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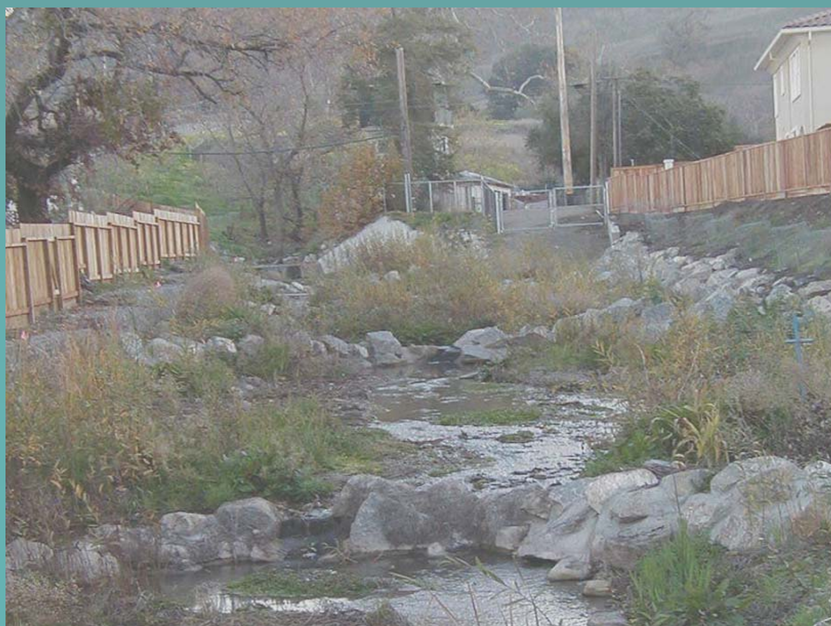
City of Milpitas
Storm Drain Master Plan



Schaaf & Wheeler
Consulting Civil Engineers

July 2013

CITY OF MILPITAS
455 East Calaveras Boulevard
Milpitas, California 95035

STORM DRAIN MASTER PLAN

July 2013

This report has been prepared under the direct supervision of the undersigned, who hereby certifies that he is a Registered Professional Engineer in the State of California.

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Abbreviations and Acronyms

ACFCWCD	Alameda County Flood Control and Water Conservation District
AMC	Antecedent Moisture Conditions
Approx	Approximately
BFE	Base Flood Elevation (100-year water surface elevation)
CEQA	California Environmental Quality Act
CFS	Cubic Feet per Second
CIP	Capital Improvement Program
CMP	Corrugated Metal (steel) Pipe
CN	Curve Number
DWR	California Department of Water Resources
Esmt	Easement
Elev	Elevation
Exist	Existing
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FPS	Feet per Second
GIS	Geographic Information System
GPM	Gallons per Minute
HGL	Hydraulic Grade Line
HP	Horsepower
HSG	Hydrologic Soil Group
IDF	Intensity-Duration-Frequency
INV	Invert
LF	Lineal feet
LiDAR	Light Detection and Ranging
MAP	Mean Annual Precipitation
MSL	Mean Sea Level
MS4	Municipal Separate Storm Sewer Systems
NAD	North American Datum (1983)
NAVD	North American Vertical Datum (1988)

NFIP	National Flood Insurance Program
NGVD	National Geodetic Vertical Datum (1929)
NPDES	National Pollutant Discharge Elimination System
NRCS	U. S. Department of Agriculture Natural Resources Conservation Service
RCB	Reinforced Concrete Box (Culvert)
RCP	Reinforced Concrete Pipe
RPM	Revolutions per Minute
RWQCB	San Francisco Bay Regional Water Quality Control Board
SCVWD	Santa Clara Valley Water District (also, “District”)
SD	Storm Drain
SDMP	Storm Drain Master Plan
SCS	U.S. Department of Agriculture Soil Conservation Service
STA	Station
TDS	Return Period-Duration-Specific
UPRR	Union Pacific Railroad
VAC	Volts, alternating current
VDC	Volts, direct current
VTA	Santa Clara Valley Transportation Authority
WSEL	Water Surface Elevation
YR	Year

EXECUTIVE SUMMARY

Milpitas completed its first comprehensive storm drainage master plan in 2001. This effort represents the first major update of that document, and has been undertaken to help guide the City of Milpitas (City) implement a prioritized capital improvement program and meet requirements of the California Environmental Quality Act (CEQA). This document represents an updated and complete Storm Drain Master Plan (SDMP).

The city is now over fifty years old and is beginning to experience the effects of aging storm drainage infrastructure, the need to maintain and replace expensive equipment and facilities, and changing regulatory requirements. This Storm Drain Master Plan identifies the capital improvements needed to maintain acceptable levels of protection against storm water runoff, and the need for a revenue stream that will allow the necessary capital improvements to be made, and the storm drain system kept 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 in relation to flood flows; and 2) inadequate capacity of local drainage facilities. Since the operation and maintenance of major creeks and channels is, 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.

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 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 storm water pumping stations.

Regional Storm Water Coordination

The Santa Clara Valley Water District (District) is Milpitas' primary partner in the management of local storm water issues. The District's stated mission is to "[manage] an integrated water resources system that includes the supply of clean, safe water, flood protection and stewardship of streams on behalf of Santa Clara County's 1.8 million residents." More specifically, the District manages most of the major drainage-ways in Milpitas including Arroyo de los Coches, Berryessa Creek, Calera Creek, Coyote Creek, Lower Penitencia Creek, Piedmont Creek, and Tularcitos Creek.

Coordination with the District is integral to the success of the storm drain master plan, since all of the City's storm drainage systems eventually discharge into a District-managed facility. The District is keenly interested in any City storm drain project that might potentially impact one of their receiving creeks. In turn, the City has a vested interest in how the District discharges 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 storm water within the City of Milpitas. The City's storm drain CIP also must address storm water infrastructure needs identified in the City Utility Asset Management System and storm water quality protection needs defined by the 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 to be used 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) are used in this master plan, with some additional provisions as discussed throughout the document.

A geographic information system (GIS) based model representing storm drain systems throughout Milpitas has been constructed using data provided by the City 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 compared to the peak rate of runoff carried in that system component, to confirm whether City drainage system performance criteria are met. If certain criteria are not met by the existing storm drainage system, the model is then used to establish the capital improvement(s) needed so that those criteria are met upon the completion of a prioritized capital improvement program.

Estimated Capital Costs and Annual Revenue Requirements

Based on the analytical evaluation of Milpitas' existing storm drainage system using the GIS-based hydrologic and hydraulic models, a prioritized capital improvement program has been established. Figure ES–1 shows the locations of city-wide high priority capital improvement projects. Table ES-1 provides an estimate of the present worth of capital expenditures needed to complete those projects shown on Figure ES–1, and provides capital costs for other medium and 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 Capital Improvement Program, except low priority projects that are either optional or expected to be built as ancillaries to other site development or public projects, long-term equipment replacement, and annual operations and maintenance.

Table ES-1
Capital Improvement Program Costs

Category	Included with CIP	Optional/Low Priority
High Priority CIP	$16,000,000	
Medium Priority CIP	$11,000,000	
Low Priority CIP		$12,000,000
Total Budget	$27,000,000	$12,000,000

Table ES-2
Summary of Storm Drainage Budget Requirements

Category	Present Worth	Annualized Cost
Capital Improvements	$27,000,000	$2,400,000
Long-Term Equipment Replacement	$38,000,000	$1,100,000
Annual Operations and Maintenance		$1,500,000
Total Budget	$65,000,000	$5,000,000

Work Products

This updated master plan is intended 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 GIS-based model is available.

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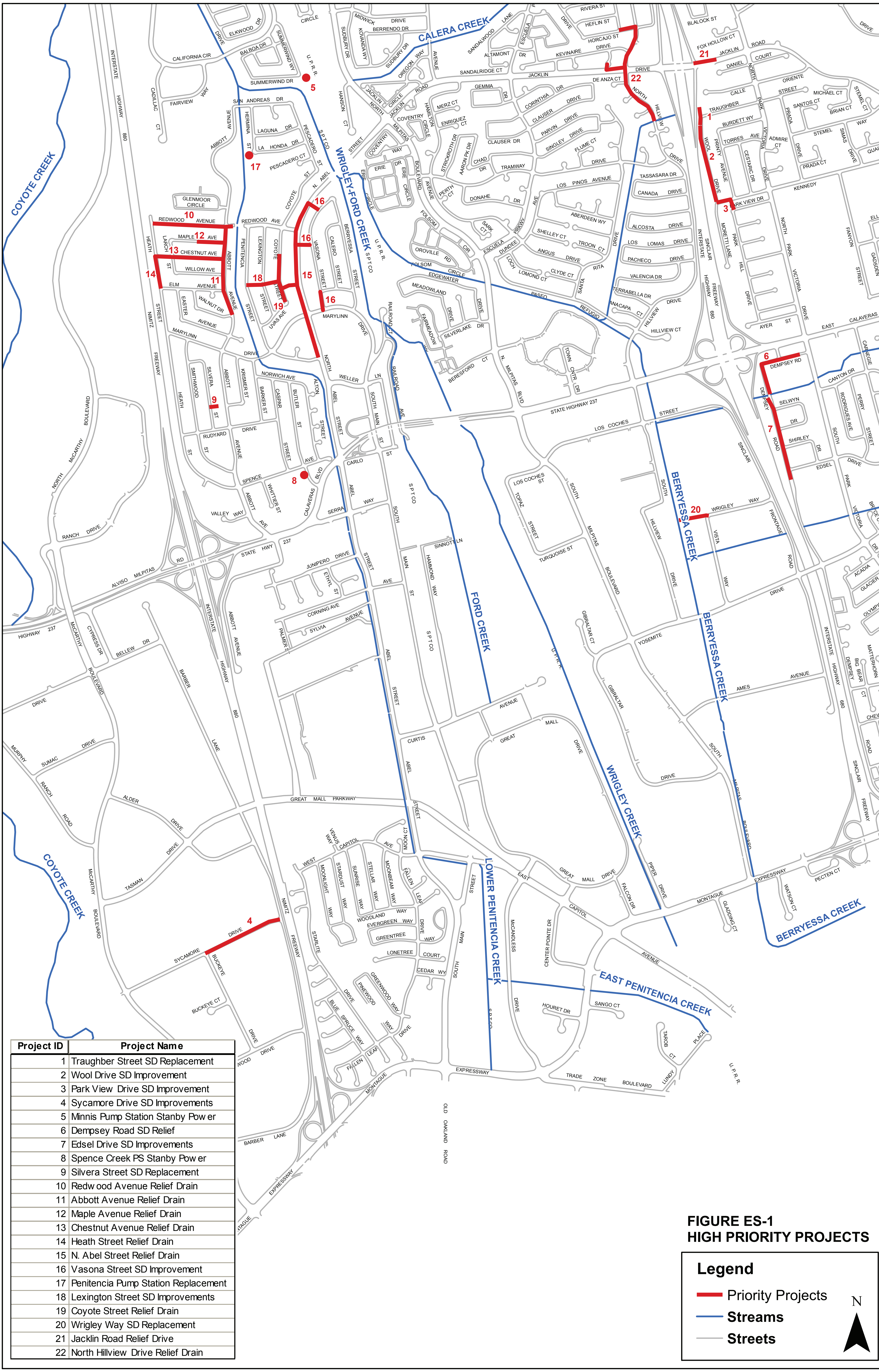


Project ID	Project Name
1	Traughber Street SD Replacement
2	Wool Drive SD Improvement
3	Park View Drive SD Improvement
4	Sycamore Drive SD Improvements
5	Minnis Pump Station Stanby Power
6	Dempsey Road SD Relief
7	Edsel Drive SD Improvements
8	Spence Creek PS Stanby Power
9	Silvera Street SD Replacement
10	Redwood Avenue Relief Drain
11	Abbott Avenue Relief Drain
12	Maple Avenue Relief Drain
13	Chestnut Avenue Relief Drain
14	Heath Street Relief Drain
15	N. Abel Street Relief Drain
16	Vasona Street SD Improvement
17	Penitencia Pump Station Replacement
18	Lexington Street SD Improvements
19	Coyote Street Relief Drain
20	Wrigley Way SD Replacement
21	Jacklin Road Relief Drive
22	North Hillview Drive Relief Drain

CHAPTER 1
INTRODUCTION

Milpitas completed a comprehensive storm drain master plan in 2001. This effort represents the first major update of that document, and has been undertaken to help guide the City of Milpitas (City) in implementing a prioritized capital improvement program and meeting requirements of the California Environmental Quality Act (CEQA). This document represents an updated and complete storm drain master plan (SDMP). Key objectives of the SDMP update include:

- Updating the geographical information systems (GIS) -based storm drain system model for the entire city to reflect all storm drain projects and operational improvements completed through 2009, as well as any changed land uses.
- Revising hydrologic and hydraulic calculations for consistency with the Santa Clara County Drainage Manual (Schaaf & Wheeler, 2007).
- Presenting flood hazard information included with the digital flood insurance rate map (DFIRM) effective May 18, 2009.
- Improving the document's graphical clarity and ease of use.
- Eliminating the reliance on the specialized commercial software previously used for hydrologic calculations, due primarily to shortcomings with respect to street capacity calculations. Storm drain capacity and hydraulic grade calculations are now based on the Microsoft Excel and ArcView software platforms.
- Categorizing storm drainage system deficiencies after the inclusion of recent upgrades to system operation, in terms of the risk to public safety and potential property damage.
- Preparing an updated Capital Improvement Program (CIP) that remediates identified system deficiencies.
- Updating projected capital improvement, operations, maintenance, and replacement schedules and costs.

Flooding within Milpitas is caused by two basic interrelated factors: 1) major creeks and channels that overflow due to limited capacity in relation to flood flows; and 2) inadequate capacity of local drainage facilities. References are made throughout this Master Plan to the larger creeks and channels and their impact on the city's storm drainage system. However, the operation and maintenance of these major facilities is, for the most part, outside the city's control. The focus of this document, therefore, is on local storm drainage collection and pumping facilities.

Authorization

Schaaf & Wheeler Consulting Civil Engineers, Inc. prepared this updated Storm Drainage Master Plan, Project CP3701, for the City of Milpitas in accordance with the provisions of an agreement executed by the City on December 16, 2008.

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 boundary between the city and neighboring Fremont. Incorporated Milpitas encompasses 13.5 square miles, all of which are 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. As stated previously, however, this Master Plan focuses on the impacts of events occurring within the city itself.

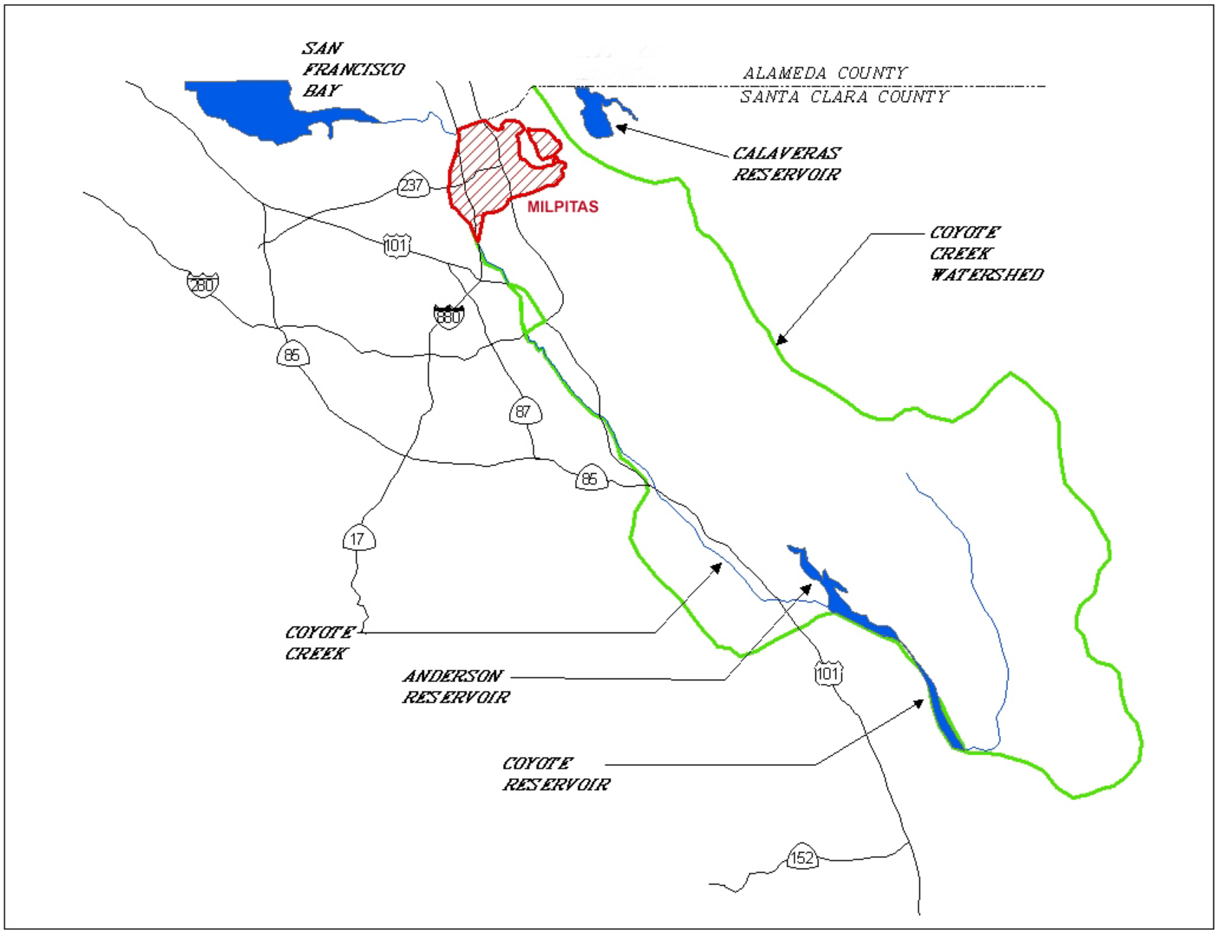


Figure 1-1: City of Milpitas within the Coyote Creek Watershed

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 storm water 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 associated with the movement of air masses from regions of higher barometric pressure 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 cold fronts are typified by higher rainfall intensities. Convective precipitation (e.g. thunderstorms) caused by the heating of air at the ground, often leads to extremely intense localized storms, but is not common to this area.

Physiography

The city lies at the base of the Diablo Range, 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 almost 800 feet at Monument Peak just west of Calaveras Reservoir. Once on the valley floor, the land falls away from the base of the hills 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 base of the foothills. Throughout the center of the city, 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. A majority of the soil within Milpitas is either clay or clayey loam with very low infiltration rates when wetted and therefore has a high runoff potential. At the western city limits near Coyote Creek, some of the soil is loamier in nature 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 city's 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 development 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 have been most concentrated within the Midtown and Transit Area Specific Plan (TASP) areas, and therefore storm drain systems serving these tributary areas are probably the most potentially impacted by new development.

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 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.

Regional Storm Water Coordination

The Santa Clara Valley Water District (District) is Milpitas' primary partner in the management of local storm water issues. The District's stated mission is to "[manage] an integrated water resources system that includes the supply of clean, safe water, flood protection and stewardship of streams on behalf of Santa Clara County's 1.8 million residents." More specifically, the District manages most of the major drainage-ways in Milpitas including Arroyo de los Coches, Berryessa Creek, Calera Creek, Coyote Creek, Lower Penitencia Creek, Piedmont Creek, and Tularcitos Creek.

Coordination with the District is integral to the success of the storm drain master plan, since all of the City's storm drainage systems eventually discharge into a District-managed facility. The District is keenly interested in any City storm drain project that might potentially impact one of their receiving creeks. In turn, the City has a vested interest in how the District discharges its legislated flood protection responsibility. A majority of the identified special flood hazard areas within Milpitas are the result of overflows from District facilities during periods of extreme runoff. The Capital Improvement Plan presented in this document recognizes the ongoing coordination required to remove these hazard areas, through they are projects over which the City has little direct control.

This master plan focuses on storm drainage and flood management, which are only two factors in the overall management of storm water within the City of Milpitas. The City's storm drain CIP also must address storm water infrastructure needs identified in the City Utility Asset Management System and storm water quality protection needs defined by the San Francisco Regional Water Quality Control Board's (RWQCB) Municipal Regional Storm Water Permit, and the National Pollutant Discharge Elimination System (NPDES) permit.

Work Products

This updated master plan is intended 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 GIS based model is available. As discussed in subsequent sections, the following information is available via the GIS:

1. ***Inventory of Drainage Facilities.*** Information pertaining to each system component may be accessed graphically through GIS, or numerically using the companion tables and spreadsheets.
2. ***Tributary Drainage Areas.*** Land areas used to generate local runoff are available in GIS and tabular format with tributary areas, runoff coefficients, and times of concentration.
3. ***Hydraulic Grade Information.*** The 10-year and 100-year hydraulic grade line information may be accessed using either the GIS or calculation spreadsheets.
4. ***Storm Drain Capacities and Street Flow Evaluation.*** Storm drain discharges and capacities are documented in the GIS. Calculation spreadsheets indicate locations where estimated storm water discharges exceed pipe capacity and flow within street rights-of-way. Capacities at the top-of-curb and right-of-way line are compared to establish system deficiencies, necessary remediation, and the priority of that remediation.

CHAPTER 2
METHODOLOGIES

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 to be used for future infrastructure design. As discussed in this chapter and the next, 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) are used in this master plan, with some additional provisions as discussed herein.

A geographic information system (GIS) based model representing storm drain systems throughout Milpitas has been constructed using data provided by the City and gathered in the field as needed. 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 compared to the peak rate of runoff carried in that system component, to confirm whether City drainage system performance criteria are met. If certain criteria are not met by the existing storm drainage system, the model is then used to establish the capital improvement(s) needed so that those criteria are met based on the capital improvement priority system described in Chapter 5.

GIS Based Model

ArcMap software has been used to construct a geographic information system (GIS) containing the City's storm runoff collection system (storm drain pipes and channels) and their tributary watersheds. The GIS is compiled on the California State Plane Coordinate System (NAD 83), with elevation data stored in feet NAVD 88. Microsoft Excel spreadsheets linked to the GIS are used to calculate the hydraulic capacity of each storm drain and street system (or open channel), and provide output data to be used in the GIS interface to assist City staff in their storm drain planning and management efforts. For each of the 20 separate drainage basins, a separate Excel workbook has been developed. Each workbook is similar in structure. The first worksheet is used for runoff and hydraulic grade calculations as described herein; the following three worksheets titled "Pipes," "Manholes" and "Basins" are used to tabulate system parameters and write .dbf files that interface with the GIS, which is supported on ArcMap software. A fourth worksheet titled "Rainfall" calculates rainfall intensities using the methods discussed herein.

Appendix A contains detailed instructions on the use of the GIS based modeling tools. Some of the data available for retrieval through Excel or GIS software are listed below, and much of these data are presented graphically and in tabular form throughout Chapter 5:

Pipe Information	Node (Manhole) Information	Basin Information
▪ System ID ▪ Material ▪ Length ▪ Diameter ▪ Discharge • 10-year • 100-year ▪ Flow Velocity • 10-year • 100-year ▪ Performance Evaluation	▪ System ID ▪ Ground Elevation ▪ Invert Elevation ▪ Hydraulic Grade Line • 10-year • 100-year	▪ Inflow to Manhole ID ▪ Tributary Area ▪ Weighted Runoff Coefficient ▪ Time of Concentration ▪ Rainfall Intensity • 10-year • 100-year

Data Sources

Most of the data required to assemble this master plan have been compiled by the City of Milpitas in the form of an AutoCAD storm drain block map. Record drawings for street improvements or tracts have been consulted as needed to fill in any new, missing, or conflicting information. In limited instances, field surveys have also been used to verify certain data. All elevations have been converted to the National Adjusted Vertical Datum of 1988 (NAVD) in order 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)

Information regarding pump station operation has been obtained from record drawings, conversations with city operations and maintenance staff, and field evaluation.

Design Storm

Since it is impossible to anticipate the effect of every conceivable storm, flood frequency analyses are often used to design facilities that control storm runoff. A common practice, and one that both the Milpitas and Santa Clara County standards follow, is to construct a design storm – a rainfall pattern used in hydrologic models to estimate surface runoff – and to 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 the frequency (probability) of a rainfall event is coincident with the frequency of direct storm water runoff, although the generation of runoff depends on a number of factors (particularly antecedent moisture conditions in the drainage basin) not necessarily dependent upon the precipitation event.

For the purposes of evaluating storm drain performance for this master plan, relevant frequencies of occurrence for precipitation (and by assumption, runoff) are 10 years and 100 years. Some statistical perspective for each of these return periods is given in Table 2-1. It may be noted 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, and the chance of experiencing at least one 100-year event is about 26 percent.

Table 2-1
Return Period Statistics

	10-year	100-year
Exceedance Probability[1]	10%	1%
Risk of at least one event in 10 years	65%	10%
Risk of at least one event in 25 years	93%	22%
Risk of at least one event in 50 years	99%	39%
Risk of at least one event in 100 years	99.997%	63%

[1]Probability of at least one event greater than or equal to a certain magnitude in a given year.

Rainfall Intensity

Over the years, the California Department of Water Resources has measured precipitation throughout the state and compiled statistics that have been reduced to intensity-duration-frequency (IDF) relationships (DWR, 1982). Santa Clara County adopted similar relationships in 2007 based on procedures set forth by the Santa Clara Valley Water District. Rainfall intensities for specified durations and frequencies of recurrence are based on recorded rainfall. The Santa Clara Valley Water District's Return Period-Duration-Specific (TDS) Regional Equation is used in the Santa Clara County Drainage Manual to establish a relationship between precipitation depth and mean annual precipitation for various storm frequencies (return periods).

In the City's Drainage Standards, rainfall intensity curves are developed using a mean annual precipitation value of 16 inches for catchments west of Interstate 680, and 20 inches for areas east of Interstate 680. The actual recorded mean annual precipitation for Milpitas ranges from 14 inches at Coyote Creek to 18 inches at the upper end of Ed Levin Park. For the master plan, IDF curves have been refined to account for a more precise determination of mean annual precipitation based on a mean annual precipitation (M.A.P.) map published by the Santa Clara Valley Water District in 1989 (which has been superimposed over a base map of Milpitas on Figure 2-1). Once the mean annual precipitation for a given location is determined, rainfall depths are calculated using the TDS Regional Equation:

$$x_{T,D} = A_{T,D} + \left(B_{T,D}\ \ MAP\right)$$

Where $x_{T,D}$ is precipitation depth for a specific return period and storm duration (inches);
T is return period (years);
D is storm duration (hours); and
A,B are coefficients determined from Table 2-2.

Table 2-2
Rainfall Coefficients for TDS Equation

Duration	10-year		100-year	
	$A_{T,D}$	$B_{T,D}$	$A_{T,D}$	$B_{T,D}$
5 minutes	0.201876	0.002063	0.269993	0.003580
10 minutes	0.258682	0.003569	0.315263	0.007312
15 minutes	0.294808	0.004710	0.421360	0.006957
30 minutes	0.367861	0.007879	0.553934	0.009857
1 hour	0.427723	0.014802	0.626608	0.019201
2 hours	0.522608	0.027457	0.732944	0.036193
3 hours	0.591660	0.038944	0.816471	0.051981
6 hours	0.625054	0.070715	0.776677	0.101053
12 hours	0.641638	0.111660	0.821859	0.162184
24 hours	0.567017	0.162550	0.814046	0.243391

The precipitation intensity, $i_{T,D}$, is given by:

$$i_{T,D} = \frac{x_{T,D}}{D}$$

Hydrologic models used to prepare this master plan use the TDS equation to directly calculate the precipitation intensity based on the time of concentration calculated at the system location of interest and the mean annual precipitation from Figure 2-1, rounded up to the nearest inch for each collection system as identified in Chapter 5.

For master plan users' convenience, the intensity-duration-frequency (IDF) curves for mean annual precipitation values of 15 and 20 inches are also provided in Appendix B. Interpolation may be used to obtain IDF relationships for MAP values between those shown in Appendix B, or the equations described herein may be applied directly.

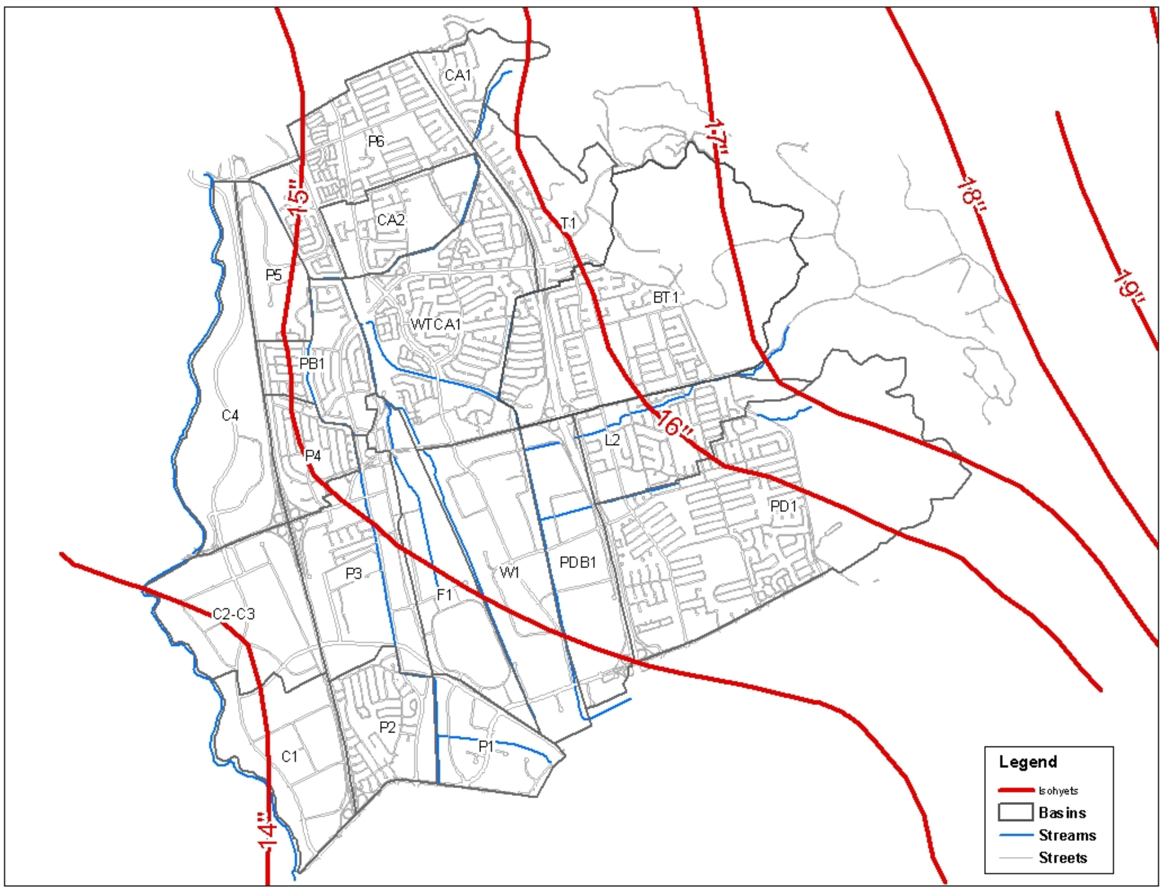


Figure 2-1: Mean Annual Precipitation in Milpitas (ref. SCVWD)

Design Storm Pattern

The County Drainage Manual specifies a 24-hour design rainfall distribution pattern for Santa Clara County. This pattern is balanced to reflect local rainfall duration-depth relationships described above so that hydrographs are consistent with peak runoff estimates made using the Rational Method. Figure 2-2 shows the balanced 24-hour rainfall patterns (which are a function of mean annual precipitation) used in this master plan, as summarized by Table 2-3.

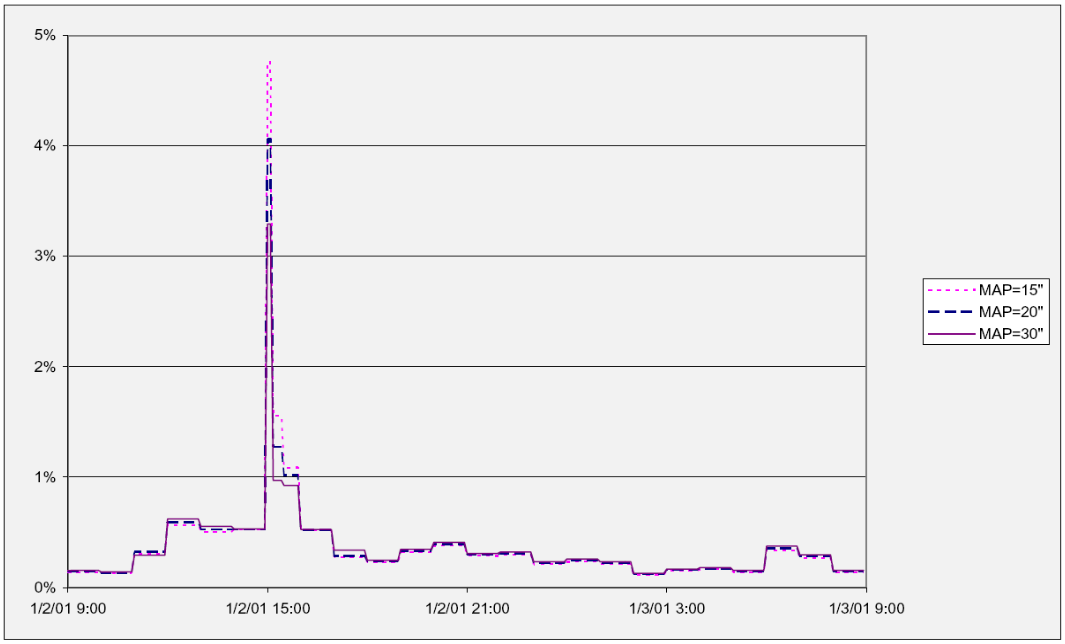


Figure 2-2: Design Storm Patterns

Table 2-3
24-Hour Design Storm (5-Minute Pattern)

Time Starting	Fraction of Total Rainfall in Percent	
	MAP = 15”	MAP = 20”
0:00	0.1412	0.1482
1:00	0.1294	0.1358
2:00	0.3080	0.3223
3:00	0.5667	0.5930
4:00	0.5051	0.5285
5:00	0.5272	0.5266
6:00	4.7600	4.0600
6:10	1.5540	1.2750
6:30	1.0850	1.0169
7:00	0.5177	0.5229
8:00	0.2763	0.2860
9:00	0.2302	0.2384
10:00	0.3223	0.3337
11:00	0.3799	0.3933
12:00	0.2878	0.2979
13:00	0.2993	0.3099
14:00	0.2118	0.2223
15:00	0.2353	0.2470
16:00	0.2118	0.2223
17:00	0.1177	0.1235
18:00	0.1530	0.1605
19:00	0.1647	0.1729
20:00	0.1412	0.1482
21:00	0.3412	0.3581
22:00	0.2706	0.2840
23:00	0.1412	0.1482

Runoff Characteristics

Storm runoff modeling requires some means of evaluating the amount (peak rate and volume) of runoff generated by the tributary watersheds. In conformance with the County Drainage Manual, the methodologies used herein rely upon lumped parameters to convert precipitation into direct runoff. The lumped parameter models all of the natural watershed processes (e.g. infiltration, depression storage, vegetation, etc.) that cause a certain percentage of precipitation to flow off of an individual catchment as runoff. Estimated values of peak basin discharge and volume, therefore, are heavily influenced by the selection of runoff coefficients, which is based on the type of land uses within a watershed and the characteristics of the underlying soil.

Two types of lumped runoff parameters are used in the Milpitas Storm Drain Master Plan:

1. Runoff Coefficients, used to generate peak runoff rates; and
2. Curve Numbers, used to generate discharge hydrographs.

Runoff Coefficients

Table 2-4 lists runoff coefficients used in master plan analysis, which are generally consistent with runoff coefficients from the 2007 County Drainage Manual. Each coefficient is a function of the underlying land use and soil type; more specifically, the NRCS "Hydrologic Soil Group" (HSG). Land use types based on the City's current land use zoning designations are shown in Figure 2-3, and soil types as defined by the Santa Clara Valley Water District are shown in Figure 2-4. A complete listing of the weighted runoff coefficients used for each basin is provided in the GIS model. It is important to remember that runoff coefficients are <u>not</u> necessarily equivalent to the percent of impervious surface within a basin.

Table 2-4
Runoff Coefficients

Land Use	Soil Type (HSG)		
	B	C	D
Low Density Residential	0.30	0.40	0.45
Medium Density Residential	0.50	0.55	0.60
High Density Residential	0.70	0.70	0.75
Mixed Use	0.70	0.70	0.75
Commercial	0.80	0.80	0.80
Industrial	0.80	0.80	0.80
Parks	0.20	0.30	0.35
Institutional	0.30	0.40	0.45
Agricultural	0.15	0.35	0.40
Shrub Land	0.20	0.40	0.50
Paved Surfaces	0.85	0.85	0.85

The runoff coefficients listed in Table 2-4 will produce estimates of peak runoff that calibrate to peak discharge based on flood frequency analyses of measured stream discharges in Santa Clara County. The County uses this approach of statistically based peak flow estimation in lieu of attempting to calibrate rainfall-runoff models to individual storm events that can be difficult to measure.

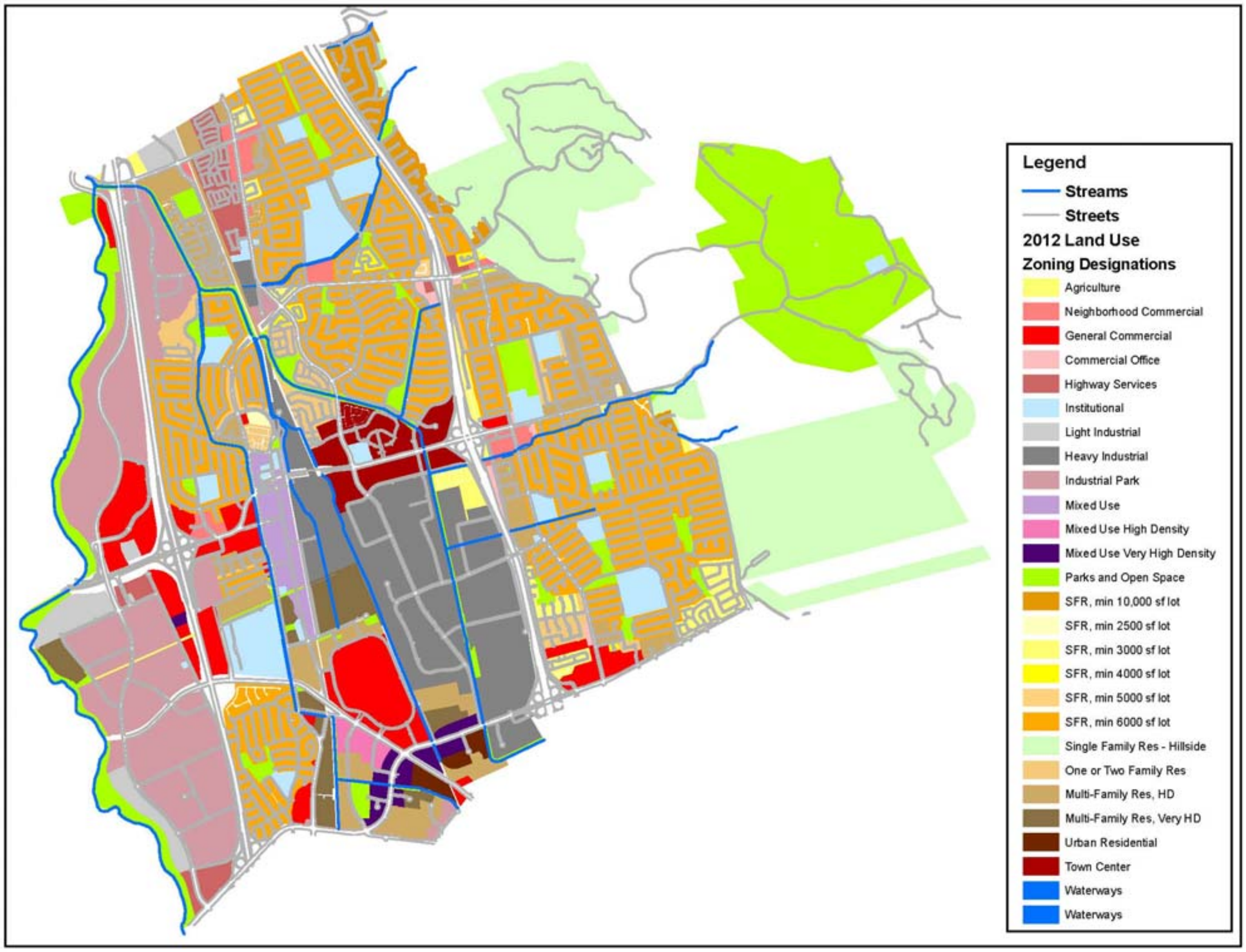


Figure 2-3: Land Use Designations within Milpitas

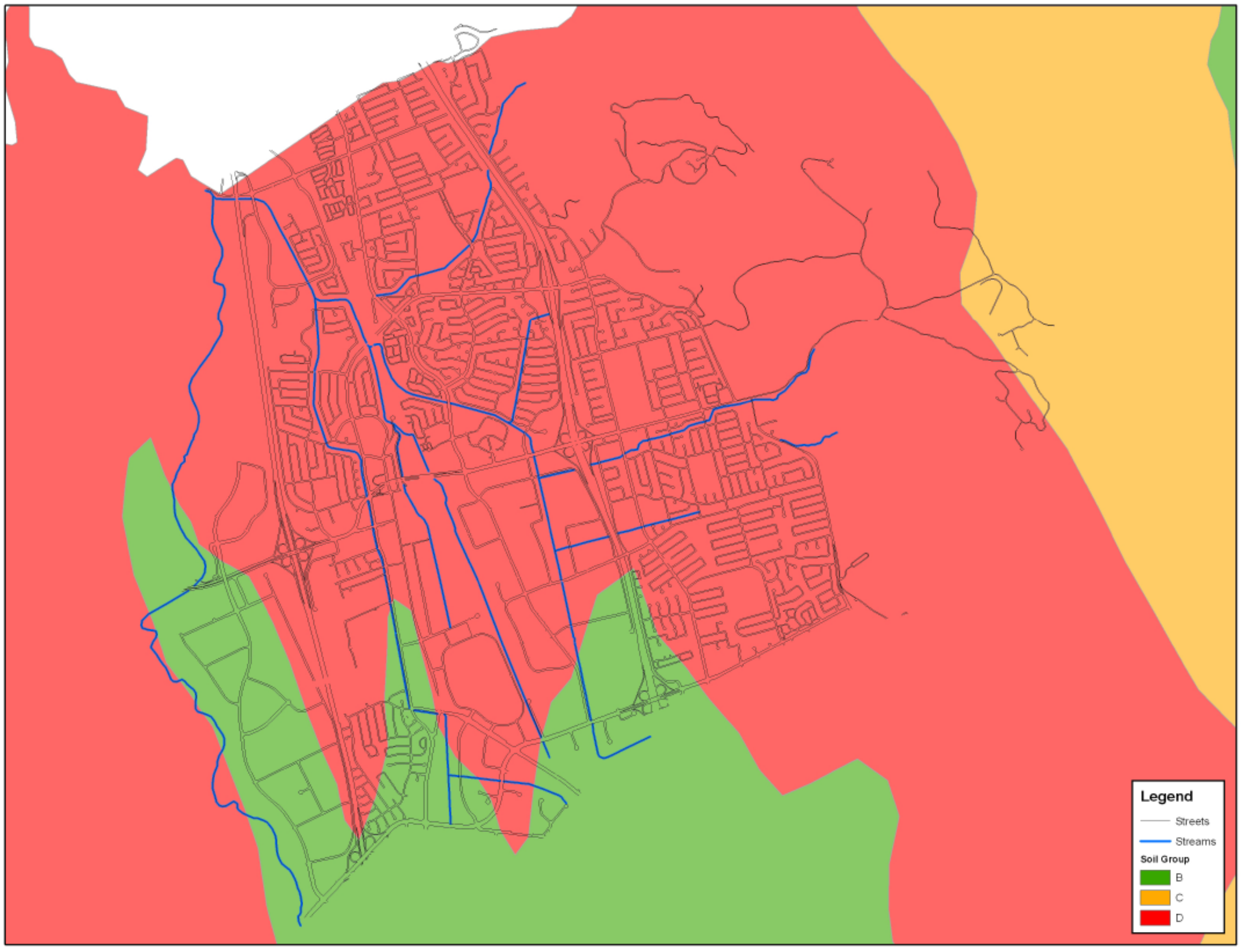


Figure 2-4: Soil Types within Milpitas

Studies have shown that runoff coefficients do not remain constant during individual storms or from storm to storm. Antecedent moisture conditions (a measure of how wet the soil is due to previous storms) and other factors can change the net runoff coefficient. It has also been observed that as rainfall intensity increases, soil permeability decreases. One may sense, therefore, that runoff coefficients should increase with rainfall intensity.

Applying such non-linearities over an area with as many small urban drainage basins as Milpitas, however, is not rewarded by significant improvements in accuracy when weighed against the difficulties imposed on computation. It is also noted that Milpitas' assumed constant coefficients compare favorably to neighboring Alameda County's (ACFCWCD, 1987), which are adjusted for rainfall intensity. Alameda's maximum C_{100} (based on a minimum t_c of 10 minutes) is 0.53 for single-family residential, 0.67 for multi-family residential, and 0.8 for industrial and commercial development respectively.

Curve Numbers

The Soil Conservation Service (SCS, now the National Resources Conservation Service) Curve Number methodology is used to estimate direct runoff by subtracting soil infiltration and other losses from the rate of rainfall. The Curve Number (CN) method is an empirical methodology wherein the CN reflects potential loss for a given soil and cover (land use) complex. After satisfying an initial abstraction – rainfall absorbed by tree cover, depressions, and soil at the beginning of a storm – the soil becomes saturated at a certain rate so that a higher percentage of the accumulated rainfall is converted to runoff. The initial abstraction for pervious areas is set to 0.2S where S = (1000/CN) – 10. For impervious areas, the initial abstraction is set equal to 0.05 inch.

Estimates of the CN are made based on the soil types (Figure 2-4) and cover (Figure 2-3) within a drainage basin. The number varies from 0 to 100, and represents the relative runoff potential for a given soil-cover complex for given antecedent moisture conditions (AMC, which measures how wet the soil is prior to a precipitation event). The County Drainage Manual lists Curve Numbers that have been calibrated to replicate peak discharges for approximately 200-acre basins using the subsequently described Rational Method and the runoff coefficients that are listed in Table 2-4. Using the balanced storm pattern of Figure 2-2, the appropriate AMC for both 10-year and 100-year simulations is II½ (Santa Clara County, 2007). Table 2-5 lists Curve Numbers used in the master plan, adjusted to reflect AMC II½.

Table 2-5
SCS Curve Numbers (AMC II1/2)

Land Use	Soil Type (HSG)		
	B	C	D
Urban Open Space	67	79	81
Agricultural	69	81	86
Shrub Land	61	74	79
Paved Surfaces	100	100	100

In urbanized areas, Curve Numbers for the pervious (open) portion of the basin are used along with the listed estimates of impervious area (Table 2-6), which are used in hydrologic modeling.

Table 2-6
Imperviousness for Urban Areas

Land Use	Percent Impervious
Low Density Residential	25
Medium Density Residential	40
High Density Residential	50
Mixed Use	50
Commercial	80
Industrial	80
Parks	10
Institutional	40

Runoff Calculations

One hydrologic value of interest when evaluating and designing a storm drain collection system is the peak rate of flow within each system element. In an urbanized area characterized by relatively small watersheds with largely impervious areas, the Rational Method has a long history of usefulness for flood peak estimation and storm water conveyance system design, where a full hydrograph is not required.

Rational Method of Peak Flow Estimation

The Rational Method has been selected for the following reasons:

1. Use of the Rational Method has been adopted by the City of Milpitas as its standard.
2. The method is simple to apply, and does not necessarily require the use of computer simulation.
3. Although the application of this seemingly simple methodology is subject to judgment and difficult to replicate among users, establishing standard parameters and equations in a master plan can promote reasonableness and design equity throughout the city. In other words, all potential storm drain system developers can be held to the same standard.
4. Use of the Rational Method is generally limited to areas roughly one square mile in size (ASCE, 1996). All of the collection systems analyzed for the Master Plan drain tributary areas that fall within this limit.

The Rational Method estimates peak discharge based on the following formula:

$$Q_T = k\, C\, i_T\, A$$

where Q_T = peak flow rate in cubic feet per second (cfs), for a return interval of T years;

k = 1.008 (often taken as 1.0);

C = a dimensionless runoff coefficient dependent upon land use;

i_T = the design rainfall intensity (inches per hour) for a return interval of T years, and a duration equal to the time of concentration for the basin; and

A = drainage area in acres.

This methodology is based on the premise that under constant rainfall intensity, peak discharge will occur at the basin outlet when the entire area above the outlet contributes runoff. Known as the "time of concentration," this value is defined as the time required for runoff to travel from the most hydraulically distant point (at a drainage divide such as a ridge) to the outlet.

Effective use of the Rational Formula depends upon the computation of the time of concentration, t_c. In this master plan, time of concentration estimation is separated into urban areas and open space. Travel time for runoff in urbanized basins occurs in three phases:

1. Initial overland flow represents rainfall collecting on roof tops and making its way to an impervious surface, where runoff begins in earnest. In accordance with the County Drainage Manual, this value is assumed to be ten minutes where a substantial area is drained, and five minutes when street or parking lot sections are drained.
2. Gutter flow represents the sheet flow of runoff over paved or other impervious surfaces (e.g. street gutters) toward an initial collection point in the city's storm drain system. Calculations for this portion of travel time are based on the overland flow chart from the County Drainage

Manual (Figure A-1) for paved areas assuming a minimum slope of 0.005 feet/foot in urban areas. The calculated relationship between pavement slope (S in feet/foot) and flow velocity (in feet per second) is:

$$V = 20.19 S^{0.5}$$

3. Pipe flow in a storm drain collection system is calculated by dividing the distance between design points by the average flow velocity in the subject reach. The average velocity is based either on partial or full pipe flow (the latter is for surcharged pipe), and data input into the system including pipe size, slope, length and Manning's roughness coefficient. The program lags each discharge hydrograph downstream based on this travel time. Table 2-7 indicates roughness coefficients used for analysis and design.

The total time of concentration used in the Rational Method calculation is the sum of the overland flow time plus any travel time in pipes, gutters, swales, or channels leading to the point at which a discharge estimate is desired.

