


2018 Integrated Master Plan

FINAL REPORT revision 1.2

March 2018

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City of Banning

INTEGRATED MASTER PLAN

FINAL | March 2018

revision 1.2 October 2018

REGISTERED PROFESSIONAL ENGINEER
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Abbreviations

AACE	Advancement of Cost Engineering
ABS	Acrylonitrile-Butadiene-Styrene
ACP	Asbestos Cement
ADD	average day demand
afy	acre-feet per year
AWWA	American Water Works Association
Carollo	Carollo Engineers, Inc.
CEQA	California Environmental Quality Act
City	City of Banning
CIP	Capital Improvement Program
CIPP	cured-in-place pipe
C&R	capacity and reliability
d/D	depth-to-pipe diameter ratio
DIP	Ductile Iron
ENR CCI	*Engineering News Recorded* Construction Cost Index
ETo	Evapotranspiration
fps	feet per second
ft/s	feet per second
IMP	Integrated Master Plan
IPR	Indirect Potable Reuse
MDD	maximum day demand
MDD+FF	MDD plus Fire Flow
NPR	Non-Potable Reuse
o.o.s.	out-of-service
PHD	peak hour demand
PRV	pressure reducing valve
psi	pounds per square inch
PS	pump station
PVC	polyvinyl chloride
PWWF	peak wet weather flow
ROW	right-of-way

R&R	repair and rehabilitation
RSG	Rancho San Gorgonio
SSO	sanitary sewer overflow
SOI	Sphere of Influence
UNK	unknown materials
UWMP	Urban Water Management Plan
VCP	Vitrified Clay Pipe

Executive Summary

The City of Banning (City) has retained Carollo Engineers, Inc. (Carollo) to prepare this Integrated Master Plan (IMP). This IMP evaluates the performance and condition of the City's potable water, wastewater, and recycled water systems under existing and future conditions through year 2040. This chapter presents the purpose, objectives, and background of this IMP. A list of references used to prepare this IMP is provided in Appendix A.

Introduction

The objective of this Integrated Master Plan (IMP) is to develop a capital improvement plan (CIP) that guides the City of Banning (City) in the planning and development of water, wastewater, and recycled water system facilities required to meet system performance criteria for existing customers, as well as to support anticipated growth through the year 2040.

Some of the key goals of this IMP are:

- Identify the existing, near-term (year 2025), long-term (year 2040), and build-out potable water demands, wastewater flows, and recycled water demands.
- Define planning and evaluation criteria for the City's potable water, wastewater, and recycled water systems.
- Determine where deficiencies exist in the City's potable water and wastewater systems under existing, long-term (year 2040), and build-out conditions.
- Identify necessary recycled water system facilities to serve the City's potential recycled water customers.
- Prepare an integrated CIP with phasing of recommended improvements and an integrated phasing plan.

Study Area

The City is located in northern Riverside County in Southern California, approximately 25 miles east of downtown Riverside and 85 miles from downtown Los Angeles. The City encompasses 23.2 square miles astride Interstate 10 in the San Gorgonio Pass and is bounded by the City of Beaumont on the west, the Morongo Band of Mission Indians in the east, the San Bernardino Mountains to the north, and the San Jacinto Mountains to the south. The study areas of each system indicate the areas that are being served and differ for each system.

The total potable water study area includes the City boundary and approximately 2.4 square miles outside of the City boundary as shown on Figure ES.1.

The City's wastewater study area consists of three basic boundaries identified in the General Plan, which include the City limits, the Sphere of Influence (SOI), and Planning Area (PA) as shown on Figure ES.2. The total area of the wastewater study area is approximately 36.8 square miles.

The recycled water study area for this IMP coincides with the wastewater service area, which includes the City limits, SOI, and Planning Area shown on Figure ES.2. This study area may change as the system develops.

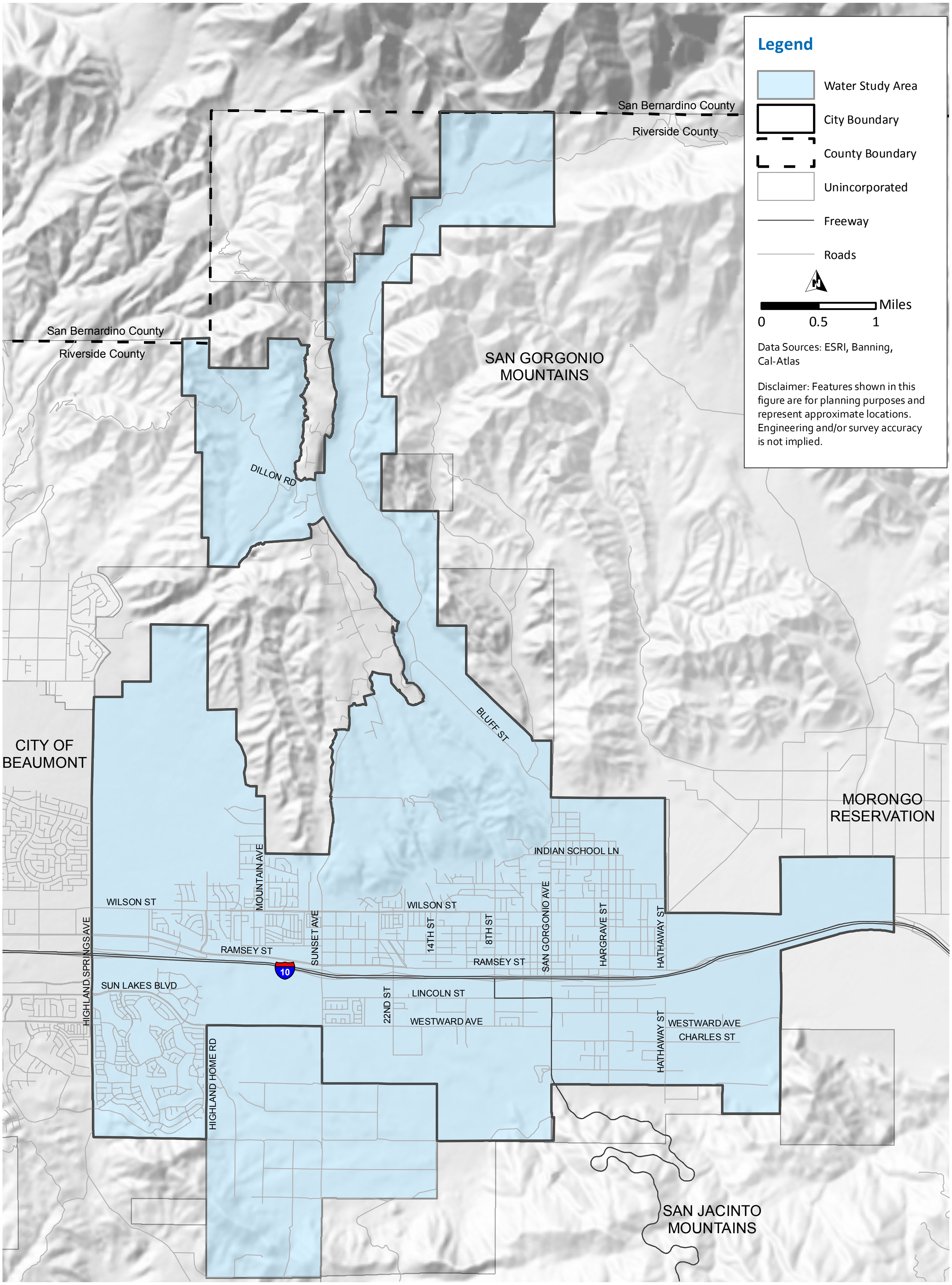

Figure ES.1 Water Study Area

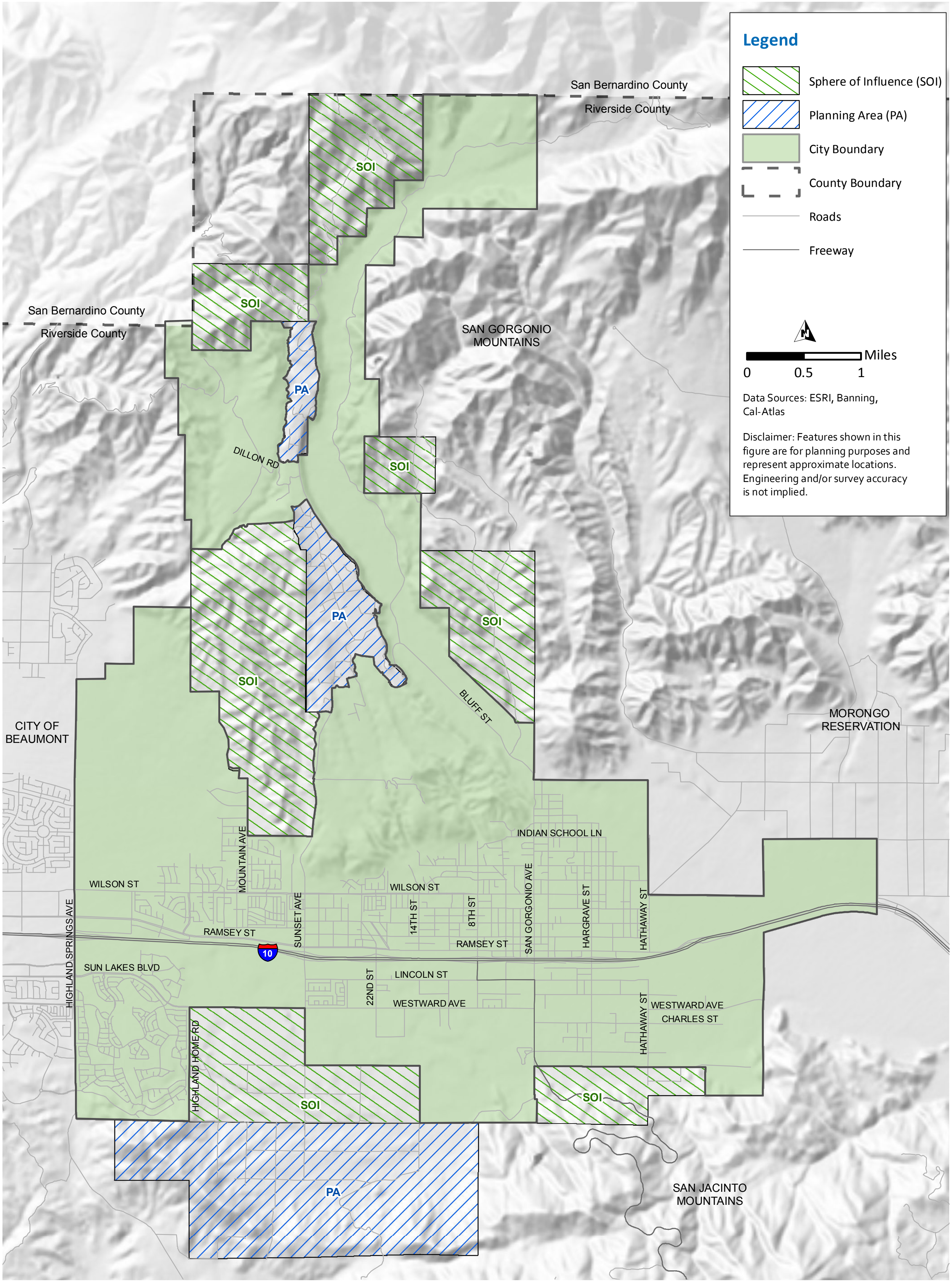

Figure ES.2 Wastewater and Recycled Water Study Area

Land Use

The City's existing land use is based on the City's General Plan, which was most recently updated in 2016. As part of this IMP, City staff provided a list of known future developments and identified each as near-term (likely to be constructed by 2025) and long-term (anticipated to constructed between 2025 and 2040). Of the 15 known developments, 2 were identified as near-term and 7 were identified as long-term.

Population

As of 2016, the total existing population within the City's boundaries was estimated at 30,834 people. Since the water and wastewater service areas extend beyond the City boundaries, the estimated population in year 2016 was 30,834 for the water service area and 29,607 for the wastewater service area.

The City's 2015 Urban Water Management Plan (UWMP) summarizes the City's service area population projection with and without two master planned communities (Butterfield and Rancho San Gorgonio). The population projections for each service area are summarized in Table ES.1 and Table ES.2.

Table ES.1 Water Service Area Population Projections

Year	Population Without Master Planned Communities [1]	Master Planned Communities Estimated Population [2]	Total Water Service Area Population
2020	31,913	3,042	34,955
2025	33,335	7,965	41,300
2030	34,757	16,177	50,934
2035	36,179	20,168	56,347
2040	37,700	23,288	60,988

Notes:
(1) City's water service area population data without master planned communities retrieved from City's 2015 UWMP.
(2) RSG population retrieved from Water Supply Assessment (WSA). Butterfield population calculated based on number of dwelling units and the 2015 UWMP assumption of 3.12 persons per connection.

Table ES.2 Wastewater Service Area Population Projections

Year	Population Without Master Planned Communities [1]	Master Planned Communities Estimated Population [2]	Total Wastewater Service Area Population
2020	30,812	3,042	33,854
2025	32,185	7,965	40,150
2030	33,558	16,177	49,735
2035	34,931	20,168	55,099
2040	36,399	23,288	59,687

Notes:
(1) Wastewater service area population calculated using Census Block data and removing parcels on septic systems.
(2) RSG population retrieved from Water Supply Assessment (WSA). Butterfield population calculated based on number of dwelling units and the 2015 UWMP assumption of 3.12 persons per connection.

For the purpose of this IMP, the service area population is assumed to include the two master planned communities.

Water Demands and Flow Forecasts

Potable Water Demands

The City's potable water supply is primarily served by groundwater from five basins: Beaumont Basin, Banning Basin, Cabazon Storage Unit, Banning Bench Storage Unit, and Banning Water Canyon Basin. The average annual water supply between 2012 and 2014 was 8,595 afy, which equates to an average day demand (ADD) of 7.7 mgd. The average maximum month demand (MMD) for this same time period was 10.1 mgd, while the maximum day demand (MDD) was calculated to be approximately 13.3 mgd. An MDD peaking factor of 1.7 was used in this IMP.

Based on a review of the available data for the City, it was determined that the most accurate demand forecasting method is a combination of a population- and land-use-based demand forecasting method. Population-based demand forecasting utilized a calculated per-capita water use and was obtained from the City's 2015 UWMP (see Section 3.1.2.1). Land-use-based demand forecasting utilizes calculated water demand factors (see Section 3.1.2.2). The water demand factors estimate the amount of water usage per area for a certain land-use type and are typically expressed in gallons per day per acre (gpd/ac).

Future demands were estimated and grouped into three categories; 1) existing customers, 2) known developments, and 3) infill development. The forecasted water demands are summarized in Table ES.3.

Table ES.3 Water Demand Projections

Year	Existing Demand(1) (afy)	Near-term (2025) Demand (afy)	Long-term (2040) Demand (afy)	Build-out Demand (afy)
Existing (including Potable Water Offset)(1)	5,302	5,262	5,262	5,262
Near-term Known Developments	0	784	784	784
Long-term Known Developments	0	0	1,581	1,581
Build-out Known Developments	0	0	0	1,409
Infill	0	972	823	3,303
Total(2)	**5,302**	**7,018**	**8,450**	**12,339**

Notes:
(1) Existing is represented as the average of years 2012 through 2014. Future demand decreases due to conversion of potable water customers to recycled water for irrigation.
(2) Retrieved from 2015 UWMP. Demand includes projected deliveries and losses.

As shown in Table ES.3, the City's future water demands are expected to increase from approximately 5,302 afy to 7,018 by the year 2025, and to 8,450 afy by the year 2040. The majority of this increase in water demands within the planning horizon is attributed to new planned developments.

Wastewater Flows

Based on historical records, the average annual flow at the City's wastewater treatment plant (WWTP) was estimated to be roughly 2.02 mgd for years 2011 through 2016. The existing average dry weather flow (ADWF) is approximately 2.08 mgd for years 2011 through 2016. The City's 5-year average per capita wastewater generation was estimated at 73 gallons per capita per day (gpcd).

Wastewater flow projections were developed using a land use based methodology. Wastewater flow factors (WWFF) were developed to correlate land use and sewer generation. Projected wastewater flows are presented by phase in Table ES.4

Table ES.4 Flow Projections

Flow Condition	ADWF (mgd)	PWWF (mgd)	PWWF Peaking Factor
Existing	2.01	13.8	6.87
Near Term (2025)	2.80	15.2	5.43
Long Term (2040)	4.29	17.5	4.08
Buildout	6.35	22.2	3.50

Notes:
(1) ADWF = Average Dry Weather Flow.
(2) PWWF = Peak Wet Weather Flow and is based on 1-hour interval.

As shown in Table ES.4, the City's ADWF is projected to increase to 2.8 mgd by year 2020 and to 4.29 mgd by year 2040.

In addition to the ADWF, a design storm was selected to predict peak wet-weather flow (PWWF) conditions. Peak wet-weather flow (PWWF) is the highest observed flow that occurs following a design storm event. Wet-weather I/I cause flows in the collection system to increase. PWWF is typically used for designing sewers and lift stations. Therefore, the PWWF and the "Design Flow" are synonymous and will be used interchangeably throughout this report.

This IMP utilizes the 10-year, 24-hour design storm rainfall pattern for generating the peak wet-weather flow. This design storm has a total rainfall of 4.46 inches in a 24-hour period with a peak intensity of 0.77 inches per hour and a total.

Recycled Water Demand

The City currently serves one customer (Sun Lakes Development Golf Course) with non-potable water from Well M7 and Well M12. Based on average production data for years 2012 through 2014, the average annual demand for Sun Lakes Development Golf Course is 850 afy (or 0.8 mgd). Aside from this customer, the City does not have any other recycled water or non-potable demands.

Future recycled water demand projections were retrieved from the 2006 Recycled Water Master Plan (RWMP). Based on an evaluation of the customer locations and estimated recycled water demand, City staff decided to keep the recycled water system south of the Interstate 10. A list of the potential recycled water customers and estimated demands are summarized in Table ES.5. As shown in Table ES.5, the total projected annual recycled water demand is approximately 2,530 afy.

Table ES.5 Potential Recycled Water Customers and Demands

Customer Name	Annual Demand (afy)	MDD (mgd)	PHD (mgd)
Existing City Customers			
Sun Lakes Development	850	2.1	2.1
Banning High School	175	1.3	1.3
Dysart Park	87	0.7	0.7
Lions Park	79	0.6	0.6
Future Customers/Developments			
Butterfield Development	864	2.2	6.5
Rancho San Gorgonio Development	217	0.5	1.6
Five Bridges Development	223	0.6	1.7
Neighborhood Park	35	0.1	0.3
Total	**2,530**	**8.1**	**14.8**

Notes:
(1) Source: 2006 Recycled Water Master Plan (Carollo, 2006) unless noted otherwise.
(2) Demands based on 2016 billing data.
(3) Butterfield and Rancho San Gorgonio demands estimated by respective developers.

Hydraulic Modeling

Three separate hydraulic models were utilized for the analysis of the potable water, wastewater collection, and recycled water systems. A summary of the hydraulic models updates is provided below. Additional details regarding the water and recycled water model updates, as well as the wastewater collection model development and calibration, are provided in Chapter 4 of the IMP.

The City's potable water hydraulic model was developed in H_20Map® Water in 2002 by MWH. Carollo Engineers, Inc. (Carollo) updated the H_2OMap® Water model for the 2015 Review of Ranch San Gorgonio Study, the 2015 Water System Storage Analysis, and the 2016 Chromium 6 Well Study. As part of the model update for this IMP, the potable water model was converted to the Infowater ® Version 12.3 Update #6. The hydraulic model was rebuilt using the City's Geographic Information Systems (GIS) Data since the City's GIS data was digitized according to as-built documents and City staff input. City staff provided details on the operation set points of facilities. Demands were allocated based on geocoding parcel demand data. The model updates included the geospatial allocation of demands, creation of demand sets for existing, 2025, and 2040 demand conditions, inputting diurnal patterns, updating pipeline network. In addition facility input was added for pump stations, wells, and reservoirs. The updated model was then calibrated using pressure logger and fire flow testing data collected in the field as part of this IMP. The updated calibrated model was used for the potable water system analysis presented in this IMP.

The City's wastewater collection system hydraulic model was previously developed in H2OMAP® Sewer. The hydraulic model was converted to InfoSWMM®. Using the converted hydraulic model as a basis, the hydraulic model network was updated using the City's GIS. The model updates included the geospatial allocation of wastewater flows, creation of wastewater flow sets for existing, 2025, and 2040 demand conditions, inputting diurnal patterns; updating pipeline network. In addition, facility input was added for all lift stations and diversion structures. The updated model was then calibrated for dry- and wet-weather conditions using flow-monitoring data collected as part of this IMP. The updated calibrated model was used for the wastewater collection system analysis presented in this IMP.

The City's recycled water hydraulic model was developed in H_20Map® Water in 2006 by Carollo. Since the City did not have an existing system at the time, the model was developed based on the different scenarios and alternatives evaluated at that time. The model was converted to InfoWater® Version 12.3 Update #6. The model updates included an update of demands, creation of demand sets for existing, 2025, and 2040 demand conditions, inputting diurnal patterns, updating the pipeline network based on the decision to have the system south of the I-10, and adding new non-potable wells. In addition facility input was added for pump stations. Since the City does not have an existing system, the model was not calibrated as part of this IMP.

Potable Water System Evaluation

Existing Potable Water System

The City potable water is primarily supplied from groundwater wells with a total capacity of 14,950 gpm. However, three (3) of these wells (Well M7, M12, 24) are currently non-potable wells, resulting in a total potable water capacity of 11,500 gpm. In the future, these non-potable wells may be converted for potable use. In addition, the City purchases imported water from the San Gorgonio Pass Water Agency to recharge to the Beaumont Basin at Beaumont Cherry Valley Water District's (BCVWD) Noble Creek spreading facility. Based on the City's 2015 UWMP, the City recharged approximately 694 afy in year 2015. Although the City purchases imported water, the imported water supply connection is only used for recharge.

The potable water distribution system consists of 165 miles of pipeline and includes 19 groundwater wells, 8 storage reservoirs, 2 booster pumping stations, 5 pressure reducing valve stations, and 6 pressure zones. The City's existing water distribution system is depicted on Figure ES.3.

Supply Analysis

Currently, 100 percent of the City's potable water system is supplied by groundwater from the wells, which can supply up to 11,500 gpm. In addition, the City has an interconnection with BCVWD with an estimated supply of 1,000 gpm. As part of this IMP, a supply analysis was performed under two different scenarios: largest supply out of service and extreme drought conditions.

Foothill East and Foothill West are the only pressure zones with excess supply under both scenarios. While the other pressure zones are deficient, all of the deficiencies can be resolved from using existing PRVs to convey the excess water in Foothill East and Foothill West to the lower zones. No recommendations were made for existing conditions. New wells are recommended for future conditions. The findings and recommendations of the analyses is described in detail in Chapter 6.

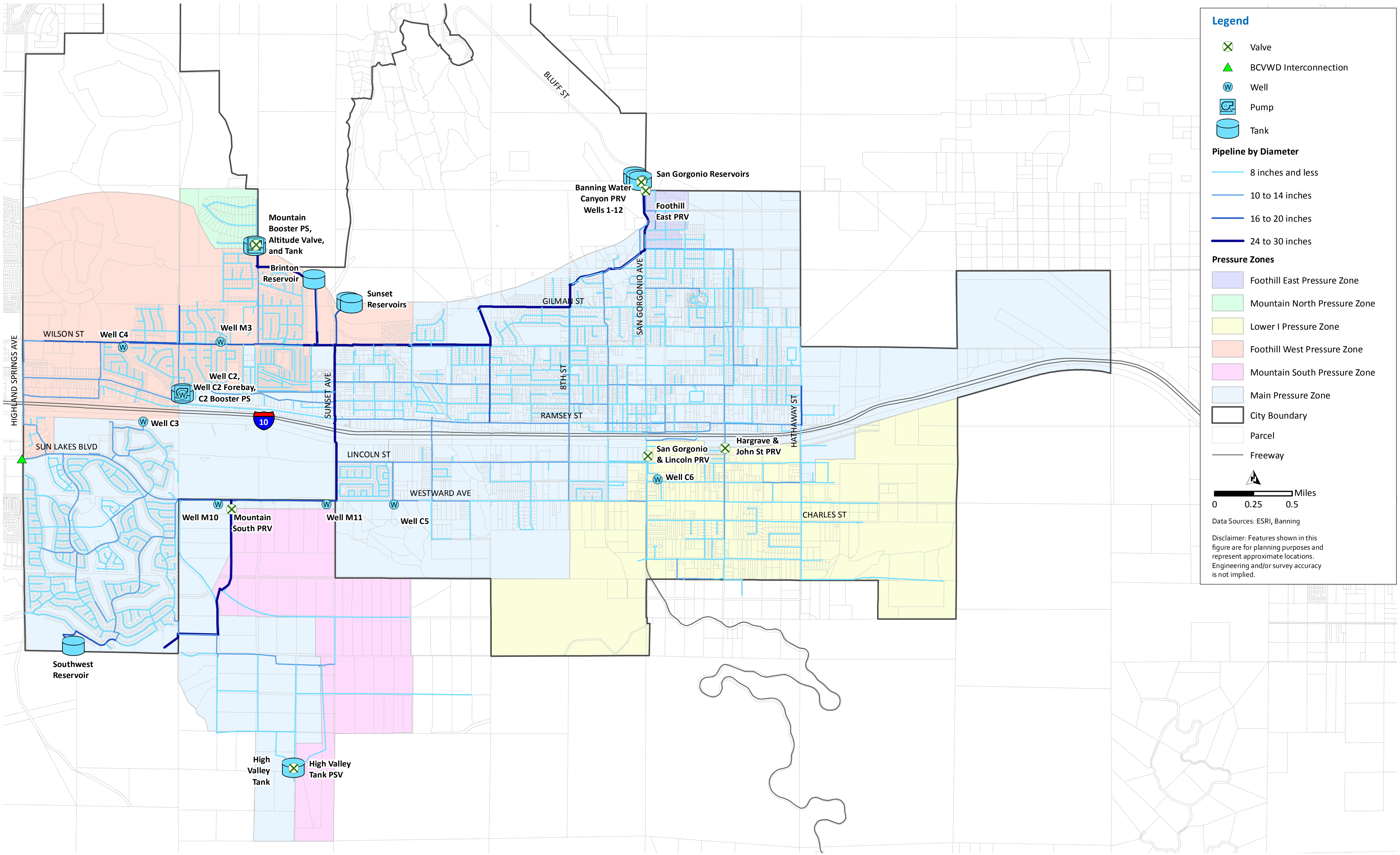

Figure ES.3 Existing Potable Water System as Modeled

Distribution System Evaluation

The water distribution system was evaluated using the hydraulic model based on the criteria described in Chapter 5 to determine system deficiencies and identify improvement projects to address these deficiencies. The distribution system evaluation included system pressures, pipeline velocity, fire flow, storage, and pump station capacity analyses under the existing, 2025, and 2040 demand conditions. The findings and recommendations of these analyses are described in detail in Chapter 6.

Water System Rehabilitation

A condition assessment was performed on June 7, 2017 by the Carollo assessment team for eight well sites, five reservoir sites, and two PRV stations that were identified by City staff as the most critical facilities of the potable water system. Projects were identified for the near-term (year 2025) and the long-term (year 2040). A summary of the recommended CIP projects listed in order of priority for the potable water facilities is provided in Chapter 6. In addition, the City's an age-based pipeline replacement analysis was reviewed and included in Chapter 6.

Other Improvements

Other miscellaneous improvement projects have been recommended to optimize the operation of the City's potable water system or provide reliability.

Water System Recommendations

In summary, the following major water system improvement projects are recommended and included in the CIP for the planning horizon of this IMP:

- **Supply Improvements**:
  - New Well C8 and Well C9, which feed the Main Zone and the Proposed Upper Main Zone.
  - The conversion of Well M7 and Well M12 from non-potable to potable.
  - VFDs at Well C6 and existing Well C8.
  - Approximately 0.4 miles of 12-inch diameter transmission mains.
- **Capacity and Reliability Improvements:**
  - The replacement of seven (7) existing PRVs to rezone the Main Zone to the Upper and Lower Main Zones.
  - Twenty-three (23) fire flow projects ranging from 8- to 12-inches in diameter and a total length of 30,000 feet (5.7 miles). This includes a PRV and check valve.
  - Five (5) new reservoirs with a combined capacity of 11.5 MG, which include Main Reservoir 1, Foothill West Reservoir 1, Mountain North Reservoir 1, Upper Main Reservoir 2, and Zone 1A Reservoir (or Upper Butterfield Reservoir).
  - Four (4) pump station projects with a combined capacity of 460 hp, which includes the Mountain Booster PS Upgrade, Foothill West PS, Mountain 2PS, and Zone 1A PS (or Upper Butterfield PS).
  - Approximately 6.0 miles of transmission main ranging in diameter from 12- to 24-inches.
- **Repair and Rehabilitation Improvements:**
  - A total of 69.1 miles of pipeline replacement due to estimated useful life.

  - Site Improvements at five (5) reservoir sites, two (2) PRV stations, and eight (8) well sites.
  - Multi-Site Rehabilitation Projects.
- **Other Projects** (See Chapter 6).

The proposed potable water system improvement projects are depicted on Figure ES.4 and the capital costs are discussed in Section ES.13 .

Build-out Water System Recommendations

In addition to improvements within the planning horizon of this IMP, the following major water system improvement projects are recommended and included in the CIP based on preliminary analyses for build-out conditions:

- **Supply Improvements:**
  - Three (3) new wells, which include Well C10, Well C11, and Well C12.
  - Approximately 0.6 miles of 12-inch diameter transmission mains.
- **Capacity and Reliability Improvements:**
  - Four (4) new reservoirs with a combined capacity of 13.5 MG, which includes Foothill West Reservoir 2, Upper Main Reservoir 3, Black Bench Reservoir 1, and Loma Linda Reservoir 1.
  - Two (2) pump station projects with a combined capacity of 160 hp, which includes the Loma Linda PS and Black Bench PS.
  - Approximately 3.8 miles of 18-inch diameter transmission main.

Wastewater Collection System Evaluation

Existing Wastewater Collection System

The City's wastewater collection system consists of approximately 112 miles of gravity sewer mains, and four lift stations that collect and convey wastewater to the City's wastewater treatment plant (WWTP). Sewer pipelines range in diameter from 4-inches to 30-inches, with 8-inch diameter pipelines accounting for 78 percent of the city's gravity sewer. A vast majority of the pipelines are vitrified Clay pipe (74 percent). A majority of the collection system pipelines with age related information were installed between 1980 and 2000. The City's existing wastewater collection system is depicted on Figure ES.5.

The City's wastewater collection system drains primarily from west to east. The average annual flow of wastewater in the previous five years is approximately 2.02 mgd.

Existing Wastewater Treatment Facility

All wastewater flows collected within the City's service area are currently treated at one facility, the Banning WWTP. The WWTP is designed to treat wastewater to secondary standards and consists of the following processes: headworks, screening, grit removal, two primary clarifiers, two trickling filters, and two secondary clarifiers. The plant currently discharges the effluent to percolation ponds.

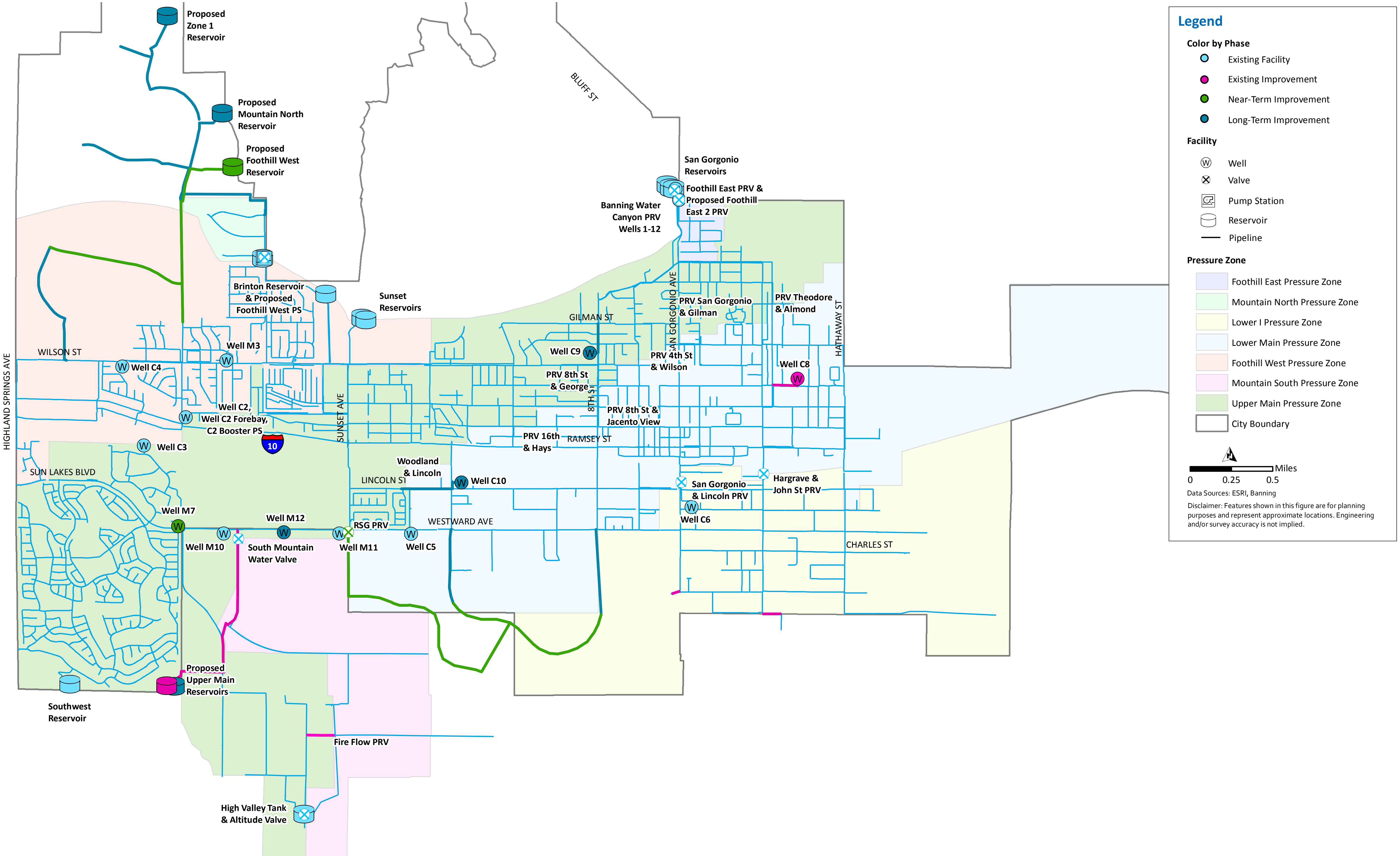

Figure ES.4 Proposed Water System Improvements

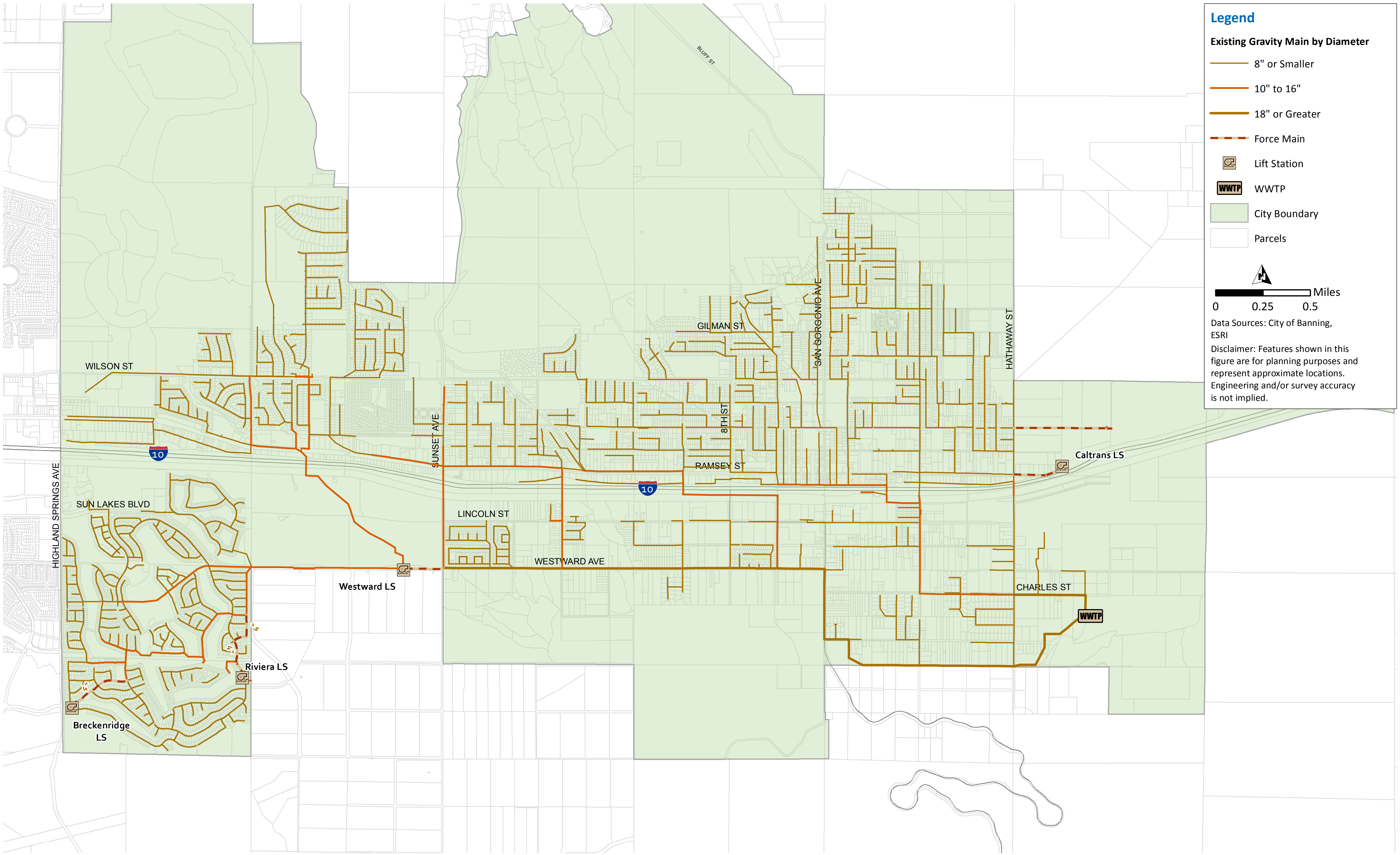

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Figure ES.5 Existing Wastewater Collection System

Wastewater Collection System Evaluation

For the existing sewer collection system, the peak wet weather flow (PWWF) was routed through the hydraulic model. In accordance with the established flow depth criteria for existing sewers, pipelines with a maximum flow depth to pipe diameter (d/D) ratio greater than 0.92 were identified as capacity deficient. Under existing conditions, the analysis showed that there are there are seven (7) gravity main alignments and one sewer lift station that require upsizing to address capacity deficiencies under PWWF conditions.

The analysis of the future system was performed in a manner similar to the existing system analysis. As part of the future system analysis, the planning years 2025 and 2040 were evaluated. In addition, a preliminary analysis was performed to identify improvements under Build-Out PWWF conditions. Therefore, the term future is a general reference to planning years 2025, 2040, and Build-Out. The future analysis identified two (2) capacity deficiencies under 2040 conditions and four (4) capacity deficiencies under Build-Out conditions. The future analysis also evaluated preliminary alignments for new development. A total of nine (9) growth related projects were identified under near term and long, while thirteen (13) growth related projects were identified under Build-Out.

Wastewater System Condition Assessment

A condition assessment was completed for two lift stations as part of the IMP. The condition assessment was conducted on June 7, 2017. The assessment consisted of visual inspection of mechanical, structural, and electrical equipment. The two lift stations evaluated in the condition assessment included the Caltrans Lift Station and the Westward Lift Station.

Satellite Treatment Plant for Pardee Development

As an alternative to the City's WWTP receiving all the wastewater within the projected service area, this IMP evaluated the potential use of a satellite facility to treat Butterfield's wastewater. The Butterfield Satellite Plant (Satellite Plant) would be located near the intersection of Highland Home Road and Wilson Street. The Satellite Plant was evaluated under future and Build-Out conditions.

Wastewater System Recommendations

In summary, the following major wastewater system improvement projects are recommended and included in the CIP:

- **Capacity Improvements:**
  - Sixteen (16) gravity main projects ranging in diameter from 8 to 30 inches with a total length of 40,500 feet (or 7.6 miles).
  - An interim upgrade to the Westward Lift Station, which includes an increase in capacity of 4.4 mgd and a force main upgrade with a pipeline diameter of 12-inches and length of 1,500 feet (or 0.28 miles).
  - Two (2) lift stations projects with a total capacity of 2.52 mgd, which include the Distribution Center Lift Station and the Business Park Lift Station. One (1) force main project with a pipeline diameter of 8-inches a total length of 4,000 feet (or 0.78 miles) as well as a bypass pipeline project is required for these new lift stations.

- **Repair and Rehabilitation Improvements:**
  - Annual sewer replacements.
  - Caltrans Lift Station and Westward Lift Station site improvements.
- **Treatment Plant Related Improvements:**
  - Five (5) treatment plant improvement projects, which include digester cleaning, heat exchanger repairs, boiler gas control valves, digester gas pipeline, and WWP upgrade to tertiary treatment.
- Other Projects:
  - Lift Station Telemetry
  - Septic Removal

The proposed sewer system improvement projects are depicted on Figure ES.6 and the capital costs are discussed in Section ES.13.

Build-out Wastewater Collection System Recommendations

In addition to improvements within the planning horizon of this IMP, the following major wastewater collection system improvement projects are recommended and included in the CIP based on preliminary analyses for build-out conditions:

- Thirteen (13) gravity main projects ranging in diameter from 8 to 24 inches with a total length of 89,500 feet (or 17.0 miles).
- Three (3) lift station projects with a total capacity of 0.90 mgd, which include Porter Street Lift Station, Roadrunner Trail Lift Station, and Bluff Street Lift Station. In addition, three (3) 6-inch diameter force main projects with a total length of 6,500 feet (or 1.2 miles).

Satellite Treatment Plant Alternative Recommendations

As part of this IMP, an alternative was evaluated with a satellite treatment plant at the Butterfield development to serve recycled water. With the addition of the Satellite Treatment Plant, the following recommendations will be required:

- Three (3) projects within the gravity system improvements may be altered in diameter and/or length, which includes the Butterfield Offsite Trunk, Porter Street Trunk, and South WWETP Trunk Parallel.

Recycled Water System Evaluation

Existing Recycled Water System

The City has constructed approximately 2.2 miles of 24-inch diameter pipeline and has begun constructing an additional 3.4 miles of pipeline to connect to the wastewater treatment plant (WWTP). The City's existing recycled water system and planned pipelines are shown on Figure ES.7.

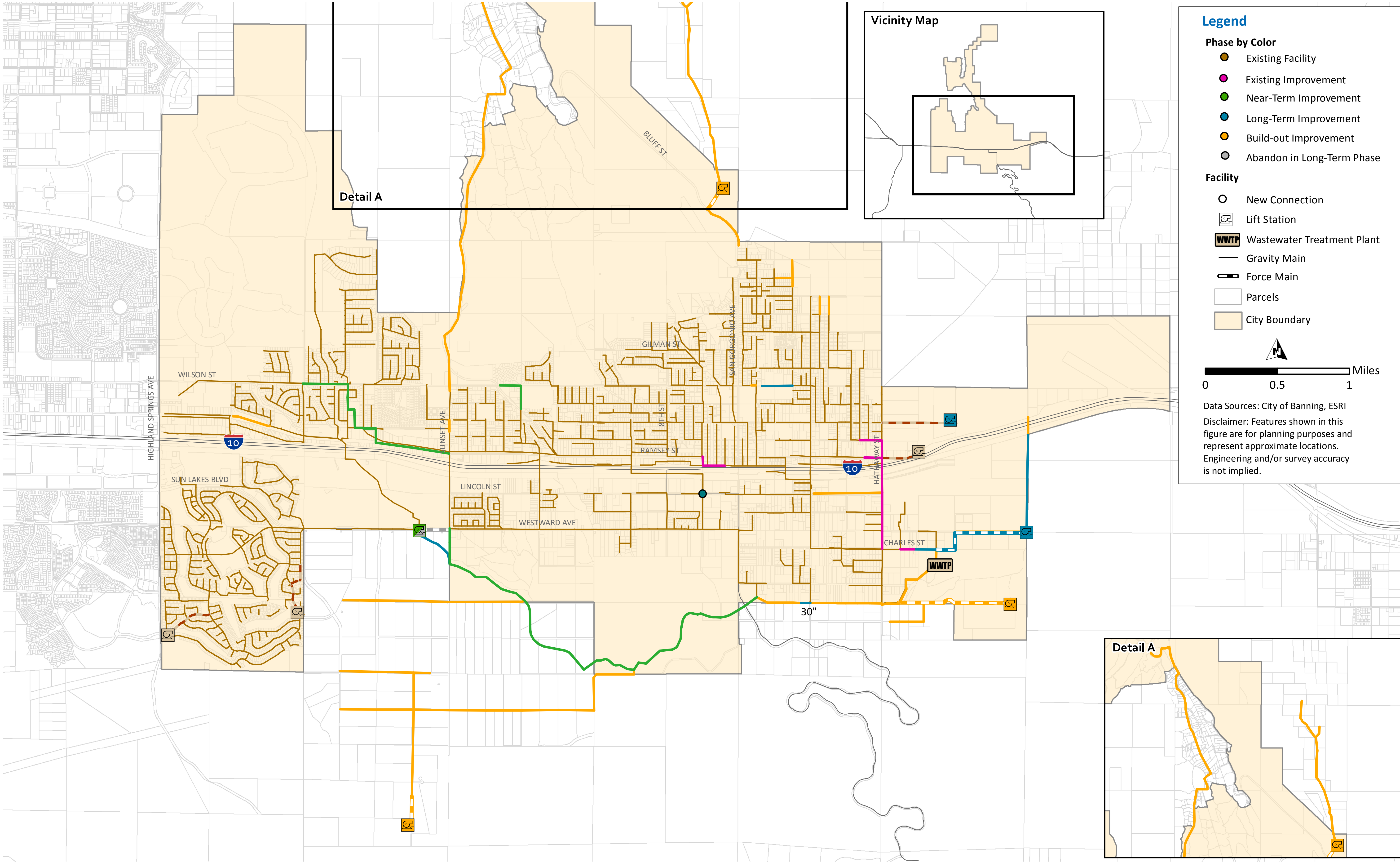


Figure ES.6 Proposed Wastewater System Improvements without Satellite Plant

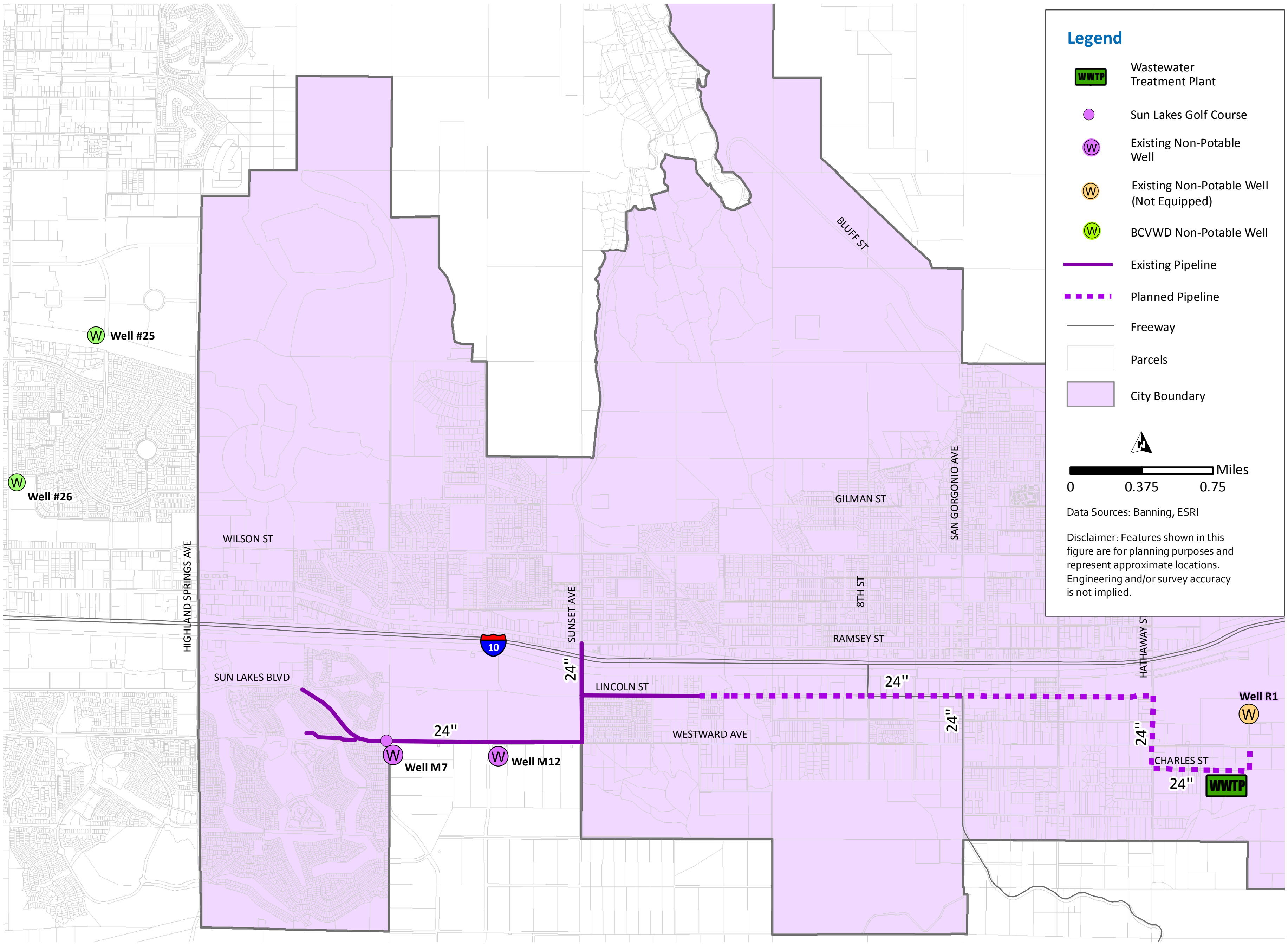


Figure ES.7 Existing Recycled Water System

Recycled Water System Evaluation

For the future system evaluation, the hydraulic model was used to develop potential system expansion alternatives that maximize the usage of recycled water within the City's service area, while meeting the evaluation criteria discussed in Chapter 5. A total of 6 alternatives were evaluated with a combination of non-potable reuse (NPR) and indirect potable reuse (IPR). The hydraulic model was used to size pipelines and cost estimates were developed for each segment. The analysis shows that the estimated cost for each alternative ranges from $717 per acre-foot to $1,300 per acre-foot. Based on the City’s objective to maximize the use of recycled water and improve local supply reliability, a hybrid alternative with both NPR and IPR are recommended and included in the CIP. The recommended alternative (Alternative 5) includes NPR and recharge to two potential basins: WWTP and Five Bridges Basin. The implementation of this alternative is proposed in the following phases:

- Phase 1: The NPR system would be constructed, starting with equipping Well R-1 and connecting Well R-1 to Lions Park and Banning High School.
- Phase 2: The backbone pipeline can be extended to the RSG development. The WWTP expansion is to be completed and the WWTP recycled water pump is constructed.
- Phase 3: The backbone pipeline can be extended to connect to the existing pipelines in Lincoln Street and connect Dysart Park to the main recycled water system.
- Phase 4: The City can begin the construction of the pipelines to the recharge basins for IPR use.

The proposed recycled water system projects are depicted on Figure ES.8 and the capital costs are discussed in Section ES.13 .

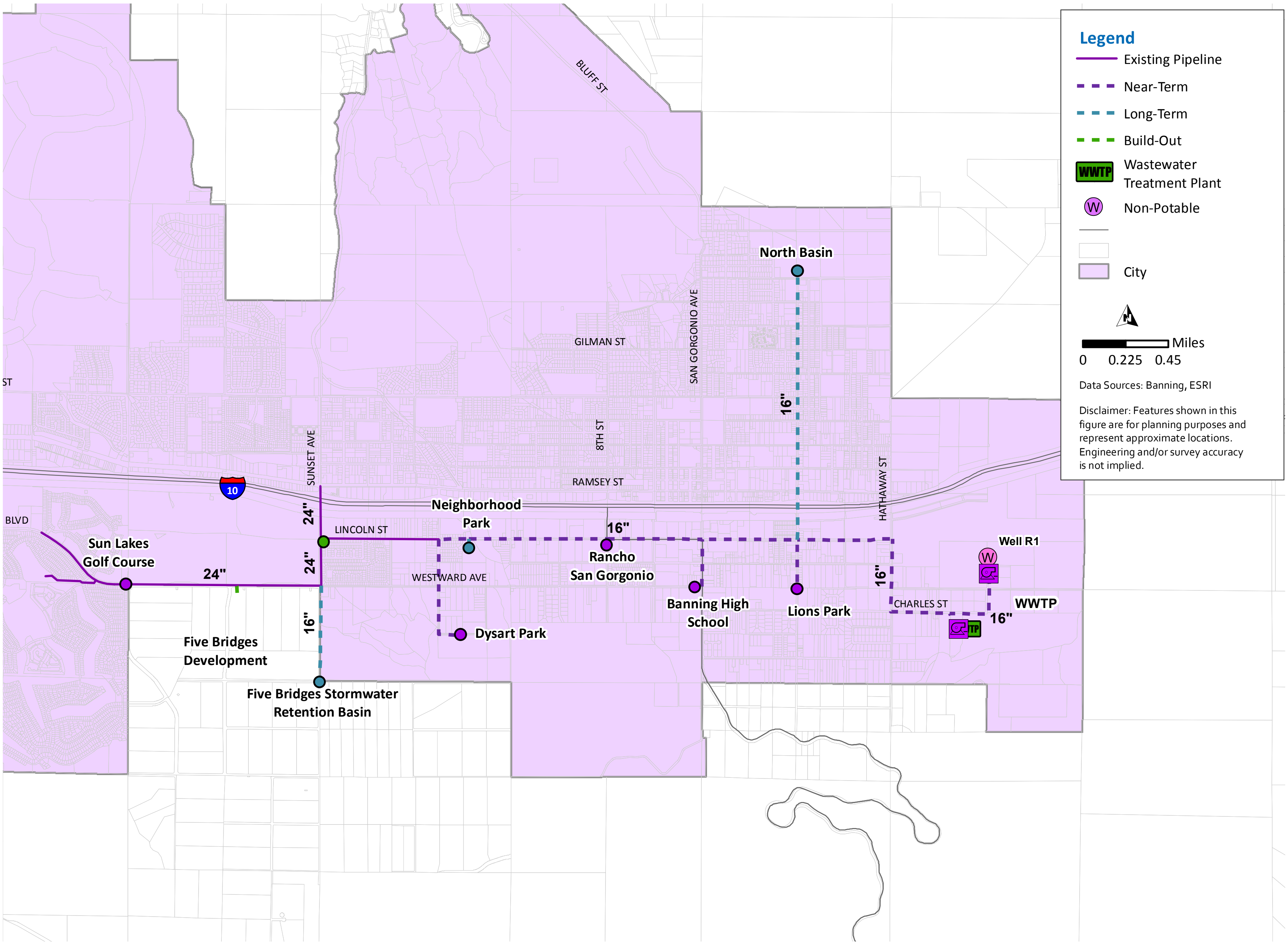
Satellite Treatment Plan Recommendation

As part of this IMP, a potential satellite treatment plant at the Butterfield development was analyzed. The flows at the Butterfield Development are not sufficient to meet recycled water demands and will need to be supplemented with additional flows from nearby neighborhoods or potable water. A satellite plant would also add a second treatment plant for City staff to operate and maintain, which increases operational cost and requires additional staff. Thus, it is not recommended to build a satellite plant at the Butterfield Development. Alternative supply sources to serve Butterfield with non-potable water are discussed in Chapter 8.

Capital Improvement Plan

The Capital Improvement Plan (CIP) is the foundation of the City's long-range capital investment and financial planning. The CIP establishes a specific list of projects to be completed for capital replacements and improvements. Looking ahead through the CIP provides an opportunity to prioritize capital expenditures, manage cash flows, project reserve balances, and establish future revenue requirements that ultimately determine rates, fees, and charges necessary to maintain the facilities for potable water, wastewater, and recycled water systems.

It should be noted that the current water rates will make it difficult to fund the projects within the near-term planning period as discussed in Chapter 9. Therefore, the CIP will need to be revised periodically to push projects out to later years. Other select projects may also be moved at the discretion of City staff. Future rate increases to raise capital funds, additional contributions from developers, and grant funding can potentially accelerate projects to the near-term planning phase.


Legend
Existing Pipeline
Near-Term
Long-Term
Build-Out
Wastewater Treatment Plant
Non-Potable
City
Miles
0 0.225 0.45
Data Sources: Banning, ESRI
Disclaimer: Features shown in this figure are for planning purposes and represent approximate locations. Engineering and/or survey accuracy is not implied.
BLUFF ST
North Basin
GILMAN ST
SAN GORGONIO AVE
16"
8TH ST
SUNSET AVE
RAMSEY ST
HATHAWAY ST
10
Neighborhood Park
24"
LINCOLN ST
16"
BLVD
Sun Lakes Golf Course
24"
24"
Rancho San Gorgonio
WESTWARD AVE
Well R1
16"
Banning High School
Lions Park
CHARLES ST
WWTP
16"
16"
Dysart Park
Five Bridges Development
Five Bridges Stormwater Retention Basin

The integrated CIP for the City's potable water, wastewater, and recycled water systems is summarized in Table ES.6. The individual CIPs for the three systems are presented in Chapter 9 of this IMP, and a complete project list with associated cost estimates for the potable water, wastewater, and recycled water systems can be found at the end of Chapter 9 in Table 9.10, Table 9.11, and Table 9.12, respectively. In addition, Figure 9.9, Figure 9.10, and Figure 9.10 present the locations of the proposed CIP projects that correspond to Table 9.10, Table 9.11, and Table 9.12 in Chapter 9.

As shown in Table ES.6, the integrated CIP costs for all three systems through planning year 2040 is estimated to be about $348.4 million, respectively. As shown on Figure ES.9, the potable water system CIP comprises the largest portion of cost with $570.2 million (80 percent) of the total combined CIP, while the wastewater system CIP represents the second largest cost with $99.2 million (14 percent). As described in Chapter 9, the vast majority of costs are related to capacity improvements, age-based replacements of pipelines that are projected to reach the end of their useful life, and treatment plant improvements.

The phasing of the integrated CIP by system is depicted on Figure ES.10. As shown on this figure, about $193.2 million of project costs are included in the near-term phase and $155.1 million are scheduled for the long-term phase. Nearly 51 percent (or $362.9) of the improvement projects are anticipated to occur in the build-out phase, which is outside of the planning horizon of this IMP.

It is anticipated that a combined total of approximately $88.5 million in developer funding will be provided within the near-term and $41.1 million within the long-term planning phases. With developer funding, the City's anticipated average annual expenditures equate to $13.1 million in the near-term phase and $7.6 million in the long-term phase, or an overall average of $9.5 million within the 23-year planning horizon of this IMP.

Table ES.6 Integrated CIP by System and Phase

Project Type	Near-Term 2018-2025 ($ Million)	Long-Term 2026-2040 ($ Million)	Build-Out 2041 &Beyond ($ Million)(2)	Total ($ Million)
Potable Water System(1)	$108.7	$137.8	$323.8	$570.2
Wastewater System(2)	$48.3	$12.0	$38.9	$99.2
Recycled Water System(3)	$36.3	$5.3	$0.2	$41.8
Grand Total	**$193.2**	**$155.1**	**$362.9**	**$711.2**
Number of Years	8	15	N/A	N/A
Total Annual Cost ($/year)	**$24.2**	**$10.3**	**N/A**	**N/A**
Anticipated Developer Funding	$88.5	$41.1	$103.3	$232.9
City Funded CIP	$104.7	$114.1	$259.6	$478.3
City Annual Cost ($/year)	**$13.1**	**$7.6**	**N/A**	**N/A**

Notes:
(1) See Table 9.10.
(2) See Table 9.11.
(3) See Table 9.12.
(4) The costs per year do not include build-out since the implementation timeline is unknown and may be outside of the 2040 planning horizon.
(5) Numbers may vary slightly due to rounding.

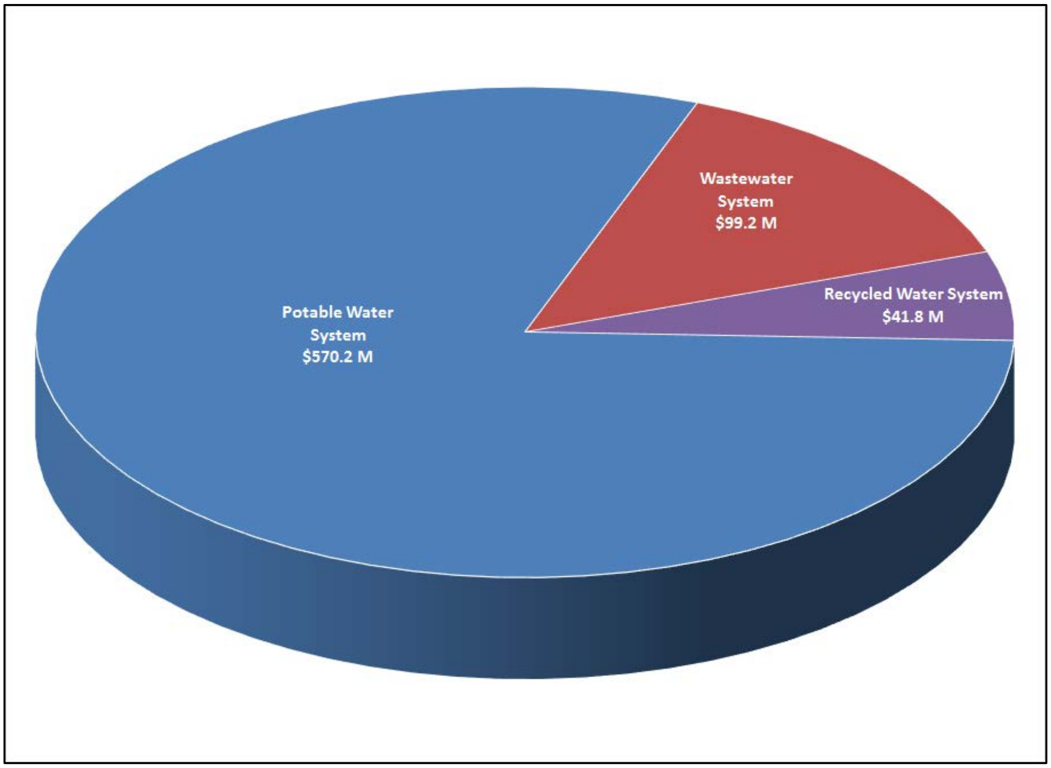


Figure ES.9 Integrated Systems CIP by Cost

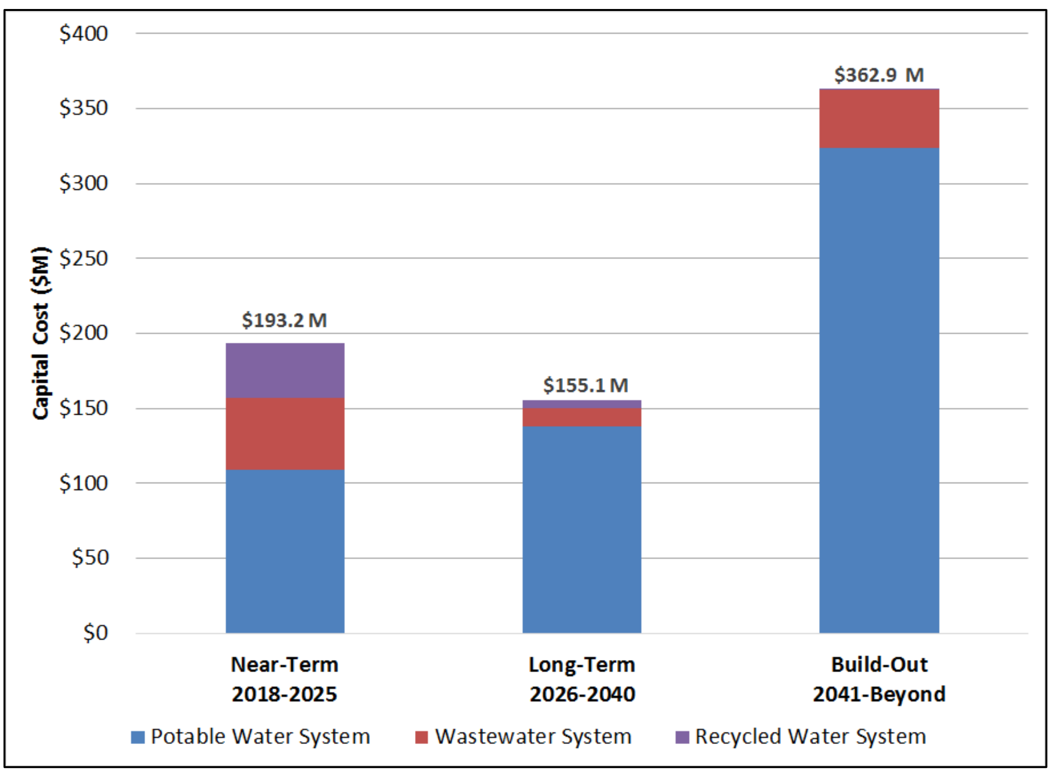


Figure ES.10 Integrated Systems CIP by Phase

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Chapter 1

INTRODUCTION

The City of Banning (City) has retained Carollo Engineers, Inc. (Carollo) to prepare this Integrated Master Plan (IMP). This IMP evaluates the performance and condition of the City's potable water, wastewater, and recycled water systems under existing and future conditions through year 2040. This chapter presents the purpose, objectives, and background of this IMP. A list of references used to prepare this IMP is provided in Appendix A.

1.1 Background

The City's Sewer and Recycled Water System Studies (2006 Studies) were last updated by Carollo Engineers, Inc. (Carollo) in year 2006. The 2006 Sewer System Study included a capacity evaluation, recommended improvements to mitigate capacity deficiencies, and a summary of capital costs associated with recommended improvements. The 2006 Recycled Water System Study defined capital improvement projects required to serve potential recycled water customers where it was cost effective.

Since the completion of the 2006 Studies, significant changes have occurred within the City's service area, including changes in water demands, wastewater flow characteristics, and potential recycled water customers. The economic recession during year 2007 through year 2009 slowed down growth within the City. Therefore, the forecasts in the 2006 Studies are too aggressive and outdated. In addition, multi-year drought conditions led to mandated statewide conservation, resulting in significant water demand reductions. Consequently, wastewater flows also decreased, which leaves less flow available for recycled water use.

The City recognizes the importance of updating the 2006 Studies and developing an integrated Capital Improvement Plan (CIP) that prioritizes potable water system infrastructure upgrades, wastewater system infrastructure upgrades and expansion projects, and recycled water infrastructure construction.

1.2 Goals and Objectives

The purpose of this IMP is to update the 2006 Studies and extend the planning horizon to year 2040. In addition, the City has identified developments that are planned, but will not likely develop until after the planning period of this IMP (after year 2040). The goal of this IMP is to assist the City in the planning and development of potable water, wastewater, and recycled water system facilities. The objectives of this IMP are:

1. Identify the existing, near-term (year 2025), long-term (year 2040), and build-out potable water demands, wastewater flows, and recycled water demands.
2. Define planning and evaluation criteria for the City's potable water, wastewater, and recycled water systems.
3. Determine where deficiencies exist in the City's potable water and wastewater systems under existing, long-term (year 2040), and build-out conditions.

4. Identify necessary recycled water system facilities to serve the City's potential recycled water customers.
5. Prepare an integrated CIP with phasing of recommended improvements and an integrated phasing plan.

1.3 City Boundary

The City, incorporated in 1913, occupies approximately 23.2 square miles astride Interstate 10 in the San Gorgonio Pass. The City is located in the northwest portion of Riverside County and approximately 30 miles east of downtown Riverside. The City is bordered by the City of Beaumont to the west, the Morongo Band of Mission Indians to the east, the San Gorgonio Mountains to the north, and the San Jacinto Mountains to the south. The City boundaries and neighboring cities can be found in Figure 1.1.

1.4 Report Organization

Chapter 1 – Introduction: This chapter presents the project background, goals, and organization of this IMP.

Chapter 2 – Land Use and Population: This chapter presents a discussion of the land use classifications, historical population trends, and projected populations for the planning period of this IMP.

Chapter 3 – Water Demand, Wastewater Flow, and Recycled Water Forecasts: This chapter summarizes the existing potable water demands for the City's potable water system. This chapter also discusses demand-forecasting methodology and provides a summary of potable and recycled water demand projections. In addition, this chapter presents the historical and existing wastewater flows and characteristics for the City's wastewater collection system. Finally, future wastewater flow projections are presented.

Chapter 4 – Hydraulic Model Update: This chapter discusses the water, wastewater collection, and recycled water models used for the preparation of this IMP. This chapter summarizes updates made to the existing hydraulic models, including a summary of the modeling software selection, a description of the modeled systems, the hydraulic model elements, the model creation process, and the model calibration process.

Chapter 5 – System Evaluation Criteria: This chapter presents the planning criteria and methodologies for the analysis used to evaluate the existing water, wastewater collection, and recycled water systems and associated facilities. The criteria described in this chapter are used to identify existing system deficiencies and size future improvements and expansions in subsequent chapters.

Chapter 6 – Potable Water System Evaluation: This chapter presents an overview of the City's existing potable water system, existing system analysis, and future system analysis. The chapter describes the existing potable water distribution system and facilities. In addition, this chapter presents the results of the capacity evaluation of the existing potable water system and the proposed improvements to mitigate the identified deficiencies. As part of the existing system analysis, this chapter summarizes the results of the condition assessment of the City's potable water system facilities performed as part of this IMP. Following the existing system analysis, the results of the capacity evaluation of the potable water system to meet the projected water demands described in Chapter 3 is discussed. This chapter also identifies the proposed improvements that are required to meet the planning and evaluation criteria under future demand conditions.

Chapter 7 – Wastewater Collection System Evaluation: This chapter presents an overview of the City's existing wastewater collection system, existing system analysis, and future system analysis. The chapter describes the existing wastewater collection system facilities. In addition, this chapter presents the results of the capacity evaluation of the existing wastewater collection system and the proposed improvements to mitigate the identified deficiencies. As part of the existing system analysis, this chapter summarizes the results of the condition assessment of the City's wastewater collection system facilities performed as part of this IMP. Following the existing system analysis, the results of the capacity evaluation of the wastewater collection system based on the projected flows described in Chapter 3 is discussed. This chapter also identifies the proposed improvements that are required to meet the planning and evaluation criteria under future flow conditions.

Chapter 8 – Recycled Water System Evaluation: This chapter presents an overview of the City's existing recycled water system, supply sources, and an analysis of recycled water system alternatives. The chapter describes the existing recycled water system and supply sources. In addition, this chapter presents the Non-Potable Reuse (NPR) and Indirect Potable Reuse (IPR) alternatives for the recycled water system to maximize the future use of recycled water in the City. This chapter identifies the proposed projects for the recommended alternative to meet the planning and evaluation criteria under future demand conditions.

Chapter 9 – Capital Improvement Plan: This chapter presents an integrated CIP for the City's water, sewer, and recycled water systems. This program incorporates all recommended projects identified in the existing system analysis, future system analysis, and the condition assessment.

1.5 Acknowledgements

We would like to thank the following City staff for their assistance and oversight of this project:

- Art Vela, P.E., Director of Public Works/City Engineer
- Luis Cardenas, P.E., Senior Civil Engineer
- Perry Gerdes, Water/Wastewater Superintendent

The following Carollo staff members were principally involved in this project:

- Inge Wiersema, P.E., Project Manager
- Ryan Orgill, P.E., Project Engineer
- Amy Martin, Project Engineer
- Joaquin Ramirez, Staff Engineer
- Ryan Hejka, Staff Engineer
- Aimee Zhao, Staff Engineer
- David Baranowski, P.E., Condition Assessment Lead
- Pei-Shin Wu, P.E., Staff Engineer
- James Doering, P.E., S.E., Structural Engineer

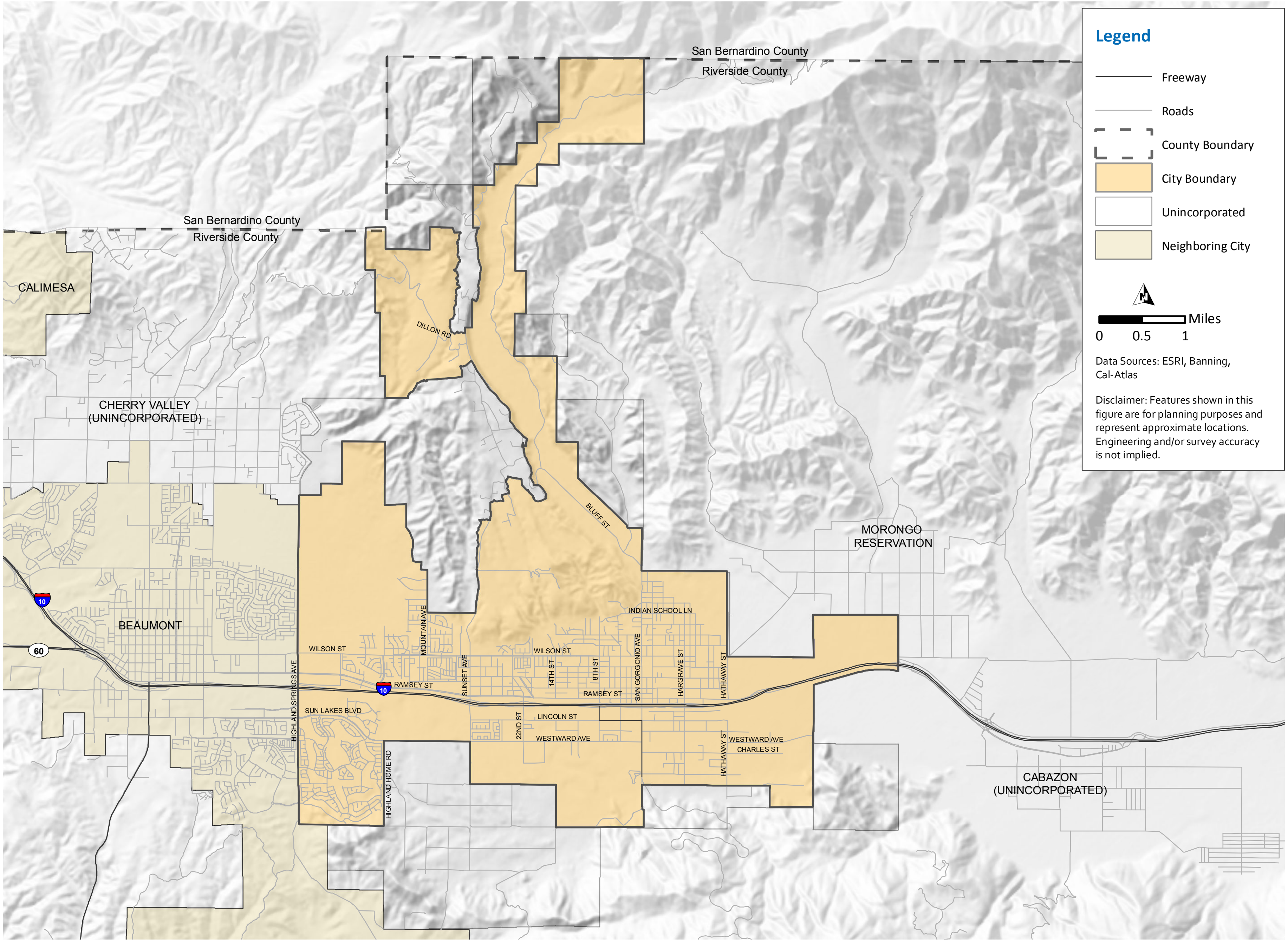

Figure 1.1 City Boundary

Chapter 2

LAND USE AND POPULATION

This chapter presents the study area of this IMP, including the City's different service areas for the potable water, recycled water, and wastewater collection systems. The land use classifications, planned developments, and information obtained on future land use are discussed next. This chapter concludes with a description of the historical population trends within the City and projected populations for the planning period of the IMP. Details presented in this chapter on new developments and population projections form the basis for the demand and flow projections presented in Chapter 3.

2.1 Study Area

The City is located in northern Riverside County in Southern California, approximately 25 miles east of downtown Riverside and 85 miles from downtown Los Angeles. The City encompasses 23.2 square miles astride Interstate 10 in the San Gorgonio Pass and is bounded by the City of Beaumont on the west, the Morongo Band of Mission Indians in the east, the San Bernardino Mountains to the north, and the San Jacinto Mountains to the south. The study areas of each system indicate the areas that are being served and differ for each system.

2.1.1 Water Study Area

The City's potable water study area coincides with the City boundary, which includes approximately 23.2 square miles, and 2.4 square miles outside of the City boundary as shown on Figure 2.1. The total potable water study area is approximately 25.6 square miles.

2.1.2 Wastewater Study Area

The City's wastewater study area consists of three basic boundaries identified in the General Plan and defines the City's current and future limits. These boundaries include the City limits, the Sphere of Influence (SOI), and Planning Area (PA) as shown on Figure 2.2. The SOI, which is part of the existing wastewater service area, extends outside of the City limits and includes an additional 8.5 square miles. The PA consists of 5.1 square miles of unincorporated lands outside of the City limits and SOI. The total area of the wastewater study area is approximately 36.8 square miles.

The City provides wastewater collection service to residents, businesses, and other institutions within its limits and in the surrounding unincorporated County lands. The existing sewer service area is estimated at 2,900 acres. Existing sewer customers are concentrated along Interstate 10, while the north and County areas in the south have remained rural and are largely on septic systems. As future development occurs, new sewer infrastructure will connect these areas to the existing sewer collection system.

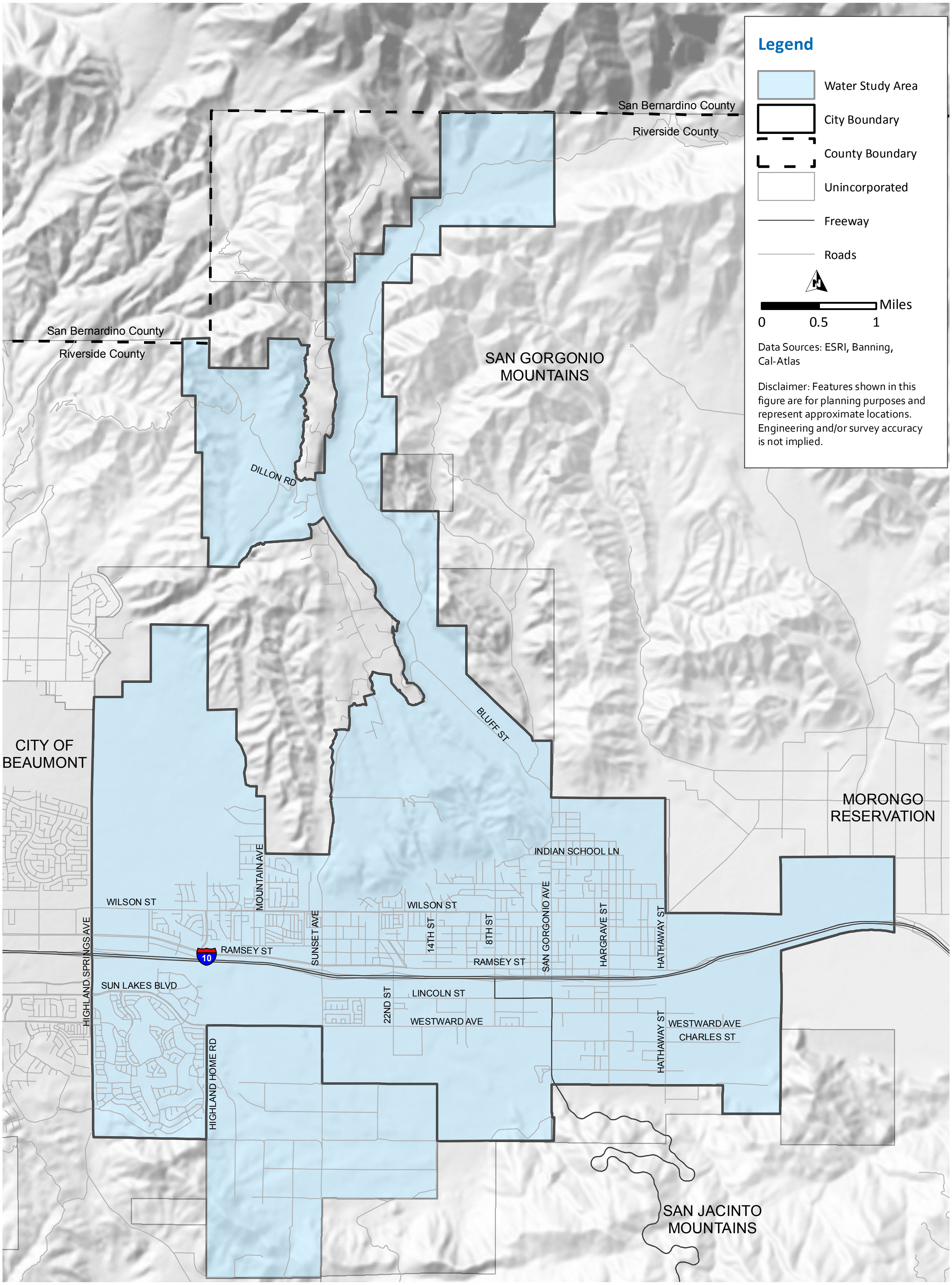

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Figure 2.1 Water Study Area

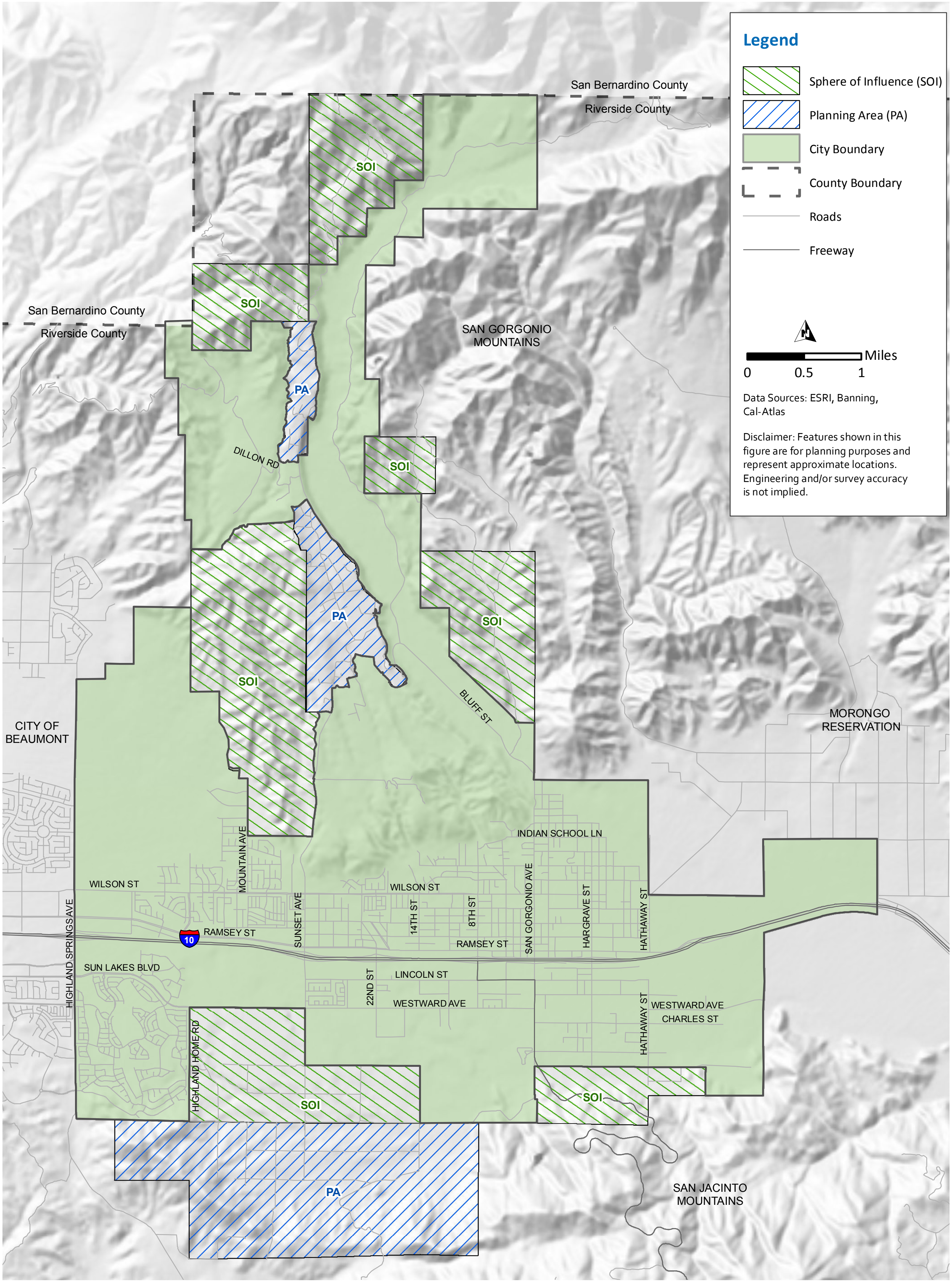

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Figure 2.2 Wastewater and Recycled Water Study Area

2.1.3 Recycled Water Study Area

The recycled water study area for this IMP coincides with the wastewater service area, which includes the City limits, SOI, and Planning Area shown on Figure 2.2. This study area may change as the system develops.

2.2 Climate

The City is located in the San Gorgonio Pass, which is characterized by short, mild winters, and hot, dry summers. Summers are hot and dry, while winters are cool with an average precipitation of about 17.8-inches per year. The study area is subject to significant variations in annual precipitation. Most of the annual precipitation occurs during the period from December through March. Temperatures range from an average minimum of 38 degrees Fahrenheit to an average maximum of 96 degrees Fahrenheit in July with annual minimum temperatures averaging 47 degrees Fahrenheit in January and annual maximum temperatures averaging 77 degrees Fahrenheit.

Evapotranspiration (ETo) is the quantity of moisture that is transpired by a reference plant, such as an irrigated grass lawn, and evaporated from soil. ETo is important to water resource management because irrigation requirements relate directly to ETo. Irrigators who are working to achieve maximum efficiency need to apply enough water to meet the crop's ETo demand. ETo for the City ranges from about 2.0 inches per month during the winter to more than 7 inches per month during the summer. Annual ETo is 59.1 inches per year. Monthly average ETo rates, rainfall, and temperature are summarized in Table 2.1.

Table 2.1 Climate

Month	Average Rainfall(2) (inches)	Average Minimum Temperature(2) (degrees F)	Average Maximum Temperature(2) (degrees F)	Average ETo(1) (inches)
January	3.52	38.4	60.3	2.27
February	3.4	38.8	63.1	2.74
March	3.12	39.9	65.8	4.33
April	1.44	42.7	71.9	5.27
May	0.55	47.5	78.6	6.64
June	0.14	52.2	87.5	7.3
July	0.23	58.2	95.5	7.94
August	0.27	58.8	95	7.63
September	0.51	55.5	90.1	6.12
October	0.65	49.1	80.1	4.19
November	1.72	42.9	69	2.7
December	2.26	39.2	61.7	2.0
Annual	**17.81**	**46.9**	**76.6**	**59.13**

Notes:
(1) Source: California Irrigation Management Information System (CIMIS), Hemet Station (239).
(2) Source: Western Regional Climate Center, Station No. 040609– Beaumont #2 (Period of record 08/1/1939-6/10/2016).

2.3 Land Use

The General Plan guides development within the City's planning boundary and establishes the long-range development policies. The General Plan also provides land use projections. Land use information is an integral component in determining the amount of water use and wastewater generation within the City. The type of land use in an area will affect the volume of water use and volume and character of the wastewater generation. Adequately estimating the water use and generation of wastewater from various land use types is important in sizing and maintaining effective water and sewer system facilities.

Table 2.2 Land Use Designation

Grouped Land Use Category	Land Use Code	General Plan Land Use Category [1]
Rural	RUR	Ranch/Agriculture – Hillside (10 ac min.) Ranch/Agriculture (10 ac min.) Rural Residential – Hillside (0-1 du/ac) Rural Residential (0-1 du/ac)
Very Low Density Residential	VLDR	Very Low Density Residential
Low Density Residential	LDR	Low Density Residential
Medium Density Residential	MDR	Medium Density Residential (0-10 du/ac) Mobile Home Parks
High Density Residential	HDR	High Density Residential (11-18 du/ac) High Density Residential-20/Affordable Housing Opportunity (20-24 du/ac) Very High Density Residential
Commercial	COM	Business Park Downtown Commercial General Commercial Highway Serving Commercial Professional Office
Industrial	IND	Airport Industrial Industrial Industrial – Mineral Resources
Open Space	OS-HP	Open Space – Hillside Preservation Open Space – Resources
Parks	OS-P	Open Space – Parks
Public Facilities	PF	Public Facilities – Airport Public Facilities – Cemetery Public Facilities – Fire Station Public Facilities – Government Public Facilities – Hospital Public Facilities – Railroad/Interstate
Schools	PF-S	Public Facilities - School

Notes:
(1) General Plan Land Use categories obtained from City's General Plan.

The City's most recent General Plan was adopted in year 2016 (Banning, 2016a). This plan classifies land use into 29 categories. For the purpose of this IMP, these categories were grouped into eleven (11) categories. The twenty-nine (29) land use categories and eleven (11) grouped categories are summarized in Table 2.2. Figure 2.3 illustrates the distribution of land use within the study area, including the SOI and Planning Area.

2.3.1 Known Future Developments

The City has plans for development of new communities, infill, and redevelopment of existing land. As shown in Table 2.3, the City has currently identified 6 master planned communities, 6 residential developments, and 3 commercial/industrial developments. The six master planned communities include a mixture of residential, public facilities, commercial, and open space. The City also has additional proposed residential, commercial, and industrial developments on record. For this IMP, the known future developments were identified as either near-term, long-term, or build-out depending on the anticipated completion. Near-term developments are assumed to be completed by year 2025, while long-term developments are assumed to be completed by year 2040. Build-out developments are planned, but not likely to develop until after the planning period of this IMP (after year 2040). These developments are considered in the build-out phase of the demand envelop, which are discussed in Chapter 3. The number of units and size of each planned development is summarized in Table 2.3, while the location of each development is shown on Figure 2.4.

Table 2.3 Known Developments

Development Name	Land Use	Residential Units	Size (Acres)	Build-out Year(1)
Master Planned Community				
Black Bench	Muti-use	1,500	2,452	Build-out
Five Bridges	Multi-use	1,924	639	Build-out
Little Europe	Multi-use	268	15	Build-out
Loma Linda	Multi-use	944	600	Build-out
Pardee Butterfield	Multi-use	4,862	1,528	2040
Rancho San Gorgonio	Multi-use	3,385	831	2040
Sub-Total	***N/A***	***12,883***	***6,065***	***N/A***
Residential				
Fiesta Development	Very Low Density	303	159	2025
St. Boniface	Low Density	172	65	2040
Wilson 97	Low Density	98	34.6	2025
RMG Residential	Low Density	48	10.7	2040
Kohavi	Low Density	2	1	Build-out
Our Savior Lutheran(2)	Medium Density	2	2.75	Build-out
Sub-Total	***N/A***	***625***	***273***	***N/A***

Table 2.3 Known Developments (Continued)

Development Name	Land Use	Residential Units	Size (Acres)	Build-out Year(1)
Commercial/Industrial				
Silverstone	General Commercial	N/A	47	2040
Banning Distribution Center	Airport Industrial	N/A	64	2040
Banning Business Park	Commercial/Industrial	N/A	65	2040
Sub-Total	***N/A***	***N/A***	***176***	***N/A***
Total	**N/A**	**13,508**	**6,514**	**N/A**

Notes:
(1) Some developments have agreements in place that allow for the extension of their build-out horizon. Conservative estimates were used for planning purposes, and developer provided phasing information was incorporated when available.
(2) This area is partially developed with a church.

As shown in Table 2.3, the known developments are estimated to result in 13,508 new residential units by build-out. Of the six master planned communities, two, namely Butterfield and Rancho San Gorgonio (RSG) are anticipated to be constructed within the planning period of this IMP, resulting in 8,247 new residential units by year 2040.

2.3.2 Projected Land Use

Future land use includes the development of vacant or underdeveloped areas not defined by known developments, which are referred to as infill. It is assumed that development, redevelopment, and infill will be according to the land use designations as depicted on Figure 2.3.

Build-out is defined as development of all land including the Planning Area of the City and is not anticipated within the planning period of this IMP. At build-out, the City will encompass approximately 36.9 square miles.

2.4 Population

This section describes the City's current population as well as projected populations throughout the planning period.

2.4.1 Historical and Existing Population

Historical population estimates from the Department of Finance from years 2010 through 2014 are presented in Table 2.4 and depicted on Figure 2.5. As of 2016, the total existing population within the City's boundaries was estimated at 30,834 people.

The water service area coincides with the City boundaries and extends to portions of the County to the south. However, since some of the population within the City boundary use septic systems in lieu of being connected to the City's wastewater collection system, the wastewater service area population is lower than the water service area population. The wastewater service area population was determined using Census Block data and removing the parcels that are on septic systems. The estimated population within each service area is summarized in Table 2.4.

As shown in Table 2.4, the estimated population in year 2016 was 30,834 for the water service area and 29,607 for the wastewater service area.

Table 2.4 Historical Service Area Population

Year	Water Service Area Population(1)	Wastewater Service Area Population(2)	Growth from Previous Year
2010	29,603	28,425	N/A
2011	29,818	28,631	0.7%
2012	30,133	28,934	1.1%
2013	30,332	29,125	0.7%
2014	30,483	29,270	0.5%
2015	30,659	29,439	0.6%
2016	30,834	29,607	0.6%

Notes:
(1) Historic population values are from Report E-4, California Department of Finance, Table 2.
(2) Calculated based on City population and adjusted by removing parcels believed to be on septic.

2.4.2 Projected Population

The City's water service area population is expected to significantly increase with the development of the identified known developments. The City's 2015 Urban Water Management Plan (UWMP) summarizes the City's service area population projection with and without two of the six master planned communities listed Table 2.3. The two communities that are not included in the lower population forecast presented in Table 2.5 are Butterfield and Rancho San Gorgonio. The population projections for each service area are summarized in Table 2.5 and Table 2.6 and graphically shown on Figure 2.5.

Table 2.5 Water Service Area Population Projections

Year	Population Without Master Planned Communities 1	Master Planned Communities Estimated Population 2	Total Water Service Area Population
2020	31,913	3,042	34,955
2025	33,335	7,965	41,300
2030	34,757	16,177	50,934
2035	36,179	20,168	56,347
2040	37,700	23,288	60,988

Notes:
(1) City's water service area population data without master planned communities retrieved from City's 2015 UWMP.
(2) RSG population retrieved from Water Supply Assessment (WSA). Butterfield population calculated based on number of dwelling units and the 2015 UWMP assumption of 3.12 persons per connection.

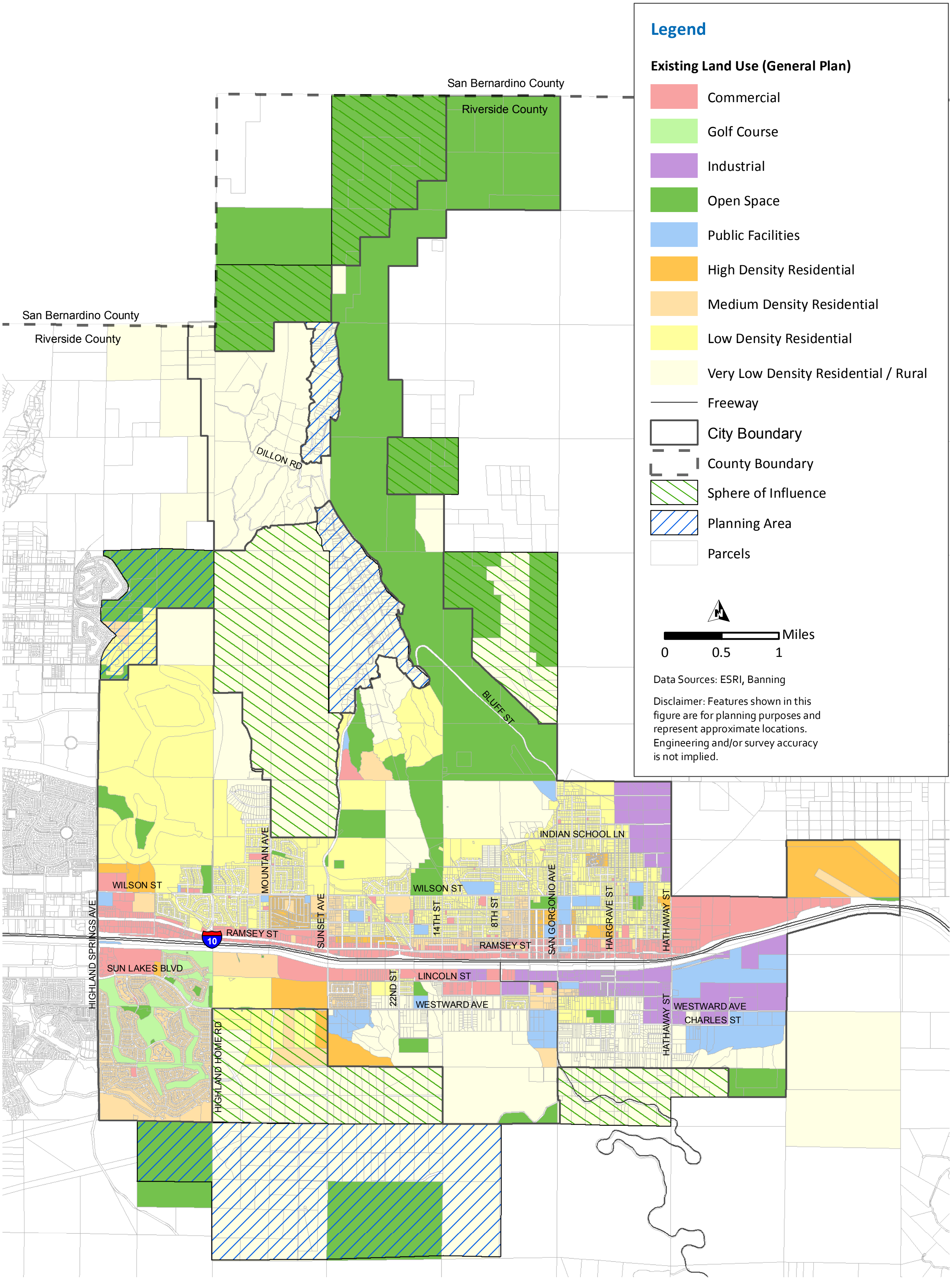

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Figure 2.3 Land Use Map

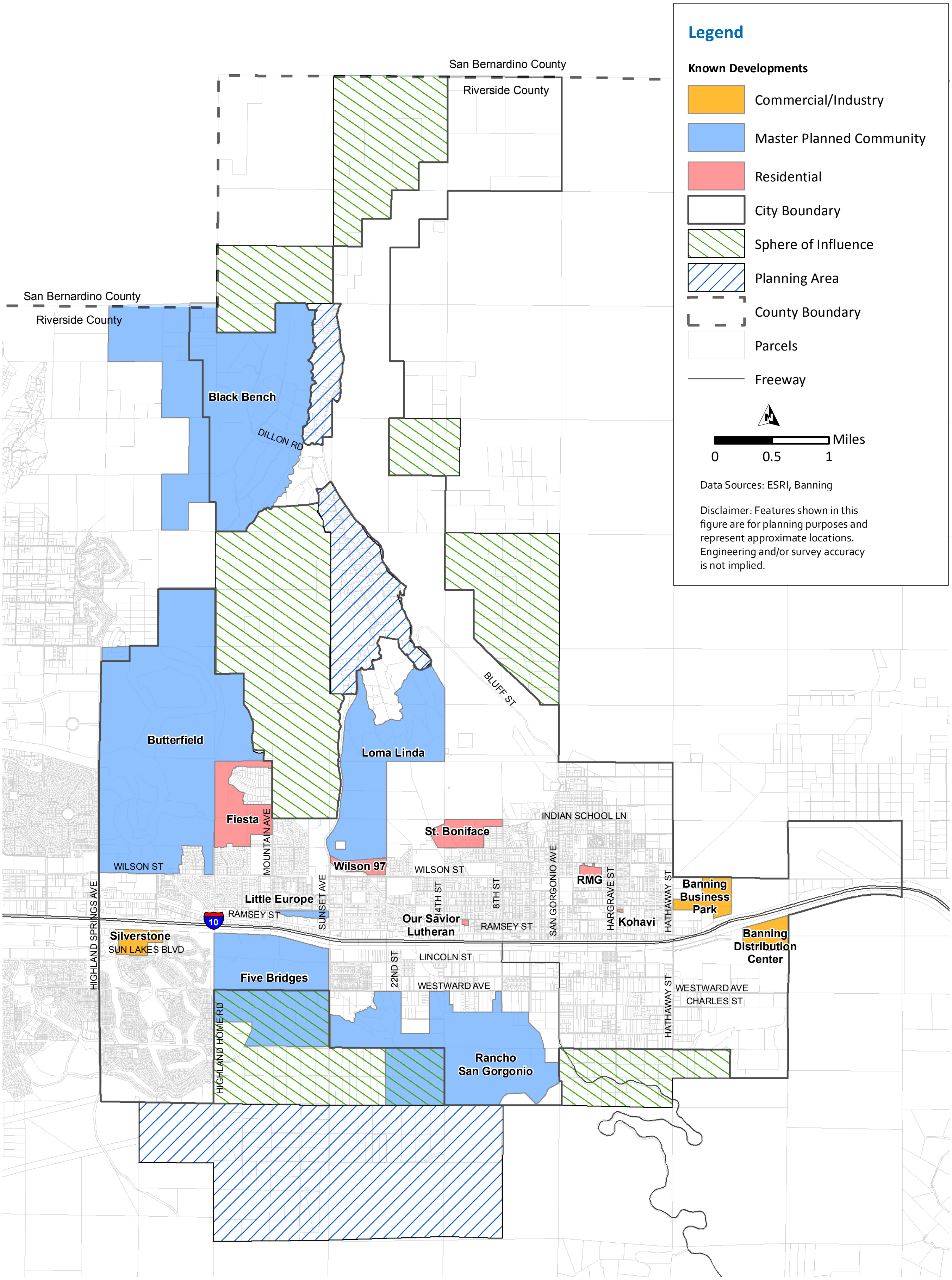


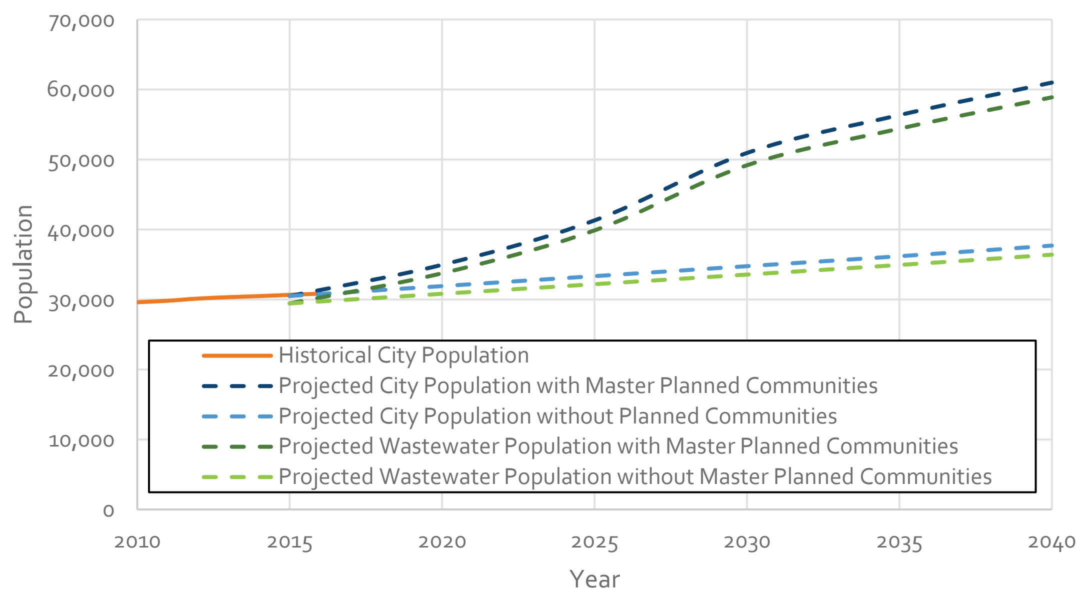
Figure 2.4 Known Developments

Table 2.6 Wastewater Service Area Population Projections

Year	Population Without Master Planned Communities [1]	Master Planned Communities Estimated Population [2]	Total Wastewater Service Area Population
2020	30,812	3,042	33,854
2025	32,185	7,965	40,150
2030	33,558	16,177	49,735
2035	34,931	20,168	55,099
2040	36,399	23,288	59,687

Notes:
(1) Wastewater service area population calculated using Census Block data and removing parcels on septic systems.
(2) RSG population retrieved from Water Supply Assessment (WSA). Butterfield population calculated based on number of dwelling units and the 2015 UWMP assumption of 3.12 persons per connection.

For the purpose of this IMP, the service area population is assumed to include the two master planned communities. As listed in Table 2.5 and Table 2.6, the projected population nearly doubles by year 2040 and is estimated to be 60,988 for the water service area and 59,687 for the wastewater service area. The average annual growth rate varies from 1.6 percent to 4.7 percent for the water service area and 1.7 percent to 4.8 percent for wastewater service area.



Source: 2015 UWMP

Figure 2.5 Historical and Projected City Population

Chapter 3

WATER DEMAND, WASTEWATER FLOW, AND RECYCLED WATER FORECASTS

This chapter summarizes the existing and projected demand and flow forecasts for the potable water, wastewater, and recycled water systems through year 2040.

3.1 Potable Water Demands

This section describes the City's existing and projected potable water demand. The existing water demand section consists of a discussion of the historical water consumption, historical water supply, water loss, and peaking factors. The future water demand section consists of a description of per-capita water use, water demand factors, water demand projections through year 2040, and the anticipated phasing of demands. This chapter concludes with a discussion of water conservation measures and the anticipated impacts these measures will have on the City's future water demands.

3.1.1 Existing and Historical Water Demands

Water demand consists of water that leaves the distribution system through metered and unmetered connections (such as fire hydrants). Additional unmetered flows contributing to water demand include maintenance flushing, reservoir cleaning, leaks to pipe joints, or breaks. The City meters all of their customer accounts, including temporary construction meters. A description of historical water consumption, water supply, and the estimated amount of water loss or unaccounted for water is presented below.

3.1.1.1 Historical Potable Water Consumption

The City provided historical customer billing records by usage type for years 2012 through 2015. The historical water use for the four years is summarized in acre-feet per year (afy) by billing classification in Table 3.1.

As shown in Table 3.1, the total annual potable water consumption has been decreasing since year 2012 due to increased conservation in response to the severe state-wide drought and the associated mandatory water use restrictions imposed by the Governor and local entities. Due to prolonged water conservation efforts, the water demands in 2015 were substantially reduced and are therefore not representative of normal conditions. For the purpose of this Integrated Master Plan (IMP), the existing water demands were defined as the average water demand of years 2012 through 2014. As shown in Table 3.1, the existing potable water demand equates to 7,475 afy.

Table 3.1 Historical Annual Consumption by Customer Class

Year	Annual Demand by Customer Class (afy)							Total Annual Demand (afy)
	Apartment/ Multi-Family	Single-Family	Commercial	Industrial	Public	Irrigation	Wholesale	
2012	91	4,477	2,087	119	108	1,015	0	7,897
2013	88	4,294	2,110	108	106	312	312	7,331
2014	85	4,186	2,095	108	77	323	323	7,197
2015	68	3,326	1,810	92	19	237	237	5,789
Average (2012-2015)	**83**	**4,071**	**2,026**	**107**	**78**	**472**	**218**	**7,054**
Average (2012-2014)	**88**	**4,319**	**2,097**	**112**	**97**	**550**	**212**	**7,475**

Notes:
Historical consumption provided by City and does not include water loss.

Seasonal variations in demands are depicted in Figure 3.1. As shown in Figure 3.1, demands are historically highest from July through October when temperatures are the highest and lowest from December through March when temperatures are lowest. The seasonal variation observed can be used to calculate monthly peaking factors, including peaking factors for maximum month demand (MMD) and minimum month demand (MinMD) conditions.

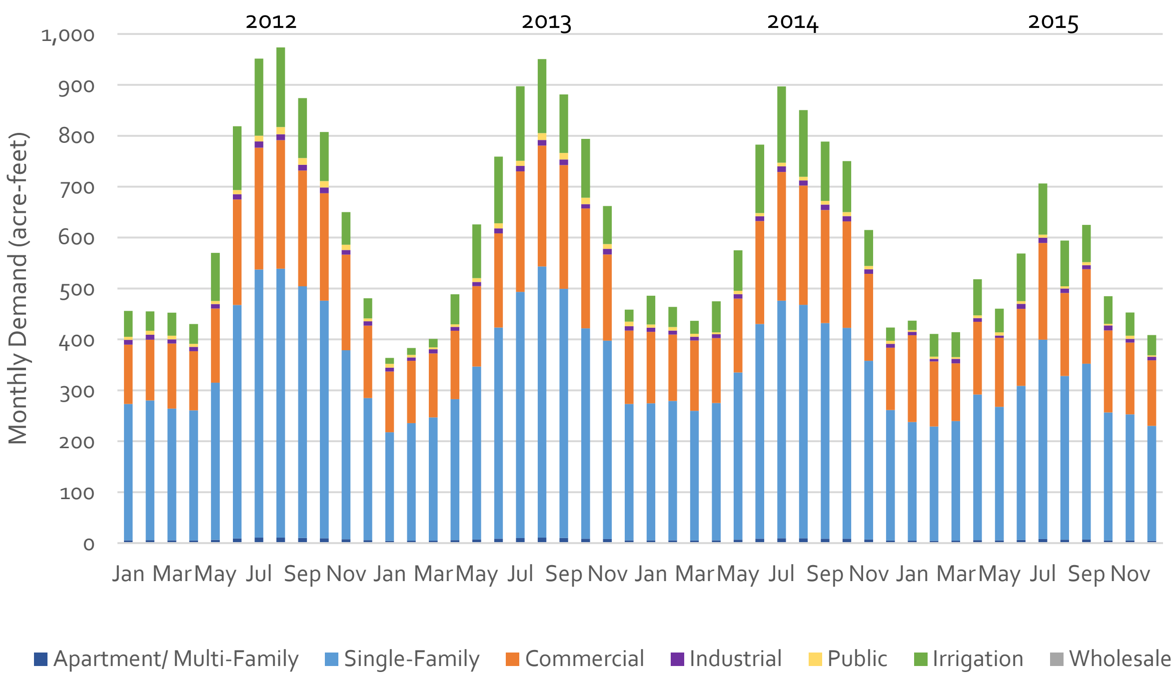


Figure 3.1 Historical Monthly Consumption

The City summarizes the water use in seven billing classifications, which include the following:

- Apartment/Multi-family.
- Single-family.
- Commercial.
- Industrial.
- Public.
- Irrigation.
- Wholesale.

A breakdown of water demands by billing classification is presented graphically in Figure 3.2. As shown in Figure 3.2, single-family residential demands account for the majority (58 percent) of the City's demands on average between years 2012 to 2015. Commercial and irrigation accounts were the two next largest consumers, representing roughly 29 percent and 7 percent, respectively. Apartments/Multi-family residential demands, wholesale, public facilities, and industrial demands represent 1 percent, 3 percent, 1 percent, and 1 percent, respectively.

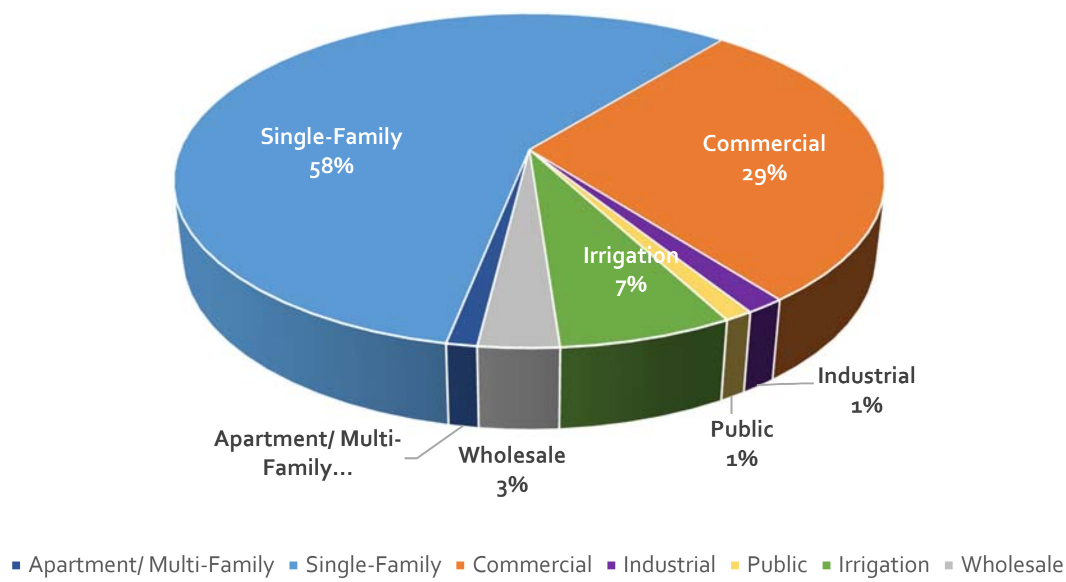


Figure 3.2 Annual Consumption Breakdown by Customer Class (Year 2012-2015)

3.1.1.2 Historical Potable Water Supply

The City's potable water supply is primarily served by groundwater from five basins: Beaumont Basin, Banning Basin, Cabazon Storage Unit, Banning Bench Storage Unit, and Banning Water Canyon Basin. The annual supply mix for years 2012 to 2015 is summarized in Table 3.2 and presented graphically in Figure 3.3.

Table 3.2 Historical Annual Supply

Year	Annual Supply (afy)					Total Annual Supply (afy)
	Beaumont Basin	Banning Basin	Cabazon Basin	Banning Bench	Banning Water Canyon	
2012	1,170	1,260	455	1,644	4,046	8,575
2013	2,136	1,747	11	1,701	3,147	8,743
2014	2,729	1,393	787	1,001	2,558	8,468
2015	1,675	527	1,207	648	2,462	6,520
Average (2012-2015)	**1,928**	**1,232**	**615**	**1,249**	**3,053**	**8,077**
Average (2012-2014)	**2,012**	**1,467**	**418**	**1,449**	**3,250**	**8,595**
Percent(2)	**24%**	**15%**	**8%**	**15%**	**38%**	**100%**

Notes:
(1) Historical production data provided by City.
(2) Percent based on average of years 2012-2015.

As listed in Table 3.2, over half of the City's supply in the period 2012-2015 (62 percent) is from to sources, the Banning Water Canyon (38 percent) and the Beaumont Basin (24 percent). The Banning Basin and Banning Bench supply about 15 percent each. The Cabazon Basin is at the east of the City's service area boundary and serves approximately 8 percent of the City's supply.

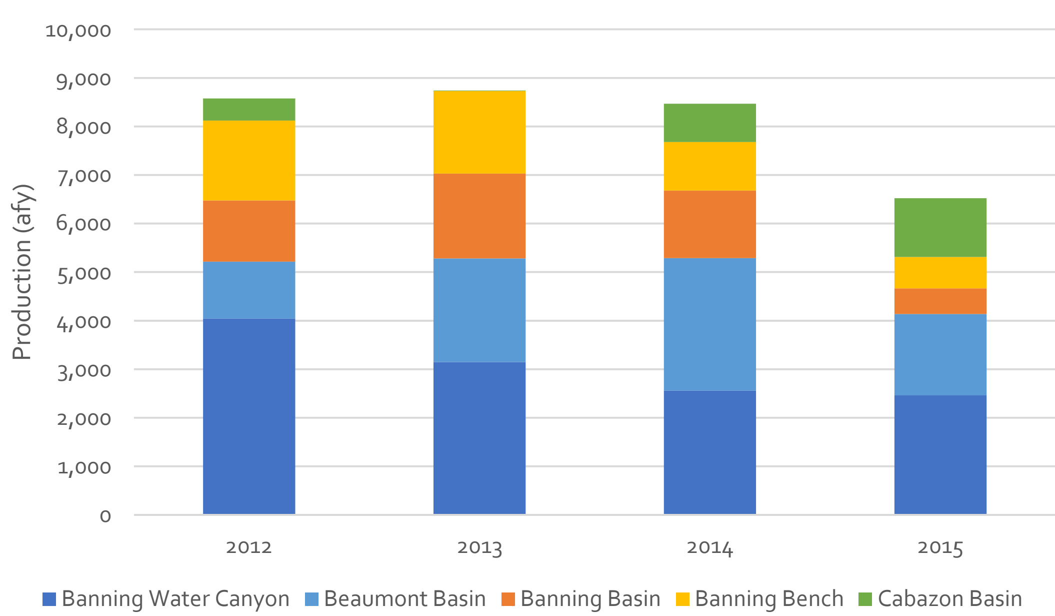


Figure 3.3 Historical Annual Production by Groundwater Unit

As shown in Figure 3.3, Cabazon Basin production decreased significantly in year 2013 due to a well being non-operational, while production in the Banning Basin increased to accommodate demands. However, supplies in the Banning Basin decreased significantly in year 2015 when demands also decreased and due to idling of the well with highest concentrations of Chromium-6.

3.1.1.3 Water Loss

The difference between water supply and consumption (billed to customers) is defined as water loss, which is also referred to as non-revenue water. Water loss may be attributed to leaking pipes, unmetered or unauthorized water use, inaccurate meters, tank overflows, hydrant testing, system flushing, reservoir cleaning, and firefighting. The City's estimated historical water loss is summarized in Table 3.3.

Table 3.3 Historical Water Loss

Year	Demand (afy)	Supply (afy)	Water Loss	
			(afy)	(%)
2012(1)	7,897	8,575	678	8%
2013(1)	7,331	8,743	1,412	16%
2014(1)	7,197	8,468	1,270	15%
2015(1)	5,789	6,520	731	11%
2016(2)	5,852	6,750	897	13%
Average	6,813	7,811	998	13%

Notes:
(1) Historical production and billing data provided by City.
(2) Year 2016 based on 2016 AWWA Water Audit. Details in Appendix C.1.

The water loss for well-operated systems is typically less than 10 percent. As shown in Table 3.3, the City's average water loss for years 2012 through 2016 is 13 percent. The City's higher water loss percentage demonstrates the need for evaluation of the City's existing potable water system. Higher water loss affect water utilities financially due to increased production costs and lost revenue. The City's 2016 American Water Works Association (AWWA) water audit provides recommendations to reduce the City's water loss percentage, including identifying data gaps, sampling, and equipment replacement. A summary of the water audit and recommendations are presented in Appendix C.1.

3.1.2 Demand Forecasting Methodology

Based on a review of the available data, it was determined that the most accurate demand forecasting method is a combination of a population- and land-use-based demand forecasting methodology. Population-based demand forecasting utilized a calculated per-capita water use, while land-use-based demand forecasting was based on calculated water demand factors (WDF). Population-based demand forecasting was used in the City's 2015 Urban Water Management Plan (UWMP). The Southern California Association of Governments (SCAG) population projections and per capita demand was used to determine the City's overall demand in the near-term (year 2025) and long-term (year 2040). Land-use based demand forecasting using WDFs was used to determine the projected demands of vacant lots and new developments.

3.1.2.1 Per Capita Water Use

An average per-capita water use expressed in gallons per capita per day (gpcd) was developed using population projections from known planned developments and SCAG. The City's 2015 UWMP summarizes the City's historical per capita water use, which is presented in Table 3.4.

Table 3.4 Historical Per Capita Water Use

Year	Estimated Service Area Population	Water Use (mgd)	Per Capita Use (gpcd)
2001	24,639	9.0	363
2005	28,250	8.4	298
2010	29,603	7.6	256
2015	30,491	6.0	196
2015 UWMP	**N/A**	**N/A**	**220**

Notes:
(1) Data obtained from City's 2015 UWMP.

As shown in Table 3.4, the City's per capita water use decreased significantly between year 2001 and year 2015. Due to increased conservation triggered by the state-wide drought and the City's water conservation programs, year 2015 experienced a low per capita use of 196 gpcd. To account for some increase in per capita water use, the City's 2015 UWMP estimates the per capita use will be approximately 220 gpcd for future non-specific plan developments.

3.1.2.2 Water Demand Factors

A WDF is defined as the estimated amount of water usage per area for a certain land-use type. WDFs are typically expressed in gallons per day per acre (gpd/ac). These factors are used to estimate the Average Day Demand (ADD) for potential development areas by multiplying the WDF with the total number of acres for each land-use category. WDFs were developed using year 2016 billing data and scaled up to the average demands 2012 through 2014 to better represent existing demands. These WDFs were used to project demands for vacant lots and planned developments where land-use details are known at this time.

The following details the steps used to develop the WDFs for this IMP:

- City provided geocoded billing addresses and assigned Assessor's Parcel Number (APN).
- Map billing addresses to 2016 billing data use.
- Map billing addresses APN to the land-use category based on the general plan and City input.
- Select 2016 billing data with demands greater than zero (0) gallons per minute (gpm) and calculate WDF, expressed in gpd per acre, for each APN.
- Calculate the average WDF for each land-use category.
- Summarize calculated and recommended WDFs for each land-use category.

Recommended WDFs are determined by rounding the calculated WDFs to the nearest hundred. The WDFs recommended for this IMP are presented in Table 3.5.

Table 3.5 Water Demand Factors

Land Use Type	Abbreviation	Calculated WDF (gpd/acre)	2017 IMP WDF (gpd/acre)
Rural	RUR	150	100
Very Low Density Residential	VLDR	1,636	1,600
Low Density Residential	LDR	2,334	2,300
Medium Density Residential	MDR	2,763	2,800
High Density Residential	HDR	3,053	3,100
Commercial	COM	5,275	5,300
Industrial	IND	1,674	1,700
Open Space - Parks	OS-P	1,019	1,000
Public Facilities	PF	373	400
Schools	PF-S	3,502	3,500

Notes:
(1) WDFs calculated based on 2016 billing data and scaled up to average of 2012-2014 demands.

As shown in Table 3.5, the calculated WDFs for the City's land uses range between 100 for rural areas to 5,300 for commercial areas.

3.1.2.3 Potable Water Peaking Factors

Peaking Factors (PF) are typically used to determine the water demands for conditions other than ADD conditions. Peaking factors account for fluctuations in demands on a seasonal or hourly basis. For example, during hot summer days, water use is typically higher than on a cold winter day due to increased irrigation demands.

Common PFs include factors for Maximum Day Demands (MDD) and Minimum Day Demands (MinDD). PFs are determined using the water system demands for a selected period and dividing the quantity by the ADD. The MDD factor, for example, is determined by comparing the water demands for the day of the year with the highest daily water demand to the ADD.

The peaking factors determined in this report include:

- Monthly Peaking Factors.
- Daily Peaking Factors.

These PFs not only reflect a different time scale, but are often calculated using different data sources. The City's PFs and data used to establish these are discussed below.

Monthly Peaking Factors

Monthly PFs represent the seasonal demand variation on a monthly basis, such as the MMD and MinMD factors. In the absence of daily production data for an entire calendar year, these factors can be established using monthly production summaries or historical billing data. The City's monthly peaking factors based on historical monthly production are summarized in Table 3.6. Since year 2015 demands were much lower due to conservation mandates, it is not included in this calculation.

Table 3.6 Monthly Peaking Factors

Year	ADD (mgd)	MMD Month	MinMD Month	MMD		MinMD	
				(mgd)	PF	(mgd)	PF
2012	7.7	July	December	11.0	1.4	4.4	0.6
2013	7.8	July	February	11.2	1.4	4.7	0.6
2014	7.6	July	December	10.4	1.4	4.0	0.5
Average	**7.2**	**N/A**	**N/A**	**10.1**	**1.4**	**4.4**	**0.6**
2017 IMP	**N/A**	**N/A**	**N/A**	**N/A**	**1.5**	**N/A**	**0.6**

Notes:
(1) Historical production data provided by City.

As shown in Table 3.6, the MMD typically occurs in July when temperatures are high, while the MinMD typically occurs between December and February when temperatures are lower. The recommended peaking factors for MMD and MinMD conditions based on historical production data are 1.5 and 0.6, respectively. These factors represent typical values observed by many other water agencies in Southern California based on Carollo experience.

Daily Peaking Factors

Historical supply records are typically used to determine the seasonal demand factors, such as Maximum Day Demand (MDD) and Minimum Day Demand (MinDD). The MDD PF represents the ratio of the largest daily demand observed in one year to the ADD for the same year. This factor can then be applied to the ADD of future planning years to project MDD. The estimated MDD is commonly used to establish water supply, storage, and pumping capacity requirements. The PFs calculated in this section should be reevaluated prior to designing the facilities.

Historical water production for maximum days in years 2012 through 2015 was provided by the City. Like the monthly PFs, year 2015 was not considered due to the low demand year. Data for Years 2012 through 2014 was used to establish the City's MDD PF by dividing the maximum day production by the average day production of the same year to obtain a ratio that represents the MDD seasonal PF. Likewise, the MinDD PF was established by dividing the minimum day production by the average day production of the same year. The City's MDD and MinDD PFs are summarized in Table 3.7.

Table 3.7 Daily Peaking Factors

Year	ADD (mgd)	Day of MDD	Day of MinDD	MDD		MinDD	
				(mgd)	PF	(mgd)	PF
2012	7.7	August 9	December 30	13.6	1.8	3.0	0.4
2013	7.8	July 3	January 30	13.5	1.7	2.9	0.4
2014	7.6	July 4	March 1	12.8	1.7	2.9	0.4
Average	**7.2**	**N/A**	**N/A**	**13.3**	**1.7**	**2.9**	**0.4**
2017 IMP	**N/A**	**N/A**	**N/A**	**N/A**	**1.7**	**N/A**	**0.5**

Notes:
(1) Historical production data provided by City.

As shown in Table 3.7, the calculated MDD and MinDD PFs are 1.7 and 0.4, respectively. For conservative planning purposes, the recommended MDD and MinDD PFs for this IMP are 1.7 and 0.5, respectively.

3.1.3 Future Water Demand Projection

Demand projections were developed using a combination of Specific Plans, vacant land information, per-capita water use, and water demand factors.

3.1.3.1 New Known Developments Demand Projections

The new development projects that would have a significant impact on water demands were identified and described in Chapter 2. Of the new developments, two master planned communities (Butterfield and RSG) will have the largest impact within the planning horizon of this IMP.

The Butterfield development involves the construction of a 1,528 acre multi-use community within the northwestern corner of the City. The master planned community will comprise mainly of single-family residential homes with space for neighborhood parks, community parks, schools, open space, and retail and commercial areas. The development is anticipated to have approximately 4,862 dwelling units. Based on input from the developer, it was assumed that the development will construct 600 dwelling units by 2020 and 200 dwelling units per year thereafter, resulting in 1,600 dwelling units by year 2025 and 4,600 units by year 2040, and 4,862 at build-out.

The Rancho San Gorgonio (RSG) development involves the construction of an 831-acre residential community within the southern portion of the City and the City's sphere of influence. The master planned community will comprise mainly of residential homes with space for common open space, an elementary school, commercial area, and parks. The development is anticipated to have approximately 3,385 dwelling units. Based on RSG's specific plan, it was assumed that the development will construct 1,126 dwelling units by year 2025 and 3,385 dwelling units by year 2040. The development is anticipated for completion by year 2040.

The estimated demands along are presented in Table 3.8 with the anticipated IMP phase completion of near-term (by year 2025), long-term (by year 2040), or build-out. As listed in this table, the City's demands are anticipated to increase by approximately 6,202 afy (or 5.5 mgd).

Table 3.8 Known Developments Demand Projections

Future Development	Development Size	Annual Demand (afy)	Anticipated Completion Phase
Residential			
Fiesta Development	303 du	117	2025
St. Boniface	172 du	66	2040
Wilson 97	98 du	38	2025
RMG Residential	48 du	19	2040
Kohavi	2 du	1	Build-out

Table 3.8 Known Developments Demand Projections (Continued)

Future Development	Development Size	Annual Demand (afy)	Anticipated Completion Phase
Our Savior Lutheran	2 du	1	Build-out
Subtotal	***625 du***	***242***	***N/A***
Master Planned Communities			
Black Bench	1,500 du & 2,452 acres	721	Build-out
Five Bridges	1,924 du &, 640 acres	1,104	Build-out
Little Europe	268 du & 15 acres	103	Build-out
Loma Linda	944 du & 600 acres	364	Build-out
Butterfield(1)	4,862 du & 1,528 acres	1,600	Build-out
Rancho San Gorgonio(1)	3,385 du & 831 acres	1,411	2040
Subtotal	***12,883 du & 6,066 acres***	***5,303***	***N/A***
Commercial/Industrial			
Silverstone	47 acres	279	2040
Banning Distribution Center	64 acres	122	2025
Banning Business Park	65 acres	256	2025
Subtotal	***176 acres***	***657***	***N/A***
Grand Total	***13,508 du & 6,242 acres***	***6,202***	***N/A***

Notes:
(1) Butterfield and RSG data provided by developers, with minor updates to estimates contained in respective Specific Plans.

3.1.3.2 Long-Term Demand Projections

Long-term demand projections were obtained from the City's 2015 UWMP, which uses a per-unit forecasting to combine population growth with average consumption to yield total demand. As discussed previously, the City's per capita usage was estimated at 220 gpcd in the 2015 UWMP. Since the 2015 UWMP, updates have been made to the demands of the master planned communities. The UWMP per capita use and the population projections were used to calculate the water demand projections along with the updated demands from the master planned communities. These updated demand projections are presented in Table 3.9.

Table 3.9 Water Demand Projections

Year	City Population Projections(2)	Demand(3) (afy)
Existing(1)	30,316	8,552
2020	34,955	10,514
2025	41,300	11,319
2030	50,934	12,046
2035	56,347	12,836
2040	60,988	13,628

Notes:
(1) Existing is represented as the average of years 2012 through 2014.
(2) Obtained from Table 2.5. City population includes the two master planned communities (Butterfield and Rancho San Gorgonio).
(3) Existing and projected demand includes 13 percent water loss.

As shown in Table 3.9, the City demand, which includes 13 percent water loss, is anticipated to increase from 8,552 afy to 13,628 afy by the year 2040. This equates to an average annual demand increase of 2.4 percent. Though the City population without the master planned communities increases steadily, the master planned communities' population increases at a much more rapid rate. However, since these communities are anticipated to have more water efficient fixtures and technology, the City's total demands do not experience the same rapid increase.

3.1.3.3 Integration with New Development Demands

The new development demands listed in Table 3.8 were integrated into the long-term demand forecast. The existing demands are based on the City's existing supply to account for the water loss (Table 3.3) and consumption (see Table 3.1). Demands from existing customers decrease due to the conversion of existing customers to recycled water, which will be discussed in Chapter 8. The integrated demand projections are shown on Figure 3.4.

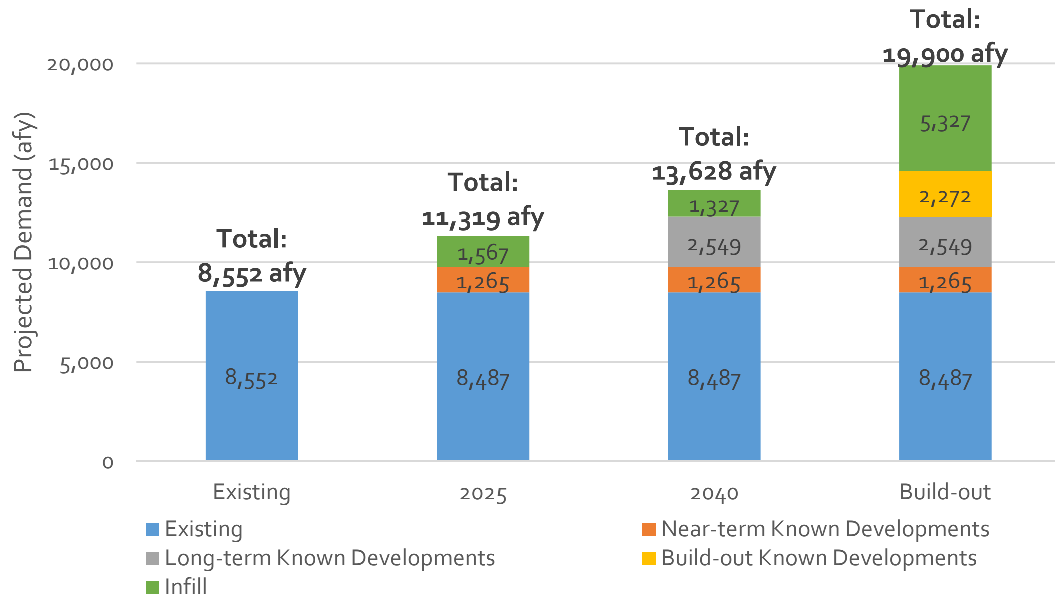


Figure 3.4 Near- and Long-Term Demands

As shown in Figure 3.4, the projected demands consist of the new developments in Table 3.8 and the background increase attributed to continuous population growth (infill and densification). Existing demands account for the majority of the usage in the planning horizon of this IMP, while known developments contribute approximately 1,265 afy (11 percent) of additional demand in the near-term (year 2025) and 3,814 afy of additional demand in the long-term (year 2040). In build-out, the City is anticipated to have an additional 3,814 afy of additional demand from known developments and a much larger increase from infill.

3.1.4 Water Conservation

Currently, the City has implemented water use restrictions to comply with the executive orders issued by Governor Brown during recent extreme drought conditions. However, the City plans to lift the ordinance in the near future, but still plans to maintain portions of the ordinance in effect for long-term conservation efforts.

The City's 2015 UWMP describes the City's Demand Management Measures, which include:

- Water waste prevention ordinances.
- Metering at water service connections.
- Conservation pricing.
- Public education and outreach.
- Programs to assess and manage distribution system real loss.
- Water Conservation program coordination and staffing support.

Due to the mandated conservation, the City's 2015 per capita demand was low compared to historical per capita demand. A demand envelop was developed to compare the impact of changes to the per capita demand. Three scenarios were evaluated as described below:

1. **Low:** The low scenario assumes increased conservation as seen in year 2015. The 2015 actual per capita demand of 196 gpcd was used to project demands.
2. **Medium:** The medium or baseline scenario assumes moderate conservation, which includes some increase from the 2015 actual per capita demand. The 2015 UWMP per capita demand of 220 gpcd was used to project demands.
3. **High:** The high scenario assumes increased per capita demand, but not to exceed the SB X7-7 goal. The 2015 UWMP SB X7-7 Calculated Target of 252 gpcd was used to project demands.

The estimated near- and long-term demands for each scenario are summarized in Table 3.10.