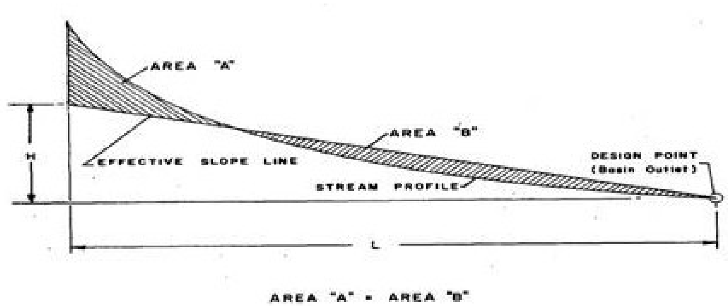
Table 2-7
Manning "n" Values for Storm Drain Elements

Drainage System Element	"n"
Reinforced Concrete Pipe	0.013
Reinforced Concrete Box Culvert	0.015
Cast-in-Place Concrete Pipe	0.018
Corrugated Metal Pipe	0.024
Street Right-of-Way	0.025

For natural watersheds in the hillside area, the Kirpich formula is used (Santa Clara County, 2007):

$$T_c = 0.0078 \left(\frac{L^2}{S} \right)^{0.385} + 10 \;\; minutes$$

where L is the length of maximum length of travel from headwater to outlet (feet); and
S is the effective slope along L (feet per foot) as illustrated below:



Hydrograph Method of Estimating Storm Water Runoff

In certain instances, particularly for the evaluation of pumping and storage systems, estimates of storm water volume may also be required. IDF relationships for precipitation are not sufficient to provide this information. Rather, temporal rainfall patterns (hyetographs) must be specified for rainfall depths of a given duration and frequency. (For instance, a 24-hour, 100-year design storm is often used to analyze pumping and detention facilities.)

The time response of catchment runoff caused by one inch of excess rainfall applied uniformly over time is numerically represented by a unit hydrograph. Unit hydrograph methods described in the 2007 County Drainage Manual are used in this master plan to generate runoff hydrographs where needed for system analysis.

Several techniques are available to estimate unit hydrographs for rainfall-runoff calculations. To be consistent with the County Drainage Manual, the SCS synthetic unit hydrograph is used. This unit hydrograph is based on a single parameter, basin lag, which is the time from the beginning of excess rainfall (that is, direct runoff) to the point in time when fifty percent of the runoff has passed the catch point. The basin lag equation (a modified version of the US Army Corps of Engineers [USACE] basin lag equation) is:

$$t_{lag} = (0.862) 24 N \left(\frac{L L_c}{\sqrt{S}} \right)^{0.38} - \frac{D}{2}$$

where t_{lag} = Basin lag (hours)

N = watershed roughness value (dimensionless)

L = longest flow path from catchment divide to outlet (miles)

L_c = length along flow path from a point perpendicular with the basin centroid to its outlet (miles)

S = effective slope along main watercourse (feet/mile)

D = duration of unit hydrograph (hours)

N should not be confused with Manning's n. Rather it is a watershed "roughness" value selected based on the level of urbanization within a basin. Table 2-8 provides values for N recommended by USACE.

Table 2-8
Urbanization Parameters for Basin Lag

Basin Condition	N
Natural channels, little or no urbanization	0.080
Urban area with natural channels	0.050
Concrete-lined channels with ~2/3 basin urbanized	0.035
Full basin urbanization with storm drain systems	0.025

If a runoff hydrograph needs to be produced with a peak discharge matching the peak runoff estimated using the Rational Method, the Clark Synthetic Unit Hydrograph may be used. The Clark Synthetic Unit Hydrograph is used by the Santa Clara Valley Water District in their standard hydrology procedures.

The Clark UH method relies on two parameters:

t_c	Time of concentration (hours)	Estimated as previously described
R	Storage coefficient (hours)	Varied to produce a hydrograph with peak discharge equivalent to that determined by the Rational Method

Storage Facilities

Often storm drain collection systems terminate in a storage facility where runoff is pumped into a receiving creek, or metered out to downstream conveyance facilities. The operation of these facilities is evaluated using the calibrated unit hydrograph method described previously using the HEC-1 program. Once calibrated and balanced, the design hydrograph is routed through the detention and pumping (or outlet) system to establish the maximum stage in the storage facility for the event of interest. This stage is then input into the GIS model to establish a starting water surface elevation for backwater analyses in tributary collection systems. Downstream flow control provided by a detention basin is modeled in the GIS by adjusting the system time of concentration to reflect the time lag provided by the basin, and adjusting the net C x A from the basin to replicate the reduced peak discharge.

Larger Watersheds

Tributary drainage areas for major drainage facilities including creeks and flood control channels have not been incorporated into the storm drain system model unless runoff from that tributary flows through a city-owned collection system (see Chapter 4). Flowrates and water surface profiles for all other major facilities have been obtained from the Santa Clara Valley Water District, Federal Emergency Management Agency reports, or the FEMA study contractor for Milpitas. Hydrologic analyses for larger watersheds are generally based on the Clark unit hydrograph and precipitation data consistent with this master plan.

Collection System Capacity Analyses

Detailed analyses of peak storm water discharge are performed in the GIS-compatible spreadsheets, which determine the flow condition in each collection system element. Depending upon the magnitude of flow and the size of the pipe in question, the pipe either flows partially flow or is surcharged and flowing under pressure.

As discussed in Chapter 3, it is permissible for runoff to be carried within the street right-of-way. When a pipe system is surcharged, hydraulic grades at each model node are adjusted to equal:

1. The calculated hydraulic grade line if it is below the ground elevation;
2. The hydraulic grade line at the downstream end of the pipe, plus the friction loss through the pipe calculated using Manning's formula;
3. The upstream invert of the pipe plus the normal depth of water in the pipe (partially full conditions only); or
4. The ground elevation.

This methodology automatically adjusts hydraulic grade line profiles to reflect spill into the street at any location within the system. After this adjustment, the spreadsheet determines the amount of flow within the street right-of-way by subtracting the pipe capacity from the design peak discharge at each location.

Flow in Streets

The depth of flow traveling down a street is determined from Manning's formula for uniform depth in an open channel:

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

where Q = peak flow rate in cubic feet per second (cfs);

n = Manning's coefficient (0.025 for streets);

A = cross sectional area of flow in the street (ft^2);

R = hydraulic radius of the flow (area/wetted perimeter in feet); and

S = longitudinal street slope (feet/feet);

Figure 2-5 shows typical cross sections for each of the street categories within Milpitas, based on the reference City standards. Street capacities are calculated as a function of slope for each street type, noting that Manning's equation may be consolidated in the form given below:

$$Q = K S^{1/2}$$

Table 2-9 presents K values for each street category, to determine capacity at the top of curb (six inches above the gutter flow line), and with a depth of six inches at the street right-of-way line (12 inches above the gutter flow line). Six inches at the right-of-way line is considered to be the acceptable limit of street flow depth during a 100-year design storm. Blockage from debris and parked cars between the curbs; and vegetation and other potential blockage within public rights-of-way near the curb and in median strips; are modeled using a composite Manning's roughness coefficient of 0.025. System performance is evaluated by comparing the predicted street overflow against the street capacity at top-of-curb and right-of-way.

Table 2-9
Street Capacity Coefficient (K)

Street Category	Top of Curb	Right of Way
Local	148	1,513
Collector	144	1,586
Minor Industrial	144	1,552
Major Industrial	144	1,655
Secondary Arterial	144	1,803
Major Arterial	144	1,692
Local Frontage	72	889
Collector Frontage	72	897
Industrial Frontage	72	819

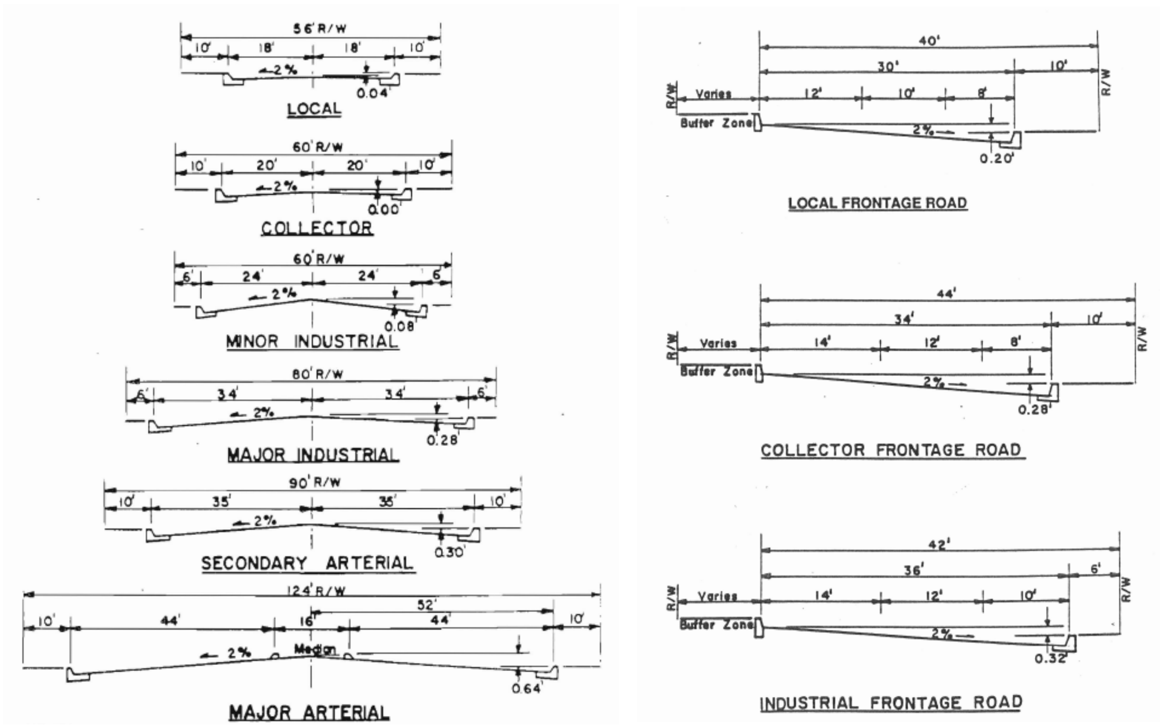


Figure 2-5: Standard Street Sections

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CHAPTER 3
DRAINAGE STANDARDS

Criteria used throughout the Master Plan to evaluate how well individual storm drainage systems are functioning – and how best to improve that function – are expanded from storm drain design criteria set forth by the City of Milpitas in the July 15, 2010 *Engineering Plans and Map Procedures and Guidelines*. Other guidance is provided by the June 15, 2010 *City of Milpitas Standard Drawings*.

Design of New Systems

Any proposed storm drainage system – whether to serve new development, to extend existing facilities, or to remedy problem areas – should be designed in conformance with Milpitas standards:

With 10-Year Design Discharge	Hydraulic grade shall be no higher than two (2) feet below top of curb elevation at any manhole or inlet.
With 100-Year Design Discharge	Hydraulic grade shall not exceed top of curb elevation.

As discussed in Chapter 5, much of Milpitas' existing collection system does not strictly meet these criteria; so when new systems tie into existing systems, it may not be possible to provide a design that meets the desired standard. The design and evaluation of new systems, particularly extensions of existing systems, must be done on a case-by-case basis. Criteria for use, when new collection systems discharge into existing systems, are suggested:

With 10-Year Design Discharge	Pipes shall be sized to carry the 10-year discharge without surcharging the pipe. When downstream surcharge effects are included, upstream hydraulic grades shall be no higher than the top of curb elevation at any manhole or inlet.
With 100-Year Design Discharge	Hydraulic grade shall not exceed the street right-of-way elevation at any location.

Manholes should be no farther than 500 feet apart, and catch basins are to be spaced so that the maximum width of gutter flow does not exceed 8 feet from the face of curb during a 10-year design storm; or 600 feet, whichever is less.

Evaluation of Existing Systems

This master plan recognizes that it may not be cost effective to replace facilities simply so that all areas within the city meet standards set for new systems. Instead, less restrictive criteria have been established at city staff's direction to balance system performance and public safety against limited capital improvement funds. By applying less restrictive criteria, fewer deficiencies are identified and this results in a commensurately shorter list of corrective projects. As such, collection system improvements are prioritized per Table 3-1.

Table 3-1
Storm System Improvement Priorities

System Acceptable	10-year design discharge is carried in the street no deeper than the top of curb, and 100-year design discharge is carried within the street right-of-way without adjacent property damage.
High Priority	A condition exists that creates a significant annual risk of flood damage. Also, where the 10-year design discharge is not carried within the street right-of-way and could cause property damage.
Medium Priority	Where the 100-year design discharge is not carried within the street-right-of-way and could cause property damage.
Low Priority	Where the 10-year flow depth in the street is over the top-of-curb, but the 100-year flow is contained within the street right-of-way. Flooding causing property damage is not expected. This category also includes those medium priority areas also exposed to 100-year flood hazards due to creek overflows.

Storage Facilities

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

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, rooftop, or park.

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 storm water detention and retention. For instance, pumps are used to move attenuated flood waves through the facility, but a permanent pool of water remains behind for aesthetic (or perhaps recreational) purposes.

Properly designed, constructed, and maintained, storm water storage facilities can 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, as well as ancillary improvements in the quality of storm runoff to receiving waters, needs to be evaluated on a case-by-case basis. However, some general design criteria should be applied to every basin:

1. Basins should be sized so that their output does not exceed the design capacity of downstream facilities.
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 basins are created by berms or levees.

4. Infiltration capacity shall not be considered when designing basins, unless percolation rates are determined by on-site soils 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. Inlet and outlet 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 be capable of fully dewatering 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 promotes good water quality.
9. Underdrain systems to minimize wetness should be considered for detention facilities not intended as permanent water features. This helps to prevent the facility from encouraging insect populations, and also 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 the point where they will not operate as designed. Therefore, some consideration 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	0.1acre-feet/mi^2/year
Hillside open space	0.4 acre-feet/mi^2/year

Pumping

Conjunctive pump and storage capacity must equal or exceed the 100-year design runoff for the area tributary to the pump station. Detailed criteria for the design and rehabilitation of storm water pump stations are provided in Chapter 6 and Appendix B. Stations need to have sufficient horsepower to discharge against the design water surface elevation.

Outfalls

Where storm drain collection systems discharge to receiving waters, analyses assume that the peak of local runoff coincides with the peak stage at the collection system outfall. Under 10-year design conditions for which the collection systems are designed, this probably provides for a conservative analysis. For 100-year conditions, however, it is generally unrealistic to expect the collection system to discharge against a coincident peak stage within a creek with a much larger tributary area, since the smaller local basins will likely peak earlier than the receiving creek. This is particularly true in Milpitas, where many of the creeks are leveed and 100-year peak stage can exceed nearby ground elevations.

If the 100-year peak stage is below the natural ground elevation at the storm drain outfall, a coincident discharge is conservatively assumed. If the receiving waters' 100-year peak stage is above the natural ground elevation at the outfall, the natural bank elevation is assumed as the starting tailwater elevation. This presumes that future capital projects to remedy 100-year flooding would not rely upon levees or floodwalls and, if they do, that project's sponsor will be responsible for mitigating adverse impacts to local storm drainage. The minimum assumed tailwater is at the top of the outfall pipe.

Where storm drain systems discharge into a pumping or detention facility, however, coincident peaks are assumed for both 10- and 100-year analyses.

Outfalls to major drainage facilities should be equipped with properly maintained flap gates or other devices to prevent creek water from flowing back into the storm drains. A full discussion of the outfall tailwater elevations assumed in master planning is provided in Chapter 5, which also identifies those locations where high creek stages preclude gravity drainage.

CHAPTER 4
MAJOR DRAINAGE FACILITIES

Each of the city's storm drainage collection systems discharges into one of Coyote Creek's tributaries, whether by gravity or by pumping. Figure 4-1 delineates these major drainage facilities and provides a representation of special flood hazards designated by FEMA as of May 18, 2009. This master plan does not intend to provide detailed documentation regarding federally regulated flood plains, nor should Figure 4-1 be used to determine if any individual areas or properties are flood prone.

It is also noted that regulatory flood hazards within Milpitas are under study as of December 2012. This re-evaluation of special flood hazard zones has been undertaken as part of the Silicon Valley BART Extension managed by the Santa Clara Valley Transportation Authority (VTA). As such, flood hazards discussed and depicted herein are subject to change, perhaps within the 2013 calendar year, although the timing of FEMA's review and eventual changes to the Flood Insurance Rate Map (FIRM) are uncertain.

This presentation of major drainage facilities is, however, useful when examined in the context of assimilating flood plain information into hydraulic modeling. For instance, street flooding during a one percent (100-year) event may be inconsequential if that area is submerged by waters overflowing from a tributary fed by 10 square miles of drainage area. And while the Capital Improvement Program can fix the street flooding, the City may have little control over how well a major facility performs.

Santa Clara Valley Water District owns and maintains most of the major drainage facilities in Milpitas. Table 4-1 indicates those drainage facilities under District jurisdiction. It is noted that the ownership of Wrigley-Ford Creek and its associated pump station were transferred to the City of Milpitas in 1993. While the City can have input into District plans and priorities, they do not have direct control over these facilities. Those wishing more detailed hydrologic or flood plain data should consult the Flood Insurance Study for Milpitas (i.e. the compiled Santa Clara County FIS). The following compiled information from previous flood insurance studies and other sources is for readers' convenience only, and is not intended as a recitation of official floodplain data.

Table 4-1
Drainage Facility Jurisdiction

Facility Name	SCVWD Jurisdiction	Milpitas Jurisdiction
Berryessa Creek	Headwaters to Lower Penitencia Creek	none
Calera Creek	Headwaters to Berryessa Creek	none
Coyote Creek	Headwaters to San Francisco Bay	none
Ford Creek	none	Sinnott Lane to Wrigley-Ford Creek
Los Coches Creek	Headwaters to Berryessa Creek	none
Lower Penitencia Creek	Montague Expressway to Coyote Creek	none
Piedmont Creek	Sequoia Drive to Berryessa Creek	Headwaters to Sequoia Drive
Tularcitos Creek	Interstate 680 to Berryessa Creek	Headwaters to Interstate 680
Wrigley Creek	none	Capitol Avenue to Wrigley-Ford Creek
Wrigley-Ford Creek	none	Confluence to Berryessa Creek

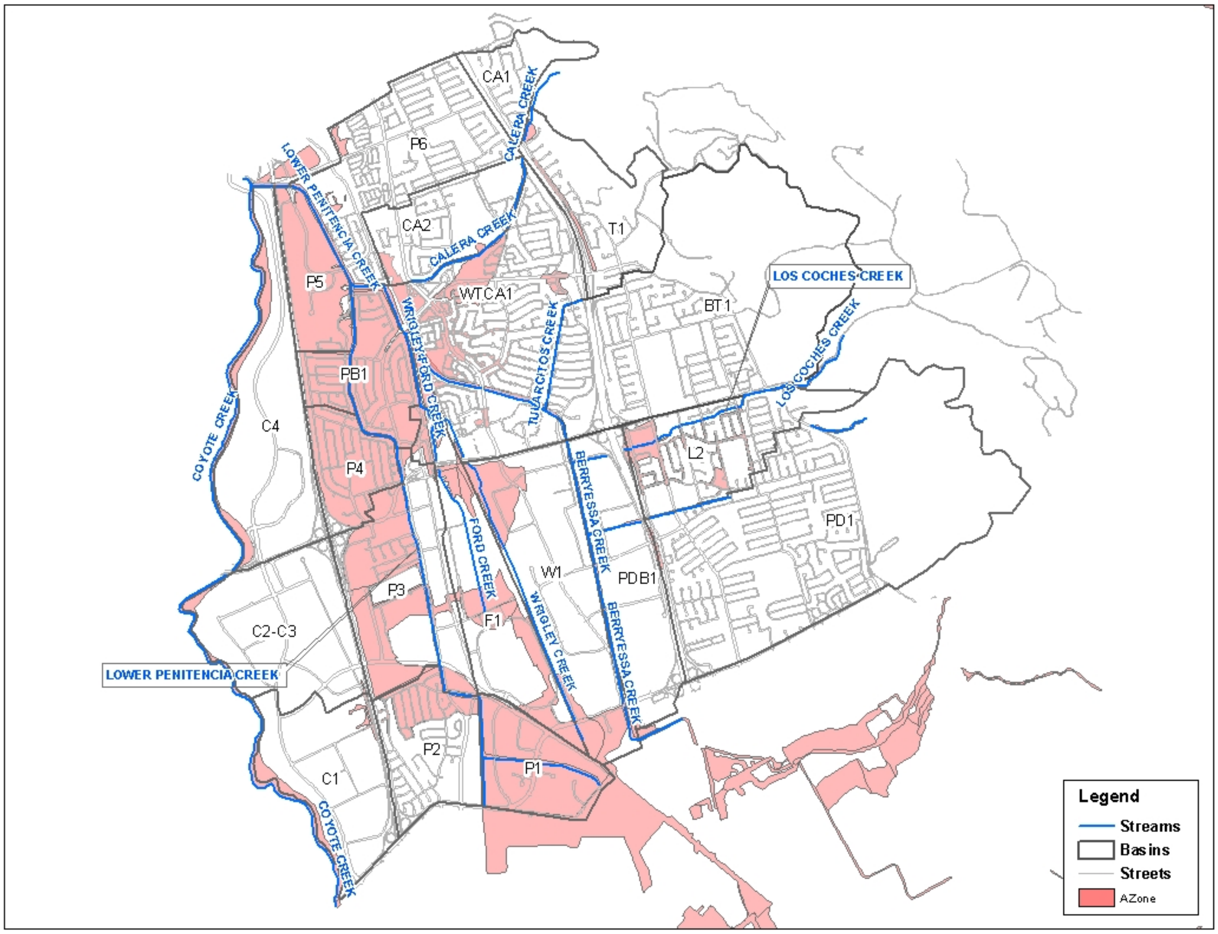

Figure 4-1: Flood Hazard Designations within Milpitas

Berryessa Creek

Berryessa Creek drains almost nine square miles of San José at the southern border of Milpitas (Montague Expressway), picking up drainage from Piedmont Creek, Los Coches Creek, Tularcitos Creek, and Calera Creek before discharging to Lower Penitencia Creek at the railroads near Milmont Drive.

Calera Creek

Extreme storm event runoff in Calera Creek spills over the south bank upstream of North Park Victoria Road and Interstate 680, flooding the adjacent Higuera Adobe Park. This spill is forced back into the creek by a series of landscape berms. South bank spills downstream from Escuela Parkway flow toward Berryessa Creek, where levees trap the water at Hidden Lake and the Berryessa Pump Station. Flood water that cannot be pumped into Berryessa Creek form a residual floodplain.

Coyote Creek

All of Milpitas eventually drains to Coyote Creek, which also drains the eastern half of the Santa Clara Valley. The Santa Clara Valley Water District operates two water supply reservoirs within the drainage area (Anderson and Coyote), which provide limited flood attenuation pools. The District has completed a levee improvement project on Coyote Creek between San Francisco Bay and Montague Expressway, which has effectively removed areas west of Interstate 880 from the flood plain north of Montague. This area is now mapped as a shaded Zone X, which represents areas of 100-year flood with average depths of less than one foot (local residual flooding), and areas protected by levees from the 100-year flood.

Los Coches Creek

Most of Los Coches Creek, from its confluence with Berryessa Creek upstream to Old Piedmont Road, is concrete lined with drop sections to dissipate energy. Upstream of Interstate 680, the channel does not have sufficient capacity to carry the 100-year discharge. Inadequate channel capacity at Old Piedmont Road causes flood water to spill to the south. Additional flows leave the channel upstream of Interstate 680, eventually reaching the highway where they pond.

Lower Penitencia Creek

Lower Penitencia Creek drains a portion of San José and Milpitas to the confluence of Berryessa Creek at Milmont Drive. After the confluence, Lower Penitencia Creek continues on to Coyote Creek at the Milpitas-Fremont border. Through Milpitas, the Santa Clara Valley Water District has lined Lower Penitencia Creek with concrete and built floodwalls to protect adjacent properties. Lower Penitencia Creek receives floodwater spilled from adjacent drainage basins at Trimble Road, but spilled water is stored behind the railroad near South Main Street, thereby reducing the discharge. Lower Penitencia Creek overflows to the west from just south of Elmwood Jail north to the Coyote Creek confluence. (Highway 880 contains this spill.) The east bank levee of Lower Penitencia Creek is fully accredited for published base flood discharges between the confluence with Berryessa Creek and Coyote Creek. Near the upstream end of Lower Penitencia Creek, the tributary East Penitencia Channel drains part of San José and Milpitas.

Piedmont Creek

Santa Clara Valley Water District jurisdiction over Piedmont Creek extends from Berryessa Creek 2,700 feet to the east. The creek is an excavated earth channel from Berryessa Creek upstream to Interstate 680. To the east until Roswell Drive, the District has built a concrete "U" frame channel. Above Roswell Drive flows are contained in larger diameter storm drains. This system drains a significant hillside area through the local pipe collection system. While some reaches of Piedmont Creek are considered to be District facilities, the entire watershed is modeled to better examine the performance of city-owned systems.

Scott Creek

Scott Creek forms the northern city boundary with Fremont; it is outside of Santa Clara Valley Water District and City of Milpitas jurisdiction. This creek discharges to Coyote Creek downstream from Lower Penitencia Creek, and no City of Milpitas facilities drain to the creek. There is no reported indication of flooding from Scott Creek that affects property within the city.

Tularcitos Creek

District jurisdiction along Tularcitos Creek extends from Berryessa Creek to the inlet of the box culvert underneath Interstate 680. The improved creek is an excavated earth channel from Berryessa to the highway. Local drainage in a storm drain collection system empties into the District facility. This system also drains a significant hillside area through the local collection system.

Wrigley and Ford Creeks

Wrigley Creek and Ford Creek drain an industrial area located between Lower Penitencia and Berryessa Creeks. The two creeks combine into Wrigley-Ford Creek, an excavated channel along the Southern Pacific and Western Pacific railroads. In 1993, the Santa Clara Valley Water District constructed a pump station at the confluence with Berryessa Creek. Under low flow conditions in Berryessa Creek, flow from Wrigley-Ford Creek can drain by gravity. Under high tailwater conditions in Berryessa the pump station takes over. The pump station is now owned by Milpitas and is described further in Chapter 6. Local storm drainage issues associated with the two creeks are discussed in Chapter 5.

CHAPTER 5
STORM DRAIN COLLECTION SYSTEMS

Analyzing Milpitas' storm drain collection system performance forms an essential core of this master plan. To better track and report results from the GIS model, collection systems are grouped and named for the creek or facility that drains them. Alphanumeric codes for the system groups are presented as Figure 5-1. The Master Plan generally follows the same codes as originally outlined by City staff in April 1997 and used in the first draft master plan document (2001). Collection system analyses are presented alphabetically herein.

For each collection system area, this chapter describes major storm drain facilities and outfalls, any historic problem areas, pumping or storage facilities (if applicable), and other known flood hazards. Within each collection system group, those areas meeting storm drain system evaluation criteria from Chapter 3 are delineated, as are those areas that do not meet the criteria, but require some form of remediation through the Capital Improvement Program (CIP). Detailed descriptions of necessary capital improvement projects and their prioritization are provided in this chapter.

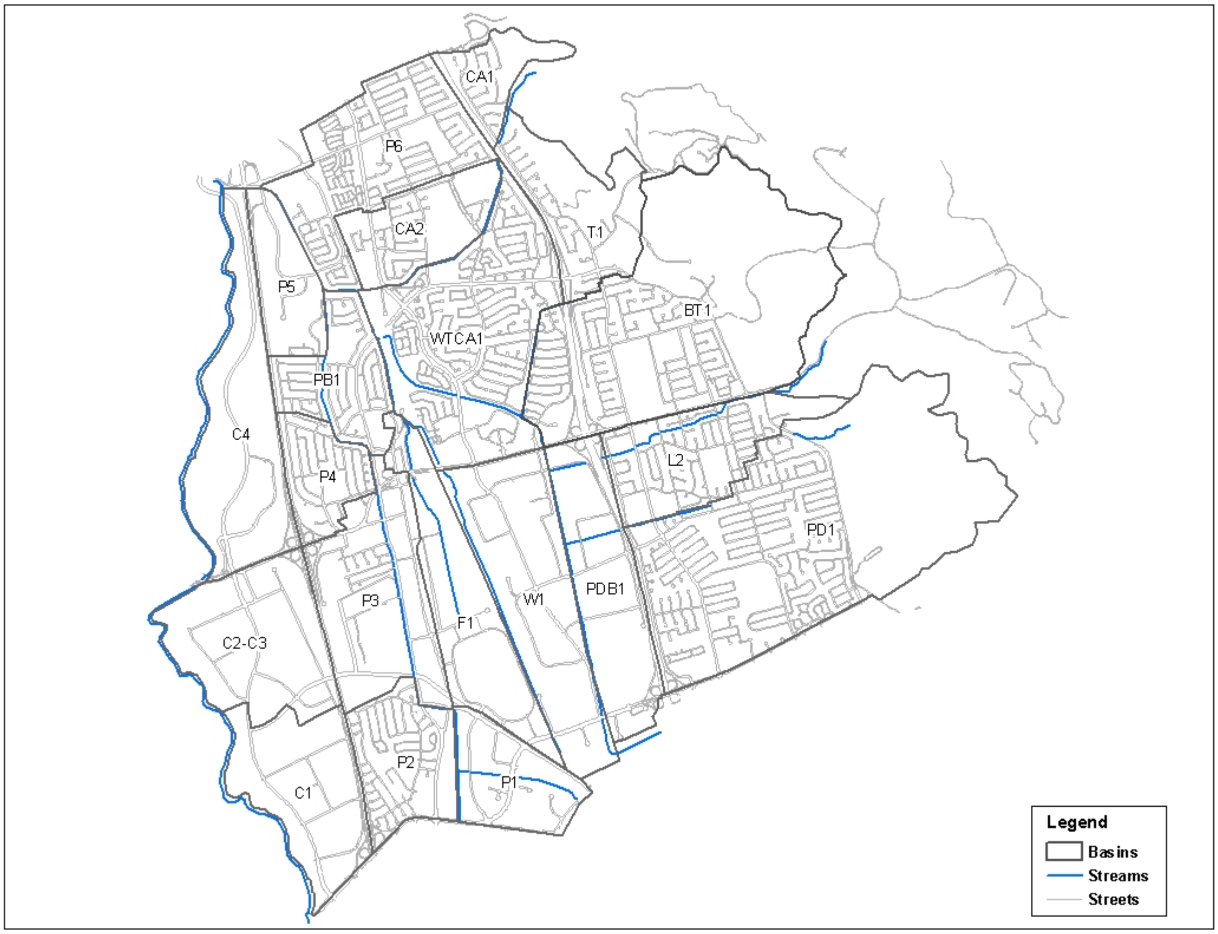

Figure 5-1: Storm Drain Collection System Grouping

System Evaluation and CIP Prioritization

Each collection system is analyzed to determine its flow condition during design ten-year and 100-year storms. Based on available storm drain pipe capacity, performance criteria for existing systems outlined in Chapter 3 are either met, or there is excess runoff flowing in the street and the criteria are not met. Table 5-1 provides the logic tree used to prioritize master plan projects necessary to meet performance criteria.

Table 5-1
Prioritization of Collection System Improvements

<table>
<tr><td colspan="5">With Existing System, is d_{10} < Top of Curb (T/C)?</td></tr>
<tr><td colspan="2">YES
Is d_{100} < Street Right-of-Way Limit (R/W)?</td><td colspan="3">NO
Is d_{10} < Street Right-of-Way Limit (R/W)?</td></tr>
<tr><td rowspan="2">YES</td><td rowspan="2">NO</td><td colspan="2">YES
d_{100} < R/W?</td><td rowspan="2">NO</td></tr>
<tr><td>YES</td><td>NO</td></tr>
<tr><td>Satisfactory</td><td>Medium Priority</td><td>Low Priority</td><td>Medium Priority</td><td>High Priority</td></tr>
</table>

Where d_{10} and d_{100} are depths of street flooding in the 10- and 100-year runoff events respectively.

Medium priority capital improvement projects are reclassified as low priority projects wherever the project is located within an identified special flood hazard area as described in Chapter 4. This recommendation is to avoid public expenditures on storm drain improvements that do not directly address the source of a substantial flood hazard. The reduction or elimination of major flood hazard zones caused by local creek overflow is beyond the City's control. Therefore it is recommended that medium priority storm drainage improvement projects within identified flood hazard zones be deferred until such zones have been corrected by others.

Collection System Groups

This chapter and the GIS based model are broken into collection system groups using alphanumeric codes devised by the City of Milpitas that generally correspond to major drainage facilities such as creeks or pump stations. Collection system group designations indicated on Figure 5-1 correspond to the GIS project directory summarized by Table 5-2.

All City-owned storm drain pipes 18 inches and larger in diameter have been evaluated. Additionally, smaller diameter storm drain pipes have been evaluated where their performance potentially affects local drainage conditions. For example, a 12-inch diameter pipe would be analyzed if it serves the downhill end of a cul-de-sac that lacks safe street flow conveyance. On the other hand, a 12-inch or 15-inch diameter storm drain serving a street with its own substantial flow conveyance would not be analyzed. Neither private drainage systems nor site-specific drainage systems are analyzed.

Table 5-2
Storm Drain Collection System Groups

Collection System Group Name (and Alpha Numeric Code)	Page Reference
Tularcitos Creek at Berryessa Confluence (BT1)	5-5
Coyote Creek at Oak Creek Pump Station (C1)	5-13
Coyote Creek at Murphy Pump Station (C2)	5-17
Coyote Creek at Bellew Pump Station (C3)	5-19
Coyote Creek at McCarthy Ranch (C4)	5-23
Calera Creek East of 680 (CA1)	5-27
Calera Creek West of 680 (CA2)	5-31
Ford Creek (F1)	5-37
Los Coches Creek East of 680 (L2)	5-43
Penitencia East Channel (P1)	5-49
Penitencia Creek West (P2)	5-55
Penitencia Creek at Calaveras Boulevard (P3)	5-61
Penitencia Creek at Manor Pump Station (P4)	5-67
Penitencia Creek at Dixon Landing (P5)	5-71
Penitencia Creek at Jurgens Pump Station (P6)	5-75
Penitencia Creek at Berryessa Confluence (PB1)	5-81
Piedmont Creek East of 680 (PD1)	5-87
Piedmont Creek at Berryessa Confluence (PDB1)	5-91
Tularcitos Creek East of 680 (T1)	5-95
Wrigley Creek (W1)	5-99
Wrigley / Tularcitos / Calera Creek at Jacklin Road (WTCA1)	5-105

For each collection system group, a table of statistics is provided to summarize the prioritized Capital Improvement Program recommended for that collection system group. 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 5-1:

Green Satisfactory Performance / No Improvement Necessary

Red High Priority Project

Orange Medium Priority Project

Yellow Low Priority Project

Project elements have been combined and CIP priorities adjusted as necessary for complete upstream to downstream storm drain remediation. For example downstream pipes could be lower priority than upstream pipe, but need to be installed at the same time to prevent the inducement of downstream problems. The tables of statistics associated with each collection system group give a general indication of the level of capital expenditure necessary to correct storm drain deficiencies. It is noted that sometimes additional pipe lengths must be installed to complete a corrective action.

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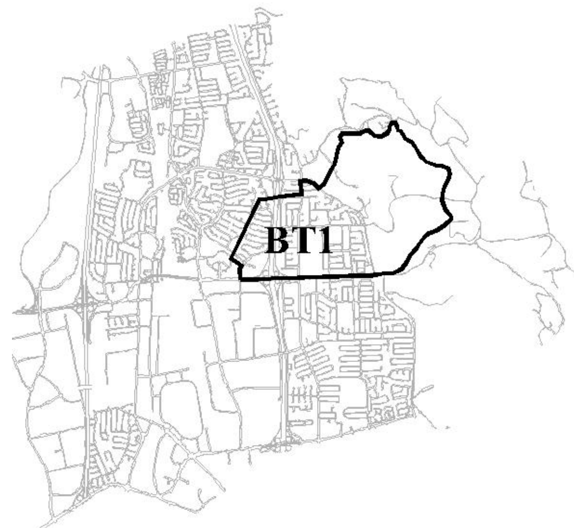
Tularcitos Creek at Berryessa Confluence (BT1)



This group of systems is on the south side of Tularcitos Creek, which drains a portion of the city's hillside areas. Between Evans Road and the freeway, the local storm drain collection systems all converge to a common outfall at the Tularcitos Creek box culvert under I-680. Two separate systems near the Evans Road intersection with Calaveras Road drain directly into Arroyo de los Coches. Several residential neighborhood storm drains located between the interstate and Tularcitos Creek have direct outfall to Tularcitos Creek. One local system draining Hillview Court actually discharges directly into Berryessa Creek.

Detention Basin at Quince Lane

Approximately 257 acres (0.4 square mile) of hillside shrub land over HSG "D", including some development and the Tularcitos Golf and Country Club, drain to a detention basin located near the intersection of Evans Road with Quince Lane. Detained runoff enters the storm drain system through a vertical 36-inch diameter CMP riser. This is a significant storage facility that attenuates peak runoff entering the storm drain system. The hydrograph method outlined in Chapter 2 is used to assess the performance of this basin. Hydrograph parameters are made compatible with the Rational Method for further downstream system analysis by modifying the effective C x A to match the peak detention basin outflow while preserving the time of concentration at the detention basin outlet. The case of detention basin outflow during the local peak runoff is also checked and the time period with maximum basin discharge controls. Table 5-3 lists the relevant hydrologic parameters.

Table 5-3
Hydrologic Parameters for Quince Lane Detention Basin

Parameter	10-year	100-year
Mean annual precipitation (inches)	17	17
24-hour rainfall (inches)	3.33	4.95
Watershed roughness (N)	0.080	0.080
Catchment length (miles)	1.15	1.15
Centroid length (miles)	0.60	0.60
Effective slope (ft/mi)	660	660
Curve Number	79	79
Percent impervious	10	10
Unit hydrograph duration (minutes)	5	5
Basin lag (hour)	0.38	0.38
Peak detention basin inflow (cfs)	82	173
Peak detention basin outflow (cfs)	78	133
Peak detention basin stage (ft NAVD)	114.9	119.0
Containment elevation (ft NAVD)	120	120
Inflow time of concentration (minutes)	38.0	38.0
Detention time (minutes)	10.2	25.2
Outflow time of concentration (minutes)	48.2	63.2
Rainfall intensity at outflow t_c (in/hr)	0.76	0.92
Effective C x A to match discharge (acres)	102.63	144.57

Figure 5-2 shows the detention basin's storage-elevation curve, calculated using Santa Clara County LiDAR topography.

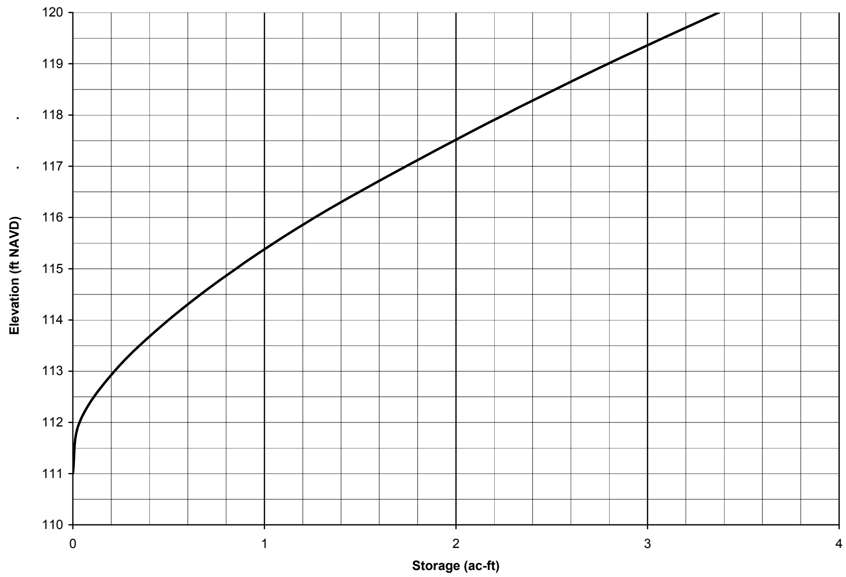


Figure 5-2: Storage Elevation Curve for Quince Lane Detention Basin

Gravity Outfalls

Due to the relative steepness of this area, storm drains discharge to receiving waters through gravity outfalls. Table 5-4 lists the ten- and 100-year starting tailwater elevations at each gravity outfall, using the criteria outlined in Chapter 3. The starting backwater for the tributary system is equivalent to the water surface elevation in the receiving water plus the exit loss at the storm drain pipe outfall.

Table 5-4
Tailwater Elevations for Storm Drain Outfalls within BT1 System

ID	Outfall Location	Ground Elev.	SD INV	Outfall Dia	Velocity (fps)		Creek WSEL (feet NAVD)		System Tailwater (feet NAVD)	
					10yr	100yr	10yr	100yr	10yr	100yr
131	Quince Lane Detention Basin	132.4	112.53	36"	2.72	4.22	114.87	119.07	115.64	119.35
152	Los Coches 475' D/S Piedmont	118.5	107.73	36"	3.66	5.25	112.92	114.41	113.13	114.84
154	Los Coches D/S Piedmont	128.3	120.73	27"	4.92	7.79	122.42	124.00	123.36	124.94
171	Berryessa Ck 110' U/S Hillview	27.8	17.33	21"	2.91	4.18	22.00	23.80	22.13	24.07
179	Tularcitos Ck at Terrabella	25.0	17.33	36"	2.54	3.87	20.94	22.04	21.04	22.27
187	Tularcitos Ck at Pacheco	25.0	19.92	24"	3.02	4.40	21.70	22.65	22.06	22.95
192	Tularcitos Ck at Alcosta	25.0	19.15	24"	2.69	3.87	22.56	23.37	22.67	23.60
197	Tularcitos Ck at Canada	26.0	19.15	21"	3.25	4.74	23.51	24.23	23.67	24.58
202	Tularcitos Ck at Tramway	27.0	20.22	18"	2.14	2.99	24.29	25.02	24.36	25.16
206	Tularcitos Ck at N Hillview	27.0	20.73	18"	2.52	3.63	25.26	26.01	25.36	26.21
208	Unnamed Hillside Creek	175.0	148.99	24"	5.56	8.36	n/a	n/a	151.47	152.08
1026	Tularcitos Ck at Interstate 680	31.0	21.62	72"	7.41	10.72	25.90	26.80	28.47	29.40

Collection System Performance

Table 5-5 presents CIP statistics by priority for the BT1 system. More than twenty percent of the analyzed system requires remediation. The storm drain on North Hillwood Drive [link 204 in Figure 5-3] does not meet the performance standard because the 100-year water surface elevation in Tularcitos Creek is higher than the street grade at the intersection with Del Rio Court. However, given the rare frequency and limited duration of street flooding in this location, installing a pump station to remedy the issue is not recommended as part of the CIP.

Table 5-5
Recommended CIP for Collection System BT1

	Lineal Feet	Percentage
System Acceptable / No Improvements	25,728	77
High Priority Improvements	1,935	6
Medium Priority Improvements	4,230	12
Low Priority Improvements	1,630	5
Total System	**33,523**	**100**

Capital Projects

Table 5-6 identifies capital projects to correct inadequate storm drain capacity caused primarily by undersized pipe and occasional flat or adverse street grades in the vicinity. Figure 5-3 shows the location and priority of each identified capital project. Options for parallel relief drains and full replacement are provided. Generally installing a parallel relief drain is less expensive, depending upon the number and location of existing street utilities.

The most critical problems are undersized storm drains within easements between Traughber Street and Calle Oriente and between Park View Drive and Kennedy Drive. In these locations, overloaded storm drains could flood properties in a design 10-year runoff event. The following CIP projects, completed in order remedy the problems: Traughber Street Storm Drain Replacement (ID 3), Wool Drive Storm Drain Improvement (ID 4), and Park View Drive Storm Drain Improvement (ID 5).

Other listed improvements are low and medium priorities. In the case of the Calaveras Ridge Drive Relief Drain, while street overflows have a safe release to a natural hillside draw that is tributary to an unnamed creek, the storm drain improvement is recommended as a medium priority to avoid hillside erosion. Storm drains discharging directly to Arroyo de los Coches (Pipe 150) and Tularcitos Creek (Pipes 201 and 204) do not have sufficient capacity to prevent storm drain backup during the design 100-year runoff event, and street flooding could occur in local depressions.

Sedimentation within this collection system is another potential problem, since steep hillside areas are tributary to the local collection system. Once topography flattens west of Evans Road, the sediment load gathered from the steep hillside could drop out and potentially block storm drains. Upstream debris basins and storm drain inlet retrofits are recommended at the locations shown on Figure 5-3 to improve this maintenance issue. Chapter 9 describes inlet retrofitting in more detail.

Table 5-6
Recommended Capital Improvements in System BT1

ID	Project	Priority	Parallel Option	Replacement Option
1	Traughber Street SD Replacement	High	Use replacement option.	Replace approx. 300 LF of existing 36-inch RCP with 72-inch RCP across Traughber Street between the existing 24-inch SD from Burdett Way and the existing 72-inch outfall to Tularcitos Creek.
2	Wool Drive SD Improvement	High	Install approx. 1,210 LF of 42-inch RCP in Wool Drive from Kennedy Drive to the existing 24-inch SD from Burdett Way near Traughber Street.	Replace approx. 1,210 LF of existing 27-inch RCP with 48-inch RCP in Wool Drive from Kennedy Drive to the existing 24-inch SD from Burdett Way near Traughber Street.
3	Park View Drive SD Improvement	High	Use replacement option.	Replace approx. 175 LF of existing 27-inch RCP with 42-inch RCP in a storm drain easement between Park View Drive and Kennedy Drive. Install approx. 250 LF of 42-inch RCP in Kennedy Drive from easement to Wool Drive.
4	Tramway Drive SD Improvement	Medium	Install approx. 530 LF of 18-inch RCP in Tramway Dr. from N. Hillview Dr. to Tularcitos Ck outfall.	Replace approx. 530 LF of existing 18-inch RCP in Tramway Dr. with 24-inch RCP from N. Hillview Dr. to Tularcitos Ck and upsize the creek outfall.
5	Calaveras Road Outfall Relocation	Medium	Use replacement option	Relocate existing 36-inch outfall to Arroyo de las Coches at Temple Drive (existing 8' x 6' RCB). Plug existing outfall and extend 36-inch storm drain pipe in Calaveras Road approx. 800 LF to new outfall.
6	Fanyon Street SD Improvement	Medium	Install approx. 1,150 LF of 24-inch RCP in Fanyon Street from Dennis Avenue to Kennedy Drive.	Replace approx. 1,150 LF of existing 27-inch RCP with 36-inch RCP in Fanyon Street from Dennis Avenue to Kennedy Drive.
7	Temple Drive SD Improvement	Medium	Install approx. 205 LF of 24-inch RCP in Temple Drive from Fair Hill Drive to Kennedy Drive. Install approx. 1,230 LF of 24-inch RCP in Kennedy Drive from Temple Drive to Fanyon Street.	Replace approx. 205 LF of existing 30-inch RCP with 36-inch RCP in Temple Drive from Fair Hill Drive to Kennedy Drive. Replace approx. 350 LF of existing 30-inch RCP and 880 LF of existing 33-inch RCP with 36-inch RCP in Kennedy Drive from Temple Drive to Fanyon Street.
8	Calaveras Ridge Dr. SD Improvement	Medium	Install approx. 315 LF of 18-inch RCP in Calaveras Ridge Drive parallel to existing 18-inch RCP storm drain.	Replace approx. 315 LF of 18-inch RCP storm drain in Calaveras Ridge Drive with 24-inch storm drain.

ID	Project	Priority	Parallel Option	Replacement Option
9	Park Hill Drive SD Improvement	Low	Install approx 820 LF of 24-inch RCP in Park Hill Dr from Park Grove Dr to Park Heights Dr. Install approx. 810 LF of 30-inch RCP in Park Hill Drive from Park Heights Dr. to Park View Dr.	Replace approx 820 LF of existing 12-inch and 15-inch RCP with 30-inch RCP in Park Hill Dr from Park Grove Dr to Park Heights Dr. Replace approx. 810 LF of existing 21-inch RCP in Park Hill Drive with 36-inch RCP from Park Heights Dr. to Park View Dr.
10	Debris Basins and Storm Drain Inlet Modifications	Medium	Per Figure 5-13. Debris basin size to be determined from criteria presented in Chapter 3 and specific conditions at each location determined during the design phase.	

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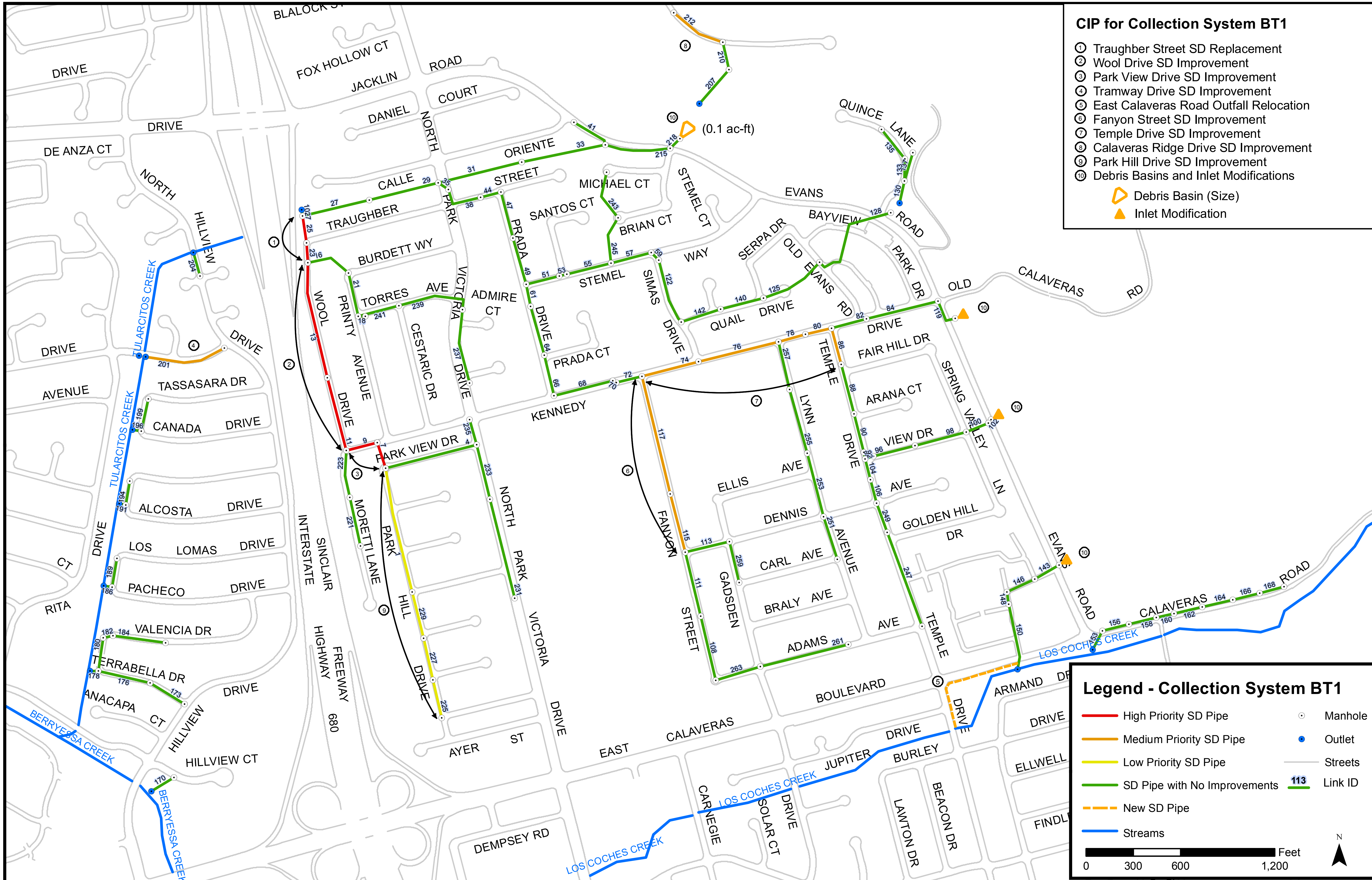
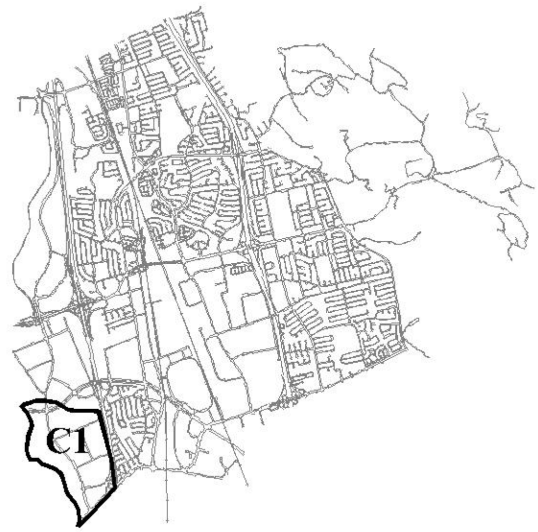

CIP for Collection System BT1
1 Traughber Street SD Replacement
2 Wool Drive SD Improvement
3 Park View Drive SD Improvement
4 Tramway Drive SD Improvement
5 East Calaveras Road Outfall Relocation
6 Fanyon Street SD Improvement
7 Temple Drive SD Improvement
8 Calaveras Ridge Drive SD Improvement
9 Park Hill Drive SD Improvement
10 Debris Basins and Inlet Modifications
Debris Basin (Size)
Inlet Modification
(0.1 ac-ft)
Legend - Collection System BT1
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
New SD Pipe
Streams
Manhole
Outlet
Streets
113 Link ID
Feet
0 300 600 1,200
N
TULARCITOS CREEK
BERRYESSA CREEK
LOS COCHES CREEK
INTERSTATE 680
SINCLAIR FREEWAY
EAST CALAVERAS BOULEVARD
CALAVERAS ROAD

Figure 5-3

Coyote Creek at Oak Creek Pump Station (C1)



Levee improvements made by the Santa Clara Valley Water District have removed this interior area from the Coyote Creek floodplain. There is no natural way for runoff to drain from this area, so all storm drain collection systems lead to the Oak Creek Pump Station, which discharges into Coyote Creek. This area is bound by Coyote Creek to the west, the Nimitz Freeway to the east, Tasman Drive on the north, and Montague Expressway to the south.

The FIRM shows a Shaded Zone X flood hazard, indicating that the area is protected by levees, and small areas of shallow residual 100-year flooding (Figure 4-1).

Outfall to Pump Station

Table 5-7 lists pump station operating parameters and their effect on backwater conditions for the storm drain analyses. The starting backwater for the tributary system is equivalent to the water surface elevation in the pump wet well plus the exit loss at the influent storm drain pipe.

Table 5-7
Hydraulics at Oak Creek Pump Station Outfall

Hydraulic Parameter	10-year	100-year
WSEL in Coyote Creek (feet NAVD)	39	40
Design Inflow (cfs)	190	288
Number of Pumps Operating	2	3
Pump Station Wet Well Level (feet NAVD)	19.00	19.50
84-inch Inflow Pipe Velocity (fps)	4.94	7.48
Pipe Exit Loss (foot)	0.38	0.87
Storm Drain Tailwater (feet NAVD)	19.38	20.37

Collection System Performance

Oak Creek Pump Station has sufficient capacity to discharge the 100-year design runoff from its tributary drainage basin. Within the collection system, however, some slightly undersized pipe contributes to surcharged and overloaded conditions, whereby there will be some unpredictable, shallow flooding through individual industrial properties where streets have no natural outlet. Since structures are likely padded up at least one foot above surrounding grade, damage should be limited to parking lots and landscape areas. However, site conditions will vary and berms or other obstructions could force excess runoff to enter buildings. Site-specific surveys are required to document this.

Table 5-8 summarizes the storm drain system as categorized in the prioritized CIP. Some storm drains need to be improved at a higher priority to avoid inducing downstream drainage problems, and some storm drains have substandard performance issues resolved without the need for direct replacement.

Table 5-8
Recommended CIP for Collection System C1

	Lineal Feet	Percentage
System Acceptable / No Improvements	11,854	70
High Priority Improvements	1,270	8
Medium Priority Improvements	3,795	22
Low Priority Improvements	0	0
Total System	**16,919**	**100**

Capital Projects

Table 5-9 identifies capital projects that correct inadequate storm drain capacity caused by the flat and adverse street grades in the vicinity. Figure 5-4 shows the location of each capital project. Options for parallel relief drains and full replacement are provided. Generally, installing a parallel relief drain is less expensive depending upon the number and location of existing street utilities. In the case of the Buckeye Court Storm Drain Replacement Project, the undersized pipe is located within a storm drainage easement between private properties, so a parallel pipe option is assumed to be infeasible given the difficulty of constructing a second storm drain within a limited public utility easement without disturbing the existing pipe.

Table 5-9
Recommended Capital Improvements in System C1

ID	Project	Priority	Parallel Option	Replacement Option
1	Sycamore Drive SD Improvements	High	Install approx. 1,270 LF of 42-inch RCP in Sycamore Drive from Barber Lane to Buckeye Drive.	Replace approx. 480 LF of exist 27-inch RCP, 360 LF of exist 30-inch RCP and 430 LF of exist 33-inch RCP with 48-inch RCP in Sycamore Drive from Barber Lane to Buckeye Dr.
2	Buckeye Court SD Replacement	Medium	Use replacement option.	Replace approx. 685 LF of existing 27-inch RCP and 440 LF of existing 33-inch RCP with 36-inch RCP in the storm drain esmt from the Barber Ct cul-de-sac to Sycamore Dr.
3	Cottonwood Drive SD Improvements	Medium	Install approx. 1,400 LF of 24-inch RCP in Barber Lane near Cottonwood Drive and in Cottonwood Drive from Barber Lane to Buckeye Drive.	Replace approx. 550 LF of existing 21-inch RCP with 30-inch RCP in Barber Lane near Cottonwood Dr; 280 LF of existing 27-inch RCP in Cottonwood Dr with 36-inch RCP; and 570 LF of existing 33-inch RCP with 42-inch RCP in Cottonwood Dr to Buckeye Dr.
4	Barber Lane SD Improvements	Medium	Install approx. 780 LF of 36-inch RCP in Barber Lane near McCarthy Boulevard.	Replace approx. 780 LF of existing 30-inch RCP with 48-inch RCP in Barber Lane near McCarthy Blvd.
5	McCarthy Blvd. SD Improvements	Medium	Install approx. 490 LF of 36-inch RCP in McCarthy Boulevard south of Barber Lane.	Replace approx. 490 LF of existing 24-inch RCP with 42-inch RCP in McCarthy Blvd south of Barber Lane.

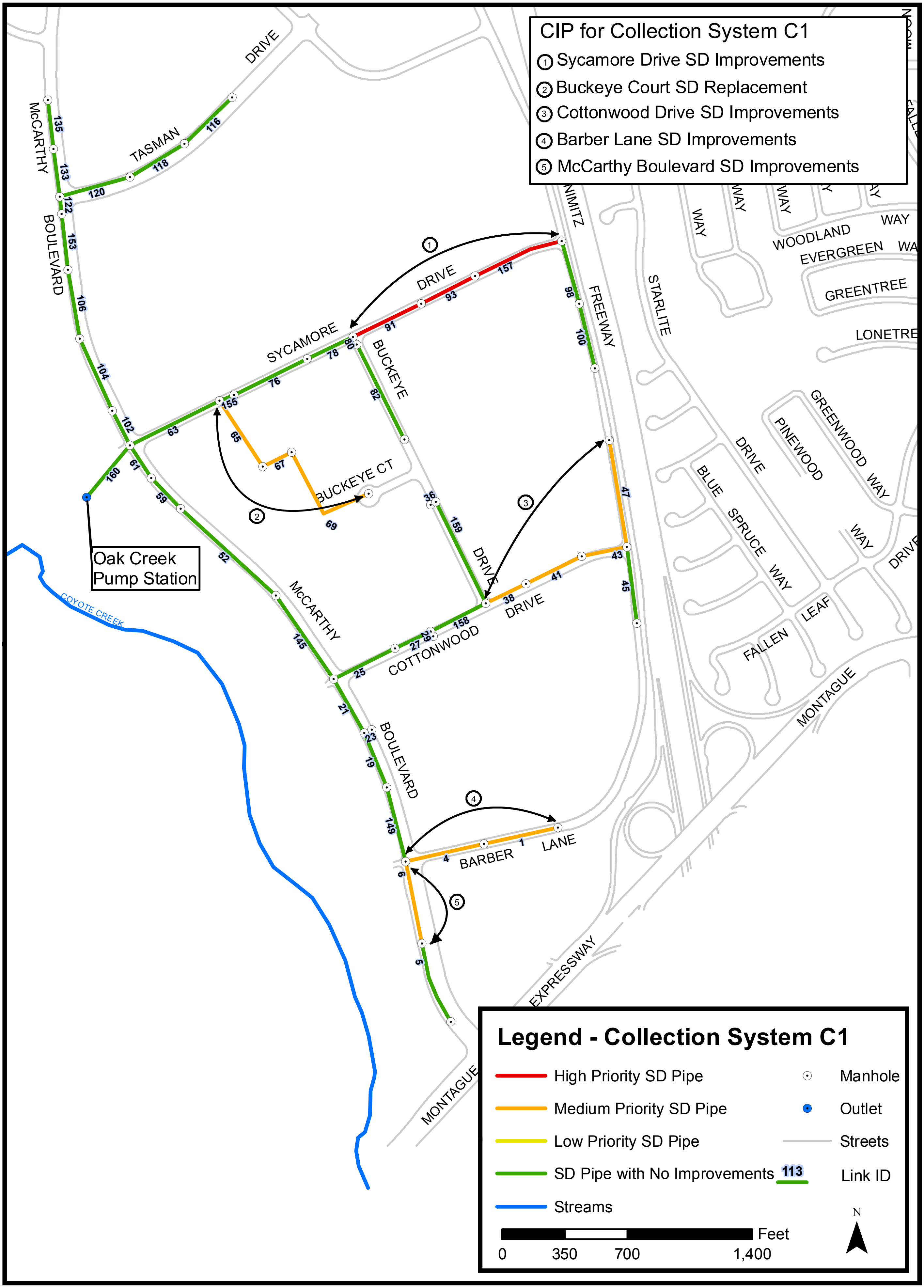
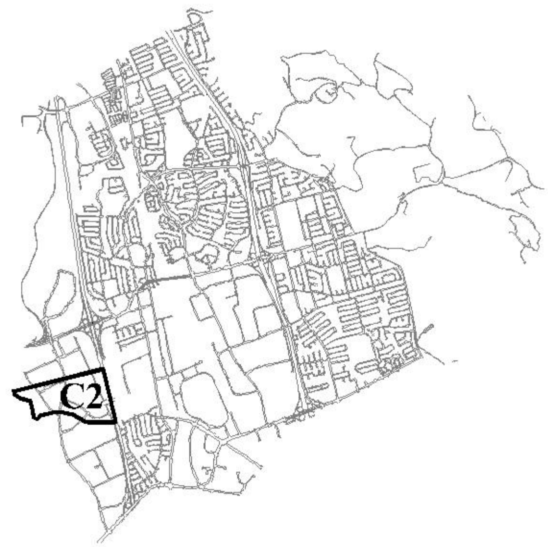

CIP for Collection System C1
1 Sycamore Drive SD Improvements
2 Buckeye Court SD Replacement
3 Cottonwood Drive SD Improvements
4 Barber Lane SD Improvements
5 McCarthy Boulevard SD Improvements
Oak Creek Pump Station
COYOTE CREEK
McCARTHY BOULEVARD
TASMAN DRIVE
SYCAMORE DRIVE
BUCKEYE DRIVE
BUCKEYE CT
COTTONWOOD DRIVE
BARBER LANE
NIMITZ FREEWAY
STARLITE DRIVE
BLUE SPRUCE WAY
PINEWOOD
GREENWOOD WAY
WOODLAND WAY
EVERGREEN
GREENTREE
LONETRE
FALLEN LEAF DRIVE
MONTAGUE EXPRESSWAY
MONTAGUE
Legend - Collection System C1
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
Streams
Manhole
Outlet
Streets
113
Link ID
Feet
0
350
700
1,400
N

Figure 5-4

Coyote Creek at Murphy Pump Station (C2)

Similar in nature to the Oak Creek storm drain system, this drainage is located immediately to the north. Storm drains also discharge to Coyote Creek, but through the Murphy Pump Station. There is a 39-inch diameter storm drain inter-tie to the Bellew Pump Station system (C3) on Ranch Road. The area is mapped entirely as Shaded Zone X, indicating levee protection.



Outfall to Pump Station

Table 5-10 lists pump station operating parameters and their effect on backwater conditions for the storm drain analyses. The starting backwater for the tributary system is equivalent to the water surface elevation in the pump wet well plus the exit loss at the influent storm drain pipe.

Table 5-10
Hydraulics at Murphy Pump Station Outfall

Hydraulic Parameter	10-year	100-year
WSEL in Coyote Creek (feet NAVD)		32.5
Design Inflow (cfs)	64	110
Number of Pumps Operating	1	2
Pump Station Wet Well Level (feet NAVD)	18.00	18.50
66-inch Inflow Pipe Velocity (fps)	2.70	4.63
Pipe Exit Loss (foot)	0.11	0.33
Storm Drain Tailwater (feet NAVD)	18.11	18.83

Collection System Performance

Murphy Pump Station has more than enough capacity to discharge the 100-year design runoff from its tributary drainage area. All analyzed storm drain pipes in the system meet storm drain master plan performance criteria as shown in Table 5-11 and Figure 5-5.

Table 5-11
Collection System C2 Performance

	Lineal Feet	Percentage
System Acceptable / No Improvements	7,107	100
High Priority Improvements	0	0
Medium Priority Improvements	0	0
Low Priority Improvements	0	0
Total System	**7,107**	**100**

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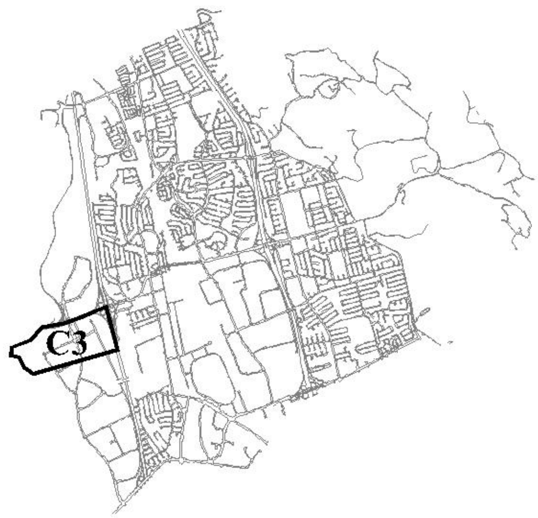
Coyote Creek at Bellew Pump Station (C3)



Located just north of Murphy Pump Station, the Bellew Pump Station also drains a closed industrial area; in this case along Bellew Drive. There is a 39-inch storm drain inter-tie to the Murphy system (C2). This area is mapped entirely as Shaded Zone X, indicating that it is protected from creek flooding by levees.

Outfall to Pump Station

Table 5-12 lists pump station operating parameters and their effect on backwater conditions for the storm drain analyses. The starting backwater for the tributary system is equivalent to the water surface elevation in the pump wet well plus the exit loss at the influent storm drain pipe.

Table 5-12
Hydraulics at Bellew Pump Station Outfall

Hydraulic Parameter	10-year	100-year
WSEL in Coyote Creek (feet NAVD)		30.0
Design Inflow (cfs)	142	243
Number of Pumps Operating	2	2
Pump Station Wet Well Level (feet NAVD)	13.50	13.50
84-inch Inflow Pipe Velocity (fps)	5.01	8.60
Pipe Exit Loss (foot)	0.39	1.15
Storm Drain Tailwater (feet NAVD)	13.89	14.65

Collection System Performance

Bellew Pump Station has sufficient capacity to discharge the 100-year design runoff from its tributary drainage area. With the exception of predicted 100-year street ponding in excess of the allowable criterion on Sumac Drive between McCarthy Boulevard and Murphy Ranch Road, all analyzed storm drain pipes in the system meet storm drain master plan performance criteria as shown in Table 5-13 and Figure 5-5. Additional storm drains in Murphy Ranch Road and Sumac Drive must be remediated at a medium priority to relieve the problem in Sumac Drive.

Table 5-13
Recommended CIP for Collection System C3

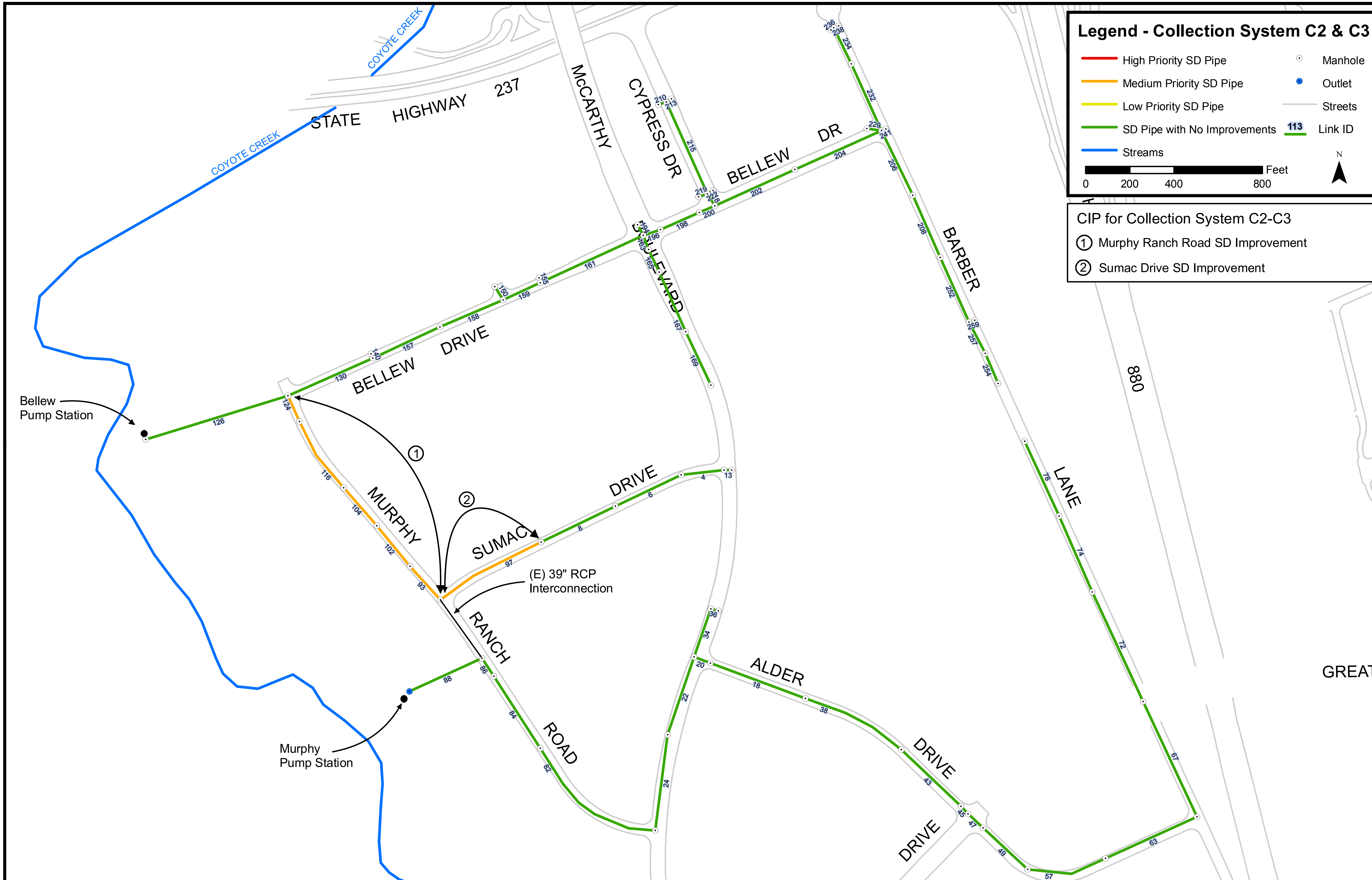
	Lineal Feet	Percentage
System Acceptable / No Improvements	7,970	83
High Priority Improvements	0	0
Medium Priority Improvements	1,610	17
Low Priority Improvements	0	0
Total System	**9,580**	**100**

Capital Projects

Table 5-14 identifies capital projects to correct inadequate storm drain capacity caused by the flat and adverse street grades along Sumac Drive. Figure 5-5 shows the location of each capital project. Options for parallel relief drains and full replacement are provided. Generally installing a parallel relief drain is less expensive, depending upon the number and location of existing street utilities.

Table 5-14
Recommended Capital Improvements in System C3

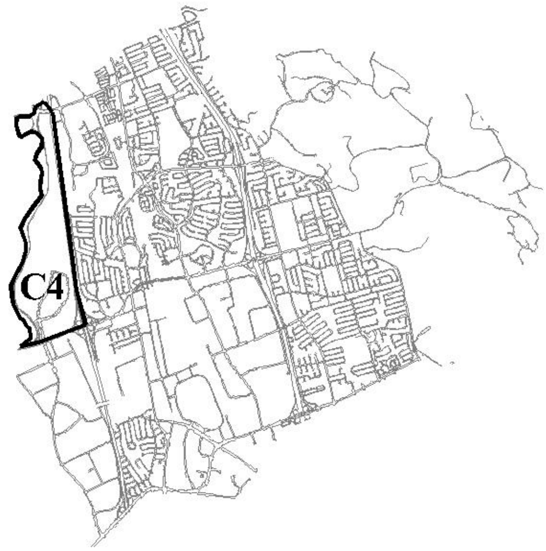
ID	Project	Priority	Parallel Option	Replacement Option
1	Murphy Ranch Road SD Improvement	Medium	Install approx. 1,160 LF of 36-inch RCP in Murphy Ranch Road from Sumac Drive to Bellew Drive.	Replace approx. 190 LF of existing 39-inch RCP with 48-inch RCP, 420 LF of existing 42-inch RCP with 54-inch RCP, and 550 LF of existing 48-inch RCP with 60-inch RCP in Murphy Ranch Road from Sumac Drive to Bellew Drive.
2	Sumac Drive SD Improvement	Medium	Install approx. 450 LF of 36-inch RCP in Sumac Drive from the sag to Murphy Ranch Road.	Replace approx. 450 LF of existing 36-inch RCP with 48-inch RCP in Sumac Drive from the sag to Murphy Ranch Road.



Legend - Collection System C2 & C3
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
Streams
Manhole
Outlet
Streets
113
Link ID
N
Feet
0
200
400
800
CIP for Collection System C2-C3
1 Murphy Ranch Road SD Improvement
2 Sumac Drive SD Improvement
STATE HIGHWAY 237
COYOTE CREEK
COYOTE CREEK
McCARTHY
CYPRESS DR
BELLEW DR
BELLEW DRIVE
BOULEVARD
BARBER LANE
880
GREAT
SUMAC DRIVE
MURPHY RANCH ROAD
ALDER DRIVE
DRIVE
Bellew Pump Station
Murphy Pump Station
(E) 39" RCP Interconnection
1
2

Figure 5-5

Coyote Creek at McCarthy Ranch (C4)



Located at the northwestern corner of Milpitas, McCarthy Ranch is a planned mixed use development north of Interstate 237 that is not yet fully built out. As is the case for its industrial neighbors to the south, Coyote Creek levees close the area to natural drainage. Hence the McCarthy Ranch Pump Station discharges all local drainage to Coyote Creek. The entire area is mapped as a Shaded Zone X, indicating levee protection against one-percent flooding.

Outfall to Pump Station

Table 5-15 lists pump station operating parameters and their effect on backwater conditions for the storm drain analyses. The starting backwater for the tributary system is equivalent to the water surface elevation in the pump wet well plus the exit loss at the influent storm drain pipe.

Table 5-15
Hydraulics at McCarthy Pump Station Outfall

Hydraulic Parameter	10-year	100-year
WSEL in Coyote Creek (feet NAVD)	14.0	14.5
Design Inflow (cfs)	173	290
Number of Pumps Operating	2	3
Pump Station Wet Well Level (feet NAVD)	1.00	1.50
78-inch Inflow Pipe Velocity (fps)	5.21	8.74
Pipe Exit Loss (foot)	0.42	1.19
Storm Drain Tailwater (feet NAVD)	1.42	2.69

Collection System Performance

The McCarthy Ranch Pump Station and its tributary collection system meet all evaluation criteria for both the 10- and 100-year design discharges (Figure 5-6).

Table 5-16
Collection System C4 Performance

	Lineal Feet	Percentage
System Acceptable / No Improvements	10,058	100
High Priority Improvements	0	0
Medium Priority Improvements	0	0
Low Priority Improvements	0	0
Total System	**10,058**	**100**

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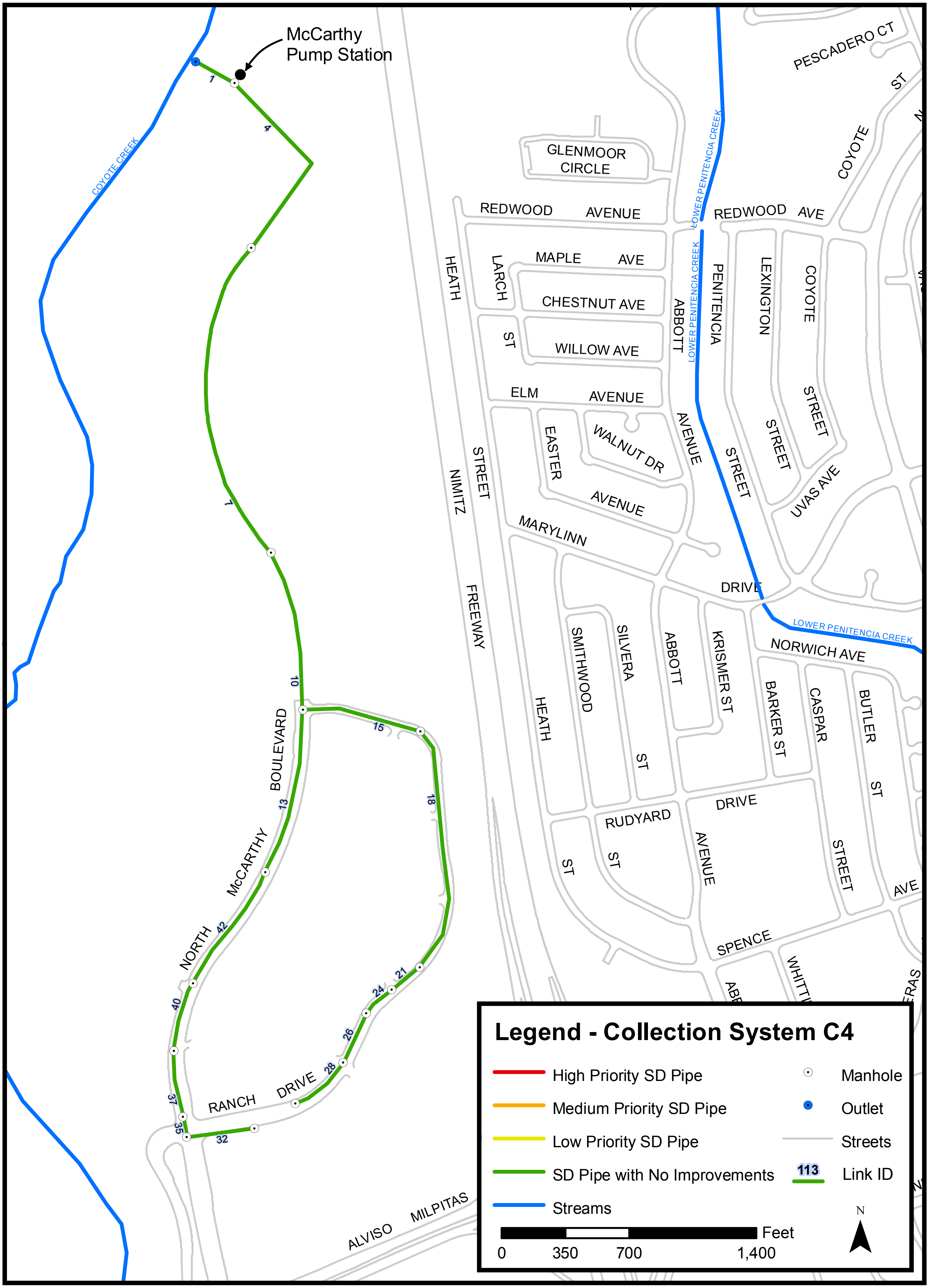
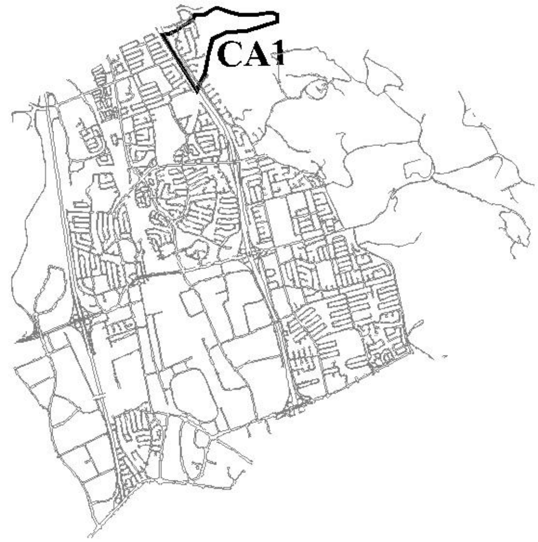

McCarthy Pump Station
COYOTE CREEK
LOWER PENITENCIA CREEK
GLENMOOR CIRCLE
PESCADERO CT
COYOTE ST
REDWOOD AVENUE
REDWOOD AVE
MAPLE AVE
CHESTNUT AVE
WILLOW AVE
ELM AVENUE
LARCH ST
HEATH STREET
NIMITZ FREEWAY
ABBOTT AVENUE
PENITENCIA STREET
LEXINGTON STREET
COYOTE STREET
WALNUT DR
EASTER AVENUE
UVAS AVE
MARYLINN DRIVE
NORWICH AVE
HEATH ST
SMITHWOOD ST
SILVERA ST
ABBOTT AVENUE
KRISMER ST
BARKER ST
CASPAR STREET
BUTLER ST
AVE
RUDYARD DRIVE
ST
SPENCE
NORTH McCARTHY BOULEVARD
RANCH DRIVE
ALVISO MILPITAS
1
4
7
10
13
15
18
21
24
26
28
32
35
37
40
42
Legend - Collection System C4
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
Streams
Manhole
Outlet
Streets
113
Link ID
N
Feet
0
350
700
1,400

Figure 5-6

Calera Creek East of 680 (CA1)



Local runoff drains through a collection system to Calera Creek, on the upstream side of North Park Victoria Drive (at Interstate 680). Scott Creek is immediately to the north. This collection system is fairly small in size and characterized by good drainage.

There are no documented problems with local drainage, nor any master plan improvements required. In fact the area is not in Calera Creek's 100-year floodplain.

Gravity Outfall at Calera Creek

Due to the relative steepness of this area, storm drains discharge to Calera Creek at North Park Victoria Drive through a gravity outfall. Table 5-17 lists the 10- and 100-year starting tailwater elevations at the gravity outfall, using the criteria outlined in Chapter 3. The starting backwater for the tributary system is equivalent to the water surface elevation in the receiving water plus the exit loss at the storm drain pipe outfall.

Table 5-17
Tailwater Elevation for Storm Drain Outfall within CA1 System

ID	Outfall Location	Ground Elev.	SD INV	Outfall Dia	Velocity (fps)		Creek WSEL (feet NAVD)		System Tailwater (feet NAVD)	
					10yr	100yr	10yr	100yr	10yr	100yr
2	Calera Ck at N Park Victoria	110.0	101.08	78”	3.75	6.06	105.00	107.00	105.30	107.57

Collection System Performance

Table 5-18 presents the analytical performance statistics for the CA1 system. All analyzed storm drains meet the stated performance criteria (Figure 5-7).

Table 5-18
Collection System CA1 Performance

	Lineal Feet	Percentage
System Acceptable / No Improvements	6,990	100
High Priority Improvements	0	0
Medium Priority Improvements	0	0
Low Priority Improvements	0	0
Total System	**6,990**	**100**

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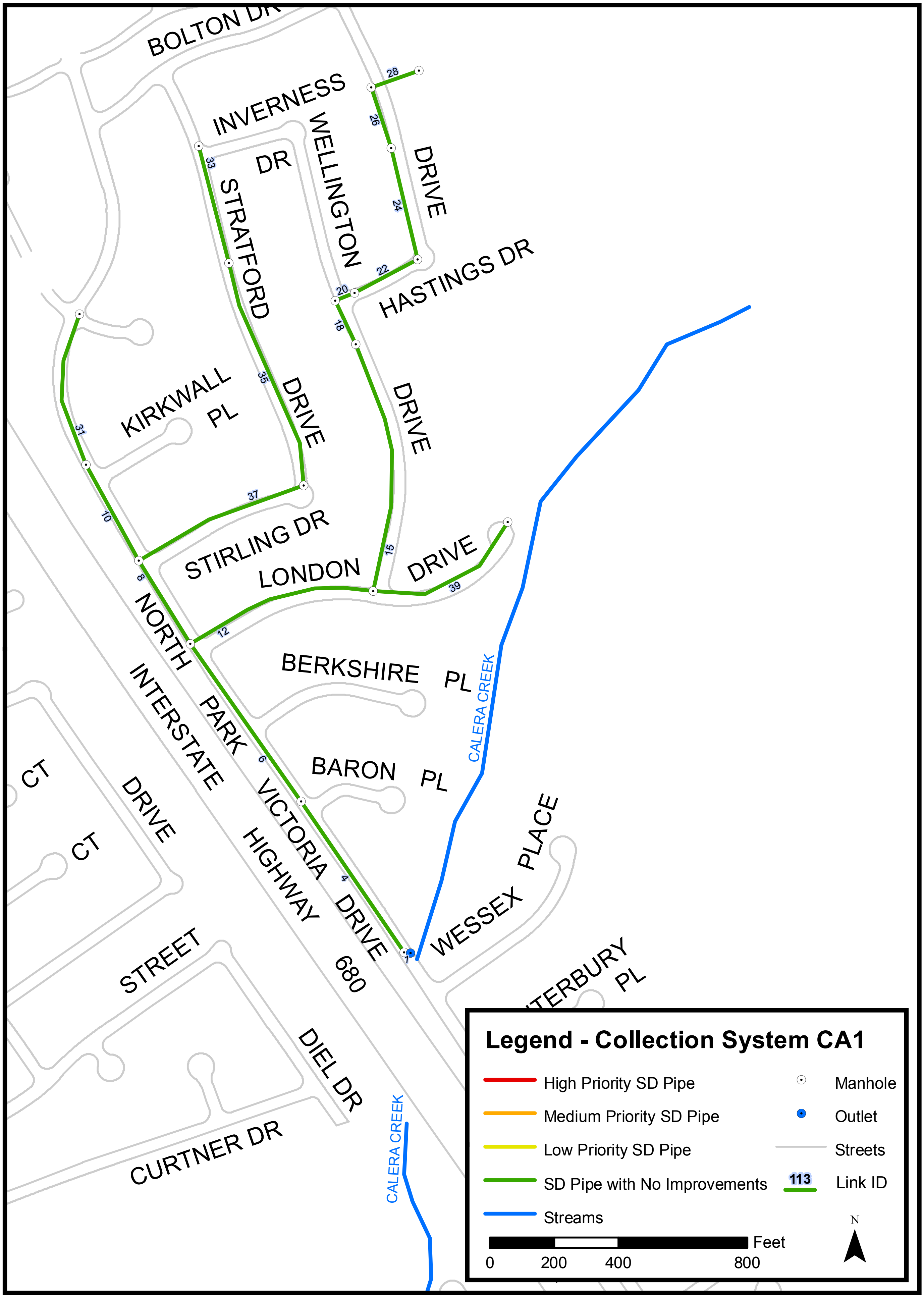
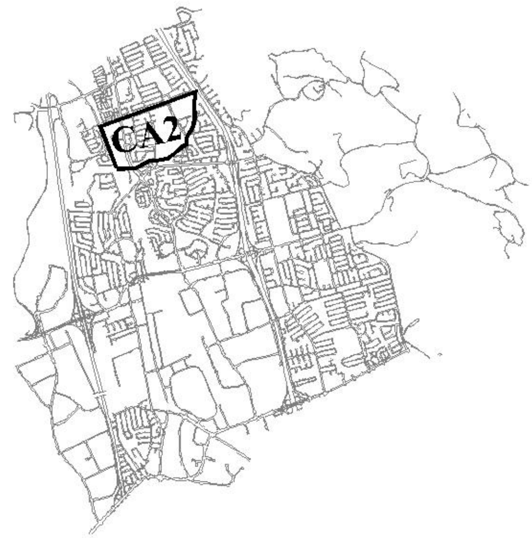


BOLTON DR
INVERNESS
DR
WELLINGTON
DRIVE
STRATFORD
DRIVE
HASTINGS DR
KIRKWALL
PL
DRIVE
STIRLING DR
LONDON
DRIVE
NORTH
BERKSHIRE
PL
CALERA CREEK
INTERSTATE
PARK
BARON
PL
CT
CT
DRIVE
VICTORIA
HIGHWAY
PLACE
WESSEX
DRIVE
680
STREET
TERBURY
PL
DIEL DR
CURTNER DR
CALERA CREEK
28
26
24
22
20
18
33
35
31
37
10
15
39
8
12
6
4
1
Legend - Collection System CA1
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
Streams
Manhole
Outlet
Streets
113
Link ID
N
Feet
0
200
400
800

Figure 5-7

Calera Creek West of 680 (CA2)



Residential areas in the eastern three-quarters of this area drain directly to Calera Creek through local outfalls. One outfall is located at Escuela Parkway, the other at North Milpitas Boulevard. The Minnis Circle area drains to the Minnis Pump Station at Calera Creek near the Union Pacific Railroad. The pump station discharges to Calera Creek just above the confluence with Berryessa Creek. Most of this area is located within a Shaded Zone X on the latest flood insurance maps, meaning properties are at risk in events with greater than 100-year occurrence intervals, or are protected by levees. A fairly small area near the pump station is located within a mapped AH zone, indicating the risk for 100-year flooding from Calera Creek.

Gravity Outfalls at Calera Creek

Most storm drains in the system discharge to Calera Creek through gravity outfalls. Table 5-19 lists the 10- and 100-year starting tailwater elevations at each gravity outfall, using the criteria outlined in Chapter 3. The starting backwater for the tributary system is equivalent to the water surface elevation in the receiving water plus the exit loss at the storm drain pipe outfall.

Table 5-19
Tailwater Elevations for Storm Drain Outfalls within CA2 System

ID	Outfall Location	Ground Elev.	SD INV	Outfall Dia	Velocity (fps)		Creek WSEL (feet NAVD)		System Tailwater (feet NAVD)	
					10yr	100yr	10yr	100yr	10yr	100yr
2	Calera at Escuela Pkwy (D/S)	42.0	40.13	18”	1.44	2.07	39.19	39.92	41.66	41.70
49	Calera at N Milpitas Blvd	20.0	9.69	42”	4.54	7.16	15.37	16.05	15.69	16.85
99	Calera at Arizona Ave (U/S)	23.0	18.77	18”	1.28	1.99	15.55	19.80	20.30	20.33
102	Calera 100’ U/S Arizona Ave	23.0	16.78	36”	2.81	4.06	16.03	20.60	19.90	20.86
1005	Calera at Escuela Pkwy (U/S)	45.0	39.05	30”	3.65	5.34	41.63	43.15	41.84	43.59

Outfall to Minnis Pump Station

Table 5-20 lists pump station operating parameters, assuming capital improvements recommended in Chapter 6 are implemented, and their effect on backwater conditions for the storm drain analyses is achieved. (Recommended capital improvements do not affect operation for the 10-year design event.)

Table 5-20
Hydraulics at Minnis Pump Station Outfall

Hydraulic Parameter	10-year	100-year
WSEL in Calera Creek (feet NAVD)	14.85	15.74
First Pump Start Level (feet NAVD)	4.0	4.0
Design Inflow (cfs)	27	40
Number of Pumps Operating	2	2
Pump Station Wet Well Level (feet NAVD)	4.50	4.50
27-inch Inflow Pipe Velocity (fps)	6.71	9.98
Pipe Exit Loss (feet)	0.70	1.55
Storm Drain Tailwater (feet NAVD)	5.20	6.05

Collection System Performance

Table 5-21 presents the CIP statistics for the CA2 system. Although about 20 percent of the analyzed storm drains do not meet the stated performance criteria, improving ten percent of the storm drain system will rectify those problems. Problem areas are concentrated at natural topographic depressions near the north bank of Calera Creek. The recommended CIP realizes some efficiency in improvement priority.

Table 5-21
Recommended CIP for Collection System CA2

	Lineal Feet	Percentage
System Acceptable / No Improvements	11,698	90
High Priority Improvements	0	0
Medium Priority Improvements	100	1
Low Priority Improvements	1,130	9
Total System	**12,928**	**100**

Minnis Circle is a four cul-de-sac loop in a natural bowl with no drainage outlet. Hence the Minnis Pump Station has been constructed to pump water into Calera Creek. If the pump station does not function, a disaster could occur even during modest runoff events. (Without an operable pump station, ponded water can reach depths of up to four feet before it releases to Calera Creek, no matter the magnitude of storm runoff.) Although the pumps are driven by electric motors, no provision has been made for standby power, which would enable the pumps to continue operation during PG&E power outages. Providing standby power a high priority project as recommended in Chapter 6.

Even assuming that the pump station functions properly, system surcharging and overflow during 100-year conditions could also cause significant off-street ponding of water that must travel overland to find other inlets available to carry the water to the pump station. Since the ponding area is also identified as a special flood hazard zone from Calera Creek flooding, the need for improvements is considered to be low priority until Calera Creek improvements are made by others.

North Milpitas Boulevard is at adverse grade near Calera Creek. Hydraulic conditions during the 100-year event force excess runoff to spill through adjacent properties to the west toward the Minnis Circle bowl, where excess runoff is blocked by the railroad. This water would be stored there and eventually pumped out once the local peak discharge has passed.

A storm system that drains the low point of Sudbury Drive is undersized. As a consequence, 100-year runoff backs up along Sudbury Drive to Midwick Drive, along Kovanda Way, and part of Berrendo Drive. Fortunately, the ponding is limited to the immediate street frontage, so the risk for substantial property damage is minimized somewhat.

Capital Improvements

Table 5-22 identifies capital projects to correct inadequate storm drain capacity caused primarily by undersized pipe within locally depressed areas without natural relief.

Table 5-22
Recommended Capital Improvements in System CA2

ID	Project	Priority	Parallel Option	Replacement Option
1	Minnis Pump Station Standby Power	High	Add automatic standby power. (See also Page 6-17)	
2	North Milpitas Boulevard SD Relief	Medium	Install approx. 100 LF of 42-inch RCP in North Milpitas Boulevard and a new 42-inch outfall to Calera Creek.	Replace approx. 100 LF of existing 42-inch RCP in North Milpitas Boulevard with 54-inch RCP and replace the existing 42-inch creek outfall with a 54-inch RCP creek outfall.
3	Minnis Circle SD Replacement	Low	Use replacement option.	Replace approx. 140 LF of existing 24-inch RCP on southwest side of Minnis Circle with 48-inch RCP; and approx. 990 LF of existing 27-inch RCP with 48-inch RCP along UPRR to Minnis Pump Station.
4	Minnis Pump Station Replacement	Low	Replace Minnis Pump Station pumps and electrical equipment to provide 100-year pumping capacity. (See also Page 6-17)	

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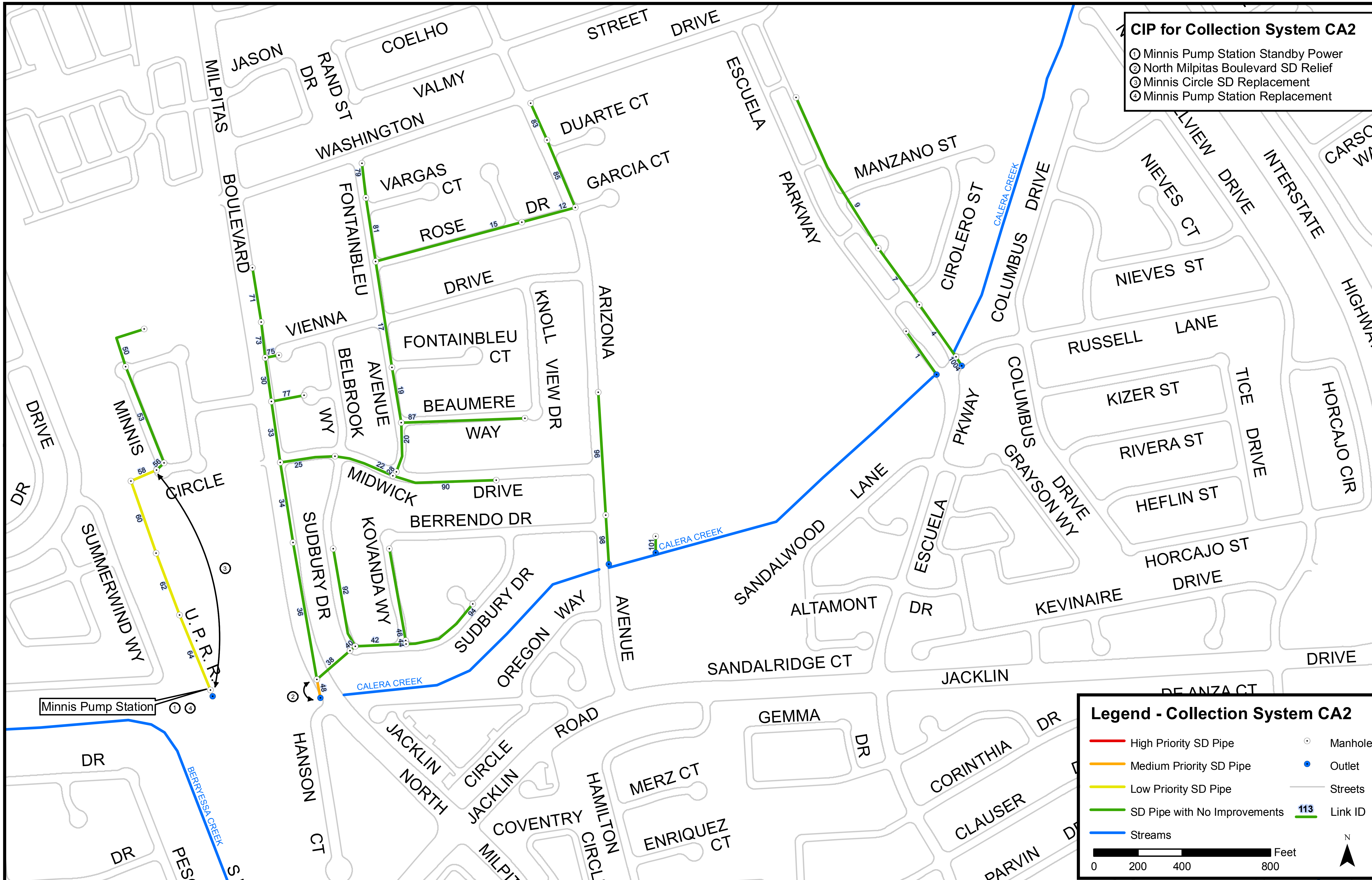
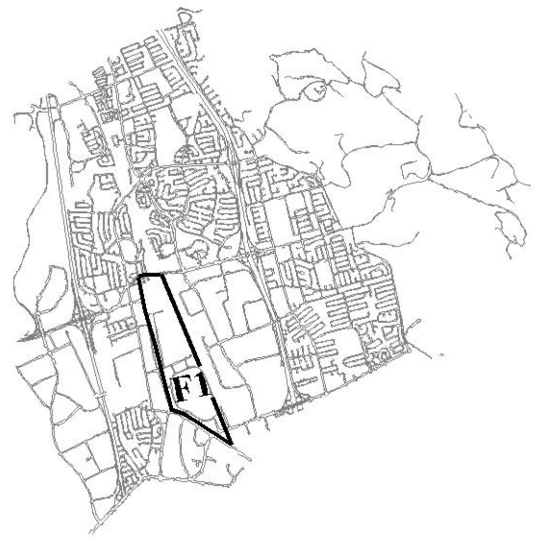

CIP for Collection System CA2
1 Minnis Pump Station Standby Power
2 North Milpitas Boulevard SD Relief
3 Minnis Circle SD Replacement
4 Minnis Pump Station Replacement
Legend - Collection System CA2
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
Streams
Manhole
Outlet
Streets
113
Link ID
Feet
0
200
400
800
N
Minnis Pump Station
CALERA CREEK
BERRYESSA CREEK
MILPITAS BOULEVARD
WASHINGTON
VALMY
COELHO
JASON
RAND ST
DUARTE CT
GARCIA CT
STREET
DRIVE
ESCUELA PARKWAY
MANZANO ST
CIROLERO ST
COLUMBUS DRIVE
NIEVES CT
NIEVES ST
RUSSELL LANE
KIZER ST
RIVERA ST
HEFLIN ST
HORCAJO ST
HORCAJO CIR
TICE DRIVE
INTERSTATE
HIGHWAY
CARSON WAY
VARGAS CT
ROSE DR
FONTAINBLEU DRIVE
FONTAINBLEU CT
VIENNA
BELBROOK WY
AVENUE
BEAUMERE WAY
KNOLL VIEW DR
ARIZONA AVENUE
MIDWICK DRIVE
BERRENDO DR
SUDBURY DR
KOVANDA WY
MINNIS CIRCLE
SUMMERWIND WY
U.P.R.R.
OREGON WAY
SANDALWOOD LANE
ALTAMONT DR
ESCUELA PKWAY
GRAYSON WY
KEVINAIRE DRIVE
SANDALRIDGE CT
JACKLIN ROAD
GEMMA DR
CORINTHIA DR
CLAUSER
PARVIN
DE ANZA CT
JACKLIN CIRCLE
NORTH MILPITAS
HANSON CT
COVENTRY
HAMILTON CIRCLE
MERZ CT
ENRIQUEZ CT
DR
PESC

Figure 5-8

Ford Creek (F1)



This is a heavily industrialized area slowly being converted to mixed land uses, located between the Union Pacific and Southern Pacific Railroads, from Montague Expressway to State Highway 237. The local collection system is made up of storm drains, laterals, and Ford Creek itself, which joins Wrigley Creek north of 237 to form Wrigley-Ford Creek. Ford Creek, Wrigley Creek, Wrigley-Ford Creek, and the Wrigley-Ford Pump Station are all maintained by the City of Milpitas.