Table 3.10 Potable Water Demand Envelop

Scenario	Description	Per Capita Demand Assumption	2025 Demand (afy)	2040 Demand (afy)
Low	Increased Conservation	196	10,423	12,615
Medium	Baseline	220	11,319	13,628
High	Increased Per Capita Demand	252	12,513	14,979

Notes:
(1) Per capita demands retrieved from 2015 UWMP. Demands for each phase were calculated based on population estimates in Chapter 2.

As shown in Table 3.10, the estimated demand in the low scenario is 12,615 afy by year 2040, while the estimated demand in the high scenario is 14,979 afy by year 2040. The low scenario is not likely realistic with current and future conditions. The City will experience some increase in demand once these watering restrictions and mandates from the drought conditions are lifted. However, as mentioned previously, the City plans to maintain some of the restrictions from the ordinance, which is expected to continue a moderate level of conservation within the City. In addition, the new developments will be constructed with more efficient water fixtures. Thus, the City is also not likely to reach the high scenario. Based on input from City staff, it was determined that the medium scenario would be the most realistic scenario to use for the system analysis.

For the purpose of this IMP, the existing demands are considered to be the average of years 2012 through 2014 and the future demands are projected using the medium scenario presented above. The existing and future demands used for this analysis are summarized in Table 3.11.

Table 3.11 Existing and Future Potable Water Demands

Phase	ADD (mgd)	MDD[1] (mgd)	PHD[2] (mgd)
Existing	7.7	13.3	23.3
Near-term (year 2025)	10.1	17.2	30.6
Long-term (year 2040)	12.2	20.7	36.9

Notes:
(1) MDD PF assumed to be 1.7.
(2) PHD PF assumed to be 1.78 (see Chapter 4).

3.2 Wastewater

This section describes the City's existing and projected wastewater flows. This section includes a discussion of the various flow components present in wastewater and summarizes the historical flow-monitoring data that was used as part of this IMP. The existing wastewater flow section summarizes the current flows generated within the City's sewer service area, and the future wastewater flow section consists of the wastewater flow projections through buildout and the anticipated phasing of the projected flows.

3.2.1 Wastewater Flow Components

This section defines the terminology used for hydraulic analysis of the wastewater collection system. Wastewater flows vary according to the season. Dry weather flow (DWF) or base flow is flow generated by routine water usage in the residential, commercial, business and industrial sectors of the collection system.

Groundwater infiltration (GWI) is an additional component of DWF. GWI enters the sewer system when the pipeline depth is lower than the groundwater. Although the water table is several hundred feet below ground surface over much of the collection system service area, undetected leaks in the potable water system can create localized conditions that contribute to GWI. Defects such as cracks, misaligned joints, and broken pipelines allow groundwater to infiltrate into the collection system.

Wet weather flow (WWF) includes inflow from storm water runoff and infiltration from rising ground water or saturated soil conditions. The storm water inflow and infiltration comprise the WWF component termed infiltration/inflow (I/I). The response in the sewer system to rainfall is seen immediately (as with inflow) or within hours after the storm (as with infiltration).

3.2.1.1 Base Wastewater Flow

The base wastewater flow (BWF) is the flow generated by the City's customers independent of wet weather influences. BWF is estimated by measuring flows during dry weather conditions. The flow has a diurnal pattern that varies depending on the type of use. Commercial and industrial patterns, though they vary depending on the type of use, typically have more consistent higher flows during business hours and lower flows at night. Furthermore, the diurnal flow pattern experienced during a weekend may vary from the diurnal flow experienced during a weekday.

3.2.1.2 Average Annual Flow

The average annual flow (AAF) is the average flow that occurs on a daily basis throughout the year, including both periods of dry and wet weather conditions.

3.2.1.3 Average Dry Weather Flow

The average dry weather flow (ADWF) is the average flow that occurs on a daily basis during the dry weather season, with June through August considered dry weather months. The ADWF includes the BWF generated by the City's residential, commercial, and industrial users, plus the dry weather GWI component. For this report BWF, will be used synonymously with DWF as any significant ground water infiltration is unlikely during the summer months.

3.2.1.4 Maximum Dry Weather Flow

The maximum day dry weather flow (MDDWF) is the highest 30-day average flow that occurs during the dry weather season.

3.2.1.5 Maximum Day Wet Weather Flow

The maximum day wet weather flow (MDWWF) is the highest 30-day average flow that occurs during the rainy season.

3.2.1.6 Peak Wet Weather

Peak wet weather flow (PWWF) is the highest observed flow that occurs following a design storm event. Wet weather I/I cause flows in the collection system to increase. PWWF is typically used for designing sewers and lift stations. Therefore, the PWWF and the "Design Flow" are synonymous and will be used interchangeably throughout this report.

3.2.1.7 Design Storm

A design storm is a rainfall event used in the evaluation of a collection system. Design storms are defined according to rainfall depth, duration and temporal distribution.

3.2.1.8 Groundwater Infiltration

GWI is the result of extraneous water entering the sewer system through defects in pipes and manholes. GWI is related to the condition of the sewer pipes, manholes, and groundwater levels. GWI may occur throughout the year, although rates are typically higher in the late winter and early spring. Dry weather GWI (or base infiltration) cannot easily be separated from BWF by flow measurement techniques. Therefore, dry weather GWI is typically grouped with BWF.

3.2.1.9 Infiltration and Inflow

Infiltration is defined as storm water flows that enter the sewer system by percolating through the soil and then through defects in pipelines, manholes, and joints. Examples of infiltration entry points are cracks in pipelines, misaligned joints, and root penetration. Inflow is defined as storm water that enters the sewer system via storm drain cross connections, leaky manhole covers, or cleanouts.

3.2.2 Flow Monitoring Data

This section describes the temporary flow monitoring program conducted as part of this study. The data and results from the flow monitoring program are summarized and discussed.

3.2.2.1 Flow Monitoring Sites

As part of the Scope of Services for this Master Plan, Carollo Engineers, Inc. (Carollo) contracted with V&A Consulting Engineers (V&A) to conduct a temporary flow monitoring program within the City's wastewater collection system. The purpose of the flow monitoring program was to assist in the development of design flow criteria and to correlate actual collection system flows to the hydraulic model predicted flows. The temporary flow monitoring program was conducted for a period of 4 weeks, which occurred from January 20, 2017 to February 22, 2017. The "Sewer Flow Monitoring and Inflow/Infiltration Study" prepared by V&A summarizes the flow monitoring program. A copy of the report is included in Appendix B.1.

Flow Monitoring Sites and Tributary Areas

A total of nine (9) open-channel flow meters were installed at locations selected by Carollo and the City. The meter sites were selected to best isolate and model the critical areas and subareas within the sewer system. Table 3.12 lists the flow monitoring locations and the diameters for the sewers where the meters were installed .The nine (9) flow monitoring locations, as well as the tributary area to each site, are shown on Figure 3.5. Figure 3.6 provides a schematic illustration of the flow monitoring locations. As shown on Figure 3.6, Basin 3 has a number of manholes that have the potential to divert flow into Basin 1. A majority of these manholes have the inverts offset and the flow would need to reach a certain depth before it is split between basins.

Flow Meter Installation and Flow Calculation

Teledyne Isco 2150 flow meters were used for this project. Isco 2150 meters use a pressure transducer to collect depth readings and ultrasonic Doppler sensors on the probe to determine the average fluid velocity. The ultrasonic sensor emits high frequency sound waves, which are reflected by air bubbles and suspended particles in the flow. The sensor receives the reflected signal and determines the Doppler frequency shift, which indicates the estimated average flow velocity. The sensor is typically mounted at a manhole inlet to take advantage of smoother upstream flow conditions. The sensor may be offset to one side to lessen the chances of fouling and sedimentation where these problems are expected to occur. Manual level and velocity measurements were taken during installation of the flow meters and again when they were removed and were compared to simultaneous level and velocity readings from the flow meters to verify proper calibration and accuracy. The pipeline diameter was also verified in order to accurately calculate the flow cross-section. The continuous depth and velocity readings were recorded by the flow meters on 5-minute intervals.

Table 3.12 Flow Monitoring Locations

Site	Pipe Diameter (in)	Location
1	24	City of Banning Water Reclamation Facility
2	30	Lot next to treatment plant
3	15	South Hargrave Street and E Westward Avenue
4	15	South 4th Street south of W Barbour Street
5	12	663 22nd Street
6	21	2435 W Westward Avenue
7	15	1170 W Ramsey Street
8	12	Westward Avenue west of Sunset Avenue
9	12	4545 W Ramsey Street

The flow at each meter was calculated at 5-minute intervals based on the continuity equation:

$Q = V \times A$

where,

Q = Pipeline flow rate, cfs

V = Average velocity, ft/s

A = Cross sectional flow area, ft2

Finally, the 5-minute flow, velocity, and level data were aggregated into 15-minute increments.

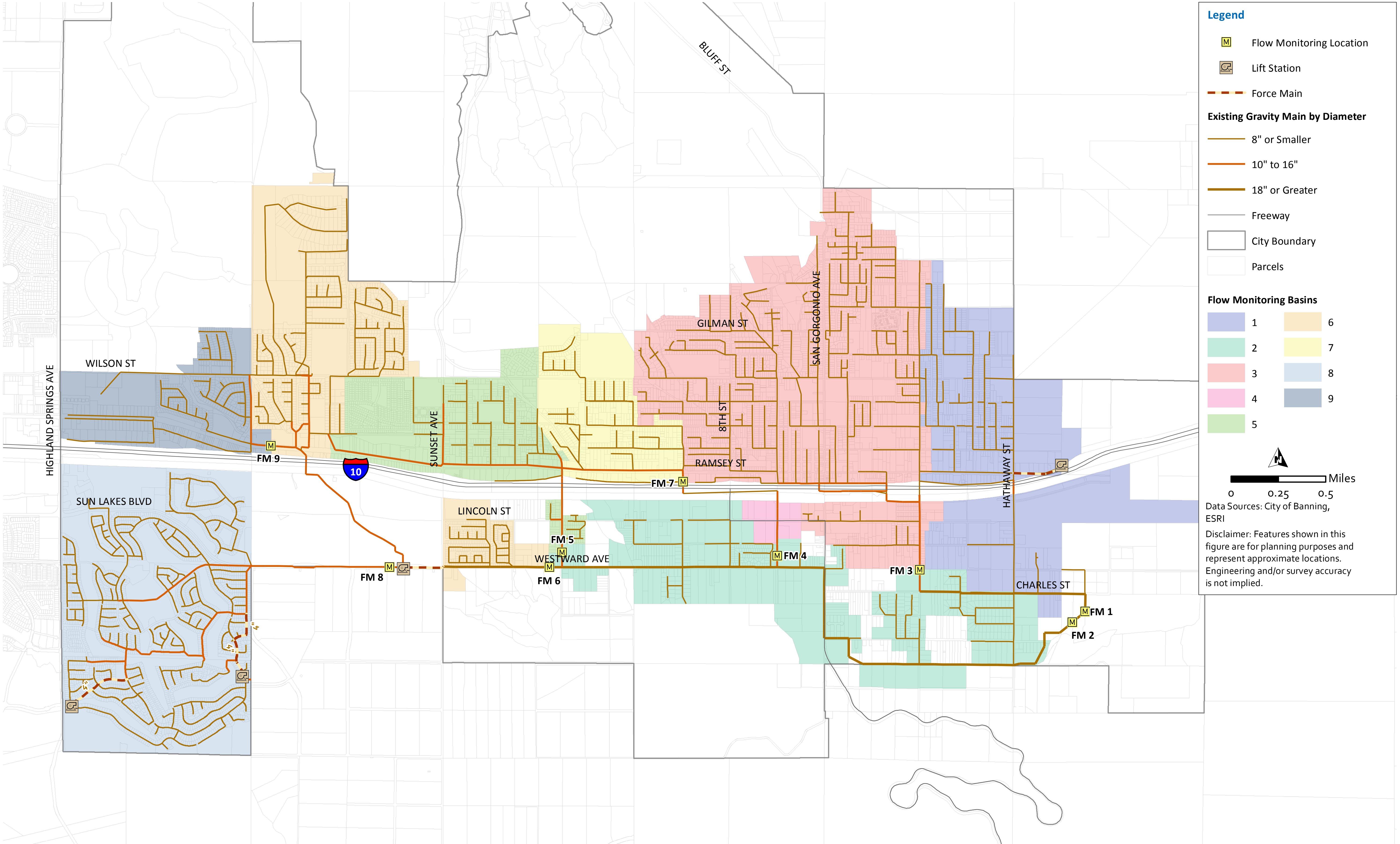


Figure 3.5 Temporary Flow Monitoring Locations

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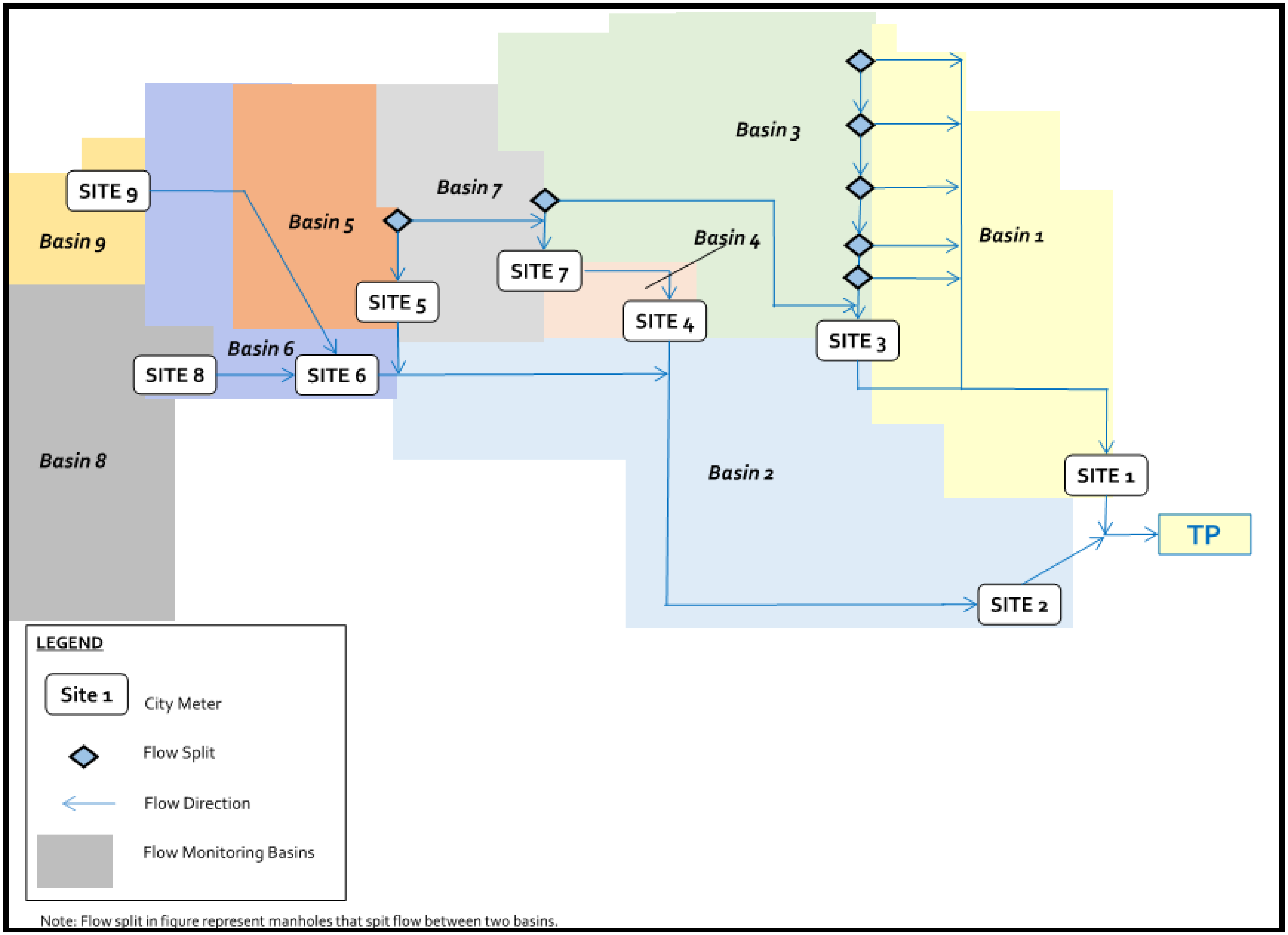

Figure 3.6 Flow Monitoring Schematic

3.2.3 Flow Monitoring Results

This section summarizes the results of the flow monitoring program, including dry weather flow data, rainfall data, and wet weather flow data.

3.2.3.1 Dry Weather Data

During the flow monitoring period, depth and velocity data were collected at each meter at 5-minute intervals. The 5-minute data was then aggregated to 15-minute data by V&A. Carollo aggregated the 15-minute data to hourly data for use in the hydraulic model. Characteristic dry weather 24-hour diurnal flow patterns for each site were developed based on the hourly data. This hourly flow data was then used to calibrate the hydraulic model for the observed dry weather flows during the flow monitoring period.

Hourly patterns for weekday and weekend flows vary and are separated to better understand dry weather flow. V&A used the data from days least affected by rainfall to estimate the weekday and weekend dry weather flows. In addition, V&A provided estimates for the average weekday and weekend levels and velocities at each site, which are used in dry weather flow calibration. Table 3.13 summarizes the dry weather flows at each meter.

Table 3.13 Dry Weather Flow Summary

Monitoring Site	Dry Weather Flow				
	(Mon – Thur) (mgd)	(Friday) (mgd)	(Saturday) (mgd)	(Sunday) (mgd)	Overall (mgd)
1	0.649	0.622	0.719	0.705	0.663
2	1.349	1.329	1.324	1.381	1.347
3	0.494	0.486	0.496	0.503	0.495
4	0.342	0.335	0.343	0.350	0.342
5	0.059	0.067	0.063	0.077	0.063
6	0.844	0.846	0.859	0.891	0.853
7	0.281	0.274	0.277	0.282	0.279
8	0.489	0.503	0.492	0.538	0.503
9	0.195	0.193	0.188	0.180	0.192
Total Influent	**1.998**	**1.951**	**2.043**	**2.086**	**2.01**

Notes:
(1) Source: Sanitary Sewer Flow Monitoring, V&A Consulting Engineers, Inc. (2017).
(2) Overall Dry Weather Flow = ((4 x Monday - Thursday)+(Friday)+(2 x Weekend))/7.
(3) Total Influent is flow entering WWTP and is equal to Site 1 plus Site 2.

Figure 3.7 illustrates a typical variation of wastewater flows in the City, which is based on the data collected from Meter 2. Similar graphics associated with the remaining sites are included in Appendix B.2.

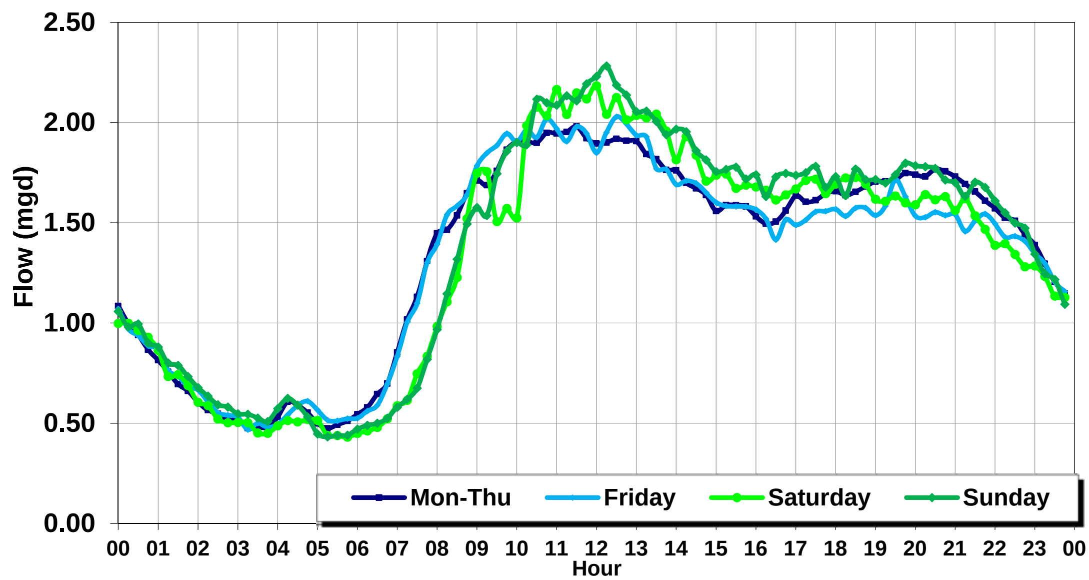

Figure 3.7 Typical Dry Weather Flow Variation (Meter 2)

3.2.3.2 Rainfall Data

Over the course of the wet weather flow monitoring period, two significant rainfall events occurred. Table 3.14 summarizes the total rainfall recorded during the two main rainfall events and over the entire flow monitoring period. The events that occurred January 20th is classified as greater than a 1-year, 24-hour storm event, while January 24nd is classified as less than a 1-year, 24-hour storm event.

Table 3.14 Rainfall Event Summary

Storm Event	Rain Gage (in)
January 19 – 20, 2017	2.78
January 22 - 24, 2017	2.33
February 17 - 19, 2017	1.25
Total Monitoring Period (January 19 – February 19)	5.01

3.2.3.3 Wet Weather Flow Data

V&A evaluated the flow monitoring data to quantify the collection system's response to wet weather events. Because the rainfall event that occurred on January 20th, 2017 captured the largest I/I response during the flow monitoring period, it was selected for the I/I analysis.

During January, Banning accepted flows from the City of Beaumont on an emergency basis. The flows where diverted into basin 9, started entering Banning's collection system on January 20th and extended for approximately one week. The wet weather flow monitoring data shows an increase in flow not related to rainfall inflow for basin 9. The extraneous flow occurred during the two largest rain events, which were used for model calibration. Wet weather calibration results in Appendix B.2 illustrate the increased flow and show how the modeling data does not account for flows unrelated to inflow. However, a distinction between external flow and flow within the basin during peak events related to inflow was difficult to distinguish.

Figure 3.8 shows an example of the wet weather response at Site 2 during the January rainfall events. The volume of I/I that entered the system from the collection system upstream of Site 2 is also illustrated in Figure 3.8. The light blue area is the base wastewater flow, and the gray area is the measured wet weather flow from the flow monitoring period. As shown, discernible amounts of I/I enter the system during wet weather events. Similar graphs were generated for the remaining monitoring sites and are shown in Appendix B.2.

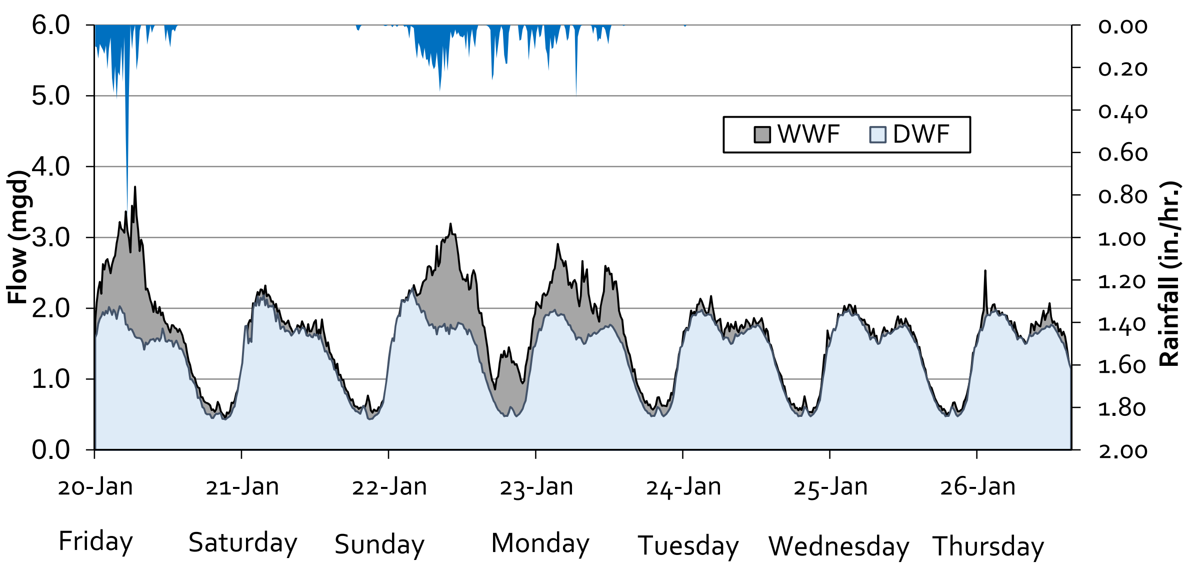

Figure 3.8 Example Wet Weather Flow Response (Meter 2)

The metric typically used to quantify the severity of the system's I/I is the R-value. The R-value is defined as the percentage of rainfall volume that makes it into the collection system as I/I. Table 3.15 summarizes the R-values for each flow monitoring basin. As shown in Table 3.15, the R-Values vary from 3.12-percent in basin 1 to 0.1-percent in multiple basins. In general, an R-Value of 5 percent or more is usually considered indicative of a significant I/I response.

The R-Value for each basin is determined by isolating I/I associated with individual flow monitoring basins and calculating the ratio of the volume of water that enters the system as I/I versus the volume of rainfall that fell over the flow monitoring basin tributary area. As shown in Table 3.15, basin 1 has the largest amount of I/I relative to the other basins.

Table 3.15 I/I Analysis (V&A)

Meter Basin	Basin Acres(2)	Peak I/I Rate (mgd)	Peak I/I Per Acre (gpd/Acre)	Combined I/I (gallons)	R Value (%)
1	556	2.03	3,654	1,309,000	3.12
2	583	0.03	59	6,000	0.1
3	1,391	1.29	928	581,000	0.6
4(3)	96	N/A	N/A	N/A	N/A
5	133	0.25	1,845	6,000	0.1
6(4)	645	0.22	343	N/A	N/A
7	358	0.44	1,238	36,000	0.1
8	882	0.48	546	137,000	0.2
9	310	0.38	1,234	123,000	0.5

Notes:
(1) Source: Data provided by V&A.
(2) Basin area is considered gross acreage.
(3) Small basin size relative to the flow quantity of upstream site. Isolated flows after subtraction may have too much uncertainty.
(4) R-value for basin is uncertain as flow monitoring site 6 includes a total of 3 basins.

Flow monitoring Site 7 can be consolidated with flow monitoring site 4, which is downstream of site 7. These sites were previously recommended to account for overflow from Basin 3, however, the overflow pipeline is now abandoned.

Adding a monitoring site upstream of Westward lift station will reduce the uncertainty of Basin 6 I/I and its hydrograph pattern, which are influenced by Westward lift station and multiple upstream basins.

Basin 1 has displayed the largest amount of I/I entering the collection system. Further investigation is recommended to identify the source(s). The addition of flow monitoring sites within Basin 1 will assist with isolating areas of high I/I.

3.2.4 Wastewater Design Flows

This section summarizes the City's Historic Flows and presents the methodology for the calculation of dry weather and wet weather flows used to model the existing and future system.

3.2.4.1 Historical Flows

The City provided historical daily influent flow data at the wastewater treatment plant (WWTP) for the period of 2011 through 2016. The data included the average monthly flow, as well as the maximum daily flow that occurred in each month.

Historical flow conditions at the WWTP are summarized in Table 3.16. As shown, ADWF decreased approximately 9-percent over six years. A decline in wastewater generation is attributed to consecutive drought years and conservation efforts.

Based on the data, the Average Annual Flow (AAF) is less than the Average Dry Weather Flow (ADWF). Data on daily average Influent flows entering the WWTP have shown a consistent trend of being greater during the summer months. With ADWF only accounting for the summer months, Table 3.16 shows a greater average for ADWF. This is not uncommon for Cities to experience higher ADWF values in comparison to AAF. During summer months base flows tend to be greater than the winter months. With an extensive drought these previous years, rainfall has not skewed the data to show annual averages as greater.

Average daily flows at the WWTP are shown on Figure 3.9. During significant rainfall, the WWTP experienced notable amounts of inflow from three storm events. The event that occurred between January 5, 2016 and January 7, 2016, produced the highest peak hour flow. Figure 3.10 illustrates the hourly flow data at the WWTP during the event that occurred between the 5th and 7th. As shown on Figure 3.10, the event produced a peak hour flow of 6.5 million gallons per day (mgd).

3.2.4.2 Historical Per Capita Wastewater Generation

Historical per capita wastewater flows were determined for the previous five years, from 2012 to the end of 2016. The City's ADWF for each year was divided by the City's estimated sewer service population. The City's 5-year average per capita wastewater generation was estimated at 73 gallons per capita per day (gpcd). The highest per capita rate occurred in 2012, while the lowest rate occurred in 2016. As shown in Table 3.17, the per capita wastewater generation has declined over the last five years. This is a result of an increased population and a decrease in wastewater generation.

Table 3.16 Historical Treatment Plant Flow Summary

Flow Condition	Year						5-Yr Average
	2011 (mgd)	2012 (mgd)	2013 (mgd)	2014 (mgd)	2015 (mgd)	2016 (mgd)	
Average Annual Flow (AAF)	2.16	2.11	2.11	1.99	1.97	1.94	2.02
Average Dry Weather Flow (ADWF)	2.21	2.17	2.15	2.05	2.01	2.01	2.08
Max Day Dry Weather Flow (MDDWF)	2.45	2.41	2.37	2.13	2.18	2.17	2.25
Max Day Wet Weather Flow (MDWWF)	2.60	2.58	2.43	2.94	2.29	2.60	2.57

Notes:
(1) ADWF is the average daily flow from June to August.
(2) Wet weather flow excludes dry months.

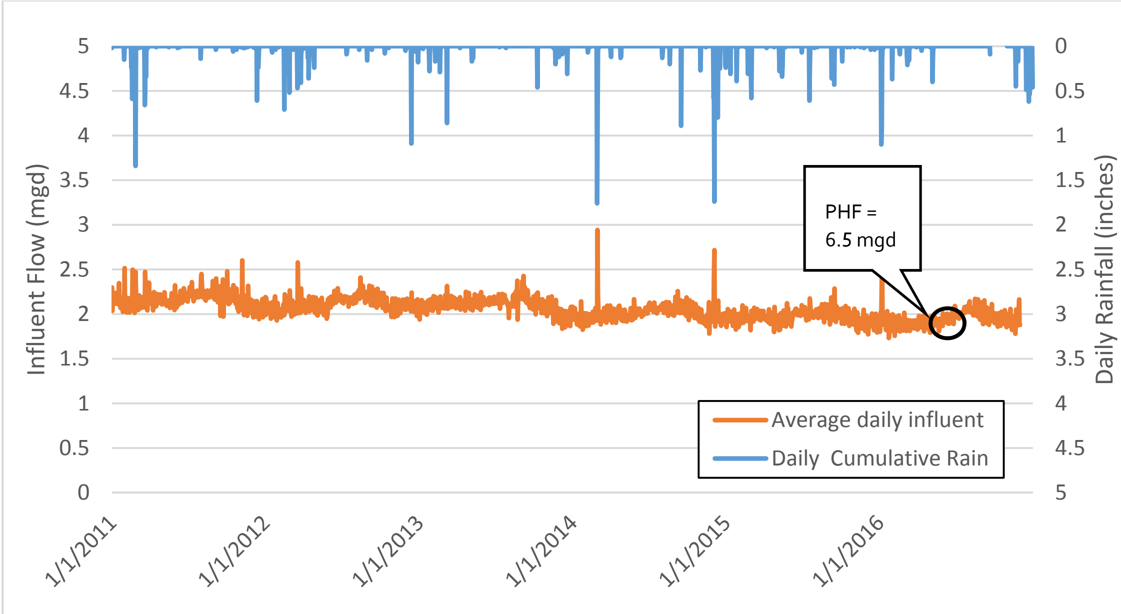

Figure 3.9 Historical Daily Flows at WWTP

Table 3.17 Per Capita Wastewater Generation

Year	Service Area Population	ADWF (mgd)	gpcd
2012	28,934	2.17	75
2013	29,125	2.15	74
2014	29,270	2.05	70
2015	29,439	2.01	68
2016	29,607	2.01	68
		Average	**73**

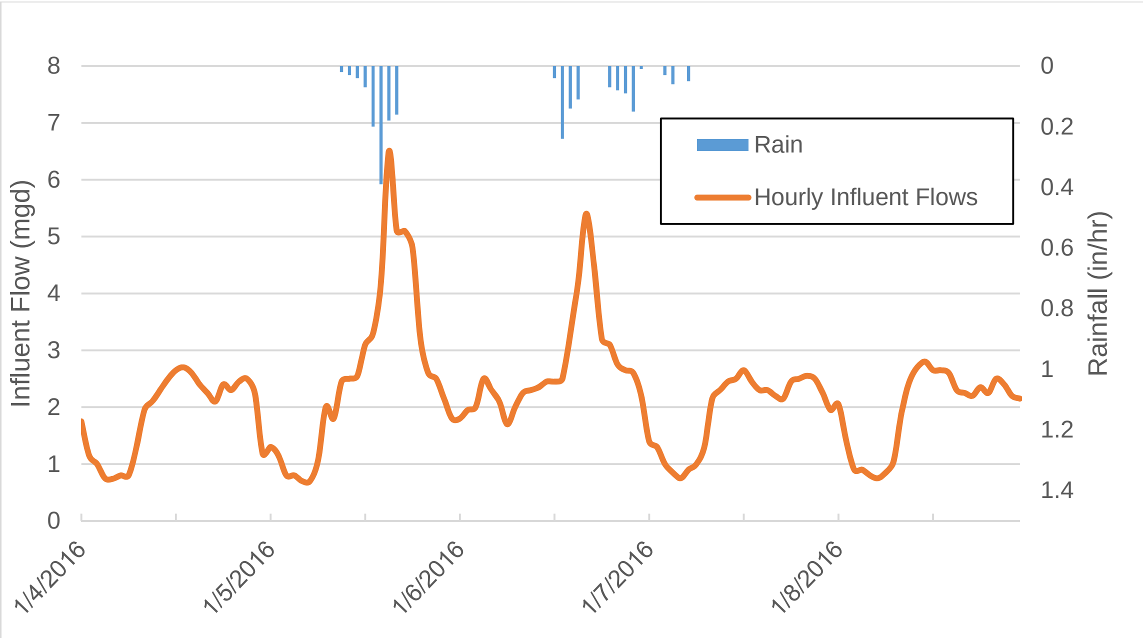

Figure 3.10 Hourly Flows at the WWTP

3.2.4.3 Per Capita Analysis

A wastewater flow envelop was developed to compare the impact of changes to the per capita demand. Three scenarios were evaluated as described below:

- **Low:** The low scenario assumes increased conservation as seen in year 2015. The lowest per capita rate of 68 gpcd was used.
- **Medium:** The medium or baseline scenario assumed the average from year 2012 to 2016. The per capita rate of 73 gpcd was used to project flows.
- **High:** The high scenario assumes increased per capita rates and uses the highest 5-year rate, or year 2012. The per capita rate of 75 gpcd was used to project flows.

The estimated near- and long-term wastewater generation for each scenario is summarized in Table 3.18.

Table 3.18 Wastewater Flow Envelop

Scenario	Description	Per Capita Demand Assumption	2025 (mgd)	2040 (mgd)
Low	Increased Conservation	68	2.64	4.10
Medium	Baseline	73	2.80	4.29
High	Increased Per Capita Demand	75	2.87	4.36

As shown in Table 3.18, the low scenario occurred during a period of mandated conservation and wastewater flows are expected to increase as drought conditions subside. However, the higher wastewater rate is unlikely to become the City's average as conservation efforts are ongoing and new development continues to install efficient water fixtures. Therefore, it was determined that the medium scenario would represent future per capita rates.

3.2.4.4 Wastewater Unit Flow Factors

To estimate the amount of flow per acre generated by each land use category, wastewater flow factors (WWFF) were developed and are a correlation between land use and sewer generation. These flow factors are based on the average wastewater flow generated for each land use type and were developed to project the ADWF for buildout of the City's General Plan.

WWFF provide a method to estimate the average quantity of flow per acre for each type of land use. The flow factors are expressed in gallons per day per acre (gpd/ac). The flow factors were developed using the following procedure:

- Average flows for each flow metering tributary area were derived from the flow monitoring data.
- Using GIS information, the acres for each existing land use type contained in each flow monitoring tributary area were calculated. Land use identified as vacant or on septic were excluded from existing estimates and added under future scenarios.
- Preliminary WWFF for each land use type were estimated based on the previous Master Plan.
- The WWFF for each flow metering tributary were then balanced (adjusted up or down) to match the calculated average flows from each tributary to the measured flows during the flow monitoring period.
- Once the WWFF for each flowmeter tributary area were balanced, the weighted average of the coefficients for each existing land use type was calculated based on the acreage contribution from each metering tributary area.
- The weighted average WWFF were then adjusted for the entire developed sewer service area until they matched the total metered ADWF of 2.01 mgd. The adjusted WWF are considered representative of the wastewater generation by land use for the entire City and are used to project Buildout average wastewater flows.

The calibrated WWFF developed for the Integrated Master Plan are summarized in Table 3.19. These flow coefficients are less than those in the previous 2006 Sewer Master Plan. The reduction of wastewater generation can be contributed to a number of reasons, including California's current drought conditions, promotion of efficient plumbing fixtures, ongoing water restrictions, and a water rate increase. The water rate increase promotes water conservation and occurred after the completion of the 2006 Master Plan.

Table 3.19 Wastewater Flow Factors

Land Use Type	Abbreviation	Total Area (acre)	Wastewater Factors	
			gpd/acre	gpd
Rural	RUR	83	50	163,000
Very Low Density Residential	VLDR	289	180	52,000
Low Density Residential	LDR	967	540	522,000
Medium Density Residential	MDR	677	1,020	691,000
High Density Residential	HDR	129	1,260	163,000
Commercial	COM	338	1,150	388,000
Industrial	IND	90	750	67,000
Public Facilities(1)	PF	299	410	122,000
Open Space	OS-P	313	0	0
Total	**-**	**3,185**	**-**	**2,010,000**

Notes:
(1) Includes schools, County jail, and hospital.

As with most Cities in California, residential land use accounts for a majority of development and wastewater flow. For the City, residential customers account for 71 percent of current flow, commercial users account for 19 percent, the industrial sector generates 4 percent, and public facilities account for 6 percent of flows.

3.2.4.5 Existing Average Dry Weather flow

To estimate existing ADWF, a combination of historical flow data from the WWTP and the temporary monitoring program were used. During the flow monitoring program dry weather flows averaged 2.01 mgd. In addition, Table 3.16 shows that the City's historical ADWF, for the previous two years, has been consistent with the results of the flow monitoring program. Therefore, the existing ADWF generated within the City is approximately 2.01 mgd.

3.2.4.6 Future Average Dry Weather Flow

Based on review of available data, it was determined that the most accurate forecasting methodology for sewer flow included a combination of population and land use flow factors. Known future development wastewater flow projections were based on Specific Plans, land use, and WWFF. These flows were then added to the appropriate planning year, based on input from the City and from Butterfield and RSG Master Planned Communities.

For Near Term (2025) and Long Term (2040) flows, a combination of projected population, and the wastewater per capita flow rate were utilized to estimate infill. Known development utilized flow projections from Specific Plans and land use. Buildout flows were projected by multiplying the WWFFs by the projected land use acreage. Existing and projected wastewater flows for the City are provided in Table 3.20. Wastewater flows for known developments are presented in Table 3.21.

Table 3.20 Average Dry Weather Flow Projections

Planning Year	Estimated ADWF (mgd)
Existing (2017)	2.01
Near Term (2025)	2.80
Long Term (2040)	4.29
Buildout	6.35

3.2.4.7 Design Storm

For wastewater collection systems, the PWWF (or design flow) is typically estimated through the use of a peaking factor, a peak I/I allowance, or by routing a "design storm" through a calibrated hydraulic model. Of these three methods, the most accurate way to develop a PWWF estimate is to route a design storm through the calibrated hydraulic model.

In California, it is an industry standard to use a 10-year, 24-hour design storm to analyze wastewater collection system performance during PWWF conditions. Figure 3.11 shows the estimated rainfall intensity generated from the 10-year, 24-hour design storm.

For this IMP, the 10-year, 24-hour design storm was modified to mimic the January 20, 2017, storm event and is shown on Figure 3.11. The design storm has a peak intensity of 0.77 inches per hour and a total rainfall volume of 4.46 inches. The design storm volume is based on NOAA Atlas 14 Point Precipitation Frequency Estimates for a 10-year, 24-hour storm event.

Table 3.21 Known Development Flow Projections

Development name	Development Size		Percent Constructed			ADWF			Total
	Area (acre)	Residential Units	2025	2040	Buildout	2025 (gpd)	2040 (gpd)	Buildout (gpd)	
Residential									
Fiesta	159	215	100%	-	-	59,721	-	-	59,721(3)
St. Boniface	65	172	0%	100%	-	0	33,901	-	33,901(3)
Wilson	35	98	100%	-	-	19,316	-	-	19,316(3)
RMG	11	48	0%	100%	-	0	9,461	-	9,461(3)
Kohavi	1	2	0%	0%	100%	0	0	394	394(3)
Our Savior Lutheran	3	2	0%	0%	100%	0	0	394	394(3)
Community									
Black Bench	2,452	1,500	0%	0%	100%	0	0	300,980	300,980(3)
Five Bridges	639	1,924	0%	0%	100%	0	0	439,320	439,320(3)
Little Europe	15	268	0%	0%	100%	0	0	52,823	52,823(3)
Loma Linda	600	944	0%	0%	100%	0	0	186,062	186,062(3)
Pardee Buterfiled	1,004	4,862	33%	62%	5%	250,800	471,200	38,000	760,000(2)
Rancho San Gorgonio	831	3,385	25%	75%	-	209,750	629,250	-	839,000(2)
Commercial/Industrial									
Silverstone	47	-	0%	100%	-	0	53,580	-	53,580
Banning Business Park	65	-	0%	100%	-	0	61,425	-	61,425
Banning Dist. Center	64	-	0%	100%	-	0	48,000	-	48,000
Total						**539,587**	**1,306,817**	**1,017,974**	**2,864,378**

Notes:
(1) Known development data provided by the City.
(2) ADWF provided by specific Plan.
(3) Based on a flow per capita of 73 gpcd and general plan capita per dwelling of 2.7.

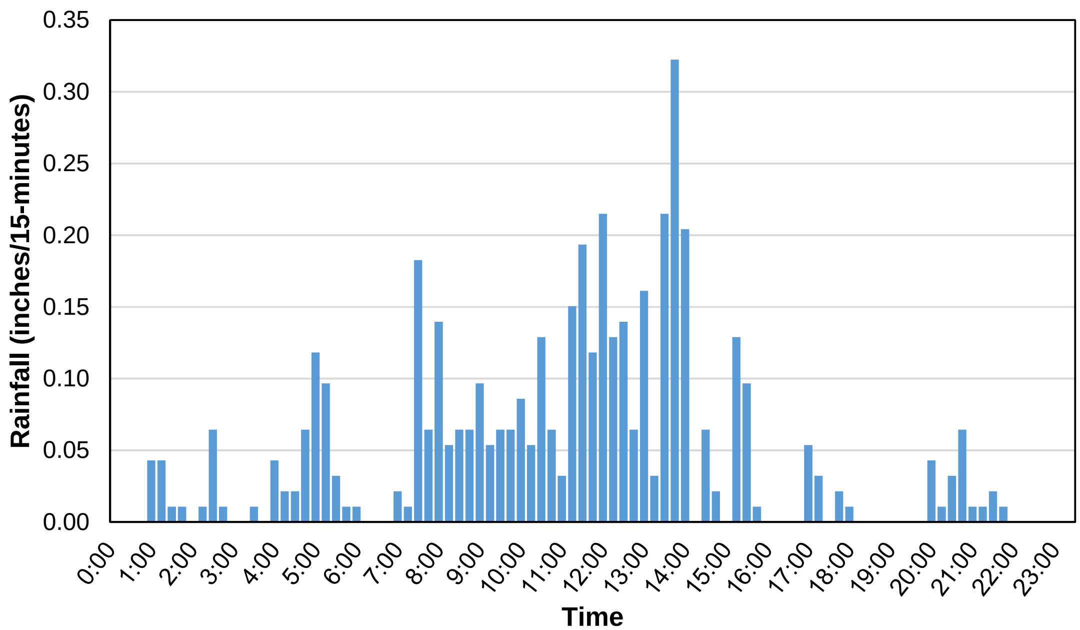

Figure 3.11 10-Year, 24-Hour Design Storm

3.2.4.1 Existing and Projected Peak Wet Weather Flow (PWWF)

Wet weather infiltration and inflow (I/I) occurring during and after rainfall events will increase flows in the collection system and cause peak wet weather flow (PWWF), which is the highest hourly flow, after the design storm event. The City's sewers and lift stations were evaluated based on their capacity to convey the PWWF.

Throughout the system, the existing PWWF was derived using the hydraulic modeling results. This was accomplished by routing the 10-year, 24-hour design storm through the hydraulic model, which was calibrated to both dry weather and wet weather conditions. Similarly, the future PWWF was derived by routing a 10-year, 24 hour design storm through the hydraulic model. Peak I/I rates for future growth areas (e.g., vacant areas within the existing service area and growth areas outside of the current service area) were developed based on a peak I/I rate of 500 gallons per day per acre (gpd/ac). In comparison to Table 3.15, 500 gpd/acre is reflective of Basin 8.

Table 3.22 presents a summary of existing and projected flows for both ADWF and PWWF. As shown, the existing PWWF is estimated at 13.8 mgd for a 10-year storm event and is projected to increase to 22.2 mgd at buildout.

Table 3.22 Flow Projections

Flow Condition	ADWF (mgd)	PWWF (mgd)	PWWF Peaking Factor
Existing	2.01	13.8	6.87
Near Term (2025)	2.80	15.2	5.43
Long Term (2040)	4.29	17.5	4.08
Buildout	6.35	22.2	3.50

Notes:
(1) ADWF = Average Dry Weather Flow.
(2) PWWF = Peak Wet Weather Flow and is based on 1-hour interval.

3.3 Recycled Water

The section presents a discussion on the estimated existing and future recycled water demand. Potential recycled water use associated with non-potable reuse (NPR) and indirect potable reuse (IPR) through groundwater recharge is discussed in Chapter 8 of this IMP.

3.3.1 Existing and Historical Recycled Water Demands

The City currently serves one customer (Sun Lakes Development Golf Course) with non-potable water from Well M7 and Well M12. Based on average production data for years 2012 through 2014, the average annual demand for Sun Lakes Development Golf Course is 850 afy (or 0.8 mgd). Aside from this customer, the City does not have any other recycled water or non-potable demands.

3.3.2 Recycled Water Peaking Factors

Similar to potable water, PFs are used to estimate recycled water demands for conditions other than average annual demand conditions. PFs are used to account for fluctuations in demands on a seasonal and hourly basis.

Since the City currently only has one customer connected to the system, existing and historical PFs are not available. Thus, peaking factors identified in the 2006 Recycled Water Master Plan (RWMP) were used to estimate MDD and Peak Hour Demand (PHD).

Table 3.23 Recycled Water Peaking Factors

Demand Condition	Peaking Factor
ADD	1.0 x ADD
MDD	2.8 x ADD
PHD	
8-Hour Irrigation	8.5 x ADD (or 3.0 x MDD)
12-Hour Irrigation	5.6 x ADD (or 2.0 x MDD)
24-Hour Irrigation	2.8 x ADD (or 1.0 x MDD)

Notes:
(1) Source: 2006 Recycled Water Master Plan (Carollo, 2006).

As shown in Table 3.23, the recycled water MDD PF is 2.8, while the PHD PF varies depending on the assumed number of hours of irrigation.

3.3.3 Future Recycled Water Demand Projection

Future recycled water demand projections are based on a review of the identified potential customers in the 2006 RWMP. This section describes the methodology used to project the future demand potential.

It should be noted that the future demands described herein do not necessarily represent the actual future demands. This section is limited to identifying the future demand potential. The system analysis (Chapter 8) determines the feasibility of serving these customers and identifies the preferred pipeline alignments to serve a portion of the potential customers described in this chapter.

3.3.3.1 Methodology

The 2006 RWMP identified 18 potential recycled water customers based on the following three criteria:

1. Location shall be near a recycled water distribution pipeline or in proximity of other potential customers.
2. ADD exceeds 10,000 gpd. Potential customers with ADD less than 10,000 gpd may be eligible if their location is near a recycled water pipeline.
3. Location within City limits.

The list of potential recycled water customers was reviewed to determine the current irrigation status of each customer and the feasibility of tying into the recycled water system, resulting in an updated list of 15 customers. The full list of the 15 potential customers and their estimated recycled water demands are summarized in Appendix C.

3.3.3.2 Potential Customers

Since the 2006 RWMP, several customers, including Caltrans, Repplier Park, and Gilman Ranch Museum, have reduced demands due to changes such as drought tolerant landscaping. Based on an initial review of the recycled water system layout, connecting these customers was not considered cost effective. In addition, customers north of the Interstate 10 would require a second pressure zone, resulting in an additional pump, large lengths of pipeline, and additional storage to serve these customers. Since those customers were not large users, City staff decided to keep the recycled water system south of the Interstate 10. The customers that are included in this analysis and their demands are summarized in Table 3.24, while the location of the recycled water customers is shown on Figure 3.12.

As listed in Table 3.24, the total potential recycled water demand is estimated to be 2,530 afy (or 2.3 mgd). The largest potential recycled water users are Butterfield Development and Sun Lakes Development, which have an estimated demand of 864 afy and 850 afy, respectively.

Table 3.24 Potential Recycled Water Customers and Demands

Customer Name	Irrigation Area (acres)	WDF (gpd/acre)	Hours of Irrigation	Annual Demand (afy)	MDD (mgd)	PHD (mgd)
Existing City Customers						
Sun Lakes Development	199	3,814	24	850	2.1	2.1
Banning High School	40	3,900	8	175	1.3	1.3
Dysart Park	20	3,900	8	87	0.7	0.7
Lions Park	18	3,900	8	79	0.6	0.6
Future Customers/Developments						
Butterfield Development	497	1,798	8	864	2.2	6.5
Rancho San Gorgonio Development	210	924	8	217	0.5	1.6
Five Bridges Development	51	3,900	8	223	0.6	1.7
Neighborhood Park	8	3,900	8	35	0.1	0.3
Total	**1,043**	**N/A**	**N/A**	**2,530**	**8.1**	**14.8**

Notes:
(1) Source: 2006 Recycled Water Master Plan (Carollo, 2006) unless noted otherwise.
(2) Demands based on 2016 billing data.
(3) Butterfield and Rancho San Gorgonio demands estimated by respective developers.

Similar to the potable water and wastewater demands and flows, the recycled water demands considered a high and low demand envelop. Since the Butterfield Development is a large future demand that can impact the supply availability significantly, the high scenario includes the Butterfield Development is connected into the main recycled water system. The total demand of 2,530 afy presented in Table 3.24 represents this scenario. The low scenario does not include the Butterfield Development, which results in a total demand of 1,966 afy. The demand envelops are used to evaluate the different recycled water alternatives in Chapter 8.

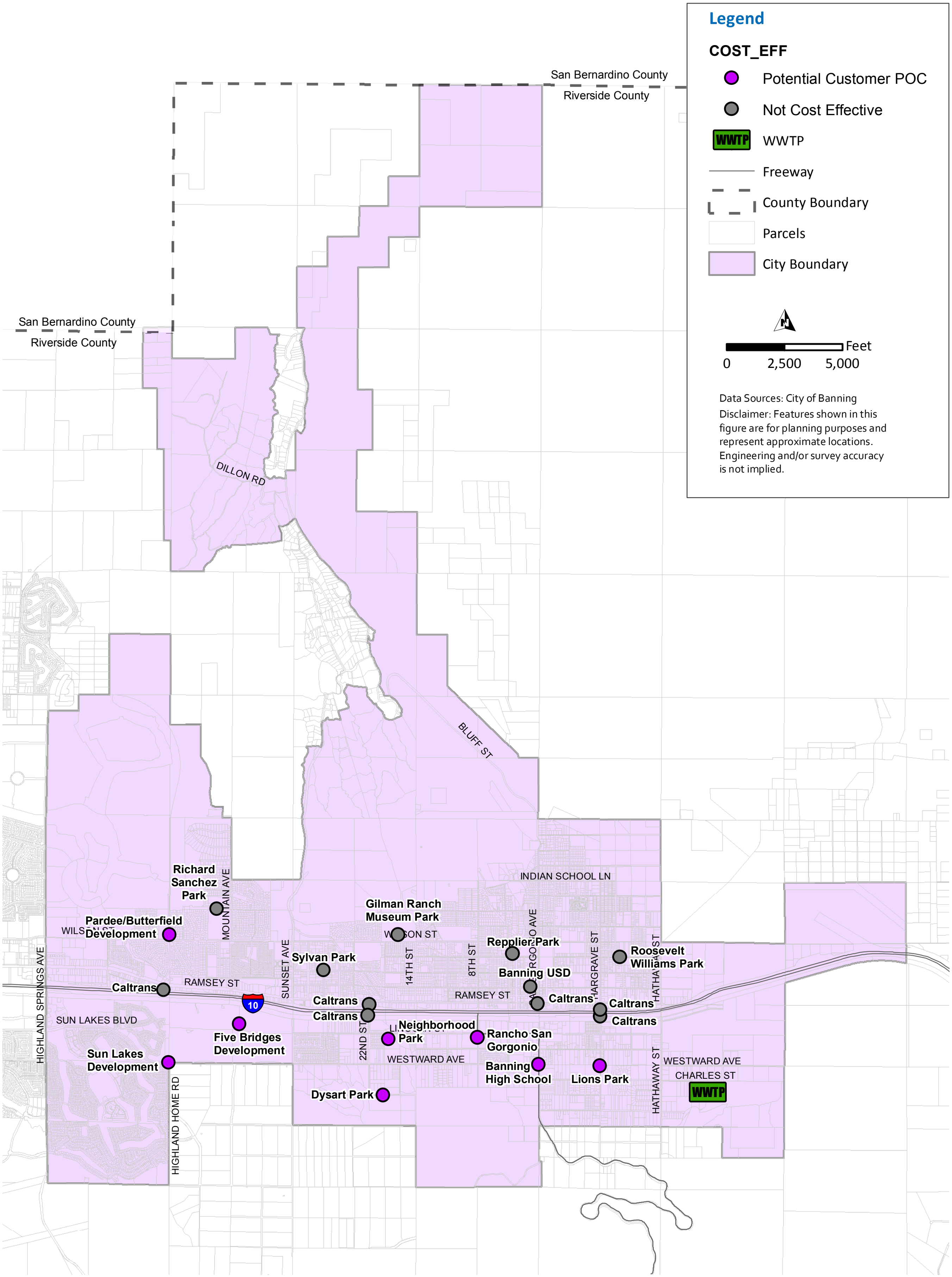

Figure 3.12 Potential Recycled Water Customers

Chapter 4

HYDRAULIC MODEL DEVELOPMENT

This chapter discusses the review and updates for the City of Banning (City)'s existing hydraulic models for potable water, wastewater, and recycled water. In addition, this chapter describes how the projected demands and wastewater flows developed in Chapter 3 were added into the existing models. The potable system hydraulic model is described in Section 4.1 . The collection system hydraulic model is described in Section 4.2 . The recycled water model is described in Section 4.3 .

4.1 Potable Water System Hydraulic Model

A potable water system hydraulic model is a simplified representation of the real potable water distribution system. Potable system models can assess the capacity of a distribution system. In addition, potable water models can perform "what if" scenarios to assess the impacts of future developments and land use changes. The City's potable water system hydraulic model was constructed using a multi-step process utilizing data from a variety of sources. This chapter summarizes the hydraulic model development process, including a summary of the modeling software selection, a description of the modeled distribution system, the hydraulic model elements, the model creation process, and the model calibration process.

4.1.1 Potable Water Hydraulic Modeling Software

There are several software applications for network analysis with a variety of capabilities and features. The selection of a particular model is generally dependent upon user preference, the requirements of the particular distribution system, and the cost associated with the software.

The City's potable water model was developed in H_20Map® Water in 2002 by MWH. Since then, Carollo Engineers, Inc. (Carollo) had updated the H_2OMap® Water model for the 2015 Review of Rancho San Gorgonio Study, the 2015 Water System Storage Analysis, and the 2016 Chromium 6 Well Study. Up to the time Carollo received the H_2OMap® Water model at the beginning of this Integrated Master Plan (IMP), the model was developed using as-built drawings. In order to more accurately input updates into the model and provide the City with better spatial approximations of potable water alignments and facilities, the hydraulic model was rebuilt as part of this Integrated Master Plan (IMP) using the City's Geographic Information Systems (GIS) Data. Furthermore, the hydraulic model was converted to InfoWater during the model update and conversion process. The current hydraulic model uses InfoWater® 12.3 Update #6. The hydraulic modeling engine for the InfoWater® software package uses the Environmental Protection Agency (EPA)'s EPANET model, which is widely used throughout the world for planning, analysis, and design related to potable water distribution systems. InfoWater® consists of multiple products that work together to bring a graphical approach to the analysis and design of potable water collection systems. The program includes seamless integration with GIS data.

4.1.2 Data Collection and Validation

The primary source for the development of the hydraulic model was the City's distribution system GIS data. The City's GIS data was digitized according to As-built documents and input from City Staff. Street centerlines were obtained from public data sources and were used for reference during model development. Additionally, City staff provided details on the City's facilities including operation set points and capacities. Section 4.1.3 describes the facilities included in the model. Figure 4.1 shows the modeled potable water distribution system.

4.1.3 Elements of the Hydraulic Model

The following provides a brief overview of the major elements of the hydraulic model and the required input parameters associated with each:

- **Junctions:** Locations where pipe sizes change or where pipelines intersect are represented by junctions in the hydraulic model. The only required inputs for junctions are the invert elevations, as well as demand and demand pattern, if any.
- **Pipes:** Transmission mains and distribution system piping are represented as pipes in the hydraulic model. Input parameters for pipes include length (which was auto calculated based on the To/From Node), friction factor (e.g., Hazen Williams C), To/From Node, diameter, and the spatial alignment.
- **Storage Tanks:** Storage tanks are used to represent reservoirs. Input parameters for storage tanks include base elevation, maximum/minimum water levels, tank diameter, and initial water level.
- **Pumps:** Pumps are included in the hydraulic model as points. Input parameters for pumps include pump curves and operational controls.
- **Reservoirs:** Reservoirs represent areas where flow enters the system. For potable modeling, a reservoir typically represents a water source. In Banning's model all sources of water are groundwater wells and every well is represented by a reservoir model element.
- **Valves:** Special valves, such as pressure-reducing, flow-control, or pressure sustaining valves are included in the hydraulic model. The input parameters include diameter and valve type (e.g., Pressure Reducing). Gate valves are typically not included in hydraulic models.

The City's hydraulic model consists of the following components:

- 5,992 junctions.
- 6,357 pipelines.
- 152 miles of pipeline (ranging from 2-inch diameter to 30-inch diameter).
- 18 pumps.
- 13 tanks.
- 4,579 Valves (ranging from 2-inch diameter to 24-inch diameter).

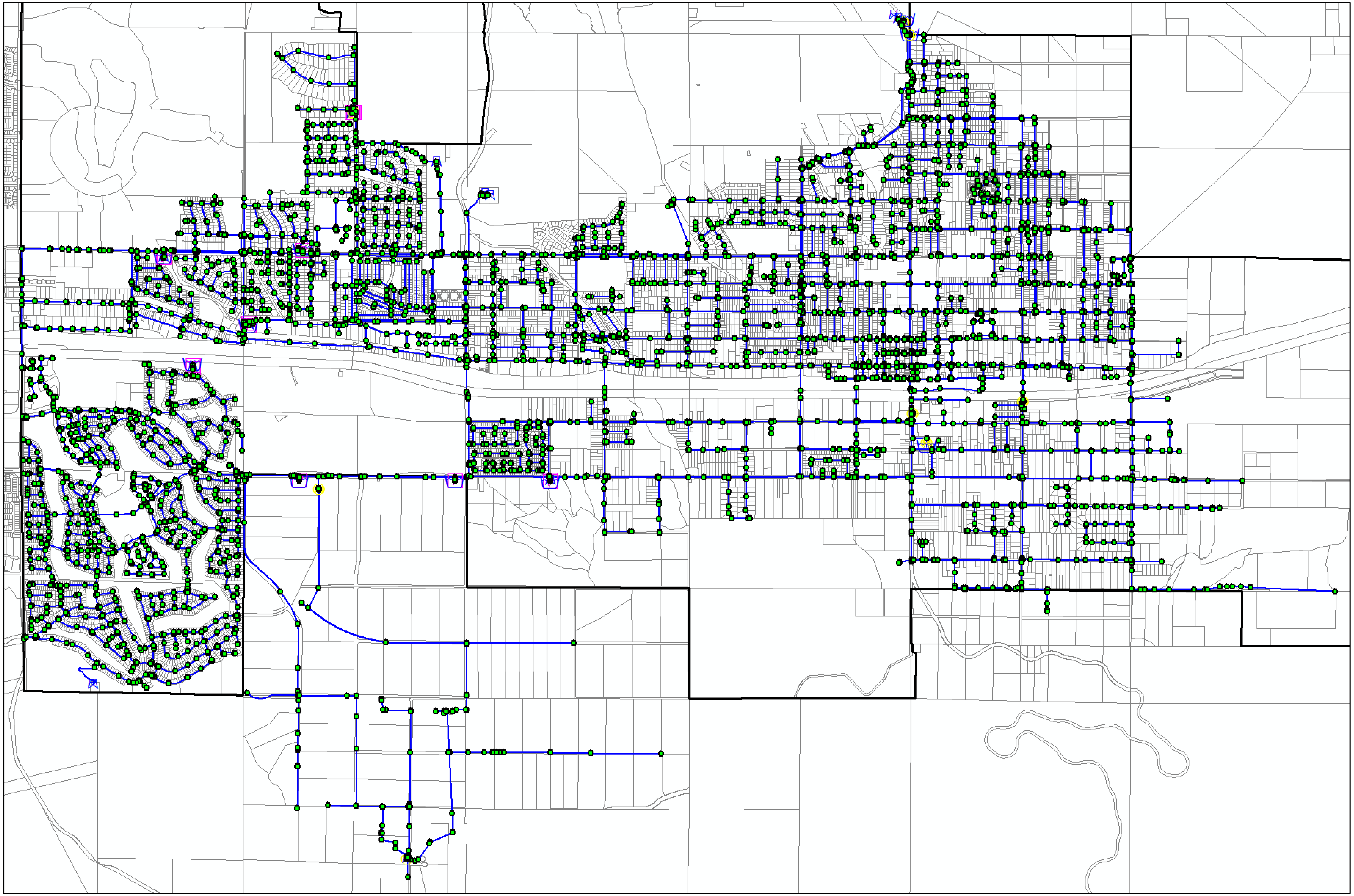

Figure 4.1 Potable Water Distribution System Model

4.1.4 Hydraulic Model Development

The City's hydraulic model combines information on the physical and operational characteristics of the potable water distribution system, and performs calculations to solve a series of mathematical equations to simulate flows in pipes.

The model construction process consisted of eight steps, as described below:

- **Step 1:** The City's GIS shapefiles for the potable water system were obtained.
- **Step 2:** The GIS data was reviewed and formatted to allow easy import into the InfoWater® modeling platform.
- **Step 3:** The distribution system pipeline data was imported into the modeling software and verified.
- **Step 4:** Junctions were generated at the intersection of pipe segments. Junction elevations were imported from the old H2OMap® model where applicable. New junctions were assigned elevation data using United States Geological Survey (USGS) contours data.
- **Step 5:** All the major facilities such as reservoirs, break tanks, well pumps, booster pumps, and specialty valves we added to the model using their GIS locations and as-built drawings when needed. Physical and operational data for the City's distribution facilities was not available from the GIS data. This type of data, such as pump on/off set points, pump capacities, valve types, valve set points, and reservoir dimensions were input manually into the model based on supervisory control and data acquisition (SCADA) data, as-built drawings, queries of City staff, and other City documentation.
- **Step 6:** Once all the relevant data was input into the hydraulic model, the model was reviewed to verify that the model data was input correctly and that the network configuration and size of the modeled pipelines were logical. Additionally, GIS topology tools were used to flag locations were pipelines should cross rather than intersect, and to flag locations where pipe segments terminated suspiciously close to the beginning of other pipe segments. These flags represented possible topography errors and were evaluated, then remedied if necessary using as-built drawings and discussions with City staff.
- **Step 7:** Potable water demands were then allocated to the appropriate model junctions, using the methods described in Section 4.1.5
- **Step 8:** The hydraulic model contains certain run parameters that need to be set by the user at the beginning of the project. These include time steps, reporting parameters, output units, and headloss equations. Once the run parameters were established, the model was debugged to ensure that it ran without errors or warnings.

4.1.5 Potable Water Demand Allocation

Determining the quantity of water demanded by City customers and how they are distributed throughout the distribution system is a critical component of the hydraulic modeling process.

Various techniques can be used to allocate water demands within the system. The preferred method is driven by the type of available information. Two common methodologies are the geocoded billing data method and the land used method. The geocoded billing data method uses the City's meters addresses from the billing database to spatially allocate the average annual water demand of each customer in the billing meter shapefile. In the land use method, the land use acreages are multiplied by a water duty factor (WDF) to obtain a spatial distribution of approximate water demands. The geocoded billing data method was used to allocate the demands for this IMP. Through the use of the City's 2016 billing records, the roughly 10,000 water meters were geocoded using the water meter address. Once the meter addresses were represented spatially throughout the water service area, demands were distributed to the model nodes. The demands were then scaled up to the average of the 2012 through 2014 supply. This average was determined to be representative of present day demands under normal, non-drought conditions, as discussed in Chapter 3. Scaling the demands to match the supply is normal practice in hydraulic modeling, to account for system losses that are not captured in the billing data.

Since the hydraulic model was not developed to represent each individual customer's service lateral, there was not a specific model node for each billing meter. In order to allocate the demands from the GIS billing meters onto the model nodes, the Thiessen polygon demand distribution method was used. The Thiessen polygon method involves using a GIS formula that generates a polygon around each of the model demand nodes. The demands from any billing meter that overlays a Thiessen polygon was applied to that demand node.

The existing annual supply is 7.7 million gallons per day (mgd), or 5,334 gallons per minute (gpm). Of the annual supply, 13 percent or 695 gpm is non-revenue water. The remaining 87 percent or 4,639 gpm represents the average annual consumption. Applying a maximum day demand (MDD) peaking factor of 1.7 (see Chapter 3), the MDD was estimated to be 9,068 gpm, or 13.1 mgd.

The hydraulic modeling software has the option of assigning 10 different demand types for each demand node. As part of the potable water demand update, 8 of the 10 different demand types were used to help identify the source of the demands in the hydraulic model. The description and demand allocated to the model for each demand type are as follows:

- **Demand Type 1:** This demand type was used to update demands for the existing system consumption (4,639 gpm). Note that this demand does not include the existing non-revenue water.
- **Demand Type 2:** This demand type was used to update demands for the existing system to account for non-revenue water (695 gpm). Non-revenue water was distributed evenly amongst the nodes that contained Demand Type 1, this demand is 13 percent of the existing consumption.
- **Demand Type 3:** This demand type was used to represent the near-term (2025) known developments (790 gpm).
- **Demand Type 4:** This demand type was used to distribute the near-term (2025) infill (741 gpm).
- **Demand Type 5:** This demand type was used to represent the long-term (2040) known developments (1,584 gpm).
- **Demand Type 6:** This demand type was used to distribute the long-term (2040) infill (0 gpm). This demand was set to zero because the demands of the known developments slightly surpassed the total projected demand based on population growth estimates in 2040, causing no need to distribute infill.
- **Demand Type 7:** This demand type was used to represent the build out (post 2040) known developments (1,426 gpm).