Ford Creek contains the estimated one-percent discharge within its banks from Calaveras Boulevard to the confluence with Wrigley-Ford Creek. A sediment and vegetation removal project has restored flow capacity to Ford Creek, although a regulatory floodplain, which results from spills both inside and outside of Milpitas, is present.

Ford Creek Discharge

Ford Creek collects local storm water runoff and discharges it to Wrigley-Ford Creek and eventually the Wrigley-Ford Pump Station and Berryessa Creek. Table 5-23 lists the 10- and 100-year design discharges in the Ford Creek and Wrigley-Ford Creek system.

Table 5-23
Storm Water Discharge in Ford Creek, Wrigley Creek, and Wrigley-Ford Creek

Creek	Location	Tributary Area (acres)	10-year Discharge (cfs)	100-year Discharge (cfs)
Ford Creek	Bothello Avenue	255	110	155
	Calaveras Boulevard	298	130	175
Wrigley Creek	Montague Expressway	50	30	50
	Piper Drive Outfall	85	55	80
	Gibralter Drive Outfall	169	100	150
	Yosemite Drive Outfall	220	130	200
	Los Coches Street Outfall	339	140	230
	Calaveras Boulevard	422	170	280
Wrigley Ford Creek	At Confluence	760	290	400

Gravity Outfalls at Ford Creek

Ford Creek carries a 100-year discharge of about 175 cfs at Highway 237 (Calaveras Boulevard). Figure 5-9 shows the 100-year water surface profile in Wrigley-Ford Creek from the Wrigley Creek and Ford Creek confluence to the Wrigley-Ford Pump Station and in Ford Creek from its confluence with Wrigley Creek to Sinnott Lane.

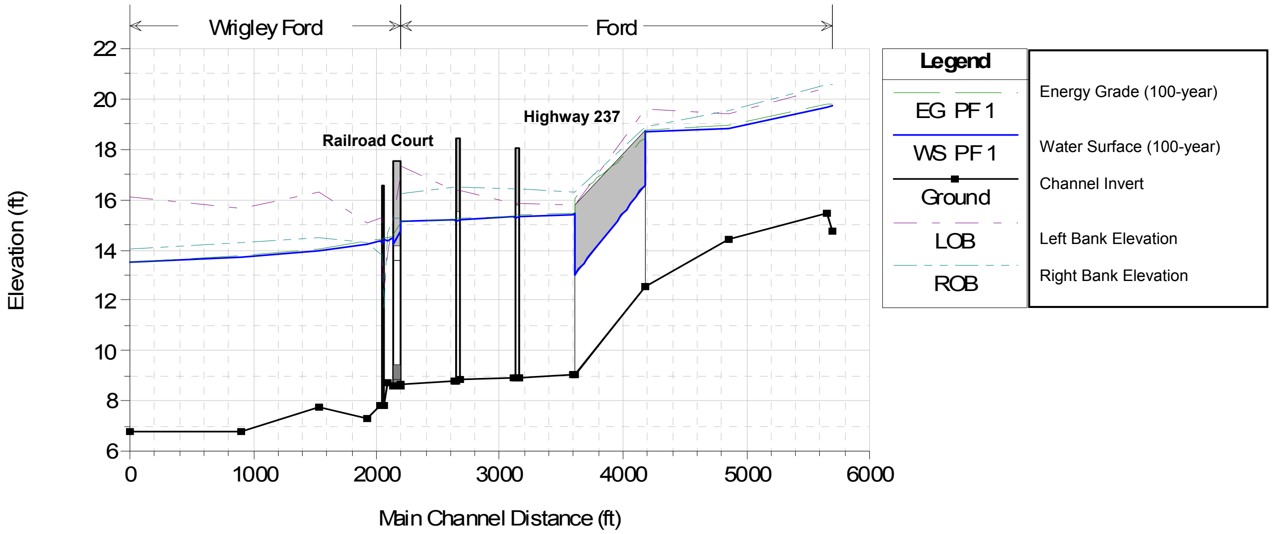


Figure 5-9: Water Surface Profile for Ford Creek and Wrigley-Ford Creek

This profile reflects the following assumptions:

1. Accumulated sediment is routinely removed from Ford Creek and Wrigley-Ford Creek so that it does not significantly block the concrete culvert inverts at the unlabeled driveway crossings shown in Figure 5-9.
2. Woody channel vegetation that grows from bank toe to bank toe is routinely removed. Such vegetation retards the flow of water during both flood and non-flood events and may promote additional sediment deposition. Emergent wetland vegetation is flexible and will bend in the direction of flow during large runoff events. However, it is important the woody vegetation does not become established within the channel, since it is not flexible.
3. Mitigation vegetation planted on the channel banks is maintained to help prevent bare channel banks along Ford Creek. Bank erosion may be a significant source of sediment and reducing channel erosion could help reduce channel sedimentation and increase the interval between periods of channel dredging required to maintain an open culvert at Highway 237.

Maintaining flow capacity through the Highway 237 culvert minimizes upstream water surface elevations, which also affect local drainage. With effective culvert and channel maintenance, the culvert capacity is roughly 160 cfs with about 15 cfs spilling to Railroad Avenue, which flows generally within the street right-of-way under Highway 237 and re-enters the creek. Adding culvert capacity to this crossing by installing an additional culvert is deemed to be cost prohibitive relative to the small benefit provided.

Table 5-24 lists the 10- and 100-year starting tailwater elevations at each gravity outfall to Ford Creek and Wrigley-Ford Creek, using the criteria outlined in Chapter 3. The starting backwater for the tributary system is equivalent to the water surface elevation in the receiving water plus the exit loss at the storm drain pipe outfall. Schaaf & Wheeler has completed field surveys of Ford Creek from Bothello Avenue to the confluence of Wrigley-Ford Creek and prepared a hydraulic model for the creek under the design one-percent (100-year) discharge. This model has been used to evaluate flow capacity in Ford Creek and determine tailwater elevations assuming that the creek conditions described previously are maintained.

Table 5-24
Tailwater Elevations for Storm Drain Outfalls within F1 System

ID	Outfall Location	Ground Elev.	SD INV	Outfall Dia	Velocity (fps)		Creek WSEL (feet NAVD)		System Tailwater (feet NAVD)	
					10yr	100yr	10yr	100yr	10yr	100yr
2	Ford Creek at Bothello Avenue	20.5	14.74	24"	2.15	3.19	19.14	19.96	19.21	20.12
50	Ford Creek at Railroad Avenue	16.6	8.81	36"	0.18	0.29	13.76	14.65	13.76	14.65
1057	Wrigley-Ford Ck at Marylinn Dr	13.9	9.16	39"	0.32	1.31	13.36	14.04	13.36	13.87

Collection System Performance

Table 5-25 presents the analytical performance statistics for the F1 system, which includes the completion of storm drain improvements for the Milpitas Library project on Weller Lane and Marylinn Drive. Further local storm drain system improvements are not needed (Figure 5-10).

Table 5-25
Collection System F1 Performance

	Lineal Feet	Percentage
System Acceptable / No Improvements	7,412	100
High Priority Improvements	0	0
Medium Priority Improvements	0	0
Low Priority Improvements	0	0
Total System	**7,412**	**100**

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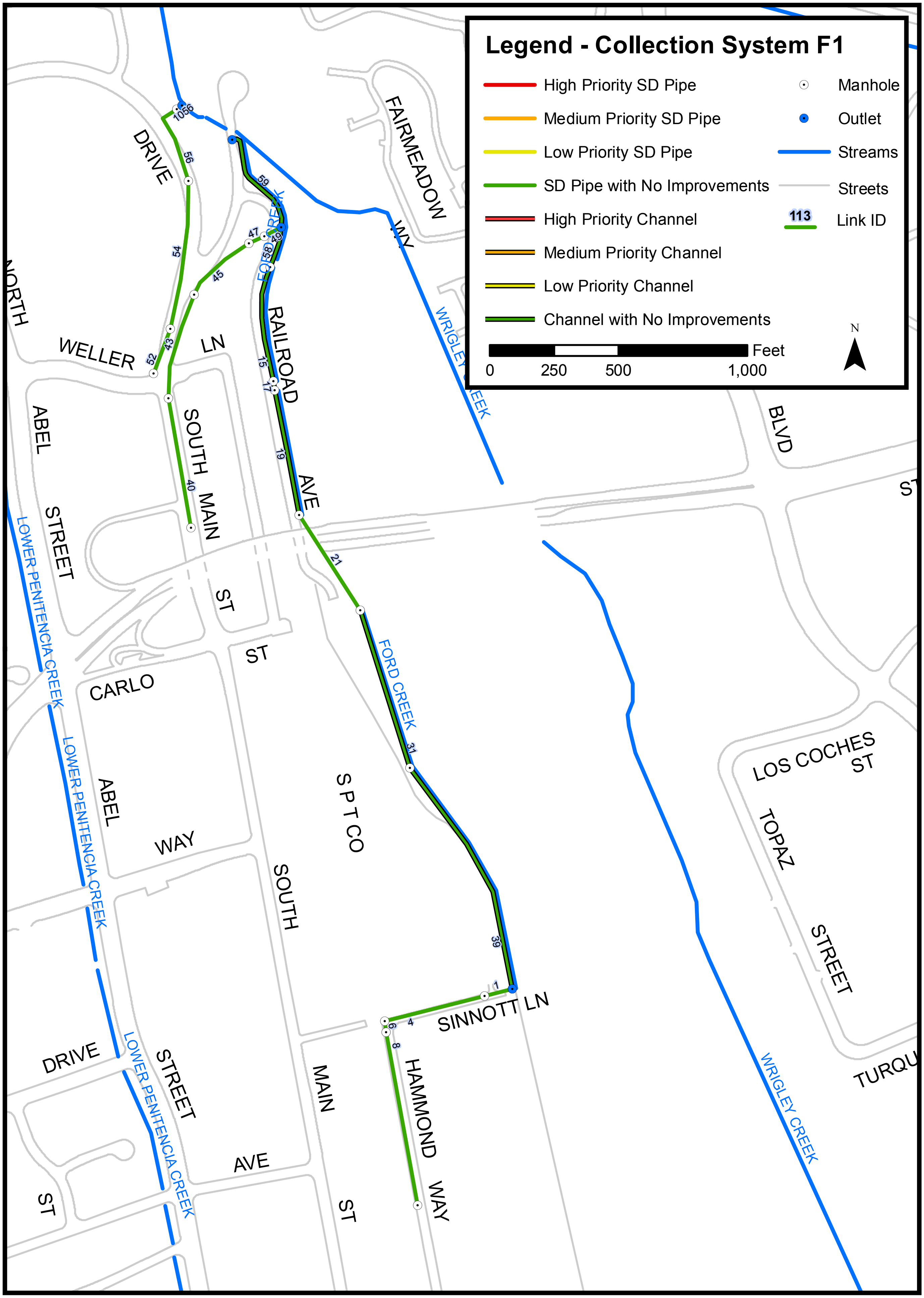
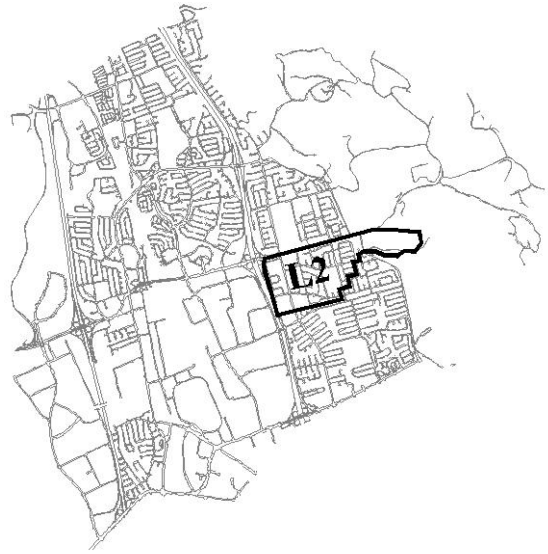

Legend - Collection System F1
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
High Priority Channel
Medium Priority Channel
Low Priority Channel
Channel with No Improvements
Manhole
Outlet
Streams
Streets
113
Link ID
Feet
0
250
500
1,000
N
FAIRMEADOW WY
DRIVE
NORTH
WELLER LN
RAILROAD AVE
FORD CREEK
WRIGLEY CREEK
ABEL STREET
SOUTH MAIN ST
CARLO ST
LOWER PENITENCIA CREEK
ABEL WAY
S P T CO
SOUTH MAIN ST
SINNOTT LN
HAMMOND WAY
DRIVE
STREET
AVE
ST
LOS COCHES ST
TOPAZ STREET
TURQU
BLVD
ST

Figure 5-10

Los Coches Creek East of 680 (L2)



This system defines an area located between Piedmont Creek to the south and Los Coches Creek to the north, from Piedmont Road on the east to Interstate 680 on the west. Storm drains within this system drain directly into Los Coches Creek, which is a Santa Clara Valley Water District facility (See Table 4-1).

Los Coches Creek is problematic between Piedmont Road and the interstate (Figure 4-1), and flooding due to creek overflows in severe events is a significant concern that must be addressed by the District. Past problems with rather frequent flooding at the Falcato Drive cul-de-sac area, and the streets adjacent to La Cross Drive near Los Coches Creek, have already been corrected by a storm drain improvement project on Piedmont Road.

Gravity Outfalls at Los Coches Creek

All storm drains in the system discharge to Los Coches Creek through gravity outfalls. Table 5-26 lists the 10- and 100-year starting tailwater elevations at each gravity outfall, using the criteria outlined in Chapter 3. The starting backwater for the tributary system is equivalent to the water surface elevation in the receiving water plus the exit loss at the storm drain pipe outfall. In some instances, the 10-year creek water surface elevation is greater than the 100-year creek water surface elevation due to upstream spills.

Table 5-26
Tailwater Elevations at Storm Drain Outfalls within L2 System

ID	Outfall Location on Los Coches Creek	Ground Elev.	SD INV	Outfall Dia	Velocity (fps)		Creek WSEL (feet NAVD)		System Tailwater (feet NAVD)	
					10yr	100yr	10yr	100yr	10yr	100yr
7	110' D/S S Park Victoria Drive	55.0	49.53	27”	5.40	7.87	52.38	51.78	52.83	52.74
9	D/S Face S Park Victoria Drive	57.0	48.03	30”	3.20	4.75	57.09	56.03	57.00	56.38
17	D/S Face Dempsey Road	48.0	36.22	21”	13.00	19.10	40.74	43.19	43.36	48.00
30	180' U/S S Park Victoria Drive	58.2	51.33	18”	2.79	4.26	59.56	59.08	58.20	58.20
44	310' U/S S Park Victoria Drive	60.0	51.98	42”	7.17	11.01	60.43	60.00	60.00	60.00
70	D/S Face Piedmont Road	128.0	121.33	21”	1.14	1.73	122.42	124.00	123.10	124.05
72	U/S Face Piedmont Road	129.0	118.78	30”	6.39	9.19	123.53	124.78	124.16	126.09
104	U/S Face Dempsey Road	48.0	40.08	18”	6.60	9.50	48.75	50.80	48.00	48.00
1061	260' D/S Temple Drive	98.0	92.20	21”	5.56	8.28	97.44	98.03	97.92	98.00

Collection System Performance

Table 5-27 presents CIP statistics for the L2 system. Performance issues arise when the water surface elevation in Los Coches Creek at a storm drain outfall is higher than the ground elevation of adjacent streets. Problem areas are concentrated at natural topographic depressions near both banks of the perched Los Coches Creek. Additional storm drains and reshuffled priorities are necessary for CIP implementation.

Table 5-27
Recommended CIP for Collection System L2

	Lineal Feet	Percentage
System Acceptable / No Improvements	8,619	52
High Priority Improvements	3,230	20
Medium Priority Improvements	3,370	20
Low Priority Improvements	1,250	8
Total System	**16,469**	**100**

Since ground elevations often run adverse to Los Coches Creek near storm drain outfalls, it is not always possible to upgrade system performance to meet storm drain performance criteria by upsizing the storm drain outfall. That is, even the 10-year water surface elevation in the creek may be higher than the ground surface a block away. It is not realistic to expect an improvement in this situation, by the SCVWD or otherwise. No amount of pipe upsizing could solve this type of problem. To avoid the construction of pumping facilities in residential and commercial areas with limited available rights-of-way for public improvements, the Master Plan proposes gravity storm drain diversions in conjunction with some outfall upsizing, as indicated in Table 5-28 and shown on Figure 5-11.

Capital Improvements

Table 5-28 identifies capital projects to correct inadequate storm drain capacity caused primarily by the perched nature of Los Coches Creek.

Table 5-28
Recommended Capital Improvements in System L2

ID	Project	Priority	Parallel Option	Replacement Option
1	Dempsey Road SD Relief	High	Use replacement option.	Install approx 1,100 LF of 36-inch RCP from the existing 27-inch storm drain that crosses Dempsey Road in an easement to Los Coches Creek in a new outfall downstream of the Dempsey Road culvert.
2	Edsel Drive SD Improvements	High	Install approx 730 LF of 36-inch RCP in Edsel Dr from South Park Victoria Drive to Dempsey Road. Install approx 1,200 LF of 42-inch RCP in Dempsey Road from Edsel Dr to the north side of Selwyn Dr. Replace approx 200 LF of existing 21-inch RCP with 48-inch RCP from Dempsey Rd/Selwyn Dr to a new outfall at Los Coches Creek.	Install approx 730 LF of 36-inch RCP in Edsel Dr from South Park Victoria Drive to Dempsey Road. Replace approx 900 LF of existing 18-inch RCP in Dempsey Road from Edsel Dr to the south side of Selwyn Dr with 48-inch RCP. Replace approx 300 LF of existing 21-inch RCP with 48-inch RCP in Dempsey Road between the Selwyn Dr intersections. Replace approx 200 LF of existing 21-inch RCP with 48-inch RCP from Dempsey Rd/Selwyn Dr to a new outfall at Los Coches Creek.

ID	Project	Priority	Parallel Option	Replacement Option
3	Carnegie Drive SD Improvements	Medium	Install approx 1,080 LF of 30-inch RCP on Carnegie Dr between Mercury Ct and Canton Dr. Install approx 160 LF of 30-inch RCP in Canton Dr. from Carnegie Dr. to the existing 42-inch outfall to Los Coches Creek west of Perry St.	Replace approx 25 LF of existing 21-inch RCP and 715 LF of existing 27-inch RCP with 36-inch RCP on Carnegie Dr between Mercury Ct and Ashland Dr. Replace approx 340 LF of existing 30-inch RCP with 42-inch RCP on Carnegie Dr between Ashland Dr and Canton Dr. Replace approx 160 LF of 36-inch RCP with 42-inch RCP in Canton Dr. from Carnegie Dr. to the existing 42-inch outfall to Los Coches Creek west of Perry St.
4	Roswell/Canton SD Improvements	Medium	Install approx 1,070 LF of 30-inch RCP in Roswell Dr. from Roswell Ct. to Canton Dr. and approx 1,060 LF of 30-inch RCP in Canton Dr from Roswell Dr to Carnegie Dr.	Replace approx 250 LF of existing 24-inch RCP in Roswell Dr with 36-inch RCP immediately north of Roswell Ct. Replace approx 820 LF existing 33-inch RCP in Roswell Dr with 42-inch RCP to Canton Dr. Replace approx 680 LF of existing 33-inch RCP and 380 LF of existing 36-inch RCP with 42-inch RCP in Canton Dr from Roswell Dr to Carnegie Dr.
5	Lawton Drive SD Relief	Low	Use replacement option.	Connect existing 21-inch storm drain at Burley Drive outfall to new Roswell/ Canton improvements at Roswell Drive with approx 1,250 LF of new 24-inch RCP on Burley Drive, Lawton Drive, and Canton Drive.

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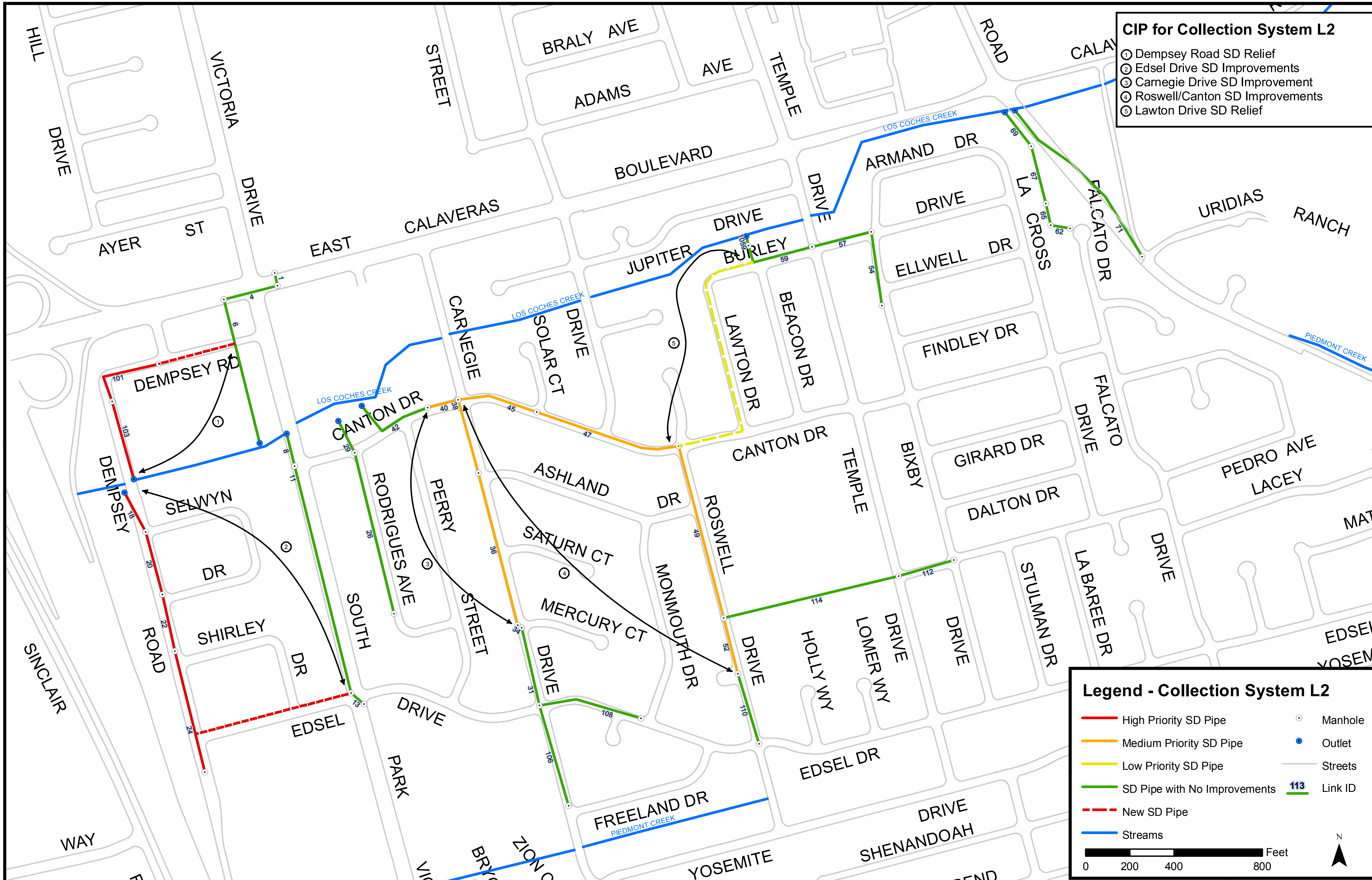
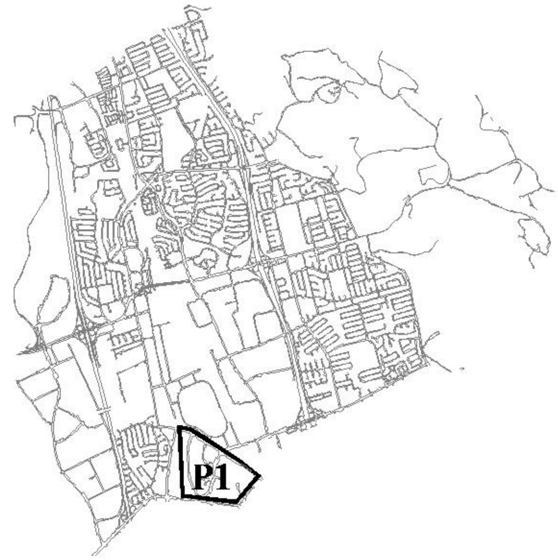

CIP for Collection System L2
1 Dempsey Road SD Relief
2 Edsel Drive SD Improvements
3 Carnegie Drive SD Improvement
4 Roswell/Canton SD Improvements
5 Lawton Drive SD Relief
Legend - Collection System L2
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
New SD Pipe
Streams
Manhole
Outlet
Streets
113 Link ID
Feet
0 200 400 800
N
LOS COCHES CREEK
PIEDMONT CREEK
DEMPSEY RD
CANTON DR
EDSEL DR
ROSWELL DRIVE
LAWTON DR
CALAVERAS BOULEVARD

Figure 5-11



Penitencia East Channel (P1)

This is a heavy industrial area at the southern boundary of Milpitas between Capitol Avenue and Montague Expressway / Trade Zone Boulevard. Local stormwater collection systems all drain either to the Penitencia East Channel or Lower Penitencia Creek, both of which are owned and maintained by the Santa Clara Valley Water District. Substantial tributary areas in San Jose also drain to these creeks at Montague Expressway (a County road) and along Lundy Place, in a siphon under the VTA right-of-way. The entire drainage basin lies within the mapped 100-year floodplain for Lower Penitencia Creek.

Gravity Outfalls

All storm drains in the system discharge to East Penitencia Creek or Lower Penitencia Creek through gravity outfalls. Table 5-29 lists the ten- and 100-year starting tailwater elevations at each gravity outfall, using the criteria outlined in Chapter 3. The starting backwater for the tributary system is equivalent to the water surface elevation in the receiving water plus the exit loss at the storm drain pipe outfall. In some instances, the ten-year creek water surface elevation is greater than the 100-year creek water surface elevation due to upstream spills.

Table 5-29
Tailwater Elevations at Storm Drain Outfalls within P1 System

ID	Outfall Location on East Penitencia Creek	Ground Elev.	SD INV	Outfall Dia	Velocity (fps)		Creek WSEL (feet NAVD)		System Tailwater (feet NAVD)	
					10yr	100yr	10yr	100yr	10yr	100yr
61	240 ft U/S McCandless Drive	35.0	27.63	42”	1.52	2.49	33.6	36.5	33.64	35.00
75	D/S face McCandless Drive	35.0	26.86	24”	1.66	2.55	32.7	36.1	32.74	35.00
209	U/S end of creek	51.0	45.00	72”	11.60	16.39	47.0	50.0	51.00	51.00
1012	575 ft U/S McCandless Drive	40.0	34.49	24”	10.45	15.14	35.8	38.0	38.19	40.00
1014	U/S face Montague Expwy	40.0	37.09	33”	2.20	3.27	40.4	41.8	40.00	40.00
1017	760 ft U/S Montague Expwy	45.0	37.38	30”	2.67	3.96	44.1	45.3	44.21	45.00
1086	575 ft U/S McCandless Drive	38.0	34.49	30”	0.94	1.51	36.3	37.0	37.00	37.04
ID	**Outfall Location on Lower Penitencia Creek**	**Ground Elev.**	**SD INV**	**Outfall Dia**	**Velocity (fps)**		**Creek WSEL (feet NAVD)**		**System Tailwater (feet NAVD)**	
					10yr	**100yr**	**10yr**	**100yr**	**10yr**	**100yr**
41	565 ft above UPRR	35.0	22.61	33”	1.95	2.85	29.8	35.8	29.86	35.00
102	D/S face Montague Expwy	38.0	30.73	54”	5.61	8.73	32.7	36.1	35.72	37.24
112	D/S face Montague Expwy	38.0	28.00	72”	14.56	20.64	32.7	36.1	37.29	38.00
1036	U/S face UPRR	34.0	28.43	24”	2.07	3.54	29.6	35.7	30.50	34.00
1080	380 ft U/S East Penitencia Ck	34.0	27.12	33”	2.60	4.15	31.2	35.9	31.33	34.00

Collection System Performance

Table 5-30 presents the recommended CIP for Collection System P1. The analysis of collection system performance is somewhat complicated by substantial areas within San José that are tributary to storm drain systems within Milpitas. Runoff from tributary San José systems is included in the contributory watershed calculations, but storm drain systems in San José are not analyzed in detail. Some capital improvements have been eliminated or reclassified as “low priority” due to questions of infrastructure ownership in Montague Expressway and improvements planned by VTA for the Silicon Valley BART Extension project.

System performance criteria are not met due to the runoff contributions from San José at the Lower Penitencia Creek outfall at Montague Expressway, and the siphon under the UPRR at Lundy Place near East Penitencia Creek. Low priority improvements result from discharge against relatively high 100-year creek levels, noting that the lowest elevations along McCandless Drive between Great Mall Parkway and East Penitencia Creek are one to two feet lower than the bank elevations of Lower Penitencia Creek and East Penitencia Creek. No amount of storm drain upsizing can overcome this adverse grade problem when flood levels in the creeks are near the bank. Furthermore, this area is subject to shallow 100-year flooding (Zone AO; see Figure 4-1) primarily due to overflows from Berryessa Creek. Therefore many nominally medium priority improvements have been reclassified as low priority, and some of these deficiencies will only be corrected by reducing the stage in Lower Penitencia Creek and East Penitencia Creek. The City will require streets like McCandless Drive be reconstructed during redevelopment to improve drainage performance.

Table 5-30
Recommended CIP for Collection System P1

	Lineal Feet	Percentage
System Acceptable / No Improvements	16,274	82
High Priority Improvements	0	0
Medium Priority Improvements	0	0
Low Priority Improvements	3,580	18
Total System	**19,854**	**100**

Capital Improvements

Table 5-31 identifies capital projects to correct inadequate storm drain capacity, which are shown on Figure 5-12. Both city storm drain block maps show storm drains in Lundy Place, Trade Zone Boulevard and Montague Expressway that outfall to Lower Penitencia Creek. Available information indicates that not all of these storm drains are owned and maintained by the City of Milpitas. It is noted that a new double 6’ x 4’ / 6’ x 5’ RCB crossing of the former UPRR and current VTA right-of-way for the Silicon Valley BART Extension Project will be completed by VTA and is not part of the CIP. It is also noted that since required improvements along Lundy Place and improvements in Montague Expressway at Lower Penitencia Creek are primarily necessitated due to tributary storm water runoff from San José, Milpitas should consider separating its own storm drainage from these systems. Since available information indicates that the storm drain system in Montague Expressway is not owned by the City of Milpitas and outside of its control, corrective action required at this location has been reclassified as “low priority”.

Table 5-31
Recommended Capital Improvements in System P1

ID	Project	Priority	Parallel Option	Replacement Option
1	Montague Expressway SD Improvements	Low	Install approx 660 LF of 18-inch RCP in Montague Expressway with new outfall to East Penitencia Creek. Install approx 610 LF of 30-inch RCP in Montague Expressway immediately north of the intersection of Trade Zone Boulevard with McCandless Drive.	Replace approx 660 LF of existing 24-inch RCP in Montague Expressway with 30-inch RCP immediately north of East Penitencia Creek and reconstruct creek outfall. Replace approx 610 LF of existing 21-inch RCP in Montague Expressway with 36-inch RCP immediately north of the intersection of Trade Zone Boulevard with McCandless Drive.
2	Montague Expressway SD Improvements at Lower Penitencia Creek (Not a Milpitas facility)	Low	Install approx 660 LF of 84-inch RCP on Montague Expressway from South Main Street to outfall at Lower Penitencia Creek.	Replace approx 660 LF of existing 60-inch RCP in Montague Expwy. with 96-inch RCP from South Main Street to the Lower Penitencia Creek outfall. Replace approx 130 LF of existing 72-inch RCP outfall to Lower Penitencia Creek with a 96-inch creek outfall.
3	Tarob Court Outfall Relocation	Low	Use replacement option.	Replace approx 620 LF of existing 30-inch RCP with 42-inch RCP from the Tarob Court cul-de-sac to the East Penitencia Creek outfall. Relocate new 42-inch outfall approximately 150 feet downstream from existing outfall location.
4	Lundy Place Relief Line	Low	Install approx 750 LF of 18-inch RCP in Lundy Place from Tarob Court to existing 72-inch East Penitencia Creek outfall.	Replace approx 750 LF of existing 18-inch RCP in Lundy Place with 30-inch RCP from Tarob Court to existing 72-inch East Penitencia Creek outfall.

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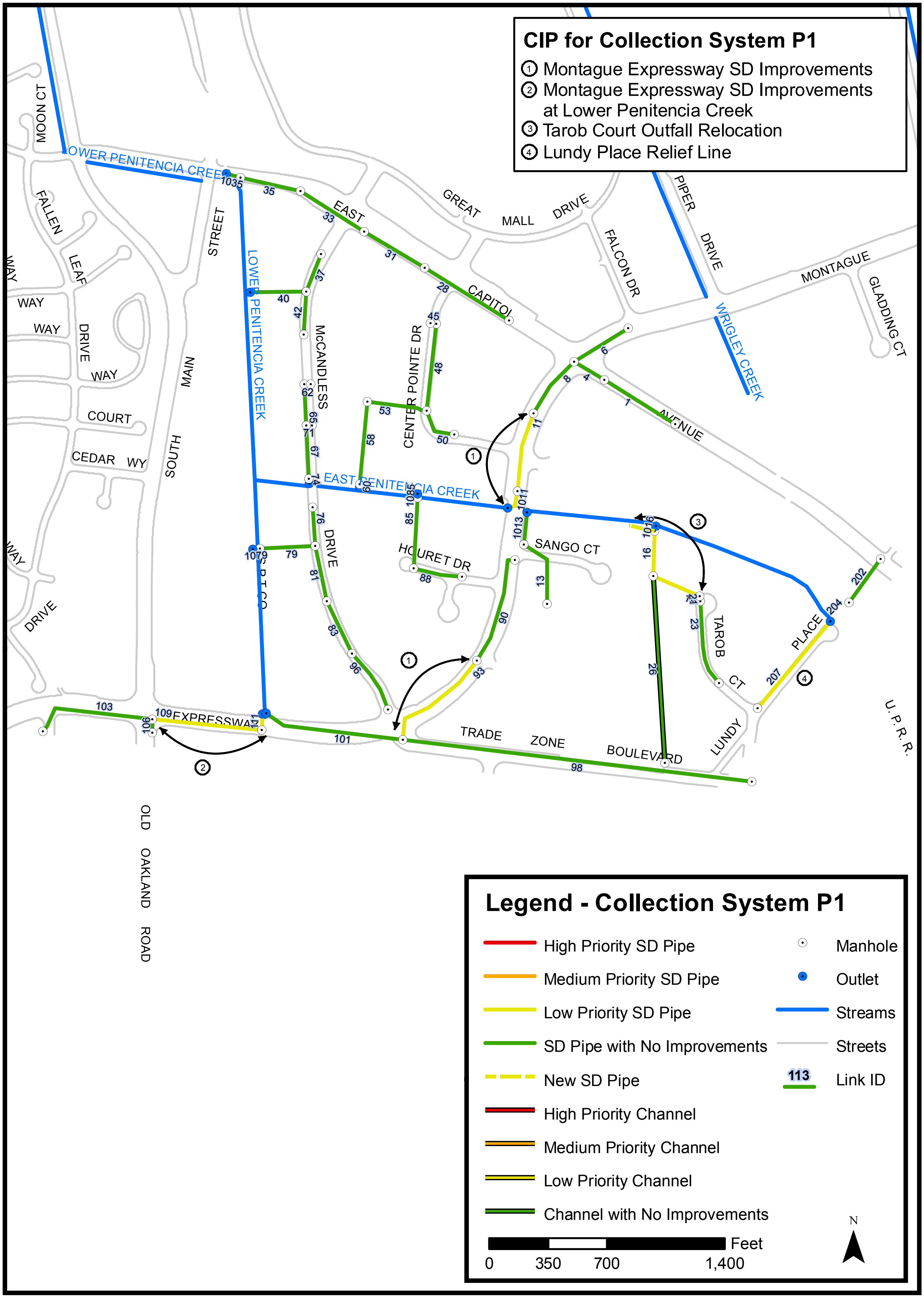
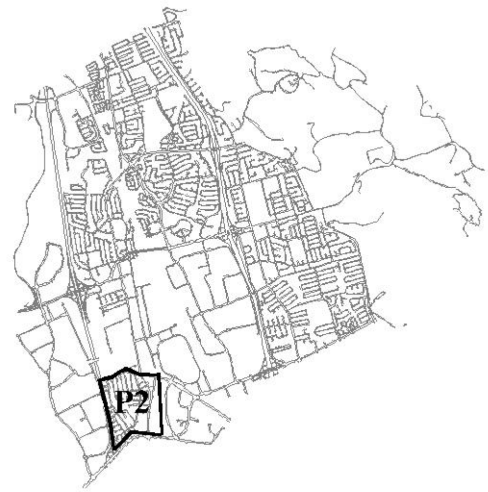

CIP for Collection System P1
1 Montague Expressway SD Improvements
2 Montague Expressway SD Improvements at Lower Penitencia Creek
3 Tarob Court Outfall Relocation
4 Lundy Place Relief Line
Legend - Collection System P1
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
New SD Pipe
High Priority Channel
Medium Priority Channel
Low Priority Channel
Channel with No Improvements
Manhole
Outlet
Streams
Streets
113
Link ID
Feet
0
350
700
1,400
N
LOWER PENITENCIA CREEK
EAST PENITENCIA CREEK
WRIGLEY CREEK
MONTAGUE EXPRESSWAY
TRADE ZONE BOULEVARD
GREAT MALL DRIVE
EAST CAPITOL AVENUE
McCANDLESS DRIVE
CENTER POINTE DR
HOURET DR
SANGO CT
TAROB CT
LUNDY PLACE
FALCON DR
PIPER DRIVE
GLADDING CT
SOUTH MAIN STREET
OLD OAKLAND ROAD
U.P.R.R.
MOON CT
FALLEN LEAF DRIVE
COURT
CEDAR WY

Figure 5-12

Penitencia Creek West (P2)



System P2 is comprised of storm drains that collect runoff from primarily single-family residences between the Nimitz Freeway (Interstate 880) and Penitencia Creek. The area is bound by Montague Expressway to the south and Great Mall Parkway on the north. Local collection systems all drain to Lower Penitencia Creek outfalls in the vicinity of West Capitol Avenue and South Main Street. Some of the area is within Penitencia Creek's 100-year floodplain; the remainder is mapped as Shaded Zone X.

Gravity Outfalls at Lower Penitencia Creek

All storm drains in the system discharge to Lower Penitencia Creek through gravity outfalls. Table 5-32 lists the 10- and 100-year starting tailwater elevations at each gravity outfall, using the criteria outlined in Chapter 3. The starting backwater for the tributary system is equivalent to the water surface elevation in the receiving water plus the exit loss at the storm drain pipe outfall. In some instances, the 10-year creek water surface elevation is greater than the 100-year creek water surface elevation due to upstream spills.

Table 5-32
Tailwater Elevations at Storm Drain Outfalls within P2 System

ID	Outfall Location on Lower Penitencia Creek	Ground Elev.	SD INV	Outfall Dia	Velocity (fps)		Creek WSEL (feet NAVD)		System Tailwater (feet NAVD)	
					10yr	100yr	10yr	100yr	10yr	100yr
89	D/S face Capitol Avenue	29.0	18.82	66"	0.72	1.19	24.8	27.6	24.81	27.62
136	D/S Great Mall Parkway	27.0	14.72	42"	0.75	1.08	22.3	24.4	22.31	24.42
1026	D/S face South Main Street	32.0	23.72	36"	2.49	3.63	27.8	30.7	27.90	30.90
1087	580 ft D/S Capitol Avenue	28.0	18.07	36"	2.26	3.54	23.8	26.8	23.88	26.97

Collection System Performance

The storm water collection systems within this basin perform well in the 10-year event but some areas do not meet the 100-year performance criterion. Because this part of Milpitas is relatively flat, street profiles are often saw-toothed to promote drainage toward individual inlets. As a consequence, there is some residual 100-year ponding that could threaten property, as excess runoff must reach certain depths before it can release to a storm drain system with sufficient capacity. In a neighborhood with many street sags, keeping inlet grades clear is more important, since the streets cannot carry as much water as might be expected in areas with steeper gradients. Table 5-33 presents the recommended CIP for the P2 system. Note that wherever medium priority improvements are located within the mapped one-percent special flood hazard zone, those medium priority improvements are reclassified as low priority improvements for the CIP.

Table 5-33
Recommended CIP for Collection System P2

	Lineal Feet	Percentage
System Acceptable / No Improvements	13,858	70
High Priority Improvements	0	0
Medium Priority Improvements	1,100	6
Low Priority Improvements	4,700	24
Total System	**19,658**	**100**

Capital Improvements

Table 5-34 identifies capital projects to mitigate areas of residual flooding under 100-year conditions caused by flat street grades. Improvements necessary to alleviate this flooding are described below and shown on Figure 5-13. As downstream drains are upsized, upstream conditions will continue to improve. All listed projects correct nominally medium priority problems, but the West Capitol Avenue Relief Line corrects local 100-year flooding in an area also subject to shallow 100-year flooding from other sources, and is therefore listed as a low priority project. While the Woodland Way Storm Drain Improvements also correct flooding in an area subject to 100-year flooding from other sources, the listed improvements between Sunrise Way and Fallen Leaf Drive are necessary to remedy upstream problems between Greenwood Way and Lonetree Court, and are therefore still considered to be medium priority improvements.

Table 5-34
Recommended Capital Improvements in System P2

ID	Project	Priority	Parallel Option	Replacement Option
1	South Main Street SD Improvements at Cedar Way	Medium	Install approx 660 LF of 24-inch RCP in South Main Street immediately south of the intersection with Cedar Way.	Replace approx 660 LF of existing 24-inch RCP in South Main Street immediately south of the intersection with Cedar Way with 36-inch RCP and extend approx 440 LF to the north on South Main Street, tying in to the existing 36-inch RCP.
2	Woodland Way SD Improvements	Low	Install 750 LF of 18-inch RCP in Starlite Dr from Gibbons Ct to the Woodland Wy SD esmt. Replace 300 LF of exist 21-inch RCP with 24-inch RCP between Starlite Dr and Moonlight Wy. Install 160 LF of 24-inch RCP in Stardust Wy between Moonlight Wy and Moonlight Cr. Replace 360 LF of exist 27-inch RCP in SD esmt between Stardust Way and Sunrise Way with 36-inch RCP. Install 890 LF of 24-inch RCP in Woodland Way between Sunrise Wy and Fallen Leaf Dr.	Replace 750 LF of 15-in RCP in Starlite Dr from Gibbons Ct to Woodland Wy esmt with 24-in. Replace 300 LF of 21-in RCP with 24-in between Starlite Dr and Moonlight Wy. Replace 160 LF of 24-in RCP in Stardust Wy between Moonlight Wy and Moonlight Cr with 36-in. Replace 360 LF of 27-in RCP in SD esmt between Stardust Way and Sunrise Way with 36-in. Replace 520 LF of 36-in RCP with 42-in and 370 LF of 42-in with 48-in RCP in Woodland Way between Sunrise Wy and Fallen Leaf Dr.

ID	Project	Priority	Parallel Option	Replacement Option
3	West Capitol Avenue Relief Lines	Low	Install approx 1,260 LF of 30-inch RCP in West Capitol Ave from Starlite Drive to Evening Star Ct and 700 LF of 30-inch RCP in Evening Star Ct from West Capitol Ave to a new 48-inch outfall at Lower Penitencia Creek. Install approx 280 LF of 18-inch RCP in West Capitol Ave from Moonbeam Way to Fallen Leaf Drive.	Along W Capitol Ave, replace 270 LF of 18-in RCP and 170 LF of 21-in RCP with 36-in RCP; 490 LF of 33-in RCP with 42-in RCP; and 330 LF of 36-in RCP with 48-in RCP. Replace 700 LF of 36-in RCP with 48-in RCP on Evening Star Ct, and install a 48-in outfall to Lower Penitencia Creek. Replace 280 LF of 15-in RCP in West Capitol from Moonbeam Way to Fallen Leaf Drive with 24-in RCP.

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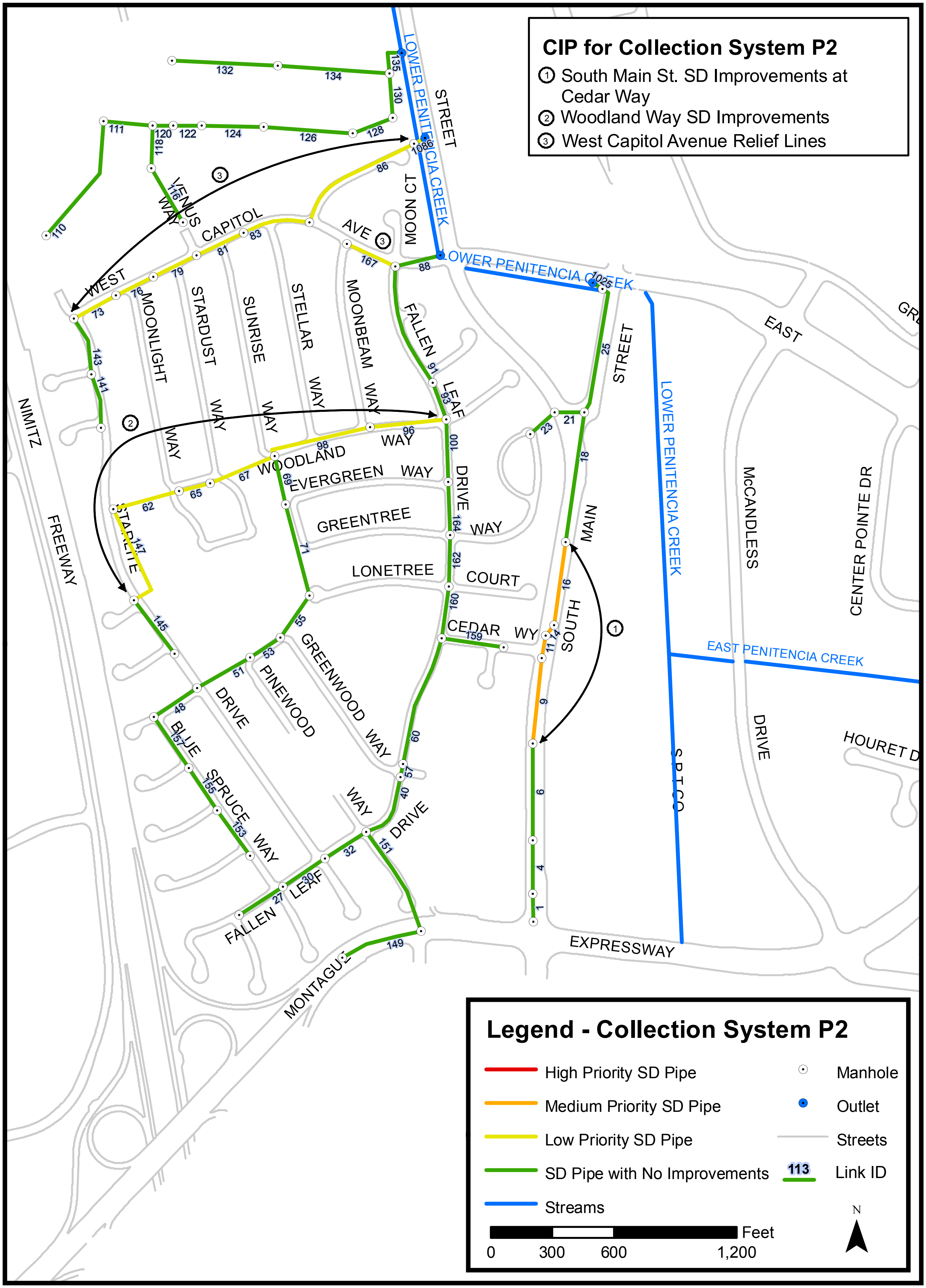
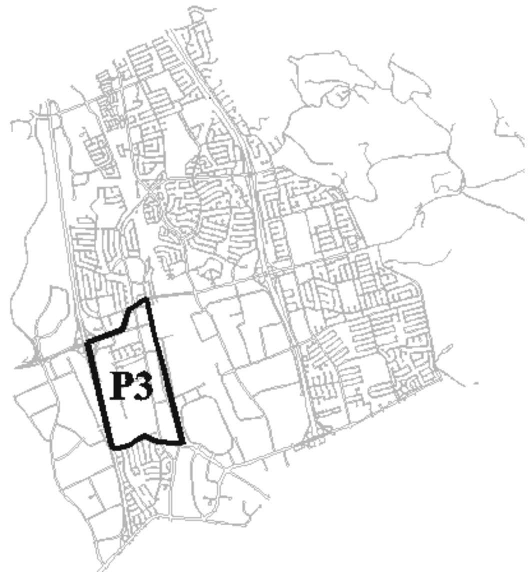

CIP for Collection System P2
1 South Main St. SD Improvements at Cedar Way
2 Woodland Way SD Improvements
3 West Capitol Avenue Relief Lines
Legend - Collection System P2
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
Streams
Manhole
Outlet
Streets
113 Link ID
Feet
0 300 600 1,200
N
WEST CAPITOL AVE
MOONLIGHT WAY
STARDUST WAY
SUNRISE WAY
STELLAR WAY
MOONBEAM WAY
MOON CT
FALLEN LEAF DRIVE
WOODLAND WAY
EVERGREEN WAY
GREENTREE WAY
LONETREE COURT
CEDAR WY
GREENWOOD WAY
PINEWOOD
BLUE SPRUCE WAY
STARLITE
VENUS WAY
SOUTH MAIN STREET
LOWER PENITENCIA CREEK
EAST PENITENCIA CREEK
McCANDLESS DRIVE
CENTER POINTE DR
HOURET D
EAST
S.P.T.CO
NIMITZ FREEWAY
MONTAGUE EXPRESSWAY

Figure 5-13

Penitencia Creek at Calaveras Boulevard (P3)



Just north of System P2, this predominately commercial area also includes recently constructed automobile dealerships and residential housing, the now vacant Divot City golf center, and Elmwood Correctional Facility. Single-family residential land use is also found within the northern part of the drainage basin. The basin is bound by Great Mall Parkway on the south and Spence Avenue to the north, the Nimitz Freeway to the east and South Main Street on the west. Elmwood Correction Facility's storm drainage system is owned and operated by Santa Clara County and drains directly to Lower Penitencia Creek; it does not connect to any City system, and is not included herein.

Local collection systems drain either directly to Lower Penitencia Creek, which parallels the west side of Abel Street, or to Spence Creek which outfalls through a 38" x 60" arch culvert to Penitencia Creek at Calaveras Boulevard. Floodwalls along the creek can cause water surface elevations within the creek to be higher than adjacent ground elevations, so when creek levels are high, a flapgate on the outfall closes to protect interior areas. When this occurs, the Spence Creek Pump Station (Chapter 6) lifts stormwater through a 42-inch outfall into Penitencia Creek at Calaveras Boulevard. Check valves on each pump also isolate the interior drainage systems from Penitencia Creek backwater. Most of this system lies within Penitencia Creek's 100-year floodplain, caused by creek overflows that become trapped against Interstate 880.

Gravity Outfalls at Lower Penitencia Creek

Storm drains in the system all discharge to Lower Penitencia Creek through gravity outfalls. Table 5-35 lists the 10- and 100-year starting tailwater elevations at each, using the criteria outlined in Chapter 3. Starting backwater for the tributary system is equivalent to the water surface elevation in the receiving water plus the exit loss at the storm drain pipe outfall. In some instances, the 10-year creek water surface elevation is greater than the 100-year creek water surface elevation due to upstream spills.

Table 5-35
Tailwater Elevations at Storm Drain Outfalls within P3 System

ID	Outfall Location on Lower Penitencia Creek	Ground Elev.	SD INV	Outfall Dia	Velocity (fps)		Creek WSEL (feet NAVD)		System Tailwater (feet NAVD)	
					10yr	100yr	10yr	100yr	10yr	100yr
81	D/S Face Corning Avenue	20.6	16.54	18"	9.10	13.33	17.0	20.0	19.33	20.60
98	290 ft US Calaveras Blvd	16.8	10.65	18"	3.52	5.07	16.2	18.1	16.34	16.75
100	825 ft US Sylvia Avenue	23.7	13.62	24"	4.17	6.02	18.9	21.8	19.12	22.31
144	D/S Face Great Mall Pkwy	28.0	16.53	18"	3.05	4.45	24.8	27.5	24.92	27.84
1008	Calaveras Boulevard	18.1	8.88	24"	2.69	4.07	16.0	17.3	16.09	17.54
1067	75 ft D/S Corning Avenue	20.0	12.62	27"	3.64	5.63	17.0	19.9	17.35	20.00
1090	Serra Way	18.5	13.23	30"	2.97	2.98	16.3	18.4	16.43	18.49
1092	Serra Way	18.6	12.44	30"	1.09	1.70	16.3	18.4	16.32	18.39
1107	Junipero Drive	20.0	11.12	27"	3.17	4.95	16.3	19.1	16.44	19.46
1118	Opposite Curtis Avenue	24.2	18.42	30"	3.62	5.37	21.6	23.5	21.76	23.95

Outfall to Pump Station

Table 5-36 lists pump station operating parameters and their effect on backwater conditions for the storm drain analyses. The starting backwater for the tributary system is equivalent to the water surface elevation in the pump wet well plus the exit loss at the influent storm drain pipe.

Table 5-36
Hydraulics at Spence Creek Pump Station Outfall

Hydraulic Parameter	10-year	100-year
WSEL in Lower Penitencia Creek (feet NAVD)	16.0	17.3
Design Inflow (cfs)	60	90
Number of Pumps Operating	2	3
Pump Station Wet Well Level (feet NAVD)	13.7	14.0
Channel Inflow Velocity (fps)	4.29	6.31
Exit Loss (foot)	0.29	0.62
Storm Drain Tailwater (feet NAVD)	13.99	14.62

Collection System Performance

Storm water drainage within this system meets 10-year performance criteria. Direct discharge against high Lower Penitencia Creek stage during a design 100-year event is the most predominant problem (Table 5-37).

Table 5-37
Recommended CIP for Collection System P3

	Lineal Feet	Percentage
System Acceptable / No Improvements	11,640	73
High Priority Improvements	0	0
Medium Priority Improvements	780	5
Low Priority Improvements	3,475	22
Total System	**15,895**	**100**

Capital Improvements

Table 5-38 identifies capital projects to mitigate areas of residual ponding under 100-year conditions caused by high stage in Lower Penitencia Creek. Many of these proposed improvements are within a special flood hazard area, and its elimination is outside City control, so these are listed as low priority. As downstream drains are upsized, upstream conditions continue to improve. A local catch basin within an isolated low topographic point in Abel Street between Serra Way and Carlo Street does not meet the performance criterion for 10-year hydraulic grade line, but since the 10-year water surface elevation at the Lower Penitencia Creek outfall is above the top of curb elevation, no amount of pipe upsizing can correct this deficiency. Furthermore the nearest location on Lower Penitencia Creek with a sufficiently low 10-year water surface elevation is at Redwood Avenue, roughly 4,700 feet downstream. Building a City pump station to drain this small area (primarily commercial parking) is not cost effective and improvements are not proposed. Figure 5-14 shows the prioritized CIP.

Table 5-38
Recommended Capital Improvements in System P3

ID	Project	Priority	Parallel Option	Replacement Option
1	Spence Creek Pump Station Standby Power	High	Add automatic standby power. (See also Page 6-27)	
2	Carlo Street Relief Drain	Medium	Install approx 780 LF of 24-inch RCP in Carlo Street from South Main Street to Lower Penitencia Creek, including a new 24-inch diameter outfall.	Replace approx 780 LF of existing 24-inch RCP in Carlo Street from South Main Street to Lower Penitencia Creek with 36-inch RCP, including a new 36-inch diameter outfall.
3	Abbott Avenue Relief Drain	Low	Install approx 840 LF of 18-inch RCP in Abbott Ave from the point adjacent to the I-880 offramp to the Palmer St. SD.	In Abbott Avenue, replace approx 400 LF of existing 24-inch RCP and approx 440 LF of existing 27-inch RCP with 30-inch RCP at the location indicated.
4	Junipero Drive Relief Drain	Low	Install approx 890 LF of 24-inch RCP in Junipero Drive from Rio Verde Pl to Ethyl St and approx 450 LF of 48-inch RCP from Ethyl St to Lower Penitenia Creek. Construct a new 48-inch outfall.	On Junipero Drive, replace approx 290 LF of existing 18-inch RCP and 600 LF of existing 24-inch RCP with 36-inch RCP. Replace approx 450 LF of existing 24- and 27-inch RCP with 54-inch RCP. Construct a new 54-inch RCP outfall to Lower Penitencia Creek.
5	Corning Avenue SD Improvements	Low	Install approx 580 LF of 18-inch RCP in Corning Ave from the existing 15-inch SD easement to the existing 24-inch SD easement and approx 180 LF of 42-inch RCP from the (E) 24-inch SD easement to Ethyl St. Install approx 535 LF of 42-inch RCP in Ethyl St from Corning Ave to Junipero Dr.	Along Corning Ave, replace approx 580 LF of existing 18-inch RCP with 24-inch RCP and 180 LF of existing 27-inch RCP with 48-inch RCP from the (E) 15-inch SD easement to Ethyl St. Replace approx 125 LF of existing 27-inch RCP with 48-inch RCP in Ethyl St from Corning Ave to the (E) 27-inch SD easement between Ethyl St and Lower Penitencia Creek. Install approx 410 LF of 48-inch RCP in Ethyl St from (E) 27-inch SD easement to Junipero Dr.

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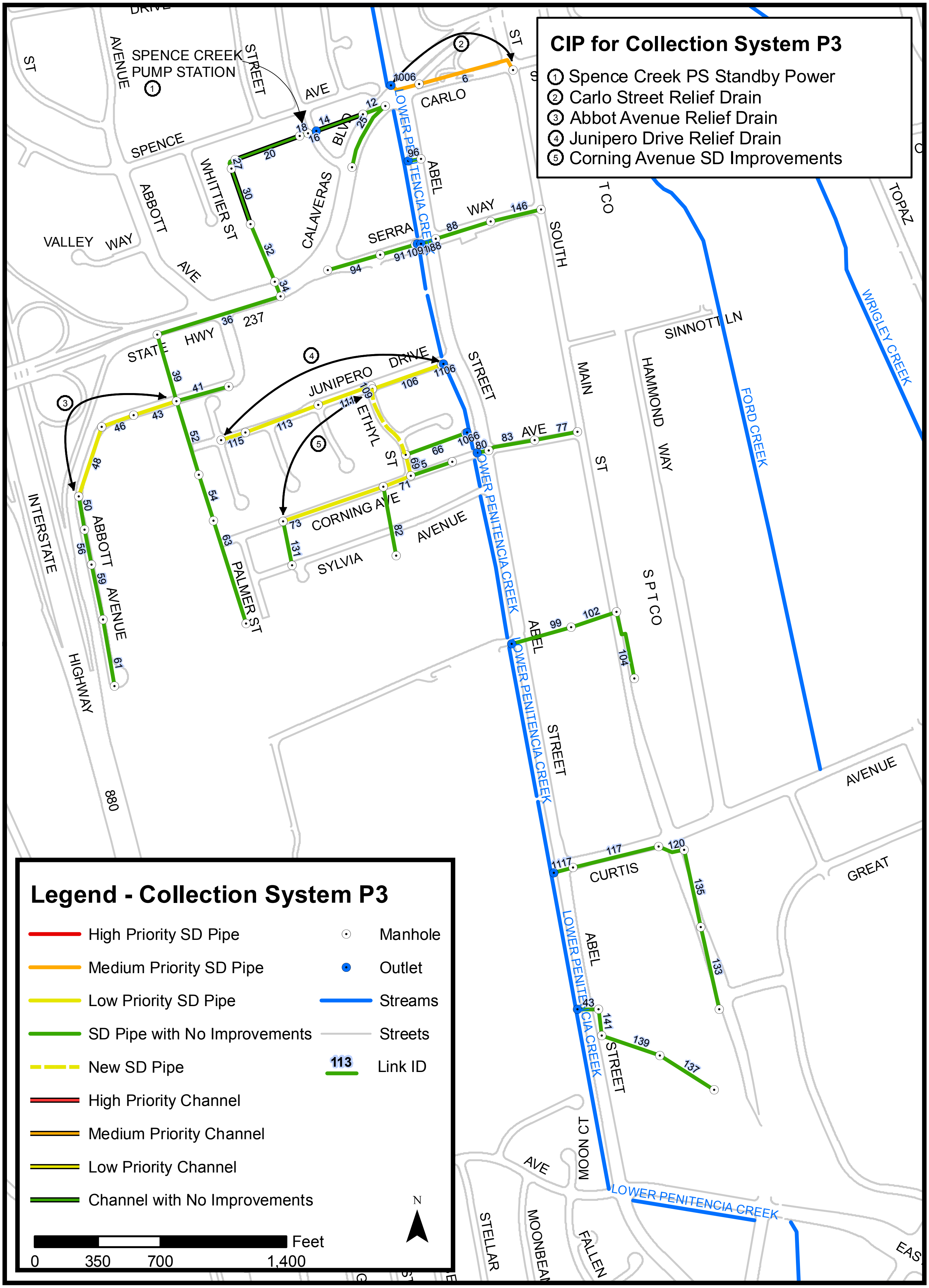


CIP for Collection System P3
1 Spence Creek PS Standby Power
2 Carlo Street Relief Drain
3 Abbot Avenue Relief Drain
4 Junipero Drive Relief Drain
5 Corning Avenue SD Improvements
Legend - Collection System P3
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
New SD Pipe
High Priority Channel
Medium Priority Channel
Low Priority Channel
Channel with No Improvements
Manhole
Outlet
Streams
Streets
113
Link ID
N
Feet
0
350
700
1,400
SPENCE CREEK PUMP STATION
SPENCE
CARLO
SERRA WAY
STATE HWY 237
JUNIPERO DRIVE
CORNING AVE
SYLVIA AVENUE
ABBOTT AVENUE
PALMER ST
ETHYL ST
CALAVERAS BLVD
WHITTIER ST
ABEL STREET
SOUTH MAIN ST
SINNOTT LN
HAMMOND WAY
S P T CO
CURTIS AVE
GREAT AVENUE
INTERSTATE HIGHWAY 880
LOWER PENITENCIA CREEK
FORD CREEK
WRIGLEY CREEK
TOPAZ
VALLEY WAY
MOON CT
MOONBEAM
STELLAR
FALLEN

Figure 5-14

Penitencia Creek at Manor Pump Station (P4)

North of System P3, the Manor Pump Station drains a single-family residential area to Lower Penitencia Creek. System P4 is located between Highway 237 and Marylinn Drive, and between the Nimitz Freeway and Lower Penitencia Creek. All local collection systems eventually discharge to the pump station.

The entire system lies within Lower Penitencia Creek's 100-year floodplain, caused by creek overflows that become trapped against Interstate 880 (Figure 4-1).

Outfall to Pump Station

A gravity bypass is located at the pump station, allowing storm runoff to drain directly to the creek, when water surface elevations permit. This bypass is closed in the 10-year and 100-year events due to creek water surface elevations that are higher than water in the pump station wet well. Table 5-39 lists pump station operating parameters and their effect on backwater conditions for the storm drain analyses. The starting backwater for the tributary system is equivalent to the water surface elevation in the pump wet well plus the exit loss at the influent storm drain pipe.

Table 5-39
Hydraulics at Manor Pump Station Outfall

Hydraulic Parameter	10-year	100-year
WSEL in Lower Penitencia Creek (feet NAVD)	15.2	16.0
First Pump Start Level (feet NAVD)	5.7	5.7
Design Inflow (cfs)	50	88
Number of Pumps Operating	2	3
Pump Station Wet Well Level (feet NAVD)	6.20	6.70
54-inch Inflow Pipe Velocity (fps)	3.15	5.52
Pipe Exit Loss (foot)	0.15	0.47
Storm Drain Tailwater (feet NAVD)	6.35	7.17

Collection System Performance

This storm water collection system generally performs well when measured against the evaluation criteria. Analysis indicates two isolated systems are in need of capital improvement; one at high priority, the other at low priority. Table 5-40 summarizes the recommended CIP for System P4.

Table 5-40
Recommended CIP for Collection System P4

	Lineal Feet	Percentage
System Acceptable / No Improvements	12,965	94
High Priority Improvements	140	1
Medium Priority Improvements	0	0
Low Priority Improvements	715	5
Total System	**13,820**	**100**

Capital Projects

Table 5-41 identifies capital projects to correct inadequate storm drain capacity caused by the flat and adverse street grades in the vicinity. Figure 5-15 shows the location of each capital project. Options for parallel relief drains and full replacement are provided. Generally installing a parallel relief drain is less expensive, depending upon the number and location of existing street utilities. In the case of the Rudyard Drive Relief Drain, the undersized pipe is located within a storm drainage easement between private residences. The parallel option utilizes an alignment in Rudyard Drive as it is assumed that a parallel pipe would be very difficult to install within the easement without disturbing the existing pipe.

Table 5-41
Recommended Capital Improvements in System P4

ID	Project	Priority	Parallel Option	Replacement Option
1	Silvera Street Storm Drain Replacement	High	Use Replacement Option	Replace approx. 140 LF of existing 15-inch RCP with 27-inch RCP in the SD easement from Silvera St. to the existing 27-inch RCP that drains to Abbott Avenue.
2	Rudyard Drive Relief Drain	Low	Install approx. 600 LF of 24-inch RCP in Rudyard Dr. from Heath St. to the 27-inch SD crossing Rudyard Dr. west of Abbott Ave. Replace approx. 115 LF of (E) 27-inch RCP with 36-inch RCP in the SD easement between Silvera St. and Abbott Ave. just north of Rudyard Dr.	Replace approx. 250 LF of existing 18-inch RCP with 30-inch RCP in the SD easement from Heath St. to Smithwood St. and approx. 350 LF of existing 24-inch RCP with 36-inch RCP in the SD easement. From Smithwood St. to the existing SD east of Silvera St. Replace approx. 115 LF of (E) 27-inch RCP with 36-inch RCP in the SD easement between Silvera St. and Abbott Ave. just north of Rudyard Dr.

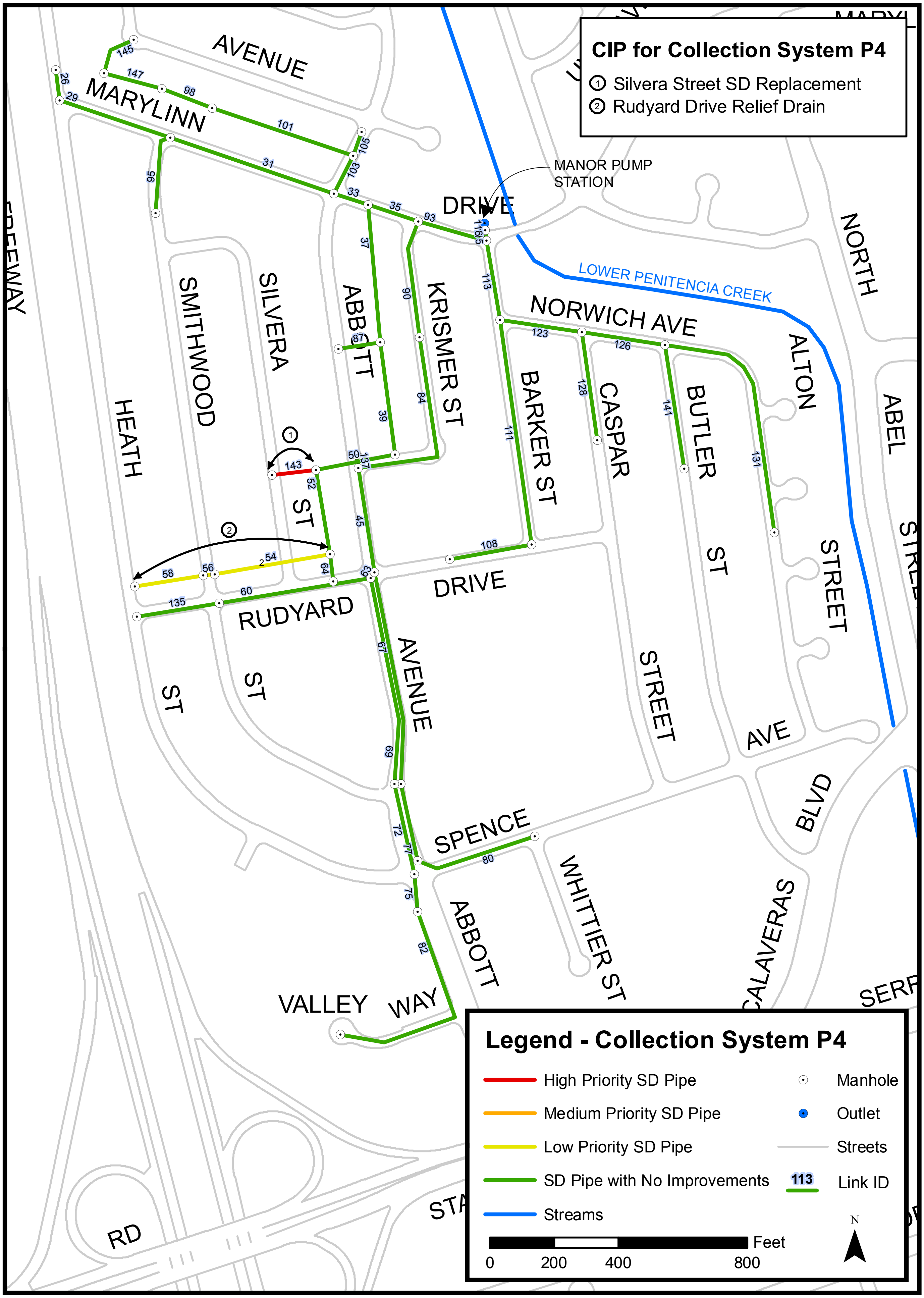
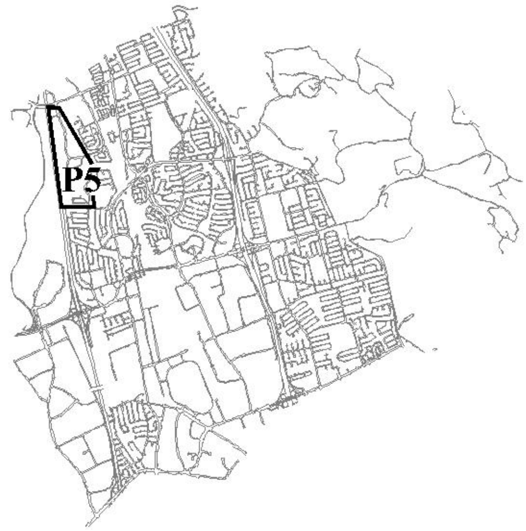

CIP for Collection System P4
1 Silvera Street SD Replacement
2 Rudyard Drive Relief Drain
MANOR PUMP STATION
LOWER PENITENCIA CREEK
AVENUE
MARYLINN
DRIVE
NORWICH AVE
NORTH
ALTON
ABEL
FREEWAY
SMITHWOOD
SILVERA
ABBOTT
KRISMER ST
BARKER ST
CASPAR
BUTLER
HEATH
ST
STREET
RUDYARD
DRIVE
AVENUE
AVE
BLVD
SPENCE
WHITTIER ST
CALAVERAS
VALLEY
WAY
RD
Legend - Collection System P4
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
Streams
Manhole
Outlet
Streets
113
Link ID
Feet
0
200
400
800
N

Figure 5-15

Penitencia Creek at Dixon Landing (P5)



The California Circle Lagoon and Pump Station drain an industrial park located between Lower Penitencia Creek and Interstate 880. The collection system also includes the Abbott Stormwater Pump Station and Lagoon, which drain a small length of Fairview Way and Cadillac Court. These lagoons also function as recreational and aesthetic amenities, as discussed in Chapter 6.

The entire system lies within Lower Penitencia Creek's 100-year floodplain, which is caused by creek overflows that become trapped against Interstate 880.

Outfalls to Lagoons

Table 5-42 lists lagoon operating parameters and their effect on backwater conditions for the storm drain analyses. The starting backwater for the tributary system is equivalent to the water surface elevation in the pump wet well plus the exit loss at the influent storm drain pipe. Lagoon and pump station operation are described in Chapter 6.

Table 5-42
Tailwater Elevations at Storm Drain Outfalls within P5 System

ID	Outfall Location	Ground Elev.	SD INV	Outfall Dia	Velocity (fps)		Lagoon WSEL (feet NAVD)		System Tailwater (feet NAVD)	
					10yr	100yr	10yr	100yr	10yr	100yr
2	California Circle Lagoon	14.0	5.15	36”	0.97	1.47	6.80	8.60	8.16	8.63
1013	California Circle Lagoon Ditch	12.0	4.58	54”	1.32	2.30	6.80	8.60	9.11	9.16
1032	Abbott Lagoon	10.8	6.73	30”	1.85	3.09	9.60	10.30	9.65	10.45

Collection System Performance

This storm water collection system performs in conformance with Storm Drain Master Plan standards.

Table 5-43
Collection System P5 Performance

	Lineal Feet	Percentage
System Acceptable / No Improvements	10,364	100
High Priority Improvements	0	0
Medium Priority Improvements	0	0
Low Priority Improvements	0	0
Total System	**10,364**	**100**

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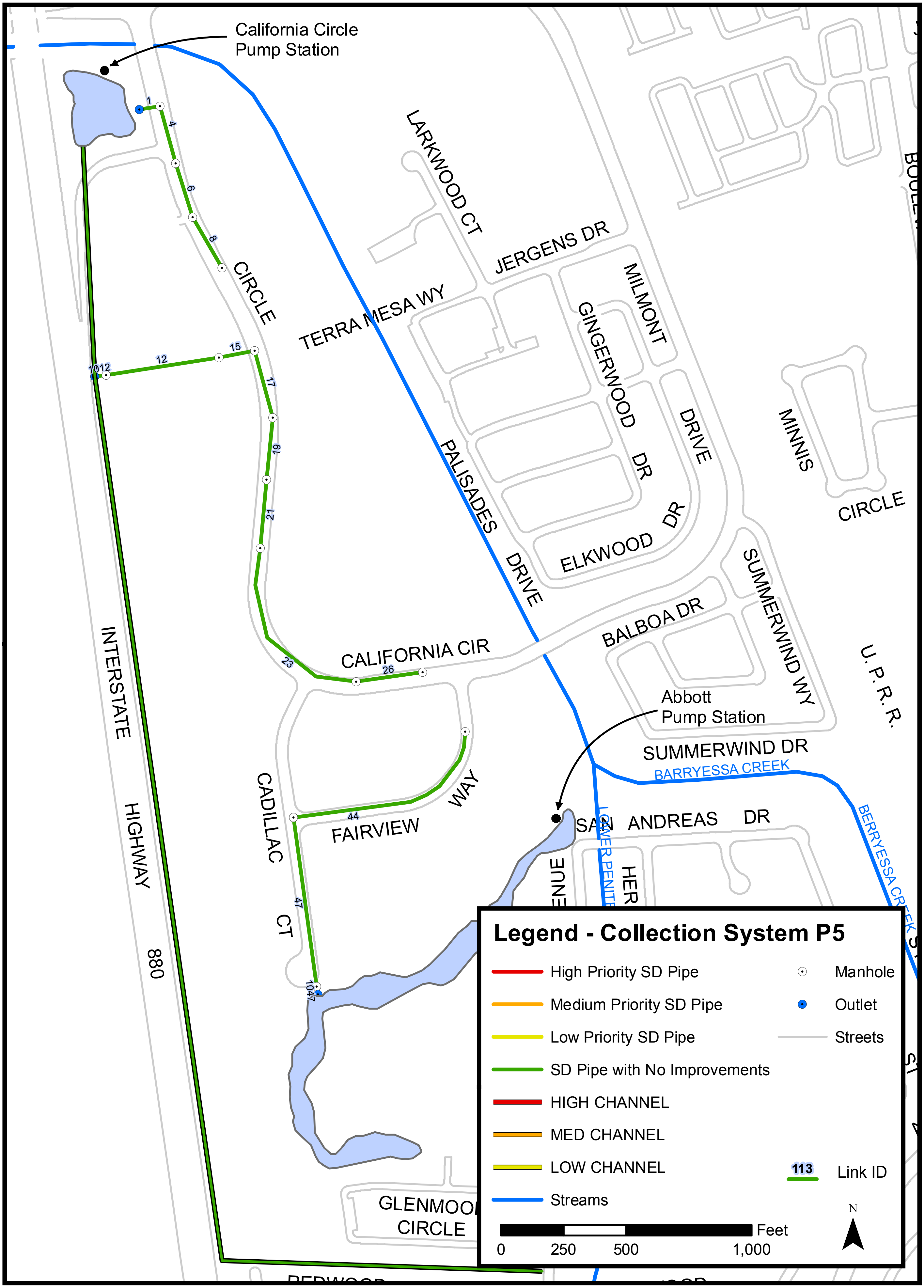
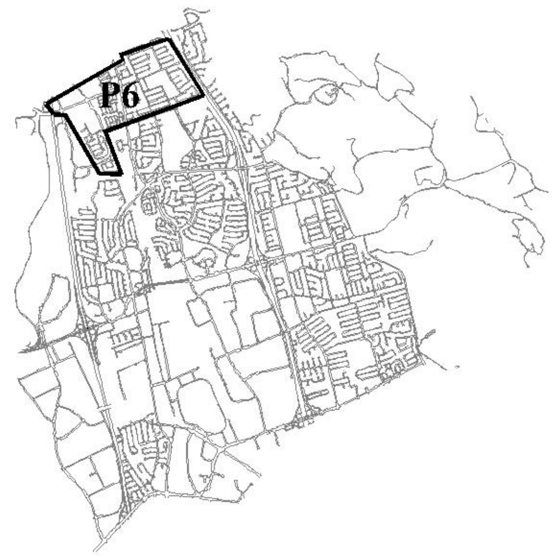

California Circle Pump Station
CIRCLE
TERRA MESA WY
LARKWOOD CT
JERGENS DR
MILMONT
GINGERWOOD DR
DRIVE
PALISADES DRIVE
ELKWOOD DR
MINNIS CIRCLE
BALBOA DR
SUMMERWIND WY
CALIFORNIA CIR
INTERSTATE HIGHWAY 880
CADILLAC CT
FAIRVIEW WAY
Abbott Pump Station
SUMMERWIND DR
BARRYESSA CREEK
BERRYESSA CREEK
SAN ANDREAS DR
U. P. R. R.
GLENMOOR CIRCLE
Legend - Collection System P5
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
HIGH CHANNEL
MED CHANNEL
LOW CHANNEL
Streams
Manhole
Outlet
Streets
113
Link ID
Feet
0
250
500
1,000
N

Figure 5-16

Penitencia Creek at Jurgens Pump Station (P6)



A primarily residential mixed-use area, Jurgens Pump Station's drainage basin is entirely enclosed with no natural drainage to any creek. Storm drains and channels lead to Dixon Landing Park and Jurgens Pump Station, which discharges water through a force main into Penitencia Creek. System P6 extends from Penitencia Creek on the west to Interstate 680 on the east, along Dixon Landing Road. All but a small portion of the basin showing intentional detention at Dixon Landing Park is mapped as a Shaded Zone X, indicating a 500-year floodplain, 100-year flood depths less than one foot, or that the area is protected by levees.

Water will back up into Dixon Landing Park during extreme runoff events. On February 3, 1998 (during what was estimated to be slightly less than a 10-year precipitation event) water backed up to a level that caused equipment to become inundated, short out, and shut off the engine driven pumps. The problem was further exacerbated when Berryessa Creek overtopped at the Union Pacific Railroad levee and contributed to the volume of water trapped behind the Penitencia Creek levee in the California Landings development. However, analyses described within Chapter 6 of this master plan indicate that even without the contribution from Berryessa Creek, the pump station would have shut itself off during the February 1998 event. Subsequently, Jurgens Pump Station was flood-proofed to prevent its shutoff during storm events. The maximum level of inundation during a 100-year storm event is 12 feet NAVD.

Collection system performance can impact the rate at which storm runoff can reach the pumping facility. In particular, an open drainage channel that parallels the east side of the Union Pacific Railroad, and a set of four 36-inch diameter reinforced concrete pipe culverts under the railroad could potentially limit the amount of water that moves through this barrier toward the pump station. Once ponded water on the east side of the UPRR exceeds elevation 15 feet NAVD, runoff will spill toward the south. (This occurs when the discharge through the culverts exceeds about 120 cfs.) As part of the SVBX project, VTA will install two additional 48-inch diameter RCP crossings under the tracks, which will solve potential flooding in this location. Local hydraulic conditions are presented in Table 5-44. The existing crossing has approximately 10-year capacity; the additional twin 48-inch diameter culverts VTA is installing will increase capacity to accommodate the 100-year discharge, once connected to the system by the City.

Table 5-44
System Capacity at UPRR Storm Drain Crossing

	10-year	100-year
Runoff at Drainage Channel	63 cfs	95 cfs
Drainage Channel Capacity when Clean (n=0.03)	125 cfs	125 cfs
Drainage Channel Capacity if Overgrown (n=0.08)	45 cfs	45 cfs
Runoff at UPRR Culverts	138 cfs	207 cfs
Upstream Hydraulic Grade with Incipient Flooding	15.0 feet	15.0 feet
Upstream Hydraulic Grade with Existing Culvert Crossing (NAVD)	14.9 feet	16.0 feet
Upstream Hydraulic Grade with Additional VTA-Installed Culverts (NAVD)	14.3 feet	14.7 feet

The results also illustrate the importance of routine channel maintenance. Allowing the ditch along the railroad to become overgrown severely restricts the conveyance of runoff, and allows storm water to back up into drainage systems along Dixon Landing Road, with the potential for inundating low lying areas.

Collection System Performance

A number of problematic storm drain systems exist, particularly when evaluated against the 100-year performance criterion (Table 5-45 and Figure 5-17). Collection system performance deficiencies identified within the area inundated by design within Dixon Landing Park are not corrected in the CIP since that area would still be inundated even with a larger subsurface storm drain system. The improved UPRR crossing to be completed by VTA is not included in the CIP.

Table 5-45
Recommended CIP for Collection System P6

	Lineal Feet	Percentage
System Acceptable / No Improvements	22,915	84
High Priority Improvements	0	0
Medium Priority Improvements	3,800	14
Low Priority Improvements	500	2
Total System	**27,215**	**100**

Capital Projects

Table 5-46 identifies capital projects to correct inadequate storm drain and channel capacities within the Jurgens Pump Station drainage basin. Figure 5-17 shows the location of each capital project. Options for parallel relief drains and full replacement are provided. Installing a parallel relief drain is generally less expensive, depending upon the number and location of existing street utilities. While identified in Table 5-45 as "high priority improvements" based on performance criteria for the existing system, capital improvements are not recommended for storm drains within Dixon Landing Park, since that area is already inundated with runoff by design.

Table 5-46
Recommended Capital Improvements in System P6

ID	Project	Priority	Parallel Option	Replacement Option
1	Arizona Avenue Relief Drain	Medium	Install approx. 1,320 LF of 30-inch RCP in Arizona Avenue between Dixon Road and Coelho Street.	Replace approx. 1,320 LF of 36-inch RCP with 48-inch RCP in Arizona Avenue between Dixon Road and Coelho Street.
2	Wilson Way SD Improvements	Medium	Install approx. 180 LF of 18-inch RCP as shown; replace approx 120 LF of existing 30-inch RCP with 48-inch RCP; and replace 840 LF of existing 33-inch RCP with 48-inch RCP, about half of which is in Dixon Landing Road.	Replace approx. 180 LF of 21-inch RCP with 30-inch RCP as shown; replace approx 120 LF of existing 30-inch RCP with 48-inch RCP; and replace 840 LF of existing 33-inch RCP with 48-inch RCP, about half of which is in Dixon Landing Road.

ID	Project	Priority	Parallel Option	Replacement Option
3	Summerwind Way Relief Drain	Medium	Install approx. 360 LF of 36-inch RCP in Summerwind Way from Balboa Drive to Milmont Drive.	Replace approx. 360 LF of existing 30-inch RCP in Summerwind Way from Balboa Drive to Milmont Drive with 48-inch RCP.
4	Milmont Drive Relief Drain	Medium	Install approx. 480 LF of 48-inch RCP in Milmont Drive from Aspenridge Drive to Jurgens Drive.	Replace approx. 480 LF of existing 36-in RCP in Milmont Drive from Aspenridge Drive to Jurgens Drive with 54-in RCP.
5	Jurgens Drive Relief Drain	Medium	Install approx. 500 LF of 54-inch RCP in Jurgens Drive from the UPRR crossing to the Jurgens Pump Station inlet and construct a new junction box at the pump station.	Replace approx. 500 LF of existing 72-inch RCP in Jurgens Drive from the UPRR crossing to the Jurgens Pump Station inlet with 84-inch RCP and modify the pump station inlet to accommodate the larger pipe.
6	Connect New RCP Crossing at UPRR/SVBX Installed by VTA	Medium	Remove bulkheads from both of the 48-inch RCP cross culverts installed by VTA and connect to upstream and downstream systems.	Same as parallel option.
7	Gingerwood Drive Relief Drain	Low	Install approx. 500 LF of 30-inch RCP in Gingerwood Drive from Aspenridge Drive to Jurgens Dr.	Replace approx. 320 LF of existing 30-inch RCP and approx 180 LF of 36-inch RCP with 48-inch RCP in Gingerwood Drive from Aspenridge Drive to Jurgens Dr.

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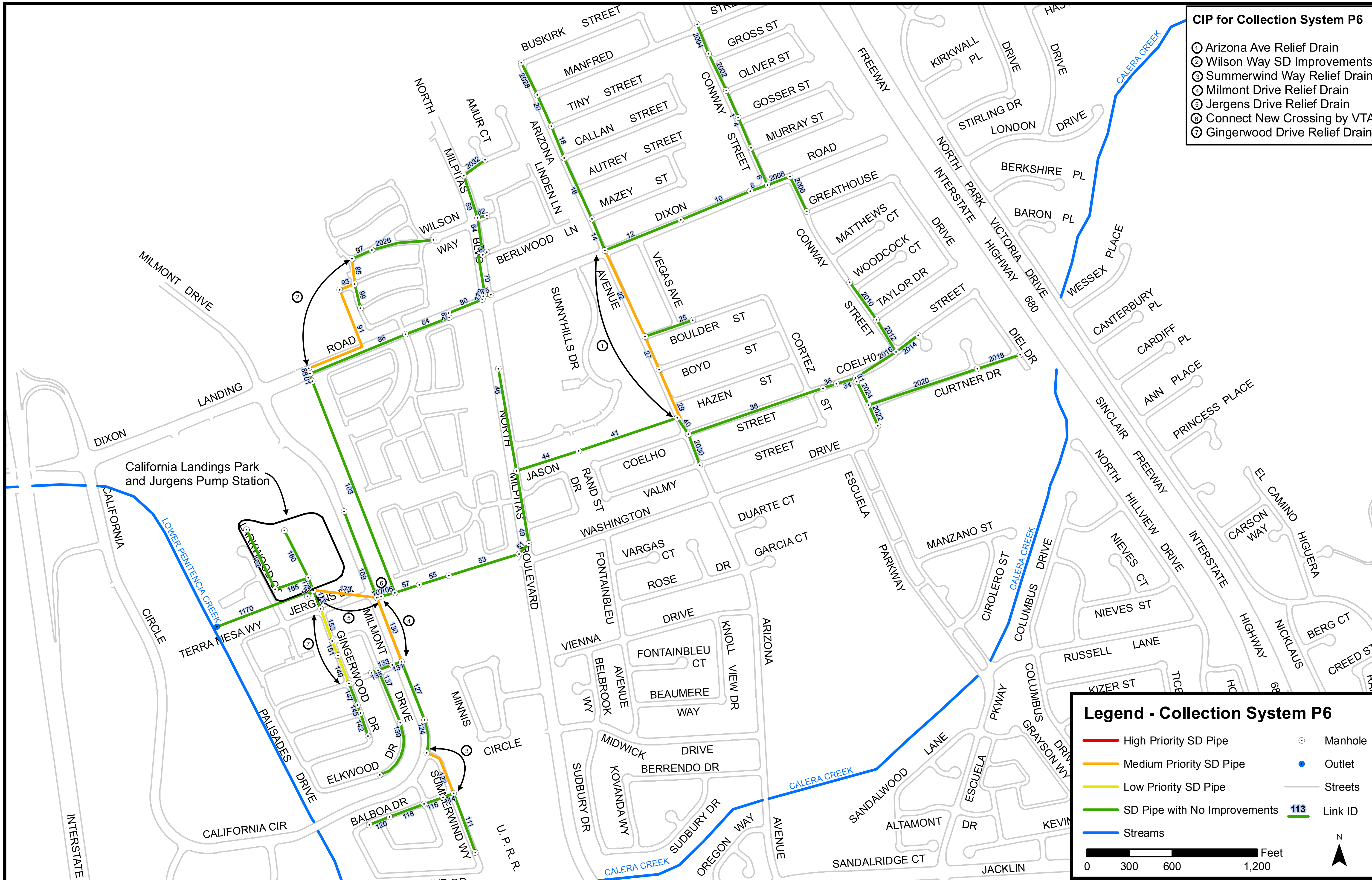
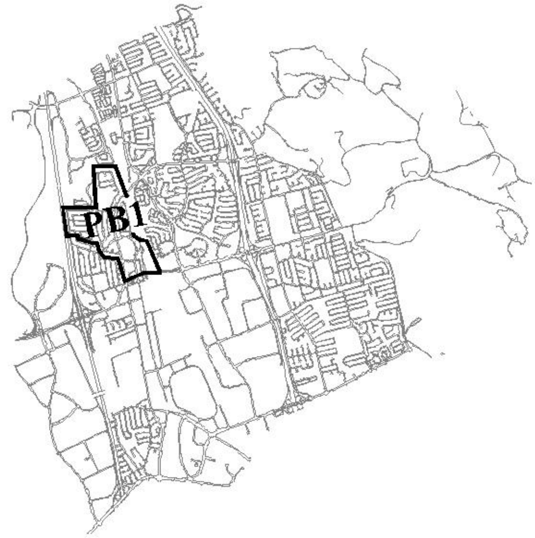


CIP for Collection System P6
1 Arizona Ave Relief Drain
2 Wilson Way SD Improvements
3 Summerwind Way Relief Drain
4 Milmont Drive Relief Drain
5 Jergens Drive Relief Drain
6 Connect New Crossing by VTA
7 Gingerwood Drive Relief Drain
California Landings Park and Jurgens Pump Station
Legend - Collection System P6
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
Streams
Manhole
Outlet
Streets
113 Link ID
0 300 600 1,200
Feet
N

Figure 5-17

Penitencia Creek at Berryessa Confluence (PB1)



Single-family residences dominate land use within this system, which is defined as the tributary to the Penitencia Creek Pump Station and Hall Memorial Park Lagoon. The area is located between Berryessa Creek on the east and Interstate 880 to the west. Local drainage extends as far south as Calaveras Boulevard. Lower Penitencia Creek travels from south to north, roughly bisecting the area.

Gradients throughout the area are flat, and several streets have no surface drainage outlet. Consequently, a strong possibility of nuisance flooding exists, particularly if any drainage inlets become plugged. Street flooding has been experienced in the area. Although many streets have no natural drainage outlet, there are positive overland release points to the Interstate 880 ditch that eventually drain to the California Circle Pump Station and to the corner of Hermina Street and La Honda Drive, where the Penitencia Pump Station is located.

Gravity Outfalls at Hall Memorial Park Lagoon

All storm drains in the system discharge to the Hall Memorial Park Lagoon, located on the west side of Lower Penitencia Creek adjacent to Abbott Avenue. Stored runoff from the lagoon is discharged to the creek by the Penitencia Pump Station, located immediately across the creek in Hall Memorial Park. Table 5-47 lists the 10- and 100-year starting tailwater elevations at each gravity outfall, using the criteria outlined in Chapter 3. The starting backwater for the tributary system is equivalent to the water surface elevation in the receiving water at the time of peak runoff plus the exit loss at the storm drain pipe outfall.

Table 5-47
Tailwater Elevations at Storm Drain Outfalls within PB1 System

ID	Outfall Location on Hall Memorial Park Lagoon	Ground Elev.	SD INV	Outfall Dia	Velocity (fps)		Lagoon WSEL (feet NAVD)		System Tailwater (feet NAVD)	
					10yr	100yr	10yr	100yr	10yr	100yr
36	North End – Pump Station	18.2	-1.86	54”	3.49	5.28	7.3	8.2	7.49	8.63
1113	South End	11.2	7.61	24”	3.22	3.24	6.2	6.8	9.77	9.78

Collection System Performance

Storm drainage within this system suffers due to the lack of relief for overloaded storm drains. Table 5-48 summarizes the recommended CIP for the PB1 system. To avoid inducing downstream flooding, much of the recommended CIP is classified as high priority.

Table 5-48
Recommended CIP for Collection System PB1

	Lineal Feet	Percentage
System Acceptable / No Improvements	7,060	42
High Priority Improvements	9,475	55
Medium Priority Improvements	0	0
Low Priority Improvements	450	3
Total System	**16,985**	**100**

Capital Improvements

Table 5-49 identifies capital projects to mitigate areas of residual ponding under 10-year and 100-year conditions caused by flat street grades and the lack of natural relief. Improvements necessary to alleviate this ponding are described below and shown in Figure 5-18. Detailed design work by the City has shown that improving the Abbott Avenue storm drain system between Redwood Avenue and Hall Memorial Park Lagoon is extremely problematic given conflicting utilities and other field constraints. Therefore, storm water will continue to be stored within the streets to the back of sidewalk, and released toward the California Avenue Pump Station through the concrete lined ditch between Redwood Avenue and Glenmore Circle, which continues to the ditch that parallels Interstate 880.

Table 5-49
Recommended Capital Improvements in System PB1

ID	Project	Priority	Parallel Option	Replacement Option
1	Redwood Avenue Relief Drain	High	Install approx 1,300 LF of 24-inch RCP in Redwood Ave from Heath St to the existing Abbott Ave SD.	Along Redwood Ave, replace approx 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.
2	Abbott Avenue Relief Drain	High	Install approx 1,425 LF of 36-inch RCP in Abbott Ave from Walnut Dr to Redwood Ave.	On Abbott Ave, replace approx 400 LF of existing 15-inch RCP with 24-inch RCP between Walnut Dr and Elm Ave; approx 255 LF of existing 21-inch RCP from Elm Ave to Willow Ave and approx 250 LF of existing 24-inch RCP with 42-inch RCP from Willow Ave to Chestnut Ave. Install approx 520 LF of 42-inch RCP in Abbott Ave from Chestnut to Redwood Ave.
3	Maple Avenue Relief Drain	High	Install approx 390 LF of 18-inch RCP in Maple Ave from the existing SD easement to Abbott Ave.	Replace approx 220 LF of 12-inch RCP with 24-inch RCP in the SD easement between Maple Ave and Redwood Ave.
4	Chestnut Avenue Relief Drain	High	Install approx 1,060 LF of 36-inch RCP in Chestnut Ave from Heath St to Abbott Ave.	Along Chestnut Ave, replace approx 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.

ID	Project	Priority	Parallel Option	Replacement Option
5	Heath Street Relief Drain	High	Install approx 520 LF of 36-inch RCP in Heath St from Elm Ave to Chestnut Ave.	Replace approx 520 LF of 15-inch RCP with 42-inch RCP in Heath St from Elm Ave to Chestnut Ave.
6	North Abel Street Relief Drain	High	Install approx 2,530 LF of 48-inch RCP in North Abel St from Penitencia St to the SD easement midway between Redwood Ave and Berryessa Creek.	Not recommended to avoid upsizing downstream storm drain pipes, which are not within street rights-of-way. Use parallel pipe option.
7	Vasona Street SD Improvement	High	Install approx 290 LF of 24-inch RCP on Vasona Street between Almaden Ave and Marylinn Dr. Replace approx 240 LF of existing 21-inch RCP with 48-inch RCP in the storm drain easement between Vasona St and North Abel St. Replace approx 200 LF of existing 12-inch RCP with 42-inch RCP in the Vasona Street cul-de-sac adjacent to N. Abel St.	Replace approx 290 LF of existing 15-inch RCP with 30-inch RCP on Vasona Street between Almaden Ave and Marylinn Dr. Replace approx 240 LF of existing 21-inch RCP with 48-inch RCP in the storm drain easement between Vasona St and North Abel St. Replace approx 200 LF of existing 12-inch RCP with 42-inch RCP in the Vasona Street cul-de-sac adjacent to N. Abel St.
8	Penitencia Pump Station Rehabilitation	High	Full rehabilitation or replacement of Penitencia Pump Station. (See Page 6-19)	
9	Lexington Street SD Improvements	High	Use replacement option.	Replace approx 220 LF of existing 15-inch RCP with 36-inch RCP in the existing SD easement between Penitencia St and Lexington St; approx 260 LF of existing 18-inch and 21-inch RCP with 42-inch RCP between Lexington St and Coyote St and approx 290 LF of existing 27-inch RCP with 48-inch RCP in the SD easement between Coyote St and North Abel St.
10	Coyote Street Relief Drain	High	Install approx 750 LF of 36-inch RCP in Coyote St from low point in street to Uvas Ave	Replace approx 400 LF of existing 15-inch RCP and approx 110 of existing 24-inch RCP with 42-inch RCP in Coyote St to the existing SD easement between Coyote St and North Abel St; and approx 240 LF of existing 15-inch RCP with 42-inch RCP between Uvas Ave and the referenced existing SD easement.
11	Berryessa Street Relief Drain	Low	Install approx 450 LF of 18-inch RCP in Berryessa St at the Calero St intersection.	Replace approx 450 LF of existing 15-inch RCP in Berryessa St with 21-inch RCP to match downstream pipe size.

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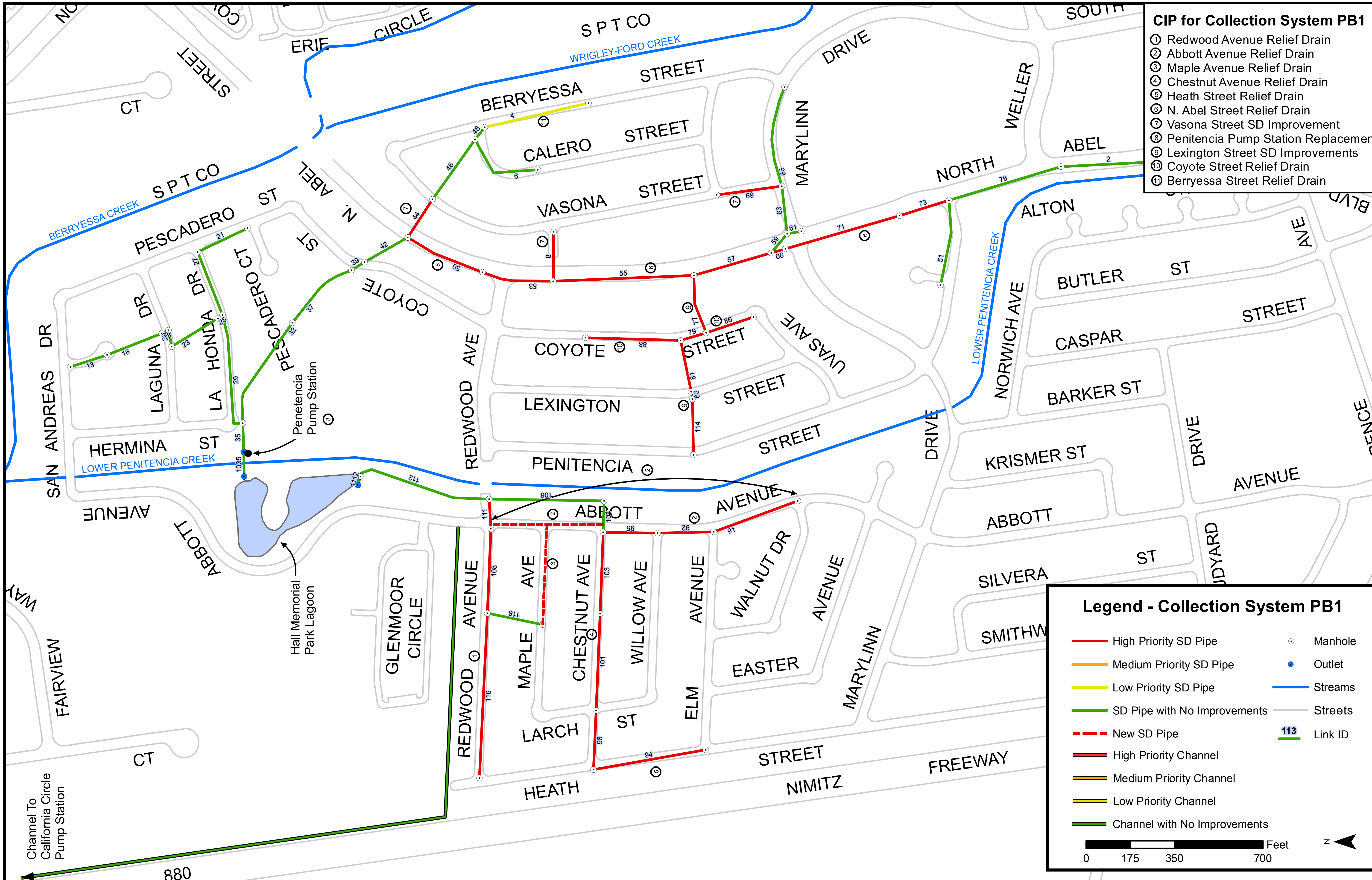
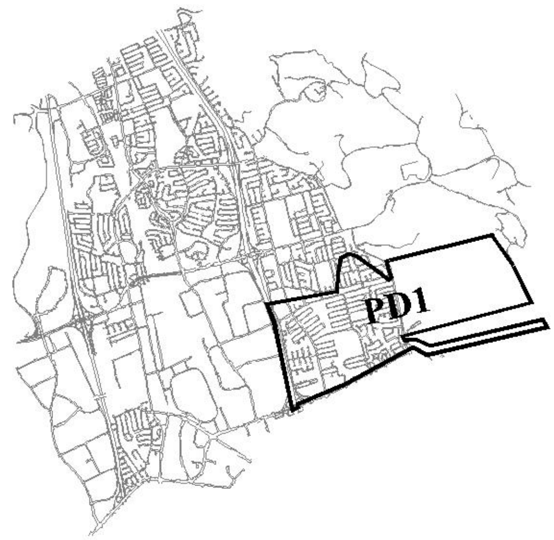


CIP for Collection System PB1
1 Redwood Avenue Relief Drain
2 Abbott Avenue Relief Drain
3 Maple Avenue Relief Drain
4 Chestnut Avenue Relief Drain
5 Heath Street Relief Drain
6 N. Abel Street Relief Drain
7 Vasona Street SD Improvement
8 Penitencia Pump Station Replacement
9 Lexington Street SD Improvements
10 Coyote Street Relief Drain
11 Berryessa Street Relief Drain
Legend - Collection System PB1
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
New SD Pipe
High Priority Channel
Medium Priority Channel
Low Priority Channel
Channel with No Improvements
Manhole
Outlet
Streams
Streets
113 Link ID
Feet
0 175 350 700
WRIGLEY-FORD CREEK
BERRYESSA CREEK
LOWER PENITENCIA CREEK
S P T CO
BERRYESSA STREET
CALERO STREET
VASONA STREET
COYOTE STREET
LEXINGTON STREET
PENITENCIA STREET
ABBOTT AVENUE
HEATH STREET
NIMITZ FREEWAY
880
Penetencia Pump Station
Hall Memorial Park Lagoon
Channel To California Circle Pump Station

Figure 5-18

Piedmont Creek East of 680 (PD1)



Piedmont Creek drains just over one square mile of the southeastern corner of Milpitas into Berryessa Creek near Yosemite Drive, between Hillview Drive and Vista Way. A significant amount of hillside area drains into the local collection system along Piedmont Road. All storm drainage collection systems eventually discharge to Piedmont Creek, which is maintained for the most part by the Santa Clara Valley Water District as described in Chapter 4.