- **Demand Type 8:** This demand type was used to distribute the build out (post 2040) infill (2,587 gpm).
- **Demand Type 9:** This demand type was used for the recycled water offset scenario for the selected project alternative listed in Chapter 8. These demands are represented as negative values in the model. All other future scenarios assume potable water usage without recycled water offset since the recycled water analysis performed was preliminary and may change as the project alternative is further developed. However, non-potable water wells were included as a supply source in the future scenarios.

Each of the eight demand types used were input as average day demand (ADD). The demands were entered into the hydraulic model as ADD in order to create a baseline demand set and thus reduced the need for excess demand sets, which reduces the overall time it takes to modify and update demands. Also, a demand set this represents the ADD condition can easily be manipulated by the model global multiplier and/or diurnals patterns, depending on the analysis to be performed. The global multiplier run parameter in the hydraulic modeling software is used to scale up the demand sets by a given number for example: the MDD peaking factor in Table 3.5 is defined as 1.7. By changing the global multiplier to 1.7, the hydraulic model can simulate a MDD model run.

In addition to adjusting the global demand multiplier for seasonal or daily variations, the hydraulic model was set up with the capability of adjusting the hourly variation through diurnal patterns. Different classes of water users require supply from the distribution system at different times of the day. A diurnal curve, or demand pattern, simplifies the typical variation of hourly demands for the City's customers over the course of a day. In general, typical diurnal curves vary for residential, commercial, and landscape irrigation water users, and will vary for individual users.

As discussed in Chapter 3, diurnal curves are typically calculated based on data gathered as a part of model calibration. The City's available data allowed for calculation of diurnal curves for the system as a whole. Due to the lack of complete hourly flow data at the City's Canyon Wells, pump stations (PSs) and pressure reducing stations (PRSs), it was not possible to develop diurnal curves for individual pressure zones. A complete set of hourly well production data, as well as daily production data for the Canyon Wells was available from May 8, 2017 to May 24, 2017. A diurnal curve was calculated for the entire distribution system from the production data from May 15, 2017. This diurnal curve is presented in Figure 4.2 and was used for all the modeling analysis in this report. The peak hour demand (PHD) on May 15, 2017 was calculated to be 1.78, which is presented in Table 3.5. Once the City's SCADA system is upgraded, the peaking factor should be reevaluated.

Since the diurnal pattern shown on Figure 4.2 does not represent a typical diurnal pattern, an example of a typical diurnal pattern has been included on Figure 4.3 for comparative purposes. This typical diurnal pattern was not used in any of the modeling analysis, but has been provided in the hydraulic model for the City's use as needed.

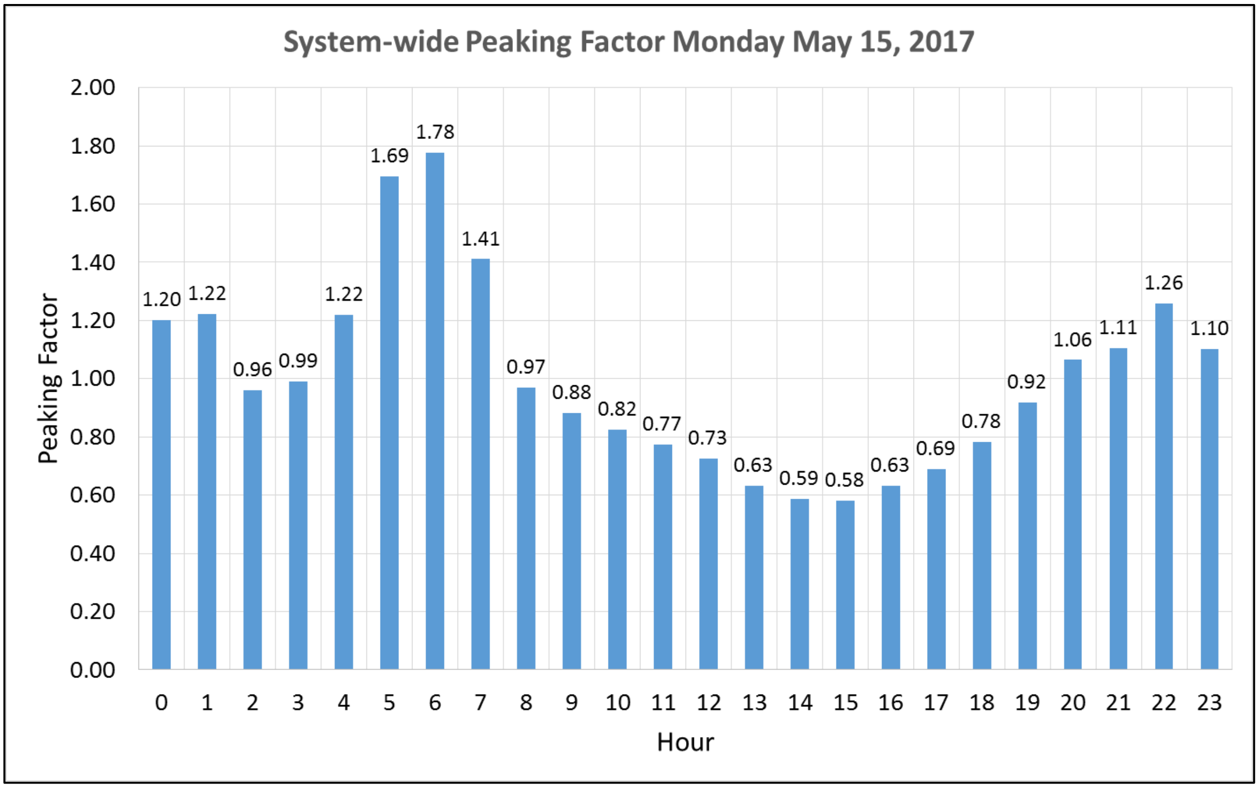

Figure 4.2 Potable Water Diurnal Pattern (Calculated)

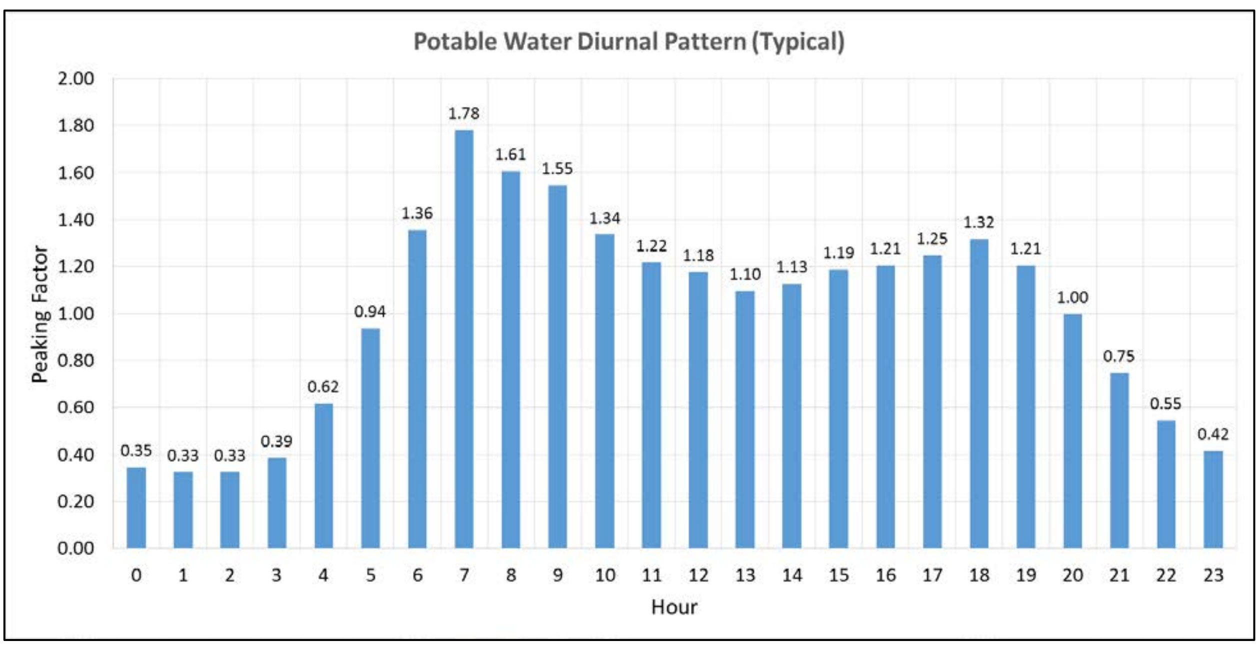

Figure 4.3 Potable Water Diurnal Pattern (Typical)

4.1.6 Hydraulic Model Calibration

The purpose of a water system hydraulic model is to predict how a water distribution system will respond under a given set of conditions. One way to test the accuracy of the hydraulic model is to create a set of known conditions in the water system and then compare the results observed in the field against the results of the hydraulic model simulation using the same conditions. Fire flow tests conducted in the field on the water system can yield a profound tool in verifying data used in the hydraulic model and a greater understanding of how the water system operates.

Field testing can indicate errors in the data used to develop the hydraulic model, or show that a condition might exist in the field not otherwise known. Valves, which are reported as being open, might actually be closed (or vice versa), an obstruction could exist in a pipeline, or pressure settings for a PRS may be slightly different than noted. Field testing can also correct erroneous model data such as incorrect pipeline diameters or connections. Aside from a few specific cases noted in the following subsections, no discrepancies were encountered during model calibration that hadn't already been addressed during the model update process. Data obtained from the field tests can be used to determine appropriate roughness coefficients for each pipeline, as roughness coefficient can vary with age and pipe material. Other parameters can also be adjusted to generate a calibrated model.

The calibration process for the City's water distribution system hydraulic model consisted of three parts, a macro calibration, and extended period simulation (EPS) calibration, and a fire flow test calibration. Prior to calibration a Calibration Plan was developed, which described in detail the methods used to collect the calibration data. The Calibration Plan and field results are provided as Appendix B.3. The following sections summarize the calibration process and results.

4.1.6.1 Macro Calibration

Initially, the model was run under existing demand conditions and necessary adjustments were made to produce reasonable system pressures and reservoir level fluctuations. Such adjustments include modifications of pipeline connectivity, operational controls, ground elevations, and facility characteristics.

The macro calibration process involved several steps to verify that the model produces reasonable results:

- **Transmission Main Connectivity.** Using the connectivity features of the modeling software, the connectivity of the water mains within the distribution system was verified. Problems found using the connectivity locators were reviewed to determine whether adjustments were needed to the connectivity of the model. Output reports of pipeline flow characteristics, such as headloss (feet per thousand feet [ft/kft]) and velocity (feet per second [fps]) were also used to locate problem areas where additional adjustments could be necessary.
- **System Pressures.** The macro calibration compared the model output to the typical pressures observed within the distribution system in psi. This process was used to locate major errors in model creation, elevations, or connectivity, as well as changes that reflect how operational controls of the system should be implemented in the model.
- **Facility Characteristics.** Hydraulic model results were compared to data provided by the City to verify that facility attributes entered into the model, such as the physical

characteristics of the tanks and pumps, produced results comparable to what the City experiences.

4.1.6.2 Extended Period Simulation Calibration

The extended period calibration is intended to calibrate the EPS capabilities of the hydraulic model by closely matching the model pressures and flows to field conditions over a 24-hour period of similar demand and system boundary conditions. Pressure data and flows from meter connections were recorded to create diurnal patterns and obtain EPS calibration data. The primary varied parameters for this calibration were operational controls and PRS set points, although other parameters were also adjusted as calibration results were generated. From the calibration period, May 15, 2017, was selected to be used for the 24-hour EPS calibration day. This was chosen because it was one of the few days when available SCADA data and pressure logger data overlapped. Additionally, the diurnal pattern used in the model was calculated from this day. The calculated daily demand for the calibration day was about 6.2 mgd (4,309 gpm), which is 1.5 mgd lower than the average day demand from 2012 to 2016, or 7.7 mgd. Hence, the model calibration day had a seasonal peaking factor of 0.8. For the EPS calibration, the ADD was adjusted by multiplying the demands on all demand nodes by 0.8 to match this estimated demand condition during the calibration day. The EPS calibration compared model simulated PS flows, discharge pressures, reservoir levels, and storage tank levels. In addition, model simulated pressures at the pressure recorder locations were compared to the actual field pressures recorded during the calibration day. The model calibration results of all comparison points are included in Appendix B.4, while a few examples are shown on Figure 4.4, and Figure 4.5.

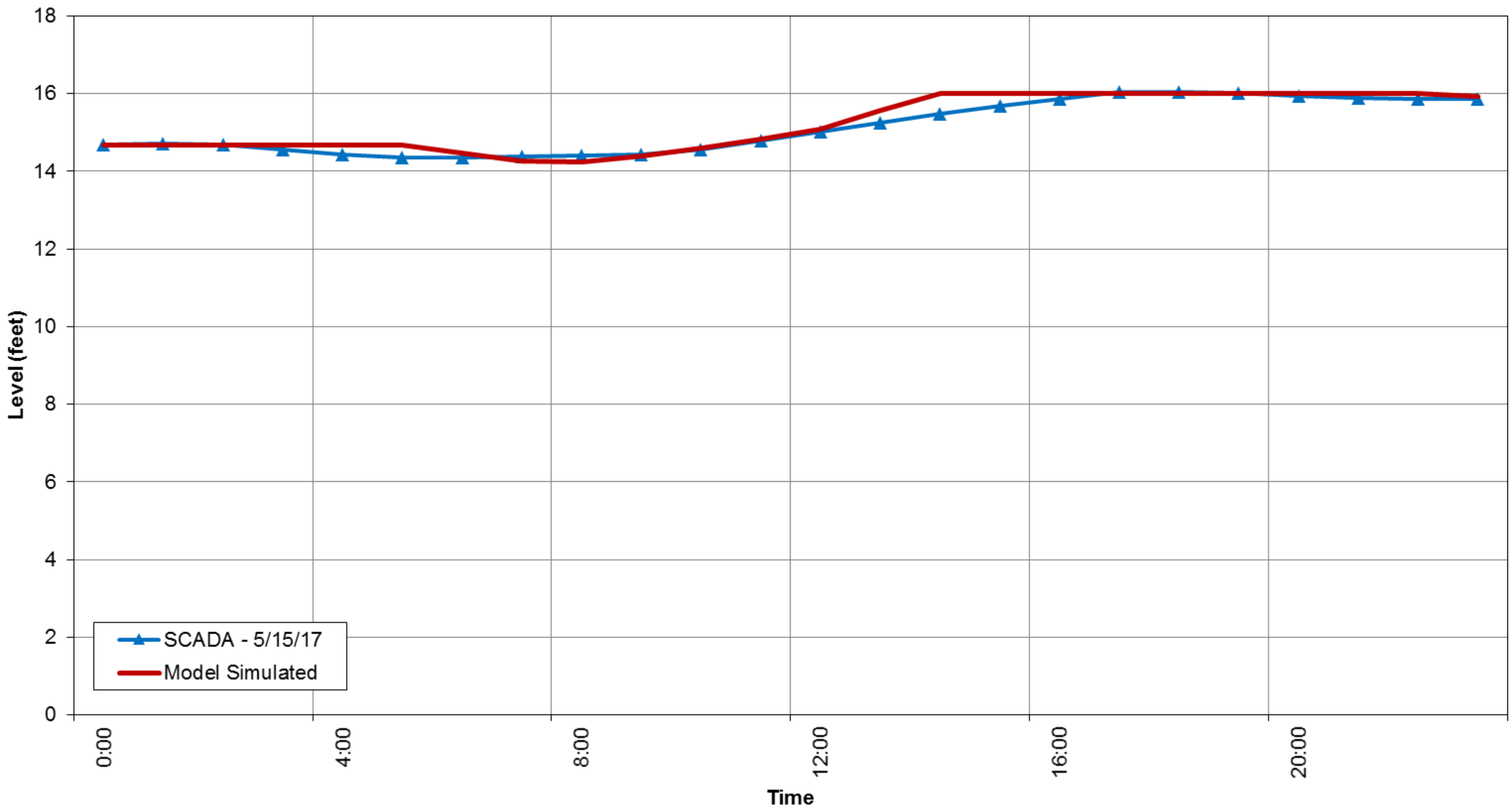

Figure 4.4 Brinton Reservoir EPS Calibration

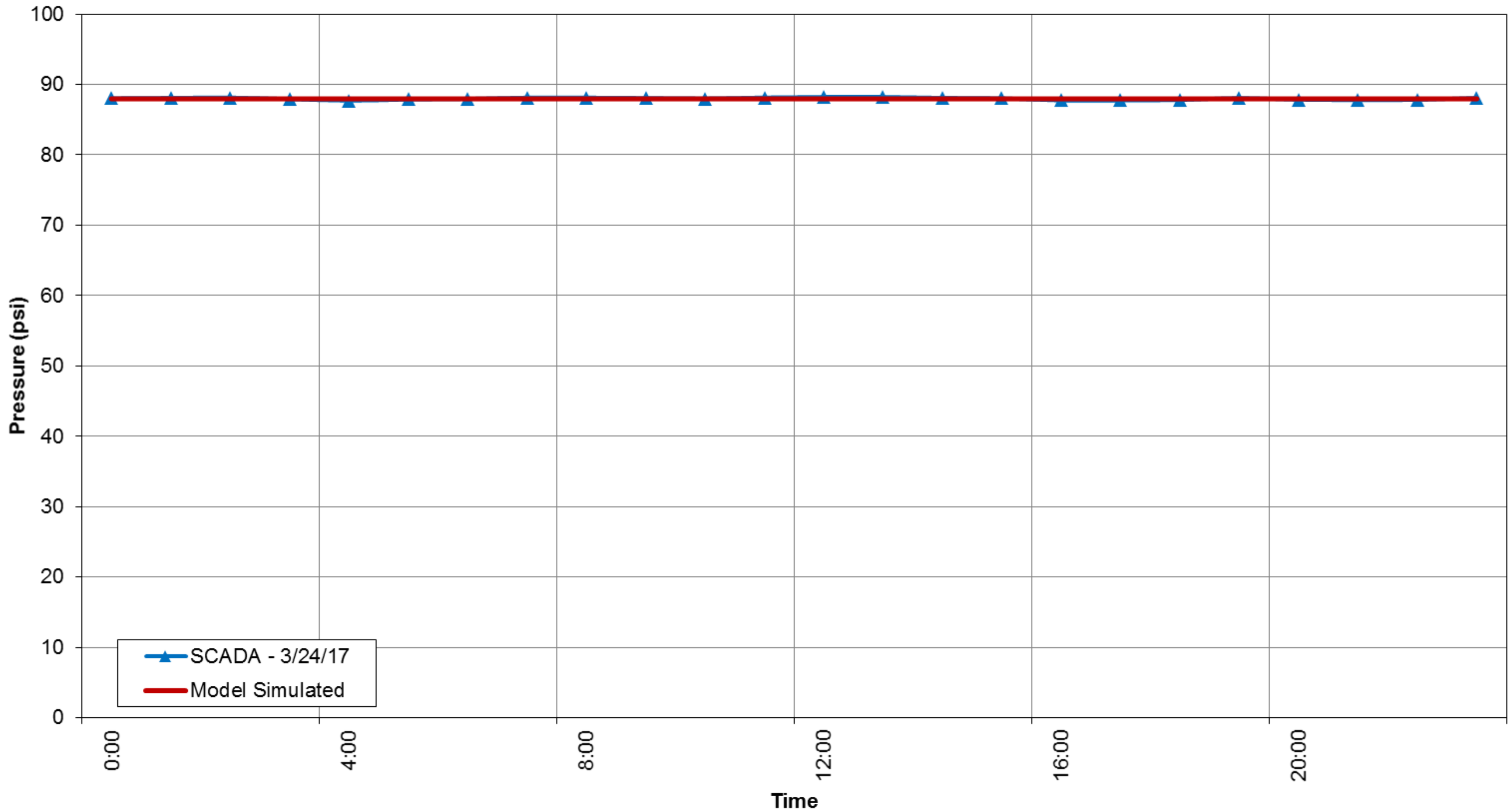

Figure 4.5 Pressure Logger 35 EPS Calibration

As shown in Figure 4.4 and Figure 4.5, the model simulated data closely matches the trend and magnitude of the SCADA data in these figures. Overall, taking into account all the calibration graphs, the trends seen in the SCADA data were consistent with the predicted planning level modeling results. Some notable model modification and observations from the EPS model calibration include:

- Groundwater well pumps design head and design flow were altered to reflect the hydraulic grade lines (HGLs) recorded by the pressure loggers in the field and flows recorded by SCADA.
- The Sunset Reservoir Levels follow the general trend seen in the SCADA data. However, at certain points during the day, the modeled levels are approximately 1 foot different from the field measured levels. Extensive effort was put into modeling the controls of the supplies in this zone, but the level difference could not be resolved. Overall, the difference in the reservoir levels was attributed to a possible difference in demand between the modeled diurnal and the actual diurnal pattern for that zone, which could not be calculated from the available SCADA data.
- All The pressure loggers had results that followed the same trend as the SCADA data and were within 1 psi of the recorded pressure data, with the exception of Pressure Logger 38. The model simulated results for Pressure Logger 38 were systematically 4 psi higher than the recorded field data.

4.1.6.3 Fire Flow Calibration

The calibration of fire flow tests is intended to closely match model simulated pressures to field pressures under similar high demand and system boundary conditions. The primary parameter that is modified during this calibration step is the pipeline roughness coefficient. However, other parameters, such as connectivity, may also be adjusted as calibration results are generated.

Hazen-Williams roughness coefficients, or C-factors, have industry accepted value ranges based on pipeline material, diameter, and age. Characteristics specific to the City water distribution system such as water quality, temperature, construction methodologies, material suppliers, and other factors may result in roughness coefficients that differ from the average of the industry accepted ranges. Fire flow calibration is used to refine the initial estimation of the roughness coefficients to better match the conditions of the City's distribution system.

During average day demand conditions, roughness coefficients have a relatively small effect on system pressure in the distribution system. However, as flow rates increase in the system on higher demand days, velocity within pipelines increase and roughness coefficients contribute more to overall system headloss and system pressures. Fire flow tests artificially create high demand conditions to generate more headloss, allowing a better estimation of the pipeline roughness coefficients.

Fire flow tests stress the distribution system by creating a differential between the HGL at the point of hydrant flow and the system HGL at neighboring hydrants. This HGL differential increases the effect of the roughness coefficients on system headloss and allows adjustments to the model to match model pressures to field pressures within an acceptable tolerance. As the model is adjusted to match system pressures, roughness coefficients should be adjusted only within a reasonable tolerance of industry accepted roughness coefficient ranges. If a model is unable to match the calibration results within the acceptable C-factor range for a given pipeline material and age, there may be cause for further investigation of a previously unknown field condition. Examples of such conditions include closed valves, partially closed or malfunctioning valves, extreme corrosion within pipelines, erroneous model network connectivity, incorrect diameter in GIS layers or record drawings, and the influence of unique diurnal patterns of large water users.

Two separate hydraulic model scenarios were created for each of the 12 flow tests, one to simulate a static pressure condition, and one to simulate fire hydrant flow conditions. The flow observed at each fire flow hydrant was assigned as a demand to the model node at the location of the hydrant. Since the fire flow calibration is a steady state simulation, model demands were adjusted in each fire test scenario to match the time that the tests were conducted. Residual pressures were then read at each hydrant location while the hydrant was flowing. Model results were considered acceptable if they were within a 10 percent tolerance. A summary of the fire test model calibration results are shown in Table 4.1.

4.1.6.4 Potable Water Calibration Summary

In summary, the calibration results indicate the model generally predicts conditions similar to those observed in the field. In the Mountain South Pressure Zone of the model, there are some unknown local conditions that cause the model results to slightly deviate from field conditions. However, the overall distribution system is well represented by the model.

Based on the results of the calibration, it can be concluded that the model is calibrated to extended period simulation and steady state fire flow. Utilizing the available field data and input from City staff, the model represents the City's distribution system and system operations to a level suitable to support the City's future hydraulic modeling analysis.

The Banning Water Canyon wells and pipes were simplified to a single input into the distribution system. This was due to a lack of SCADA data and missing as-built information at the time of model creation. Although the pipes and wells were drafted into the model, those facilities were deactivated during all model runs.

Table 4.1 Fire Flow Calibration Results

Fire Flow Test Results
City of Banning
Potable Water Hydraulic Modeling
Test Date: 03/16/2017

Test Site	Time	Hydrant Type	Hydrant ID	Field Measured Data: Hydrant Flow (F-1) (gpm)	Field Measured Data: Hydrant Pressure (psi) Static	Field Measured Data: Hydrant Pressure (psi) Residual	Model Simulated Data: Hydrant Pressure (psi) Static	Model Simulated Data: Hydrant Pressure (psi) Residual	Percent Difference: Static	Percent Difference: Residual	Pressure Drop: Measured (psi)	Pressure Drop: Modeled (psi)	Pressure Drop: Difference
1	9:02	P-1	R04308	932	92.5	41	98	38	5.9%	-7.3%	51.5	60	9
		P-2	R04310		100	43	100	40	0.0%	-7.0%	57	60	3
2	--	P-1	T05300	1,320	82	80	--	--	--	--	--	--	--
	--	P-2	S05313		76	72.5	--	--	--	--	--	--	--
3	--	P-1	U03327	1,610	112.5	111	--	--	--	--	--	--	--
	--	P-2	T03350		111	109	--	--	--	--	--	--	--
4	--	P-1	W02312	1,409	85	84	--	--	--	--	--	--	--
	--	P-2	W02304		83	82	--	--	--	--	--	--	--
5	--	P-1	V05313	1,650	121.5	118	--	--	--	--	--	--	--
	--	P-2	V05311		117	117	--	--	--	--	--	--	--
6	--	P-1	S07301	1,412	77.5	77	--	--	--	--	--	--	--
	--	P-2	S07308		94	93	--	--	--	--	--	--	--
7	--	P-1	R09311	1,118	101	58	--	--	--	--	--	--	--
	--	P-2	R09308		108	62	--	--	--	--	--	--	--
8	--	P-1	T09324	1,730	141	130	--	--	--	--	--	--	--
	--	P-2	T08321		138	--	--	--	--	--	--	--	--
9	13:45	P-1	W07303	1,890	169	159	163	150	-3.6%	-5.7%	10	13	-3
		P-2	--		--	--	--	--	--	--	--	--	--
10	14:01	P-1	W10312	1,456	110	93	110	91	0.0%	-2.2%	17	19	2
		P-2	W10314		111	100	114	99	2.7%	-1.0%	11	15	4
11	9:34	P-1	Z05305	953	117.5	57.5	116	56	-1.3%	-2.6%	60	60	0
		P-2	Z05306		91	38	93	33	2.2%	-13.2%	53	60	7
12	14:01	P-1	Z05301	500	118	27	117	25	-0.8%	-7.4%	91	92	1
		P-2	--		--	--	--	--	--	--	--	--	--

Fire Flow Test Results
City of Banning
Potable Water Hydraulic Modeling
Test Date: 03/16/2017

				Field Measured Data			Model Simulated Data		Percent Difference		Pressure Drop		
		Hydrant		Hydrant Flow (F-1 and F2-)	Hydrant Pressure (psi)		Hydrant Pressure (psi)				Measured	Modeled	
Test Site	Time	Type	Hydrant ID	(gpm)	Static	Residual	Static	Residual	Static	Residual	(psi)	(psi)	Difference
1	--	P-1	R04308	--	--	--	--	--	--	--	--	--	--
	--	P-2	R04310		--	--	--	--	--	--	--	--	--
2	8:49	P-1	T05300	2,685	82	78	82	78	0.0%	0.0%	4	4	0
		P-2	S05313		76	72.5	82	76	7.9%	4.8%	3.5	6	-3
3	12:12	P-1	U03327	2,610	112.5	110	113	103	0.4%	-6.4%	2.5	10	-8
		P-2	T03350		111	109	111	100	0.0%	-8.3%	2	11	9
4	11:32	P-1	W02312	2,789	85	82.5	80	75	-5.9%	-9.1%	2.5	5	3
		P-2	W02304		83	80	78	75	-6.0%	-6.3%	3	3	0
5	13:29	P-1	V05313	3,300	121	117	114	105	-5.8%	-10.3%	4	9	5
		P-2	V05311		119	117	117	115	-1.7%	-1.7%	2	2	0
6	8:25	P-1	S07301	2,680	77.5	75	76	75	-1.9%	0.0%	2.5	1	2
		P-2	S07308		94	94	91	90	-3.2%	-3.7%	0.5	1	-1
7	7:50	P-1	R09311	2,388	101	58	99	60	-2.0%	3.4%	43	39	4
		P-2	R09308		108	62	102	62	-5.6%	0.0%	46	40	6
8	14:23	P-1	T09324	3,495	141	133	140	119	-0.7%	-10.9%	8	21.5	14
		P-2	T08321		138	135	136	131	-1.4%	-3.0%	3	5	2
9	--	P-1	W07303	--	--	--	--	--	--	--	--	--	--
	--	P-2	--		--	--	--	--	--	--	--	--	--
10	--	P-1	W10312	--	--	--	--	--	--	--	--	--	--
	--	P-2	W10314		--	--	--	--	--	--	--	--	--
11	--	P-1	Z05305	--	--	--	--	--	--	--	--	--	--
	--	P-2	Z05306		--	--	--	--	--	--	--	--	--
12	--	P-1	Z05301	--	--	--	--	--	--	--	--	--	--
	--	P-2	--		--	--	--	--	--	--	--	--	--

4.2 Sewer Collection System Hydraulic Model

A sewer collection system model is a simplified representation of the real sewer system. Sewer system models can assess the conveyance capacity for a collection system. In addition, sewer system models can perform "what if" scenarios to assess the impacts of future developments and land use changes. The City's collection system hydraulic model was constructed using a multi-step process utilizing data from a variety of sources. This chapter summarizes the hydraulic model development process, including a summary of the modeling software selection, a description of the modeled collection system, the hydraulic model elements, the model creation process, and the model calibration process.

4.2.1 Sewer Collection System Hydraulic Modeling Software

There are several software applications for network analysis with a variety of capabilities and features. The selection of a particular model is generally dependent upon user preference, the requirements of the particular collection system, and the cost associated with the software.

The City was previously using H2OMAP Sewer® software as their hydraulic modeling platform for their collection system. However, H2OMAP Sewer® uses simplified routing solutions (Muskingum-Cunge equation) and is often considered a semi dynamic model, with limited capabilities for backwater conditions and surcharging. Therefore, InfoSWMM was recommended to provide a fully dynamic model. The hydraulic modeling engine for the InfoSWMM® software package uses the EPA's Storm Water Management Model (SWMM), which is widely used throughout the world for planning, analysis, and design related to stormwater runoff, combined sewers, sanitary sewers, and other drainage systems. InfoSWMM® routes flows through the model using the Dynamic Wave method, which solves the complete Saint Venant, one dimensional equations of fluid flow.

InfoSWMM® consists of multiple products that work together to bring a graphical approach to the analysis and design of wastewater and stormwater collection systems. The program includes seamless integration with GIS data.

4.2.2 Data Collection and Validation

The source for the development of the hydraulic model was the City's hydraulic model and sewer system GIS data. The existing system was update with the City's GIS, while the hydraulic model was used to fill in unavailable data from the GIS and to include growth related projects. The City's GIS data was digitized according to As-built documents and input from City Staff. Street centerlines were obtained from public data sources and were used for reference during model development. Additionally, City staff provided details on the City's lift stations. Figure 4.6 shows the modeled wastewater collection system.

4.2.3 Elements of the Wastewater Hydraulic Model

The following provides a brief overview of the major elements of the hydraulic model and the required input parameters associated with each:

- **Junctions:** Sewer manholes, cleanouts, as well as other locations where pipe sizes change or where pipelines intersect are represented by junctions in the hydraulic model. Required inputs for junctions include rim elevation, invert elevation, and surcharge depth (used to represent pressurized systems). Junctions are also used to represent locations where flows are split or diverted between two or more downstream links.

- **Pipes:** Gravity sewers and force mains are represented as pipes in the hydraulic model. Input parameters for pipes include length, friction factor (e.g., Manning's n for gravity mains, Hazen Williams C for force mains), invert elevations, diameter, and whether or not the pipe is a force main.
- **Storage Nodes:** For sewer system modeling, storage nodes typically are used to represent lift station wet wells (although other storage basins, etc. can be modeled as storage nodes). Input parameters for storage nodes include invert elevation, wet well depth, and wet well cross section.
- **Pumps:** Pumps are included in the hydraulic model as links. Input parameters for pumps include pump curves and operational controls.
- **Outfalls:** Outfalls represent areas where flow leaves the system. For sewer system modeling, an outfall typically represents the connection to the influent pump station or headworks of a wastewater treatment plant (WWTP).
- **Rain Gauges:** Rain gauges are input into the hydraulic model to simulate historical or theoretical hourly rainfall events.

- **Inflows:** The following are the three types of wastewater flow sources that can be injected into individual model junctions (and storage nodes):

  - External. External inflows can represent any number of flows into the collection system, such as metered flow data or groundwater inflow. External inflows are applied to a specific model junction by applying a baseline flow value and a pattern that varies the flow by hour, day, or month of the year.
  - Dry Weather. Dry weather inflows simulate base sanitary wastewater flows and represent the average flow. The dry weather flows can be multiplied by up to four patterns that vary the flow by month, day, hour, and day of the week (e.g., weekday or weekend). The dry weather diurnal patterns are adjusted during the dry weather calibration process.
  - Rainfall Derived Infiltration and Inflow (RDII). RDII flows are applied in the model by assigning a unit hydrograph and a corresponding tributary area to a given junction. The unit hydrographs consists of several parameters that are used to adjust the volume of RDII that enters the system at a given location. These parameters are adjusted during the wet weather calibration process.

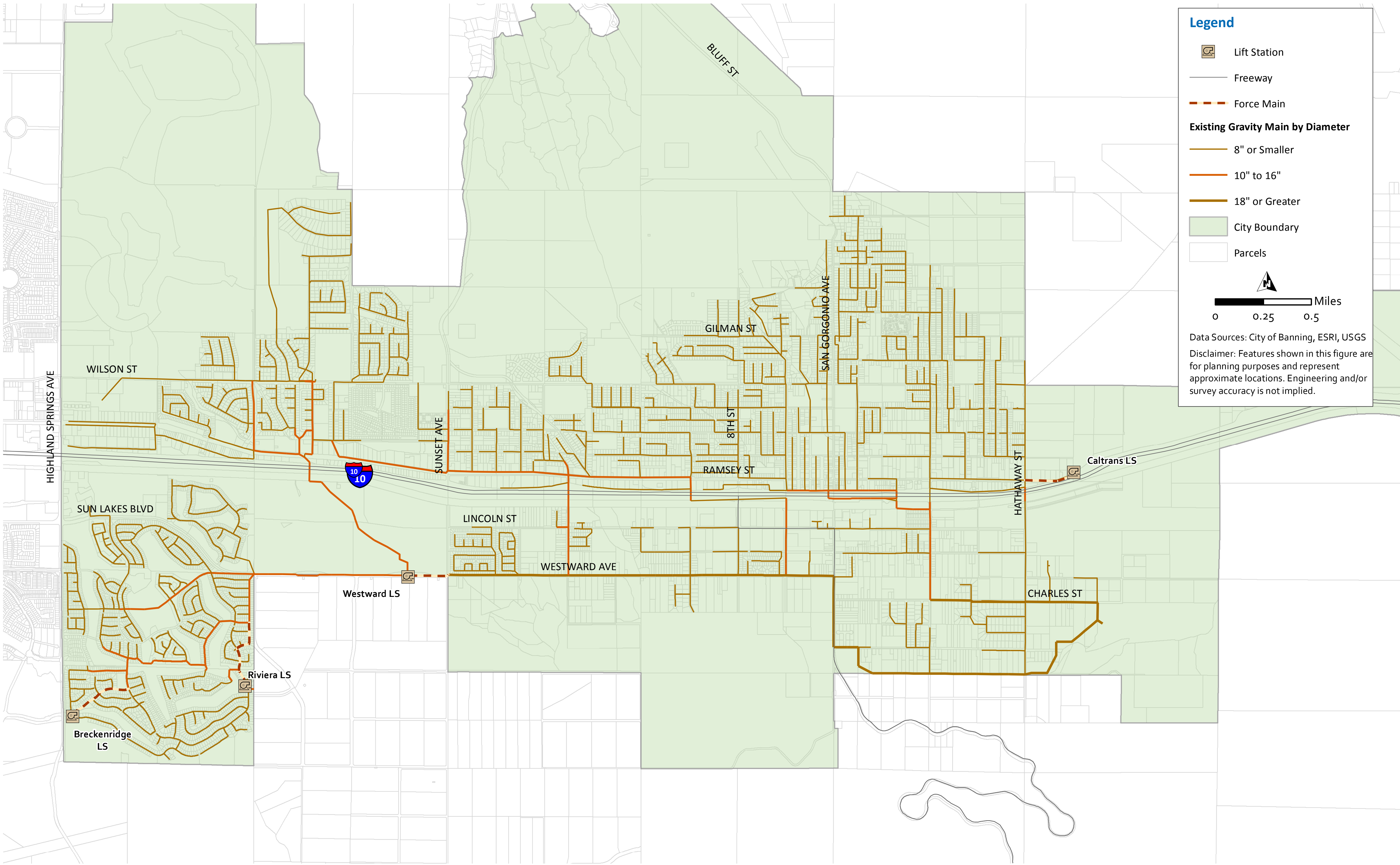

Figure 4.6 Wastewater Collection System Model

4.2.4 Wastewater Hydraulic Model Construction

The City's hydraulic model combines information on the physical and operational characteristics of the wastewater collection system and performs calculations to solve a series of mathematical equations to simulate flows in pipelines.

The model construction process consisted of six steps, as described below:

- **Step 1:** The City's GIS shapefiles for the sewer collection system were obtained.
- **Step 2:** The GIS data was reviewed and formatted to allow easy import into the InfoSWMM® modeling platform.
- **Step 3:** The collection system pipeline and facility data were imported into the modeling software and verified. Physical and operational data for the City's wastewater collection facilities was not available from the GIS data. This type of data, such as wet well dimensions, pump stations, and other special features, were input manually into the model based on available information. In addition, pipelines and junctions with missing inverts or invert discrepancies were reviewed and manually input or modified based on City records, field reconnaissance, and engineering judgment.
- **Step 4:** Once all the relevant data was input into the hydraulic model, the model was reviewed to verify that the model data was input correctly and that the flow direction and size of the modeled pipelines were logical. Additionally, the modeled lift stations were also checked to verify that they operated correctly.
- **Step 5:** Dry weather wastewater flows were then allocated to the appropriate model junctions. These flows were scaled up or down, as necessary, to match the dry weather flows recorded during the flow monitoring period.
- **Step 6:** The hydraulic model contains certain run parameters that need to be set by the user at the beginning of the project. These include run dates, time steps, reporting parameters, output units, and flow routing method. Once the run parameters were established, the model was debugged to ensure that it ran without errors or warnings.

4.2.5 Wastewater Load Allocation

Determining the quantity of base wastewater flows generated by a municipality and how they are distributed throughout the collection system is a critical component of the hydraulic modeling process.

Various techniques can be used to assign wastewater flows to individual model junctions, depending on the type of data that is available. Adequate estimates of the volume of wastewater are important in maintaining and sizing sewer system facilities, both for present and future conditions. Baseline wastewater loads were allocated (assigned to specific nodes) in the hydraulic model based on a combination of water billing records and land use data provided by the City, as well as the flow data from the temporary flow monitoring program. The following steps outline the wastewater load allocation process:

- **Step 1:** The City's service area was broken up into 1,283 individual loading polygons. In a "skeletonized" (i.e., truncated model) model, a loading polygon will usually encompass a particular subdivision or grouping of lots. In an all pipe model, such as the City's hydraulic model, a loading polygon could be as small as a few parcels. Each loading polygon represents the geographic area that contributes flows into a single model node (i.e., manhole), and was developed using GIS based on the City's parcel and sewer pipeline shapefiles.

- **Step 2:** One approach for estimating the existing dry weather wastewater flow associated with each loading polygon is based on land use designations, flow coefficients, and land use area.
  - In reality, the wastewater generation rates of each existing customer will vary from an average flow coefficient (significantly in some cases). For this reason, water billing records can be considered as an alternative to the land use based load allocation method for existing dry weather flows. For this project, water consumption billing records by parcel were available. For each parcel within the collection system service area, the annual average water consumption for 2016 was calculated in GIS. Winter water demand is used because landscape water use is minimal. The parcel demands were then merged with the loading polygons in GIS and the total demand for each loading polygon was calculated. The water demands were imported into the hydraulic model using InfoSWMM's "Load Allocator" tool.
- **Step 3:** Once the existing wastewater flows were allocated into the model, they were adjusted as needed during model calibration to closely match the dry weather flows recorded during the flow monitoring program. This adjustment accounts for the "return to sewer" ratio, which varies throughout the system.

4.2.6 Wastewater Hydraulic Model Calibration

Hydraulic model calibration is a crucial component of the hydraulic modeling effort. Calibrating the model to match data collected during the flow monitoring program to achieve the most accurate results possible. The calibration process consists of calibrating to both dry and wet weather conditions.

For this project, dry weather flow monitoring was conducted at nine metering sites for a period of approximately one month. Dry weather flow (DWF) calibration provides an accurate depiction of base wastewater flow generated within the study area. The wet weather flow (WWF) calibration consists of calibrating the hydraulic model to a specific storm event or events to accurately simulate the peak and volume of infiltration/inflow (I/I) into the sewer system. The amount of I/I is essentially the difference between the WWF and DWF components.

4.2.6.1 Wastewater Calibration Standards

The hydraulic model was calibrated in accordance with international modeling standards. The Wastewater Planning Users Group (WaPUG), a section of the Chartered Institution of Water and Environmental Management, has established generally agreed upon principles for model verification. The dry weather and wet weather calibration focused on meeting the recommendations on model verification contained in the "Code of Practice for the Hydraulic Modeling of Sewer Systems," published by the WaPUG (WaPUG 2002), as summarized below:

- **Dry Weather Calibration Standards:** Dry weather calibration should be carried out for two dry weather days and the modeled flows and depths should be compared to the field measured flows and depths. Both the modeled and field measured flow hydrographs should closely follow each other in both shape and magnitude.

In addition to the shape, the flow hydrographs should also meet the following criteria as a general guide:

- The timing of flow peaks and troughs should be within one hour.
- The peak flow rate should be within the range of ±10 percent.
- The volume of flow (or the average rate of flow) should be within the range of ±10 percent. If applicable, care should be taken to exclude periods of missing or inaccurate data.

- **Wet Weather Calibration Standards:** The model simulated flows should be compared to the field measured flows. The flow hydrographs for both events should closely follow each other in both shape and magnitude, until the flow has substantially returned to dry weather flow rates.

  In addition to the shape, the flow hydrographs should also meet the following criteria as a general guide:

  - The timing of the peaks and troughs should be similar with regard to the duration of the events.
  - The peak flow rates at significant peaks should be in the range of +25 percent to -15 percent and should be generally similar throughout.
  - The volume of flow (or the average flow rate) should be within the range of +20 percent to -10 percent.

4.2.6.2 Dry Weather Flow Calibration

The DWF calibration process consists of several elements, as outlined below:

- **Divide the system into flow meter tributaries.** The first step in the calibration process was to divide the City into flowmeter tributary areas. Nine tributary areas were created, one for each flow meter from the temporary flow monitoring program. A map showing the locations of each flow monitoring site and their associated tributary area are provided in Chapter 3 along with a schematic of the flow meters.

- **Define flow volumes within each area.** The next step was to define the flow volumes within each area, which was accomplished in the flow allocation step.

- **Create diurnal patterns to match the temporal distribution of flow.** A diurnal curve is a pattern of hourly multipliers that are applied to the base flow to simulate the variation in flow that occurs throughout the day. Two diurnal curves were developed for each flow monitoring tributary area, one representing weekday flow and one representing weekend flow. The diurnal patterns were initially developed based on the flow monitoring data and adjusted as part of the calibration process until the model simulated flows matched the field measured flows as closely as possible. Figure 5.2 shows the calibrated weekday and weekend diurnal patterns for the area tributary to Site 2. Similar diurnal curves were developed for each of the meters and its tributary area. These additional curves are available in Appendix B.2.

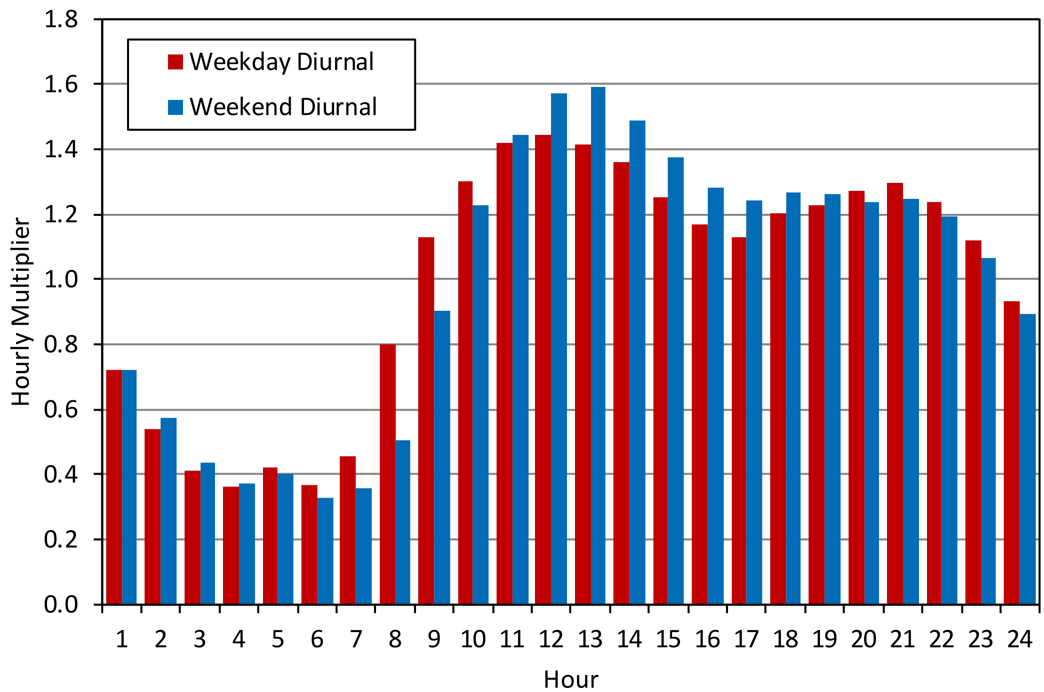

Figure 4.7 Meter Diurnal Pattern

- **Adjust model variables to match field-measured velocity and flow depths.** After the model-simulated flows satisfactorily matched the field-measured flows, the model-simulated velocity and flow depth were compared to the field-measured velocity and flow depth. Adjustments were then made to various model parameters until the modeled and measured velocity and depth closely matched each other. For this process, the primary varied parameters were pipeline roughness (Manning's n) and sediment buildup in the pipe, although other parameters can also be adjusted as calibration results are generated.

  Manning's roughness coefficients, or n values, have industry-accepted ranges based on a number of variables. Roughness coefficients increase over time depending on the construction methods, installation quality, system maintenance, and other environmental factors. Additionally, certain factors within the City's collection system can result in roughness coefficients that differ from the typical range. For example, pipeline bellies, joint misalignment, cracks, and debris (e.g., root intrusion) lead to increased turbulence in a pipe, which in turn increases the apparent Manning's n factor.

  If the model is unable to reasonably match the field-measured flow depth and velocity without leaving the acceptable range of Manning's roughness coefficients, further investigation is conducted to determine the cause of the discrepancy. Causes of the discrepancy can include errors in a pipeline's slope or diameter, downstream blockages, pipeline sags, and, in some cases, influences from downstream lift station operations.

Table 4.2 provides a summary of the dry weather flow calibration using the average and daily peak flow results for both weekday and weekend conditions. As shown in Table 4.2, the model simulated average and peak flows for both weekday and weekend DWF within 10 percent.

Appendix B.2 contains a detailed dry weather flow calibration summary sheet for each of the nine metering sites. Each calibration sheet provides plots that compare the model simulated and field measured flow, velocity, and level data for both weekday and weekend conditions. Figure 4.8 shows an example of the dry weather calibration.

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Table 4.2 Dry Weather Flow Calibration Summary

		Weekday									Weekend									Overall ADWF		
		Measured Data[1]			Modeled Data[1]			Percent Error[2]			Measured Data[1]			Modeled Data[1]			Percent Error[2]					
Meter Number	Pipe Diameter (in)	Avg. Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (%)	Avg. Velocity (%)	Avg. Level (%)	Avg. Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (%)	Avg. Velocity (%)	Avg. Level (%)	Measured (mgd)	Modeled (mgd)	Percent Error (%)
Site 1	24	0.64	3.88	3.2	0.669	4.13	3.5	4.0%	6.5%	7.7%	0.71	3.96	3.3	0.69	4.15	3.5	-3.3%	4.6%	5.3%	0.66	0.67	1.8%
Site 2	30	1.35	1.81	8.5	1.398	1.87	8.3	3.9%	3.3%	-2.5%	1.35	1.76	8.7	1.42	1.87	8.3	5.0%	6.3%	-4.2%	1.35	1.40	4.2%
Site 3	15	0.49	5.94	2.5	0.491	5.83	2.4	-0.3%	-1.8%	-6.2%	0.50	5.82	2.5	0.50	5.83	2.4	-0.3%	0.2%	-3.5%	0.49	0.49	-0.3%
Site 4	15	0.34	4.09	2.4	0.313	4.14	2.3	-8.2%	1.1%	-5.9%	0.35	4.06	2.4	0.31	4.12	2.3	-9.8%	1.3%	-7.1%	0.34	0.31	-8.7%
Site 5	12	0.06	1.40	1.6	0.062	1.53	1.5	3.0%	9.5%	-5.3%	0.07	1.47	1.7	0.07	1.57	1.6	3.0%	6.5%	-7.1%	0.06	0.06	3.0%
Site 6	21	0.84	2.10	6.3	0.866	2.25	6.0	2.6%	7.6%	-5.2%	0.88	2.11	6.4	0.88	2.26	6.0	0.7%	7.4%	-5.8%	0.85	0.87	2.0%
Site 7	15	0.28	4.85	2.0	0.307	4.86	1.9	9.9%	0.3%	-1.3%	0.28	4.80	1.9	0.31	4.84	1.9	9.8%	0.8%	0.9%	0.28	0.31	9.9%
Site 8	12	0.50	8.26	2.1	0.484	7.82	2.0	-2.9%	-5.3%	-1.5%	0.51	8.27	2.1	0.51	7.90	2.1	-1.5%	-4.5%	-2.1%	0.50	0.49	-2.5%
Site 9	12	0.19	3.25	2.1	0.197	3.23	2.1	1.3%	-0.8%	-2.4%	0.18	3.21	2.1	0.19	3.18	2.0	1.2%	-0.8%	-3.0%	0.19	0.19	1.2%

Notes:
Source: City of Banning 2017 Temporary Flow Monitoring Program, V&A Consulting Engineers. Average flows are calculated from flow monitoring data. Maximum flow values are hourly peaks.
Percent Difference = (Modeled - Measured)/Measured*100.

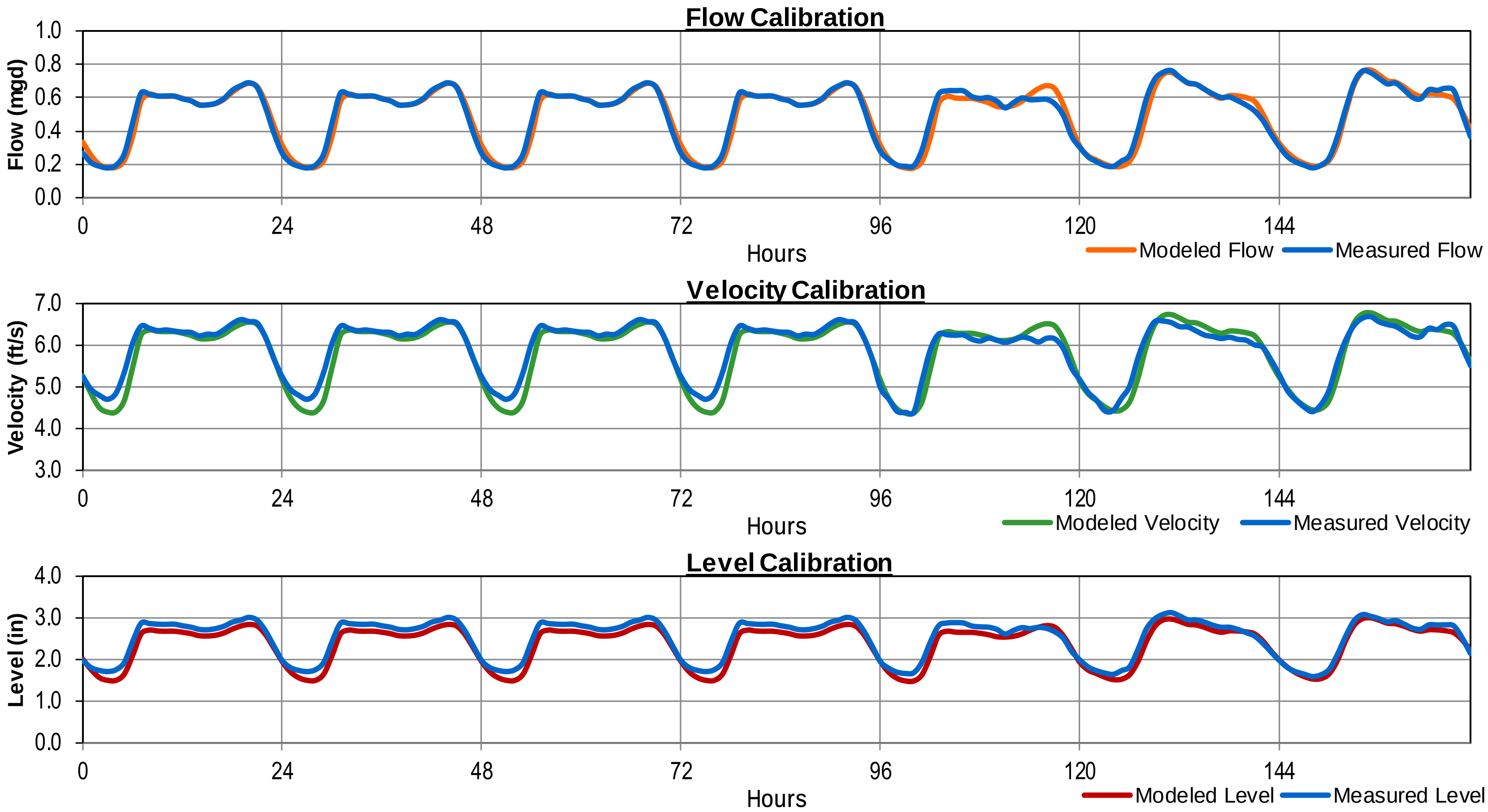

Figure 4.8 Example of Dry Weather Calibration (Site 3)

4.2.7 Wet Weather Calibration

The WWF calibration enables the hydraulic model to accurately simulate I/I entering the collection system during a large storm. As outlined below, the WWF calibration process consists of several elements:

- **Identify calibration rainfall events.** The WWF calibration process consists of running model simulations of historical rainfall. The goal of any WWF calibration is to capture and characterize a system's response to a significant rainfall event, preferably during wet antecedent moisture conditions.
  - The selection of a particular calibration storm or group of storms is based on a review of flow and rainfall data. In this case, the model was run from January 19, 2017 to January 26, 2017, and was calibrated to the main rainfall event that occurred during the flow monitoring period.
  - In order to run a model simulation for the January 2017 rainfall event, the hourly rainfall data was input into the model.
- **Define RDII tributary areas.** For the WWF calibration, RDII flows are superimposed on top of the DWF. The model calculates RDII by assigning "RDII Inflows" to each node in the model. RDII inflows consist of both a unit hydrograph and the total area that is tributary to the model node. The RDII tributary areas were calculated in GIS using the loading polygons. The tributary area provides a means to transform hourly rainfall depth from the rainfall hyetographs into a rainfall volume. The rainfall volume is transformed into actual RDII flows using the unit hydrograph, as described in the next step.

  Create I/I parameter database and modify to match field measured flows. The main step in the WWF calibration process involved creating a custom unit hydrograph for the City service area using the "RTK Method," which is widely used in collection system master planning. Using the RTK Method, the RDII unit hydrograph is the summation of three separate triangular hydrographs (short term, medium term, and long term), which are each defined by three parameters: R, T, and K. R represents the fraction of rainfall over the sewershed that enters the collection system; T represents the time to peak of the hydrograph; and K represents the ratio of time to recession to the time to peak. Therefore, there are a total of nine separate variables associated with a unit hydrograph.

  Figure 4.9 shows the shape of an example unit hydrograph. The hydrograph utilizes the R-values (percent of rainfall that enters the collection system) calculated for each basin to simulate I/I. The nine variables in each unit hydrograph were initially set based on engineering judgment and then adjusted until the model-simulated flows (both peak flows and average flows) matched closely with the field-measured flows.

  As with the dry weather calibration, the wet weather calibration process compared the measured flow data with the model output. Comparisons were made for average and peak flows as well as the temporal distribution of flow until flows returned to their baseline levels. According to the WaPUG criteria, a hydraulic model is generally considered to be satisfactorily calibrated to WWF conditions if the modeled peak flows are within +25 percent to - 15 percent of the field measured data, and if the average modeled flows are within +20 percent to -10 percent of the field measured data.

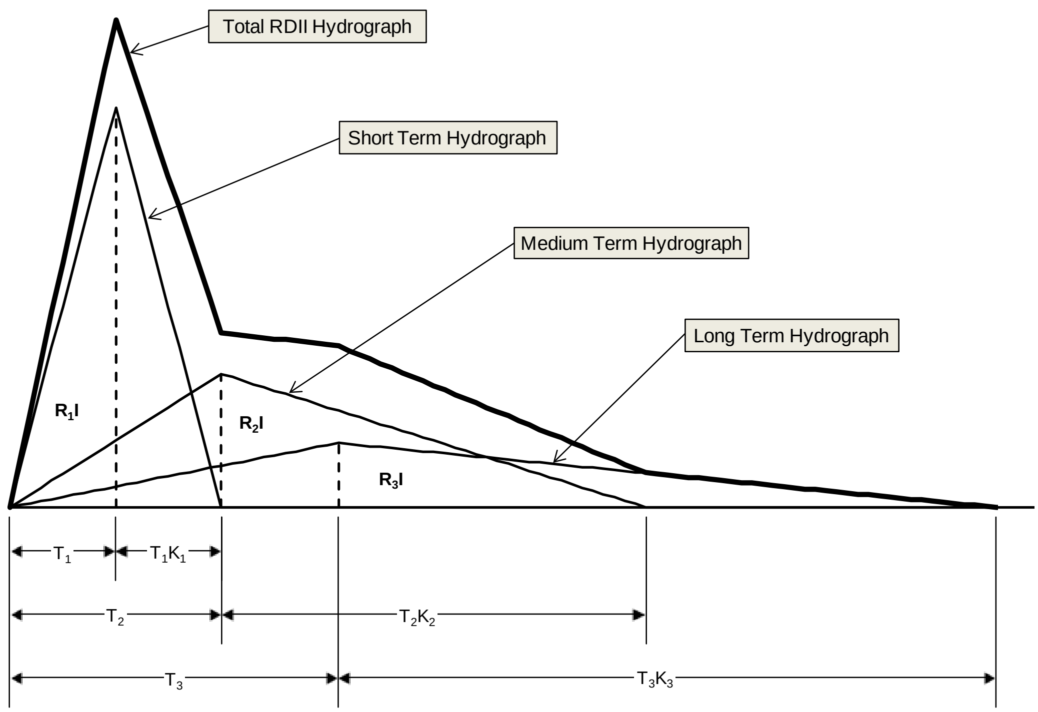

Figure 4.9 Example RDII Unit Hydrograph

Refine model variables to match field-measured velocity and flow depths. After the model was deemed satisfactorily calibrated for wet weather flows, its simulated velocities and flow depths were checked against the field-measured velocities and flow depths during the calibration storms. Refinements were also made to the various model parameters so the modeled and measured velocity and depth closely matched each other. If any adjustments were made to Manning's n values or other parameters, the DWF calibration was rechecked to verify that the flow depth and velocities still matched properly under DWF conditions. Appendix B.2 contains a detailed wet weather flow calibration summary sheet for each of the nine meter sites. Each calibration sheet provides plots that compare the model-simulated and field-measured flow, velocity, and level data for the calibration storms. An example of the wet weather calibration for Site 3 is shown on Figure 4.10.

A summary of the wet weather flow calibration is shown in Table 4.3 and displays the average and peak flow results. Table 4.3 shows excellent overall correlation between the field-measured data and the model output results. However, in some sites, the modeled flows, levels, or velocities were outside the generally accepted calibration tolerances. These sites were further investigated and deemed acceptable. The model was then considered calibrated and ready to use for capacity analysis.

- **Site 1:** The average wet weather velocity for storm 1 and average flow for storm 2 are above the calibration tolerance. The inability to match velocities is associated with the operation at the headworks and the effect of a large HGL increase at the entrance of the WWTP. Upon inspection of the calibration sheet, it is evident that Site 1 has experienced a significant amount of infiltration. The modeled flow data is unable to simulate the extensive infiltration, which skews the average data.
- **Site 2:** The increase in the water depth is created by the operation of the headworks in the WWTP. Therefore, the model was unable to simulate the water depth within the calibration tolerance.
- **Site 5:** Upon inspection of the wet weather calibration sheet for Site 5, the flow pattern has a sudden shift.

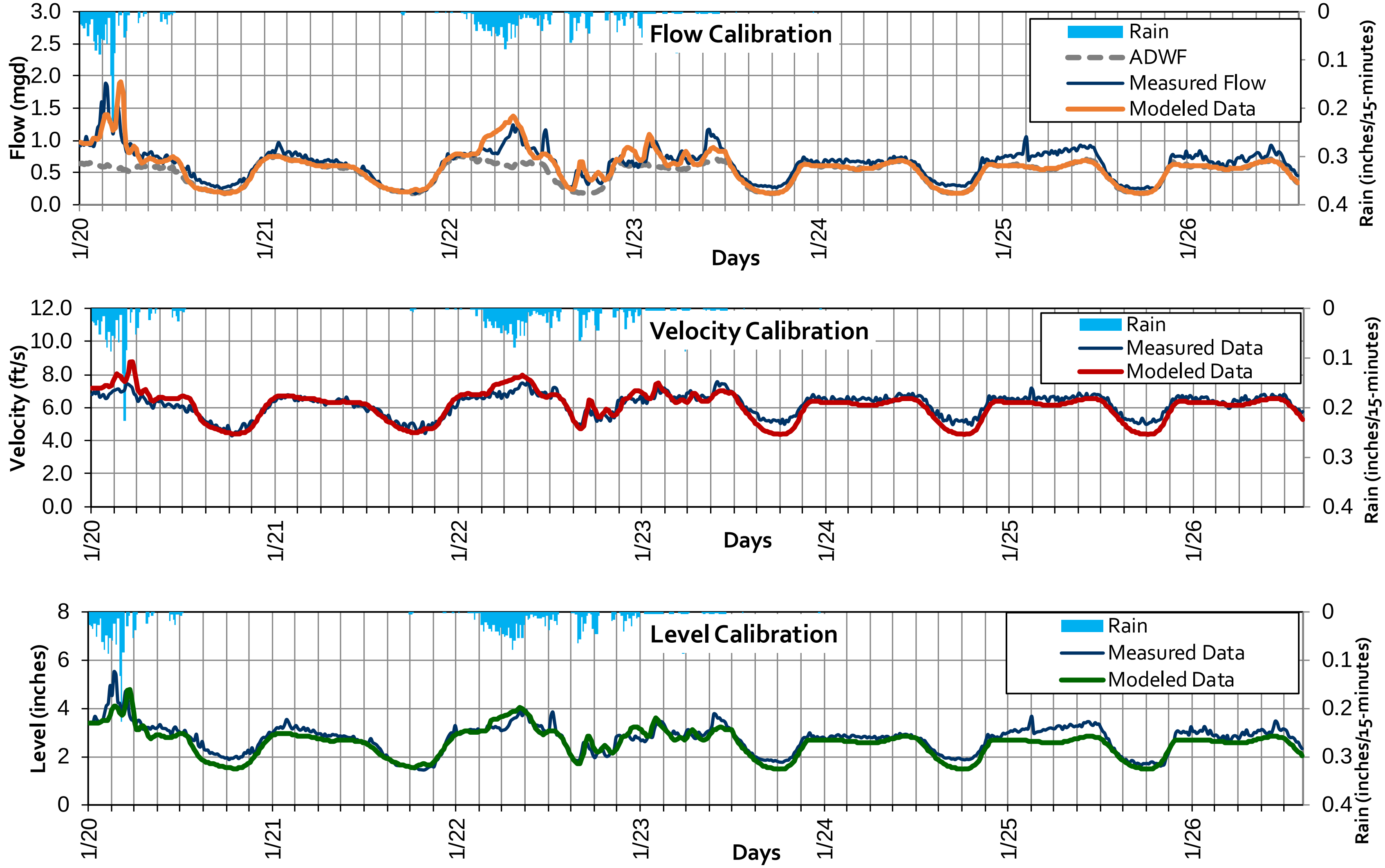

Figure 4.10 Example of Wet Weather Calibration (Site 3)

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Table 4.3 Wet Weather Flow Calibration Summary

		Storm 1 (1/20/2017-1/20/2017)												Storm 2 (1/22/2017-1/23/2017)											
		Measured Data(1)				Modeled Data(1)				Percent Error(2)				Measured Data(1)				Modeled Data(1)				Percent Error(2)			
Meter Number	Pipe Diameter (in)	Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (%)	Peak Flow (%)	Avg. Velocity (%)	Avg. Level (%)	Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (mgd)	Peak Flow (mgd)	Avg. Velocity (ft/s)	Avg. Level (in)	Avg. Flow (%)	Peak Flow (%)	Avg. Velocity (%)	Avg. Level (%)
Site 1	24	1.822	4.039	3.95	7.5	1.669	3.865	5.34	7.4	-8.4%	-4.3%	35.0%	-0.8%	1.435	3.258	4.36	5.2	1.133	2.803	4.74	6.1	-21.1%	-14.0%	8.7%	16.9%
Site 2	30	2.336	3.717	1.93	12.6	2.105	4.047	2.12	10.0	-9.9%	8.9%	9.6%	-20.9%	1.840	3.196	1.92	10.0	1.884	3.562	2.06	9.5	2.4%	11.4%	7.1%	-4.3%
Site 3	15	0.915	1.885	6.44	3.5	0.899	1.911	6.95	3.2	-1.7%	1.4%	7.9%	-7.2%	0.650	1.253	6.26	2.7	0.663	1.378	6.31	2.7	2.0%	10.0%	0.8%	0.3%
Site 4	15	0.629	1.142	5.24	3.1	0.575	1.311	4.95	3.0	-8.6%	14.9%	-5.5%	-3.8%	0.462	0.838	4.86	2.6	0.422	0.863	4.49	2.5	-8.8%	2.9%	-7.7%	-1.6%
Site 5	12	0.100	0.290	1.85	1.8	0.118	0.280	1.84	2.1	18.2%	-3.8%	-0.2%	17.5%	0.072	0.215	1.63	1.6	0.090	0.188	1.68	1.8	23.7%	-12.4%	3.1%	15.4%
Site 6	21	1.372	2.259	2.52	7.8	1.590	2.389	2.70	8.3	15.9%	5.7%	7.1%	6.4%	1.112	1.928	2.32	7.0	1.143	2.192	2.43	6.9	2.8%	13.7%	5.0%	-2.1%
Site 7	15	0.418	1.152	5.40	2.3	0.448	1.241	5.37	2.3	7.2%	7.8%	-0.6%	2.1%	0.375	0.817	5.23	2.1	0.403	0.796	5.24	2.2	7.4%	-2.6%	0.2%	4.1%
Site 8	12	0.686	0.924	9.16	2.4	0.810	1.098	9.27	2.3	18.0%	18.8%	1.2%	-7.6%	0.675	1.242	8.84	2.4	0.610	1.119	8.35	2.3	-9.7%	-10.0%	-5.6%	-4.8%
Site 9	12	0.341	0.722	3.90	2.6	0.382	0.858	3.88	2.8	12.0%	18.7%	-0.5%	7.4%	0.305	0.617	3.72	2.6	0.281	0.572	3.57	2.5	-7.6%	-7.3%	-4.0%	-4.1%

Notes:
(1) Source: City of Banning 2017 Temporary Flow Monitoring Program, V&A Consulting Engineers. Average flows are calculated from flow monitoring data. Peak flow values are hourly peaks. Averages were adjusted to account for data not recorded.
(2) Percent Difference = (Modeled - Measured)/Measured*100.