Between Piedmont Road and Roswell Drive, the creek is confined to a District-maintained storm drain system ranging in size from 48-inch diameter pipe to 84-inch pipe. Downstream of Roswell Drive the creek is contained within reinforced box culverts (8' x 5' to 8' x 7' in size) at road crossings, and by concrete “U-frame” channels and excavated earth trapezoidal channels. Analyses show that the District facilities function properly with both 10-year and 100-year design discharges.

There have been reports of sedimentation within this collection system, since steep hillside areas are tributary to the local storm drains. (North Branch Piedmont Creek drains directly into a local system through a debris capture device.) Once topography flattens west of Piedmont Road much of the sediment load gathered from the steep hillside drops out and can block storm drains. Upstream debris basins and storm drain inlet retrofits are recommended at the locations shown on Figure 5-19 to improve this maintenance issue. Chapter 9 describes inlet retrofitting in more detail.

Collection System Performance

This storm water collection system generally performs well against the design criteria due to the relatively steep topography and resulting storm drain capacities. Table 5-50 summarizes the recommended CIP for Collection System PD1, which involves relatively little improvement.

Table 5-50
Recommended CIP for Collection System PD1

	Lineal Feet	Percentage
System Acceptable / No Improvements	52,971	94
High Priority Improvements	0	0
Medium Priority Improvements	570	1
Low Priority Improvements	2,980	5
Total System	**56,521**	**100**

Capital Improvements

Table 5-51 identifies capital projects shown on Figure 5-19 needed to mitigate substandard collection system performance.

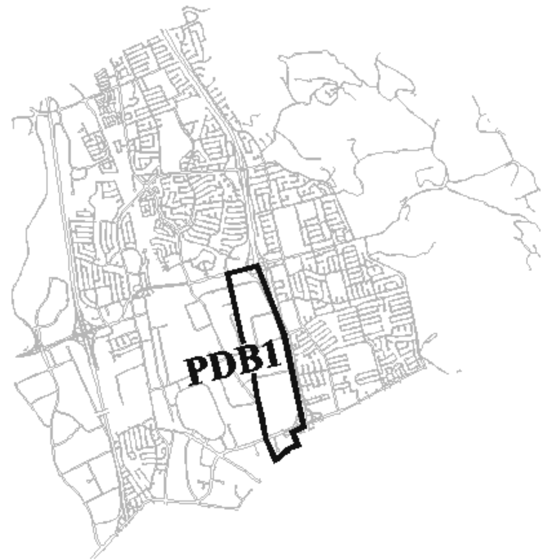
Table 5-51
Recommended Capital Improvements in System PD1

ID	Project	Priority	Parallel Option	Replacement Option
1	Vista Way Relief Drain	Medium	Install approx 260 LF of 36-inch RCP in Vista Way from the low point north of Yosemite Drive to Piedmont Creek.	Replace approx 260 LF of existing 33-inch RCP in Vista Way from the low point north of Yosemite Drive to Piedmont Creek with 48-inch RCP.
2	Falcato Drive Relief Drain	Medium	Install approx 310 LF of 24-inch RCP in Falcato Drive from Frank Court to Sepulveda Drive.	Replace approx 310 LF of existing 15-inch RCP in Falcato Drive from Frank Court to Sepulveda Drive with 30-inch RCP.
3	South Park Victoria Drive Relief Drain	Low	Install approx 430 LF of 24-inch RCP in South Park Victoria Drive from the existing 24-inch SD south of Big Basin Drive to Clear Lake Ave; and 790 LF of 30-inch RCP from Clear Lake Ave to Mt. Shasta Ave.	Along South Park Victoria Drive, replace approx 170 LF of existing 27-inch RCP and 260 LF of existing 30-inch RCP with 36-inch RCP; and 790 LF of existing 36-inch RCP with 48-inch RCP.
4	Dempsey Road Relief Drain	Low	Install approx 1,760 LF of 30-inch RCP in Dempsey Road from Cuciz Lane to Mt. Shasta Ave.	On Dempsey Road, replace approx 670 LF of existing 18-inch RCP and 500 LF of existing 24-inch RCP with 36-inch RCP; and 590 LF of existing 30-inch RCP with 42-inch RCP.
5	Debris Basins and Storm Drain Inlet Modifications	Medium	Per Figure 5-19. Debris basin size to be determined from criteria presented in Chapter 3 and specific conditions at each location determined during the design phase.	


CIP for Collection System PD1
1 Vista Way Relief Drain
2 Falcato Drive Relief Drain
3 South Park Victoria Drive Relief Drain
4 Dempsey Road Relief Drain
5 Debris Basins and Inlet Modifications
Debris Basin (Size)
Inlet Modification
(0.15 ac-ft)
(0.1 ac-ft)
Legend - Collection System PD1
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
High Priority Channel
Medium Priority Channel
Low Priority Channel
Channel with No Improvements
Manhole
Outlet
Streams
Streets
Link ID
Feet
0 375 750 1,500
N

Figure 5-19

Piedmont Creek at Berryessa Confluence (PDB1)



This grouping contains two small collection systems that drain industrial areas between Interstate 680 and Berryessa Creek to the west. The northerly system drains an area off Wrigley Way, while the southerly system drains a small area tributary to Montague Expressway. Both systems drain directly into Berryessa Creek, which is maintained by the Santa Clara Valley Water District (Figure 4-1). The Piedmont Creek confluence with Berryessa Creek is located just upstream from the Wrigley Way outfall. Some of the drainage basins in this area drain directly to Berryessa Creek.

Gravity Outfalls to Berrryessa Creek

All storm drains in the system discharge to Lower Penitencia Creek through gravity outfalls. Table 5-52 lists the 10- and 100-year starting tailwater elevations at each gravity outfall, using the criteria outlined in Chapter 3. The starting backwater for the tributary system is equivalent to the water surface elevation in the receiving water plus the exit loss at the storm drain pipe outfall.

Table 5-52
Tailwater Elevations at Storm Drain Outfalls within PDB1 System

ID	Outfall Location on Berryessa Creek	Ground Elev.	SD INV	Outfall Dia	Velocity (fps)		Creek WSEL (feet NAVD)		System Tailwater (feet NAVD)	
					10yr	100yr	10yr	100yr	10yr	100yr
53	Montague Expressway	64.00	56.69	30	4.85	7.11	62.78	62.78	63.15	63.56
74	Ames Avenue	52.00	43.50	30	7.01	11.31	45.78	45.78	46.76	47.99
1020	1,470' D/S Yosemite Drive	34.00	29.44	27	8.13	11.79	33.78	34.00	34.00	34.00

Collection System Performance

Table 5-53 summarizes the recommended CIP within the PDB1 system and presents the relevant statistics. On-site storm drains recently replaced by the Sinclair Renaissance II project are not considered part of the City system.

Table 5-53
Recommended CIP for Collection System PDB1

	Lineal Feet	Percentage
System Acceptable / No Improvements	3,922	79
High Priority Improvements	370	7
Medium Priority Improvements	680	14
Low Priority Improvements	0	0
Total System	**4,972**	**100**

Capital Improvements

Table 5-54 and Figure 5-20 identify capital projects to mitigate areas of residual ponding caused primarily by the lack of street right-of-way to convey storm drain overflows.

Table 5-54
Recommended Capital Improvements in System PDB1

ID	Project	Priority	Parallel Option	Replacement Option
1	Wrigley Way SD Replacement	High	Use replacement option.	Replace approx 40 LF of existing 24-inch RCP and 330 LF of existing 27-inch RCP with 36-inch RCP from Wrigley Way to Berryessa Creek.
2	Watson Court Relief Drain	Medium	Install approx 310 LF of 18-inch RCP in Watson Court from its low point to Montague Expressway and 370 LF of 24-inch RCP in Montague Expressway to Berryessa Creek.	Replace approx 310 LF of existing 24-inch RCP with 30-inch RCP in Watson Court and replace approx 370 LF of existing 30-inch RCP with 36-inch RCP in Montague Expressway to Berryessa Creek.

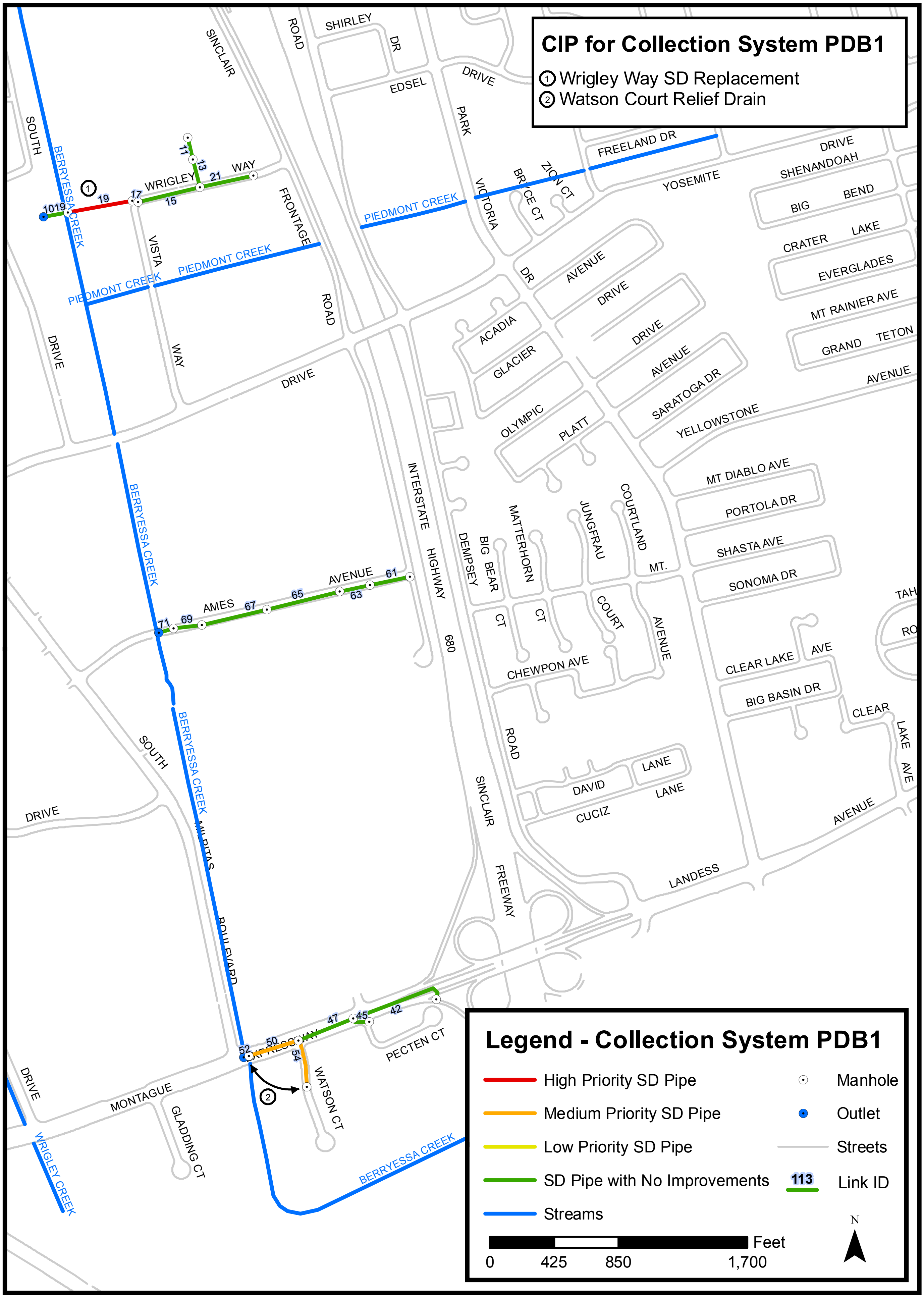
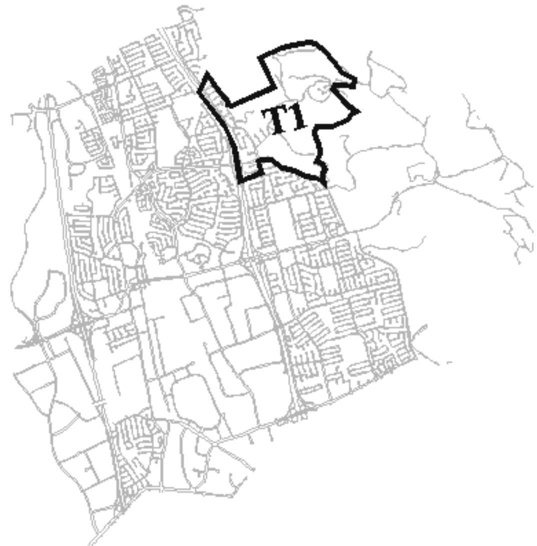

CIP for Collection System PDB1
1 Wrigley Way SD Replacement
2 Watson Court Relief Drain
Legend - Collection System PDB1
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
Streams
Manhole
Outlet
Streets
113 Link ID
Feet
0 425 850 1,700
N
WRIGLEY WAY
AMES AVENUE
BERRYESSA CREEK
PIEDMONT CREEK
WRIGLEY CREEK
INTERSTATE HIGHWAY 680
SINCLAIR FREEWAY
MONTAGUE EXPRESSWAY
WATSON CT
PECTEN CT
GLADDING CT
LANDESS AVENUE
YOSEMITE DRIVE
FREELAND DR

Figure 5-20

Tularcitos Creek East of 680 (T1)



Tularcitos Creek drains hillside areas to the east of Interstate 680 in the northeastern corner of Milpitas. The system is located east of the freeway, north of Jacklin Road. Ann Place is the demarcation between this system and the Calera Creek system (CA1).

Tularcitos Creek and its tributaries drain through local collection pipes into a ditch that parallels the northbound lane of Interstate 680. Eventually, all of the local and hillside drainage discharges to a box culvert underneath the highway. On the west side of Interstate 680, the Santa Clara Valley Water District facility contains the storm water runoff, and eventually also drains the WTCA1 and BT1 systems.

Because the area is generally so steep, the upper Tularcitos Creek system is not subject to frequent flooding, but has experienced landslides in the past, particularly along Country Club Drive and Calaveras Ridge Drive. While this master plan does not address geotechnical issues associated with such natural phenomena, a storm drain inlet modification potentially help with the associated debris and sediment loads. The potential inlet modification location is shown on Figure 5-21. There have also been reports of runoff bubbling up from inlets near the intersection of Calaveras Ridge Drive with Country Club Drive.

Gravity Outfall at Tularcitos Creek

All storm drains in the system discharge to the ditch that parallels Interstate 680, which discharges to the Tularcitos Creek box culvert at Interstate 680 south of Jacklin Road. Table 5-55 lists the 10- and 100-year starting tailwater elevations at the gravity outfall, using the criteria outlined in Chapter 3. The starting backwater for the tributary system is equivalent to the water surface elevation in the receiving water plus the exit loss at the outfall.

Table 5-55
Tailwater Elevations at Storm Drain Outfall within T1 System

ID	Outfall Location on Tularcitos Creek	Ground Elev.	SD INV	Outfall Dia	Velocity (fps)		Creek WSEL (feet NAVD)		System Tailwater (feet NAVD)	
					10yr	100yr	10yr	100yr	10yr	100yr
74	U/S face Interstate 680	37.50	23.15	n/a	3.19	3.41	25.80	26.67	25.90	26.80

Collection System Performance

Table 5-56 summarizes the recommended CIP within the T1 system and presents the relevant statistics. Most of the drainage systems are adequate and the recommended CIP is efficient.

Table 5-56
Recommended CIP for Collection System T1

	Lineal Feet	Percentage
System Acceptable / No Improvements	13,182	97
High Priority Improvements	300	2
Medium Priority Improvements	150	1
Low Priority Improvements	0	0
Total System	**13,632**	**100**

Capital Improvements

Table 5-57 and Figure 5-21 identify the capital project that solves identified drainage system performance problems near Jacklin Road.

Table 5-57
Recommended Capital Improvement in System T1

ID	Project	Priority	Parallel Option	Replacement Option
1	Jacklin Road Relief Drain	High	Install approx 300 LF of 72-inch RCP in Jacklin Road from the perpendicular 36-inch storm drain line to the channel adjacent to Interstate 680 in a new outfall.	Along Jacklin Road, replace approx 300 LF of existing 30-inch RCP with 84-inch RCP and construct a new 84-inch outfall to the channel adjacent to Interstate 680.
2	Calaveras Ridge SD Outfall	Medium	Install approx 150 LF of 24-inch RCP or HDPE pipe, crossing Country Club Drive and outfall at the new debris basin.	Same as parallel option.
3	Storm Drain Inlet Modification	Medium	Per Figure 5-21. (See also Chapter 9.)	

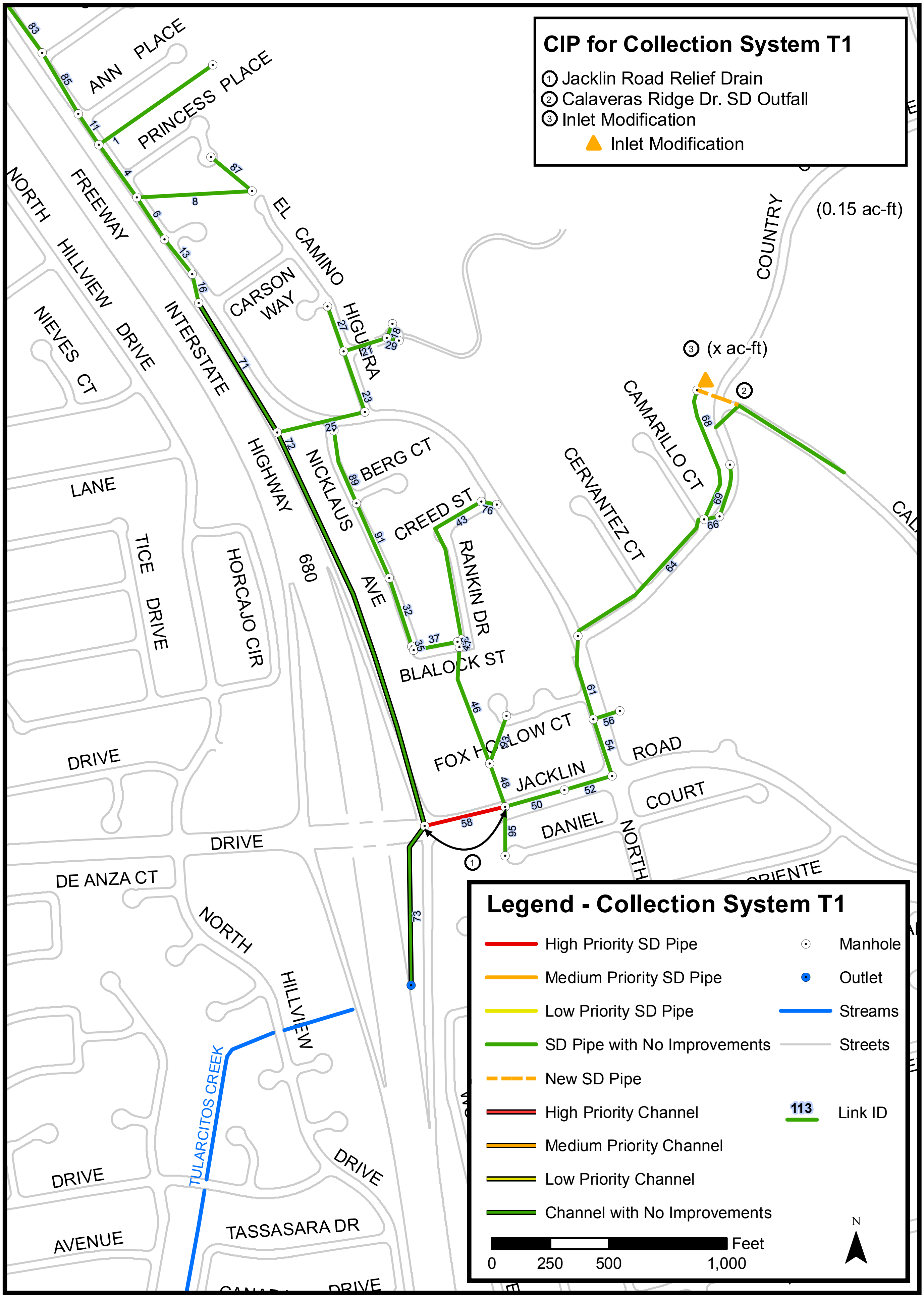
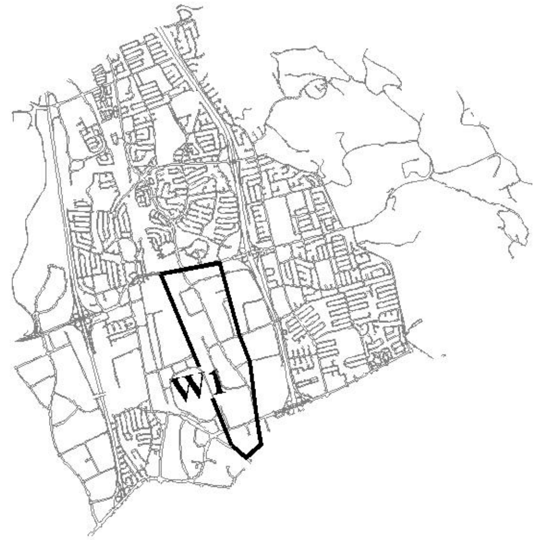

CIP for Collection System T1
1 Jacklin Road Relief Drain
2 Calaveras Ridge Dr. SD Outfall
3 Inlet Modification
Inlet Modification
(0.15 ac-ft)
3 (x ac-ft)
Legend - Collection System T1
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
New SD Pipe
High Priority Channel
Medium Priority Channel
Low Priority Channel
Channel with No Improvements
Manhole
Outlet
Streams
Streets
113
Link ID
Feet
0 250 500 1,000
N
ANN PLACE
PRINCESS PLACE
EL CAMINO
CARSON WAY
HIGUERA
NORTH HILLVIEW DRIVE
FREEWAY
INTERSTATE HIGHWAY 680
NIEVES CT
BERG CT
CREED ST
NICKLAUS AVE
RANKIN DR
BLALOCK ST
FOX HOLLOW CT
JACKLIN ROAD
DANIEL COURT
CERVANTEZ CT
CAMARILLO CT
COUNTRY
LANE
TICE DRIVE
HORCAJO CIR
DRIVE
DE ANZA CT
NORTH HILLVIEW DRIVE
TULARCITOS CREEK
TASSASARA DR
AVENUE
ORIENTE

Figure 5-21

Wrigley Creek (W1)



This is a heavy industrial area located between Berryessa Creek and Wrigley Creek, from Montague Expressway to State Highway 237. The local collection system is made up of storm drains and laterals, and Wrigley Creek itself, which joins Ford Creek north of Highway 237.

During the February 1998 storm event, localized flooding was experienced on Hillview Drive, on South Milpitas Boulevard at Montague Expressway, and on Gladding Court. Analysis does not indicate systemic problems at each location; rather the ponding appears to be caused primarily by low gutter gradients. As part of VTA's SVBX project improvements, new drainage systems will be installed within the Milpitas Station south of Montague Expressway and along the SVBX alignment in Piper Drive. Hydraulic analyses performed for VTA indicate that these project improvements will function adequately.

Wrigley Creek Discharge

Wrigley Creek collects local storm water runoff and discharges it to Wrigley-Ford Creek and eventually to the Wrigley-Ford Pump Station and Berryessa Creek. Table 5-58 lists the 10- and 100-year design discharges in the Ford Creek and Wrigley-Ford Creek system. (Reference is also made to System F1 for Ford Creek discharges.)

Table 5-58
Storm Water Discharge in Ford Creek, Wrigley Creek, and Wrigley-Ford Creek

Creek	Location	Tributary Area (acres)	10-year Discharge (cfs)	100-year Discharge (cfs)
Ford Creek	Bothello Avenue	255	110	155
	Calaveras Boulevard	298	130	175
Wrigley Creek	Montague Expressway	50	30	50
	Piper Drive Outfall	85	55	80
	Gibralter Drive Outfall	169	100	150
	Yosemite Drive Outfall	220	130	200
	Los Coches Street Outfall	339	140	230
	Calaveras Boulevard	422	170	280
Wrigley Ford Creek	At Confluence	760	290	400

Gravity Outfalls at Wrigley Creek

Wrigley Creek carries a 100-year discharge of about 280 cfs at Highway 237 (Calaveras Boulevard). VTA has replaced three 60-inch diameter CMP arch culverts at the UPRR crossing of Wrigley Creek with a double 12-foot by 6-foot reinforced concrete box culvert. Although the new box culvert is longer than the pre-existing multiple barrel CMP culvert, VTA did not change the invert grade of Wrigley Creek and analyses using HEC-RAS indicate that with all culvert crossings clean, the creek can safely discharge its one percent base flood. Wrigley Creek is generally well maintained and free of obstructions from Calaveras Boulevard upstream to Yosemite Drive.

New VTA Box Culvert

Figure 5-22 illustrates water surface elevations in Wrigley-Ford Creek and Wrigley Creek to the culvert at Calaveras Boulevard. This profile assumes that downstream sediment surveyed in the field is not removed by discharges during flood events, because the sediment is so pervasive downstream. Schaaf & Wheeler completed limited field surveys of Wrigley Creek from Yosemite Drive to the confluence of Wrigley-Ford Creek and prepared a hydraulic model for existing conditions in the creek under the design 10-year and 100-year discharges. This model has been used to evaluate flow capacity in Wrigley Creek and determine tailwater elevations.

Wrigley Creek Parallel to Topaz

Table 5-59 lists the 10- and 100-year starting tailwater elevations at each gravity outfall to Wrigley Creek using the criteria outlined in Chapter 3. The starting backwater for the tributary system is equivalent to the water surface elevation in the receiving water plus the exit loss at the storm drain pipe outfall.

Table 5-59
Tailwater Elevations at Storm Drain Outfalls within W1 System

ID	Outfall Location on Wrigley Creek	Ground Elev.	SD INV	Outfall Dia	Velocity (fps)		Creek WSEL (feet NAVD)		System Tailwater (feet NAVD)	
					10yr	100yr	10yr	100yr	10yr	100yr
108	Gibraltar Court SD Outfall	26.12	21.45	RCB	3.32	5.07	22.90	23.50	23.07	23.90
233	1140' U/S Calaveras Boulevard	21.79	13.94	42	1.94	3.21	17.51	18.50	17.57	18.66
235	600' U/S Calaveras Boulevard	20.00	11.66	48	2.83	4.56	16.94	18.08	17.06	18.40
255	2675' U/S Calaveras Boulevard	24.05	19.30	42	2.06	3.43	20.86	21.47	22.87	22.98
259	Calaveras Boulevard	19.33	11.49	36	2.92	4.27	16.20	17.65	16.33	17.93

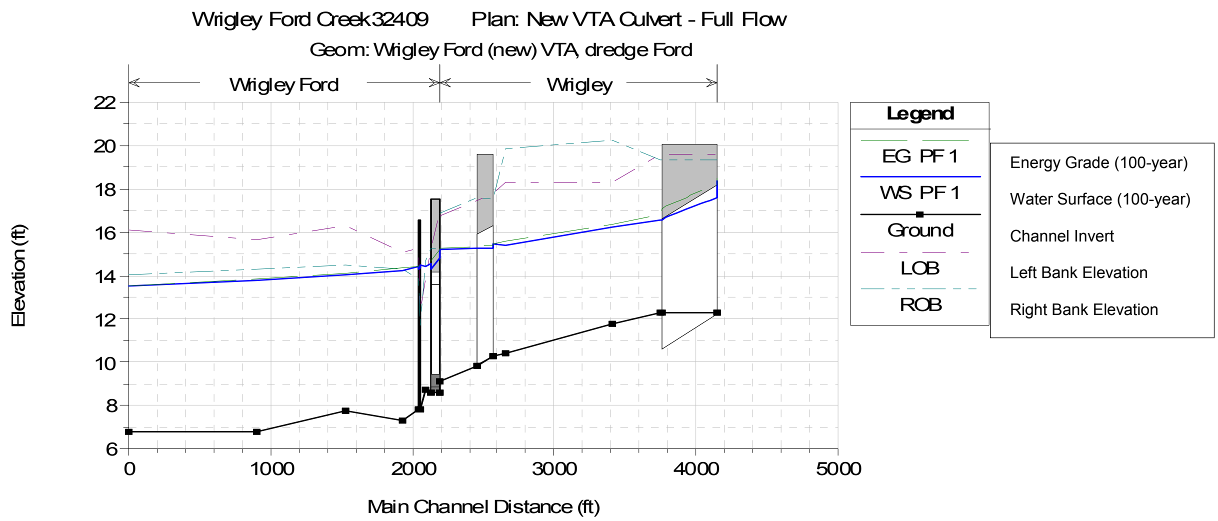


Figure 5-22: Wrigley-Ford Creek and Wrigley Creek 100-Year Water Surface Profile

Collection System Performance

Table 5-60 presents the analytical performance statistics for the W1 system. The entire storm drain system meets the design criteria (Figure 5-23). The one area with a potential low priority improvement will be remedied when VTA installs a planned storm drain improvement on Piper Drive that improves flow conveyance from Montague Expressway into Wrigley Creek.

Table 5-60
Collection System W1 Performance

	Lineal Feet	Percentage
System Acceptable / No Improvements	33,598	100
High Priority Improvements	0	0
Medium Priority Improvements	0	0
Low Priority Improvements	0	0
Total System	**33,598**	**100**

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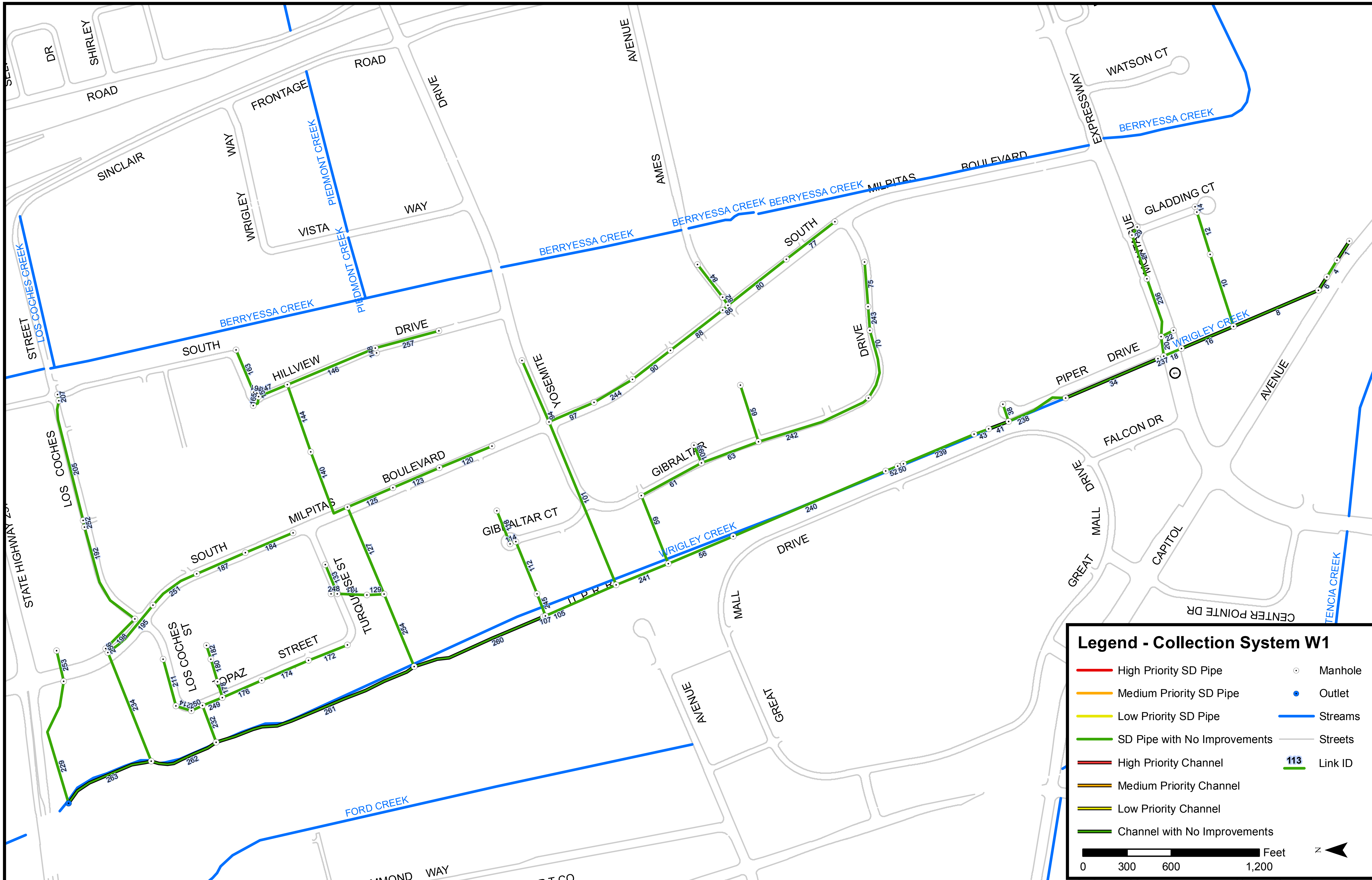
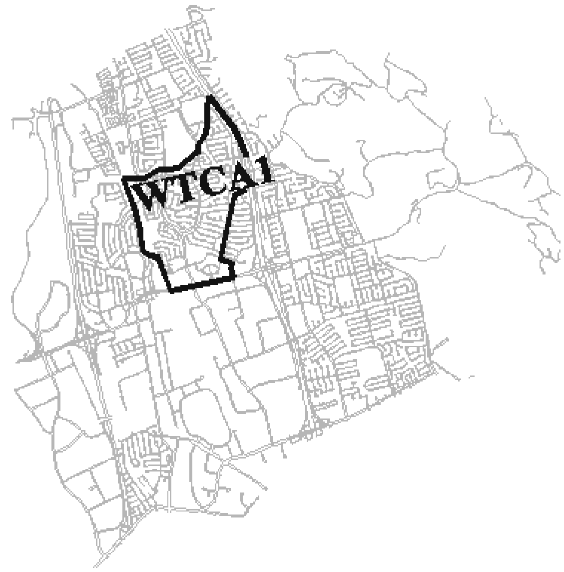

Legend - Collection System W1
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
High Priority Channel
Medium Priority Channel
Low Priority Channel
Channel with No Improvements
Manhole
Outlet
Streams
Streets
113
Link ID
Feet
0
300
600
1,200
N
BERRYESSA CREEK
PIEDMONT CREEK
LOS COCHES CREEK
WRIGLEY CREEK
FORD CREEK
FRONTAGE ROAD
SINCLAIR
WRIGLEY WAY
VISTA WAY
SOUTH HILLVIEW DRIVE
MILPITAS BOULEVARD
YOSEMITE DRIVE
AMES AVENUE
SOUTH MILPITAS BOULEVARD
EXPRESSWAY
MONTAGUE
WATSON CT
GLADDING CT
PIPER DRIVE
FALCON DR
GIBRALTAR DRIVE
GIBRALTAR CT
GREAT MALL DRIVE
GREAT MALL AVENUE
CAPITOL AVENUE
CENTER POINTE DR
STATE HIGHWAY 237
LOS COCHES STREET
LOS COCHES ST
SOUTH PAZ STREET
TURQUOISE ST
U P R R
HAMMOND WAY
S P T CO

Figure 5-23

Wrigley / Tularcitos / Calera Creek at Jacklin Road (WTCA1)



This primarily residential area is bounded by Calera Creek on the north, Tularcitos Creek on the east, Interstate 680 to the northeast, Calaveras Boulevard to the south, and the Union Pacific Railroad on the west. Berryessa Creek bisects the southern half of the system.

Two collection systems have outfalls that discharge directly to Calera Creek above Escuela Parkway and the North Hillview Drive drain discharges to Tularcitos Creek immediately downstream from Interstate 680. A portion of Calaveras Boulevard drains directly into Wrigley Creek. The remainder of the system drains into Hidden Lake, where the Berryessa Pump Station discharges into Berryessa Creek. Runoff from local collection systems south of Beryessa Creek crosses under the creek in a depressed 60-inch diameter sewer.

Most of this area is located within the mapped 100-year floodplain (Figure 4-1). Overflow from both Calera Creek and Berryessa Creek becomes trapped on the backside of the Berryessa Creek northern levees. Solutions to these problems must come from the Santa Clara Valley Water District.

Gravity Outfalls

All system storm drains discharge to one of the creeks or Hidden Lake through gravity outfalls. Table 5-62 lists the 10- and 100-year starting tailwater elevations at each gravity outfall, using the criteria outlined in Chapter 3. The starting backwater for the tributary system is equivalent to the water surface elevation in the receiving water plus the exit loss at the storm drain pipe outfall.

Table 5-61
Tailwater Elevations at Storm Drain Outfalls within WTCA1 System

ID	Outfall Location	Ground Elev.	SD INV	Outfall Dia	Velocity (fps)		Creek/Lake WSEL (feet NAVD)		System Tailwater (feet NAVD)	
					10yr	100yr	10yr	100yr	10yr	100yr
37	Hidden Lake	13.50	2.73	66	3.19	5.27	9.60	10.70	9.76	11.13
131	Hidden Lake	13.50	2.73	66	3.41	5.35	9.60	10.70	9.78	11.14
158	Calera Creek at Escuela Pkwy	44.50	38.39	30	1.97	3.28	41.78	41.78	41.84	41.95
165	Calera Creek at Founders Ln	60.00	58.05	21	2.66	3.94	58.00	58.00	59.91	60.00
167	Tularcitos Creek at Hillview Dr	28.00	19.33	48	2.39	3.58	25.07	25.77	25.16	25.97
190	Hidden Lake	13.50	2.73	21	2.73	4.22	9.60	10.70	9.72	10.98
1205	Wrigley Creek at Hwy 237	18.50	11.27	33	1.39	2.31	15.02	15.68	15.05	15.76

Collection System Performance

Table 5-63 summarizes the recommended CIP within the WTCA1 system and presents the relevant statistics. Most of the storm drains within this collection system are adequate, although the recommended CIP becomes slightly more intensive in priority to avoid inducing downstream flooding.

Table 5-62
Recommended CIP for Collection System WTCA1

	Lineal Feet	Percentage
System Acceptable / No Improvements	46,533	88
High Priority Improvements	2,000	4
Medium Priority Improvements	1,455	3
Low Priority Improvements	2,730	5
Total System	**52,718**	**100**

Capital Improvements

Table 5-64 and Figure 5-24 identify capital projects to mitigate scattered areas of residual ponding. It is noted that medium priority capital improvements will not be effective in areas subject to flooding caused by Calera Creek overflows. These improvements have been relabeled as low priority and should not be constructed until after the Santa Clara Valley Water District has improved Calera Creek capacity to pass the one-percent discharge.

Table 5-63
Recommended Capital Improvements in System WTCA1

ID	Project	Priority	Parallel Option	Replacement Option
1	North Hillview Drive Relief Drain	High	Install approx 900 LF of 42-inch RCP in Horcajo Street from Tice Drive to North Hillview Drrive and in North Hillview Drive from Horcajo Street to Jacklin Road. Install approx 800 LF of 72-inch RCP in North Hillview Drive from Jacklin Road to Tularcitos Creek in a new parallel outfall. Replace approx 300 LF of existing 12-inch RCP in Jacklin Road from Heather Court cul-de-sac to North Hillview Drive with 24-inch RCP.	Replace approx 260 LF of existing 24-inch RCP with 48-inch RCP in Horcajo Street from Tice Drive to North Hillview Drive. In North Hillview Drive replace approx 640 LF of existing 27-inch RCP with 48-inch RCP; and use 84-inch RCP to replace 90 LF of existing 36-inch RCP and 710 LF of existing 48-inch RCP. Replace approx 300 LF of existing 12-inch RCP in Jacklin Road from the Heather Court cul-de-sac to North Hillview Drive with 24-inch RCP.
2	Glasgow Court Relief Drain	Medium	Install approx 310 LF of 24-inch RCP in Glasgow Ct and approx 455 LF of 24-inch RCP in Dundee Ave from Glasgow Ct to Angus Drive.	Replace approx 310 LF of exiting 21-inch RCP in Glasgow Ct with 30-inch RCP. Replace approx 455 LF of existing 27-inch RCP with 36-inch RCP in Dundee Ave from Glasgow Ct to Angus Drive.

ID	Project	Priority	Parallel Option	Replacement Option
3	Loch Lomond Court Relief Drain	Medium	Install approx 390 LF of 18-inch RCP in Loch Lomond Ct and approx 300 LF of 18-inch RCP in Dundee Ave from Loch Lomond Ct to existing SD easement crossing toward Escuela Parkway.	Replace approx 390 LF of exiting 18-inch RCP in Loch Lomond Ct with 24-inch RCP. Replace approx 300 LF of existing 21-inch RCP with 27-inch RCP in Dundee Ave from Loch Lomand Ct to the existing 27-inch SD in the easement crossing toward Escuela Parkway.
4	Los Pinos Avenue SD Improvement	Low	Replace approx 170 LF of existing 27-inch RCP in the storm drain easement between Los Pinos Ave and Escuela Parkway with 42-inch RCP and install approx 210 LF of 48-inch RCP from the easement to Tramway Drive.	Replace approx 170 LF of existing 27-inch RCP in the storm drain easement between Los Pinos Avenue and Escuela Parkway with 42-inch RCP and replace approx 210 LF of existing 27-inch RCP with 54-inch RCP from the storm drain easement to Tramway Drive.
5	Tramway Drive Relief Drains	Low	Install approx 1,300 LF of 66-inch RCP in Tramway Drive from Singley Drive to North Milpitas Blvd. Install approx 1,050 LF of 24-inch RCP in Tramway Drive from existing SD easement to Escuela Parkway.	In Tramway Drive from Singley Drive to North Milpitas Blvd, replace approx 480 LF of existing 54-inch RCP and 820 LF of existing 60-inch RCP with 84-inch RCP. Replace approx 250 LF of existing 18-inch RCP and approx 260 LF of existing 24-inch RCP with 30-inch RCP in Tramway Drive from existing SD easement to Wyoma Place and approx 540 LF of existing 24-inch RCP in Tramway Drive from Wyoma Place to Escuela Parkway with 36-inch RCP.

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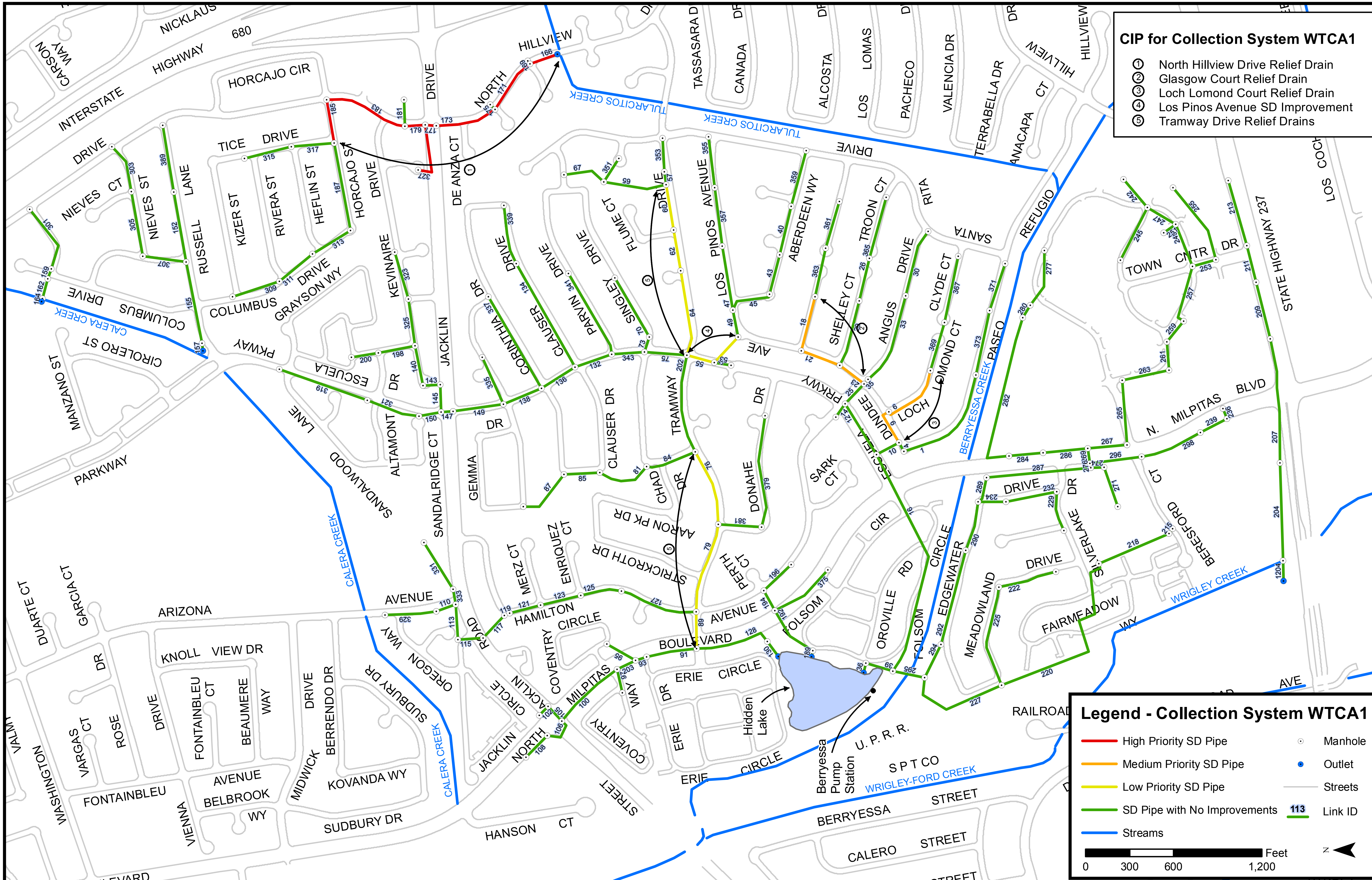

CIP for Collection System WTCA1
1 North Hillview Drive Relief Drain
2 Glasgow Court Relief Drain
3 Loch Lomond Court Relief Drain
4 Los Pinos Avenue SD Improvement
5 Tramway Drive Relief Drains
Legend - Collection System WTCA1
High Priority SD Pipe
Medium Priority SD Pipe
Low Priority SD Pipe
SD Pipe with No Improvements
Streams
Manhole
Outlet
Streets
113 Link ID
Feet
0 300 600 1,200
N
Hidden Lake
Berryessa Pump Station

Figure 5-24

CHAPTER 6
PUMP STATIONS

Each of Milpitas' 13 storm water pumping stations is evaluated based on the set of criteria described herein. Detailed pump station assessment evaluation criteria are presented in the Storm Drain Master Plan Appendix. This chapter describes how well each of the City's pumping facility 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

Storm water pump stations owned and operated by the City of Milpitas must meet, at a minimum, the criteria established herein. If a pump station is going to be 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. A combination of pumping capacity and retention storage may be used to accomplish this. Pump stations with lesser capacity (e.g. 10-year) may be considered only if there is a fail-safe way to overflow excess flows without causing property damage. Nearly all of the pumping facilities within the city meet these criteria. Table 6-1 compares current pump station capacities to the potential 100-year inflow.

Number of Pumps

For redundancy, at least two identical pumps must be installed in every storm water pump station. It is not necessary 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. (Pump maintenance should also be scheduled 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 storm water pumps. Permanent retention ponds are maintained at the California Circle and Abbott stations eliminating the utility of a small dewatering pump, while the Minnis station utilizes submersible pumps capable of nearly completely dewatering the wet well.

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 storm water pump station that does not utilize engines for prime pump drivers. The lack of adequate automatic standby power is considered to be a potentially significant deficiency. When mapping special flood hazards, FEMA will only consider pumping capacity for those pumps with motor drivers that can be started and operated with an automatic standby power generator installed at the station itself. Portable generators and manual power transfer capabilities are not sufficient.

Pump Station Evaluations

Table 6-1 provides a summary of pump station capacities and emergency readiness throughout Milpitas. More detailed evaluations for each station follow where deficiencies are identified and recommended improvements discussed. All engine drive units, where installed, run on diesel fuel. Figure 6-1 shows pump station locations within the city. Available storage is considered when evaluating pump station capacity.