4.3 Recycled Water System Hydraulic Model

This section summarizes the hydraulic model development process, including a summary of the modeling software selection, a description of the modeled distribution system, the hydraulic model elements, the model creation process, and the model calibration process.

4.3.1 Recycled Water Hydraulic Modeling Software

Similar to the City's potable water hydraulic model, the City's recycled water hydraulic model was developed in H_20Map® Water in 2006 by Carollo. Since the City did not have an existing system at the time, the model was developed based on the different scenarios and alternatives evaluated at that time. Initially, the original model was converted to InfoWater®, which is the same software used for the potable water system hydraulic model. Similar to the potable water hydraulic model, the current hydraulic model uses InfoWater® 12.3 Update #6.

4.3.2 Data Collection and Validation

The primary sources for the development of the hydraulic model were the as-built drawings for existing pipelines and drawings for planned pipeline projects for the backbone system. Street centerlines were obtained from public data sources and were used for reference during model development.

4.3.3 Elements of the Hydraulic Model

The major elements of the recycled water hydraulic model are similar to the potable water hydraulic model (see Section 4.1.3). The City's recycled water model consists of the following components:

- 22 junctions.
- 26 pipelines (8.4 miles ranging from 6-inch to 24-inch diameter).
- 2 pumps.

4.3.4 Hydraulic Model Development and Updates

To develop and update the City's recycled water hydraulic model, the following steps were performed:

- **Step 1:** Convert City's existing recycled water model to InfoWater®.
- **Step 2:** Delete customers no longer being considered in analysis.
- **Step 3:** Update demands for potential customers.
- **Step 4:** Update pipeline alignments to reflect as-built drawings and planned pipelines.
- **Step 5:** Add and size pipes connecting to customers based on updated demands.
- **Step 6:** Determine HGL required to serve customers and set WWTP pump operating point to deliver the required HGL to meet minimum pressure requirements.
- **Step 7:** Create diurnal pattern with appropriate peaking factors for 24-hour irrigation and 8-hour irrigation.
- **Step 8:** Assign diurnal patterns to customers based on anticipated irrigation schedule.
- **Step 9:** Create ADD and MDD scenarios for each phase (near-term and long-term).

4.3.5 Diurnal Patterns

Two irrigation patterns were created in the model based on the number of hours of irrigation. As mentioned in Chapter 3, the 24-hour irrigation pattern assumes a peaking factor of 1, while the 8-hour irrigation pattern assumes a peaking factor of 3. The diurnal patterns created for this model are shown in Figure 4.12 and Figure 4.13.

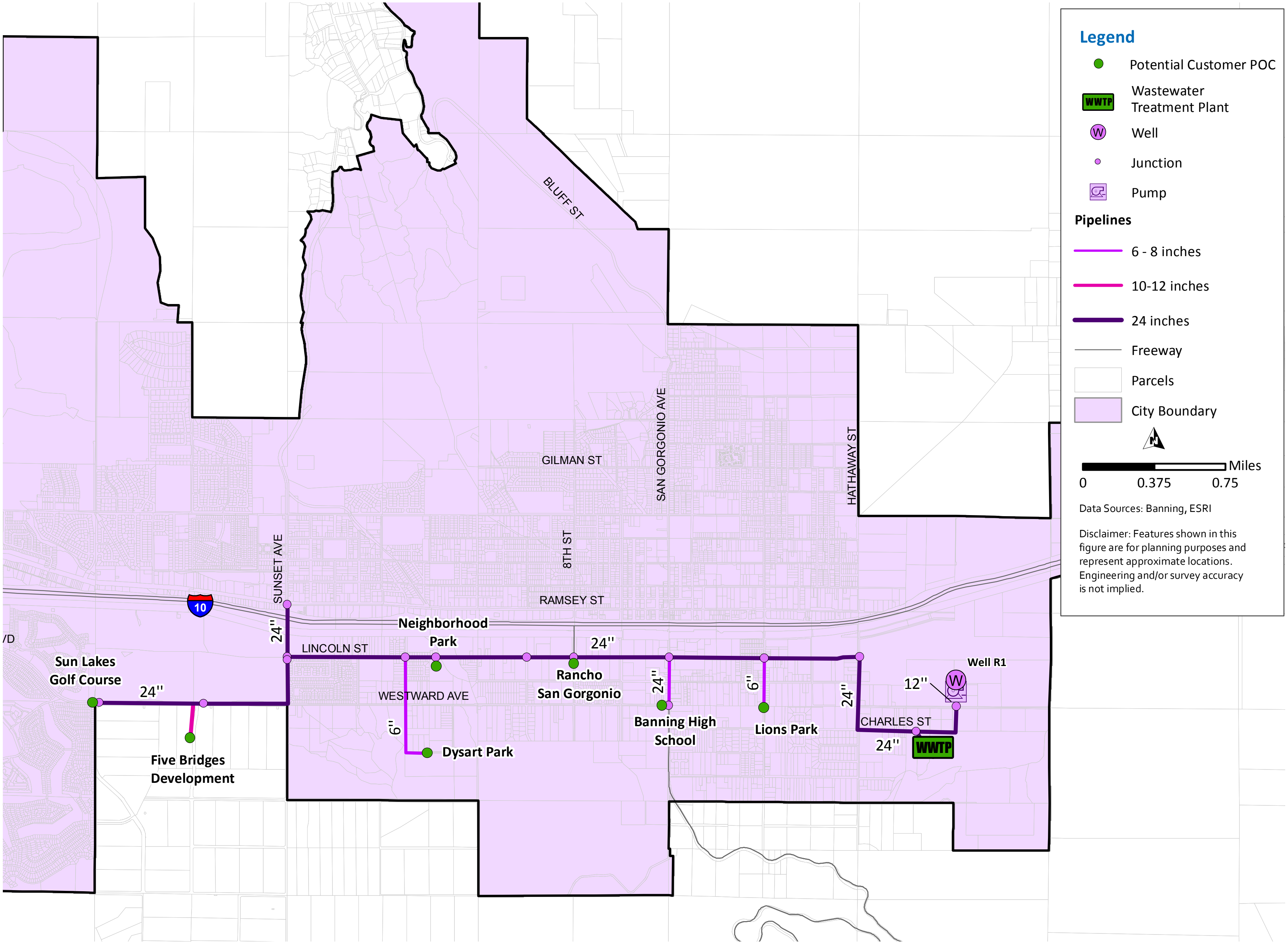

Figure 4.11 Recycled Water Distribution System Model

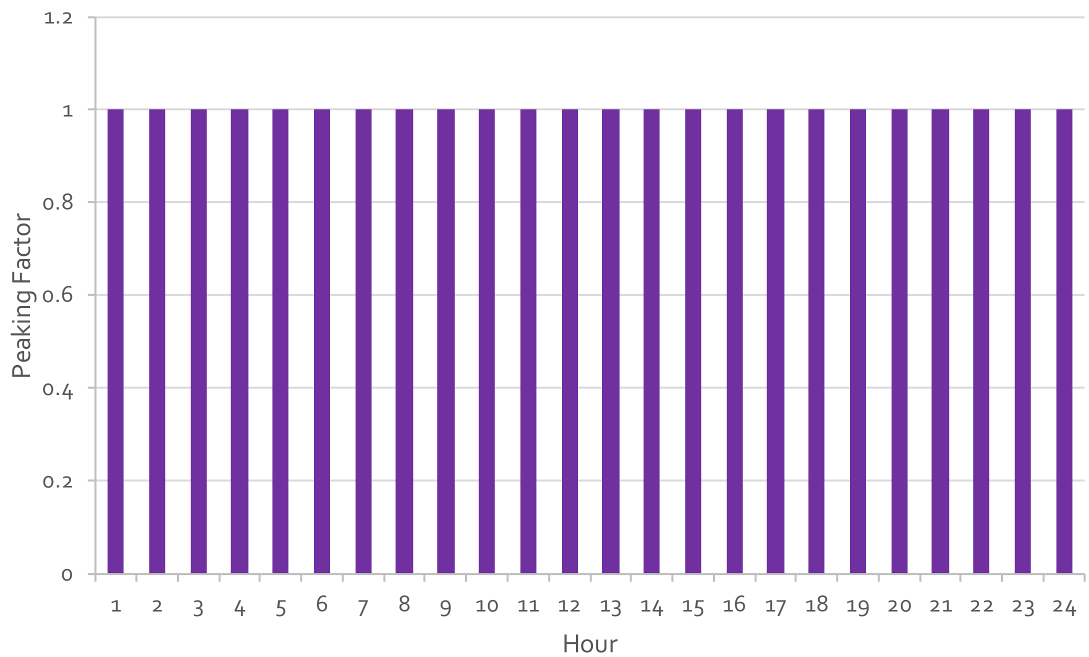

Figure 4.12 Recycled Water 24-Hour Diurnal Pattern

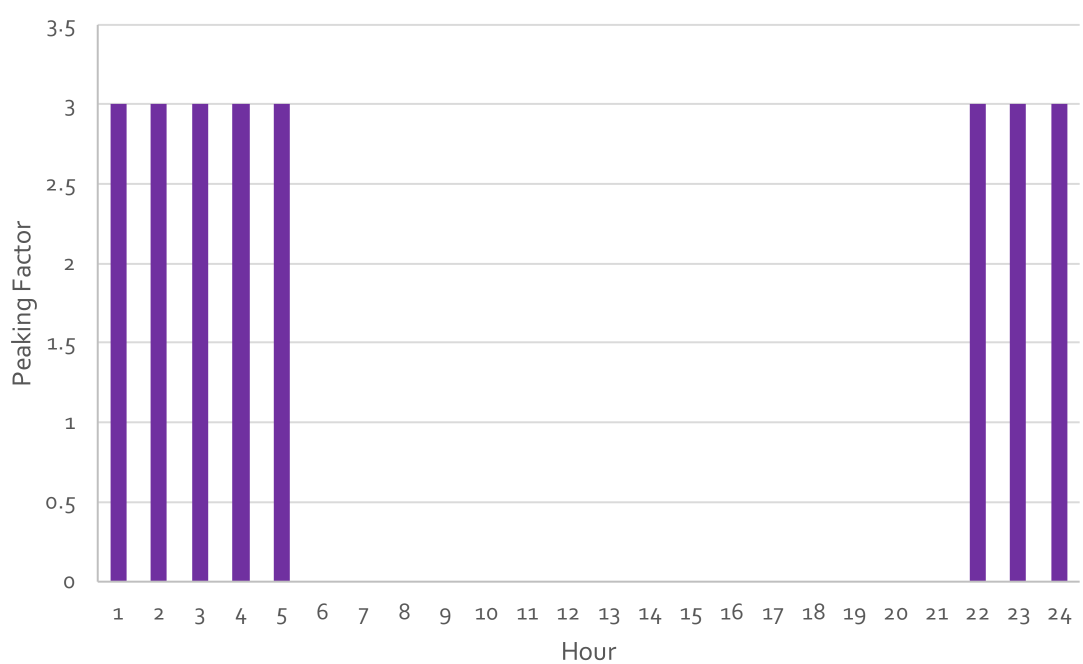

Figure 4.13 Recycled Water 8-Hour Diurnal Pattern

Chapter 5

EVALUATION CRITERIA

This chapter presents the planning criteria and methodologies for the analysis used to evaluate the existing potable water system, wastewater system, and recycled water systems and the associated facilities to identify existing system deficiencies and size future improvements and expansions. The planning criteria are used in the existing and future system analyses in Chapters 6, 7, and 8 and to define capital improvement projects in Chapter 9.

5.1 Potable Water System Evaluation Criteria

The City's water system is evaluated under a range of normal and emergency operating conditions and demand scenarios. The normal operating conditions are:

- Average Day Demand (ADD)
- Peak Hour Demand (PHD)
- Maximum Day Demand (MDD)
- MDD plus Fire Flow (MDD+FF)

Distribution system evaluation criteria are required to determine the performance of the City's water system under the range of operating conditions as discussed above and to identify system deficiencies and improvement projects. Under each operating condition, the capacities and performance of the water system are compared to the evaluation criteria to determine which pipelines or water facilities need to be upgraded or replaced. The evaluation criteria for the potable water system consist of the following categories:

- System Pressure
- Pipeline Velocity
- Storage Volume
- Pump Station (PS) Capacity
- Pressure Reducing Valve (PRV) Capacity

The evaluation criteria used for the evaluation of the City's potable water system are summarized in Table 5.1. Detailed descriptions for each evaluation criteria are provided following the table.

Table 5.1 Potable Water System Evaluation Criteria

Description	Value(1)	Units
Maximum Pressure		
Without Individual Pressure Regulator at Meter	80	psi
With Individual Pressure Regulator at Meter	115	psi
Minimum Pressure		
Peak Hour Demand (PHD)	40	psi
Maximum Day Demand (MDD) + Fire Flow	20	psi
Pipeline Criteria		
Maximum Velocity with ADD	5	fps
Maximum Velocity with PHD	8	fps
Maximum Velocity with MDD + Fire Flow	10	fps
Hazen-Williams C-Factor		
Pipelines Greater Than 50 Years in Age	110	N/A
Pipelines Between 20 to 50 Years in Age	120	N/A
Pipelines Less Than 20 Years in Age	130	N/A
Minimum Size for Pipeline Replacement	8	inches
Fire Flow Requirements(2)		
Low Density Residential	1,500	gpm for 2 hrs.
Medium Density Residential	2,000	gpm for 2 hrs.
High Density Residential	2,500	gpm for 3 hrs.
Commercial	3,500	gpm for 4 hrs.
Industrial	4,000	gpm for 4 hrs.
Public	4,000	gpm for 4 hrs.
Open Space	1,000	gpm for 2 hrs.
Storage Volume		
Operational	25% of MDD	MG
Fire Fighting Storage	Max FF in Zone	MG
Emergency	100% MDD	MG
Pump Station Capacity		
Zones with Gravity Storage	Meet MDD with largest unit out of service	gpm
Zones Without Gravity Storage	Meet MDD + FF with largest unit out of service	gpm
Pressure Reducing Valve Capacity		
Zones without Gravity Storage	Meet MDD + FF with largest valve in the pressure zone out of service	gpm

Notes:
(1) Use for planning purposes only.
(2) Criterion was reviewed by the City of Banning Fire Marshall. Values may be reduced with the use of fire sprinklers.

5.1.1 Potable Water System Pressures

Minimum system pressures are evaluated under both PHD and MDD plus fire flows conditions. Maximum system pressures are evaluated under ADD. The minimum pressure criterion for PHD demand conditions is 40 pounds per square inch (psi), while the minimum pressure criterion under MDD with fire flow conditions is 20 psi. The pressure analysis is limited to demand nodes, because only locations with service conditions need to meet such pressure requirements. Lower pressures are only acceptable for junctions at water system facilities and on transmission mains. However, no pressure shall be less than 5 psi to avoid potential water quality issues.

Maximum system pressures are evaluated under the ADD conditions. The maximum pressure criterion for normal ADD conditions is 80 psi for service connections without individual pressure-reducing valves. In areas where the maximum pressure exceeds 80 psi, individual pressure-reducing valves are required on service connections. However, the system pressure shall generally not exceed 115 psi.

5.1.2 Potable Water Pipeline Velocities

Pipeline velocities are evaluated using three different maximum velocity criteria for selected flow conditions under both existing and future demand scenarios. For transmission and distribution pipelines, a maximum velocity of 5 feet per second (fps) and 8 fps was used for ADD and PHD conditions, respectively. Fire hydrant laterals are excluded from these criteria, as higher velocities are acceptable. Under fire conditions, velocities of up to 10 fps were allowed. Ideally, all transmission and distribution pipelines should have maximum velocities less than 8 fps in order to minimize head loss. However, higher velocities in existing pipelines are not, by them self, sufficient justification for pipeline replacement.

5.1.3 Potable Water Storage Capacity

The total storage required for a water system is evaluated in three components.

- Storage for operational use.
- Storage for firefighting.
- Storage for emergencies

These three components are determined for each pressure zone to evaluate the ability of the water system to meet the storage criteria on both a zone-by-zone basis, as well as a system-wide basis. These three storage requirements are discussed in more detail below.

- Operational Storage. Operational storage is defined as the quantity of water that is supplied to meet daily fluctuations in demand beyond the quantity of water that is produced on a daily basis. It is necessary to coordinate the production rates of water sources and the available storage capacity in a water system to provide a continuous flow of treated water supply to the system. Water systems are often designed to supply the average flow on the day of maximum demand. Water storage is then used to supply water for peak hour flows that may occur throughout the day. This operational storage is continuously replenished throughout the day to maintain water quality.

  The American Water Works Association (AWWA) recommends an operational supply volume ranging from one-quarter to one-third of the demand experienced during one maximum day. It is recommended that pressure zones in the City's water system have operational storage of 25 percent of the MDD supplied by that reservoir.

- Fire Flow Storage. The governing fire department provides the City with the fire flow rate and duration to determine if fire storage is required for a pressure zone. The values provided in Table 5.1 are provided as a reference and are based on typical values for water utilities. Fire flow storage is determined based on the single greatest fire flow requirement (flow and duration) within each pressure zone.
- Emergency Storage. Storage is also required to meet system demands during emergencies. Emergencies cover a wide range of rare but probable events, such as water contamination, failure at a water treatment plant, power outages, transmission pipeline ruptures, several simultaneous fires, and earthquakes. The volume of water that is needed during an emergency is usually based on the estimated amount of time expected to elapse before the disruptions caused by the emergency are corrected. The occurrence and magnitude of emergencies is difficult to predict. The City's recommended emergency storage is set to 100 percent of the MDD.

5.1.4 Potable Water Pump Station Capacity

Typically, a pump station consists of multiple pump units, including one spare pump to provide reliability in case of a breakdown or repair. In addition, critical booster pumps may be equipped with emergency power supplies in case of failure of the primary power source.

For the purpose of this IMP, the capacity and design criteria were modified to reflect system conditions typically evaluated as part of a master plan. These criteria are the sizing of pump stations under normal demand conditions using MDD and MDD plus maximum fire flow for zones with and without gravity storage, respectively. Each station shall have sufficient capacity to meet the required MDD and the maximum zone fire flow with the largest unit out of service, or based on the available backup power.

5.1.5 Potable Water Pressure Reducing Valve Capacity

Typically, a pressure reducing valve station includes multiple valves of varying sizes. For pressure zones without gravity storage, supply sources, or pump stations, the PRV stations serve as the primary source of supply for that pressure zone. The criteria used in this situation requires that all PRVs suppling the pressure zone must meet the required MDD and maximum zone fire flow with the largest valve out of service.

5.2 Wastewater System Evaluation Criteria

The capacity of the City's sanitary sewer collection system will be evaluated based on the planning criteria defined in this section. The planning criteria address the collection-system capacity, gravity sewer pipe slopes, and maximum allowable depth of flow within a sewer.

The evaluation criteria used for the evaluation of the City's sewer system are summarized in Table 5.2. Detailed descriptions for each evaluation criteria are provided following the table.

Table 5.2 Wastewater System Evaluation Criteria

Minimum Slopes for New Circular Pipes		
	Pipe Size (in)	**Minimum Slope (ft/ft)**
	8	0.004
	10	0.003
	12	0.0024
	15	0.0017
	18	0.0014
	21	0.0011
	24	0.0010
Note: Minimum Slope values are based on pipeline flowing half full at 2 ft/s. Values are from 2006 Master Plan.		
Flow Depth, d/D		
Maximum Flow Depth for Existing Sewers		
Pipe Diameter		**Maximum d/D Ratio (PWWF)**
12" and Smaller		0.92
15" and Larger		0.92
Maximum Flow Depth for New Sewers		
Pipe Diameter		**Maximum d/D Ratio (PWWF)**
12" and Smaller		0.67
15" and Larger		0.75
Head Loss in Existing Pipelines		
Gravity Pipeline	Manning's n =	0.013
Pressure Pipelines	Hazen William's C =	120
Lift Stations and Force Mains		
Minimum Velocity		3 ft/s
Maximum Velocity		8 ft/s
Lift Station Capacity		Firm Capacity under Peak flows
Note: firm capacity represents the lift stations capacity with the largest pump out of service.		

5.2.1 Manning's n Coefficient

The Manning's n coefficient is a friction coefficient that varies with respect to pipe material, size of pipe, depth of flow, smoothness of joints, root intrusion, and other factors. For gravity pipelines, the Manning's n coefficient value is typically 0.013. The Manning's n factor was refined as necessary during model calibration to accurately simulate field-measured levels and velocities.

5.2.2 Flow-Depth Criteria

The primary criterion used to identify capacity-deficient sewers or to size new sewer improvements is the maximum flow depth-to-pipe diameter ratio (d/D). The d/D value is defined as the depth of flow (d) in a pipe during peak (design) flow conditions divided by the pipe's diameter (D). Based on Carollo's experience and industry standards, the following criteria were recommended.

- Flow Depth for Existing Sewers. Maximum flow-depth criteria for existing sanitary sewers are established based on a number of factors, including the acceptable risk tolerance of the utility, local standards and codes, and other factors. Using a conservative d/D ratio when evaluating existing sewers may lead to unnecessary replacement of existing pipelines. Conversely, lenient flow-depth criteria could increase the risk of sanitary sewer overflows (SSOs). Ultimately, the maximum allowable flow-depth criteria should be established to be as cost-effective as possible, while at the same time reducing the risk of SSOs to the greatest extent possible.

  The maximum flow depth for an existing sewer 12-inches in diameter or smaller was 0.92. The maximum flow depth for an existing sewer 15-inches in diameter or larger was 0.92. The following criteria was based on the on the 2006 Master Plan.

  A capacity-deficient sewer (i.e., system bottleneck) raises the hydraulic grade line of upstream sewers, leading to backwater conditions. The greater the capacity deficiency, the higher the water levels will surcharge upstream of the bottleneck pipeline (or pipelines). The hydraulic model is used to determine "backwater" pipelines in order to specify which specific pipelines are the actual root causes of the capacity deficiency. Capital projects are proposed to provide greater flow capacity for the deficient sewers, which eliminates the backwater conditions that cause surcharging.

- Flow Depth for New Sewers. When sizing new sewer pipelines, it is common practice to adopt variable flow depth criteria for various pipe sizes. Design d/D ratios typically range from 0.5 to 0.92, with the lower values typically used for smaller pipes, which may experience flow peaks greater than design flow or blockages from debris, paper, or rags. For pipelines 12-inches in diameter and smaller, the maximum d/D value is 0.67 or 67 percent of the pipeline depth. For Pipelines 15-inches and larger, the maximum d/D is 0.75.

5.2.3 Design Velocities and Minimum Slope

In order to minimize the settlement of sewage solids, it is standard practice in the design of gravity sewers to specify that a minimum velocity of 2 feet per second (ft/s) be maintained when the pipeline is half-full. At this velocity, the sewer flow will typically provide self-cleaning for the pipe. Due to hydraulics of a circular conduit, velocity of half-full flow in pipes approaches the velocity of nearly full flow in pipes.

Table 5.2 lists the recommended minimum slopes and their corresponding maximum flows for maintaining self-cleaning velocities (equal to or greater than 2 ft/s) when the pipe is flowing at its maximum depth (d/D ratio).

5.2.4 Changes in Pipe Size

When a smaller sewer joins a large one, the invert of the larger sewer should be lowered sufficiently to maintain the same energy gradient. An approximate method for securing these results is to place the 0.8 depth point of both sewers at the same elevation. For planning purposes and designing new pipes, and in the absence of field data, sewer crowns were matched at the manholes.

5.2.5 Lift Stations and Force Mains

Industry standard practice is to require that sewage lift stations have sufficient capacity to pump the PWWF with the largest pump out of service (firm capacity).

Force main piping should be sized to provide a minimum velocity of 3 ft/s at the design flow rate of the lift station and no more than 8 ft/s. For the determination of head loss, the Hazen Williams Equation is used with a C-factor of 120. These factors are typical for sewer system master planning purposes.

5.3 Recycled Water System Evaluation Criteria

This section presents the evaluation criteria that was used to analyze the City's future recycled water system and size facilities. The criteria discussed includes system pressures, pipelines velocities, storage reservoirs volumes, and pump station capacities.

A list of recommended criteria used in the evaluation of the City's recycled water system is presented in Table 5.3.

5.3.1 Recycled Water Pipeline Criteria

System pressures and velocity are criteria that are used to size future recycled water pipelines. Since the City currently does not have a built out recycled water system, the criteria developed was focused on new infrastructure rather than the analysis of existing infrastructure. In addition, the Hazen William's C-factor used for pipelines equal to or less than 12-inches in diameter was 120 and the Hazen William's C-factor used for pipelines greater than 12-inches was 130. The minimum pipeline size used was 6-inches.

5.3.1.1 Recycled Water System Pressures

The recycled water system pressure is ideally designed to be slightly lower than the potable water system pressure. This pressure differential reduces the risk of potable water contamination from recycled water, in the event that an adjacent recycled water main breaks. There are circumstances where this requirement is not met since it is preferred to maintain a static pressure in the recycled water system of approximately 60 psi to meet operating requirements for most sprinkler systems. However, the minimum pressure in potable water systems is typically 40 psi.

The maximum pressure criteria used for the analysis of the future recycled water system was 115 psi and the minimum system pressure used for pipeline sizing in this IMP was 60 psi under static conditions.

Table 5.3 Recycled Water System Evaluation Criteria

Description	Value	Units
Pipeline Criteria		
Maximum Pressure	115	psi
Minimum Pressure Under Static Condition	60	psi
Maximum Velocity with MDD	8	fps
Hazen Williams C-factor for Pipelines 12-inches in Diameter or Less	120	n/a
Hazen Williams C-factor for Pipelines Greater than 12-inches in Diameter	130	n/a
Minimum Size for Pipelines	6	inches
Storage Volume		
Operational	Difference Between PHD and MDD	MG
Pump Station Capacity		
Normal Conditions	Meet PHD with largest unit out of service	gpm

5.3.1.2 Recycled Water Pipeline Velocities

The maximum velocity criteria used for sizing future pipelines was 8 fps under MDD conditions. Ideally, all transmission and distribution pipelines should have maximum velocities less than 8 fps in order to minimize head loss. However, higher velocities in existing pipelines are not, by themselves, sufficient justification for pipeline replacement.

5.3.2 Recycled Water Storage Capacity

The total storage required for a recycled water system is evaluated in operational storage. The operational storage is defined as the quantity of water that is required to meet daily fluctuations in demand beyond the quantity of water that is produced on a daily basis. It is necessary to coordinate the production rates of recycled water sources and the available storage capacity in a recycled water system to provide a sufficient buffer to meet the diurnal variations in demand for the system. Recycled water systems are often designed to produce the average flow on the day of maximum demand. Water storage is then used to supply water for peak hour flows that may occur throughout the day. This operational storage is replenished during off-peak hours when the demand is lower. Therefore, the criterion used for sizing future storage reservoirs in the IMP was the difference between the PHD and MDD total. However, the criteria can be supplemented with the use of wells or on-site storage, such as lakes.

5.3.3 Recycled Water Pump Station Capacity

Pump stations shall be sized to maintain a level of service during normal operating conditions. The pump stations shall be able to meet PHD conditions with the largest unit out of service.

Chapter 6

POTABLE WATER SYSTEM EVALUATION

This chapter presents an overview of the City's existing and future potable water distribution systems, water supplies, and storage facilities. In this chapter, the water systems are identified and evaluated. Then, based on the system evaluation results, improvement projects are identified to address the identified deficiencies. This chapter is divided into the following sections:

- Existing System Description: This section discusses the facilities that make up the existing potable water system.
- Existing System Analysis: This section presents the findings and improvement recommendations for the potable water system under existing demand conditions.
- Future System Analysis: This section presents the findings and improvement recommendations for the potable water system under future demand conditions with the existing system recommendations in place.
- Proposed Improvements: This section summarizes the improvement recommendations, which are prioritized and phased in the capital improvement program (CIP) described in Chapter 9 of this IMP.

6.1 Existing Potable Water System

The existing potable water system facilities include 6 pressure zones, 19 groundwater wells, 8 storage reservoirs, 2 booster pump station (PSs), 5 pressure reducing valves (PRVs), and approximately 165 miles of pipeline. Information regarding the existing potable water system is discussed in further detail in the sections below.

6.1.1 Water Supply Sources

The City potable water is primarily supplied from groundwater wells. The City overlies the Coachella Valley Groundwater Basin, which is underlain by several large sub-basins. The City overlies the San Gorgonio Pass (SGP) Sub-basin, which is divided into water storage units. The City extracts groundwater from the Banning Storage Unit, Banning Bench Storage Unit, Cabazon Storage Unit, Beaumont Basin, and Banning Canyon Storage Unit.

The City purchases imported water from the San Gorgonio Pass Water Agency to recharge to the Beaumont Basin at Beaumont Cherry Valley Water District's (BCVWD) Noble Creek spreading facility. Based on the City's 2015 UWMP, the City recharged approximately 694 afy in year 2015. Although the City purchases imported water, the imported water supply connection is only used for recharge.

In addition to the 21 groundwater wells within the City boundary, the City also jointly owns and operates 3 potable water well with BCVWD. A summary of the 24 groundwater wells and their capacities are listed in Table 6.1.

Table 6.1 Existing Groundwater Wells

Name	Groundwater Basin	Supply to Zone	Capacity (gpm)
Canyon Wells(1) Wells 1-3 Wells 4-5, 7-12	 Banning Bench Storage Unit Banning Canyon Storage Unit	Main and Foothill East	3,000
Well C2	Beaumont Basin	C2 Booster PS	1,100
Well C3	Beaumont Basin	Main	1,100
Well C4	Beaumont Basin	Foothill West	1,300
Well C5	Banning Storage Unit	Main	900
Well C6	Cabazon Storage Unit	Main	900
Well M3	Beaumont Basin	Foothill West	800
Well M7(2)	Beaumont Basin	Main	350(4)
Well M10	Banning Storage Unit	Main	800
Well M11	Banning Storage Unit	Main	600
Well M12(2)	Banning Storage Unit	Main	1,100
Well 24 (BCVWD)(3)	Beaumont Basin	Foothill West	1,000
Well 25 (BCVWD)(2,3)	Beaumont Basin	Foothill West	1,000
Well 26 (BCVWD)(2,3)	Beaumont Basin	Foothill West	1,000
Total Well Capacity	**N/A**	**N/A**	**14,950**
Total Potable Water Well Capacity	**N/A**	**N/A**	**11,500**

Notes:
(1) The capacity is lower during drought conditions. The minimum reliability capacity is approximately 1,700 gpm.
(2) Wells are currently used for pumping into the non-potable system, but may be converted to the potable water system in the future.
(3) City of Banning is allocated half of the nominal capacity of 6,000 gpm for the three wells co-owned by BCVWD.

As shown in Table 6.1, the City has 3 wells in the Banning Bench Storage Unit, 8 wells in the Banning Canyon Storage Unit, 8 wells in the Beaumont Basin, 3 wells in the Banning Storage Unit, and 1 well in the Cabazon Storage Unit. The City's total potable supply capacity from groundwater is approximately 14,950 gpm or 21.5 mgd. Since four of the wells (Wells M7, M12, 25, and 26) are currently used to serve non-potable water, the total potable water capacity from groundwater is 11,500 gpm or 16.6 mgd. This may change in the future when the State finalizes a new MCL for Chromium-6.

6.1.2 Water Distribution System

This section describes the existing distribution system facilities and provides an understanding of the existing system operations. The following sections provide a description of the system pressure zones and water system facilities that comprise the City's distribution system, including booster pump stations, reservoirs, PRVs, and pipes. A map of the City's distribution system is presented on Figure 6.1.

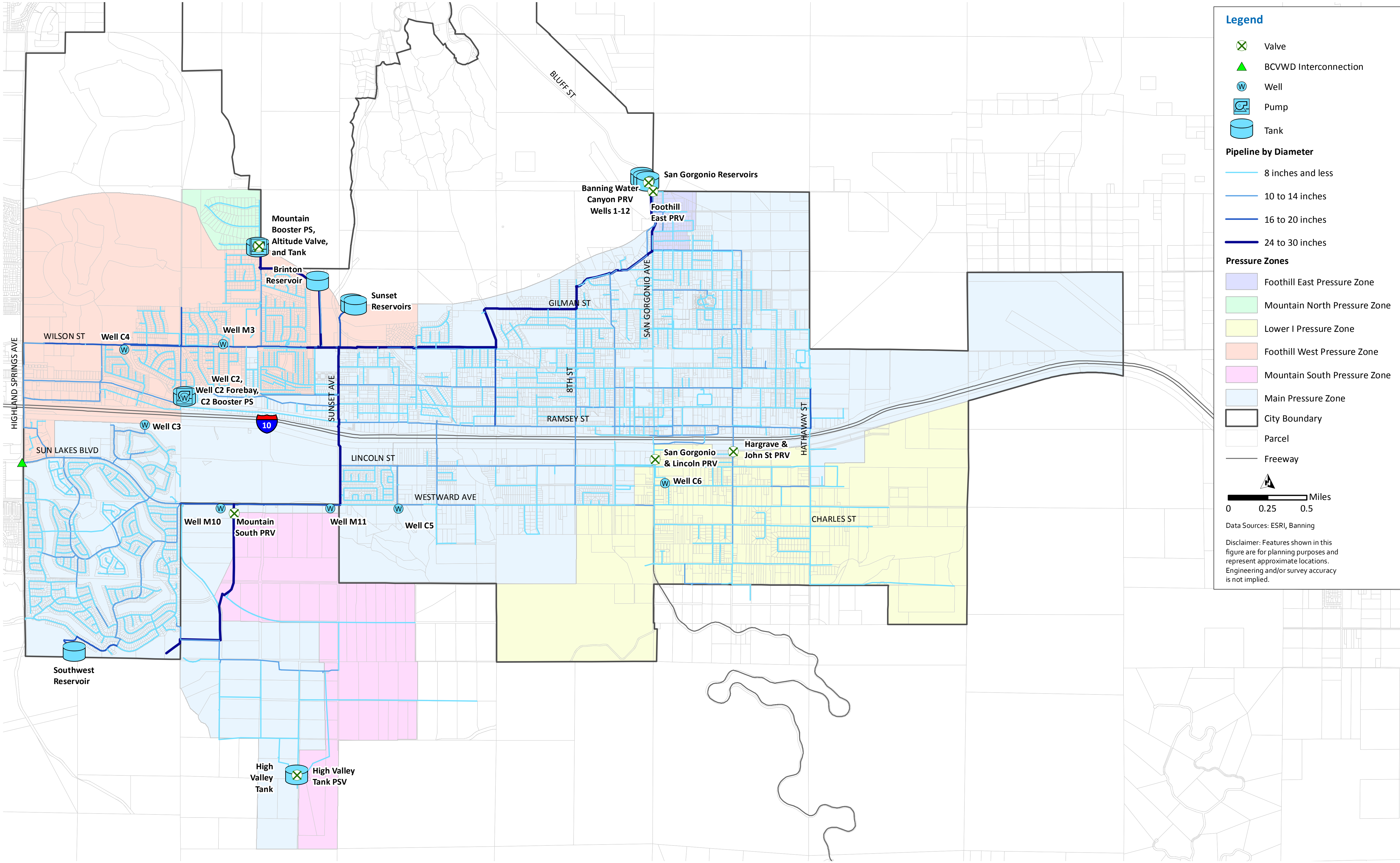

Figure 6.1 Existing Potable Water System as Modeled

6.1.2.1 Pressure Zones

Potable water systems are typically divided into different hydraulic regions, known as pressure zones, to maintain adequate pressures throughout the distribution system due to varying topography. A hydraulic grade line (HGL) is established for each pressure zone. The high water levels in reservoirs are set to maintain these HGLs. The City's service area ranges in ground elevation from approximately 2,106 feet above sea level (ft-msl) to about 2,796 ft-msl.

Although there are 3 water service connections from the Banning Water Canyon system, the City has not traditionally considered this as a part of the City's distribution system. The various HGLs along the Canyon are not given pressure zone names. This Integrated Master Plan (IMP) focuses on areas that the City considers as its distribution system. The Banning Water Canyon was represented in the hydraulic water model as a PRV discharging into the Main Zone and Foothill West Zone.

The City's distribution system is divided into 6 pressure zones. The HGLs, reservoirs, pump stations, and PRVs of each pressure zone are listed in Table 6.2. The existing water facilities and delineation of the pressure zones are shown on Figure 6.1.

Table 6.2 Existing Pressure Zones

Pressure Zone	HGL (ft msl)	Storage Reservoirs	Pump Stations (Discharge Zone)	PRV (Discharge Zone)
Foothill East	3,000	N/A	N/A	Foothill East PRV
Foothill West	2,822	Sunset Res. 1 & 2	C2 PS	N/A
Lower I	2,450	N/A	N/A	San Gorgonio & Lincoln PRV Hargrave & John St PRV
Main	2,721	Brinton Res. Southwest Res. San Gorgonio Res. 1, 2, 3	N/A	Well 1 PRV
Mountain North	2,932	N/A	Mountain PS	N/A
Mountain South	2,546	N/A	N/A	Mountain South PRV

As shown in Table 6.2, the City's existing storage is in the Foothill West Zone and Main Zone. The majority of the storage capacity is located in the Main Zone. The City's two booster pumping stations (PSs) pump into the Foothill West and Mountain North Zones. The Canyon Wells supply are conveyed through two PRVs; namely the Well 1 PRV and the Foothill East PRV. The Well 1 PRV conveys water from the Canyon Wells to the Main Zone, while the Foothill East PRV conveys water from the Canyon Wells to the Foothill East Zone. The Foothill East, Lower I, and Mountain South Zones are supplied exclusively by PRVs.

The existing water demands within each zone are presented in Table 6.3. As mentioned in Chapter 3, since year 2015 the City experienced low demands due to conservation mandates in response to the state-wide drought. The existing demands in this IMP refers to the average demand of year 2012 through year 2014. As shown in Table 6.3, the City's Average Day Demand (ADD) is 7.7 mgd and the Maximum Day Demand (MDD) is 13.0 mgd. The majority (72.4 percent) of the City's existing demand is located in the Main Zone, which has an existing ADD of 5.6 mgd.

The second highest demand (15.1 percent) is in the Foothill West Zone, which has an existing ADD of 1.2 mgd.

Table 6.3 Existing Pressure Zone Demands

Name	HGL (ft msl)	ADD(1) (mgd)	MDD(2) (mgd)	Percent (%)
Foothill East	3,000	0.1	0.2	1.2%
Foothill West	2,822	1.2	2.0	15.1%
Lower I	2,450	0.7	1.2	9.5%
Main	2,721	5.6	9.4	72.4%
Mountain North	2,932	0.1	0.2	1.2%
Mountain South	2,546	<0.1	0.1	0.6%
Total	**N/A**	**7.7**	**13.0**	**100%**

Notes:
(1) Billing data from year 2016 geocoded and scaled up to average of years 2012-2014 production.
(2) Existing ADD multiplied by MDD peaking factor of 1.7.

A hydraulic profile of the City's existing water distribution system is shown on Figure 6.4. This hydraulic profile illustrates the hydraulic connectivity of the distribution system facilities in each pressure zone.

6.1.2.2 Pipelines

The City's distribution system consists of approximately 165 miles of pipeline ranging from 2 inches to 30 inches in diameter. A breakdown of pipelines by diameter and material type is presented in Table 6.4, while this data is graphically depicted on Figure 6.2.

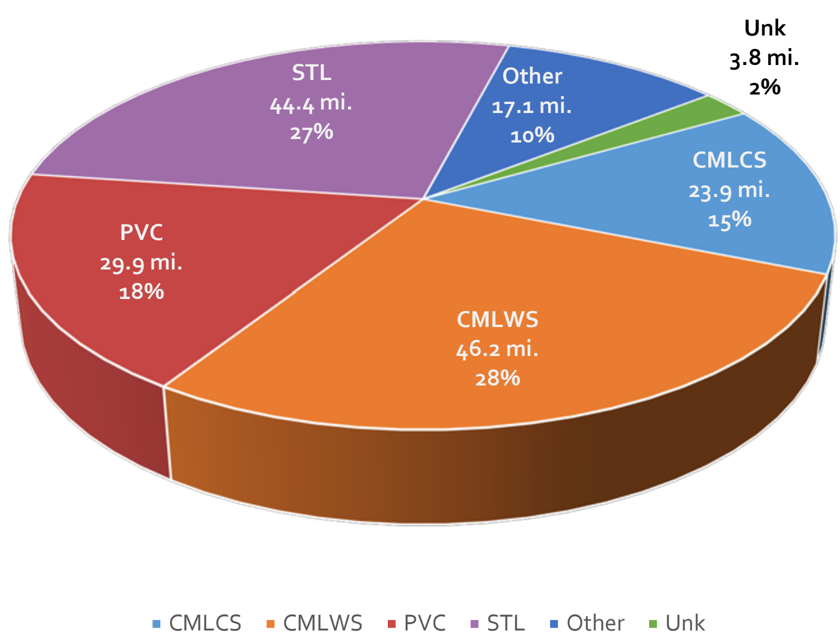

Figure 6.2 Pipelines by Material Type

Table 6.4 Potable Water Distribution System Pipelines

Diam. (in)	Pipeline Length (ft) by Material Class[1,2]						Total	
	CMLCS	CMLWS	PVC	STL	Other	Unk.	(ft)	(mi)
2	0	34	176	36,191	36	0	**36,437**	**6.9**
2.5	119	0	0	0	0	0	**214**	**0.0**
3	0	0	665	24	20	1,413	**2,122**	**0.4**
4	734	6,038	3,046	60,548	2,055	27	**72,448**	**13.7**
5	0	0	20	5,366	0	0	**5,386**	**1.0**
6	609	24,585	10,861	37,164	40,611	8,622	**122,452**	**23.2**
8	37,442	145,147	113,311	50,465	34,606	6,245	**387,216**	**73.3**
10	2,323	3,662	2,181	5,789	1,955	10	**15,919**	**3.0**
12	11,535	41,476	25,869	16,103	5,504	626	**101,113**	**19.2**
14	10,525	9,539	0	13,780	0	10	**33,855**	**6.4**
16	7,207	0	1,544	3,571	4,210	0	**16,531**	**3.1**
18	26,094	11,446	7	1,447	0	0	**38,994**	**7.4**
20	7,826	115	0	807	1,170	0	**9,918**	**1.9**
24	5,087	884	0	0	0	0	**5,970**	**1.1**
30	16,427	796	0	2,982	0	2,103	**22,309**	**4.2**
Unk.	0	0	0	0	0	904	**904**	**0.2**
Total (ft)	**125,928**	**243,722**	**157,679**	**234,237**	**90,167**	**19,961**	**871,694**	**N/A**
Total (mi)	**23.9**	**46.2**	**29.9**	**44.4**	**17.1**	**3.8**	**N/A**	**165.1**

Notes:
(1) Pipeline data retrieved from City's GIS.
(2) CMLCS = Cement-Mortar Lined & Coated Steel, CMLWS = Cember-Mortar Lined & Wrapped Steel, PVC = Polyvinyl Chloride, STL = Steel. Other category includes ACP = Asbestos-Cement Pipe, CMLS = Cement-Mortar Lined Steel, DIP = Ductile Iron Pipe, RS = Riveted Steel

As shown in Table 6.4, the majority (over 73 miles) of the City's transmission and distribution mains consist of 8-inch diameter. The City's GIS data has 904 feet or 0.2 miles of pipeline with unknown diameter. As shown on Table 6.4, the majority of the pipelines are made of cement-mortar lined and wrapped steel (CMLWS), which equates to 46.2 miles or 28 percent, and Steel (STL), which equates to 44.4 miles or 27 percent. The City's GIS also has 3.8 miles (or 2 percent) of pipeline with unknown material.

The pipeline length distribution by material and installation year is summarized in Table 6.5 and graphically depicted on Figure 6.3.

Table 6.5 Pipelines by Installation Year and Material Type

Material	Pipeline Length[1] (ft) by Installation Year							Total (mi)
	Prior to 1950	1951 to 1960	1961 to 1970	1971 to 1980	1981 to 2000	2001 to 2017	Unk.	
CMLCS	10	4,012	6,775	44,035	3,442	33,853	33,803	**23.9**
CMLWS	2,040	33,853	48,147	39,744	27,192	26,851	65,895	**46.2**
PVC	0	20	1,722	78,921	35,189	27,916	13,911	**29.9**
STL	83,583	111,124	8,238	1,411	260	4,115	25,507	**44.4**
Other	4,495	2,359	22,028	15,979	4,180	18,953	22,173	**17.1**
Unk	0	3,318	4,267	4,263	10	1,428	6,675	**3.8**
Total (ft)	**90,128**	**154,685**	**91,177**	**184,352**	**70,273**	**113,116**	**167,963**	**871,694**
Total (mi)	**17.1**	**29.3**	**17.3**	**34.9**	**13.3**	**21.4**	**31.8**	**165.1**

Notes:
(1) Pipeline data retrieved from City's GIS data. For pipes without an install date, the approved date was used.
(2) CMLCS = Cement-Mortar Lined & Coated Steel, CMLWS = Cember-Mortar Lined & Wrapped Steel, PVC = Polyvinyl Chloride, STL = Steel. Other category includes ACP = Asbestos-Cement Pipe, CMLS = Cement-Mortar Lined Steel, DIP = Ductile Iron Pipe, RS = Riveted Steel

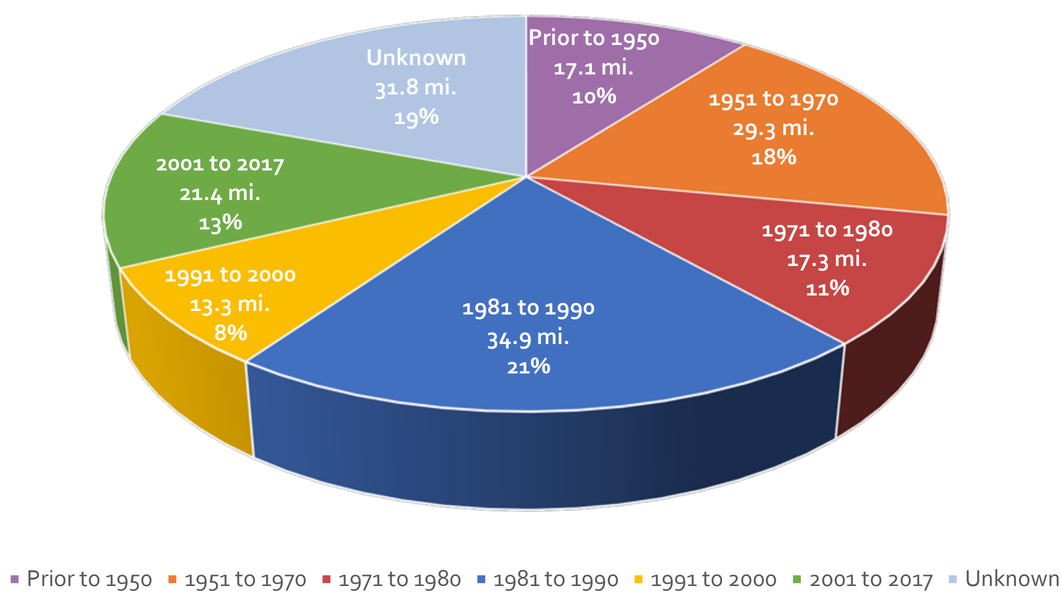

Figure 6.3 Pipelines by Installation Year

Upon initial review of the pipeline data, the City's GIS database was missing nearly half of the install dates. Reasonable assumptions were made by City staff to estimate these pipeline installation years based on the approved date to develop a pipeline replacement program, which will be discussed in Section 6.2.8. As shown in Figure 6.3, 19 percent of the pipeline installation dates remain unknown after these assumptions were made.

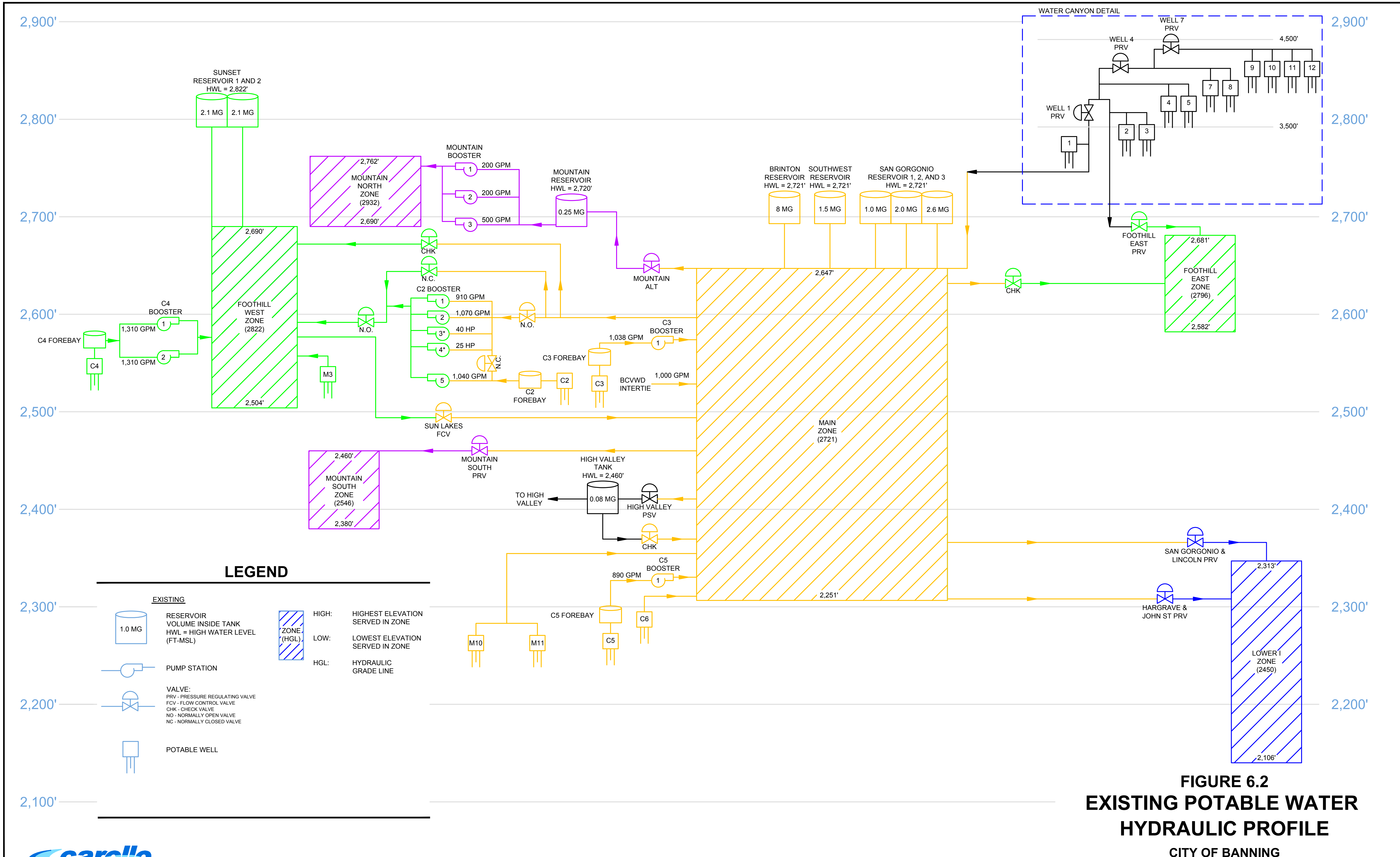

FIGURE 6.2
EXISTING POTABLE WATER HYDRAULIC PROFILE
CITY OF BANNING

6.1.2.3 Booster Pump Stations

The City's potable water distribution system uses two pump stations to move water between pressure zones. The C2 Booster PS pumps water from the Main Zone and Well C2 to the Foothill West Zone. The Mountain Booster PS pumps water from the Foothill West Zone to the Mountain North Zone. Table 6.6 lists some of the key characteristics for each pump station, while their operational functionality is described below.

Table 6.6 Existing Pumping Stations

Pump Station Name	From Zone	To Zone	Design Capacity (gpm)	Firm Capacity (gpm)
C2 Booster	Main Well C2	Foothill West Foothill West	1,980 1,040	910 0
Mountain Booster	Main	Mountain North	900	400

Notes:
(1) Capacities provided by City staff.

- **C2 Booster PS** pumps water from the Main Zone and Well C2 into the Foothill West Zone. The C2 Booster PS consists of five pump units. Two of the pumps that pump to the Main Zone have unknown design flows and are currently not operating. The two operating pump units that pump from the Main Zone are sized at 910 gpm and 1,070 gpm that pump from the Main Zone. One pump unit sized at 1,040 gpm pumps from Well C2.
- **Mountain Booster PS** pumps water from the Main Zone into the Mountain North Zone. The Mountain Booster PS consists of three pump units with two pumps sized at 200 gpm each and one pump sized at 500 gpm.

6.1.2.4 Storage Reservoirs

Water distribution systems rely on stored water to help equalize fluctuations between supply and demand. The storage criteria discussed in Chapter 5 determines the storage required within each pressure zone to provide adequate water supply for firefighting, emergency, or unplanned outages of a major source of supply, and to meet demands. Currently, the City's potable water system has 8 reservoirs that provide storage for the distribution system and 1 reservoir that is only used to pump water to High Valleys Water District.

Since the majority of the City's water supplies originate at the Main Zone, the majority (6) of the reservoirs are located in the Main Zone. The other reservoirs are configured to be replenished from lower pressure zones. This is achieved by using booster pump stations (seeTable 6.6) that pump water from the lower pressure zones to the higher pressure zones. Detailed information for each reservoir is summarized in Table 6.7.

Table 6.7 Existing Potable Water Reservoirs

Storage Tank Name[1]	Pressure Zone	HGL (ft)	Max Depth (ft)	Diameter or Length & Width (ft)	Capacity (MG)
Mountain Reservoir	Mountain North	2,720	28	39	0.25
Sunset Reservoir 1	Foothill West	2,822	28	107	2.1
Sunset Reservoir 2	Foothill West	2,822	28	107	2.1
Brinton Reservoir	Main	2,721	19	440 x 140	8.0
San Gorgonio Reservoir 1	Main	2,721	28	76	1.0
San Gorgonio Reservoir 2	Main	2,721	28	110	2.0
San Gorgonio Reservoir 3	Main	2,721	26	128	2.6
Southwest Reservoir	Main	2,721	26	100	1.5
High Valley Tank[2]	Main	2,460'	14	30	0.08
Total Storage Capacity[3]	**N/A**	**N/A**	**N/A**	**N/A**	**19.63**
Total City Storage Capacity[4]	**N/A**	**N/A**	**N/A**	**N/A**	**19.55**

Notes:
(1) Reservoir details provided by City Staff
(2) High Valley Tank does not provide City storage and is used to pump water to High Valleys Water District.
(3) Total Storage Capacity includes High Valley Tank capacity.
(4) Total City Storage Capacity does not include High Valley Tank capacity.

As shown in Table 6.7, the City has nearly 20 (19.55) million gallons (MG) of storage capacity. The majority of this is located in the Main Zone, with 15.1 MG of storage (77 percent of total), while the remaining 4.45 MG (23 percent) of storage is located in higher pressure zones.

6.1.2.5 Pressure Reducing Valves

PRVs allow distribution systems to transfer water from higher-pressure zones to lower-pressure zones without exceeding the allowable pressures in the lower zones or completely draining the pressure out of the higher zone. Water is transferred through a valve that reduces the pressure to a specified pressure setting (pressure-reducing feature), while maintaining the pressure in the upper pressure zones (pressure-sustaining feature).

The pressure-sustaining feature prevents transfer of water into the lower pressure zone if the pressure in the upper zone drops below a certain level. This helps prevent a problem or emergency in the upper pressure zone draining too much water into the lower pressure zone.

The City utilizes five major PRVs that transfer water between pressure zones. The characteristics of these major PRVs are summarized in Table 6.8.

Table 6.8 Existing Pressure Reducing Valves

PRV Name	From Zone	To Zone	No. of Valves	Valve Sizes (inches)	Pressure Setpoint (psi)
Foothill East PRV	Canyon Wells	Foothill East	3	2 2 6	60 55 50
Well 1 PRV	Canyon Wells	Main	1	10	78
Mountain South PRV	Main	Mountain South	2	8	70
San Gorgonio & Lincoln PRV	Main	Lower I	3	2 4 8	Off 55 52
Hargrave & John St PRV	Main	Lower I	3	4 6 8	70 67 64

6.2 Existing System Analysis

The goal of the existing system analysis is to evaluate the existing distribution system under various operating conditions utilizing the evaluation criteria described in Chapter 5 and the existing system demands listed in Table 6.3. The following analyses are described in this section:

- Existing Water Supply Analysis
- Existing System Pressure Analysis
- Existing Pipeline Velocity Analysis
- Existing Fire Flow Analysis
- Existing Storage Analysis
- Existing Pump Station Analysis
- Existing Condition Assessment

6.2.1 Water Supply Analysis

Currently, 100 percent of the City's potable water system is supplied by groundwater from the wells listed in Table 6.1. A supply analysis was performed for two different scenarios: largest supply out of service and extreme drought conditions.

The first scenario was conducted to determine potential supply sources in the event the largest supply was out of service. During normal and wet years, the Canyon Wells (Wells 1-5 and 7-12), which are the largest supply into the system, supply an average of approximately 3,000 gpm to the Main Zone and Foothill East Zone. The largest of the Canyon Wells are Wells 7 and 10 with a capacity of 1,000 gpm each. In this scenario, it is assumed that either Well 7 or 10 is out of service, reducing the Canyon Wells supply from 3,000 gpm to 2,000 gpm.

In addition, a second scenario was conducted in the event of extreme drought conditions. Based on recent experience during extreme drought conditions, City staff estimates that at least 1,700 gpm of the combined 3,000 gpm capacity can be supplied from the Canyon Wells, even after multi-year drought conditions. Based on this recent experience, this scenario assumed that the supply from the Canyon Wells is 1,700 gpm and demands remain the same. A summary of the analyses are presented in Table 6.9 and Table 6.10, while details are presented in Appendix E.3.

Table 6.9 Existing Water Supply Analysis with Largest Supply Out of Service

Pressure Zone	Total Supply Capacity (gpm)	Supply Capacity w/ Largest Supply out of service (gpm)	Existing MDD(2) (gpm)	Existing Capacity Balance (gpm)
Foothill East	3,000	2,000	110	1,890
Foothill West	4,200	4,200	1,371	2,829
Mountain North	0	0	107	(107)
Mountain South	0	0	58	(58)
Main	5,300	5,300	6,567	(1,266)
Lower I	0	0	855	(855)
Total	**12,500**	**11,500**	**9,067**	**2,433**

Notes:
(1) Supply capacities retrieved from City staff.
(2) MDD peaking factor assumed to be 1.7.
(3) Detailed calculations are found in Appendix E.3.

Table 6.10 Existing Water Supply Analysis with Extreme Drought Conditions

Pressure Zone	Total Supply Capacity (gpm)	Supply Capacity w/ Largest Supply out of service (gpm)	Existing MDD(2) (gpm)	Existing Capacity Balance (gpm)
Foothill East	3,000	1,700	110	1,590
Foothill West	4,200	4,200	1,371	2,829
Mountain North	0	0	107	(107)
Mountain South	0	0	58	(58)
Main	5,300	5,300	6,567	(1,266)
Lower I	0	0	855	(855)
Total	**11,200**	**7,600**	**9,067**	**2,133**

Notes:
(1) Supply capacities retrieved from City staff.
(2) MDD peaking factor assumed to be 1.7.
(3) Detailed calculations are found in Appendix E.3.

As shown in Table 6.9 and Table 6.10, Foothill East and Foothill West are the only pressure zones with excess supply under both scenarios. While the other pressure zones are deficient, all of the deficiencies can be resolved from using existing PRVs to convey the excess water in Foothill East and Foothill West to the lower zones by gravity. Based on the existing system supply reliability analysis summarized in Table 6.9 and Table 6.10, no recommendations are made.

6.2.2 System Pressure Analysis

Based on the evaluation criteria listed in Chapter 5, the system pressures were evaluated for the distribution system under existing demand conditions. The hydraulic model was used to identify areas with pressures above 150 psi under minimum demand (MinDD) conditions, while peak hour demand (PHD) conditions were used to identify areas with pressures below 40 psi. The results from this pressure analysis were predicted by the model, but have not been verified by pressure logger data in the field. It is strongly recommended that the City evaluates pressures from field data prior to addressing the predicted deficiencies.

6.2.2.1 High Pressures

When conducting the analysis of the existing system using the hydraulic model, several areas with pressures greater than 150 psi were identified, which are presented on Figure 6.5. The majority of the high pressures are located in the lower portion of the Main Zone.

The high pressures have been confirmed by the City's operations staff. However, the City has plans to rezone this area. Valves were constructed to create this pressure zone separation. However, the project was never completed. Based on recommendations from City staff, the high pressures in the existing system will be addressed as part of the future system analysis, found in Section 6.3.2.

6.2.2.2 Low Pressures

Instances of low pressures (less than 40 psi) in the existing system were minimal and occurred near tanks and reservoirs, which are presented on Figure 6.6. Since the low pressures are a result of static pressure, recommendations were not made. The majority of the low pressures occur near the High Valley Tank as a result of operational conditions in which the City only fills the tank during limited number of hours each day. This results in low pressures in parts of the Main Zone during these hours. It is recommended that the City fill the tank for longer periods throughout the day to mitigate this issue. This may require installation of a flow control valve or pressure sustaining valve to control the fill rate and avoid that this does not lead to low pressures upstream.

6.2.3 Pipeline Velocity Analysis

The hydraulic model was used to evaluate pipeline velocities in the existing system with existing system demands under MDD conditions. The hydraulic model results indicated that some pipelines in the Main Zone towards the High Valley Tank have high pipeline velocities above 7 feet per second (fps), as shown on Figure 6.7. The high velocity is a result of the operational conditions of the High Valley Tank and the fact that this portion of the Main Zone receives water from a single undersized pipe. Fire flow improvements, which are presented in Section 6.2.4, are identified to mitigate these velocity issues. Thus, no further improvements are recommended to mitigate the high velocities.