Table 6-1
Pumping Station Summary

ID	Facility	Year Built	Approximate Capacity	Primary Drivers	Standby Power	Description
1	California Circle Pump Station	1983	100-year	Engines	YES	page 6-5
2	Jurgens Pump Station	1989	10-year	Engines	YES	page 6-7
3	McCarthy Pump Station	1994	100-year	Engines	YES	page 6-9
4	Abbott Pump Station	1983	100-year	Motors	NO	page 6-11
5	Minnis Pump Station	1978	10-year	Motors	NO	page 6-13
6	Penitencia Pump Station	1960	100-year	Engines	YES	page 6-15
7	Wrigley-Ford Pump Station	1993	100-year	Engines	YES	page 6-17
8	Berryessa Pump Station	1977	100-year	Engines	YES	page 6-19
9	Manor Pump Station	1993	100-year	Motors	YES	page 6-21
10	Spence Creek Pump Station	1988	100-year	Motors	NO	page 6-23
11	Bellew Pump Station	1985	100-year	Motors/ Engine	YES	page 6-25
12	Murphy Pump Station	1983	100-year	Engines	YES	page 6-27
13	Oak Creek Pump Station	1979	100-year	Engines	YES	page 6-29

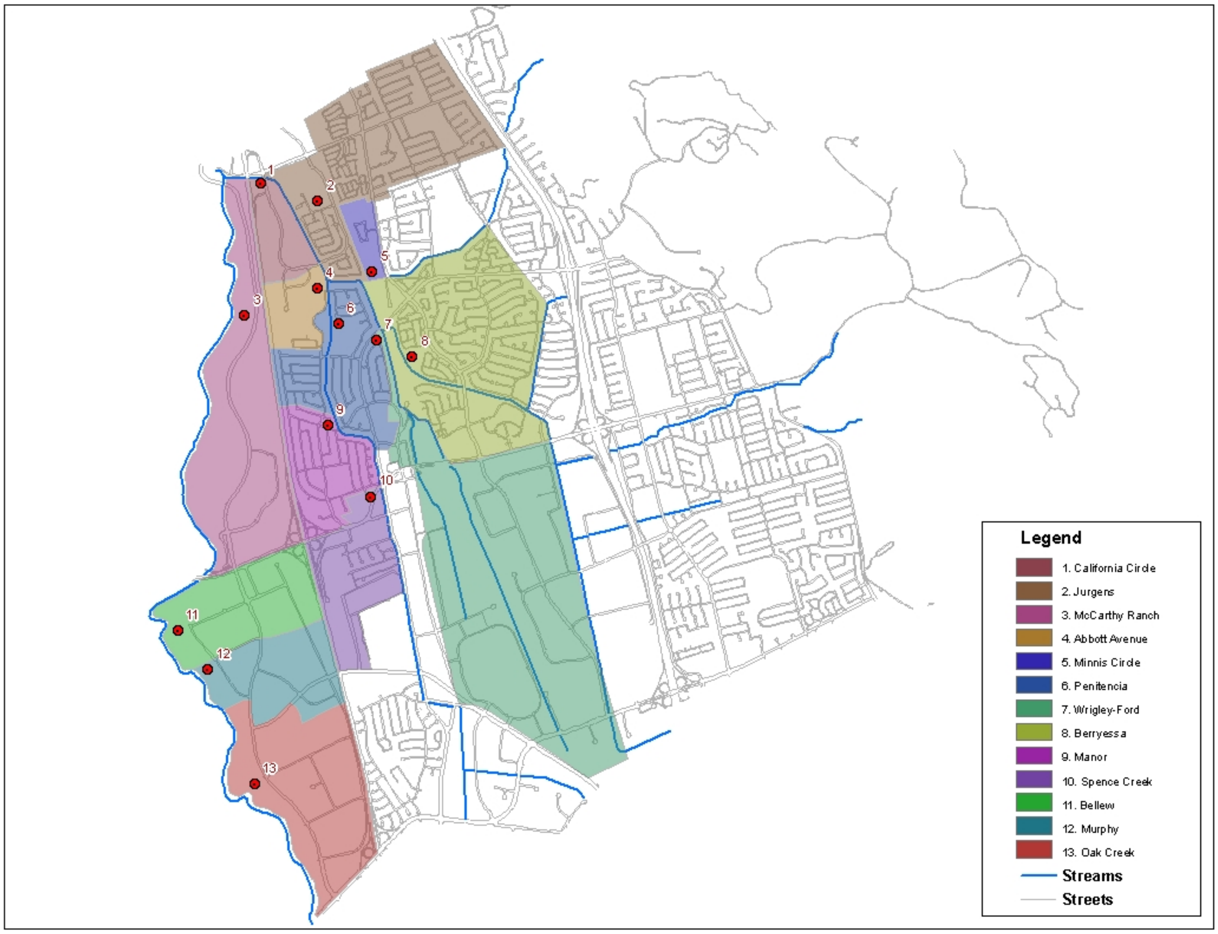

Figure 6-1: Storm Water Pump Stations in Milpitas

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California Circle Pump Station

Facility No:	**SD-1**
Location:	California Circle at Dixon Landing Road
Discharge to:	Lower Penitencia Creek (Sta. 15+00)
1% WSEL:	11.8 feet (NAVD 88), published FIS
Pipe Discharge Elev:	Invert 13.8 feet (NAVD 88)
Storage:	2.5 acre wet pond
Design Lagoon Elev.	9.9 feet (NAVD 88)
Top of Lagoon Bank:	14.0 feet (NAVD 88)
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 at all times; the normal minimum water surface elevation is 4.5 NAVD. Storm water is pumped through three 28-inch in 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. A detailed accounting of tributary area based on new Interstate 880 / Highway 237 freeway interchange plans, however, indicates that 263 acres are potentially tributary to the lagoon as tabulated below.

Table 6-2
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, but runoff in excess of its capacity (20 cfs) flows into the ditch running between Glenmoor Circle and Redwood Avenue, and then into the freeway channel. (See also Chapter 5 beginning on Page 5-77.) 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 6-2.

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 175 HP (2,400 rpm)
Standby Power:	Not required
Fuel Storage:	2,000 gallons; 96 hours at peak load (3 pumps)
Finished Floor:	14.3 feet (NAVD 88)
BFE:	14.8 feet (NAVD 88)

Previously Identified Deficiencies

1. It is noted that the finished floor elevation is six inches below the base flood elevation as currently mapped. The base flood in this location is due to potential spills from Lower Penitencia Creek becoming trapped behind the downstream levee. However, flood hazard mapping efforts for the Valley Transportation Authority's Silicon Valley BART Extension project, underway at the time of Storm Drain Master Plan publication, indicate that this is not an issue.
2. The discharge pipe invert at elevation 13.8 (NAVD) is two feet above the 100-year water surface elevation in Lower Penitencia Creek; 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), but since this situation is beyond the design condition, this deficiency does not require remedial action.

Therefore capital improvements are not proposed for California Circle Pump Station.

California Circle Lagoon Operation

Surcharging storm drains within the California Circle area controls 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 is not drained. 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 6-3.

Table 6-3
California Circle Lagoon Operation

	10-year	100-year
Lowest Adjacent Street Grade (feet NAVD)	12.28	12.28
Maximum Lagoon Stage (feet NAVD)	7.49	9.87
Time of Peak Local Runoff (hours)	10.58	10.58
Lagoon Stage at Peak Local Runoff (feet NAVD)	6.80	8.60

Jurgens Pump Station

Facility No:	**SD-2**
Location:	345 Jurgens Drive
Discharge to:	Lower Penitencia Creek (Sta. 26+50)
Design WSEL:	12.0 feet (NAVD 88)
Storage:	1 ac-ft in City Park
Tributary Area:	433 acres (residential)
Station Capacity:	150 cfs
Required Capacity:	285 cfs for peak pumping
Deficit:	Not applicable due to storage in Dixon Landing Park

Located in Dixon Landing Park, this facility drains mixed residential areas located between Penitencia Creek and Interstate 680 at the northern end of Milpitas. Apparently the system was designed to function in tandem with detention storage available in the park itself, since the pump station is undersized even for a ten-year event (Q_{10} = 190 cfs). 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 in excess of pump 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.

The pump station was subsequently “flood-proofed” by sealing floor openings and raising essential control equipment above the floor so that the equipment does not shut off during a flooding event. As submitted to FEMA in May 2009 with the levee recertification package for Lower Penitencia Creek, water will pond to the following elevations with the current pumps in operation as shown on Figure 6-2.

$WSEL_{10}$ = 10.2 feet NAVD (2 inches above finished floor)

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

At the peak of storm water inflow, the respective ponding elevations are:

$WSEL_{10}$ = 9.5 feet NAVD

$WSEL_{100}$ = 10.8 feet NAVD

Maximum one-percent flood limits are shown along with the area protected by the Lower Penitencia Creek levee (labeled as “Levee Protected Flooding”). 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.

To eliminate the temporary storage of excess runoff within Dixon Landing Park, a new station with a capacity of at least 285 cfs (128,000 gpm) would be required. It is not feasible to retrofit the existing pumping facility to nearly double its capacity. Such a project would entail demolishing the existing facility, building an upsized replacement pump station, and replacing the existing 72-inch diameter discharge pipe to Lower Penitencia Creek with at least a 96-inch diameter discharge pipe. An order of magnitude estimate of construction cost is $10 million. This is not seen as economically justified.

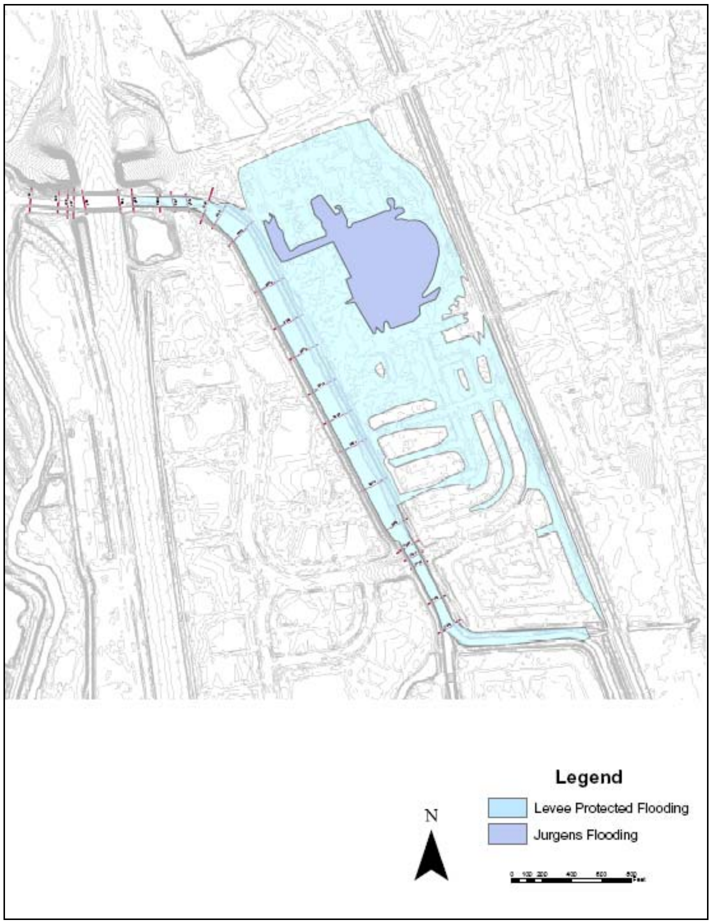

Figure 6-2: Ponding Adjacent to Jurgens Pump Station

Equipment Schedule

Pumps:	(4) Johnston 24PO axial flow rated 16,000 gpm at 10 feet TDH (700 rpm, 60 hp)
	(1) 3,000 gpm 25 hp electric jockey
Prime Power:	(4) Caterpillar 3208 diesel engines rated at 150 hp (2,400 rpm)
	Randolph right angle gear drive (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)
Finished Floor:	10.0 feet (NAVD 88)
BFE:	12.0 feet (NAVD 88)

Station Operation

In response to the February 1998 station shutdown, the City flood-proofed the equipment by sealing access openings in the floor and relocating the controls. Thus the station can continue to operate even with a base flood elevation two feet above the finished floor.

McCarthy Pump Station

Facility No:	**SD-3**
Location:	1005 N McCarthy Boulevard
Discharge to:	Coyote Creek (Sta. 145+00)
Design WSEL:	18.6 feet (NAVD '88)
Storage:	Wet Well
Tributary Area:	185 acres (mixed use)
Station Capacity:	400 cfs
10-year Inflow:	90 cfs (1 of 3 pumps operating)
100-year Inflow:	150 cfs (2 of 3 pumps operating)
Excess Capacity:	250 cfs

Located in the McCarthy Ranch Development, this facility drains mixed-use areas located 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. This is a relatively new facility, and every indication is that the pumping plant is 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)
Finished Floor:	18.5 (NAVD '88)
BFE:	(Shaded Zone X)

Pump Station Operation

Capital improvements are not necessary for the McCarthy Pump Station. To enhance operational efficiencies and minimize pump cycling, however, it is recommended that pump starts rotate so that motors will start no more than five times per hour.

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Abbott Pump Station

Facility No:	**SD-4**
Location:	1225 N Abbott Avenue
Discharge to:	Lower Penitencia Creek (Sta. 46+50)
Design WSEL:	16.9 feet (NAVD 88)
Outfall Invert Elevation:	18.3 feet (NAVD 88)
Storage:	27 ac-feet in lagoon
Tributary Area:	53 acres (park and industrial)
Station Capacity:	24 cfs
Required Capacity:	6 cfs
Excess:	18 cfs
10-year design lagoon level:	9.6 feet NAVD 88
100-year design lagoon level:	10.3 feet NAVD 88

Located on Abbott Avenue, the facility serves as a recreational and aesthetic feature inside an industrial park. As long as the pump station is functioning properly, there is no problem with flooding in the area. However, the prime drivers are electric motors without any provision for standby power. If the power supply to the pump station were to fail during a 24-hour storm, the lagoon could reach the following elevations:

$$WSEL_{10} = 11.9 \text{ feet NAVD}$$

$$WSEL_{100} = 13.7 \text{ feet NAVD}$$

Ponding levels above 12.0 feet NAVD will begin to flood adjacent property, so provisions for standby power should be made to reduce the risk of flooding in extreme events. ***[Medium Priority]***

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:	13.7 feet NAVD
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 the 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, and should water levels ever exceed design freeboard, the situation would exceed design condition. 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 ***Medium Priority*** project associated with the Abbott station. (Note that engine-generator sizing is approximate only, and requires a full load analysis.) To preserve the aesthetic feel of this station, the engine-generator should be housed in a building similar to the pump house. Estimated capital costs include:

125 kW engine-generator set		$ 120,000
Automatic Transfer Switch		40,000
Electrical modifications		50,000
15' x 15' building w/ acoustic treatment	$300/sf	70,000
		$ 280,000
Engineering and Administration (20%)		56,000
Contingency (50%)		164,000
CIP Cost		$ 500,000

Supplemental Recommendation

A style of pump with fewer maintenance requirements might be more appropriate at this pump station. In 2005 a pump specialist recommended 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 a ***Low Priority***.

Repair and replacement of parts for the two existing pumps cost about $35,000. If the impellers are not available "off the shelf" it is likely that the disassembled pump(s) would 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 type of pump would be about $175,000. Replacement of the existing pumps with axial flow submersible pumps requiring less maintenance is about $150,000.

Minnis Pump Station

Facility No:	**SD-5**
Location:	1125 N Milpitas Boulevard
Discharge to:	Calera Creek (Sta. 1+50)
Design WSEL:	15.5 feet (NAVD 88)
Storage:	None
Tributary Area:	30 acres (commercial and industrial)
Station Capacity:	33 cfs
10-year Inflow:	27 cfs (2 of 2 pumps operating)
100-year Inflow:	40 cfs (2 of 2 pumps operating)
Deficit:	7 cfs

Located off of 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, but even if this capacity deficit were to be corrected, the area would still be subject to 100-year flooding from Calera Creek until the Santa Clara Valley Water District solves capacity issues for Calera Creek. Therefore, improving pump station capacity has been downgraded from medium priority to low priority, although when the Minnis station is scheduled for long-term replacement (Chapter 9), pumping capacity should be increased to 100-year as described below.

The station is equipped with submersible electric pumps and motors, with no provision for standby power. Should the power supply to the pump station fail during almost any significant event, runoff becomes trapped behind the Calera Creek floodwall and it would reach the City's corporation yard.

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 pumps on the rail system 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.

Equipment Schedule

Pumps:	(2) Flygt CP 3300 submersible electric rated 4,500 gpm at 45 feet
Standby Power:	none
Control Power:	120 VAC (no backup)
Fuel Storage:	n/a
Finished Slab:	16.7 feet (NAVD 88)
BFE:	15.7 feet (NAVD 88, Zone AH)

Deficiencies

This pump station is not provided with automatic standby power and significant property damage could occur if the pumping facility is not operational as water becomes trapped behind the Calera Creek floodwall. To avoid the need for additional flood fighting at such a critical facility, it is recommended that automatic standby power be added as a ***High Priority***. A battery backup should also be provided for the control systems (at minimal cost) so power outages do not disrupt the pump level settings.

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.

Capital Improvement Recommendations

Providing emergency standby power is a ***High Priority*** project associated with the Minnis station. (Note that engine-generator sizing is approximate only, and requires a full load analysis.) The engine-generator should be sized to start the ultimate low priority replacement motors (to avoid eventual replacement of the generator) and housed in an appropriately sound attenuated weather-tight enclosure. Estimated capital costs include:

200 kW engine-generator set in enclosure	$ 120,000
Automatic Transfer Switch	50,000
Electrical modifications	30,000
	$ 200,000
Engineering and Administration (20%)	40,000
Contingency (50%)	120,000
CIP Cost	$ 360,000

Capital improvements are also required at this pumping facility to match 100-year inflow. Since the recommended improvement would not be effective until the Calera Creek floodplain is eliminated, this capital project remains a ***Low Priority*** until Calera Creek is improved. However, if the pumps and motor controls are replaced as part of scheduled maintenance, they should be upsized at that time for efficiency. Pump station capacity can be increased by replacing the existing submersible pumps and motors with two Flygt model 3356 LT pumps with 150hp motors. The larger pump discharge diameter is 14 inches, which matches the existing configuration so the wet well structure and pump discharge piping and valves do not need to be replaced. Given the larger pumping capacity and discharge velocity, it would be prudent to re-evaluate the discharge structure at Calera Creek. With larger horsepower motors (an upgrade from the existing 70hp motors), new motor starters would be required. Estimated capital costs include:

Remove (2) existing submersible pumps, motors and guide rails	$25,000
Remove existing motor starter and control panel	$15,000
Furnish and install (2) Flygt 3356 pumps and guide rails	$100,000
Furnish and install new pedestal mounted motor control panel	$60,000
Outfall modifications for erosion control	$20,000
	$220,000
Engineering and Administration (20%)	44,000
Contingency (50%)	136,000
CIP Cost	$400,000

Penitencia Pump Station

Facility No:	**SD-6**
Location:	La Honda Drive
Discharge to:	Penitencia Creek (Sta. 57+50)
Design WSEL:	14.7 feet (NAVD 88)
Storage:	Hall Memorial Park Lagoon
Tributary Area:	215 acres (residential)
Station Capacity:	65 cfs
10-year lagoon level:	8.4 feet NAVD
100-year lagoon level	10.1 feet NAVD
Top of lagoon bank:	14 feet NAVD

This 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. Backflow protection from Penitencia Creek is provided by a discharge standpipe that is located above the creek floodwall elevation.

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).

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, which are 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 help 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). The services of a microbiologist could also be retained to 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 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 6-3 shows the storage-elevation curve for the lagoon.

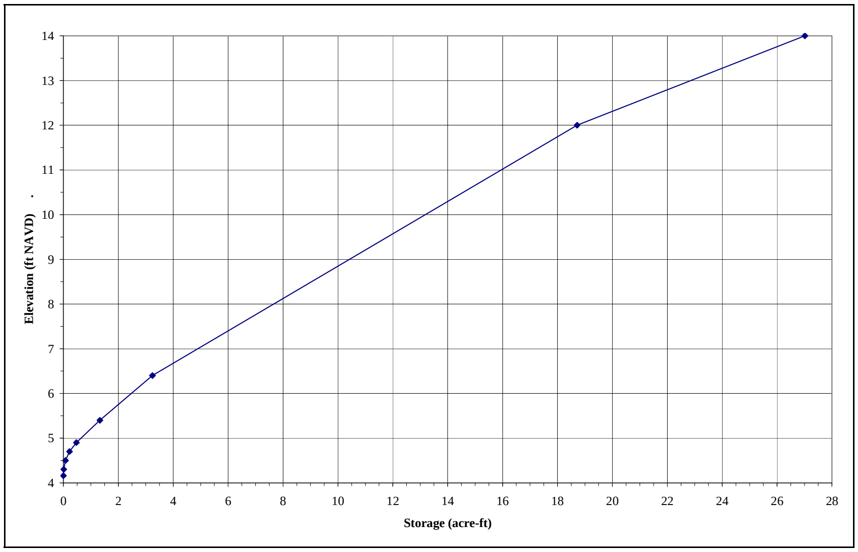

Figure 6-3: Storage Elevation Curve for Hall Memorial Park Lagoon

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
Finished Floor:	14.3 feet (NAVD 88)
BFE:	14.7 feet (NAVD 88)

Capital Improvement Recommendation

Given its age and the condition of the equipment, a complete station replacement is recommended for the Penitencia Pump Station as a ***High Priority***, which would include 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 and 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. It is likely that the new axial flow pumps will be electric motor driven with a standby diesel engine-generator set. Estimated capital costs include:

Demolish existing structure and equipment	$240,000
Furnish and install (3) axial flow pumps	$300,000
Furnish and install new motors and electrical panels	$600,000
New pump station building	$300,000
Standby generator	$300,000
Site and outfall modifications	$200,000
	$1,940,000
Engineering and Administration (20%)	390,000
Contingency (50%)	1,170,000
CIP Cost	$3,500,000

Wrigley-Ford Pump Station

Facility No:	**SD-7**
Location:	Levee access from Marylinn Dr
Discharge to:	Berryessa Creek (Sta. 24+00)
Design WSEL:	17.5 feet (NAVD 88)
Storage:	Forebay and channel storage
Tributary Area:	760 acres (commercial and industrial)
Station Capacity:	432 cfs
Required Capacity:	400 cfs
Excess:	32 cfs
10-year design WSEL:	12.7 feet NAVD
100-year design WSEL:	13.2 feet NAVD

The downstream reach of Wrigley-Ford Creek was created when the Santa Clara Valley Water district 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. Unfortunately, high flows in Wrigley-Ford Creek 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 the District built the Wrigley-Ford Pump Station to pump tributary creek flows into Berryessa Creek, thereby 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 is 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 drives
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.
Finished Floor:	20.7 feet (NAVD 88)
BFE:	13.2 feet (NAVD 88)

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 rotate on a regular basis, allowing all three pumps to be alternated for lesser storm events, and both forebay and channel storage are used to prevent cycling.

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Berryessa Pump Station

Facility No:	**SD-8**
Location:	Folsom Circle
Discharge to:	Berryessa Creek (Sta. 48+75)
Design WSEL:	18.8 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
Normal lake level:	9.0 feet NAVD
10-year lake level:	9.6 feet NAVD
100-year lake level:	10.7 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 Town Center. Current operating practice is to use this lake as an aesthetic amenity throughout the year. Local 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 6-4). 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 the installation of replacement equipment and the elevations of all controls to the flood-proofed elevation of 16.78 feet NAVD. 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 back up 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 despite the building itself being flooded. Recent analyses indicates that with the pump station remaining in operation during a spill event from Calera Creek, the one-percent base flood elevation is 15 feet NAVD, or nearly two feet below the flood-proofed elevation.

Occasional problems with odors during low lake levels have been resolved using aerators.

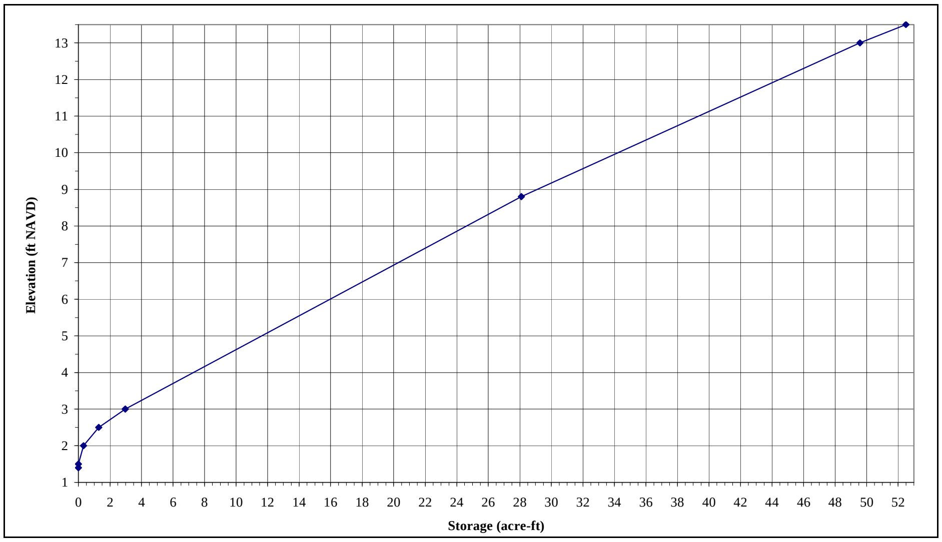

Figure 6-4: Storage Elevation Curve for Hidden Lake

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 El:	16.8 feet (NAVD 88)
BFE:	15.1 feet (NAVD 88)

No Identified Deficiencies

There are no identified pump station deficiencies.

Manor Pump Station

Facility No:	**SD-9**
Location:	Marylinn Ave. and Barker St.
Discharge to:	Lower Penitencia Creek (Sta. 90+00)
Design WSEL:	17.4 feet (NAVD '88)
Storage:	Wet Well Only
Tributary Area:	146 acres (residential and commercial)
Station Capacity:	95 cfs
Required Capacity:	90 cfs
Excess Capacity:	5 cfs
10-year design WSEL:	6.2 feet NAVD
100-year design WSEL:	6.7 feet NAVD

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 creek stage is high.

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:	18.2 feet (NAVD 88)
BFE:	14.7 feet (NAVD 88)

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 100-year inflow.

No Identified Deficiencies

There are no identified deficiencies requiring capital improvements at Manor Pump Station.

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Spence Creek Pump Station

Facility No:	**SD-10**
Location:	11 Butler Street.
Discharge to:	Penitencia Creek (Sta.110+00)
Design WSEL:	17.7 feet (NAVD 88)
Storage:	Wetwell Only
Tributary Area:	109 acres (res. and commercial)
Station Capacity:	94 cfs
Required Capacity:	90 cfs
Excess:	4 cfs
10-year design WSEL:	12.7 feet NAVD
100-year design WSEL:	14.7 feet NAVD

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 Penitencia Creek through 600 feet of 42" diameter RCP force main.

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:	18.2 feet (NAVD 88)
BFE:	14.7 feet (NAVD 88)

Deficiency

While a plug and manual transfer switch is 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 should be retrofitted 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. Also, the current bubbler level sensor needs replacement. ***[High Priority]***

Capital Improvement Recommendation

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

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
	$ 420,000
Engineering and Administration (20%)	80,000
Contingency (50%)	250,000
CIP Cost	$ 750,000

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Bellew Pump Station

Facility No:	**SD-11**
Location:	481 Murphy Ranch Road
Discharge to:	Coyote Creek (Sat. 616+00)
Design WSEL:	32.7 feet (NAVD 88)
Storage:	Wet well only
Tributary Area:	270 acres (industrial)
Station Capacity:	375 cfs
10-year Inflow:	145 cfs (2 of 3 pumps operating)
100-year Inflow:	205 cfs (2 of 3 pumps operating)
Excess Capacity:	170 cfs

Located at the end of Bellew Drive in 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.

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
	(1) Caterpillar 3412 diesel engine 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:	2,500 gallons; 72 hours at peak load (3 pumps)
Finished Floor:	25.2 feet (NAVD 88)
BFE:	n/a (Shaded Zone X)

No Identified Deficiencies

Capital improvements are not necessary for the Bellew Pump Station.

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Murphy Pump Station

Facility No:	**SD-12**
Location:	801 Murphy Ranch Road
Discharge to:	Coyote Creek (Sta. 636+00)
Design WSEL:	34.0 feet (NAVD 88)
Storage:	Wet well only
Tributary Area:	130 acres (industrial)
Station Capacity:	200 cfs
10-year Inflow:	65 cfs (1 of 3 pumps operating)
100-year Inflow:	110 cfs (2 of 3 pumps operating)
Excess Capacity:	90 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.

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)
Finished Floor:	27.7 (NAVD '88)
BFE:	n/a (Shaded Zone X)

No Identified Deficiencies

Capital improvements are not necessary for the Murphy Pump Station.

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Oak Creek Pump Station

Facility No:	**SD-13**
Location:	1521 McCarthy Boulevard
Discharge to:	Coyote Creek (Sat. 678+00)
Design WSEL:	38.3 feet (NAVD 88)
Storage:	Wet well and Pipe
Tributary Area:	280 acres (industrial)
Station Capacity:	320 cfs
10-year Inflow:	190 cfs (2 of 3 pumps operating)
100-year Inflow:	290 cfs (3 of 3 pumps operating)
Excess Capacity:	30 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 Coyote Creek stage is high, they tend to run warm.

Equipment Schedule

Pumps:	(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
Control Power:	120 VAC backed up by 24 VDC batteries with charger.
Fuel Storage:	2,000 gallons; 80 hours at peak load (3 pumps)
Finished Floor:	33.7 feet (NAVD 88)
BFE:	n/a (Shaded Zone X)

No Identified Deficiencies

Capital improvements are not necessary for the Oak Creek Pump Station.

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CHAPTER 7
STORM DRAIN MASTER PLAN IMPACTS

This chapter discusses the ramifications that continued development within Milpitas may have on Storm Drain Master Plan recommendations, and the impact that CIP implementation may have on Milpitas, including major drainage facilities.

Development Impacts

Recommendations made in Chapters 5 and 6, and the Capital Improvement Program proposed in Chapter 8; are all based on ***full planned development*** within Milpitas. Figure 7-1 shows a generalized version of the city's most current zoning map. Land use categories have been combined into the following categories, to which runoff coefficients are assigned (see Chapter 2).

- Agricultural (A)
- Single Family Residential (R1)
- Single Family Hillside (R1-H)
- 1 or 2 Family (R2)
- Multi Family (R3)
- Commercial
- Industrial
- Mixed Use

The Master Plan is based on ultimate build-out within Milpitas' boundaries according to the 2012 land use plan reflected in Figure 7-1. This approach has been taken because Milpitas has, for the most part, developed a significant portion of its available land. Less than 10 percent of developable parcels are still available for new development and the remaining vacant developable land is scattered throughout the city as fairly small parcels.

As a result, the master plan proposes improvements necessary to achieve desired storm drain performance goals as if the city were fully developed. Major developments or re-developments that are more intense (e.g. have a higher runoff coefficient) than the general land use zones shown in Figure 7-1, will need to be evaluated for their potential impact to Milpitas' storm drainage systems and Capital Improvement Program on a development-by-development basis using the GIS-based model described herein.

Recently, both the Transit Area Specific Plan and Midtown Specific Plan developments have been analyzed to ascertain their impacts on the storm drain CIP.

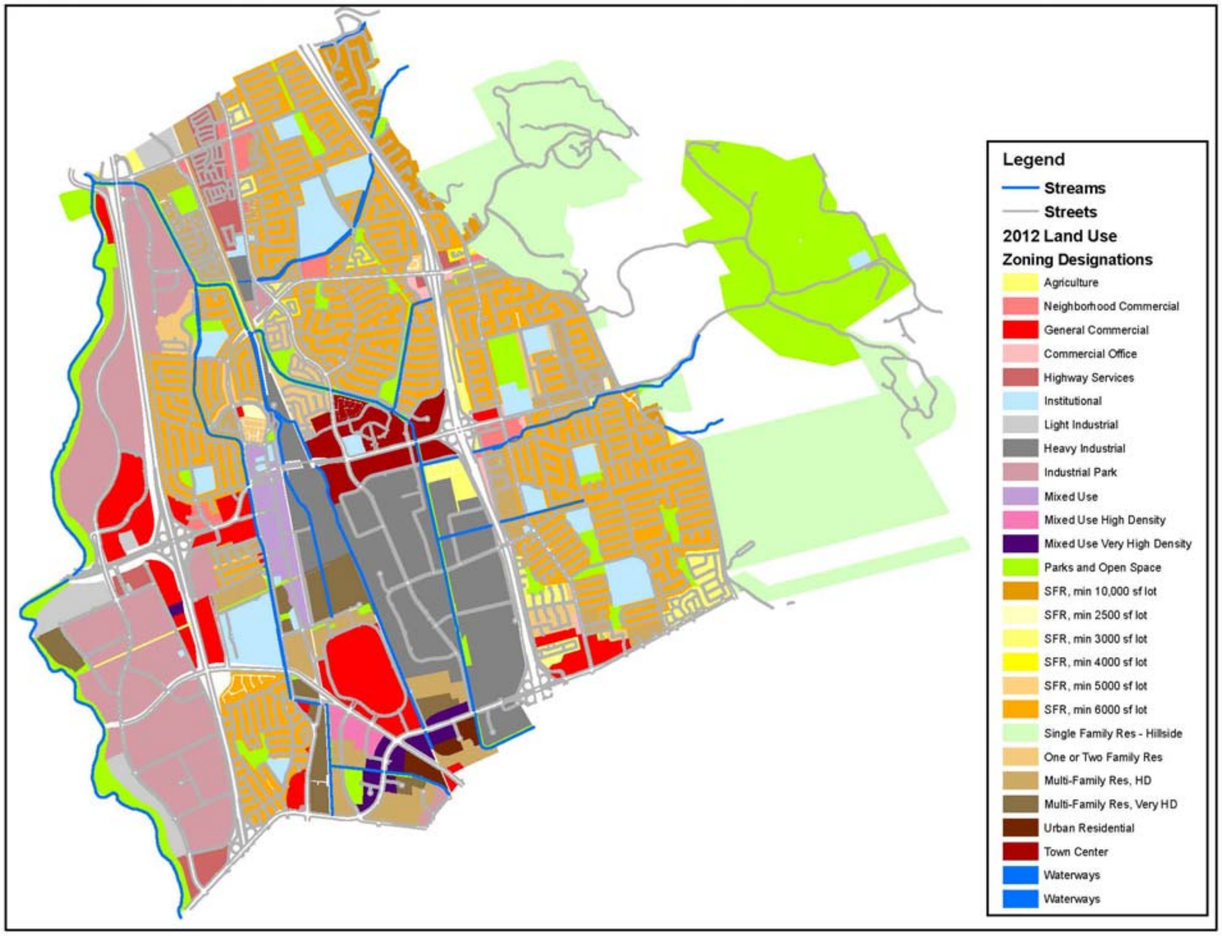

Figure 7-1: Land Use Zoning Designations in Milpitas

Transit Area Specific Plan

Proposed higher density land uses within the Milpitas Transit Area Specific Plan (TASP; Figure 7-2) have been incorporated into the storm drain master plan. Runoff coefficients for the mixture of uses including high density mixed use, very high density transit-oriented residential development, and transit-oriented retail development are not substantially different than the current commercial and industrial uses, and with additional green spaces, storm water runoff from the entire specific plan area is actually reduced. The CIP proposes only low priority improvements within the TASP.

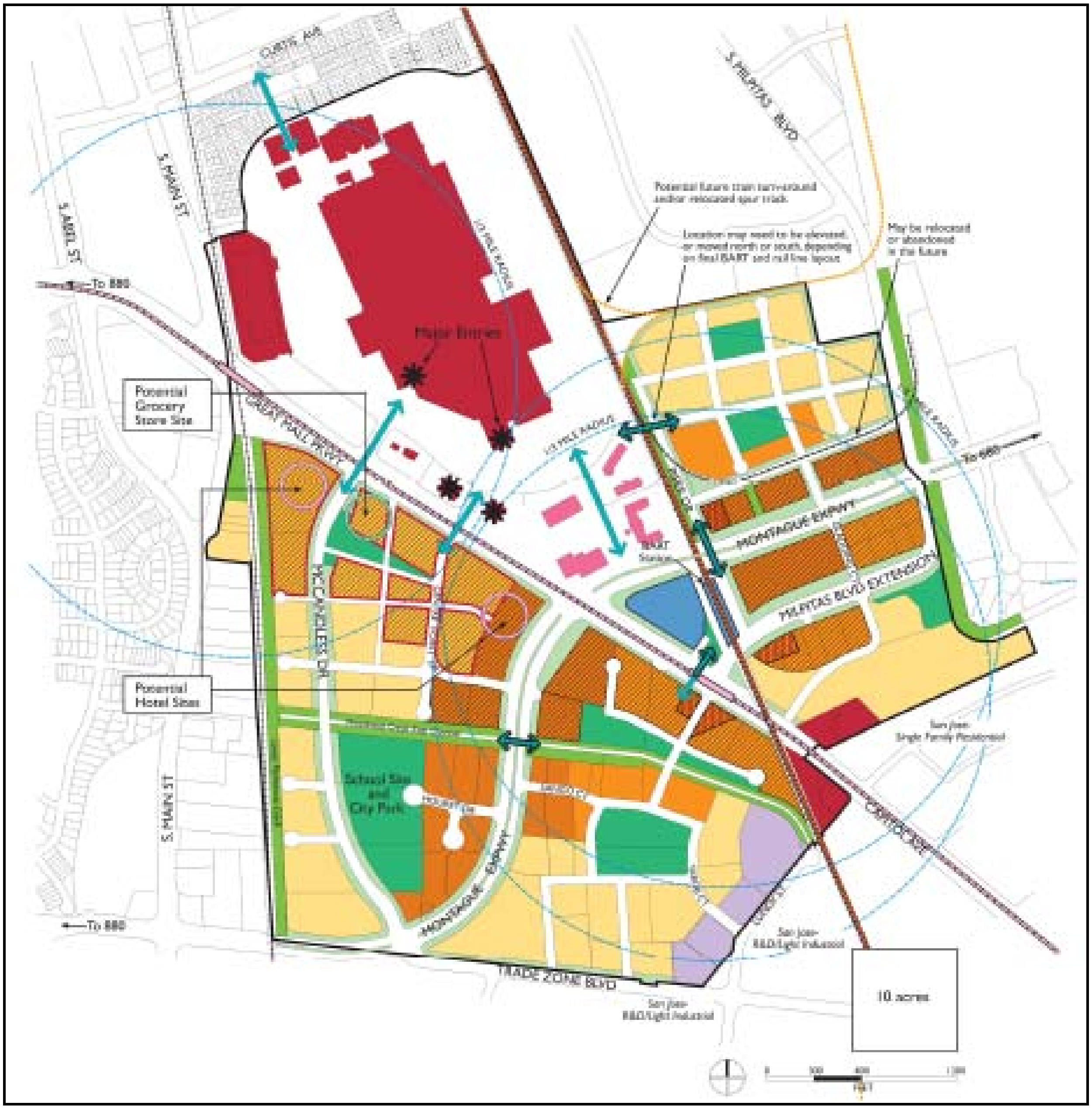

Figure 7-2: Milpitas Transit Area Specific Plan

Midtown Specific Plan

The high density land uses shown within the Milpitas Midtown Specific Plan (Figure 7-3) have also been incorporated into the 2012 land use plan and storm drain master planning. Similarly, runoff coefficients for the mixture of high density residential, commercial, industrial, and transit-oriented development are not higher than the previous commercial and industrial uses. In fact, much of the specific plan area was previously occupied by asphalt concrete parking lots. The CIP proposes only low priority improvements and one medium priority improvement within the Midtown Specific Plan area.

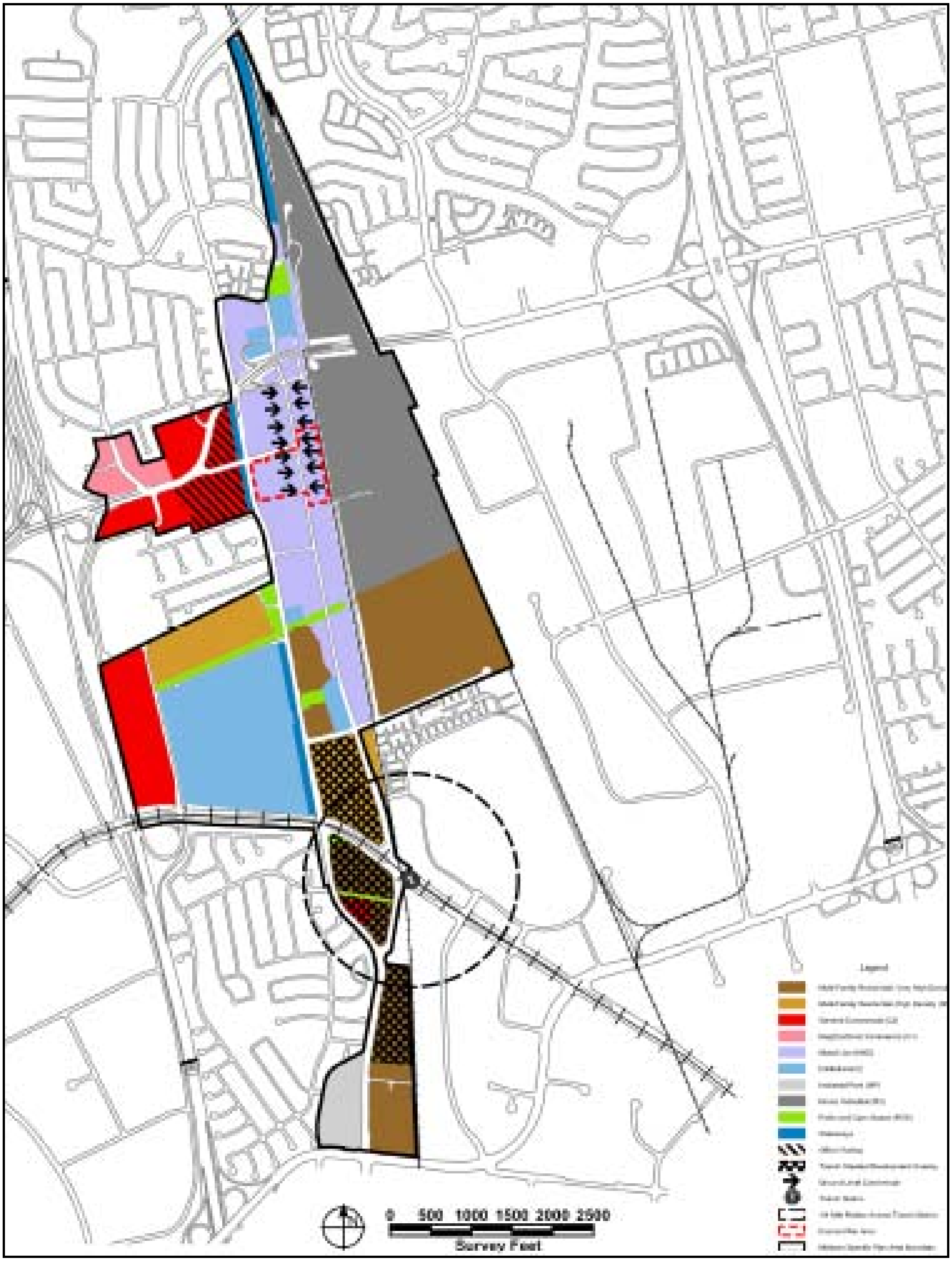

Figure 7-3: Milpitas Midtown Specific Plan

Capital Improvement Program Impacts

The most prominent impact of implementing the storm drain CIP is to improve drainage conditions within areas of Milpitas with identified deficiencies, based upon the performance criteria outlined in Chapter 3. With full implementation of the CIP in conjunction with improvements to the system of major drainages within the city (by others), Milpitas should be free of regulatory flood hazards.

Changing the interior storm drainage system could, however, potentially impact the conveyance of water through the major drainage system. The evaluation of such potential impacts herein focuses on the completion of the high priority CIP projects, which are scheduled for the near-term.

Medium priority CIP projects are time-indefinite and there is a substantial risk that any analysis of potential impacts will no longer be valid when such CIP projects commence.

Low priority CIP projects that fall into that priority rating because they are located within existing special flood hazard areas would not be constructed, if at all, until at least the associated drainage system improvements are completed by other agencies. Consequently it is necessary that those major drainage system improvement projects need to accommodate any storm drain system improvement impacts. Low priority projects that are not located within existing special flood hazard areas correct deficiencies where excess 100-year discharge is contained within the street right-of-way, but excess 10-year discharge is carried above the top of curb. In such cases, the deficiency correction does not substantially change the discharge of storm water runoff into receiving waters, and so has no potential significant impact.

Drainage Impacts of High Priority CIP Projects

Table 7-1 lists the high priority capital improvement projects identified in Chapter 5. Potentially impacted outfall locations due to CIP improvements are provided and changes to the 10-year and 100-year discharges and discharge velocities at each outfall location are given. Outfall locations with potential impacts are highlighted.

Table 7-1
Impact of High Priority CIP Projects on Major Drainage Facilities

ID	Project	Impacted Outfall Location	10-year Discharge (cfs)		100-year Discharge (cfs)	
			Existing	CIP	Existing	CIP
BT1.3	Park View Drive SD Replacement	Tularcitos Ck at I-680	210	210	303	303
C1.1	Sycamore Drive SD Improvements	Oak Creek Pump Station*				
L2.1	Dempsey Road SD Relief	Los Coches Creek at South Park Victoria Dr	16	0	16	0
L2.2	Edsel Drive SD Improvements	Los Coches Creek at Dempsey Road	31	51	45	80
P4.1	Silvera Street SD Replacement	Manor Pump Station*				
PB1.1	Redwood Ave. Relief Drain	Penitencia Pump Station*				
PB1.2	Abbott Ave. Relief Drain	Penitencia Pump Station*				
PB1.3	Maple Ave. Relief Drain	Penitencia Pump Station*				
PB1.4	Chestnut Ave. Relief Drain	Penitencia Pump Station*				

ID	Project	Impacted Outfall Location	10-year Discharge (cfs)		100-year Discharge (cfs)	
			Existing	CIP	Existing	CIP
PB1.5	Heath Street Relief Drain	Penitencia Pump Station*				
PB1.6	North Abel St Relief Drain	Penitencia Pump Station*				
PB1.7	Vasona St SD Improvement	Penitencia Pump Station*				
PB1.9	Lexington St SD Improvements	Penitencia Pump Station*				
PB1.10	Coyote St Relief Drain	Penitencia Pump Station*				
PDB1.1	Wrigley Way SD Replacement	Berryessa Creek near Piedmont Creek	32	31	47	46
T1.1	Jacklin Road Relief Drain	Tularcitos Ck at I-680	76	74	125	122
WTCA1.1	North Hillview Dr Relief Drain	Tularcitos Ck at Hillview Drive	30	24	45	38

*Discharge to pump station with no proposed increase in capacity; therefore no impact to receiving waters.

Potential impacts from high priority projects may occur to Los Coches Creek at Dempsey Road.

Los Coches Creek Impacts

To reduce the impact of 10-year street flooding, the high priority CIP will re-route storm water runoff that currently discharges from the north bank at South Park Victoria Drive to a new outfall on the north bank of Los Coches Creek at the downstream face of the Dempsey Road crossing where, despite 100-year overbanking, the creek elevation is lower. Adverse street grade prevents the discharge of storm water runoff from the south bank at South Park Victoria Drive during high creek flows. As part of the high priority CIP, flows in this storm drain system will be diverted at Edsel Drive and redirected into a new outfall on the south bank of Los Coches Creek at Dempsey Road, also at a lower discharge elevation.

The effective FIS shows that the 100-year discharge in Los Coches Creek is not contained within the creek banks between South Park Victoria Drive and Interstate 680. A special flood hazard area stretches from Dempsey Road to Selwyn Drive (Figure 4-1). Yet despite this flood hazard area, the impact of reconfiguring storm drain outfalls is to allow about 20 cfs of additional discharge to the creek at the conservatively assumed coincident peak of the 100-year flow event.

The FIS hydraulic model for Los Coches Creek has been used to evaluate the change in creek stage resulting from a decrease of 16 cfs in creek discharge between South Park Victoria Drive and an increase in discharge of 35 cfs between Dempsey Road and Interstate 680. Table 7-2 summarizes changes in base flood elevations at selected locations that result from this change in coincident storm drain discharge.

Table 7-2
Los Coches Creek Impacts

Location	Base Flood Elevation (feet NAVD)		Difference (foot)
	Effective FIS	After High Priority Storm Drain CIP	
Upstream Confluence with Berryessa Creek	34.01	34.04	0.03
Downstream Face of I-680 Culvert	38.21	38.27	0.06
Upstream Face of I-680 Culvert	42.33	42.39	0.06
Downstream Face of Dempsey Road Culvert	47.34	47.40	0.06
Upstream Face of Dempsey Road Culvert	51.04	50.98	(0.06)
Downstream Face of South Park Victoria Drive Culvert	56.03	55.95	(0.08)
Upstream Face of South Park Victoria Drive Culvert	56.14	56.05	(0.09)

The change in 100-year water surface elevation in Los Coches Creek resulting from high priority CIP storm drain project discharge modifications is less than 0.1 foot. Previous CEQA work within Milpitas and Santa Clara County has established an impact of 0.1 foot to be less than significant.

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CHAPTER 8
CAPITAL IMPROVEMENT PROGRAM

A proposed long-range capital improvement program (CIP) is laid out in this chapter by priority, according to recommendations made in Chapters 5 and 6. Chapter 10 provides costs that are totaled by drainage system and by priority.

High Priority projects are those necessary to protect property that could be endangered during a 10-year (or less) magnitude event, and are shown city-wide in Figure 8-1. Capital improvements required to mitigate less frequent (100-year) flooding that could lead to property damage are categorized as ***Medium Priority***, which should be undertaken after the completion of identified high priority projects. Improvements that remedy residual flooding not posing a risk to life or property are strictly optional, or ***Low Priority***. These portions of the CIP could be completed as funding becomes available, either through additional local development or as ancillary projects to street or other utility redevelopment. Low priority projects in particular may also fall into the category of responding to citizen complaints. The City may also modify priority levels to reflect field experience and funding realities. Tables 8-1 through 8-3 summarize the proposed Capital Improvement Program by priority and drainage system.

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 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 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 likely that the Storm Drain Capital Improvement Program for Milpitas will more often utilize parallel relief drains, unless right-of-way and utility constraints appear to favor the actual replacement of pipe.

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 to be removed. Given the 50 percent contingency applied to unit cost estimates, no differentiation is made between the cost of pipe replacement and parallel drain installation in the Capital Improvement Program. (That is, the cost of existing pipe removal is included in the large contingency.)