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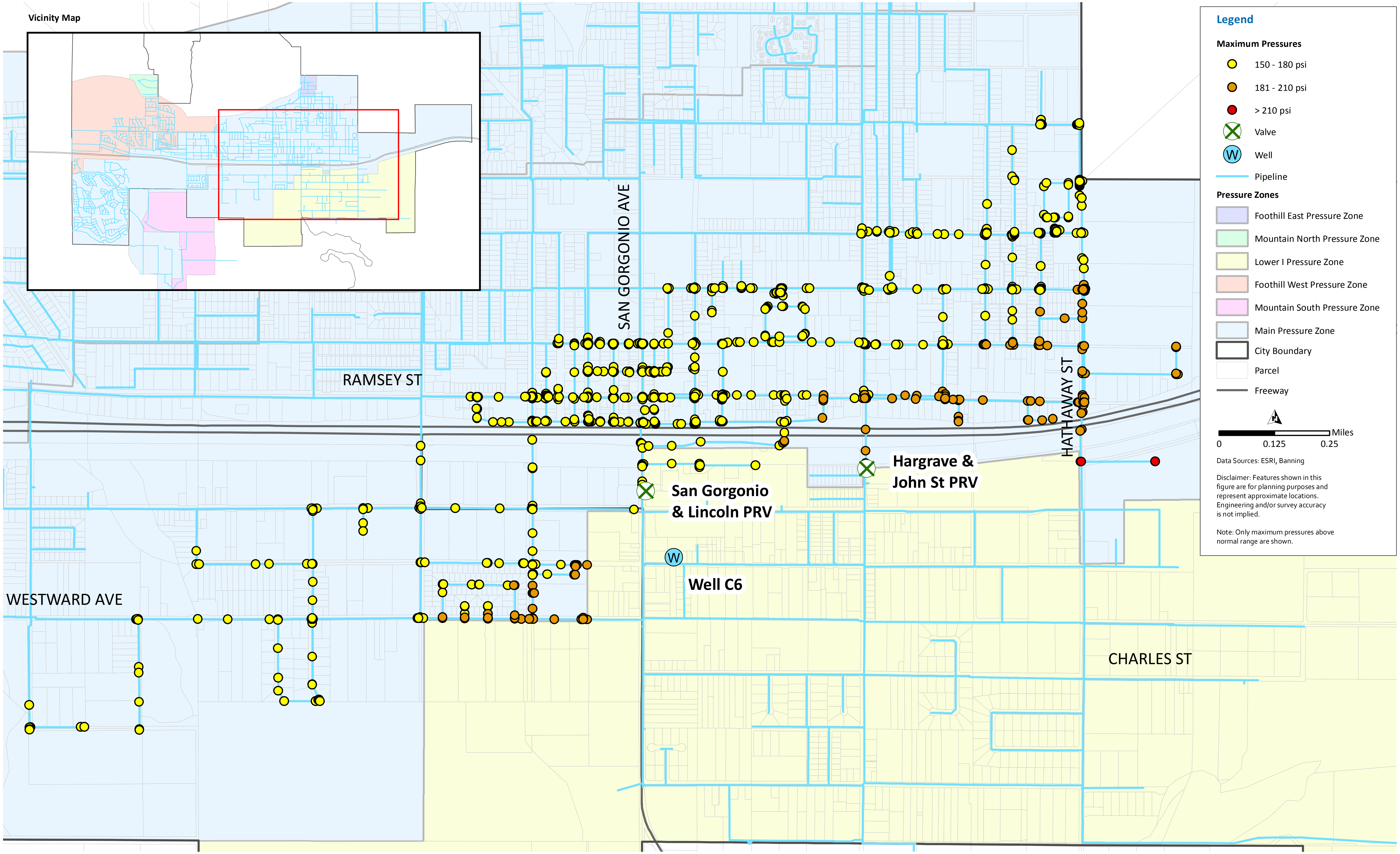


Figure 6.5 Existing System Maximum Pressures

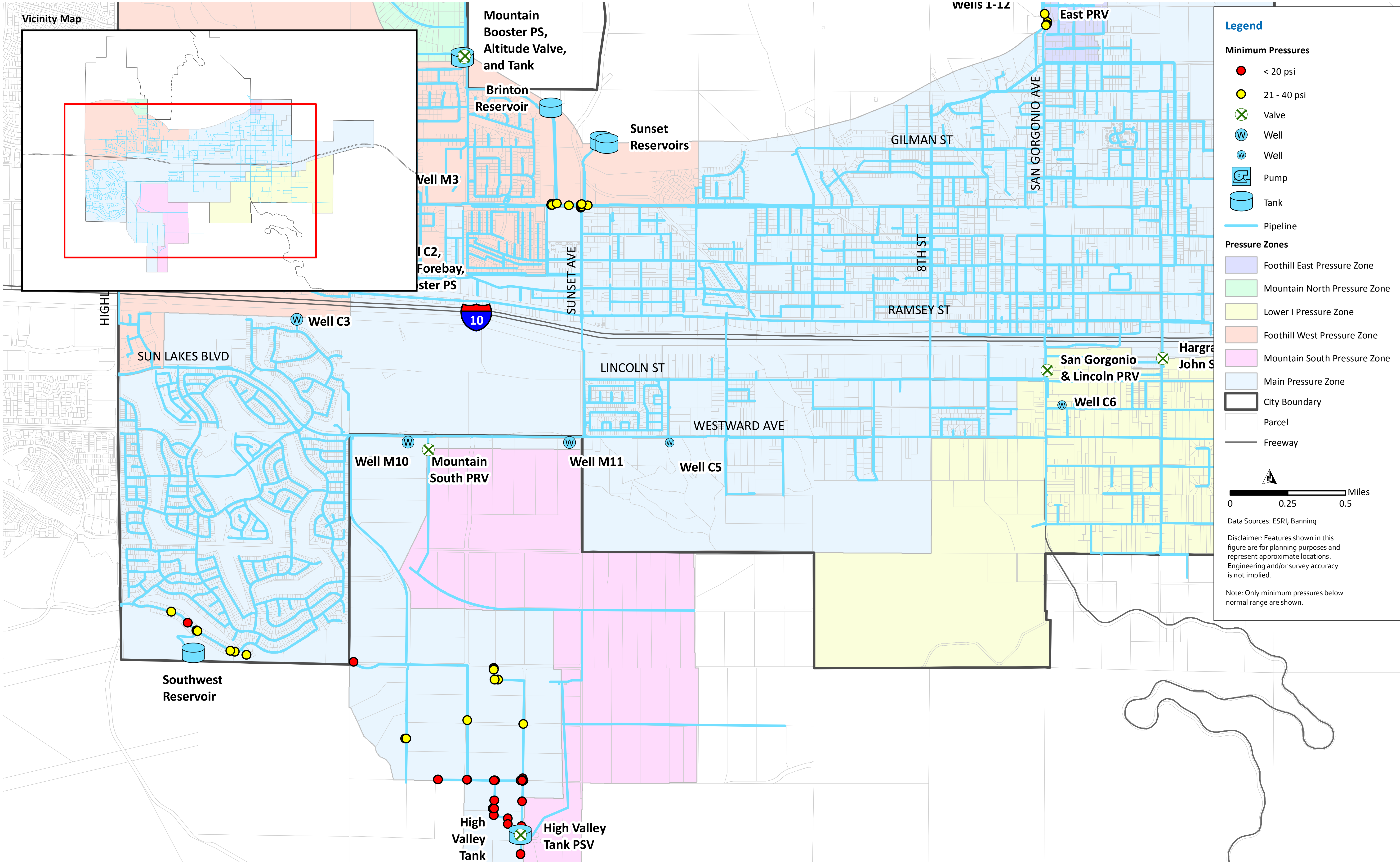


Figure 6.6 Existing System Minimum Pressures

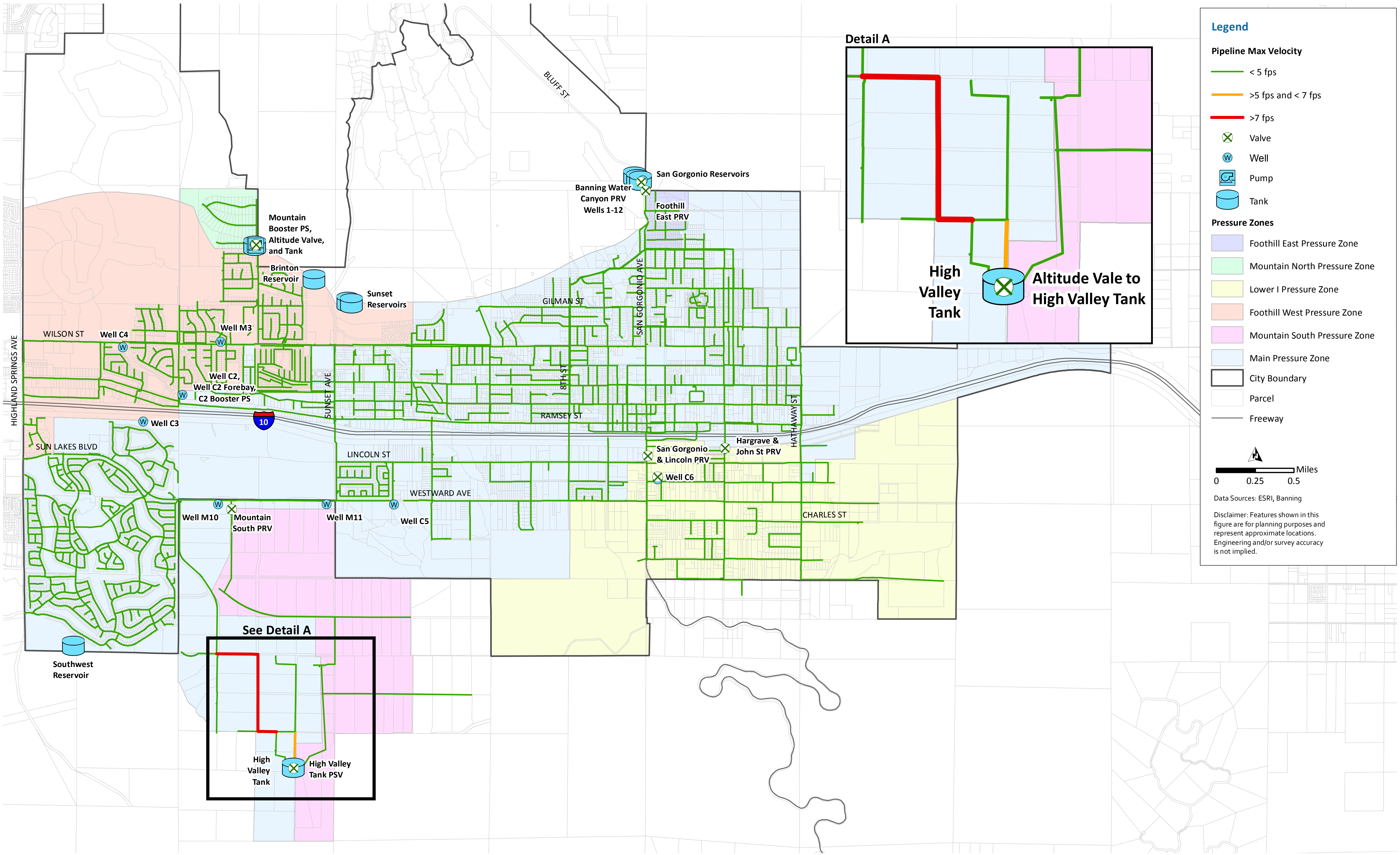


Figure 6.7 Existing System Maximum Velocities Under MDD Conditions

6.2.4 Fire Flow Analysis

A fire-flow analysis was completed using the evaluation criteria listed in Chapter 5. Based on these criteria, the existing fire-flow system was evaluated to verify that a minimum residual pressure of 20 psi was met under MDD conditions while maintaining a flow ranging from 1,500 gpm to 4,000 gpm within the corresponding land-use category. Fire Flow improvements are summarized in Table 6.11. Since pipeline replacement projects may overlap some of these fire flow improvement projects, they were only counted as fire flow projects and removed from the pipeline replacement program.

As shown in Table 6.11, a total of 23 improvements have been proposed involving upsizing existing pipelines and/or completing pipeline loops with a combined length of approximately 30,000 feet or 5.7 miles. In addition to pipelines, one fire flow project (PWFF-3) includes the addition of a PRV and check valve. The projects in this table are ranked by importance, as determined by the proximity to critical facilities, land use type, and the severity of the deficiencies resolved. The ranking of the fire flow improvements and detailed maps for each individual improvement is presented in Appendix E.1. The locations of the recommended improvement projects are shown on Figure 6.8.

It should be noted that at the time when the existing facilities were constructed, less stringent fire-flow criteria was in place. Hence, this analysis may identify insufficient pipeline conveyance capacity at certain locations. As it is beyond the scope of this IMP to verify the historic fire-flow criteria, it is recommended that City staff verify the actual fire-flow criteria for the governing fire protection district to evaluate alternatives to improve fire protection while minimizing the need to upsize existing pipelines. In addition, the fire flow criteria may be reduced in select locations that have indoor fire sprinkler systems. These locations will be reviewed by the City to determine if some of the fire flow projects can be eliminated with a reduced flow rate requirement, as allowed by the California Fire Code.

Table 6.11 Proposed Fire Flow Improvements

CIP ID	Street	Number of Nodes Resolved	Existing Diameter (in)	New Diameter (in)	Length of Pipeline (ft)
PWFF-1	W Jacinto View Road	1	6	8	400
PWFF-2	Wilson Street	1	n/a	8	100
PWFF-3(1)	Westward Avenue	26	3, 6	8 12	2,400 6,900
PWFF-4	Linda Vista Drive	1	2	8	100
PWFF-5	Jennifer Way	1	4	8	700
PWFF-6	Cottonwood Road	1	4	8	500
PWFF-7	Sloping View Drive	4	n/a	8	100
PWFF-8	Park Avenue	2	n/a	8	100
PWFF-9	Nicolet Street	1	4	8	600
PWFF-10	Alessandro Road	1	4	8	500
PWFF-11	Vista Serena Avenue Gillman Street	1	4	8 12	200 2,000
PWFF-12	Florida Street Hoffer Street	2	4, 6, 8	8 12	1,300 1,500
PWFF-13	Hay Street	2	8	8	1,000
PWFF-14	Ramsey Street	2	4, 6	8	1,400
PWFF-15	Sloping View Drive	3	4	8	1,400
PWFF-16	Park Avenue Sunview Drive	4	4, 6	8	4,800
PWFF-17	Nicolet Street 22nd Street	1	n/a	8	200
PWFF-18	Wilson Street Linda Vista Drive	2	4	8	1,500
PWFF-19	Williams Street	1	6	6	100
PWFF-20	Business east of Well C2	1	6	8	300
PWFF-21	Jacinto View Road	1	6	8	300
PWFF-22	Alessandro Road	1	4	8	900
PWFF-23	San Gorgonio Avenue First Street	1	6	8	700
Total	**N/A**	**62**	**n/a**	**n/a**	**30,000**

Note:
(1) Project also requires an 8-inch diameter PRV.

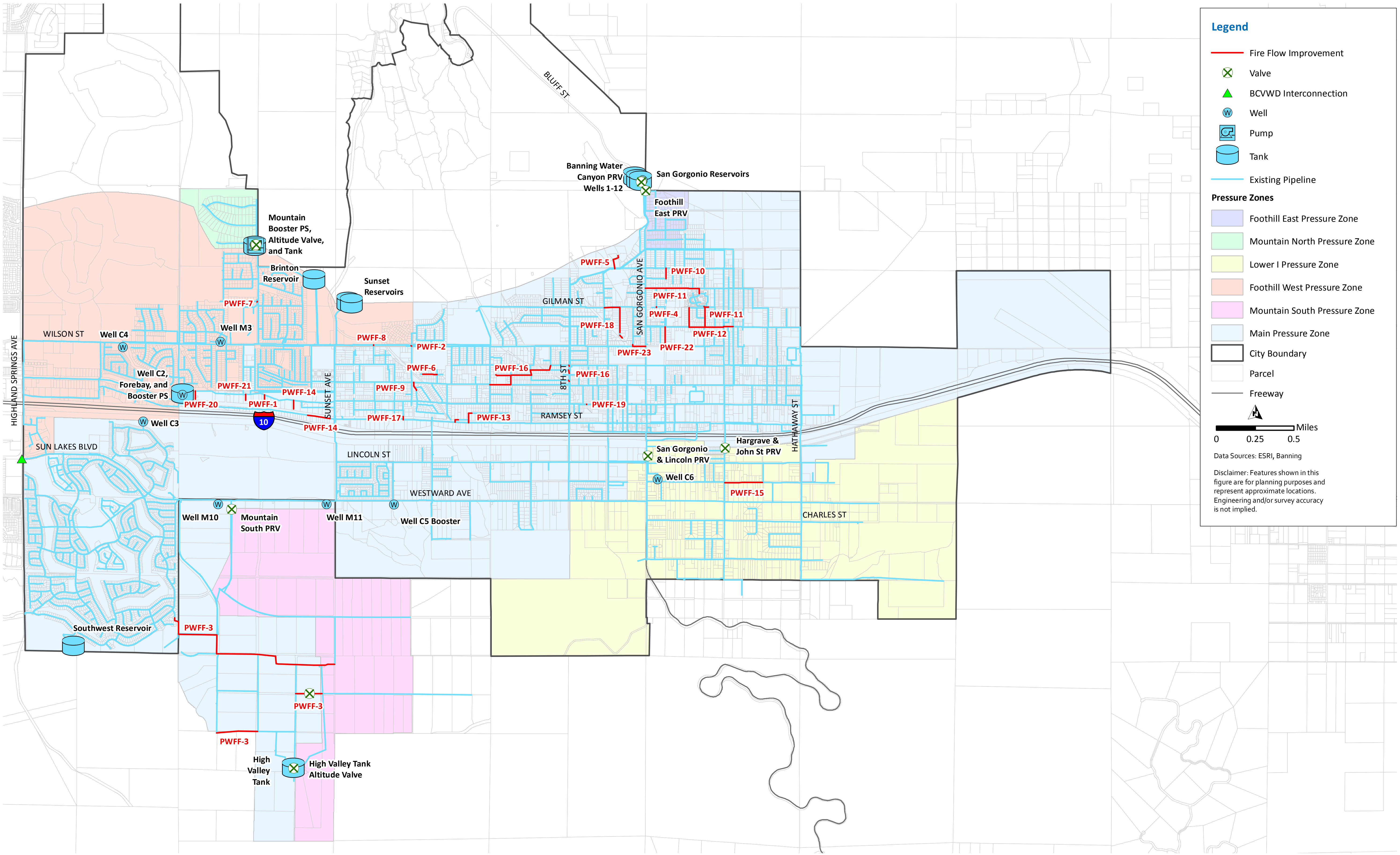


Figure 6.8 Existing Fire Flow Improvements

6.2.5 Storage Analysis

The storage analysis evaluates the existing storage capacity based on the evaluation criteria listed in Chapter 5. These storage criteria include 3 components, namely operational, fire-flow, and emergency storage. Based on the criteria listed, a storage analysis was completed under existing MDD conditions. The results of this analysis are summarized in Table 6.12, while details of this analysis are presented in Appendix E.2. An example of the calculations are presented below:

Existing Zone Storage Capacity	0.78 MG
Required Storage Capacity (Existing Conditions)	2.55 MG
Required Storage Capacity (Future 2040 Conditions)	3.29 MG
Existing Storage Deficit (2.55 MG – 0.78 MG)	1.77 MG
Future (2040) Storage Deficit (3.29 MG – 2.55 MG)	0.74 MG
Proposed Storage Capacity	**2.50 MG**
Existing User Benefit (1.77 MG / 2.5 MG)	**71 Percent**
Future (2040) User Benefit (0.74 MG / 2.5 MG)	**29 Percent**

As shown in Table 6.12, there are seven gravity reservoirs within the existing potable water system that have a total storage capacity of 19.3 MG. The storage evaluation demonstrated that the City's current system has a storage deficit of 0.65 MG under existing MDD conditions. The deficiencies and recommended storage improvements are as follows (see Table 6.12):

- Main Zone (Project PWS-1 & PWP-2): The Main Zone is in the middle of the City's potable water system and has a combined MDD of 9.46 mgd. The required storage for the Main Zone is 12.78 MG. Although the zone has 15.1 MG of storage available, resulting in a surplus of 2.3 MG, additional storage is required to accommodate for the deficiencies in the Foothill East, Mountain South, and Lower I Zones. One new storage tank (Reservoir 1) is proposed with a total capacity of 4 MG. Based on the ratio of existing versus future customer benefit, 79 percent is allocated to existing users. Based on the configuration of the existing system, a new 24-inch diameter transmission main with a total length of 6,500 feet will be required to connect this new storage facility with the distribution system of the Main Zone.
- Foothill East Zone: The Foothill East Zone is in the northern part of the City's potable water system. With a combined MDD of 0.16 mgd, the required storage is 0.38 MG. However, since the zone does not have any storage, it is recommended to mitigate the deficiency by diverting more Canyon Wells supply (approximately 262 gpm more) to the Foothill East Zone during emergency conditions. No storage improvements are recommended in this zone.
- Mountain North Zone: The Mountain North Zone is in the northwest part of the City's potable water system. With a combined MDD of 0.15 mgd, the required storage is 0.37 MG. However, since the zone does not have any storage, it is recommended to mitigate the deficiency by pumping this deficit from the Foothill West Zone, which has surplus storage to supply the deficits in the Mountain North Zone. No storage improvements are recommended in this zone.
- Mountain South Zone: The Mountain South Zone is in the southwest part of the City's potable water system. With a combined MDD of 0.08 mgd, the required storage is 0.34 MG. However, since the zone does not have any gravity storage, it is recommended to mitigate the deficiency by conveying water through the PRV from the Main Zone. No storage improvements are recommended in this zone.

Table 6.12 Existing Storage Analysis

Zone	Existing Storage Facility	Existing MDD[1] (mgd)	Required Storage (MG)	Available Storage (MG)	Zone Balance (MG)	Proposed Facilities	Proposed Capacity (MG)	Balance with New Storage (MG)
Foothill East	None	0.16	0.38	0.0	(0.38)	None. Pump from Main Zone	0	0.00
Foothill West	Sunset Reservoirs	1.97	3.31	4.2	0.89	N/A	0	0.52
Mountain North	Mountain Reservoir[3]	0.15	0.37	0.0	(0.37)	None. Pump from Foothill West Zone	0	0.00
Mountain South	None	0.08	0.28	0.0	(0.28)	PRV from Main Zone	0	0.00
Main	Brinton Reservoir San Gorgonio Reservoirs Southwest Reservoir	9.46	12.78	8.0 5.6 1.5	2.32	New Main Reservoir 1	4.00	3.16
Lower I		1.23	2.50	0.0	(2.50)	PRV from Main Zone	0	0.00
Total		**13.06**	**19.62**	**19.3**	**(0.32)**	**N/A**	**4.00**	**3.68**

Notes:
(1) MDD assumed to be ADD x 1.7.
(2) Reservoir capacities provided by City staff.
(3) Mountain Reservoir (0.25 MG) is not a gravity storage and thus, is not included in this analysis.

- Lower I Zone: The Lower I Zone is in the southeast part of the City's potable water system. With a combined MDD of 1.23 mgd, the required storage is 2.50 MG. However, since the zone does not have any storage, it is recommended to mitigate the deficiency by conveying water through the two PRVs from the Main Zone. No additional storage improvements are recommended in this zone.

6.2.6 Pump Station Analysis

The pump station analysis evaluates the existing pump station capacities based on the evaluation criteria listed in Table 5.1. These pump station evaluation criteria define that in zones with gravity storage, the firm capacity of the booster pump station shall be able to supply MDD of the zone it feeds into (including upstream zones), unless the zone it pumps into has its own production capacity, such as the Foothill West Zone. In zones without gravity storage, the firm capacity of the pump station shall be able to supply MDD of the zone it feeds into (including upstream zones), as well as, the maximum fire-flow demand in that zone.

The results of the pump station analysis is summarized in Table 6.13, while the details are presented in Appendix E.4. The same methodology listed in Section 6.2.5 was utilized to determine existing and future user benefit of new or upgraded pump stations. If there was a surplus in proposed capacity, it was applied to the future user benefit.

The City currently has two pump stations with a combined capacity of 3,920 gpm. The firm pumping capacity of these two pump stations is about 2,350 gpm or 3.4 mgd. As listed in Table 6.13, the pump station evaluation demonstrated a pumping deficiency of 4,671 gpm under existing demand conditions. The deficiencies and recommended improvements are as follows:

- Mountain Booster PS Upgrade (Project PWPU-1a): The Mountain Booster PS pumps from the Main Zone to the Mountain North Zone, which has a pumping deficiency of 1,207 gpm. Although the existing Mountain Booster PS has sufficient capacity to supply the existing MDD, it does not have enough capacity to supply fire flow. To mitigate this deficiency, it is recommended to add two pumps with a capacity of 725 gpm each. This 80 hp upgrade would provide fire flow protection to the Mountain North Zone. Additionally, it will provide supply redundancy and mitigate pumping deficiencies in the Mountain North Zone by increasing the total and firm pumping capacity of the PS to 2,350 gpm and 1,625 gpm, respectively. Based on the ratio of existing versus future customer benefit, 100 percent is allocated to existing users.
- New Mountain South 2 PRV Station (Project PWFF-3): A new Mountain South 2 PRV is recommended to provide sufficient supply to the Mountain South Zone, which has a pumping deficiency of 1,558 gpm. Although the existing Mountain South PRV has sufficient capacity to supply the existing MDD, it does not have enough capacity to supply fire flow. This new 8-inch diameter PRV is also part of a fire flow project (PWFF-3). Based on the ratio of existing versus future customer benefit, 100 percent is allocated to existing users.
- New Foothill East 2 PRV (Project PWV-3): A new Foothill East 2 PRV is proposed to provide redundancy from the Canyon Wells to the Foothill East Zone, which has a pumping deficiency of 1,400 gpm. This new 6-inch diameter PRV would provide supply redundancy. Based on the ratio of existing versus future customer benefit, 100 percent is allocated to existing users.

Table 6.13 Existing Pump Station Analysis

Discharge Pressure Zone	Existing MDD(1) (gpm)	Total Required Capacity (gpm)	Existing Firm Capacity(2) (gpm)	Existing Capacity Balance (gpm)	Proposed Facilities	Proposed Capacity (gpm)	Proposed PS Capacity (hp)
Foothill East	110	110	110	(1,400)	New Foothill East 2 PRV (8-inch)	1,800	n/a
Foothill West	1,371	1,371	3,820	3,093	None.	-	-
Mountain North	107	107	400	(1,207)	Mountain Booster PS Upgrade (2 new pumps @ 725 gpm)	1,450	80
Mountain South	58	58	0	(1,558)	New Mountain South 2 PRV(3)	2,100	n/a
Main	6,566	7,586	5,300	(586)	None.	-	-
Lower I	855	855	4,896	41	None.	-	-
Total	**9,067**	**10,673**	**19,370**	**197**	**N/A**	**5,350**	**80**

Notes:
(1) MDD assumed to be ADD x 1.7.
(2) PS capacities provided by City staff
(3) New Mountain South 2 PRV is part of a fire flow improvement.
(4) Detailed calculations in Appendix E.4.

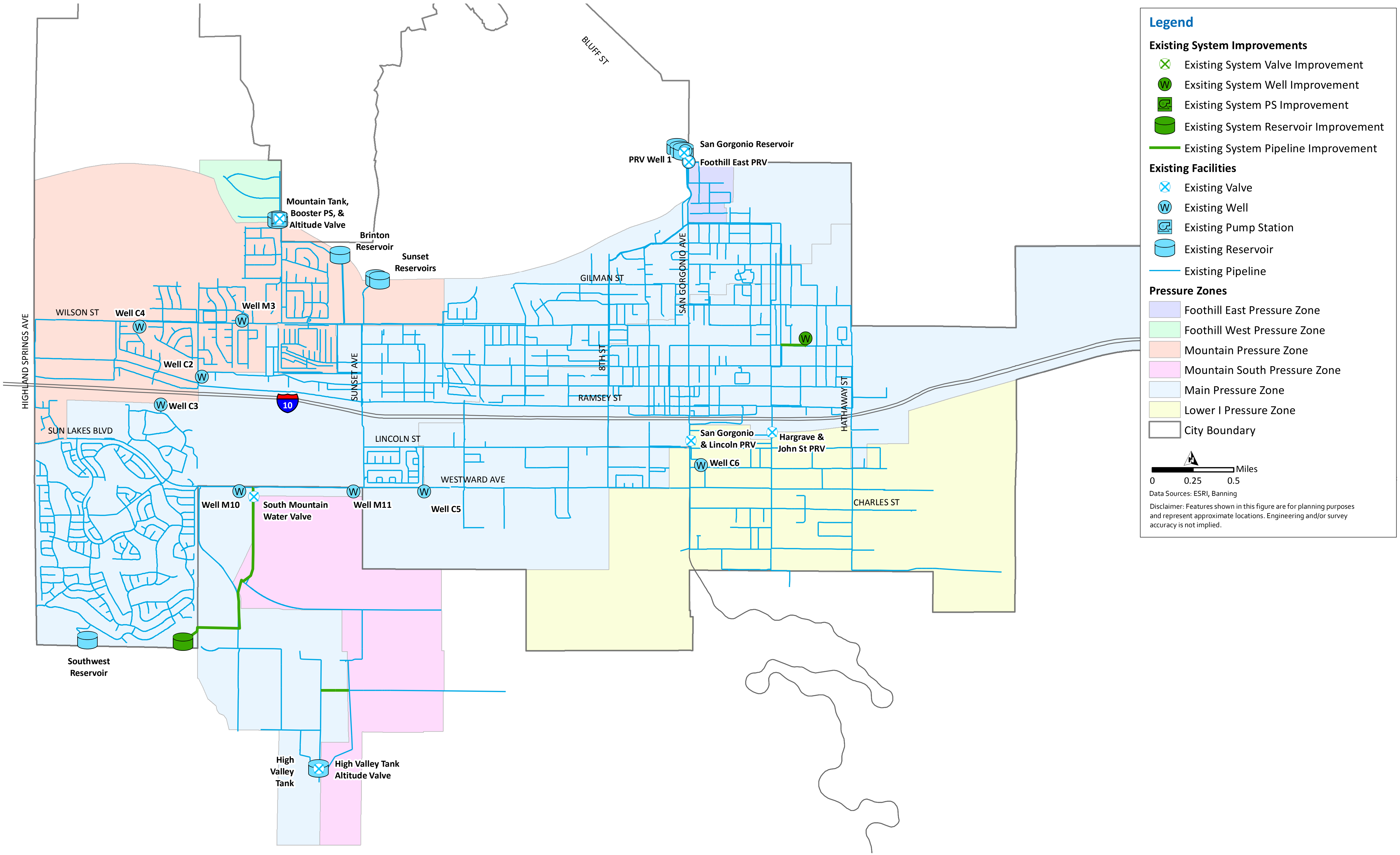


Figure 6.9 Existing System Capacity Improvements

6.2.7 Condition Assessment

A condition assessment was performed on June 7, 2017 by Carollo Engineers' assessment team for eight well sites, five reservoir sites, and two PRV stations that were identified by City staff as the most critical facilities of the potable water system. The following facilities were assessed:

- Well Sites
  - Well 1
  - Well 3
  - Well C-2
  - Well C-5
  - Well C-6
  - Well M-3
  - Well M-11
  - Well M-12
- Reservoir Sites
  - San Gorgonio Reservoirs 1, 2, and 3
  - Sunset Reservoirs 1 and 2
  - Mountain Reservoir (including hydropneumatic pump station)
  - Southwest Reservoir
  - High Valley Tank
- PRV Stations
  - Foothill East
  - Hargrave & John St.

Improvement projects were identified and grouped into a near-term phase (by year 2025) and a long-term phase (year 2026-2040). A summary of the recommended CIP projects listed in order of priority for the potable water facilities is provided in Table 6.14. The detailed Condition Assessment Report is provided in Appendix D.

Table 6.14 Condition Assessment Recommendation and Improvements

Project ID	Facility	Recommended Improvements
PWRR-5	San Gorgonio Reservoirs	**Near-Term:** Seismic evaluation/piping upgrades, overflow Title 17 compliance, tank base repair, replace vault roof, install bollards and site lighting. **Long-Term:** Pavement repair.
PWRR-6	Southwest Reservoir	**Near-Term:** Install mesh cover on ladder, overflow Title 17 compliance. **Long-Term:** Secure electrical equipment.
PWRR-7	Mountain Reservoir	**Near-Term:** Seismic evaluation/piping upgrades, overflow Title 17 compliance, replace ladder, install tank vent, replace tank interior gaskets, recoat tank floor, install anchors for pressure vessels, install air compressor, **Long-Term:** Replace booster pumps and install VFDs, level gauge, and floating assembly.
PWRR-8	High Valley Reservoir	**Near-Term:** Seismic evaluation/piping upgrades, overflow Title 17 compliance, repair tank leak, clean and protect tank bolts, replace exterior ladder, install internal ladder/replace access hatch

Project ID	Facility	Recommended Improvements
		Long-Term: Replace tank
PWRR-9	Sunset Reservoirs	**Near-Term:** Seismic evaluation/piping upgrades, tank recoating, install overflow screen, fence repairs and improvements, site security, install bollards. **Long-Term:** Repair pavement, replace level gauges, replace floating assemblies.
PWRR-10	Foothill East PRV	**Long-Term:** Install pressure transmitters and a SCADA antenna.
PWRR-11	Hargrave and John PRV	**Near-Term:** Install pipe supports **Long-Term:** Install pressure transmitters, install a SCADA antenna, and replace the site fencing.
PWRR-12	Well 1	**Near-Term:** Bypass reservoir inspection and rehab, flow meter vault locks, and bypass repair and supports. **Long-Term:** Replace bypass reservoir.
PWRR-13	Well 3	**Near-Term:** Install proper pipeline supports and security or removal of accu-tab system. **Long-Term:** PRV flow meter and piping modifications for flow meter.
PWRR-14	Well C-2	**Near-Term:** Seismic evaluation/ piping verification, tank base repair, recoating, cleaning, and inspection, installation of well pump VFD, replace A/C unit and frame, rehab pump 5, install bollards, and booster station roof seismic anchorage installation and repair. **Long-Term:** Install ATS for standby generator, check valve vault rehab, pipe support upgrades, and replacement of entry gate.
PWRR-15	Well C-5	**Near-Term:** Seismic evaluation/piping verification, tank base repair, overflow discharge modifications, and install well pump VFD, bollards, emergency generator hookup, and anchors on electrical equipment. **Long-Term:** Repaint and recoating and replace vault lid and electrical cabinets.
PWRR-16	Well C-6	**Near-Term:** Install bollards. **Long-Term:** Repair pavement.
PWRR-17	Well M-3	**Near-Term:** Install bollards. **Long-Term:** Install proper pipe supports, construct well house, repaint piping.
PWRR-18	Well M-11	**Near-Term:** Install electrical equipment and bollards. **Long-Term:** Repaint piping, repair pavement, install proper pipe supports.
PWRR-19	Well M-12	**Long-Term:** Install ATS for standby generator.
PWRR-22	Multi-Site	**Near-Term:** Emergency power and safety retrofits.

Note:
(1) See Appendix D for condition assessment technical memorandum.

6.2.8 Pipeline Replacement Analysis

As presented in 6.1.2.2, the City's GIS currently has approximately 165 miles of potable water pipelines that were installed between 1914 and 2012. Based on the GIS, 19 percent of the pipelines had missing information on either pipeline material and/or year of installation. As a full asset-management analysis is beyond the scope of this IMP, a cursory level pipeline replacement analysis was conducted along with planning level cost estimates using a number of general planning assumptions. Since a large amount of the pipeline material was also unknown, the estimated average useful life was assumed to be 80 years. To estimate the pipeline age for those pipes with unknown installation dates, the following method was used by City staff:

- The approved date field, if populated, in the GIS was used as an approximation for the installation date.
- Remaining pipelines without an installation date or approved date were assigned an installation date based on field observations and the age of surrounding pipelines.

Based on these assumptions, approximately 31.6 miles of pipelines would require replacement by year 2025 and an additional 37.5 miles of pipelines would require replacement between year 2026 and 2040. Details on this calculation are included in Appendix G. This estimate does not include the pipelines that remained with unknown diameter after using the two methods above, which equated to 19.3 percent (xx miles) of the total pipelines. To account for the remaining unknown diameters, City staff used an adjustment factor to estimate the cost. To assist with identifying and replacing pipelines that are at the end of their useful life, the following projects have been recommended:

- Pipeline Rehabilitation Asset Study (PWO-1): This project is recommended to better understand the characteristics of the City's existing pipelines and refine the pipeline replacement program.
- Pipeline Replacement Program (PWRR-1): This project is recommended to maintain the existing distribution system and replace pipelines that have already reached or are nearing the end of their useful life. This is estimated to be a total of approximately 40 (39.1 calculated) miles of pipeline.

6.2.9 Other Improvements

Other miscellaneous improvement projects have been recommended to optimize the operation of the City's potable water system or provide reliability. The projects listed are included in the City's existing CIP. The other improvements include:

- Water Canyon Pipe Phase 2 (PWP-13)
- Altitude Valves (PWV-1)
- Water Line Replacements (PWRR-2 through PWRR-4)
- Well Enclosures (PWRR-20)
- Well Rehabilitation (PWRR-21)
- Security Cameras at Water Yard (PWO-2)
- Replace SCADA Computer Hardware/Software (PWO-3)
- Work Truck (PWO-4)
- Automatic Meter Reading (AMR) (PWO-5)
- Advanced Metering Infrastructure (AMI) (PWO-6)
- Computer Information System/ERP (PWO-7)

- Chromium 6 Treatment Pilot Study, Design, and Construction (PWO-8)
- Water Master Plan Update (PWO-9)

6.3 Future System Analysis

The goal of the future system analysis is to evaluate the water distribution system under various operating conditions utilizing the evaluation criteria summarized in Chapter 5 and the future demand projections described in Chapter 3. As part of the future system analysis, a preliminary analysis was performed to identify improvements under build-out demand conditions. Since the timing and amount of growth under build-out conditions is unknown, the analysis presented in this chapter will need to be updated in the following master planning update cycle (every 5-10 years) or earlier if critical new information becomes available.

Similar to the existing system analysis, the following analyses were conducted and are described in this section:

- Future Water Supply Analysis
- Future System Pressure Analysis
- Future Pressure Zone Analysis
- Future Pipeline Velocity Analysis
- Future Fire Flow Analysis
- Future Storage Analysis
- Future Pump Station Analysis
- Pipeline Replacement Analysis

The future system analysis was conducted with the water demand projected for year 2040. As listed in Table 6.15, the ADD and MDD projected for year 2040 are 8,411 gpm (or 12.1 mgd) and 14,298 gpm (or 20.6 mgd), respectively.

Due to the Butterfield development, a new pressure zone (Zone 1A) is added into the City's service area. This pressure zone has been included in all the future system analyses. The future demands were added to the existing potable water hydraulic model. It was assumed that all existing system improvements identified in Section 6.2 are installed for the future system analyses described below. It is also assumed that the pressure zone split discussed in Section 6.2.2.1 will be included as part of this phase.

The future demands and the recommended existing system improvements described in the previous section were incorporated into the hydraulic model that was used for the future system analysis and sizing of improvement projects described in the following subsections.

6.3.1 Water Supply Analysis

As previously described in Section 6.2.1, the City's potable water system is solely supplied by groundwater wells. Similar to the existing system water-supply analysis (see 6.2.1), the City's local supply was evaluated under future demand conditions assuming the same two scenarios: largest supply out of service and extreme drought conditions.

As mentioned in 6.2.1, the first scenario was conducted assuming the largest supply was out of service, which reduces the Canyon Wells supply from 3,000 gpm to 2,000 gpm. The second scenario was conducted in the event of extreme drought conditions, which reduces the Canyon Wells supply from 3,000 gpm to 1,700 gpm. Details of this analysis are provided in Appendix E.3. A summary of the analyses results are presented in Table 6.15 and Table 6.16.

Table 6.15 Future (2040) Water Supply Analysis with Largest Well Out of Service

Pressure Zone	Supply Capacity w/ Largest Well o.o.s. (gpm)	Future (2040) MDD[(2)] (gpm)	Future (2040) Capacity Balance (gpm)	Recommendation
Zone 1A	0	315	(315)	None. Pump from Mountain North.
Foothill East	2,000	124	1,876	None.
Foothill West	4,200	2,733	1,467	None.
Mountain North	0	390	(390)	None. Pump from Foothill West.
Mountain South	0	65	(65)	None. PRV from Upper Main.
Upper Main	5,700	5,948	(1,548)	New Well C9, Convert Well M7, Convert Well M12
Lower Main	2,100	3,754	(2,854)	New Well C8, Install VFD on Wells C6 & C8, PRV from Upper Main.
Lower I	0	968	(968)	None. PRV from Lower Main.
Total	**11,500**	**14,298**	**(2,798)**	**N/A**

Notes:
(1) Supply capacities provided by City staff.
(2) MDD peaking factor assumed to be 1.7.
(3) Detailed calculations in Appendix E.3.

Table 6.16 Future (2040) Water Supply Analysis in Extreme Drought Conditions

Pressure Zone	Supply Capacity w/ Largest Well o.o.s. (gpm)	Future (2040) MDD[(2)] (gpm)	Future (2040) Capacity Balance (gpm)	Recommendation
Zone 1A	0	315	(315)	None. Pump from Mountain North.
Foothill East	1,700	124	1,576	None.
Foothill West	4,200	2,733	1,467	None.
Mountain North	0	390	(390)	None. Pump from Foothill West.
Mountain South	0	65	(65)	None. PRV from Upper Main.
Upper Main	5,700	5,948	(1,548)	New Well C9, Convert Well M7, Convert Well M12
Lower Main	2,100	3,754	(3,754)	New Well C8, Install VFD on Wells C6 & C8, PRV from Upper Main.
Lower I	0	968	(968)	None. PRV from Lower Main.
Total	**10,300**	**14,298**	**(3,998)**	**N/A**

Notes:
(1) Supply capacities provided by City staff.
(2) MDD peaking factor assumed to be 1.7.
(3) Detailed calculations in Appendix E.3.

As listed in Table 6.15 and Table 6.16, Foothill East and West are the only zones with excess supply under the both scenarios. While the other pressure zones are all deficient, most of the deficiencies can be resolved by using existing PRVs to convey more water to the lower zones.

For the remaining deficiencies, the same improvements are recommended in both scenarios. Based on the future system supply reliability analysis summarized in Table 6.15 and Table 6.16, the recommendations include:

- New Well C8 (PWW-1 & PWP-1): A new well is proposed to pump into the Main Zone in the near-term (2025) with a capacity of 1,400 gpm. Due to the proposed rezoning of the Main Zone in the long-term (2040), which will be discussed in Section 6.3.2, this well is recommended to be located in the Lower Main Zone. Based on the ratio of existing versus future customer benefit, 100 percent is allocated to future users. The existing system configuration requires a new 12-inch diameter transmission main with a total length of 1,000 feet.
- Convert Well M7 to Potable Water (Project PWW-2): Conversion of the existing Well M7 from non-potable to potable water will add an additional capacity of 350 gpm. However, due to the potential need for treatment, this capacity may decrease and will need to be re-evaluated at that time. Based on the ratio of existing versus future customer benefit, 100 percent is allocated to future users.
- Convert Well M12 to Potable Water (Project PWW-3): Conversion of the existing Well M12 from non-potable to potable water will add an additional capacity of 1,100 gpm. However, due to the potential need for treatment, this capacity may decrease and will need to be re-evaluated at that time. Based on the ratio of existing versus future customer benefit, 100 percent is allocated to future users.
- New Well C9 (Project PWW-4 & PWP-8): A new well is proposed to pump into the Upper Main Zone with a capacity of 1,800 gpm. Based on the ratio of existing versus future customer benefit, 100 percent is allocated to future users. The existing system configuration requires a new 12-inch diameter transmission main with a total length of 1,000 feet.
- VFDs on Wells C6 and C8 (PWPU-4 & PWPU-5): Based on recommendation from City staff, VFDs are recommended on Wells C6 and C8 to provide operational flexibility to pump to the Lower Main Zone. These VFDs would avoid the need of constructing additional pipelines to convey water to the Upper Main Zone and reducing pressure through PRVs to convey water back to the Lower Main Zone. Based on the ratio of existing versus future customer benefit, 100 percent is allocated to future users.

6.3.2 System Pressure Analysis

As part of the system-pressure evaluation, the future distribution system was analyzed with the hydraulic model to identify areas with pressures above 150 psi under MinDD conditions, while MDD conditions were used to identify areas with pressures below 40 psi. It was assumed that new the supply sources, storage reservoirs, and pump stations recommended in Sections 6.3.1, 6.3.5, and 6.3.6 would have been implemented.

6.3.2.1 High Pressures

Since it is assumed that the rezoning will occur in the long-term phase between the years of 2026 and 2040, areas of high pressures are predicted to remain in the southern part of the Main Zone until year 2025 as shown in Figure 6.10.

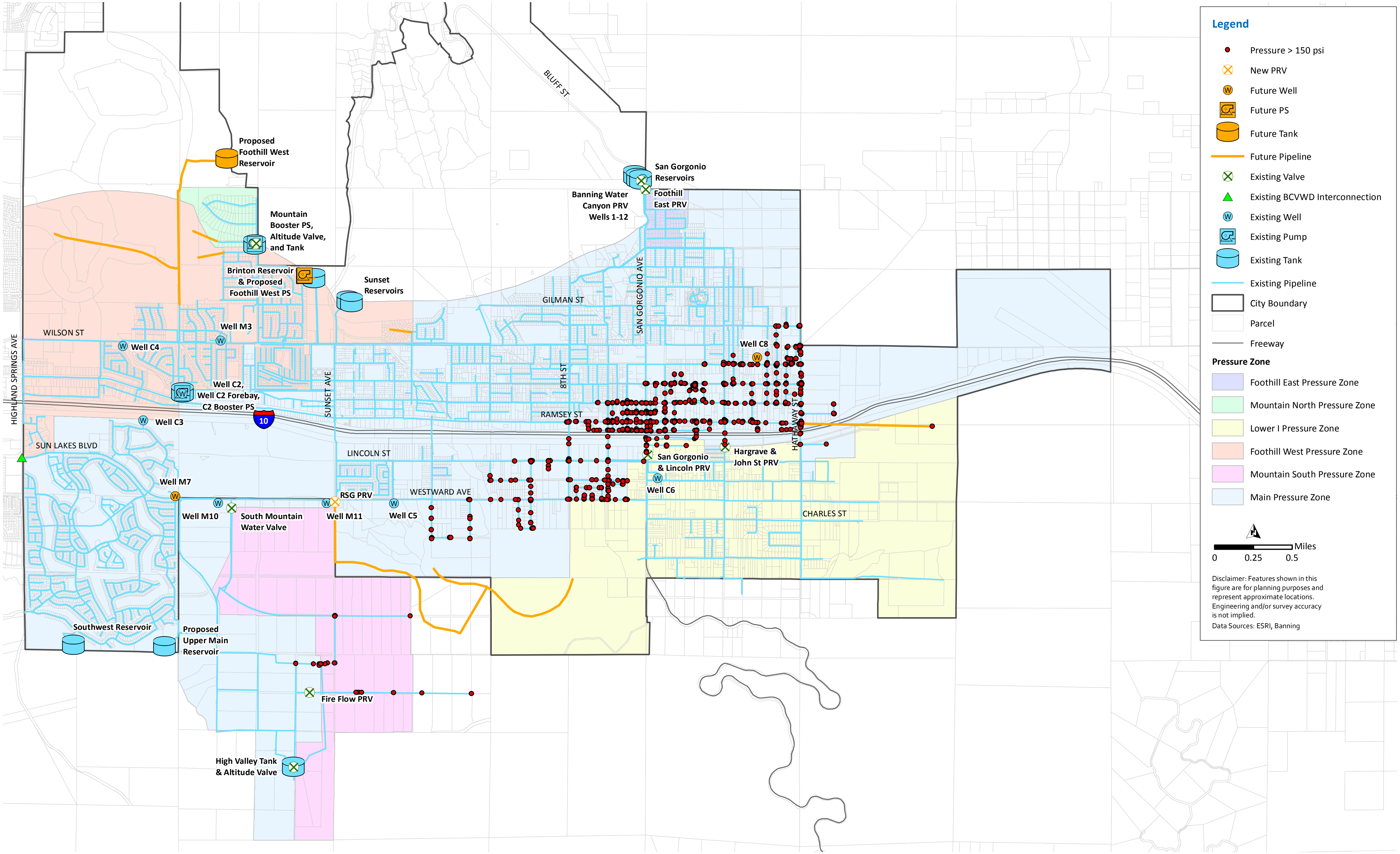


Figure 6.10 Future (2025) System Maximum Pressures without Re-zoning

To mitigate these high pressures, it is recommended that the City proceeds with rezoning of the Main Zone by creating an "Upper Main Zone" and "Lower Main Zone". This rezoning involves the construction of seven (7) PRVs along the new pressure zone boundary, which are summarized in Table 6.17. As mentioned previously, PRVs were constructed in anticipation of the rezoning, but are not currently operational. Based on City staff input, it is recommended that the existing valves be replaced and telemetry be added at each site. The locations of these PRVs, as well as the maximum pressures predicted with the hydraulic model after the rezoning changes are implemented are presented in Table 6.17.

Table 6.17 New PRVs for Rezoning of Main Zone

PRV Name	Size	Setpoint(1) (psi)
4th Street and Wilson Street PRV	1.5 6 12	55 52 49
8th Street and George PRV	4 8	59 53
8th Street and Jacinto View PRV	1.5 6 8	68 65 62
16th Street and Hays PRV	1.5 6 12	56 53 50
San Gorgonio and Gilman PRV	1.5 4 6	47 44 41
Theodore and Almond Way PRV	6 10	48 42
Woodland and Lincoln Street PRV	6 8	58 52

Notes:
(1) Setpoints provided by City staff.

As shown on Table 6.18, the replacement of the seven (7) PRVs (Project PWRZ-1) and creation of a Lower Main Zone mitigates the high pressures. This increases the City's number of pressure zones from six (6) to seven (7) pressure zones. A description of the City's future pressure zone HGLs and demand within each of the zones is presented in Table 6.18.

Table 6.18 Future (2040) Pressure Zones – HGLs and Future Demand Distribution

Name	HGL (ft)	Future ADD (gpm)	Future MDD(2) (gpm)	Percent (%)
Foothill East	2,810	73	124	0.9%
Foothill West	2,822	1,607	2,733	17.1%
Mountain North	2,932	415	705	6.9%
Mountain South	2,546	38	65	0.5%
Upper Main(1)	2,721	3,499	5,948	41.6%
Lower Main(1)	2,560	2,208	3,754	26.3%
Lower I	2,450	570	968	6.7%
Total	**N/A**	**8,411**	**14,298**	**100%**

Notes:
(1) Upper and Lower Main split dependent on locations of the existing PRVs.
(2) MDD PF is assumed to be 1.7.

The Rancho San Gorgonio Development is anticipated to start by year 2025, prior to the re-zoning, and is planned for the Mountain South Zone. Based on the development's specific plan, the development will be connecting into the City's existing Main Zone distribution system at three different locations. One point of connection is planned in the future Upper Main Zone, while two of the connection points are planned in the future Lower Main Zone. However, the HGL from the existing Main Zone and future Upper Main Zone will result in high pressures in the development.

Since the development is anticipated to start construction before the rezoning improvements, the first point of connection will require a PRV to maintain lower pressures than in the existing Main Zone. It is recommended that the development construct the first point of connection on Sunset and Westward, which is connected to the future Upper Main Zone. By doing so, the development can continue using the same PRV once the rezoning occurs. This project will require a new PRV (PWV-2) to regulate the pressure entering the development and incorporate the development into the Mountain South Zone.

As shown in Figure 6.11, the HGL of the Mountain South Zone and Lower Main Zone only differ by 4 feet. It is recommended to conduct a separate analysis to evaluate the possibility of combining these two zones.

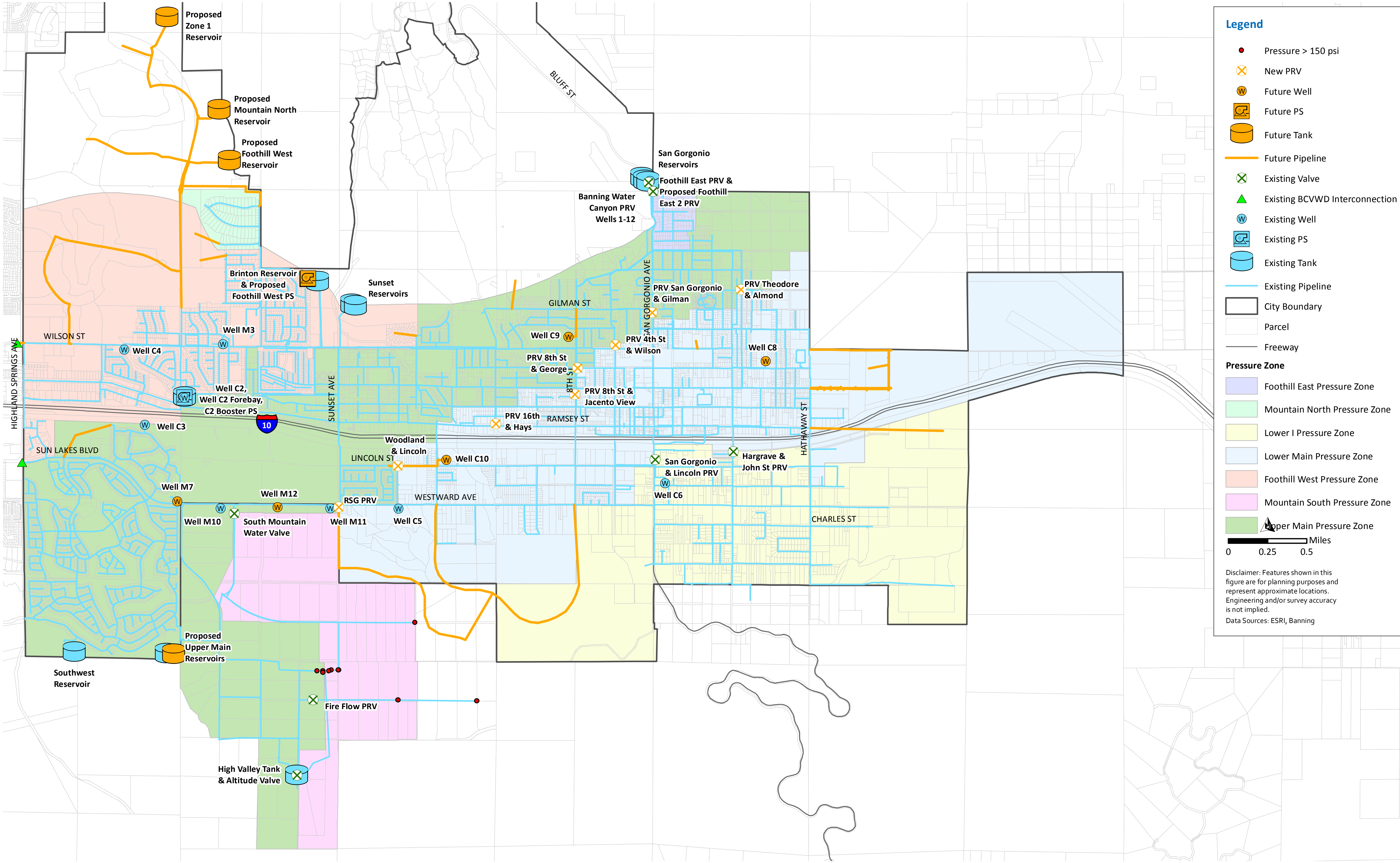


Figure 6.11 Future (2040) System Maximum Pressures with Re-zoning

6.3.2.2 Low Pressures

Based on the modeling analysis under year 2040 MDD conditions, no new low-pressure areas with pressures below 40 psi were identified. The results are presented on Figure 6.12.

6.3.3 Pipeline Velocity Analysis

The hydraulic model was used to evaluate pipeline velocities with future system demands. It was concluded that velocities throughout the distribution system were within an acceptable range below 7 fps. It was assumed that new the supply sources, storage reservoirs, and pump stations recommended in Sections 6.3.1, 6.3.5, and 6.3.6 would have been implemented. The results are presented on Figure 6.13.

6.3.4 Fire Flow Analysis

The hydraulic model was used to evaluate the conveyance capacity of the future distribution system to meet the fire flow requirements listed in Chapter 5 with a minimum residual pressure of 20 psi.

No additional fire-flow deficiencies and improvements were identified, assuming that the City has implemented all existing system fire-flow improvements listed in Table 6.11. It is also assumed that the distribution systems of the future developments, mostly modeled as point demands, will be adequately sized to the land-use-based fire-flow criteria used in the IMP. Hence, no future fire-flow improvements projects are made.

6.3.5 Storage Analysis

A future storage analysis was completed using year 2040 demands and the evaluation criteria listed in Chapter 5. The results of this analysis are summarized in Table 6.19, while details of this analysis are presented in Appendix E.2.

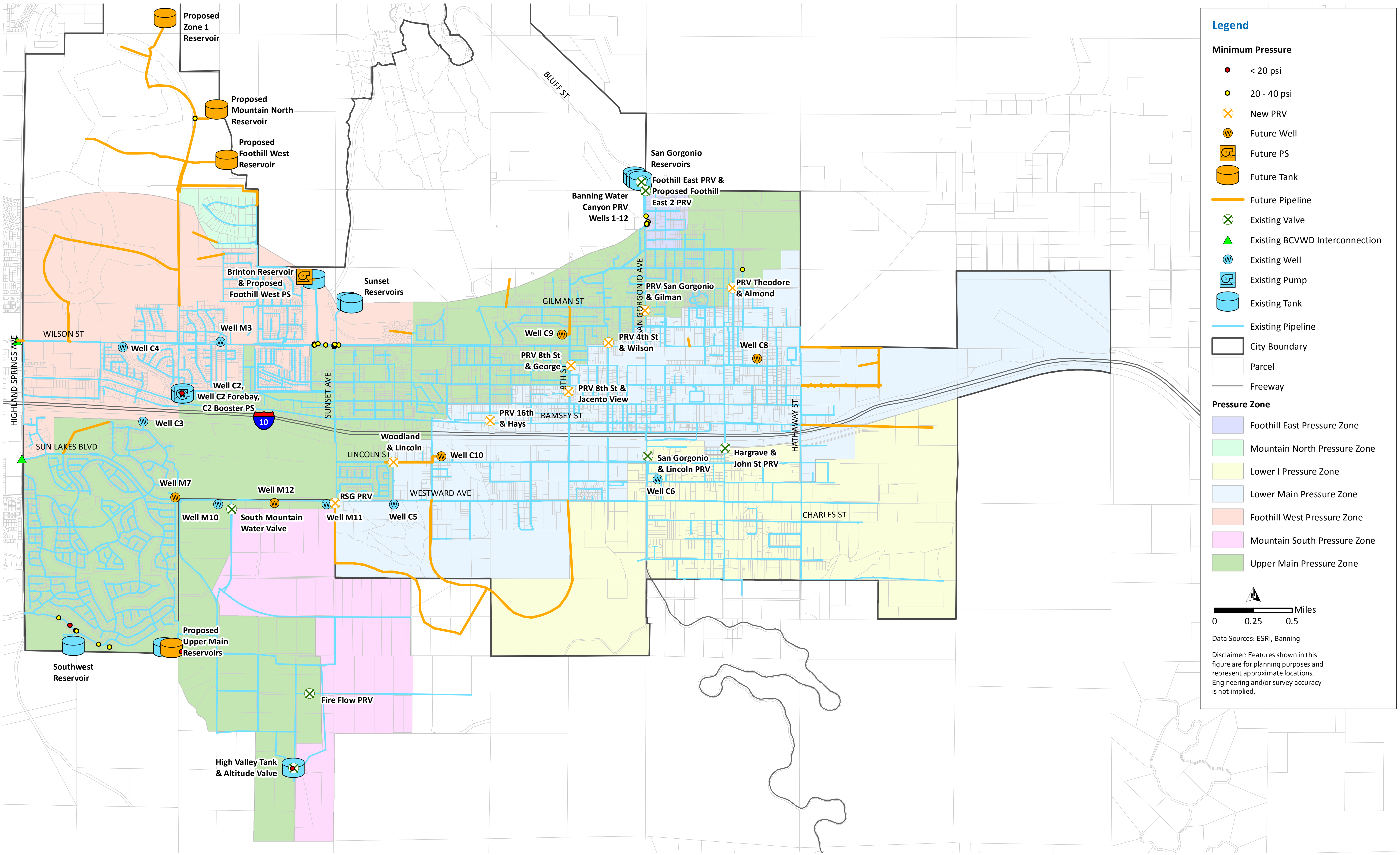
As shown in Table 6.19, after the existing improvements have been completed, the City will have eight (8) reservoirs with 23.3 MG of total storage. Based on the evaluation criteria and projected demands, the total required storage is 30.1 MG, resulting in a deficiency of 6.8 MG.

As shown in Table 6.19, the following storage improvements are recommended with a combined new storage volume of 7.5 MG:

- Upper Main Zone (Project PWS-4 & PWP-10): The Upper Main Zone is in the center of the City's potable water system. With a combined future MDD of 8.56 mgd, the required storage is 11.7 MG. This zone has 23.3 MG of storage available, resulting in a storage surplus of 7.43 MG. However, additional storage is required to accommodate the storage deficiencies in the Foothill East, Mountain South, and Lower I Zones. A new reservoir (Upper Main Reservoir 2) is proposed with a total capacity of 4.0 MG. Based on the ratio of existing versus future customer benefit, 100 percent is allocated to future users. The existing system configuration requires a new 24-inch diameter transmission main with a total length of 500 feet.
- Foothill East Zone: The Foothill East Zone is in the northern part of the City's potable water system. With a combined future MDD of 0.18 mgd, the required storage is 0.4 MG. However, since the zone does not have any storage, it is recommended to mitigate the deficiency by increasing the flow from the Banning Water Canyon Wells

through the existing Foothill East PRV or the new Foothill East 2 PRV. No storage improvements are recommended in this zone.

- Foothill West Zone (Project PWS-2 & PWP-5): The Foothill West Zone is in the northern part of the City's potable water system. Due to the new Butterfield development, the combined future MDD increases from 2.0 mgd to 3.93 mgd, resulting in a required storage of 5.76 MG. However, the zone only has 4.2 MG of available storage. A new reservoir (Foothill West Reservoir 1) is proposed with a total capacity of 1.5 MG. Based on the ratio of existing versus future customer benefit, 100 percent is allocated to future users. The existing system configuration requires a new 18-inch diameter transmission main with a total length of 6,000 feet.
- Mountain North Zone (Project PWS-3 & PWP-9): The Mountain North Zone is in the northwest part of the City's potable water system. Due to the new Butterfield development, the combined future MDD increases to from xxx mgd 0.56 mgd, which results in a required storage of 0.9 MG. However, since the zone does not have any available storage, a new reservoir (Mountain North Reservoir) is proposed with a total capacity of 1.0 MG. Based on the ratio of existing versus future customer benefit, 100 percent is allocated to future users. The existing system configuration requires a new 18-inch diameter transmission main with a total length of 6,500 feet.
- Zone 1A (Project PWS-5 & PWP-12): Zone 1A is a new zone created at the upper part of the Butterfield Development with a future MDD of 0.45 mgd and a required storage of 0.75 MG. Since this is a new zone with no storage, a new reservoir (Zone 1A Reservoir) is proposed with a total capacity of 1.0 MG. Based on the ratio of existing versus future customer benefit, 100 percent is allocated to future users. The existing system configuration requires a new 12-inch diameter transmission main. Since the location of the storage reservoir has not yet been identified by the developer, it was estimated that the total length of pipeline required is approximately 4,500 feet.
- Mountain South Zone: The Mountain South Zone is in the southwest part of the City's potable water system. With a combined MDD of 0.09 mgd, the required storage is 0.30 MG. However, since the zone does not have any storage, it is recommended to mitigate the deficiency by conveying water through the PRV from the Upper Main Zone. No storage improvements are recommended in this zone.
- Lower Main Zone: The Lower Main Zone is in the southern part of the City's potable water system. With a combined MDD of 3.18 mgd, the required storage is 7.72 MG. However, since the rezoning results in all of the existing storage located in the Upper Main Zone, the Lower Main Zone does not have any existing storage. It is recommended to mitigate the deficiency by conveying water through the seven PRVs from the Upper Main Zone. No storage improvements are recommended in this zone.
- Lower I Zone: The Lower I Zone is in the southeast part of the City's potable water system. With a combined MDD of 1.39 mgd, the required storage is 2.70 MG. However, since the zone does not have any storage, it is recommended to mitigate the deficiency by conveying water through the two PRVs from the Lower Main Zone. No storage improvements are recommended in this zone.



Legend
Minimum Pressure
< 20 psi
20 - 40 psi
New PRV
Future Well
Future PS
Future Tank
Future Pipeline
Existing Valve
Existing BCVWD Interconnection
Existing Well
Existing Pump
Existing Tank
Existing Pipeline
City Boundary
Parcel
Freeway
Pressure Zone
Foothill East Pressure Zone
Mountain North Pressure Zone
Lower I Pressure Zone
Lower Main Pressure Zone
Foothill West Pressure Zone
Mountain South Pressure Zone
Upper Main Pressure Zone
Miles
0 0.25 0.5
Data Sources: ESRI, Banning
Disclaimer: Features shown in this figure are for planning purposes and represent approximate locations. Engineering and/or survey accuracy is not implied.
Proposed Zone 1 Reservoir
Proposed Mountain North Reservoir
Proposed Foothill West Reservoir
San Gorgonio Reservoirs
Foothill East PRV & Proposed Foothill East 2 PRV
Banning Water Canyon PRV Wells 1-12
BLUFF ST
Brinton Reservoir & Proposed Foothill West PS
Sunset Reservoirs
GILMAN ST
PRV San Gorgonio & Gilman
PRV Theodore & Almond
SAN GORGONIO AVE
WILSON ST
Well M3
Well C4
Well C9
PRV 4th St & Wilson
Well C8
PRV 8th St & George
HIGHLAND SPRINGS AVE
Well C2, Well C2 Forebay, C2 Booster PS
SUNSET AVE
8TH ST
PRV 8th St & Jacento View
PRV 16th & Hays
RAMSEY ST
10
Well C3
HATHAWAY ST
SUN LAKES BLVD
Woodland & Lincoln
LINCOLN ST
Well C10
San Gorgonio & Lincoln PRV
Hargrave & John St PRV
Well M7
Well M12
RSG PRV
WESTWARD AVE
Well C6
Well M10
South Mountain Water Valve
Well M11
Well C5
CHARLES ST
Proposed Upper Main Reservoirs
Southwest Reservoir
Fire Flow PRV
High Valley Tank & Altitude Valve

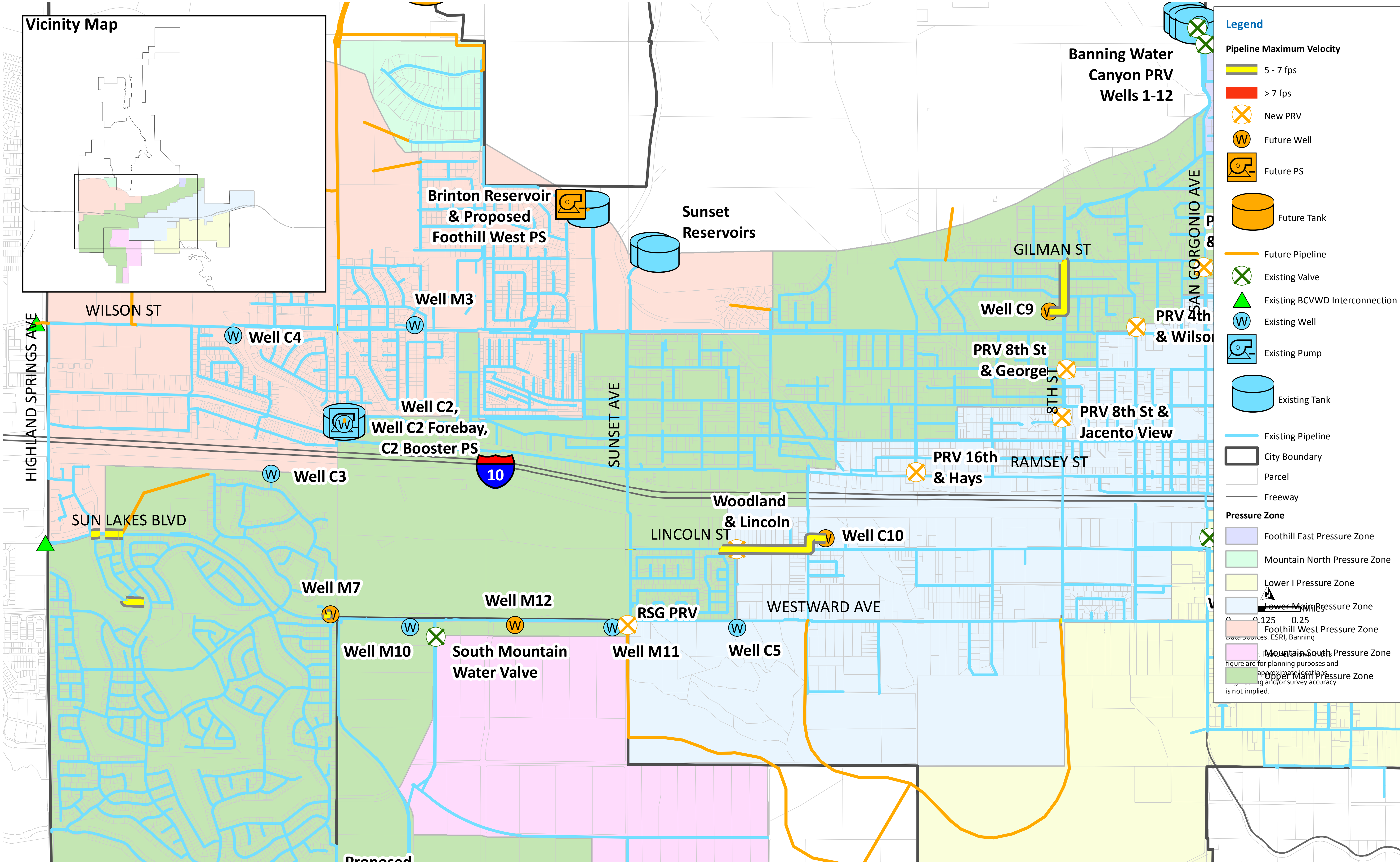


Figure 6.13 Future (2040) System Velocity under MDD Conditions

Table 6.19 Future (2040) Storage Analysis

Zone	Future MDD(1) (mgd)	Required Storage (MG)	Available Storage (MG)	Additional Storage from Existing System Improvement (MG)	Zone Balance (MG)	Proposed Facilities	Proposed Capacity (MG)
Zone 1A	0.45	0.7	0	0	(0.75)	New Zone 1A Reservoir	1.0
Foothill East	0.18	0.40	0	0	(0.40)	None. PRV from Banning Water Canyon	-
Foothill West	3.93	5.76	4.2	0	(1.56)	New Foothill West Reservoir	1.5
Mountain North	0.56	0.9	0	0	(0.88)	New Mountain North Reservoir	1.0
Mountain South	0.09	0.36	0	0	(0.30)	None. PRV from Upper Main Zone	
Upper Main	8.56	11.67	15.1	4.0	7.43	New Upper Main Reservoir 2	4.0
Lower Main	5.41	7.72	0	0	(7.72)	None. PRV from Upper Main Zone	-
Lower I	1.39	2.70	0	0	(2.70)	None. PRV from Lower Main Zone	-
Total	**20.59**	**30.06**	**19.3**	**4.0**	**(6.88)**	**N/A**	**7.5**

Notes:
(1) MDD assumed to be ADD x 1.7.
(2) Reservoir capacities provided by City staff.
(3) Detailed Calculations in Appendix E.2.

6.3.6 Pump Station Analysis

The pump station analysis evaluates the future pump station capacities based on the evaluation criteria listed in Chapter 5. These pump station evaluation criteria define that in zones with gravity storage, the firm capacity of the booster pump station shall be able to supply MDD of the zone it feeds into (including upstream zones. In zones without gravity storage, the firm capacity of the pump station shall be able to supply MDD of the zone it feeds into (including upstream zones), as well as, the maximum fire-flow demand in that zone.

The results of the pump station analysis is summarized in Table 6.20, while the details are presented in Appendix E.4. It was assumed that all existing system improvements identified in Section 6.2 including the rezoning modifications would have been implemented.

As shown in Table 6.20, with the existing system improvements completed, the pump station evaluation demonstrated a total pumping deficiency of 9,263 gpm. The deficiencies and recommended improvements are as follows:

- New/Converted Wells: New wells are recommended to provide supply redundancy. In addition, a few non-potable wells are recommended to be converted for potable water use. All of these wells are also recommended in the supply analysis in Section 6.3.2. This includes the following:
  - New Well C8 (Project PWW-1 & PWP-1): As mentioned in Section 6.3.2, one new well (Well C8) is proposed in the Upper Main Zone with a capacity of 1,400 gpm to increase supply reliability and mitigate pumping deficiencies. Based on the existing configuration, a new 12-inch diameter transmission main with a total length of 1,000 feet is required to connect the well to the distribution system.
  - New Well C9 (Project PWW-4 & PWP-8): As mentioned in Section 6.3.2, one new well (Well C9) is proposed in the Upper Main Zone with a capacity of 1,800 gpm to increase supply reliability and mitigate pumping deficiencies. Based on the existing configuration, a new 12-inch diameter transmission main with a total length of 1,000 feet is required to connect the well to the distribution system.
  - Convert Wells M7 and M12 (Project PWW-2 & PWP-3): As mentioned in Section 6.3.2, two existing non-potable wells (Well M7 and M12) are proposed to be converted to potable water with a capacity of 350 gpm and 1,100 gpm, respectively, to increase supply reliability and mitigate pumping deficiencies. However, due to the potential need for treatment, the capacity of these wells may decrease and will need to be re-evaluated at that time.
  - VFDs on Wells C6 and C8 (PWPU-4 & PWPU-5): As mentioned in Section 6.3.2, VFDs are recommended on Wells C6 and C8 to provide operational flexibility to pump to the Lower Main Zone rather than constructing additional pipelines to convey water to the Upper Main Zone and reducing pressure through PRVs to convey water back to the Lower Main Zone. Based on the ratio of existing versus future customer benefit, 100 percent is allocated to future users.

Table 6.20 Future (2040) Pump Station Analysis

Discharge Pressure Zone	Future MDD(1) (gpm)	Total Required Capacity (gpm)	Existing Firm Capacity (gpm)	Additional Firm Capacity from Existing System Improvements (gpm)	Existing Capacity Balance (gpm)	Proposed Facilities	Proposed PS Capacity (gpm)	Proposed PS Capacity (hp)
Zone 1A	315	315	0	0	(315)	New Zone 1A PS (1 pump @ 400 gpm + 1 SB)	800	50
Foothill East	124	1,624	210	1,800	386			
Foothill West	2,733	3,438	5,050	0	1,612	New Foothill West PS(3) (3 pumps @ 950 gpm + 1 SB)	3,800	200
Mountain North	390	705	400(4)	1,450(4)	920	Abandon Existing Mountain North PS New Mountain Norther 2 PS (1 pump @ 850 gpm + 1 SB)	1,700	80
Mountain South	65	2,065	0	3,100	1,535			
Upper Main	5,948	10,134	6,100	0	(4,034)	New Well C9 Convert Well M7 Convert Well M12 New Well C10	1,800 350 1,100 1,800	
Lower Main	3,754	8,722	17,450	0	8,728	New Well C8	1,400	
Lower I	968	968	6,710	0	1,742			
Total	**14,298**	**20,157**	**35,520**	**4,900**	**(9,263)**	**N/A**	**12,750**	**330**

Notes:
(1) MDD assumed to be ADD x 1.7.
(2) Estimate based on current groundwater level.
(3) New Foothill West PS for redundancy and not capacity related.
(4) Existing Mountain North PS is abandoned in the future.

- New/Upgrade Pump Stations: New pump stations and pump station upgrades are recommended to increase pumping capacity and provide redundancy. This includes the following:
  - New Foothill West PS (PWPU-2): A new Foothill West PS is recommended to provide redundant supply to the Foothill West Zone. Although the Foothill West Zone does not have a pumping deficiency, City staff wanted to provide redundancy by pumping from the Brinton Reservoir site in the Main Zone to the Foothill West Zone via an existing 30-inch diameter pipeline. The new PS will consist of three pumps at 950 gpm each and one stand-by, resulting in a total capacity of 3,800 gpm and a firm capacity of 2,850 gpm. Based on the ratio of existing versus future customer benefit, 100 percent is allocated to future users.
  - New Mountain 2 PS and Demolish Old Mountain PS (PWPU-3 & PWPU-1b & PWP-6): With the increase in demand from the Butterfield Development, a new Mountain 2 PS is recommended to supply the Mountain North Zone. As part of this project, the old Mountain PS will be demolished and the Mountain North PS will be served primarily by the new Mountain 2 PS. Due to the addition of a storage reservoir in this zone as mentioned in Section 6.3.5, the pumping criteria in this zone reduces and no longer needs to meet the capacity of the maximum fire flow requirement. The new PS will consist of one pump at 850 gpm and one stand-by, resulting in a total capacity of 1,700 gpm and a firm capacity of 850 gpm. Based on the ratio of existing versus future customer benefit, 100 percent is allocated to future users. The existing system configuration requires a new 12-inch diameter transmission main with a total length of 3,500 feet.
  - New Zone 1A PS (PWPU-6 & PWP-12): Zone 1A is a new zone created at the upper part of the Butterfield Development. A new Zone 1A PS is recommended to provide sufficient supply to the new Zone 1A Zone, which has a pumping deficiency of 1,558 gpm. The new PS will consist of one 400 gpm pump and one stand-by, resulting in a total capacity of 800 gpm and a firm capacity of 400 gpm Based on the ratio of existing versus future customer benefit, 100 percent is allocated to future users. The existing system configuration requires a new 12-inch diameter transmission main with a total length of 4,500 feet.

6.3.7 Other Improvements

Other miscellaneous improvement projects have been recommended to optimize the operation of the City's potable water system or provide reliability. The projects listed were recommended by City staff. The other improvements include:

- Replace C2 Booster PS pumps 3 and 4 with PRVs (PWV-4)

6.3.8 Build-out

A preliminary analysis was conducted for Build-Out based on information provided at the time of this IMP to identify potential supply, storage, and pump station improvements under Build-Out conditions. Since the timing of growth under Build-Out conditions is unknown, the analysis will need to be updated when additional information is available. Projects identified are summarized in this section, while details are in Appendices E.2, E.3, and E.4.

The following projects were identified as potential Build-Out projects:

- **Supply Improvements**
  - New Well C10 (Project PWW-5 & PWP-14): A new well (Well C10) is proposed in the Upper Main Zone with an assumed capacity of 1,800 gpm. Based on the existing configuration, a new 12-inch diameter transmission main with a total length of 2,000 feet is required to connect the well to the distribution system.
  - New Well C11 (Project PWW-6 & PWP-15): A new well is proposed in the Foothill West Zone with an assumed capacity of 1,800 gpm. Based on the configuration of the existing system, this project will require a new 12-inch diameter pipeline with an estimated length of 1,000 feet.
  - New Well C12 (Project PWW-7 & PWP-16): A new well is proposed in the Upper Main Zone with an assumed capacity of 1,800 gpm. Based on the configuration of the existing system, this project will require a new 12-inch diameter pipeline with an estimated length of 1,000 feet.
- **Storage Improvements**
  - New Foothill West Reservoir 2 (Project PWS-6 & PWP-17): A new 1.5 MG storage tank is recommended in the Foothill West Zone. Based on the configuration of the existing system, this project will require a new 18-inch diameter transmission main with an estimated length of 5,000 feet.
  - New Upper Main Reservoir 3 (Project PWS-7 & PWP-18): A new 9 MG storage tank is recommended in the Upper Main Zone. Based on the configuration of the existing system, this project will require a new 30-inch diameter transmission main with an estimated length of 5,000 feet.
  - New Black Bench Reservoir 1 (Project PWS-8 & PWP-19): A new 1.5 MG storage tank is recommended for the Black Bench Development. Based on the configuration of the existing system, this project will require a new 18-inch diameter transmission main with an estimated length of 5,000 feet.
  - New Loma Linda Reservoir 1 (Project PWS-9 & PWP-20): A new 1.0 MG storage tank is recommended for the Loma Linda Development. Based on the configuration of the existing system, this project will require a new 18-inch diameter transmission main with an estimated length of 5,000 feet.
- **Pump Station Improvements**
  - Upgrade Foothill West PS (Project PWPU-2): Since this pump station serves as a reliability project, it is assumed that up to 600 gpm of the stand-by pump capacity may be utilized under build out conditions.
  - New Loma Linda Pump Station (Project PWPU-7): A new pump station is recommended to supply the future Loma Linda Development. Based on estimated demands presented in Chapter 3, it is proposed to have a firm capacity of 2,700 gpm.
  - New Black Bench Pump Station (Project PWPU-8): A new pump station is recommended to supply the future Black Bench Development. Based on estimated demands presented in Chapter 3, it is proposed to have a firm capacity of 2,700 gpm.

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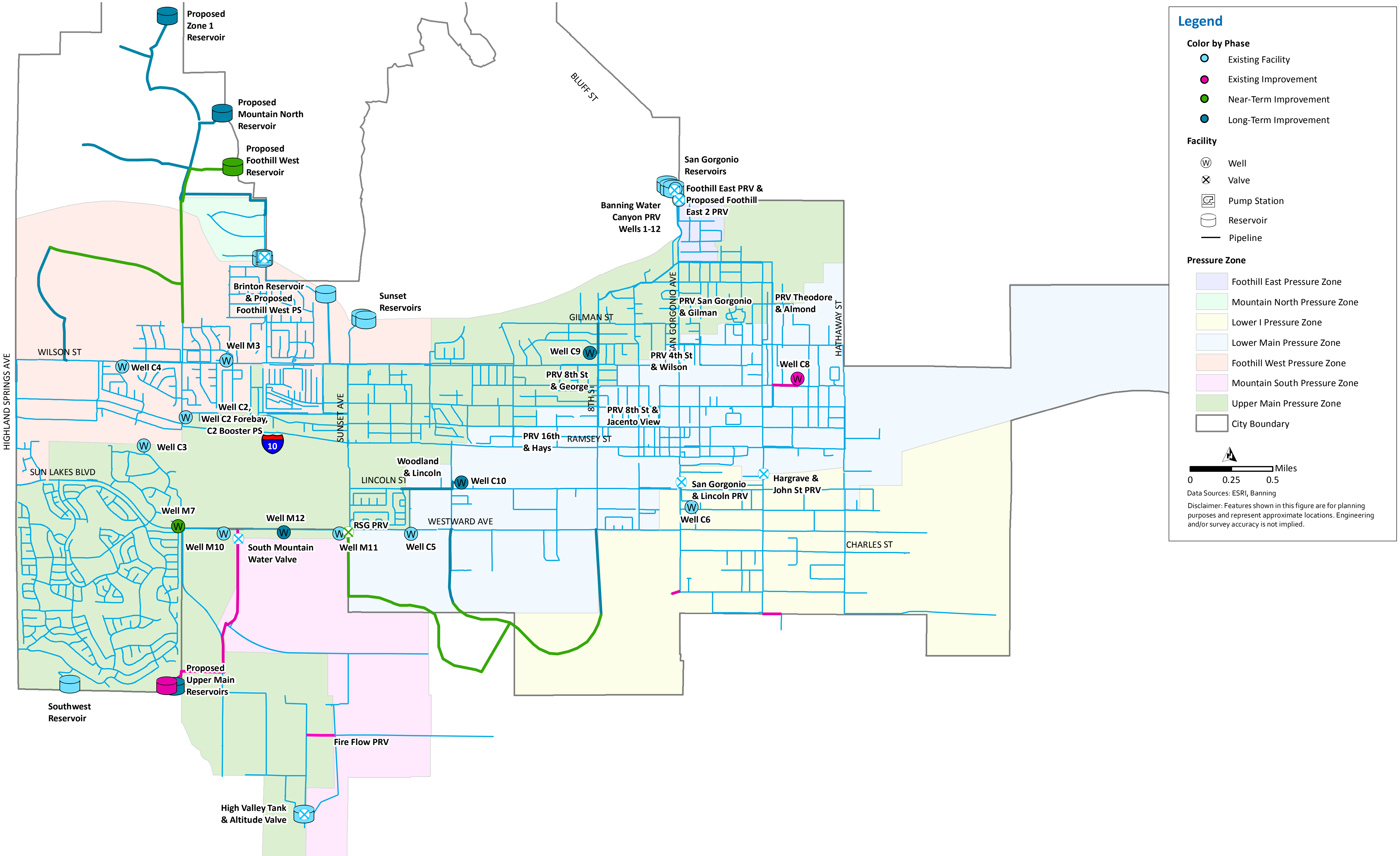

Figure 6.14 Future (2040) System Capacity Improvements

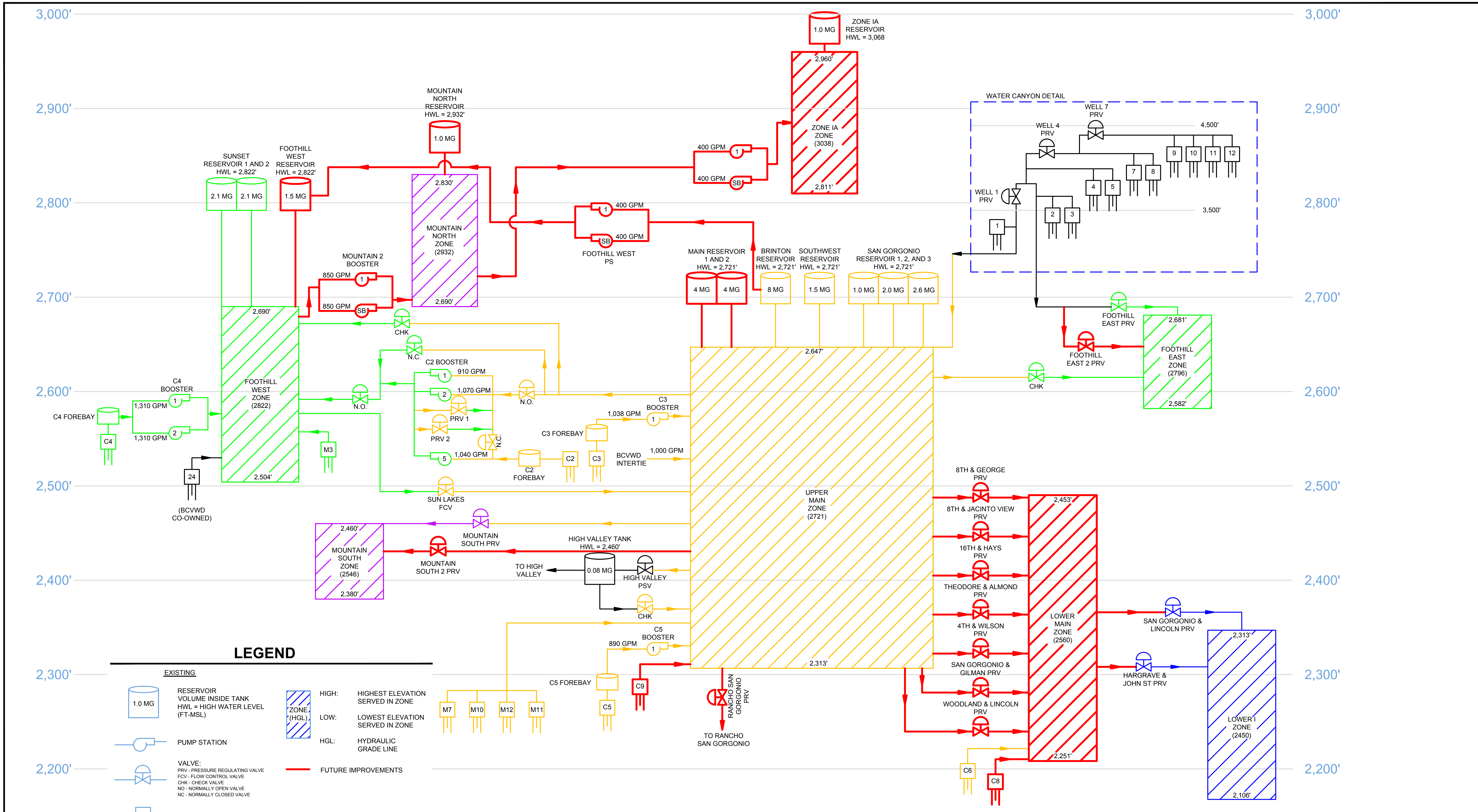

FIGURE 6.15
FUTURE (2040) POTABLE WATER HYDRAULIC PROFILE
CITY OF BANNING

6.4 Proposed Improvements

The recommendations for existing and future conditions identified in this chapter are summarized in this section. Detailed cost estimates for each of these recommendations are included in the CIP of this IMP (see Chapter 9). Based on the analysis of the existing water system under existing and future demand conditions, the following improvements are proposed:

- Supply Improvements
  - **Existing System:** None.
  - **Future System:**
    - New Well C8 in the Main Zone with a capacity of 1,200 gpm and 12-inch diameter transmission main with estimated length of 1,000 feet (Project PWW-1 & PWP-1).
    - Convert Well M7 to potable water in the Upper Main Zone with a capacity of 500 gpm (Project PWW-2).
    - Convert Well M12 to potable water in the Upper Main Zone with a capacity of 1,100 gpm (Project PWW-3).
    - New Well C9 in the Upper Main Zone with an assumed capacity of 1,800 gpm and 12-inch diameter transmission main with estimated length of 1,000 feet (Project PWW-4 & PWP-8).
    - VFDs on Wells C6 and C8 (Project PWPU-4 & PWPU-5).
  - **Build-Out:**
    - New Well C10 in the Upper Main Zone with n assumed a capacity of 1,800 gpm and 12-inch diameter transmission main with estimated length of 2,000 feet (Project PWW-5 & PWP-14).
    - New Well C11 in Foothill West Zone with an assumed capacity of 1,800 gpm (Project PWW-6 & PWP-15).
    - New Well C12 in Upper Main Zone with an assumed capacity of 1,800 gpm and 12-inch diameter transmission main with an estimated length of 1,000 feet (Project PWW-7 & PWP-16)
- Pressure Improvements
  - **Existing System:** None.
  - **Future System:**
    - Re-zoning the Main Zone to the Upper and Lower Main Zones and replace the seven existing PRVs (Project PWRZ-1).
    - Rancho San Gorgonio PRV (Project PWV-2).
  - **Build-Out:** Not evaluated as part of this IMP.
- Fire Flow Improvements
  - **Existing System:** Twenty-three (23) fire flow pipeline projects ranging from 8- to 12-inches in diameter and a total length of 30,000 feet (5.7 miles). One of the projects includes a PRV and check valve. (Projects PWFF 1 through PWFF-28)
  - **Future System:** None.
  - **Build-Out:** Not evaluated as part of this IMP.

- **Storage Improvements**
  - **Existing System:**
    - New Main Reservoir 1 with a proposed capacity of 4 MG and 24-inch diameter transmission main with estimated length of 6,500 feet (Project PWS-1 & PWP-2).
  - **Future System:**
    - New Foothill West Reservoir 1 with a proposed capacity of 1.5 MG and 18-inch diameter transmission main with estimated length of 6,000 feet (Project PWS-2 & PWP-5).
    - New Mountain North Reservoir 1 with proposed capacity of 1.0 MG and 18-inch diameter transmission main with estimated length of 6,500 feet (Project PWS-3 & PWP-9).
    - New Upper Main Reservoir 2 with proposed capacity of 4 MG and 24-inch diameter transmission main with an estimated length of 500 feet (Project PWS-4 & PWP-10).
    - New Zone 1A Reservoir with proposed capacity of 1.0 MG and 12-inch diameter transmission main with an estimated length of 4,500 feet (Project PWS-5 & PWP-12).
  - **Build-Out:**
    - New Foothill West Reservoir 2 with proposed capacity of 1.5 MG and 18-inch diameter transmission main with an estimated length of 5,000 feet (Project PWS-6 & PWP-17).
    - New Upper Main Reservoir 3 with proposed capacity of 9.0 MG and 30-inch diameter transmission main with an estimated length of 5,000 feet (Project PWS-7 & PWP-18).
    - New Black Bench Reservoir 1 with proposed capacity of 1.5 MG and 18-inch diameter transmission main with an estimated length of 5,000 feet (Project PWS-8 & PWP-19).
    - New Loma Linda Reservoir 1 with proposed capacity of 1.5 MG and 18-inch diameter transmission main with an estimated length of 5,000 feet (Project PWS-9 & PWP-20).
- **Pump Station Improvements**
  - **Existing System:**
    - Mountain Booster PS Upgrade with an additional capacity of 1,450 gpm (Project PWPU-1a).
    - New 6-inch diameter Foothill East 2 PRV (Project PWV-3).
  - **Future System:**
    - New Foothill West PS with a total design capacity of 3,800 gpm and a firm capacity of 2,850 gpm (Project PWPU-2).
    - New Mountain 2 PS with a total design capacity of 1,700 gpm and a firm capacity of 850 gpm, as well as a 12-inch diameter pipeline with an estimated length of 3,500 feet. The old Mountain Booster PS will also be demolished (Project PWPU-3, PWPU-1b, and PWP-6).
    - New Zone 1A PS with a design capacity of 800 gpm and a firm capacity of 400 gpm, as well as a 12-inch diameter pipeline with an estimated length of 4,500 feet (Project PWU-6 & PWP-12).

- **Build-Out:**
  - New Loma Linda PS with a design capacity of 800 gpm and a total firm capacity of 400 gpm (Project PWPU-7).
  - New Black Bench PS with a design capacity of 1,600 gpm and a firm capacity of 800 gpm (Project PWPU-8).

- **Repair and Rehabilitation Improvements**
  - A total of approximately 70 miles of pipeline replacement due to estimated useful life (Project PWRR-1).
  - Site Improvements at 5 reservoir sites, 2 PRV stations, and 8 well sites (PWRR-5 through PWRR-19).
  - Multi-Site Projects (PWRR-22).
- **Other Projects**
  - Water Canyon Pipe Phase 2 (PWP-13).
  - Pipeline Replacement Program (PWRR-1).
  - Altitude Valves (PWV-1).
  - Water Line Replacements (PWRR-2 through PWRR-4).
  - Well Enclosures (PWRR-20).
  - Well Rehabilitation (PWRR-21).
  - Pipeline Rehabilitation Asset Study (PWO-1).
  - Security Cameras at Water Yard (PWO-2).
  - Replace SCADA Computer Hardware/Software (PWO-3).
  - Work Truck (PWO-4).
  - Automatic Meter Reading (AMR) (PWO-5).
  - Advanced Metering Infrastructure (AMI) (PWO-6).
  - Computer Information System/ERP (PWO-7).
  - Chromium 6 Treatment Pilot Study, Design, and Construction (PWO-8).
  - Water Master Plan Update (PWO-9).
  - Replace C2 Booster PS pumps 3 and 4 with PRVs (PWV-4)

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Chapter 7

WASTEWATER COLLECTION SYSTEM EVALUATION

This chapter presents an overview of the City of Banning's (City) existing and future wastewater collection system. In this chapter, the existing and future wastewater collection systems are evaluated under various operating conditions utilizing the evaluation criteria described in Chapter 5 and the flow conditions listed in Chapter 3.

This chapter is divided into the following sections:

- Existing Wastewater Collection System: This section provides an overview of the City's existing wastewater collection system facilities.
- Existing Collection System Analysis: This section presents the findings and improvement recommendations for the wastewater collection system under existing flow conditions.
- Future Collection System Analysis: This section presents the findings and improvement recommendations for the wastewater collection system under future flow conditions with the existing system recommendations in place. An alternative analysis was performed to review the system impacts with the addition of a Satellite Treatment Plant for the Butterfield development.
- Summary of Recommendations: This section summarizes the recommended improvements, which are prioritized and phased in Capital Improvement Program (CIP) described in Chapter 9 of this Integrated Master Plan (IMP).

7.1 Existing Wastewater Collection System

The City's wastewater collection system consists of gravity sewers, lift stations, and force mains that collect and convey wastewater. Figure 7.1 presents the City's existing wastewater collection system.

7.1.1 Wastewater Treatment Facility

All wastewater flows collected within the City's service area are currently treated at one facility, the Banning WWTP. As shown on Figure 7.1, the plant is located in the southeast portion of the City adjacent to Smith Creek and east of Hathaway Street. The City contracts with United Water Services for the operation and maintenance of the WWTP. The WWTP is designed to treat wastewater to secondary standards and consists of the following processes: headworks, screening, grit removal, two primary clarifiers, two trickling filters, and two secondary clarifiers.

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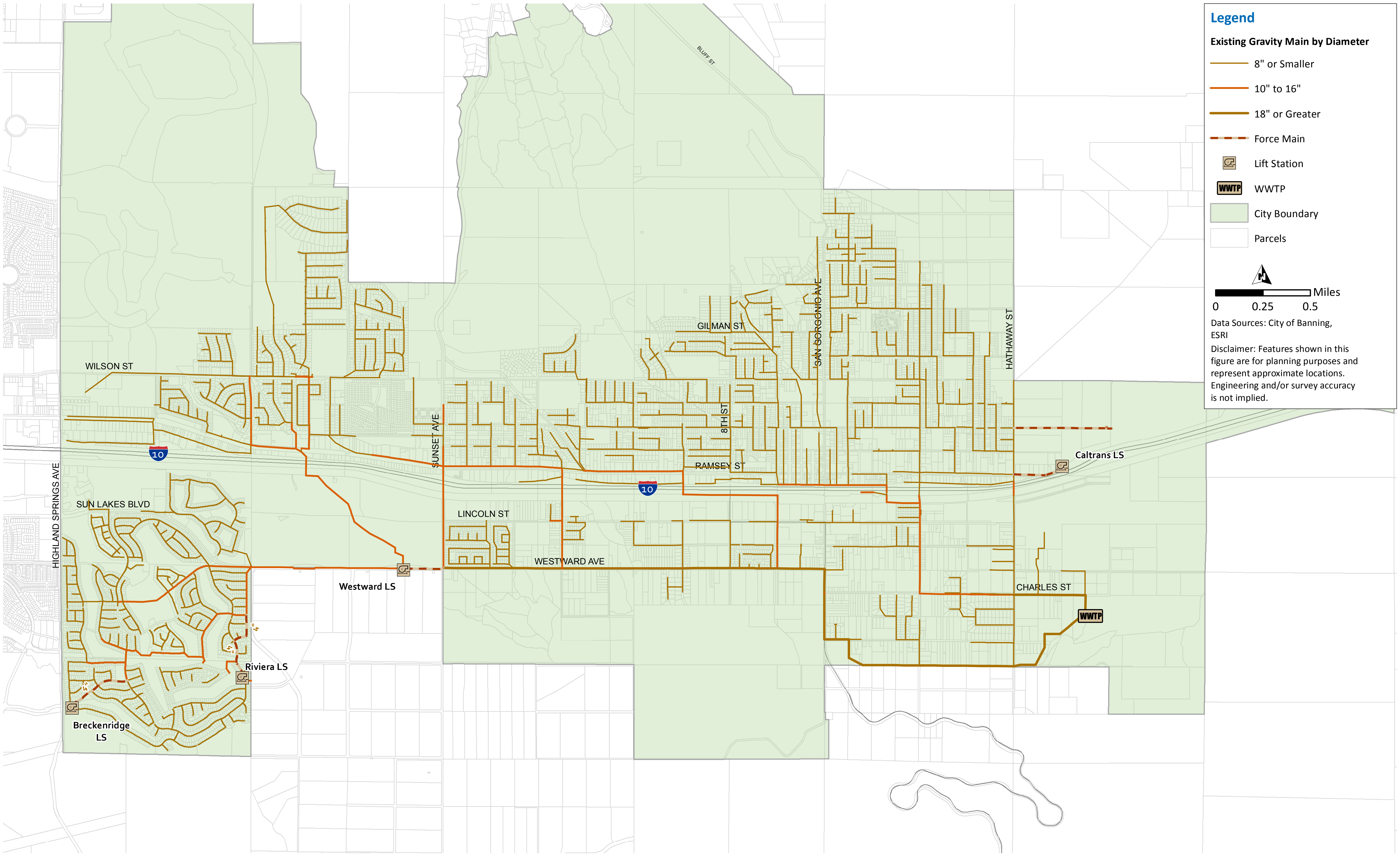

Figure 7.1 Existing Wastewater Collection System

The solids treatment at the Banning WWTP consists of a gravity thickener, two primary digesters, a secondary digester, sludge drying beds, and percolation ponds. Anaerobic digesters and sludge drying beds are used for sludge stabilization and dewatering. The plant currently discharges the effluent to percolation ponds. Solids are periodically removed from the drying beds and hauled off by a private contractor. Once off site, the sludge is disposed of by a reputable solids hauler at a designated landfill site.

The City has plans to upgrade the existing WWTP treatment to meet tertiary standards and facilitate infrastructure to supply recycled water. The design of the upgraded WWTP will allow for expansion of the treatment capacity when it becomes necessary.

7.1.2 Gravity Wastewater Collection System

The existing wastewater collection system consists of approximately 112 miles of sanitary sewer pipelines ranging in diameter from 4 inches to 30 inches, as well as 4 active wastewater lift stations. Figure 7.1 shows the City's existing collection system.

7.1.2.1 Pipeline Distribution by Diameter

Table 7.1 summarizes the total length of pipeline for each diameter in the domestic collection system. The table is based on geographic information system (GIS) data provided by City staff. The table excludes private sewer pipelines within the study area and does not account for pipelines within the WWTP, which range from 4-inch to 36-inch diameter. Figure 7.2 illustrates the distribution of pipeline diameters. As listed, approximately 78-percent of the City's gravity sewers are 8-inches in diameter.

Table 7.1 Pipeline Diameter Overview

Diameter	Length (ft)	Length (mi)	Percent (%)
4(1)	5,400	1.0	0.9%
6(2)	33,800	6.4	5.7%
8	465,200	87.9	78.6%
10(3)	8,800	1.7	1.5%
12	22,300	4.2	3.8%
15	28,900	5.5	4.9%
18	6,300	1.2	1.1%
21	17,200	3.3	2.9%
24	600	0.1	0.1%
30	2,300	0.4	0.4%
Total	**589,800**	**112**	**100.0%**

Notes:
(1) Force main length equals approximately 5,400 feet.
(2) Force main length equals approximately 2,800 feet
(3) Force main length equals approximately 1,200 feet.

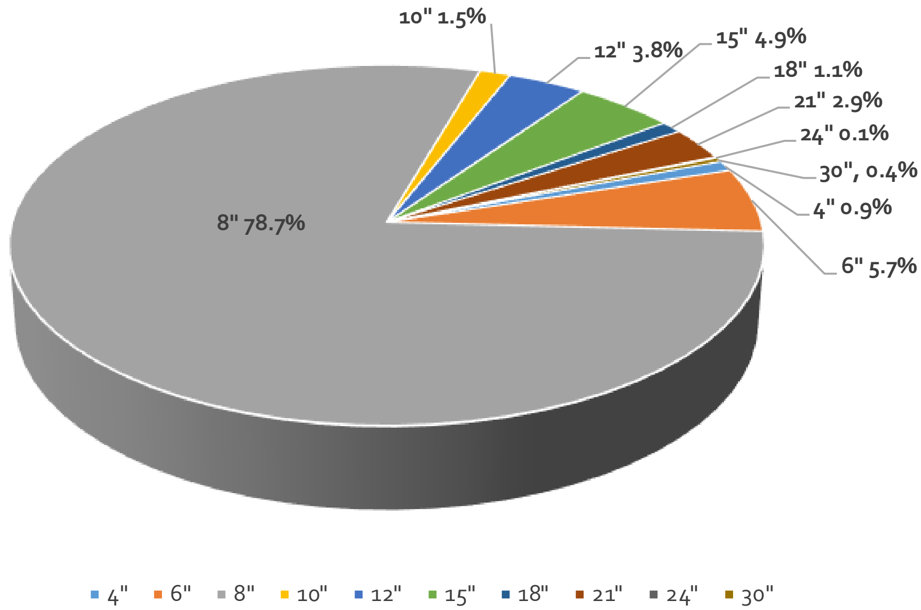

Figure 7.2 Pipelines by Diameter

7.1.2.2 Gravity Sewer Distribution by Material

The distribution of pipeline by material is graphically presented on Figure 7.3 and summarized in Table 7.2. The material categories are Standard Dimension Ratio 35 (SDR 35), Asbestos Cement (ACP), Ductile Iron (DIP), polyvinyl chloride (PVC), Vitrified Clay Pipe (VCP), and unknown materials (UNK). The vast majority of the pipelines (74-percent) are VCP, followed by SDR 35 (22-percent) which occurs primarily in the Sun Lakes Development.

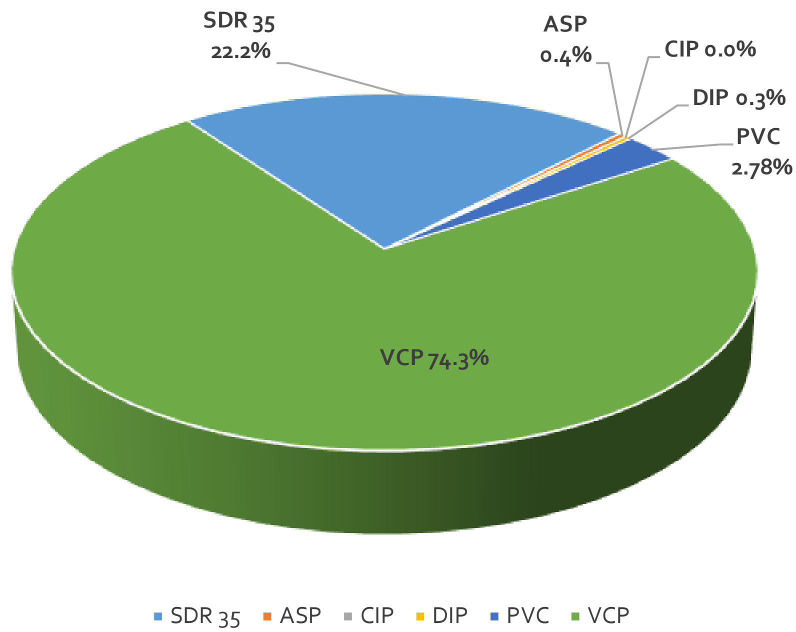

Figure 7.3 Pipelines by Material Type

Table 7.2 Pipeline Material Overview

	Pipeline Length by Material (feet)						Total	
Diameter (in)	Standard Dimension 35 (SDR 35)	Asbestos Cement (ACP)	Cast Iron (CIP)	Ductile Iron (DIP)	Polyvinyl Chloride (PVC)	Vitrified Clay (VCP)	(feet)	(%)
4	0			1,400	4,000	0	5,400	0.92%
6	200	0	0	0	2,912	30,700	33,800	5.73%
8	106,600	2,300	40	500	8,300	346,400	464,100	78.69%
10	3,900	0	0	0	1,200	3,700	8,800	1.49%
12	10,800	0	0	0	0	11,500	22,300	3.78%
15	9,500	0	0	0	0	19,500	29,000	4.92%
18	0	0	0	0	0	6,300	6,300	1.07%
21	0	0	0	0	0	17,200	17,200	2.92%
24	0	0	0	0	0	600	600	0.10%
30	0	0	0	0	0	2,300	2,300	0.39%
Total (feet)	131,000	2,300	0	1,900	16,400	438,200	589,800	-
Total (miles)	24.8	0.4	0.0	0.4	3.1	83.0	112	-
Percent of total	22.1%	0.4%	0.0%	0.3%	2.8%	74.3%	100.00%	100.00%

7.1.2.1 Gravity Distribution by Age

The distribution of gravity pipeline by age is graphically presented in Figure 7.4 and summarized in Table 7.3. As shown in Figure 7.4, approximately 50-percent of the collection system's age is unknown. A majority of the collection system pipelines with age related information were installed between 1980 and 2000. During this timeline, VCP was the common pipeline installed followed by SDR 35.

The pipeline age summary is a combination of installation dates and approved dates. To further expand on probable installation dates, the City may utilize upstream and downstream pipelines with known dates or review nearby utilities such as water lines to get an approximate timeline.

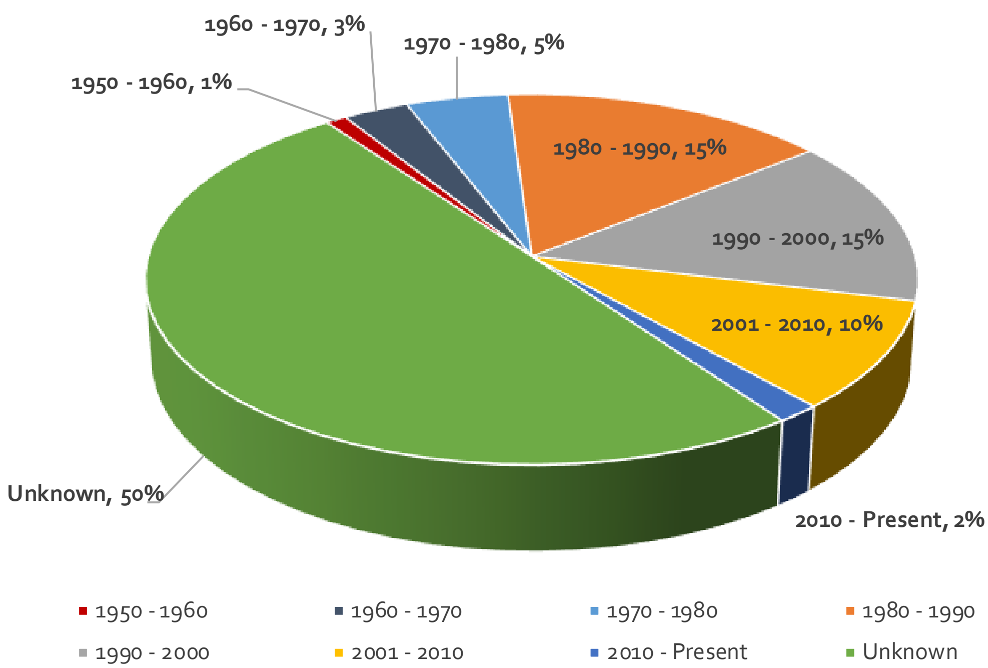

Figure 7.4 Pipelines by Age

7.1.2.2 CCTV Program

Asset management of buried infrastructure consists of two primary areas: 1) operation and maintenance activities, and 2) rehabilitation and replacement actives. Inspections, repairs, and preventative maintenance efforts aim to optimize the useful life of pipelines and appurtenances.

Table 7.3 Pipeline Age Overview

	Pipeline Length by Estimated Installation Year (feet)								Total	
Material	1950-1960	1960-1970	1970-1980	1980-1990	1990-2000	2000-2010	2010-Present	Unknown	(feet)	(%)
SDR 35	-	-	-	55,600	47,500	24,800	-	3,200	131,100	22.1%
ACP	-	-	-	-	-	-	-	2,300	2,300	0.4%
CIP	-	-	-	-	-	40	-	-	40	0.0%
DIP	-	-	-	1,600	50	-	-	300	1,900	0.3%
PVC	-	-	-	9,600	2,100	1,900	2,800	-	16,400	2.8%
VCP	5,800	18,300	28,300	24,100	32,000	35,200	7,700	289,400	438,100	74.3%
Total (feet)	5,800	18,300	28,300	90,900	81,650	61,940	10,500	295,200	589,800	100.0%
Total (miles)	1	3	4	17	16	11	2	56	112	-
Percent of total	1%	3%	5%	15%	15%	10%	2%	50%	100%	-

The City has a closed circuit television (CCTV) inspection program. According to the City's GIS database, approximately 33 miles of sewer pipeline have been inspected to date. This equated to approximately 30-percent of the City's collection system. Depending on the CCTV findings, the City uses a scoring system that categorizes pipelines based on service and structural conditions. Currently, the City uses video inspection as part of its Fats, Oil, and Grease (FOG) program, inspection of new construction, and routine inspections.

The City may consider expanding the use of the CCTV program to inspect critical pipelines with unknown installation dates, and pipelines approaching their useful life. An age-based analysis can be performed to provide a statistical evaluation of decay and potential failure of pipelines based on material. This type of analysis typically uses assumed "useful life" values, which are based on industry literature. In conjunction with an age-based analysis, the CCTV program can be used to correlate the actual conditions of pipelines approaching their assumed useful life.

7.1.2.3 Lift Stations

The City owns and operates four (4) lift stations that pump wastewater from low points in the collection system to manholes at higher elevation. Table 7.4 summarizes the characteristics of each active lift station. As shown, the City's lift stations have firm capacities that range from 0.3 mgd to 2.88 mgd. Each of the lift stations includes one duty pump and one standby pump, with the exception of Westward Lift Station, which has two duty pumps and one standby pump.

7.2 Existing Sewer System Analysis

The goal of the existing sewer system analysis is to evaluate the system under various operating conditions utilizing the evaluation criteria described in Chapter 5 and the existing flows listed in Chapter 3. The evaluation identified areas in the sewer system where pipeline capacity was inadequate to convey design flows. Sewers that lack sufficient capacity create bottlenecks in the sewer and potentially contribute to sanitary sewer overflows (SSOs).

The City's sewer system was evaluated with a hydraulic computer model, which provides a platform for effectively managing and identifying capacity deficiencies within the sewer system. Using the model, an analysis was performed on over 100 miles of pipeline.

The following analyses are described in this section:

1. Gravity System Evaluation
2. Lift Station and Force Main Evaluation
3. Rehabilitation and Replacement Improvements
4. Condition Assessment
5. Treatment Plant Improvements
6. Other Improvements

7.2.1 Gravity System Evaluation

For the existing sewer collection system, the peak wet weather flow (PWWF) was routed through the hydraulic model. In accordance with the established flow depth criteria for existing sewers, pipelines with a maximum flow depth to pipe diameter (d/D) ratio greater than 0.92 were identified as capacity deficient.

It is important to understand that not all of the existing pipelines with a d/D greater than 0.92 are necessarily capacity deficient. In some cases, a surcharged condition within a given pipeline segment is due to backwater effects created by a downstream bottleneck (i.e., upstream surcharging is caused by downstream pipeline deficiencies). An illustration of backwater effects is shown on Figure 7.5. For this reason, the hydraulic model was analyzed to identify the pipeline segments that are the cause of the surcharged conditions. These capacity deficient sewers are shown on Figure 7.6.

Following the completion of the existing system analysis, improvement projects and alternatives were identified to mitigate pipeline capacity deficiencies while maintaining a maximum d/D for new sewers (0.67 for pipes 12" and smaller, 0.75 for pipes 15" and larger). These sewers will need to be replaced by larger-diameter sewers or constructed in parallel to bypass flow around hydraulically deficient sewers. The decision on whether to upsize or parallel a particular sewer should be confirmed during the preliminary design of each proposed project and is based on a number of factors, including the condition of the existing pipeline, pipeline velocities during dry-weather flow conditions, pipeline slopes, and other relevant factors. The proposed improvements to address existing deficiencies are shown on Figure 7.7 The recommended projects range in size from 10 inches to 21 inches in diameter and include adding a parallel sewer to distribute flow directly upstream of the WWTP. The upgraded pipelines generally followed the same slope as the existing pipeline. The following summarizes the purpose and locations of existing facilities that would need to be replaced or paralleled in order to address existing system deficiencies.

- Williams Street Sewer (Project WWGM-1): This project will replace approximately 1,000 feet of 8-inch diameter pipeline located in Williams street, between Allen and Hathaway Street. The flow levels within the gravity sewer cause the existing pipeline to surcharge under PWWF, exceeding the maximum d/D criteria. To mitigate the risk of SSO occurring during PWWF conditions, it is recommended that the existing pipeline be replaced with a 10-inch diameter pipeline.
- North Hathaway Street Trunk (Project WWGM-2): This project will replace approximately 1,000 feet of 8-inch diameter pipeline located in Hathaway Street, between Williams Street and Interstate 10. To mitigate existing deficiencies, it is recommended that the existing pipeline be replaced with a 12-inch diameter pipeline.
- Casing under Interstate 10 (Project WWGM-3A): This project requires the replacement of approximately 500 feet of 8-inch diameter pipeline located under Interstate 10, extending from Hathaway Street. To mitigate existing deficiencies, it is recommended that the existing pipeline be replaced with a 15-inch diameter pipeline. This segment will also require a 30-inch diameter steel casing.
- South Hathaway Street Trunk (Project WWGM-3B): This project will replace approximately 3,000 feet of 8-inch diameter pipeline located in Hathaway Street and extends from Interstate 10 to Charles Street. To mitigate existing deficiencies, it is recommended that the existing pipeline be replaced with a 15-inch diameter pipeline.
- Ramsey Street Sewer (project WWGM-4): This project will replace approximately 1,000 feet of 8-inch diameter pipeline located in Ramsey Street and extends east of Phillips Street to Hathaway. To mitigate existing deficiencies, it is recommended that the existing pipeline be replaced with a 12-inch diameter pipeline.
- Charles Street Trunk (Project WWGM-5): This project will replace approximately 1,000 feet of 18-inch diameter pipeline located in Charles Street, east of Hathaway Street. To mitigate existing deficiencies, it is recommended that the existing pipeline be replaced with a 21-inch diameter pipeline. The upstream and downstream pipelines have steeper slopes and are not capacity deficient under existing PWWF conditions.

Table 7.4 Lift Station Information

Lift Station Name	Location	Pump Data							Force Main Data	
		Installation Date	Pump No.	Design Head (feet)	Capacity, Per Pump (gpm)	Capacity, Per Pump (mgd)	Firm Capacity (mgd)	Total Capacity (mgd)	Diameter (in)	Length (feet)
Riviera LS	Southwest of Riviera Ave. And Crenshaw Cir.	1998	1	99	210	0.30	0.30	0.60	4	2,100
		1998	2	99	210	0.30				
Breckenridge LS	South of Breckenridge Ave. and Myrtle Beach Dr.	2003	1	59	135	0.19	0.19	0.38	4	1,900
		2003	2	59	135	0.19				
Caltrans LS	West of Sunset Ave. and Westward Ave.	1998	1	50	180	0.26	0.26	0.52	4	1,400
		1998	2	50	180	0.26				
Westward LS	East of Hathaway St. and Westward Ave.	2011	1	41	1,100	1.58	2.88	4.46	10	1,100
		2003	2	38	1,100	1.58				
		2001	3	38	900	1.30				

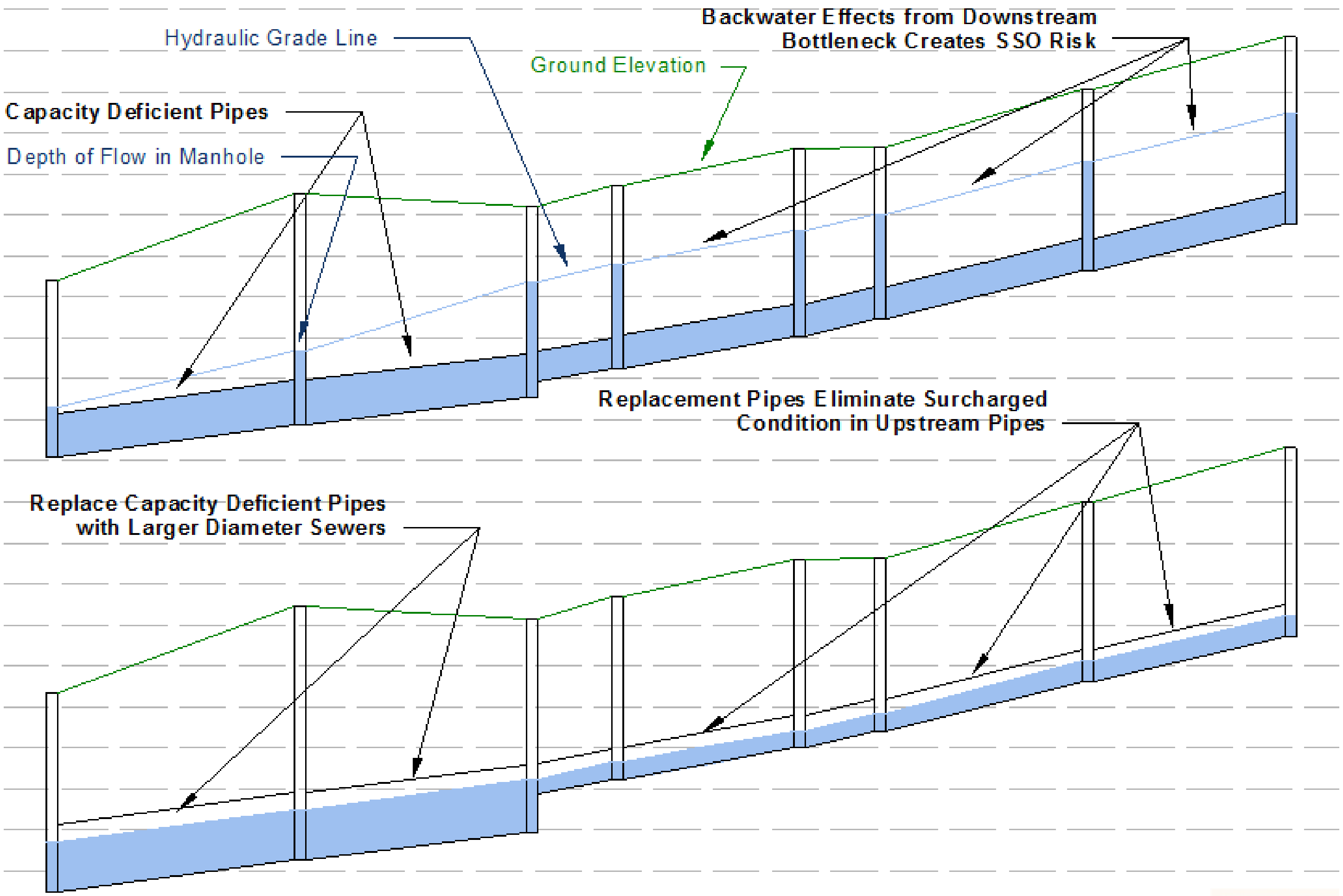

Figure 7.5 Sample Illustration of Back Water Effects in a Sewer

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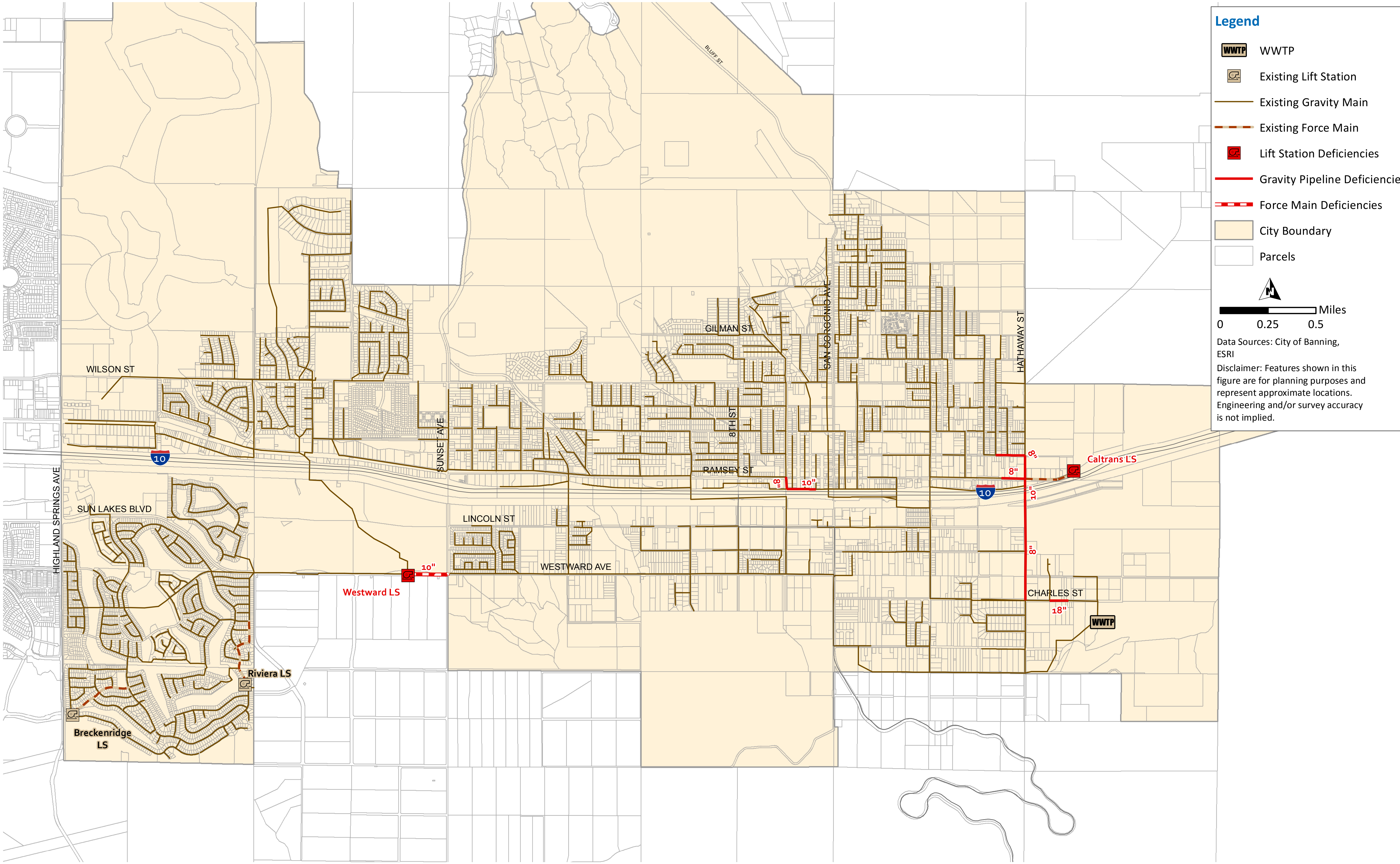

Figure 7.6 Existing System Deficiencies

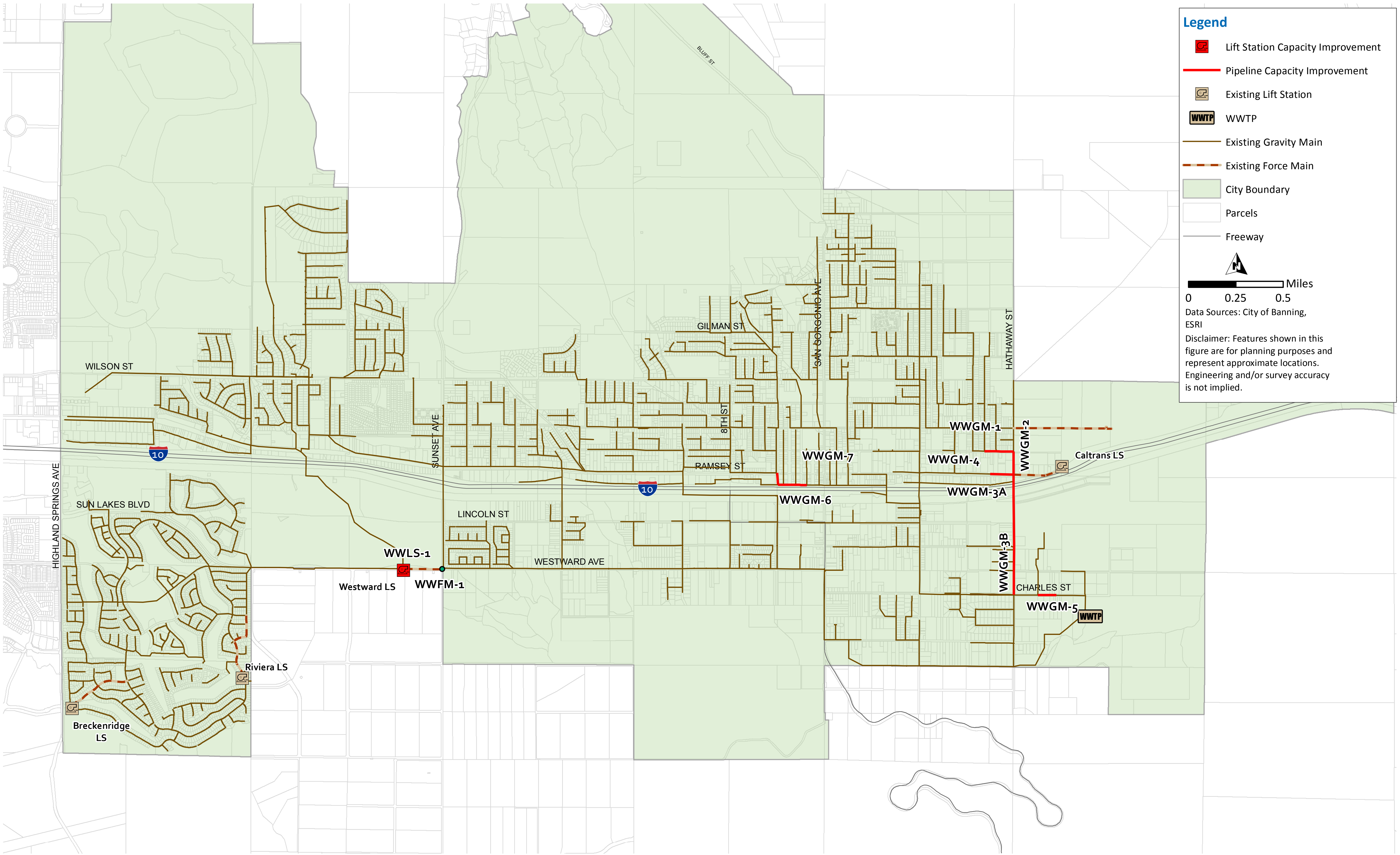

Figure 7.7 Existing System Improvements

- Livingston Street Sewer (Project WWGM-6): This project will replace approximately 1,000 feet of 10-inch diameter pipeline located in Livingston Street, between Fourth Street and Second Street. To mitigate existing deficiencies, it is recommended that the existing pipeline be replaced with a 12-inch diameter pipeline.
- Fourth Street Sewer (Project WWGM-7): This project will replace approximately 500 feet of 8-inch diameter pipeline located in Fourth Street, from Ramsey to Livingston Street. To mitigate existing deficiencies, it is recommended that the existing pipeline be replaced with a 12-inch diameter pipeline.

7.2.2 Lift Station and Force Main Evaluation

The City's hydraulic model includes each of the four (4) operational lift stations. The modeled lift stations were evaluated to determine if they have sufficient capacity to convey existing PWWFs. Lift Stations with an influent PWWF above the firm capacity were flagged as deficient. Table 7.5 summarizes the results of the lift station evaluation.

Table 7.5 Lift Station Capacity Evaluation

Lift Station	Firm Capacity (mgd)	Total Capacity (mgd)	Existing PWWF (mgd)	2025 PWWF (mgd)	2040 PWWF (mgd)	Build-Out PWWF (mgd)	Capacity Deficient?
Riviera	0.30	0.60	0.28	0.28	0.29	0.29	No
Breckenridge	0.19	0.38	0.13	0.13	0.13	0.13	No
Caltrans	0.26	0.52	0.21	0.22	0.23	0.23	No
Westward	2.88	4.46	4.01	4.24(1)(2)	4.30(1)(2)	4.35(1)(2)	Yes

(1) Notes:
(2) The Westward Lift Station is recommended to be abandoned. This table shows the estimated flows that are expected to enter the lift station if it remains active.
(3) Flows do not include Five Bridges development.

As listed in Table 7.5, the Westward Lift Station was flagged as deficient under existing PWWF conditions. With existing capacity improvements implemented, the Westward lift station has an insufficient firm capacity to convey existing PWWFs of approximately 4.0 mgd. To mitigate the existing system deficiency, the following is recommended:

- Westward Lift Station Interim Upgrade (Project WWLS-1, WWFM-1): The Westward Lift Station was identified as capacity deficient under existing conditions. This project (WWLS-1) would upsize the existing lift station and force main (WWFM-1) until Five Bridges and RSG are developed. When the two developments are constructed, Project WWGM-16 is considered viable.

  For Westward Lift Station, the existing pumps are recommended to be replaced with larger pumps to increase the firm capacity from 2.88 mgd to 4.4 mgd. Approximately 1,500 feet of existing 10-inch diameter force main will be replaced with a 12-inch force main.

  The Westward Lift Station upgrades are recommended to mitigate an existing deficiency and sized for future flows from infill and new development. Wastewater flows from Five Bridges or other major communities such as Butterfield were not considered in development of this project. If flows from these major communities are routed to the Westward Lift Station, the required pump station and force main size will need to be re-evaluated.

- Westward Lift Station Bypass (Project WWGM-16): Based on the previous Master Plan, the Westward lift Station was recommended to be abandoned when the Rancho San Gorgonio (RSG) development comes online. Based on this assumption, the project recommendation includes approximately 2,000 feet of 18-inch diameter pipeline that would bypass the lift station with a new gravity main. The project extends southeast from Westward Avenue to Sunset Avenue, through the proposed Five Bridges development and connects to Rancho San Gorgonio's proposed sewer main. Because this project depends on coordination with the Five Bridges and RSG developments, it is not considered immediately feasible and is not shown in Figure 7.7, Existing System Improvements, but instead is shown in Figure 7.8, Future System Improvements.

7.2.3 Rehabilitation and Replacement Improvements

The City's annual sewer pipeline replacement program is a City wide initiative to replace or repair aging sewer infrastructure. These projects are determined on an annual basis and are considered preventative maintenance. Typical methods for replacement and repair include cured-in-place pipe (CIPP) liner or pipe bursting. Costs associated with the annual sewer replacement program cover a range of techniques used in the pipeline rehabilitation industry today and are supplemented with cost contingencies. Since an aged based analysis was not included in the IMP, an Annual Sewer Replacement Program (Project WWRR-1) was recommended.

7.2.4 Condition Assessment

A condition assessment was completed for two lift stations as part of the IMP. The condition assessment was conducted on June 7, 2017. The assessment consisted of visual inspection of mechanical, structural, and electrical equipment. The two lift stations evaluated in the condition assessment included the Caltrans Lift Station and the Westward Lift Station.

The purpose of the lift station condition assessment was to provide a planning document that gives the City guidance and direction for facility improvements, project budgeting, and implementation schedules. This condition assessment evaluated and categorized projects identified by Carollo and City staff input. Appendix D (Critical Facilities Condition Assessment) provides a technical memorandum that describes the methodology and results of the lift station condition assessment in detail. The key findings and recommendations are summarized in Table 7.6.

Table 7.6 Lift Station Condition Assessment Recommendations

Facility	2025 Improvements	2040 Improvements
Caltrans Lift Station	• Install ladder and safety grating in wet well • Install step ladder into valve vault	• Install SCADA System. • Replace Submersible Pumps.
Westward Lift Station	• Install Ventilation fan in the valve vault. • Install an access ladder for wet well. • Replace level sensor support. • Coat interior of wet well.	• Improve security with surveillance. • Add concrete paving. • Replace submersible pumps. • Replace generator.

Based on the results of the condition assessment, the recommended improvements are as follows:

- Caltrans Lift Station (Project WWRR-2): According to the condition assessment results, this facility is in need of repairs to address safety and operation issues. The Long Term recommendations include the installation of SCADA for remote monitoring and control.. However, this facility is a low flow station and automation may not be cost effective.
- Westward Lift Station (Project WWRR-3): Based on results of the condition assessment, this facility is in need of repairs to address safety, maintenance, and operation. This lift station has been identified as an existing capacity deficiency and is recommend for abandonment in favor of a new trunk main. For this lift station to be abandoned, downstream projects for RSG must be completed. Therefore, improvement projects for this lift station under Near Term are recommend to be implemented, while Long Term projects may not be required and will not be included in the CIP. If, however, flows from Butterfield are anticipated before the RSG trunk line is constructed, the long term improvements should be completed in the near-term, along with upsizing of the force main.

7.2.5 Treatment Plant Improvements

The City has identified various treatment plant projects, which are currently included in the City's existing CIP. The projects include:

- Digester Cleaning (Project WWTP-1)
- Heat Exchanger Repairs (Project WWTP-2)
- Boiler Gas Control Valves (Project WWTP-3)
- Digester Gas Pipeline (Project WWTP-4)
- WWTP Upgrade (Project WWTP-5)

The most critical of the treatment plant improvement is the WWTP upgrade, which was discussed in Section 7.1.1. The flows that were utilized to perform all future (2025, 2040, and Build out) sewer system analyses as well as the recycled water analysis performed in Chapter 8 was based on the assumption that the WWTP would be expanded as needed. . The WWTP is estimated to reach capacity by 2025, when flows are expected to average 2.8 mgd.Other Improvements

Other miscellaneous improvement projects have been recommended to optimize the operation of the City's sewer system. The projects include:

- Septic Removal (Project WWO-1): These projects are recommended to connect septic users throughout the City to the wastewater collection system. Septic users within the City include residential, commercial, and industrial users.
- Lift Station Telemetry (Project WWO-2): This is a project in the City's existing CIP.

7.3 Future Sewer System Analysis

The goal of the future system analysis is to evaluate the collection system under various operating conditions utilizing the evaluation criteria summarized in Chapter 5 and the future flow projections described in Chapter 3. As part of the future system analysis, the planning years 2025 and 2040 were evaluated. In addition, a preliminary analysis was performed to identify improvements under Build-Out PWWF conditions. Therefore, the term future is a general reference to planning years 2025, 2040, and Build-Out.

Since the timing of growth under Build-Out conditions is unknown, the analysis performed in Chapter 7 under Build-Out conditions will need to be updated when additional information is available. In addition to the future system analysis, an alternative analysis was performed to review the system impacts with the addition of a Satellite Treatment Plant for the Butterfield Development.

The following analyses are described in this section:

- Gravity System Evaluation
- Lift Station Evaluation
- Satellite Treatment Plant for Butterfield Development

7.3.1 Gravity System Evaluation

The future system analysis of the gravity system was performed in a manner similar to the existing system evaluation. In accordance with the established flow depth criteria for existing sewers, pipelines with a maximum flow depth to pipe diameter (d/D) ratio greater than 0.92 were identified. In addition, pipelines improvements were identified and sized to mitigate capacity deficiencies under future flow conditions.

Figure 7.8 shows the locations of the future deficiencies under the future flow conditions for the planning horizon of the IMP, which are years 2025 and 2040. In addition, buildout deficiencies are shown on Figure 7.8. The proposed improvements that address future system deficiencies of this IMP are shown on Figure 7.9. The following summarizes the purpose and locations of facilities that would need to be replaced, paralleled, or added to the system to address future system deficiencies and projected growth. The projects are presented in chronological order, with capacity projects (WWGM-8 through WWGM-13) summarized first, then followed by projected growth projects (WWGM-14 through WWGM-32).

- Charles Street Trunk (Project WWGM-8): This project will replace approximately 1,000 feet of 18-inch diameter pipeline located in Charles Street, east of Hathaway Street. The flow levels within the gravity sewer cause the existing pipeline to exceed the maximum d/D criteria under 2040 PWWF conditions. To mitigate deficiencies at year 2040 flow conditions, it is recommended that the existing sewer be replaced with a 21-inch diameter pipeline.

- Porter Street Trunk (Project WWGM-9): This project will replace approximately 500 feet of 21-inch diameter pipeline located in Porter Street, west of Hargrave Street. The flow levels within the gravity sewer cause the existing pipeline to surcharge under 2040 PWWF, creating a bottleneck effect. The upstream and downstream pipelines have steeper slopes and are not capacity deficient under 2040 PWWF conditions. Improvements include replacing the existing sewer with a 30-inch diameter pipeline.
- Porter Street Trunk (Project WWGM-10): This project will replace approximately 5,000 feet of 21-inch diameter pipeline located in Porter Street and extends from Old Banning Idyllwild Road to Hathaway Street. To mitigate deficiencies at Build-Out flows, it is recommended that the existing pipeline be replaced with 4,500 feet of 24-inch and 500 feet of 30-inch diameter pipeline. The 30-inch diameter sewer is recommended due to the pipelines slope, which has a lesser grade.
- South WWTP Trunk Parallel (Project WWGM-11): This project consists of a 24-inch diameter parallel pipeline. The project extends approximately 3,000 feet and is located along an unimproved surface, from Porter Street and extends northeast, ending at the WWTP. To mitigate deficiencies at Build-Out flows, it is recommended that a 24-inch pipeline parallel the existing 24-inch and 30-inch diameter pipelines.
- North WWTP Trunk (Project WWGM-12): This project will replace approximately 500 feet of 18-inch diameter pipeline. The project extends south of Charles Street to the WWTP. To mitigate deficiencies at Build-Out flows, it is recommended that the existing pipeline be replaced with 500 feet of 21-inch diameter pipeline.
- Wilson Street Sewer (Project WWGM-13): This project will replace approximately 500 feet of 6-inch diameter pipeline located in Wilson Street and extends from Murry Street to Alessandro Road. To mitigate deficiencies at Build-Out flows, it is recommended that the existing 6-inch diameter pipeline be replaced with an 8-inch diameter pipeline.
- Butterfield Offsite Trunk (Project WWGM-14): This project has been carried forward from Butterfield's Specific Plan (Draft, Nov 2016 Exhibit 10B) as a proposed offsite trunk sewer for a multi-use community. The existing collection system downstream of Butterfield consist of an 8-inch and 12-inch pipeline. Both pipelines conveys flows to the Westward Lift Station, which has been identified as capacity deficient. Based on a d/D criteria of 0.75, the 8-inch is capable of conveying an additional 0.3 mgd of peak flow capacity. If flows continue to increase, surcharging will occur when a total of 0.37 mgd of peak flow are added to the line. If Butterfield and the City agree to an interim connection to the existing system, further actions should be taken to confirm the pipeline diameter and inverts of the 8-inch pipeline.