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

Table 8-1
High Priority Capital Improvement Plan

ID	Name	Location	From	To	Parallel Option Size (in)	Parallel Option Lineal Feet	Replacement Option Size (in)	Replacement Option Lineal Feet
(1)	Traughber Street Storm Drain Replacement (BT1.1)	SD easement	Wool Drive	Tularcitos Creek Outfall	72	300	72	300
(2)	Wool Drive Storm Drain Improvement (BT1.2)	Wool Drive	Kennedy Drive	SD easement	42	1,210	48	1,210
(3)	Park View Drive Storm Drain Improvement (BT1.3)	SD easement	Park View Drive	Kennedy Drive	42	175	42	175
		Kennedy Drive	SD easement	Wool Drive	42	250	42	250
(4)	Sycamore Drive Storm Drain Improvements (C1.1)	Sycamore Drive	Barber Lane	Buckeye Drive	42	1,270	48	1,270
(5)	Minnis Pump Station Standby Power (CA2.1)	Minnis Pump Station						
(6)	Dempsey Road Storm Drain Relief (L2.1)	Dempsey Road	SD easement	Los Coches Ck	36	1,100	36	1,100
(7)	Edsel Drive Storm Drain Improvements (L2.2)	Edsel Drive	S Park Victoria Dr	Dempsey Road	36	730	36	730
		Dempsey Road	Edsel Drive	Selwyn Drive	42	1,200	48	1,200
		SD easement	Selwyn Drive	Los Coches Ck	48	200	48	200
(8)	Spence Creek Pump Station Standby Power (P3.1)	Spence Creek Pump Station						
(9)	Silvera Street Storm Drain Replacement (P4.1)	SD easement	Silvera Street	Existing Storm Drain	27	140	27	140
(10)	Redwood Avenue Relief Drain (PB1.1)	Redwood Avenue	Heath Street	Abbott Avenue	24	1,300	30	1,140
							36	160
(11)	Abbott Avenue Relief Drain (PB1.2)	Abbott Avenue	Walnut Drive	Redwood Ave.	36	1,425	42	1,425
(12)	Maple Avenue Relief Drain (PB1.3)	Maple Avenue	SD easement	Abbott Avenue	18	390	24	220
(13)	Chestnut Avenue Relief Drain (PB1.4)	Chestnut Avenue	Heath Street	Abbott Avenue	36	1,060	42	1,060
(14)	Heath Street Relief Drain (PB1.5)	Heath Street	Elm Avenue	Chestnut Avenue	36	520	42	520
(15)	North Abel Street Relief Drain (PB1.6)	North Abel Street	Penitencia Street	SD easement	48	2,530	48	2,530
(16)	Vasona Street Storm Drain Improvements (PB1.7)	Vasona Street	Almaden Avenue	Marylinn Drive	24	290	30	290
		SD easement	Vasona Street	North Abel Street	48	240	48	240
		Vasona St cul-de-sac	as shown	North Abel Street	42	200	42	200
(17)	Penitencia Pump Station Replacement (PB1.8)	Penitencia Pump Station						
(18)	Lexington Street Storm Drain Improvements (PB1.9)	SD easement	Penitencia Street	Lexington Street	36	220	36	220
			Lexington Street	Coyote Street	42	260	42	260
			Coyote Street	North Abel Street	48	290	48	290
(19)	Coyote Street Relief Line (PB1.10)	Coyote Street	as shown	Uvas Avenue	36	510	42	510
			Uvas Avenue	SD easement	36	240	42	240
(20)	Wrigley Way Storm Drain Replacement (PDB1.1)	SD easement	Wrigley Way	Berryessa Creek	36	370	36	370
(21)	Jacklin Road Relief Drain (T1.1)	Jacklin Road	SD easement	I-680 Channel	72	300	84	300
(22)	North Hillview Drive Relief Drain (WTCA1.1)	Horcajo Street	Tice Drive	North Hillview Dr	42	260	48	260
		North Hillview Drive	Horcajo Street	Jacklin Drive	42	640	48	640
			Jacklin Drive	Tularcitos Creek	72	800	84	800
		Jacklin Road	Heather Court	North Hillview Dr	24	300	24	300

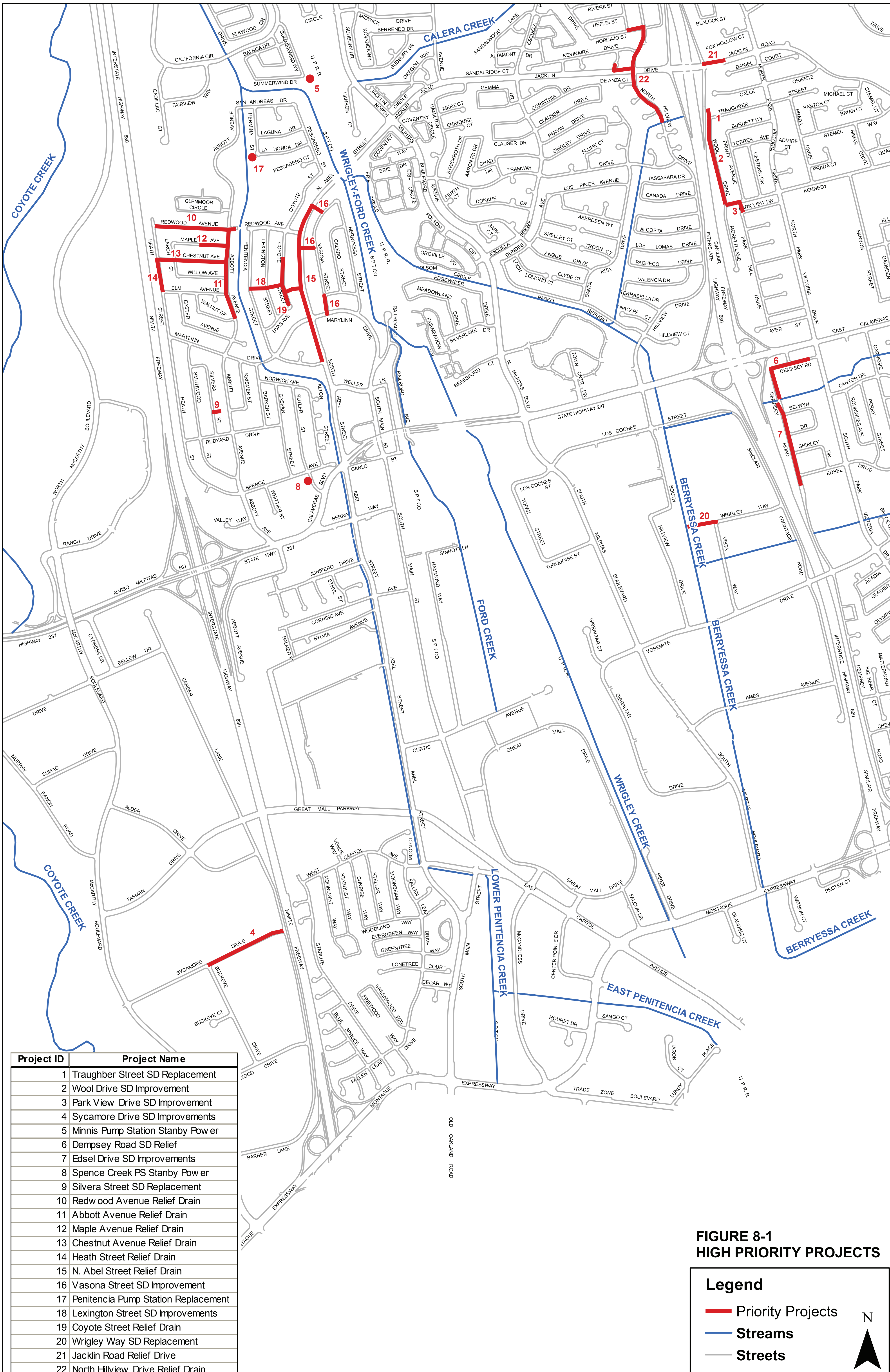

FIGURE 8-1
HIGH PRIORITY PROJECTS

Project ID	Project Name
1	Traughber Street SD Replacement
2	Wool Drive SD Improvement
3	Park View Drive SD Improvement
4	Sycamore Drive SD Improvements
5	Minnis Pump Station Stanby Power
6	Dempsey Road SD Relief
7	Edsel Drive SD Improvements
8	Spence Creek PS Stanby Power
9	Silvera Street SD Replacement
10	Redwood Avenue Relief Drain
11	Abbott Avenue Relief Drain
12	Maple Avenue Relief Drain
13	Chestnut Avenue Relief Drain
14	Heath Street Relief Drain
15	N. Abel Street Relief Drain
16	Vasona Street SD Improvement
17	Penitencia Pump Station Replacement
18	Lexington Street SD Improvements
19	Coyote Street Relief Drain
20	Wrigley Way SD Replacement
21	Jacklin Road Relief Drive
22	North Hillview Drive Relief Drain

Table 8-2
Medium Priority Capital Improvement Plan

ID	Name	Location	From	To	Parallel Option		Replacement Option	
					Size (in)	Lineal Feet	Size (in)	Lineal Feet
BT1.4	Tramway Drive Storm Drain Improvement	Tramway Drive	N Hillview Drive	Tularcitos Creek	18	530	24	530
BT1.5	Calaveras Road Outfall Relocation	Calaveras Road and Temple Drive	Temple Drive	Los Coches Creek	36	800	36	800
BT1.6	Fanyon Street Storm Drain Improvement	Fanyon Street	Dennis Avenue	Kennedy Drive	24	1,150	36	1,150
BT1.7	Temple Drive Storm Drain Improvement	Temple Drive	Fair Hill Drive	Kennedy Drive	24	205	36	205
		Kennedy Drive	Temple Drive	Fanyon Street	24	1,230	36	1,230
BT1.8	Calaveras Ridge Drive Storm Drain Improvement	Calaveras Ridge Drive	as shown	as shown	18	315	24	315
BT1.10	Debris Basins and Inlet Mods	varies						
C1.2	Buckeye Court Storm Drain Replacement	SD easement	Barber Court	Sycamore Drive	36	1,125	36	1,125
C1.3	Cottonwood Drive Storm Drain Improvements	Barber Lane	as shown	Cottonwood Dr	24	550	30	550
		Cottonwood Drive	Barber Lane	Buckeye Drive	24	850	36	280
							42	570
C1.4	Barber Lane Storm Drain Improvements	Barber Lane	as shown	McCarthy Blvd	36	780	48	780
C1.5	McCarthy Boulevard Storm Drain Improvements	McCarthy Boulevard	as shown	Barber Lane	36	490	42	490
C3.1	Murphy Ranch Road Storm Drain Improvement	Murphy Ranch Road	Sumac Drive	Bellew Drive	36	1,160	48	190
							54	420
							60	550
C3.2	Sumac Drive Storm Drain Improvement	Sumac Drive	as shown	Murphy Ranch Road	36	450	48	450
CA2.2	North Milpitas Boulevard Storm Drain Relief	North Milpitas Blvd	as shown	Calera Creek	42	100	54	100
L2.3	Carnegie Drive Storm Drain Improvements	Carnegie Drive	Mercury Court	Ashland Drive	30	740	36	740
			Ashland Drive	Canton Drive	30	340	42	340
		Canton Drive	Carnegie Drive	Los Coches Ck	30	160	42	160
L2.4	Roswell/Canton Storm Drain Improvements	Roswell Drive	Roswell Court	Canton Drive	30	1,070	36	250
							42	820
		Canton Drive	Roswell Drive	Carnegie Drive	30	1,060	42	1,060
P2.1	South Main Street Storm Drain Improvements at Cedar Way	South Main Street	as shown	North of Cedar Way	24	660	36	1,100
P3.2	Carlo Street Relief Drain	Carlo Street	South Main Street	Lower Penitencia Creek	24	780	36	780
P5.1	Abbott PS Improvements	Abbott Pump Station						
P6.1	Arizona Avenue Relief Drain	Arizona Avenue	Dixon Road	Coelho Street	30	1,320	48	1,320
P6.2	Wilson Way Storm Drain Improvements	Wilson Way	as shown	Dixon Landing Road	18	180	30	180
					48	960	48	960
P6.3	Summerwind Way Relief Drain	Summerwind Way	Balboa Drive	Milmont Drive	36	360	48	360
P6.4	Milmont Drive Relief Drain	Milmont Drive	Aspenridge Drive	Jergens Drive	48	480	54	480
P6.5	Jergens Drive Relief Drain	Jergens Drive	UPRR	Jergens PS	54	500	84	500
P6.6	Connect Twin RCPs at SVBX	Jurgens Drive	UPRR	Milmont Drive				
PD1.1	Vista Way Relief Drain	Vista Way	Yosemite Drive	Piedmont Creek	36	260	48	260
PD1.2	Falcato Drive Relief Drain	Falcato Drive	Frank Court	Sepulveda Drive	24	310	30	310
PD1.5	Debris Basins and Inlet Mods	varies						
PDB1.2	Watson Court Relief Drain	Watson Court	as shown	Montague Expwy	18	310	30	310
		Montague Expwy	as shown	Berryessa Creek	24	370	36	370
T1.2	Calaveras Ridge Dr SD Outfall	Calaveras Ridge Dr	Country Club Dr	Adjacent ravine	24	150	24	150
T1.3	Inlet Modification	Calaveras Ridge Dr						
WTCA1.4	Glasgow Court Relief Drain	Glasgow Court	as shown	as shown	24	310	30	310
		Dundee Avenue	Glasgow Court	Angus Drive	24	455	36	455
WTCA1.5	Loch Lomond Court Relief Drain	Loch Lomond Court	as shown	as shown	18	390	24	390
		Dundee Avenue	Loch Lomond Ct	SD easement	18	300	27	300

Table 8-3
Low Priority Capital Improvement Plan

ID	Name	Location	From	To	Parallel Option		Replacement Option	
					Size (in)	Lineal Feet	Size (in)	Lineal Feet
BT1.9	Park Hill Drive Storm Drain Improvement	Park Hill Drive	Park Grove Dr	Park Heights Dr	24	820	30	820
			Park Heights Dr	Park View Dr	30	810	36	810
CA2.3	Minnis Circle Storm Drain Replacement	Minnis Circle	as shown	Minnis Pump Sta	48	140	48	140
			along UPRR	Minnis Pump Sta	48	990	48	990
CA2.4	Minnis Pump Station Rehabilitation	Minnis Pump Station						
L2.5	Lawton Drive Storm Drain Relief	Burley Drive/Lawton Drive/Canton Drive	Beacon Drive	Roswell Drive	24	1,250	24	1,250
P1.1	Montague Expressway Storm Drain Improvements	Montague Expressway	as shown	East Penitencia	18	660	30	660
			as shown	Trade Zone Blvd	30	610	36	610
P1.2	Montague Expressway Storm Drain Improvements at Lower Penitencia Creek	Montague Expressway	South Main Street	Lower Penitencia Creek	84	660	96	790
P1.3	Tarob Court Outfall Relocation	SD easement	Tarob Court	East Penitencia	42	770	42	770
P1.5	Lundy Place Relief Line	Lundy Place	Tarob Court	East Penitencia	18	750	30	750
P2.2	Woodland Way Storm Drain Improvements	Starlite Drive	Gibbons Court	SD easement	18	750	24	750
		SD easement	Starlite Drive	Moonlight Way	24	300	24	300
		Stardust Way	Moonlight Way	Moonlight Circle	24	160	36	160
		SD easement	Moonlight Circle	Sunrise Way	36	360	36	360
		Woodland Way	Sunrise Way	Moonbeam Way	24	890	42	520
			Moonbeam Way	Fallen Leaf Drive			48	370
P2.3	West Capitol Avenue Relief Lines	West Capitol Avenue	Starlite Drive	Evening Star Ct	30	1,260	36	440
							42	490
							48	330
		Evening Star Court	West Capitol Ave	Lower Penitencia	30	700	48	700
		West Capitol Avenue	Moonbeam Way	Fallen Leaf Dr	18	280	24	280
P3.3	Abbott Avenue Relief Drain	Abbott Avenue	I-880 offramp	Palmer St SD	18	840	30	840
P3.4	Junipero Drive Relief Drain	Junipero Drive	Rio Verde Place	Ethyl Street	24	890	36	890
			Ethyl Street	Lower Penitencia	48	450	54	450
P3.5	Corning Avenue Storm Drain Improvements	Corning Avenue	SD easement	SD easement	18	580	24	580
		Corning Avenue	SD easement	Ethyl Street	42	180	48	180
		Ethyl Street	Corning Avenue	Junipero Drive	42	530	48	530
P4.2	Rudyard Drive Relief Drain	Rudyard Drive	Heath Street	Smithwood St	24	600	30	250
			Smithwood Street	Silvera Street			36	350
		SD easement	Silvera Street	as shown	36	115	36	115
P6.7	Gingerwood Drive Relief Drain	Gingerwood Drive	Aspenridge Drive	Jergens Drive	30	500	48	500
PB1.11	Berryessa Street Relief Drain	Berryessa Street	as shown	Calero Street	18	450	21	450
PD1.3	South Park Victoria Drive Relief Drain	South Park Victoria Drive	Big Basin Drive	Clear Lake Ave	24	430	36	430
			Clear Lake Ave	Mt. Shasta Ave	30	790	48	790
PD1.4	Dempsey Road Relief Drain	Dempsey Road	Cuciz Lane	Mt. Shasta Ave	30	1,760	36	1,170
							42	590
WTCA1.4	Los Pinos Avenue Storm Drain Improvement	SD easement	Los Pinos Ave	Escuela Parkway	42	170	42	170
		Escuela Parkway	SD easement	Tramway Drive	48	210	54	210
WTCA1.5	Tramway Drive Relief Drains	Tramway Drive	Singley Drive	N Milpitas Blvd	66	1,300	84	1,300
			SD easement	Wyoma Place	24	510	30	510
			Wyoma Place	Escuela Parkway	24	540	36	540

CHAPTER 9
OPERATIONS, MAINTENANCE, AND REPLACEMENT

This Master Plan document is not intended 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 is provided into anticipated ongoing maintenance schedules, which include periodic replacement of major storm drain system components.

Milpitas is over 50 years old, and some of its older storm drainage infrastructure, particularly pumping equipment, is reaching the end of its useful life. Major equipment replacements are needed over the next several decades and the City needs to set aside sufficient funds for annual facility maintenance and a systematic long-term replacement program, as outlined in Chapter 10.

General Maintenance Regimen

Table 9-1 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 9-1
Storm System Maintenance Guidelines

Category	Schedule
Inlet Inspection	annually (summer-fall)
Inlet Cleaning	as required (ongoing)
Storm Drain Pipe Cleaning	continuous if possible (ongoing)
Channel Cleaning	annually (fall)
Detention Basin Dredging	every ten years
Pump Exercising	monthly (year round)
Engine Exercising	monthly at full load (year round)
Equipment Lubrication	per manufacturers' recommendations
Drain and fill diesel fuel tank	every six months
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; and 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, it is important that problems do 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 is 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, is deposited as the topography flattens out to the west.

Adding debris basins and modifying inlets along Evans Road and Piedmont Road as shown in Chapter 5 could help with the maintenance effort. A discussion of debris basin sizing criteria, which is related to the frequency that accumulated sediments need to be removed, is presented in Chapter 3. Retrofitting certain storm drain inlets to mimic the existing inlet for Piedmont Creek on Piedmont Road, as shown in Figure 9-1, would also help ease downstream maintenance.

Figure 9-1: 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, where the collection system has been designed to handle the 100-year discharge and where pipes are continuously submerged in water.

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, it is also recommended that the City retrofit 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.

Appendix C outlines pump station design, maintenance, and operation features that can help further the maintenance effort. Table 9-2 summarizes the recommended frequency

Table 9-2
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, grease bearings	1000 hr	----

Municipal Regional Stormwater Permit Requirements

Milpitas participates in the Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP) as a co-permitee under the California Regional Water Quality Control Board San Francisco Bay Region (Water Board) Municipal Regional Stormwater NPDES Permit (Order No. R2-2009-0074). Also referred to as the "MS4 Permit", it became effective December 1, 2009, and expires November 30, 2014.

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 10 for typical annual costs to satisfy system wide permit requirements.

Regulatory Background

The Water Board has found that storm water runoff from urban and developing areas within the San Francisco Bay region contains significant sources of pollutants that contribute to water quality impairment in 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 impairment of water bodies and attain water quality standards.

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

Routine Practices

Best management practices (BMPs) must be implemented to control and reduce polluted storm water 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 repair, maintenance, and cleaning
- Structural maintenance (e.g. bridge repair) and graffiti removal
- Storm water 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 pollution of storm water runoff. 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 for the detection and elimination of 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, are also permit requirements.

New Development and Redevelopment

Milpitas will administer the implementation of new development and redevelopment projects, so that they are in compliance 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 storm water 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 storm water treatment facilities in lieu of treatment on individual building sites. Such regional storm water treatment facilities have not been factored into capital planning for the storm water system as described in this master plan document.

Trash Load Reduction

The MS4 Permit requires Milpitas to implement control measures and take other actions to reduce trash loads from its municipal separate storm sewer systems (MS4s) by 40% by 2014, 70% by 2017, and 100% by 2022. During the permit term Milpitas must develop and implement a short-term trash load reduction plan, and develop and begin a long-term trash load reduction plan.

Establishing a Trash Load Baseline

Milpitas will submit a Progress Report to SCVURPPP as required by the Permit. This Progress Report will include a summary of the methodology chosen to establish the baseline trash load. Alternatively, Milpitas can accept the final methodology chosen by SCVURPPP. Trash reduction goals in the NPDES permit are stated in terms of a percentage reduction and not volume. These reduction goals are intended to reflect the percentage of trash produced that will be captured. Therefore, a baseline trash load must be established to set the trash load currently being generated within City limits. SCVURPPP can determine the trash load reduction tracking method that will be used to account for trash load reduction actions. The City will need to apply this method to demonstrate progress and attainment of trash load reduction levels.

Short-Term Planning

The MS4 Permit states that each permittee should have submitted a Short-Term Trash Load Reduction Plan, including an implementation schedule, to the Water Board by February 1, 2012. This plan described control measures and BMPs, including any trash reduction ordinances, that are currently being implemented and the current level of implementation. The plan would also propose additional control measures and best management practices (BMPs) whose implementation or increased level of implementation is designed to attain a 40% trash load reduction from its established MS4 baseline by July 1, 2014. The Short-Term Plan should account for the required mandatory minimum full trash capture device(s) and trash hot spot cleanup described herein. The City should be collaborating with SCVURPPP regarding the implementation of its short-term plan.

Trash Capture Devices

Milpitas is required to install and maintain a mandatory minimum number of full trash capture devices by July 1, 2014. The City must install one or more trash capture devices that trap all particles retained by a 5 mm mesh screen with a design treatment capacity at least equal to the 1-year (generally 85th percentile), 1-hour storm for a 20 acre area of commercial land use.

Such a trash capture device has been installed at the inlet to the Wrigley-Ford Pump Station, where it also protects the gravity bypass outfall to Berryessa Creek. This trash capture device filters low-flow runoff from 760 primarily commercial and industrial acres.

Hot Spot Requirements

Co-permittees must clean up selected trash hot spots to a level of "no visual impact" at least one time per year for the term of the permit. Trash hot spots in Milpitas have been identified by SCVURPPP.[1] Four trash hot spots are listed including:

- Berryessa Creek directly south of Gill Park, adjacent to Paseo Refugio, west of North Hillview Drive;
- Coyote Creek immediately north of State Highway 237;
- Coyote Creek immediately south of State Highway 237;
- Tularcitos Creek at Paseo Refugio, west of North Hillview Drive, adjacent to Gill Park.

All of these creek reaches are under the jurisdiction of the Santa Clara Valley Water District.

[1] Santa Clara Valley Urban Runoff Pollution Prevention Program, "Trash Hot Spot Selection Final Report," 2010.

Long-Term Planning

Milpitas must submit a Long-Term Trash Load Reduction Plan, including an implementation schedule, to the Water Board by February 1, 2014. This plan describes control measures and BMPs, including any trash reduction ordinances, that will be implemented and the level of implementation. Control measures and BMPs will be designed to attain a 40% trash load reduction by July 1, 2014; a 70% reduction by July 1, 2017; and 100% trash load reduction by July 1, 2022. Since some of these deadlines fall after the expiration of the current permit, it is possible that these requirements could change. Figure 9-2 shows trash problem areas in Milpitas as identified by SCVURPPP in 2004. Table 9-3 provides an accounting of land use types within each drainage system and SCVURPPP's preliminary trash loading rates for each land use type. This information is intended to provide a basis for longer term trash capture plans prepared by the City.

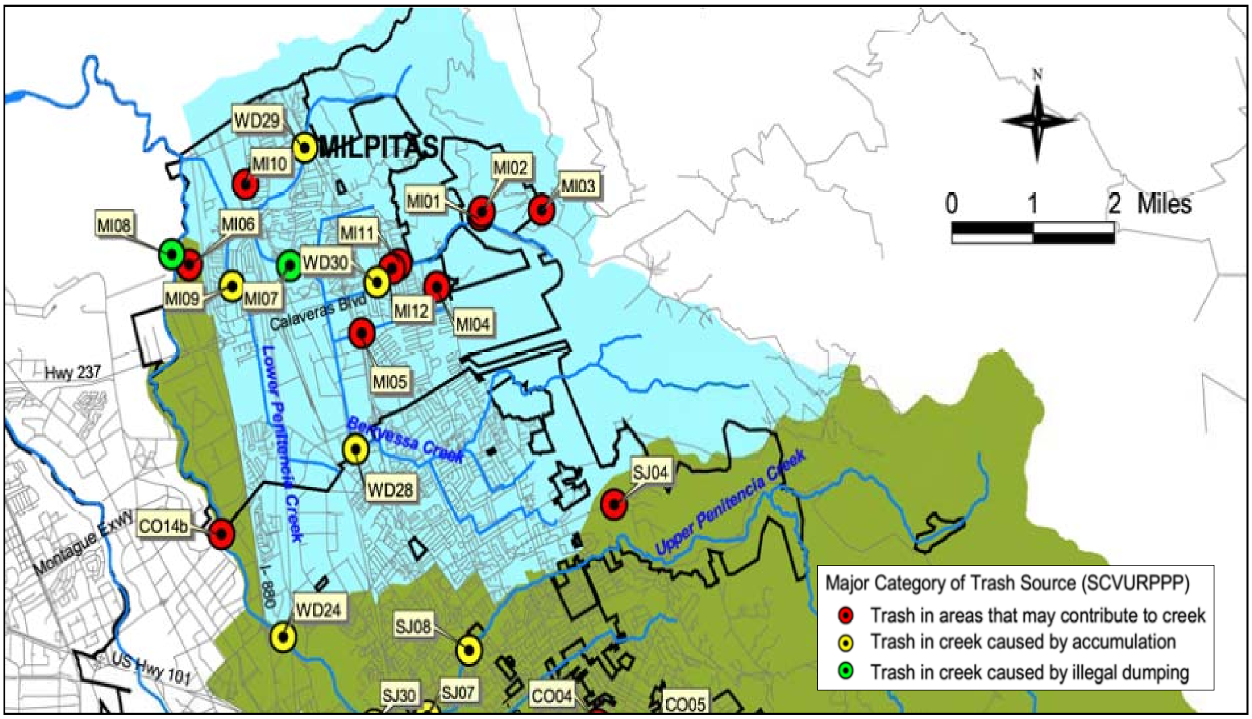

Figure 9-2: Trash Problem Areas Identified by SCVURPPP

Reporting and Schedule Requirements

Milpitas is required to submit annual reports to the Water Board showing progress toward meeting the regulatory requirements. Annual reporting requirements specific to trash reduction include a summary of trash load reduction actions (control measures and BMPs) including the types of actions and levels of implementation and the total trash loads and dominant types of trash removed by its actions both collectively and individually. Trash hot spot data shall also be included. Beginning with the 2012 Annual Report, each permittee shall report its percent annual trash load reduction relative to its baseline trash load. The permittees shall retain records which provide supporting documentation of trash load reduction actions. These records shall also include volume and dominant type of trash removed from full trash capture devices, each Trash Hot Spot cleanup, and additional control measures or BMPs implemented. Data may be combined for specific types of full trash capture devices deployed in the same drainage area. Figure 9-3 provides a flowchart showing typical trash capture plan reporting activities with a schedule that meets the MS4 Permit requirements.

Table 9-3
Major Land Use Types by MS4 Drainage Systems

System ID	Area by Land Use Type (acres)					Total
	Single Family Residential	Multi-Family Residential	Commercial	Industrial	Parks/Other/ Open Space	
BT1	391	55	25	27	504	1,002
C1	---	---	56	201	77	332
C2-C3	---	21	84	194	97	396
C4	---	---	79	129	127	335
CA1	51	---	---	---	53	104
CA2	65	7	18	90	34	214
F1	2	71	143	90	21	327
L2	162	25	21	22	75	305
P1	---	85	55	10	44	194
P2	111	29	14	8	93	255
P3	27	74	112	25	49	287
P4	80	---	15	10	40	145
P5	37	26	---	109	38	210
P6	138	134	24	10	127	433
PB1	134	50	5	12	12	213
PD1	732	49	40	46	412	1,279
PDB1	19	---	11	222	99	351
T1	170	6	8	---	66	250
W1	---	30	59	328	11	428
WTCA1	236	50	105	31	124	546
Total	2,355	712	874	1.564	2,103	7,608

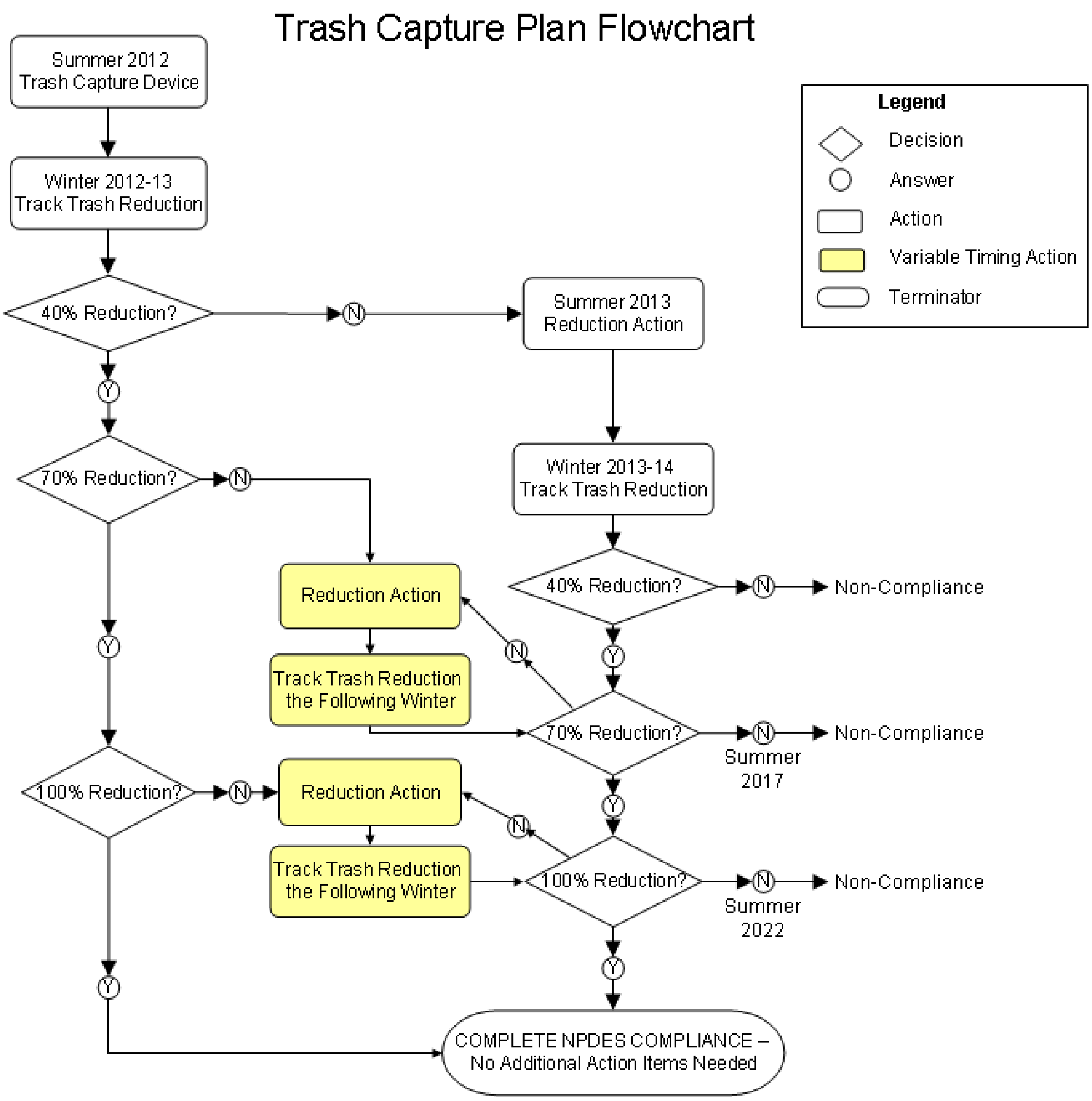

Figure 9-3: Trash Capture Plan Flowchart

System Replacement

With predominantly reinforced concrete pipe, collection system materials can be expected to last indefinitely, so a major replacement schedule for 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.

Pumping facilities, on the other hand, rely heavily on mechanical and electrical equipment that will wear out, particularly since the stations are not operated on a constant basis. 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 9-4 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 9-4
Pumping Facility Replacement

ID	Station Name	Originally Built	Age (years)	Recent Equipment Replacement	Proposed Schedule for	
					Equipment Replacement	Major Rehabilitation
1	California Circle	1983	27		2020	2050
2	Jurgens[1]	1989	21		2030	2060
3	McCarthy Ranch	1994	16		2040	2055
4	Abbott Avenue[2]	1983	27	2002	2015	2045
5	Minnis[3]	1978	32		2015	2045
6	Penitencia[3]	1960	50		2015	2015
7	Wrigley-Ford	1993	17		2035	2065
8	Berryessa[4]	1977	33	2006	2040	2040
9	Manor	1993	17		2035	2065
10	Spence Creek	1988	22		2030	2060
11	Bellew	1985	25		2025	2055
12	Murphy	1983	27		2025	2055
13	Oak Creek	1979	31		2020	2050

[1]Flood-proofed in 2002
[2]Equipment rehabilitated in 2002
[3]Scheduled as High-priority CIP
[4]All pumping, electrical, and control equipment replaced and flood-proofed in 2006

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CHAPTER 10
STORM DRAINAGE FUNDING REQUIREMENTS

This chapter summarizes budget requirements to fund Capital Improvement Program projects described in Chapter 8, and facility maintenance and replacement as outlined in Chapter 9.

Table 10-1 summarizes estimated annual costs for implementing the proposed priority Capital Improvement Program, near-term equipment replacement, facility maintenance, and future facility replacement. All cost estimates are in 2010 dollars (ENR Index = 10,000). 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. CIP implementation estimates in Table 10-1 assume that where feasible, the parallel pipe alternative will be selected to save cost. For the purpose of setting aside sufficient funds for future work, the 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.

Table 10-1
Storm Drainage Funding Requirements

Category	Present Worth	Annualized Cost
CIP Implementation	$27,000,000	$2,400,000
Long-Term Equipment Replacement	$38,000,000	$1,100,000
Annual Operations and Maintenance	---	$1,500,000
Total Budget	$65,000,000	$5,000,000

Spread over Milpitas' 6,048 acres of developed or developable land, the average annual cost per acre is $830 to fund Master Plan improvements and maintain storm drainage facilities. Based on land use equivalent, which is related to a site's runoff coefficient, a typical single-family residence's budget responsibility would be about $70 per annum. Commercial and industrial properties would need to contribute about $1,300 per gross acre per year.

If only the high priority CIP projects are to be completed, the annual storm drainage budget requirement decreases to about $4 million, and the cost per typical household would be about $60 per year.

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 are recommended to bring systems into compliance with performance criteria. This is a master plan level effort. Hence, many of the practical constraints that will govern the detail design and construction of actual infrastructure improvements are not known 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 a 50% contingency to those cost estimates. Table 10-2 provides unit cost information for storm drain collection systems. Piping costs are based on bids from past storm drain projects, adjusted to the current ENR Index, supplemented as necessary by cost data contained in 2010 Current Construction Costs, Saylor Publications, Inc. Unit costs for pumping equipment including industrial engines are derived from past projects, and data collected over the years by Schaaf & Wheeler and the East Bay Municipal Utility District.

Table 10-2
Storm Drain Collection Costs per Lineal Foot
(All costs in 2010 dollars; ENR = 10,000)

Diameter	**18"**	**24"**	**30"**	**36"**	**42"**	**48"**	**54"**	**60"**	**66"**	**72"**	**84"**	**96"**
Pipe Installation	75	99	139	174	207	241	289	332	387	443	607	749
Street Repairs	35	40	45	50	55	60	65	70	75	80	90	100
Connections	51	52	52	53	53	54	55	55	56	56	56	59
20% Eng & Admin	32	38	47	55	63	71	81	91	104	116	116	182
50% Contingency	97	114	142	166	189	213	244	274	311	348	348	545
Total Unit Cost	**$270**	**$343**	**$425**	**$499**	**$568**	**$639**	**$732**	**$823**	**$932**	**$1,013**	**$1,043**	**$1,635**

Table 10-3 details the calculation of estimated CIP cost by drainage system, and by Master Plan improvement priority. Cost estimates for the estimation of required annual revenue streams are based on a rounded midpoint between parallel pipe and replacement pipe options. These costs are $13 million for high priority projects, $12 million for medium priority projects, and $12 million for low priority projects.

Table 10-3
Capital Improvement Program Cost by System and Priority

		Parallel Option						Replacement Option					
				Estimated Capital Cost						Estimated Capital Cost			
Project I.D.	Project Name	Size	Lineal Feet	High Priority	Medium Priority	Low Priority	By Basin	Size	Lineal Feet	High Priority	Medium Priority	Low Priority	By Basin
BT1.1	Traughber St SD Replacement	→	300	330,900				72	300	330,900			
BT1.2	Wool Drive SD Improvements	42	1,210	687,280				48	1,210	833,690			
BT1.3	Park View Dr SD Improvement	→	425	262,650				42	425	262,650			
BT1.4	Tramway Dr SD Improvement	18	530		153,700			24	530		241,150		
BT1.5	Calaveras Rd Outfall Relocation	36	800		399,200			←	800		399,200		
BT1.6	Fanyon Street SD Improvement	24	1,150		488,750			36	1,150		619,850		
BT1.7	Temple Drive SD Improvement	24	1,435		609,875			36	1,435		773,465		
BT1.8	Calaveras Ridge Dr. SD Impvt.	18	315		91,350			24	315		143,325		
BT1.9	Park Hill Drive SD Improvement	24	820			348,500		30	820			381,300	
		30	810			344,250		36	810			436,590	
BT1.10	Debris Basins and Inlet Modifications	→			400,000						400,000		
Subtotal			**7,795**	**$ 1,280,830**	**$ 2,142,875**	**$ 692,750**	**$ 4,116,455**		**7,795**	**$ 1,427,240**	**$ 2,576,990**	**$ 817,890**	**$ 4,822,120**
C1.1	Sycamore Drive SD Improvements	42	1,270	721,360				48	1,270	875,030			
C1.2	Buckeye Court SD Replacement	→	1,125		606,375			36	1,125		606,375		
C1.3	Cottonwood Dr SD Improvements	24	1,400		595,000			30	550		255,750		
								36	280		150,920		
								42	570		352,260		
C1.4	Barber Lane SD Improvements	36	780		389,220			48	780		537,420		
C1.5	McCarthy Blvd SD Improvements	36	490		244,510			42	490		302,820		
Subtotal			**5,065**	**$ 721,360**	**$ 1,835,105**	**-**	**$ 2,556,465**		**5,065**	**$ 875,030**	**$ 2,205,545**	**-**	**$ 3,080,575**
C2	NO CAPITAL IMPROVEMENTS												
C3.1	Murphy Ranch Road SD Improvement	36	1,160		578,840			48	190		130,910		
								54	420		328,440		
								60	550		485,650		
C3.2	Sumac Drive SD Improvement	36	450		224,550			48	450		310,050		
Subtotal			**1,610**	**$ -**	**$ 803,390**	**-**	**$ 803,390**		**1,610**	**$ -**	**$ 1,255,050**	**-**	**$ 1,255,050**
C4	NO CAPITAL IMPROVEMENTS												
CA1	NO CAPITAL IMPROVEMENTS												
CA2.1	Minnis Pump Station Standby Power			360,000						360,000			
CA2.2	North Milpitas Blvd SD Relief	42	100		56,800			54	100		78,200		
CA2.3	Minnis Circle SD Replacement	→	1,130			778,570		48	1,130			778,570	
CA2.4	Minnis Pump Station Rehabilitation					400,000						400,000	
Subtotal			**1,230**	**$ 360,000**	**$ 56,800**	**$ 1,178,570**	**$ 1,595,370**		**1,230**	**$ 360,000**	**$ 78,200**	**$ 1,178,570**	**$ 1,616,770**
F1	NO CAPITAL IMPROVEMENTS												
L2.1	Dempsey Road SD Relief	→	1,100	592,900				36	1,100	592,900			
L2.2	Edsel Drive SD Improvements	36	730	364,270				←	730	364,270			
		42	1,200	681,600				48	1,200	826,800			
		→	200	137,800				48	200	137,800			
L2.3	Carnegie Drive SD Improvements	30	1,240		527,000			36	740		398,860		
								42	500		309,000		
L2.4	Roswell/Canton SD Improvements	30	2,130		905,250			36	250		134,750		
								42	1,880		1,161,840		
L2.5	Lawton Drive SD Relief	→	1,250			568,750		24	1,250			568,750	
Subtotal			**7,850**	**$ 1,776,570**	**$ 1,432,250**	**$ 568,750**	**$ 3,777,570**		**7,850**	**$ 1,921,770**	**$ 2,004,450**	**$ 568,750**	**$ 4,494,970**
P1.1	Montague Exwy SD Impvts	18	660			191,400		30	660			306,900	
		30	610			259,250		36	610			328,790	
P1.2	Montague SD Impvts at Lower Pen	84	660			897,600		96	790			1,339,050	
P1.3	Tarob Court Outfall Relocation	→	770			475,860		42	770			475,860	
P1.4	Lundy Place Relief Line	18	750			217,500		30	750			348,750	
Subtotal			**3,450**	**$ -**	**$ -**	**$ 2,041,610**	**$ 2,041,610**		**3,580**	**$ -**	**$ -**	**$ 2,799,350**	**$ 2,799,350**
P2.1	S Main Street SD Improvements	24	660		280,500			36	1,100		592,900		
P2.2	Woodland Way SD Improvements	18	750			217,500		24	1,050			477,750	
		24	1,350			573,750		36	520			280,280	
		36	360			179,640		42	520			321,360	
								48	370			254,930	
P2.3	West Capitol Ave Relief Line	18	280			81,200		24	280			127,400	
		30	1,960			833,000		36	440			237,160	
								42	490			302,820	
								48	1,030			709,670	
Subtotal			**5,360**	**$ -**	**$ 280,500**	**$ 1,885,090**	**$ 2,165,590**		**5,800**	**$ -**	**$ 592,900**	**$ 2,711,370**	**$ 3,304,270**
P3.1	Spence Creek PS Standby Power			$ 750,000						$ 750,000			
P3.2	Carlo Street Relief Drain	24	780		331,500			36	780		420,420		
P3.3	South Abbott Avenue Relief Drain	18	840			243,600		30	840			390,600	
P3.4	Junipero Drive Relief Drain	24	890			378,250		36	890			479,710	
		48	450			287,550		54	450			351,900	
P3.5	Corning Ave SD Improvements	18	580			168,200		24	580			263,900	
		42	715			406,120		48	715			492,635	
Subtotal			**4,255**	**$ 750,000**	**$ 331,500**	**$ 1,483,720**	**$ 2,565,220**		**4,255**	**$ 750,000**	**$ 420,420**	**$ 1,978,745**	**$ 3,149,165**
P4.1	Silvera Street SD Replacement	→	140	61,320				27	140	61,320			
P4.2	Rudyard Drive Relief Drain	24	600			255,000		30	250			116,250	
								36	350			188,650	
		→	115			61,985		36	115			61,985	
Subtotal			**855**	**$ 61,320**	**$ -**	**$ 316,985**	**$ 378,305**		**855**	**$ 61,320**	**$ -**	**$ 366,885**	**$ 428,205**
P5.1	Abbott Pump Station Improvement				500,000						500,000		
Subtotal			**-**	**$ -**	**$ 500,000**	**$ -**	**$ 500,000**		**-**	**$ -**	**$ 500,000**	**$ -**	**$ 500,000**

Table 10-4
Capital Improvement Program Costs by System and Priority (continued)

		Parallel Option						Replacement Option					
				Estimated Capital Cost						Estimated Capital Cost			
Project I.D.	Project Name	Size	Lineal Feet	High Priority	Medium Priority	Low Priority	By Basin	Size	Lineal Feet	High Priority	Medium Priority	Low Priority	By Basin
P6.1	Arizona Avenue Relief Drain	30	1,320		561,000			48	1,320		909,480		
P6.2	Wilson Way SD Improvements	18	180		52,200			30	180		83,700		
		→	960		661,440			48	960		661,440		
P6.3	Summerwind Way Relief Drain	36	360		179,640			48	360		248,040		
P6.4	Milmont Drive Relief Drain	48	480		306,720			54	480		375,360		
P6.5	Jergens Drive Relief Drain	54	500		366,000			84	500		710,000		
P6.6	Connect Twin RCP Crossing at SVBX				250,000						250,000		
P6.7	Gingerwood Drive Relief Drain	30	500			212,500		48	500			344,500	
Subtotal			**4,300**	**$ -**	**$ 2,377,000**	**$ 212,500**	**$ 2,589,500**		**4,300**	**$ -**	**$ 3,238,020**	**$ 344,500**	**$ 3,582,520**
PB1.1	Redwood Avenue Relief Drain	24	1,300	552,500				30	1,140	530,100			
								36	160	86,240			
PB1.2	South Abbott Avenue Relief Drain	36	1,425	711,075				42	1,425	880,650			
PB1.3	Maple Avenue Relief Drain	18	390	113,100				24	220	100,100			
PB1.4	Chestnut Avenue Relief Drain	36	1,060	528,940				42	1,060	655,080			
PB1.5	Heath Street Relief Drain	36	520	259,480				42	520	321,360			
PB1.6	North Abel Street Relief Drain	48	2,530	1,616,670				←	2,530	1,616,670			
PB1.7	Vasona Street SD Improvement	24	290	123,250				30	290	134,850			
		→	240	165,360				48	240	165,360			
		→	200	123,600				42	200	123,600			
PB1.8	Penitencia Pump Station Replacement			3,500,000						3,500,000			
PB1.9	Lexington Street SD Improvements	→	220	118,580				36	220	118,580			
		→	260	160,680				42	260	160,680			
		→	290	199,810				48	290	199,810			
PB1.10	Coyote Street Relief Line	36	750	463,500				42	750	463,500			
PB1.11	Berryessa Street Relief Drain	18	450			130,500		21	450			167,850	
Subtotal			**9,925**	**$ 8,636,545**	**$ -**	**$ 130,500**	**$ 8,767,045**		**9,755**	**$ 9,056,580**	**$ -**	**$ 167,850**	**$ 9,224,430**
PD1.2	Vista Way Relief Drain	36	260		129,740			48	260		179,140		
PD1.2	Falcato Drive Relief Drain	24	310		131,750			30	310		144,150		
PD1.3	South Park Victoria Dr Relief Drain	24	430			182,750		36	430			231,770	
		30	790			335,750		48	790			544,310	
PD1.4	Dempsey Road Relief Drain	30	1,760			748,000		36	1,170			630,630	
								42	590			364,620	
PD1.5	Debris Basins and Inlet Modifications				500,000						500,000		
Subtotal			**3,550**	**$ -**	**$ 761,490**	**$ 1,266,500**	**$ 2,027,990**		**3,550**	**$ -**	**$ 823,290**	**$ 1,771,330**	**$ 2,594,620**
PDB1.1	Wrigley Wa SD Replacement	→	370	199,430				36	370	199,430			
PDB1.2	Watson Court Relief Drain	18	310		89,900			30	310		144,150		
		24	370		157,250			36	370		199,430		
Subtotal			**1,050**	**$ 199,430**	**$ 247,150**	**$ -**	**$ 446,580**		**1,050**	**$ 199,430**	**$ 343,580**	**$ -**	**$ 543,010**
T1.1	Jacklin Road Relief Drain	72	300	312,900				84	300	426,000			
T1.2	Calaveras Ridge Dr. SD Outfall	24	150		63,750			←	150		63,750		
T1.3	Inlet Modification				100,000						100,000		
Subtotal			**450**	**$ 312,900**	**$ 163,750**	**$ -**	**$ 476,650**		**450**	**$ 426,000**	**$ 163,750**	**$ -**	**$ 589,750**
W1	NO CAPITAL IMPROVEMENTS												
WTCA1.1	North Hillview Drive Relief Drain	42	900	511,200				48	900	620,100			
		72	800	834,400				84	800	1,136,000			
		→	300	136,500				24	300	136,500			
WTCA1.4	Glasgow Court Relief Drain	24	765		325,125			30	310		144,150		
								36	455		245,245		
WTCA1.5	Loch Lomond Ct. Relief Drain	18	690		200,100			24	390		177,450		
								27	300		131,400		
WTCA1.2	Los Pinos Ave SD Improvement	→	170			105,060		42	170			105,060	
		48	210			134,190		54	210			164,220	
WTCA1.3	Tramway Drive Relief Drains	66	1,300			1,211,600		84	1,300			1,846,000	
		24	1,050			446,250		30	510			237,150	
		---	---					36	540			291,060	
Subtotal			**6,185**	**$ 1,482,100**	**$ 525,225**	**$ 1,897,100**	**$ 3,904,425**		**6,185**	**$ 1,892,600**	**$ 698,245**	**$ 2,643,490**	**$ 5,234,335**
TOTAL				**$ 15,581,055** ***High Priority***	**$ 11,457,035** ***Medium Priority***	**$ 11,674,075** ***Low Priority***	**$ 38,712,165** ***All Projects***			**$ 16,969,970** ***High Priority***	**$ 14,900,440** ***Medium Priority***	**$ 15,348,730** ***Low Priority***	**$ 47,219,140** ***All Projects***

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 9. Based on these regimens and input from City staff, the following annual funding levels 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.

Annual Operations	$ 500,000
Preventative Maintenance	$ 500,000
NPDES Permit Compliance	$ 200,000
Programmed Replacement	$ 300,000
Total Annual Costs	$ 1,500,000

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 at the City's request. Estimated costs in 2010 dollars for other pump station replacement projects are based on the unit costs indicated in Table 10-4. Equal payment series capital-recovery fund amounts for equipment replacement and major rehabilitation are given in Table 10-5, based on an interest rate of six percent, and beginning to accumulate the annual fund in 2015. Near-term replacement costs for Penitencia Pump Station and Minnis Pump Station (Table 10-5) are included with the CIP implementation cost given in Table 10-1.

Table 10-5
Storm Pumping and Storage Unit Costs
(All costs in 2010 dollars; ENR = 10,000)

Category	Unit Cost
Axial Flow Pump and Driver	$2,600 per cfs of capacity
Direct Drive Engine	$700 per horsepower
Engine-Generator Set	$500 per kilowatt
Pump Building	$300 per square foot
Storage Excavation	$30 per cubic yard

Table 10-6
Pumping Facility Replacement

ID	Facility	Next Scheduled Replacement			Second Scheduled Replacement		
		Year	Cost	Annual Fund	Year	Cost	Annual Fund
1	California Circle	2020	$750,000	$133,000	2050	$1,500,000	$14,000
2	Jurgens	2030	$2,000,000	$85,000	2060	$2,500,000	$12,000
3	McCarthy	2040	$2,500,000	$45,000	2055	$3,500,000	$24,000
4	Abbott		---	---	2045	$750,000	$10,000
5	Minnis	2015	incl. in CIP	---	2045	$750,000	$10,000
6	Penitencia	2015	incl. in CIP	---	2065	$2,000,000	$6,000
7	Wrigley-Ford	2035	$1,500,000	$40,000	2065	$2,500,000	$9,000
8	Berryessa		---	---	2040	$2,000,000	$37,000
9	Manor	2035	$600,000	$16,000	2065	$1,000,000	$5,000
10	Spence Creek	2030	$750,000	$31,000	2060	$1,000,000	$6,000
11	Bellew	2025	$2,000,000	$189,000	2055	$3,500,000	$24,000
12	Murphy	2025	$1,500,000	$113,000	2045	$2,000,000	$25,000
13	Oak Creek	2020	$1,400,000	$248,000	2050	$2,000,000	$18,000
	Total		**$13,000,000**	**$900,000**		**$25,000,000**	**$200,000**