  Segments of the 15-inch diameter pipeline upstream of the Westward Lift Station have a d/D of 0.68 under existing PWWF conditions. With an increase of 0.3 mgd to existing peak flows the d/D will increase to approximately 0.75.

  To provide the Butterfield community with sewer service at the existing WWTP, an offsite trunk would be required. The project is considered near term and will consist of approximately 7,500 feet of 15-inch diameter pipeline. The proposed trunk would extend from the intersection of Highland Home Road and Wilson Street, continue east on Wilson Street, south on Omar Street, and east on Ramsey Street to Sunset Avenue.
- Butterfield-Loma Linda Offsite Trunk (Project WWGM-15): This project will provide service to Butterfield and areas of growth North West of the City. The projects total

length consists of approximately 2,000 feet of 15-inch diameter pipeline. The proposed trunk would extend from the intersection of Westward Avenue /Sunset Avenue and continue south along Sunset Avenue to Pershing Creek

- Westward Lift Station Bypass (Project WWGM-16): This project is discussed in Section 7.3.2.
- RSG Main Trunk (Project WWGM-17): This project has been carried forward from RSG Master Plan and only identifies the backbone infrastructure for RSG. The major pipelines identified in this project extend approximately 14,500 feet and connect to the existing sewer system south of Wesley Street. The Master Plan recommends 4,000 feet of 18-inch, 7,500 feet of 21-inch, and 3,000 feet of 24-inch diameter pipeline. The proposed infrastructure will convey wastewater flows from existing and future development. The backbone infrastructure follows the incline of the terrain and pipeline diameters are sized to fit flow and varying slope conditions. This project consists of the following segments:
  - WWGM-17A: This section extends approximately 3,700 feet and includes 21-inch and 24-inch diameter pipeline. This segment parallels Smith Creek and connects to an existing 21-inch diameter pipeline South of Wesley Street.
  - WWGM17B: This segment consists of approximately 6,100 feet of 24 inch and 21-inch diameter pipeline. This reach crosses the confluence of the Pershing Creek and Smith Creek.
  - WWGM17C: This section consists of approximately 5,100 feet of 18 inch and 21-inch diameter pipeline. This segment parallels the Pershing Creek and extends downstream from projects WWGM-15 and WWGM-16.
- Wilson 97 Offsite Sewer (Project WWGM-18): This project encompasses approximately 2,000 feet of 8-inch diameter pipeline. The project extends along Wilson Street and connects to the existing sewer system in Sunrise Avenue. These pipelines are needed to service the Wilson 97 development
- RMG Sewer (Project WWGM-19): The project extends along Wilson Street and connects to the existing sewer system in Florida Street. These pipelines are needed to service known residential development (RMG) and additional infill along Wilson Street. This project recommends approximately 1,500 feet of 8-inch diameter pipeline.
- Lincoln Street Sewer (Project WWGM-20): This project will connect an existing 8-inch diameter dry pipeline in Lincoln Street to the collection system along Fourth Street.
- Cottonwood Road Sewer (Project WWGM-21): This project will service future growth along the eastern portion of the City. The project consists of 4,000 feet of 8-inch diameter pipeline and extends south of Interstate-10 to Westward Avenue. This project is upstream of Lift Station WWLS-2.
- Fountain Street Sewer (Project WWGM-22): This project will service future growth along the south eastern portion of the City and will extend the area served by the City's wastewater collection system. The project consists of 5,500 feet of 8-inch diameter pipeline and extends east along Fountain Street and Porter Street. The project will utilize a lift station (WWLS-4) and force main (WWFM-3) to convey flows and provide service to rural residents.
- Longhorn Road Sewer (Project WWGM-23): This project will service future growth along the south western portion of the City and will extend the area served by the City's wastewater collection system. The project consists of approximately 20,000 feet of

8-inch diameter pipeline and extends east along Longhorn road and Old Banning Idyllwild Road.

- Bobcat road Sewer (Project WWGM-24): This project will service future growth along the south western portion of the City. The project consists of approximately 7,000 feet of 12-inch diameter pipeline and extends east on Bobcat Road. This pipeline will service residential users and is a potential connection for Five Bridges.
- Sunset Avenue Sewer (Project WWGM-25): This project will service future growth along the northern portion of the City. The project consists of 24,500 feet of 12-inch diameter pipeline and extends along Sunset Avenue. The project would expand the collection system to the north and provide service to the proposed Black Bench and Loma Linda Communities. The project will connect to an existing 12-inch diameter pipeline that flows into an 8-inch pipeline. The 8-inch pipeline is approximately 600 feet in length and would require replacement with a 12-inch pipeline.
- Westward Avenue Sewer (WWGM-26): The project will provide future service to industrial users and consists of 3,000 feet of 8-inch diameter pipeline. The proposed sewer extends east on Westward Avenue.
- Mias Canyon Road Sewer (Project WWGM-27): This project will service future growth along the north eastern portion of the City's sphere of influence. The project consists of approximately 12,500 feet of 8-inch diameter pipeline and extends the collections system northward on Mias Canyon Road. This pipeline will service rural residential users and is upstream of lift station (WWLS-6) and force main (WWFM-5).
- Florida Street Sewer (Project WWGM-28): This project will service future growth along the north eastern portion of the City. The project consists of approximately 1,500 feet of 8-inch diameter pipeline and extends south on Florida Street to Santa Rita Place. This pipeline will service low density residential users.
- Almond Street Sewer (Project WWGM-29): This project will service future growth along the north eastern portion of the City. The project consists of approximately 1,500 feet of 8-inch diameter pipeline and extends south on Almond Street and Blanchard Street, connecting to the existing system east of Theodore Street. These pipelines will service low density residential users.
- Interstate 10 Sewer Crossing (Project WWGM-30): The project consists of approximately 1,000 feet of 12-inch diameter pipeline and will cross Interstate-10. This project will provide service to residential and commercial users in the north east area of the City. This segment will also require a 24-inch diameter steel casing.
- Lincoln Street Sewer (WWGM-31): This project will service future growth along the eastern portion of the City. The project consists of approximately 3,000 feet of 8-inch diameter pipeline and extends east on Lincoln Street to Hathaway Street. This pipeline will service industrial users.
- Ramsey Street Sewer (Project WWGM-32): The project will provide future service to commercial users and recommends 1,500 feet of 8-inch diameter pipeline. The proposed sewer extends east on Ramsey Street and connects to the existing sewer system at Lori Way.

7.3.2 Lift Station Evaluation

The City's lift stations were evaluated under 2025, 2040 and Build-Out PWWF conditions. As listed in Table 7.4, the Westward Lift Station is deficient under existing conditions. The recommended improvement to mitigate the existing deficiency is discussed in Section 7.2.2. The remaining lift stations have sufficient capacity to convey future and Build-Out PWWFs. However, additional lift stations will be required to serve new developments and growth within the City. The recommended improvements include:

- Distribution Center LS (Projects WWLS-2, WWFM-2): Lift Station WWLS-2 is estimated to have a firm capacity of 0.95 mgd and total capacity of 1.9 mgd. The proposed lift station is located in Westward Avenue and east of Scott Street. The lift station will have an 8-inch diameter force main. The force main extends 4,000 feet and connects to the existing sewer system in Charles Street, west of Scott Street. The project is sized to service future commercial and Build-Out users.
- Business Park LS (Project WWLS-3): Lift Station WWLS-3 is estimated to have a firm capacity of 0.31 mgd and total capacity of 0.62 mgd. The proposed lift station is located east of Hathaway and Nicolet intersection. The project is sized to service industrial and commercial users in the east quadrant of the City. The force mains and gravity mains have been constructed.
- Porter Street LS (projects WWLS-4, WWFM-3): Lift Station WWLS-4 is estimated to have a firm capacity of 0.08 mgd and total capacity of 0.16 mgd. The proposed lift station is located in Porter Street, south of Hathaway Street. The lift station will have a 6-inch diameter force main. The force main extends 4,500 feet and connects to the existing sewer system at the intersection of Porter Street and Hathaway Street. The project is sized to service rural residential users in the south east quadrant of the City.
- Roadrunner Trail LS (Projects WWLS-5, WWFM-4): Lift Station WWLS-5 is estimated to have a firm capacity of 0.17 mgd and total capacity of 0.34 mgd. The proposed lift station is located south of Roadrunner Trail and Shirleon Drive. The project is sized to service rural residential users in the south west quadrant of the City and is recommended to overcome the rugged terrain. The lift station will utilize a 6-inch diameter force main. The force main extends approximately 1,000 feet and connects to the future sewer system south of the Shirleon Drive and Roadrunner intersection.
- Bluff Street LS (Projects WWLS-6, WWFM-5): Lift Station WWLS-6 is estimated to have a firm capacity of 0.20 mgd and total capacity of 0.40 mgd. The proposed lift station is located north east of Bluff Street and Mias Canon Road, near Banning's Sportsman Club. The project is sized to service rural residential users in the north east quadrant of the City. The lift station will have a 6-inch diameter force main. The force main extends 1,000 feet and connects to the future sewer system at the intersection of Bluff Street and Mias Canyon Road.

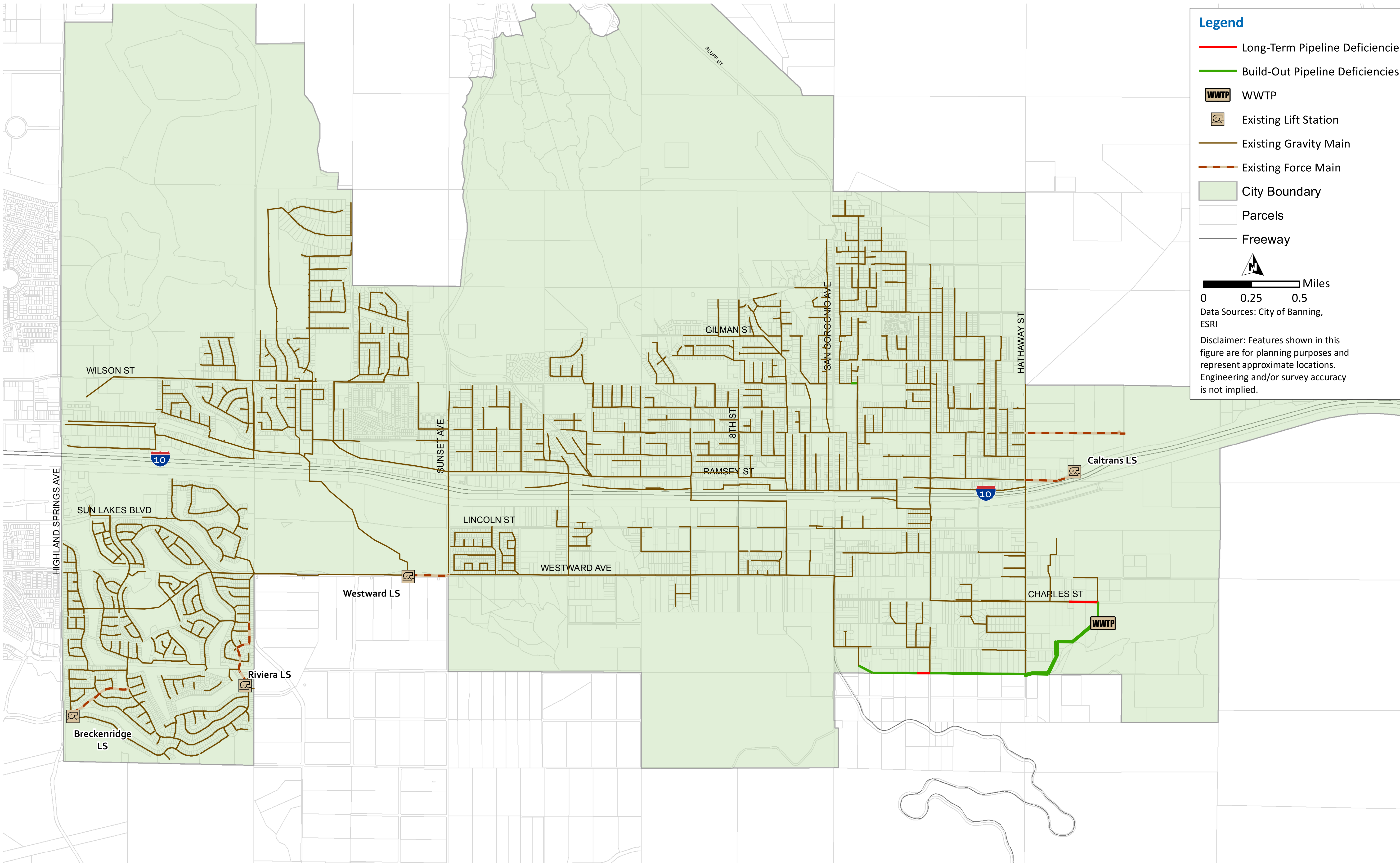

Figure 7.8 Future System Deficiencies

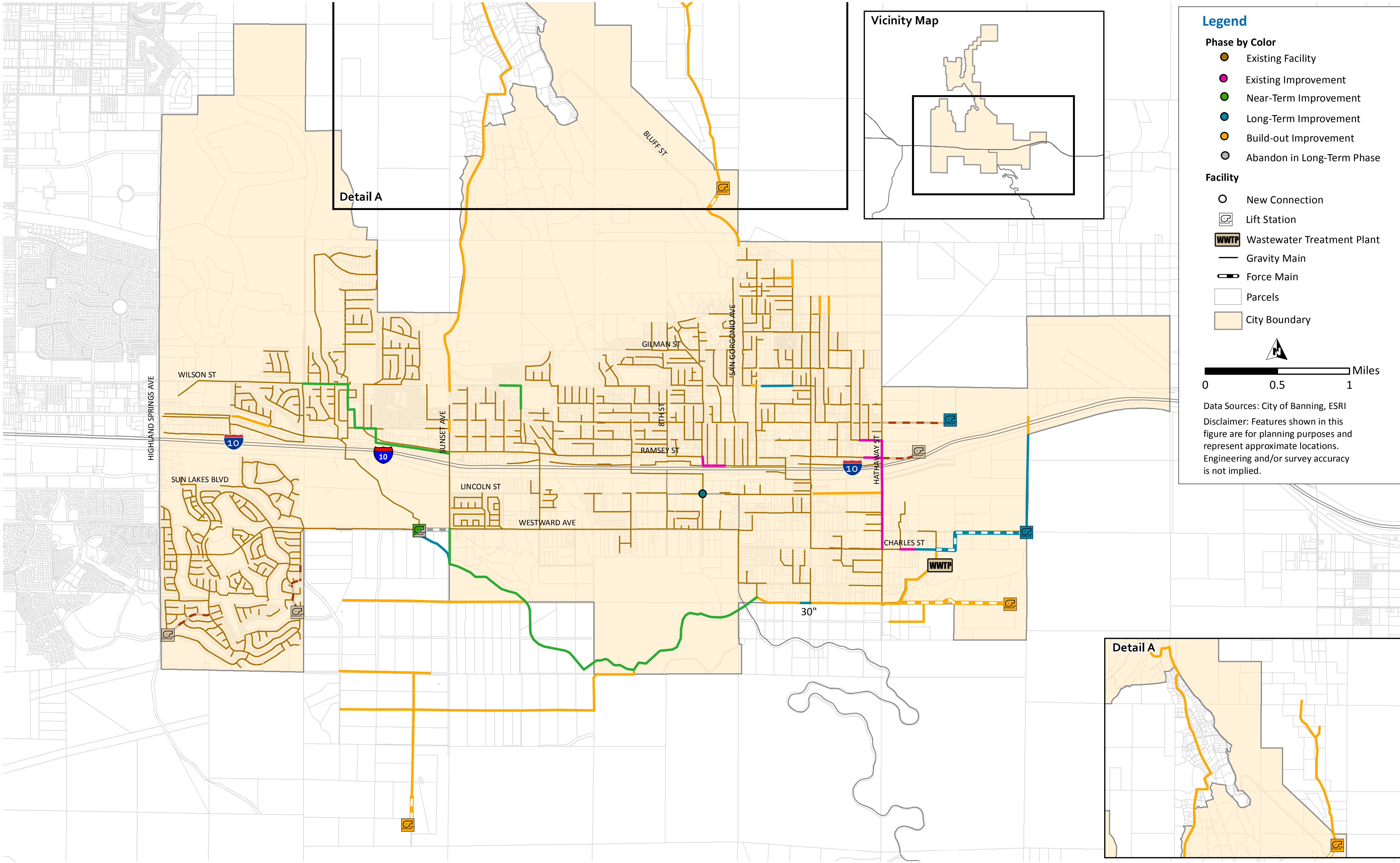

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Figure 7.9 Future System Improvements without Satellite Treatment Plant

7.3.3 Satellite Treatment Plant for Butterfield Development

As an alternative to the City's WWTP receiving all the wastewater within the projected service area, this IMP evaluated the potential use of a satellite facility to treat Butterfield's wastewater. The Butterfield Satellite Plant (Satellite Plant) would be located near the intersection of Highland Home Road and Wilson Street. The Satellite Plant was evaluated under future and Build-Out conditions.

Butterfield's estimated ADWF at Build-Out is 0.76 mgd and the capacity for the Satellite Plant was evaluated at 0.71 mgd. An analysis was performed to determine the amount of flow needed in the collection system to prevent solid deposition. It was determined that approximately 7-percent of Butterfields ADWF would need to be discharged into the sewer system. The remaining 93-percent of Butterfield's ADWF could theoretically go the satellite Plant. The remaining flow would be conveyed into the City's collection system. This minimum amount of flow is needed to provide a daily velocity of 2 feet per second (ft/s) in the gravity sewer downstream of the Satellite Plant. The purpose is to maintain a peak velocity of 2 ft/s in the gravity sewer downstream of the Satellite Plant and allow for sufficient flushing in order to prevent solids deposition in the line. The actual amount of flow that could be diverted to the Satellite Plant will depend on the selected treatment technology and is beyond the scope of this IMP. Figure 7.10 illustrates the recommended improvements with a Satellite Plant at the Butterfield development.

Due to a reduction of wastewater flows, the following proposed improvements would be revised as follows:

- Porter Street Trunk (Project WWGM-9): This project had recommended replacement of 500 feet of 21-inch pipeline with 30-inch diameter pipeline. With the Satellite Plant, this project is triggered under Build-Out and reduced in diameter to a 27-inch pipeline.
- Porter Street Trunk (Project WWGM-10): With centralized treatment at the WWTP, this project had recommended replacement of approximately 5,000 feet of 21-inch diameter pipeline with a 30-inch diameter pipeline. With treatment at the Satellite Plant, this project is reduced to approximately 500 feet, with a recommended 27-inch diameter pipeline. This project is directly upstream of WWGM-9 and is similar in diameter and length.
- South WWTP Trunk Parallel: This project can be reduced from a 24-inch to a 21-inch diameter pipeline. However, to offer additional redundancy in case the Satellite Plant experiences any down time, it is recommended that a 24-inch parallel pipeline be installed.
- Butterfield Offsite Trunk (Project WWGM-14): This project can be reduced from a 15-inch to a 10-inch diameter pipeline. However, to offer additional redundancy in case the Satellite Plant experiences any down time, it is recommended that a 15-inch pipeline be installed.

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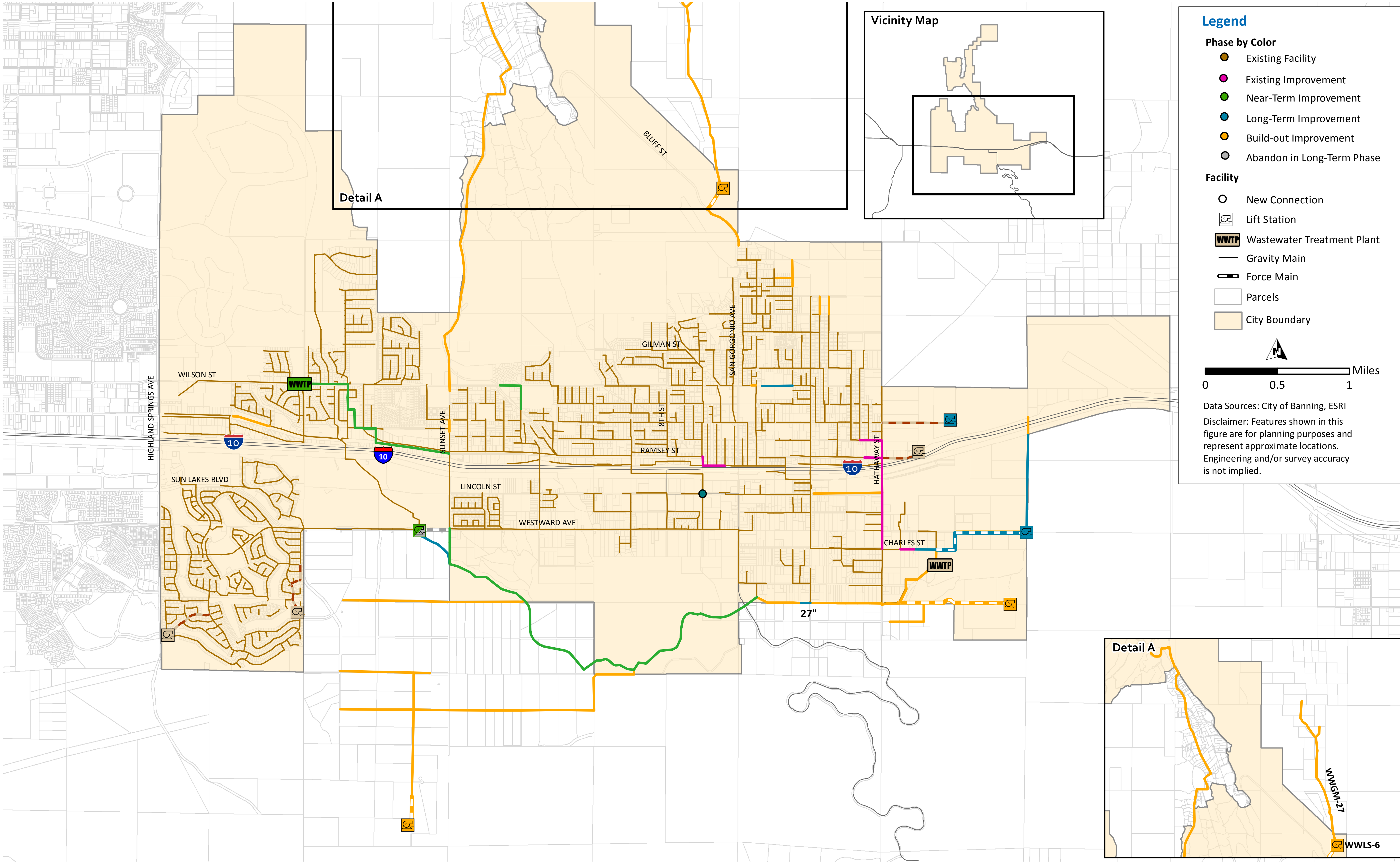

Figure 7.10 Future System Improvements with Satellite Treatment Plant

7.4 Summary of Recommendations

The recommendations identified in this chapter are summarized in this section. Detailed cost estimates for each of these recommendations are included in the CIP chapter (see Chapter 9) of this IMP. The recommendations are conceptual and should be refined during the design phase.

- **Gravity System Improvements:**
  - **Existing System**: Seven (7) gravity main projects ranging in diameter from 10 to 21 inches with a total length of 9,000 feet (Projects WWGM-1 through WWGM-7).
  - **Future System**: Ten (9) gravity main projects ranging in diameter from 8 to 30 inches with a total length of 31,500 feet (Projects WWGM-8 , WWGM-9, WWGM-14, WWGM-15, WWGM-17 through WWGM-21).
  - **Build-Out**: Thirteen (13) gravity main projects ranging in diameter from 8 to 24 inches with a total length of 89,500 feet (Projects WWGM-10 through WWGM-12, WWGM-22 through WWGM-32).
- **Lift Station and Force Main Improvements:**
  - **Existing System**: West Ward Lift Station Interim Upgrade (Project WWLS-1, WWFM-1) recommends upgrading the lift station capacity and force main.
  - **Future System**: Three (3) lift stations projects with a total capacity of 2.52 mgd and one (1) force main project with a pipeline diameter of 8-inches a total length of 4,000 feet. The recommended improvements include:
    - One (1) bypass pipeline project with an 18-inch diameter and a total length of 2,000 feet (Project WWGM-16).
    - Distribution Center Lift Station with a proposed capacity of 1.90 mgd (Project WWLS-2) and an 8-inch diameter force main with a total length of 4,000 feet (Project WWFM-2)
    - Business Park Lift Station with a proposed capacity of 0.62 mgd (Project WWLS-3). The force main has already been constructed.
  - **Build-Out**: Three (3) lift stations projects with a total capacity of 0.90 mgd and three (3) force main projects with a diameter of 6-inches and a total length of 6,500 feet. The recommended improvements include:
    - Porter Street Lift Station with a proposed capacity of 0.16 mgd (Project WWLS-4) and a 6-inch diameter force main with a total length of 4,500 feet (Project WWFM-3).
    - Roadrunner Trail Lift Station with a proposed capacity of 0.34 mgd (Project WWLS-5) and a 6-inch diameter force main with a total length of 1,000 feet (Project WWFM-4).
    - Bluff Street Lift Station with a proposed capacity of 0.40 mgd (Project WWLS-6) and a 6-inch diameter force main with a total length of 1,000 feet (Project WWFM-5).
- **Rehabilitation and Replacement Improvements**
  - **Existing System**: Annual sewer replacements (Project WWRR-1).
  - **Future System**: Project WWRR-1 continues into Future.
  - **Build-Out**: Project WWRR-1 is considered an indefinite project.
- **Condition Assessment Improvements:**
  - **Existing System**: Caltrans Lift Station (Project WWRR-2) and Westward Lift Station site improvements (Project WWRR-3)

  - **Future System**: None:
  - **Build-Out**: Projects WWRR-2 and WWRR-3 have Buildout Recommendations..
- **Treatment Plant Improvements:**
  - **Existing System**: Four (4) projects were identified, which include:
    - Digester Cleaning (Project WWTP-1)
    - Heat Exchanger Repairs (Project WWTP-2)
    - Boiler Gas Control Valves (Project WWTP-3)
    - Digester Gas Pipeline (Project WWTP-4)
  - **Future System**: One (1) project was identified, which includes the WWTP upgrade to Tertiary Treatment (Project WWTP-5).
  - **Build-Out**: None
- **Other Improvements:**
  - **Existing System**: One (1) project was identified, which include:
    - Lift Station Telemetry (Project WWO-2)
  - **Future System**: One (1) project was identified, which include:
    - Septic Removal (WWO-1)
  - **Build-Out**: One (1) project was identified:
    - Septic Removal (WWO-1) continues into build-out.
- **Satellite Treatment Plant Alternative:**
  - **Existing System**: None
  - **Future System**: With the addition of the Satellite Treatment Plant to serve the Butterfield development, three (3) projects within the gravity system improvements may be altered. The projects include:
    - Butterfield Offsite Trunk (Project WWGM-14)
    - **Build-Out**: With the addition of the Satellite Treatment Plant to serve the Butterfield development, three (3) projects within the gravity system improvements may be reduced in size. The projects include:Porter Street Trunk (Projects WWGM-9) reduced in length and diameter.
    - Porter Street Trunk (Project WWGM-10) reduced in diameter and length.
    - South WWTP Trunk Parallel (Project WWGM-11) reduced in diameter.

Chapter 8

RECYCLED WATER SYSTEM EVALUATION

This chapter describes the evaluation of alternatives for expansion of the existing non-potable water system to maximize service to the potential customers identified in Chapter 3. The evaluation and sizing criteria described in Chapter 5 were used to size these system expansions. This chapter is divided into the following sections:

- **Existing Recycled Water System**. This section discusses the existing non-potable water supply sources and the facilities that make up the existing recycled water system.
- **Alternative Analysis**. This section discusses the development of the future recycled water system layout alternatives that serve potential customers and/or recharge into the groundwater basins based on the availability of supply during the near-term (by year 2025), long-term (year 2026-2040), and build-out (beyond year 2040) phases. The pipelines and facilities required for each alternative are identified, which were sized using the criteria described in Chapter 5. For comparative purposes, planning level cost estimates were developed for each alternative.
- **Conclusions and Recommendations**. The alternatives are compared and the top-ranking system configuration is selected for the planning horizon of the recycled water system in this Integrated Master Plan (IMP).

The Capital Improvement Plan (CIP) for the recommended recycled water system alternative is described in Chapter 9 of this IMP.

8.1 Existing Recycled Water System

The City's existing non-potable water system delivers recycled water to one existing customer, the Sun Lakes Golf Course. As described in Chapter 3, the City has served an average of 850 afy (or 0.8 MGD) from Well M7 to Sun Lakes Development in years 2012 through 2014.

In addition, the City has constructed approximately 2.2 miles of 24-inch diameter pipeline and has begun constructing an additional 3.4 miles of pipeline to connect the existing recycled water pipes to the wastewater treatment plant (WWTP). The existing recycled water system and planned pipelines are shown on Figure 8.1.

8.1.1 Recycled Water Supply Sources

The City currently serves Sun Lakes Development with non-potable water from Well M7. In addition to Well M7, the City has one other existing well (Well M12) and one future well (R-1) for non-potable water use. The City also plans to upgrade the existing WWTP and treat its wastewater treatment process to meet tertiary standards for recycled water irrigation use. A description of each source is described in the proceeding sections.

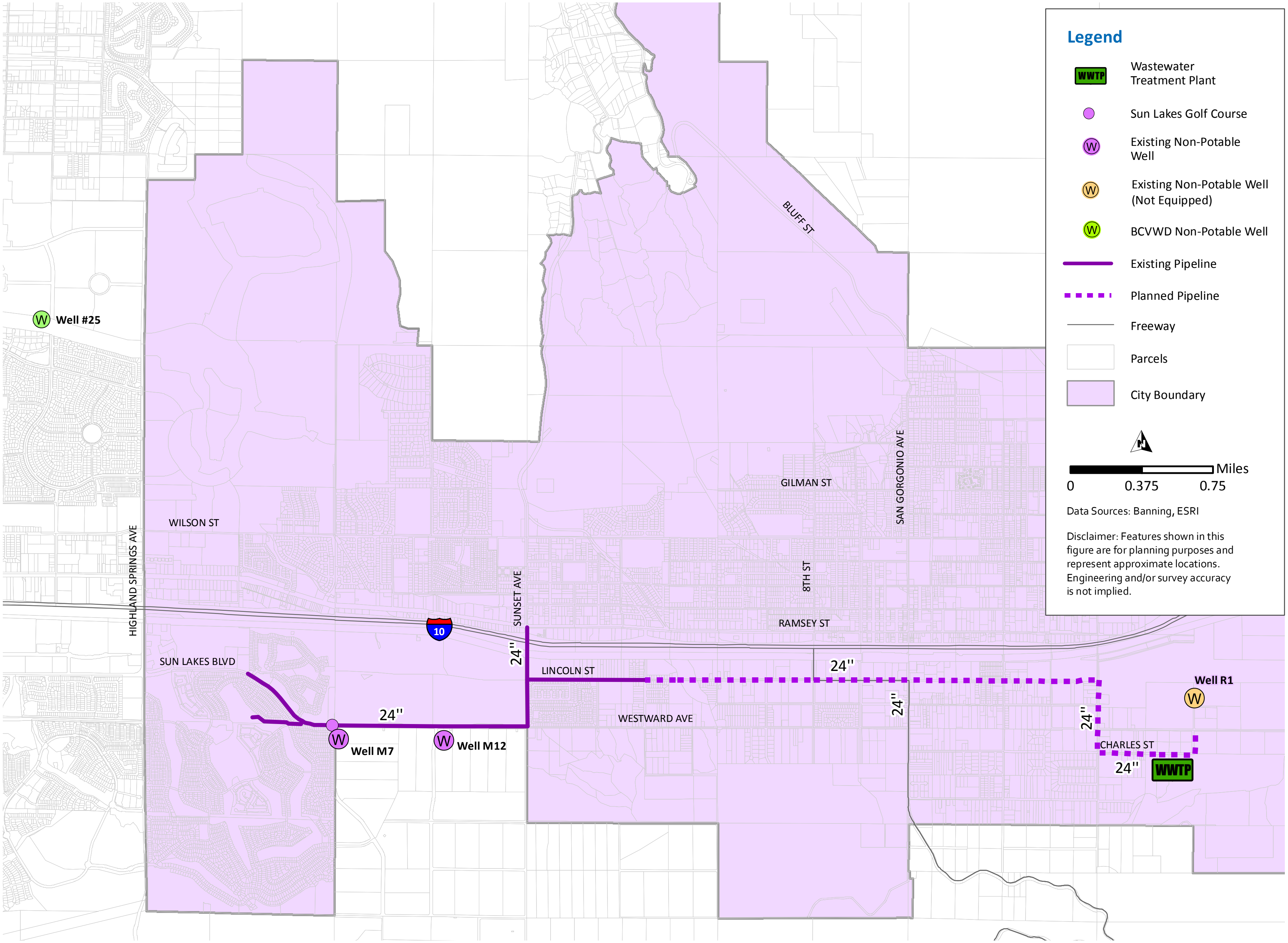

Figure 8.1 Existing Recycled Water System

8.1.1.1 Groundwater Wells

The City has been operating one well (Well M7) and estimates that this well will produce an average capacity of approximately 350 gpm. In addition, the City also started to operate a second well (Well M12) mid-year 2017 with an estimated capacity of approximately 1,000 gpm. A third well (Well R-1) is located near the WWTP, but is not yet equipped. The location of these wells can be found on Figure 8.1, while the status and capacities of the wells are summarized in Table 8.1.

Table 8.1 Existing City Non-Potable Groundwater Well Capacities

Well Number	Current Status	Future Status	Estimated Capacity(1) (gpm)	Estimated Capacity(2) (MGD)
Well M7	Operational	Potable	350	0.5
Well M12	Operational	Potable	1,000	1.4
Well R-1	Not Equipped	Equipped for Non-Potable	1,150	1.7
Total	**N/A**		**2,500**	**3.6**

Note:
(1) Well capacities provided by City staff.
(2) Assumes 24 hour operation of groundwater well.

As listed in Table 8.1, the total estimated capacity of the three wells with the addition of Well R-1 is approximately 2,500 gpm (or 3.6 MGD) of non-potable water. Although Wells M7 and M12 are both equipped, City staff plans to eventually convert Well M7 and Well M12 over to the potable water system by the end of year 2025 and year 2040, respectively, to supplement the potable water supply. Along with this conversion, it is assumed that Well R-1 will be equipped by the end of year 2025 to supplement the recycled water system supply. With only Well R-1 and Well M12 online by the end of year 2025, the groundwater well supply is estimated to provide up to 2,150 gpm (or 3.1 MGD) of non-potable water. By the end of year 2040, only Well R-1 is assumed to be online, which results in a groundwater well supply of 1,150 gpm (or 1.7 MGD). The capacities listed in MGD reflect the assumption that wells would be pumping 24 hours per day. In addition to the three wells within the City boundaries, the City jointly owns and operates two non-potable groundwater wells (Wells 25 and 26) with the Beaumont Cherry Valley Water District (BCVWD) of which the City is entitled to 50 percent of the production capacity. These two wells could potentially be converted back to potable wells in the future. Based on input from City staff, the capacity of the two wells is assumed to be 1,000 gpm (or 1.1 MGD) each, equating to a total of 2,000 gpm (or 2.9 MGD) of additional supply the City. Currently, there are no plans to connect these wells to the City's main recycled water system, but would instead provide non-potable water for the Butterfield Development.

8.1.1.2 Wastewater Treatment Plant

As mentioned in Chapter 7, the City plans to upgrade the existing WWTP treatment processes to meet tertiary standards. Based on the ADWF projections presented in Chapter 3, the available recycled water supply from the WWTP was estimated assuming 10 percent losses to treat to secondary standards and an additional 10 percent losses to treat to tertiary standards. A summary of the estimated recycled water availability is identified in Table 8.2.

As listed in Table 8.2, there is no existing recycled water supply capacity from the WWTP since the WWTP does not currently include tertiary treatment. The recycled water supply capacity is projected to increase to 2.4 MGD and 3.5 MGD in the near-term and long-term, respectively. At build-out, the WWTP recycled water supply capacity is projected to increase to 5.14 MGD.

Table 8.2 Projected WWTP Recycled Water Capacity

Planning Year	Estimated ADWF(1,2) (MGD)	Estimated Recycled Water Capacity(3)	
		(MGD)	(afy)
Existing (2017)	2.0	0	0
Near-term (2025)	2.8	2.4	2,703
Long-term (2040)	4.3	3.5	3,892
Build-out (beyond year 2040)	6.4	5.1	5,761

Notes:
(1) Estimated ADWF from Table 3.18. See section 3.2.4 for assumptions and methodology.
(2) WWTP expansion is triggered at 2.88 MGD per existing permit. See Chapter 7 for details.
(3) Assumes 10 percent losses to treat to secondary standards and an additional 10 percent losses to treat to tertiary standards. Assumes all WWTP inflows are to be treated to tertiary standards.

8.1.1.3 Projected Recycled Water Supply

Based on the projections from the groundwater well supply and WWTP, the total projected recycled water supply without the BCVWD wells is summarized in Table 8.3. As mentioned previously, Wells M7 and M12 will be converted to the potable water system in the future, which is anticipated to be completed by the end of the near-term and long-term phases, respectively.

Table 8.3 Total Projected Recycled Water Availability

Planning Year	Non-Potable Well Capacity(1,2) (MGD)	WWTP Recycled Water Availability(1) (MGD)	Total	
			(MGD)	(afy)
Existing (2017)	1.9	0	1.9	2,177
Near-term (2025)	3.1	2.4	5.5	6,171
Long-term (2040)	1.7	3.5	5.1	5,747
Build-out	1.7	5.1	6.8	7,615

Notes:
(1) Capacities and recycled water availability obtained from Table 8.1 and 8.2. Capacity does not include BCVWD wells. Assumes 24 hour production.
(2) Existing non-potable wells include Wells M7 and M12, which are assumed to convert to potable water by the near-term phase and long-term phase, respectively. Long-term phase and beyond only includes Well R-1.

As listed in Table 8.3, the total existing available supply is estimated at 1.9 MGD (or 2,177 afy). With the upgrade of the WWTP to include tertiary treatment and the conversion of Wells M7 potable water, the recycled water supply availability is projected to increase to 5.5 MGD (or 6,171 afy). By the end of the planning period of this IMP (year 2040), the recycled water availability is projected to decrease to 5.1 MGD (or 5,747 afy) due to the conversion of Well M12. At build-out, the recycled water supply is projected to increase to 6.8 MGD (or 7,615 afy). For planning purposes, it is assumed that the recycled water MDD may not exceed 6.8 MGD. While MDD will be met by the recycled water supply, the supply fluctuations required to meet PHD is assumed to be met through storage.

8.1.2 Existing Facilities

The City's existing recycled water facilities consists of the backbone 24-inch diameter pipeline and the two non-potable groundwater wells. As shown 0n Figure 8.1, the City has constructed approximately 2.2 miles of 24-inch diameter pipeline and has begun to develop plans for an additional 3.4 miles of pipeline to connect to the WWTP. In addition, the City has the capability to serve non-potable water from Well M7 and Well M12. The planned recycled water pipelines will also connect to Well R-1 to serve non-potable water.

8.2 Recycled Water Alternatives

For the future system evaluation, the hydraulic model was used to develop potential system expansion alternatives that maximize the usage of recycled water within the City's service area, while meeting the evaluation criteria discussed in Chapter 5. This section discusses the methodology used for the creation of alternatives and the selection of recommended recycled water system project. This methodology includes the following steps:

- Demand and supply balance based on planning phases
- Development of the initial system layout
- Division of the initial layout into phases based on available recycled water supply
- Analysis of non-potable reuse (NPR) and indirect potable reuse (IPR)
- Selection of recommended system

The recycled water system expansion is split into three phases, namely near-term (year 2025), long-term (year 2040), and build-out (beyond year 2040). The supply and demand balances developed to determine the phasing of the customers is presented in Appendix F.1.

8.2.1 Non-Potable Reuse Alternative (Alternative 1)

Non-potable reuse involves treating wastewater for purposes other than drinking, such as industrial uses, agriculture, or landscape irrigation at public parks and golf courses. Connecting all of the potential customers identified in Appendix C would maximize the use of recycled water for irrigation. However, due to supply limitations and the distance of the customers to existing and planned backbone pipelines, it was determined that it is not cost effective to connect to some of the potential customers. Thus, a condensed prioritized list of eight (8) customers mentioned in Chapter 3 (Table 3.24) was considered for this evaluation. These eight customers have a total potential recycled water demand of 2,530 afy (or 2.3 MGD) and a MDD of 6.4 MGD.

Utilizing the assumptions that were agreed upon with the City, Alternative 1 was developed as part of the NPR analysis with the following assumptions made to evaluate potential recycled water customers and identify facilities required to serve these customers:

1. Since the BCVWD Wells 25 and 26 are located near the Butterfield Development and are projected to have enough capacity to supply the Butterfield Development demands, it is assumed that the Butterfield Development recycled water demand will be served by these two non-potable wells.
2. Although Wells 25 and 26 could be converted to potable water in the future, it is assumed that these wells will be used to serve the Butterfield Development recycled water demands for the purpose of this analysis. If in the future, Wells 25 and 26 are converted to potable water use, Butterfield irrigation demands would need to be met

with potable water or imported water purchased from the SGPWA. Thus, the Butterfield Development is not included in the analysis of the main recycled water system.

3. For the purpose of this analysis, it is assumed that BCVWD's pipeline from Wells 25 and 26 will be used to convey water to the Butterfield Development distribution system.
4. The connections to the Butterfield Development's proposed recycled water system are assumed to be located at the intersection of Cougar Way and Highland Springs Avenue and the intersection of Oak Valley Parkway and Highland Springs Avenue. PRV stations will be required at these locations to convey water into the Butterfield distribution system.
5. The BCVWD co-owned wells will not connect to the City's main recycled water system.
6. The Butterfield Development may choose to construct a lake for storage, which will require additional piping from the PRV stations to the lake. This pipeline is not included as part of this analysis.
7. The Butterfield Development will be connected to the City's wastewater collection system and contribute to the available recycled water supply.
8. Available supply must meet MDD conditions. PHD conditions will be met through storage at the WWTP. This storage is assumed to be completed as part of the WWTP expansion.
9. Only customers south of the I-10 freeway will be considered for NPR based on cost-effectiveness.
10. Based on discussions with City staff, Five Bridges, and Neighborhood Park are not likely to be constructed until later phases. Five Bridges is estimated for the build-out phase, while Neighborhood Park's status remains unknown. It was assumed that Neighborhood Park will not come online in the near-term, but is assumed to come online in the long-term.

As mentioned in the assumptions, the Butterfield development is not included in the City's main recycled water system. Thus, seven customers are assumed to be connected into the City's main recycled water system, resulting in a potential recycled water demand of 1,666 afy (or 1.5 MGD) and an MDD of 4.2 MGD. The potential demand of the seven customers and required pipeline length are summarized in Table 8.4. The Butterfield Development demands are not included in Table 8.4, because this development is assumed to be served from the BCVWD co-owned wells.

Existing customer demands currently served by potable water for irrigation are listed as the potable water conversion demands, whereas new development and customers are listed as new demands. A more detailed breakdown of customer demands is presented in Table 3.24.

As listed in Table 8.4, the total potential non-potable reuse demand on the main system equates to approximately 1,666 afy. Of this demand, 1,151 afy is attributed to potable water conversion, while 515 afy is projected from new developments and customers. A balance of 5,949 afy of recycled water remains when compared to the total available supply by build-out of 7,615 afy listed in Table 8.3. This excess supply has the potential to be used for IPR. The proposed system layout and associated phasing is presented on Figure 8.2. The analysis of alternatives and recommendation is listed in Section 8.3.

Table 8.4 Potential Non-Potable Reuse Demands by Phase

Phase	Pipeline Length (ft)	Potable Water Conversion Demands (afy)	New Demands(1) (afy)	Total NPR Demands(2) (afy)
Near-term	26,000	1,151	257	1,408
Long-term	500	0	35	35
Build-out	500	0	223	223
Total	**27,000**	**1,151**	**515**	**1,666**

Note:
(1) New demands includes the new expansion of Lions Park.
(2) Available supply must meet MDD conditions. PHD will be met with storage at the wastewater treatment plant.
(3) Demand projections exclude Butterfield Demands because this development is assumed to be supplied from its own dedicated supply sources.

8.2.1.1 Near-Term Phase

The near-term phase includes a new recycled water pump station at the WWTP and equipping Well R-1 to supply the recycled water system. In addition, a forebay is included at the WWTP to store Well R-1 supply, which will then feed into the recycled water pump at the WWTP. This will require approximately 2,500 feet (or 0.5 miles) of 12-inch diameter pipeline to connect Well R-1 to the forebay.

This phase also includes the planned 24-inch diameter backbone pipeline along Lincoln Street. This pipeline alignment begins near Well R-1 and continues west on Charles Street to the WWTP. From the WWTP, the pipeline heads west on Charles, then north on Hathaway Street to Lincoln Street, where it heads west to connect to the existing pipeline at the intersection of Lincoln Street and 22nd Street. Banning High School, Dysart Park, Lions Park, and Rancho San Gorgonio, with a total demand of 558 afy, are connected into the main system with pipeline diameters ranging between 6 to 12 inches. With the new backbone system, Sun Lakes is also connected into the main system with a demand of 850 afy, resulting in a total system demand of 1,408 afy. The total pipeline required for the main recycled water system is approximately 19,000 feet (or 3.6 miles) of 24 inch diameter, 1,500 feet (or 0.3 miles) of 12 inch diameter, and 6,500 feet (or 1.2 miles) of 6-inch diameter.

The Butterfield Development is also anticipated to be connected to the BCVWD co-owned wells in the near-term phase with an initial demand of 162 afy. As mentioned previously, this requires an additional 2.5 miles of pipeline with diameters ranging between 12 to 16-inches to fill storage reservoirs or alternatively, PRV stations to connect BCVWD recycled water system to the City's recycled water system.

8.2.1.2 Long-Term Phase

The long-term phase includes a new connection to Neighborhood Park with a demand of 35 afy. During this phase, the Rancho San Gorgonio Development demand is anticipated to increase from 376 afy to 613 afy, resulting in a total system demand of 1,443 afy. The additional pipeline required to connect Neighborhood Park to the main recycled water system is 500 feet (or 0.1 mile) of 6-inch diameter pipeline.

The Butterfield Development demand is anticipated to increase to 900 afy in this phase, which will be served by the BCVWD co-owned wells or alternatively by potable sources or imported surface water if those wells are converted to potable use.

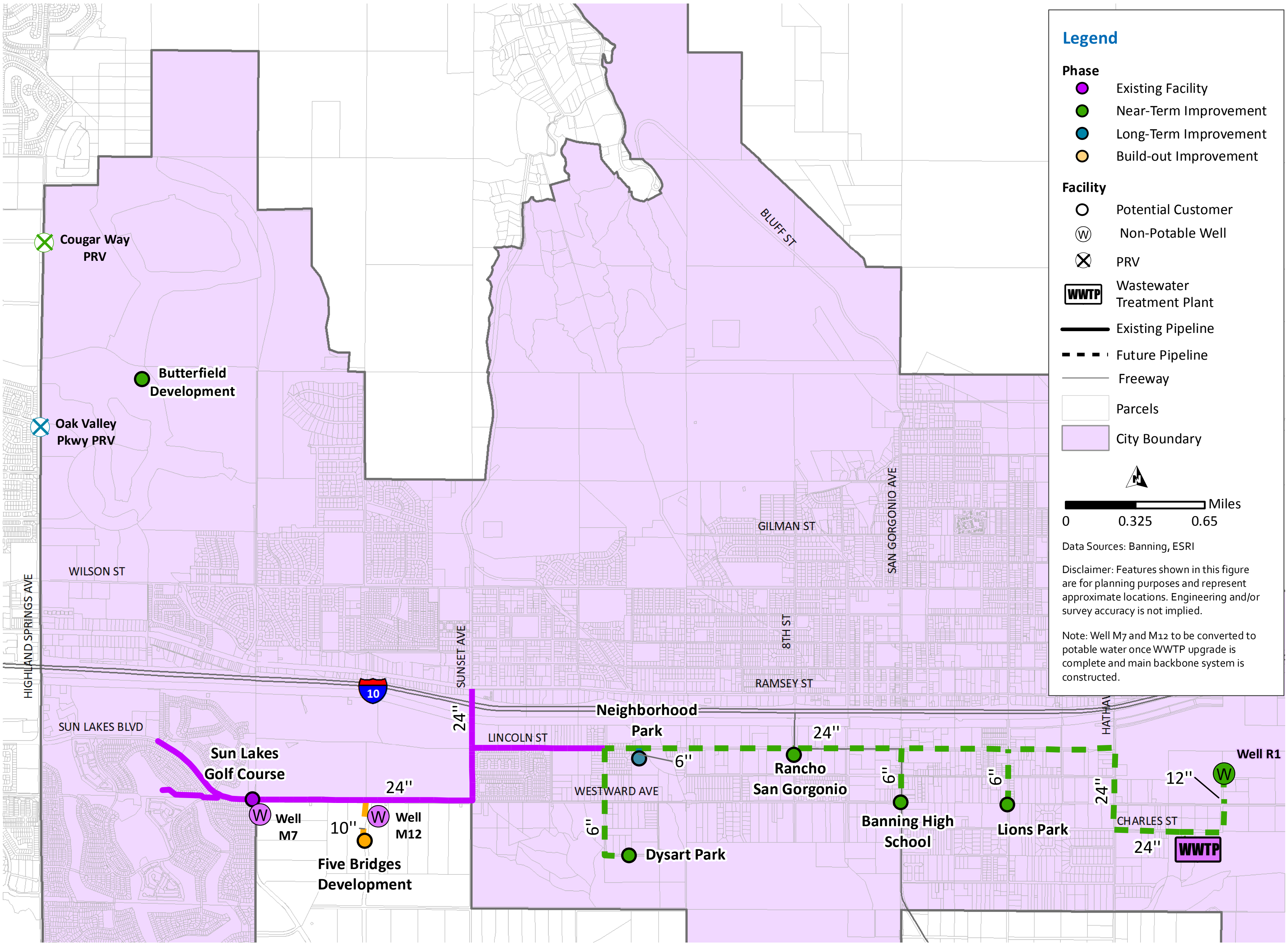

Figure 8.2 Proposed Non-Potable Reuse System

8.2.1.3 Build-Out Phase

The build-out phase connects the Five Bridges Development with a demand of 223 afy, resulting in a total system demand of 1,666 afy. The total additional pipeline required to connect the Five Bridges Development to the main recycled water system is 500 feet (or 0.1 mile) of 10-inch diameter pipeline.

The Butterfield Development demand is anticipated to decrease to 864 afy once the vegetation has matured. This demand which will be served by the BCVWD co-owned wells.

8.2.1.4 Preliminary Cost Estimates

The estimated cost of the required facilities to serve the potential customers is summarized by phase in Table 8.5. Details for the recycled water system cost estimates are in Appendix F.2.

Though the Butterfield Development is not included in the City's main recycled water system, the costs of the two PRVs to connect the Butterfield Development are included in the table. However, the Butterfield Development demand is not included. If the City chooses to connect the Butterfield Development to the main recycled water system, this cost estimate will need to be re-evaluated to include piping and any necessary facilities, such as pump stations and storage.

Table 8.5 Preliminary Cost Estimates for Alternative 1

Phase	Potential Demand(4) (afy)	Capital Cost(1)	O&M Cost(2)	Annual Cost ($/year)	Unit Cost(4) ($/af)
Near-term	1,408	$29,521,000	$987,000	$1,965,000	$1,400
Long-term	35	$138,000	$15,000	$20,000	$600
Build-out	223	$215,000	$90,000	$97,000	$400
Total (by 2040)	**1,443**	$29,659,000	$1,002,000	$1,985,000	$1,400
Total (by Build-out)	**1,666**	$29,874,000	$1,092,000	$2,082,000	$1,200

Notes:
(1) Capital cost includes a construction contingency of 20 percent and additional markups for engineering and administrative costs of 27.5 percent. Cost estimates and cost assumptions are provided in detail in Appendix G.
(2) O&M costs assume 0.5 percent of initial capital cost for pipelines, 2 percent of initial capital cost for pump stations, and 1 percent of initial capital costs for storage tanks. O&M cost also assumes a pump station energy cost of $0.12 per kWh and a recycled water treatment cost of $400 per af.
(3) Annual cost assumes a useful life of 30 years for pump stations, 50 years for storage tanks and 80 years for pipelines, and 3.0 percent interest.
(4) Butterfield Demand is not included in potential demand and unit cost calculations

As listed in Table 8.5, the total estimated capital cost within the planning period of this IMP (year 2040) equates to $29.7 million, while the total capital cost at build-out is estimated to be $29.9 million. The majority of the total capital cost (nearly $29.5 million) occurs in the near-term phase due to the construction of the backbone system from the WWTP to the existing pipelines. The estimated capital costs in the long-term and build-out phases are $138,000 and $215,000, respectively. By the end of the long-term planning phase, the estimated annual cost is approximately $2.0 million with an average unit cost of approximately $1,400/af. At build-out, the estimated annual cost is approximately $2.1 million with an average unit cost of approximately $1,200/af.

8.2.2 Indirect Potable Reuse Alternatives

Indirect Potable Reuse (IPR) with groundwater augmentation involves recharging tertiary or advanced treated wastewater into groundwater aquifers through spreading basins or injection. To provide a preliminary planning level discussion of the IPR alternatives, the following assumptions were made for this cursory level IPR alternatives analysis:

1. The Butterfield Development will be connected to the City's wastewater collection system and contribute to the available recycled water supply availability.
2. Groundwater underflow can be used for diluent blending.
3. The City will receive credit for recharge in the Cabazon Storage Unit and will be able to extract water with this credit upgradient of the recharge basin. Details on the Sustainable Groundwater Management Act (SGMA) are in Appendix F.3.

Based on discussions with City staff, three potential recharge basins for spreading were identified. The locations of the recharge basins and the proposed pipelines are presented on Figure 8.3.

1. WWTP Basin (39.5 acres)
2. North Basin (14.9 acres)
3. Five Bridges Spreading Basin (22.5 acres)

Using the assumptions that were agreed upon with the City, Alternatives 2 through 4 were developed as part of the IPR analysis.

8.2.2.1 Groundwater Basin Recharge Potential

The groundwater basin recharge potential is dependent on the effective recharge area of the basin and the infiltration rate of the soils at the basin. The basin areas were estimated based on an outline of the potential recharge basin area in GIS. The effective recharge areas were assumed to be 75 percent of the total area based on a 3 to 1 ratio side slope and the assumption that infiltration will only occur at the flat bottom of the recharge basin. Though some infiltration may occur at the slopes, this was considered negligible for the purposes of this planning level analysis. With limited data available for the recharge basins, it was assumed that the infiltration rate for the WWTP and Five Bridges Basins would be approximately 1 foot per day (ft/d). Based on input from City staff, the North Basin was assumed to have a higher infiltration rate of 2 ft/d. The recharge potential for each basin is summarized in Table 8.6.

Table 8.6 Groundwater Basin Recharge Potential

Basin Name	Infiltration Rate(1) (ft/d)	Basin Area(2) (acres)	Estimated Effective Recharge Area(3) (acres)	Potential Recharge Capacity(4) (afy)
WWTP Basin	1.0	39.5	29.6	10,813
North Basin	2.0	14.9	11.2	8,158
Five Bridges Basin	1.0	22.5	16.9	6,159

Notes:
(1) Infiltration rate assumed based on City input.
(2) Basin area estimated based on approximate GIS outline.
(3) Effective recharge area assumed to be 75% of estimated basin area.

(4) Assumes recycled water will only be used for recharge throughout the year.

As listed in Table 8.6, the WWTP Basin has an estimated effective recharge area of 29.6 acres, resulting in a recharge potential of approximately 10,813 afy. The North Basin has an estimated effective recharge area of 11.2 acres, resulting in a recharge potential of approximately 8,158 afy. The Five Bridges Basin has an estimated effective recharge area of 16.9 acres, resulting in a recharge potential of approximately 6,159 afy. Based on the recharge potential, all three recharge basins have sufficient capacity to recharge the projected recycled water production through the planning period of this IMP of 5,152 afy (4.6 MGD). At build-out, the Five Bridges Basin does not have sufficient capacity to recharge the entire estimated recycled water production of 6,944 afy (6.2 MGD). To verify the recharge potential at each site, a supplemental hydrogeological study is recommended. Once this study is completed and more definitive information is available, the analysis should be updated to revise any conclusions and/or recommendations made herein.

8.2.2.2 Alternative 2 - Indirect Potable Reuse at WWTP Basin

Alternative 2 involves groundwater recharge at the WWTP evaporation/percolation ponds located east of the WWTP. The ponds consist of an existing City-owned basin with cells. Minimal site improvements, including a turn-out, would be needed. Since the recharge basin is located near the WWTP, the proposed infrastructure would include a 16-inch diameter pipeline with a length of 1,000 feet, a turn-out, and a 100 hp pump to convey the water from the WWTP to the recharge basin. Although, two existing monitoring wells are downgradient of the ponds, the City may need to additional monitoring wells and lysimeters.

This alternative is the most cost-effective because the conveyance distance is minimal. However, the location of the recharge basin is not desired due to its close proximity near the City Boundary and absence of any extraction wells downgradient (southeast) of the City boundary. The City must therefore verify that the amount of water recharged into the basin would be allows to be credited so that the City can extract the recharged volume somewhere else in the storage unit, upgradient of the recharge basin.

In addition, the Morongo Indian Reservation is located southeast of the City boundary. The Morongo Indian Reservation is exempt from the SGMA agreement and is not required to report the amount of water extracted. City staff noted that if the Morongo Indian Reservation demands increase in the future, they are able to extract as much water as needed to meet demands. Thus, without groundwater recharge upstream of the City’s wells, the City is uncertain that they will be able to take advantage of the credited volume when pumping upstream of the WWTP recharge basin.

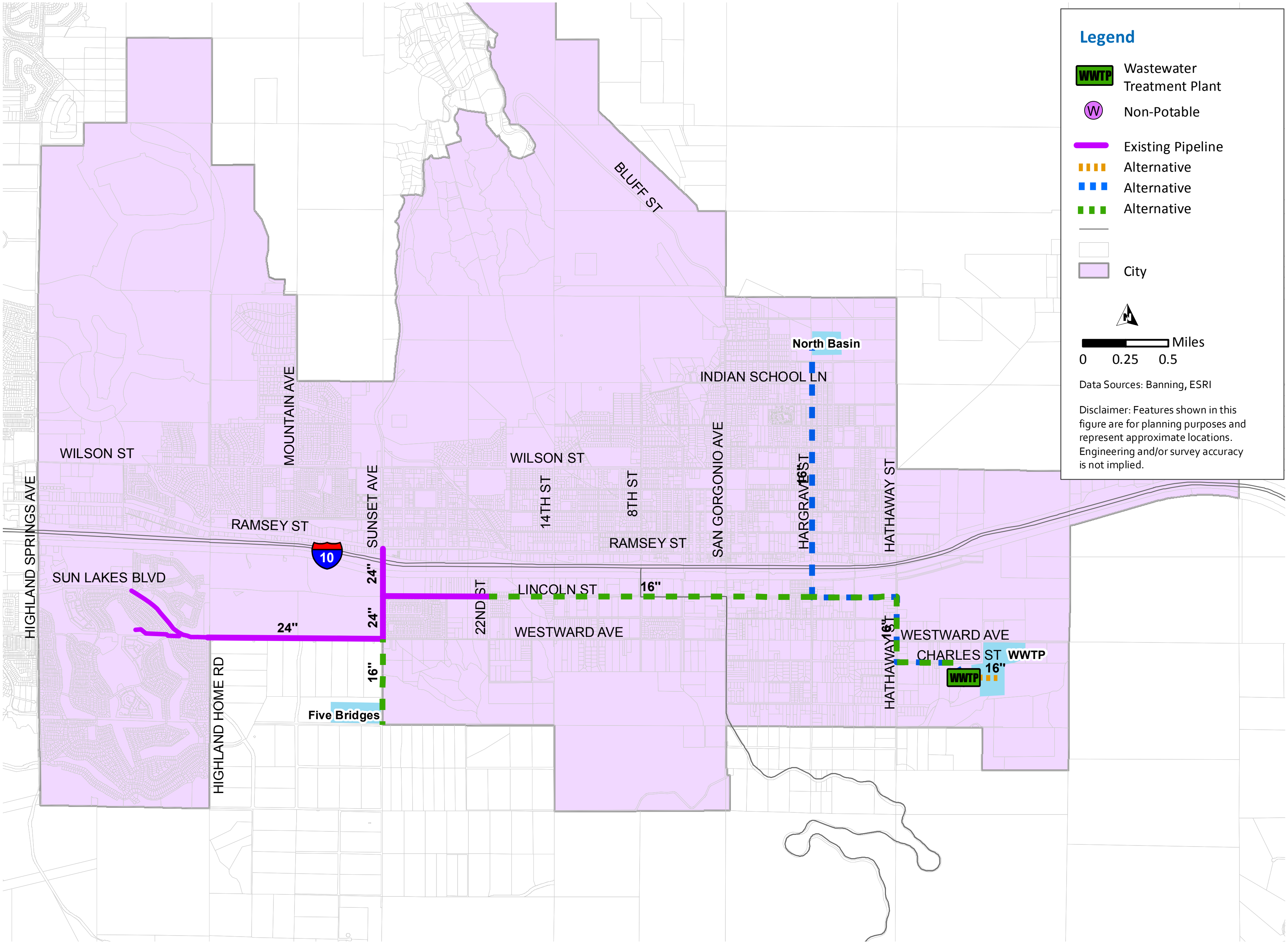

Figure 8.3 Indirect Potable Reuse Alternatives

8.2.2.3 Alternative 3 - Indirect Potable Reuse at North Basin

Alternative 3 involves groundwater recharge at the North Basin located in the northeastern part of the City boundary. This site is not owned by the City. The North Basin was previously used for mining activities and now remains as a large pit. Since the site has been mined and a large pit exists, it is assumed that minimal earthwork would be required. The site improvements included as part of this analysis include a berms on all four sides of the basin, a berm to create two cells for operational flexibility, basin piping, maintenance ramps, security, and monitoring equipment. Since the potential recharge basin is at a higher elevation than the WWTP, an 800 hp pump and 15,000 feet of 16-inch diameter pipeline will be required to convey water from the WWTP to the recharge basin. The City will also need to add monitoring wells and lysimeters.

Although this basin requires a longer pipeline and larger pump than Alternative 2, the City is planning to construct two potable wells downstream of this basin. Assuming the required travel time is met, the potable water wells can be used to extract the water that has been recharged. However, since the land is not owned by the City, land acquisition or a land lease would be required.

8.2.2.4 Alternative 4 - Indirect Potable Reuse at Five Bridges Basin

Alternative 4 involves groundwater recharge at the future Five Bridges Recharge Basin, which is located at the southwest part of the City within the Five Bridges Development. The site will likely be owned by the City. Although the Five Bridges development is not anticipated to occur until the build-out phase, City staff believes that the basin can be constructed beforehand. It is anticipated that site improvements would include earthwork, berms on all four sides of the basin, a berm to create two cells for operational flexibility, maintenance ramps, security, and monitoring equipment. This alternative would require a 600 hp pump and 20,000 feet of 16-inch diameter pipeline to convey water from the WWTP to the recharge basin. The City would also need to add monitoring wells and lysimeters.

Since the pipeline alignment for Alternative 4 extends along the same alignment as the NPR analysis in Alternative 1, potential customers may be connected to the system. This was evaluated as a separate alternative in Section 8.2.3.

Similar to the WWTP Basin Alternative, the City does not have any extraction wells downgradient of the Five Bridges Basin. Thus, the City may not be able to take advantage of the credited volume of water that is recharged into the basin when extracting upgradient of the basin. However, Wells C5 and C6 may potentially benefit from recharge at the Five Bridges Basin since they are located downgradient within the same hydrologic unit (Banning Unit). Further evaluation will be required to determine the use of these wells.

8.2.2.5 Preliminary Cost Estimates

The preliminary cost estimates for Alternatives 2 through 4 are summarized in Table 8.5, while calculations detailed are included in Appendix F.2.

Table 8.7 Preliminary Cost Estimates for Alternatives 2 through 4

Alternative	Estimated Annual Yield(1) (afy)	Capital Cost(2)	O&M Cost(3)	Annual Cost ($/year)	Unit Cost ($/af)
WWTP	3,900	$3,205,000	$283,000	$1,982,000	$508
North Basin	3,900	$16,440,000	$627,000	$2,822,000	$724
Five Bridges	3,900	$20,616,000	$477,000	$2,798,000	$717

Notes:
(1) Annual yield based on WWTP recycled water production and any well production (assuming 24 hour production).
(2) Capital cost includes a construction contingency of 20 percent and additional markups for engineering and administrative costs of 27.5 percent. Cost estimates and cost assumptions are provided in detail in Appendix G.
(3) O&M costs assume 0.5 percent of initial capital cost for pipelines, 2 percent of initial capital cost for pump stations, and 1 percent of initial capital costs for storage tanks. O&M cost also assumes a pump station energy cost of $0.12 per kWh and a recycled water treatment cost of $400 per af.
(4) Annual cost assumes a useful life of 30 years for pump stations, 50 years for storage tanks and 80 years for pipelines, and 3.0 percent interest.

As listed in Table 8.7, the capital costs are estimated to range from $3.2 million to $20.6 million. As mentioned previously, the WWTP Basin location (Alternative 2) is closer to the WWTP and requires minimal site improvements, resulting in a lower capital cost of approximately $3.2 million and the lowest estimated unit cost of approximately $508/af. The North Basin Alternative would require more pumping due to the elevation differences and a longer pipeline, resulting in a capital cost of approximately $16.4 million and a unit cost of approximately $723/af. The Five Bridges Basin Alternative requires the most site improvements, resulting in the highest capital cost of approximately $20.6 million. However, this site is not located at a lower elevation than the North Basin, resulting in a slightly lower unit cost of approximately $717/af.

8.2.3 Hybrid Non-Potable Reuse and Indirect Potable Reuse Alternatives

To maximize the use of recycled water each year and provide more operational flexibility, two alternatives have been developed that include a combination of NPR and IPR. Since the ability to implement NPR is anticipated to be easier and faster than any of the IPR alternatives s, it is assumed that NPR will be implemented in the near-term, The IPR alternative is assumed to be implemented in the long-term to allow sufficient time to acquire property and obtain regulatory approvals such as the ability to extract recharged water upgradient from the recharge site.

Using the assumptions that were agreed upon with the City, Alternatives 5 through 6 were developed as part of the combination NPR and IPR analysis. The proposed layout for each alternative is presented on Figure 8.4.

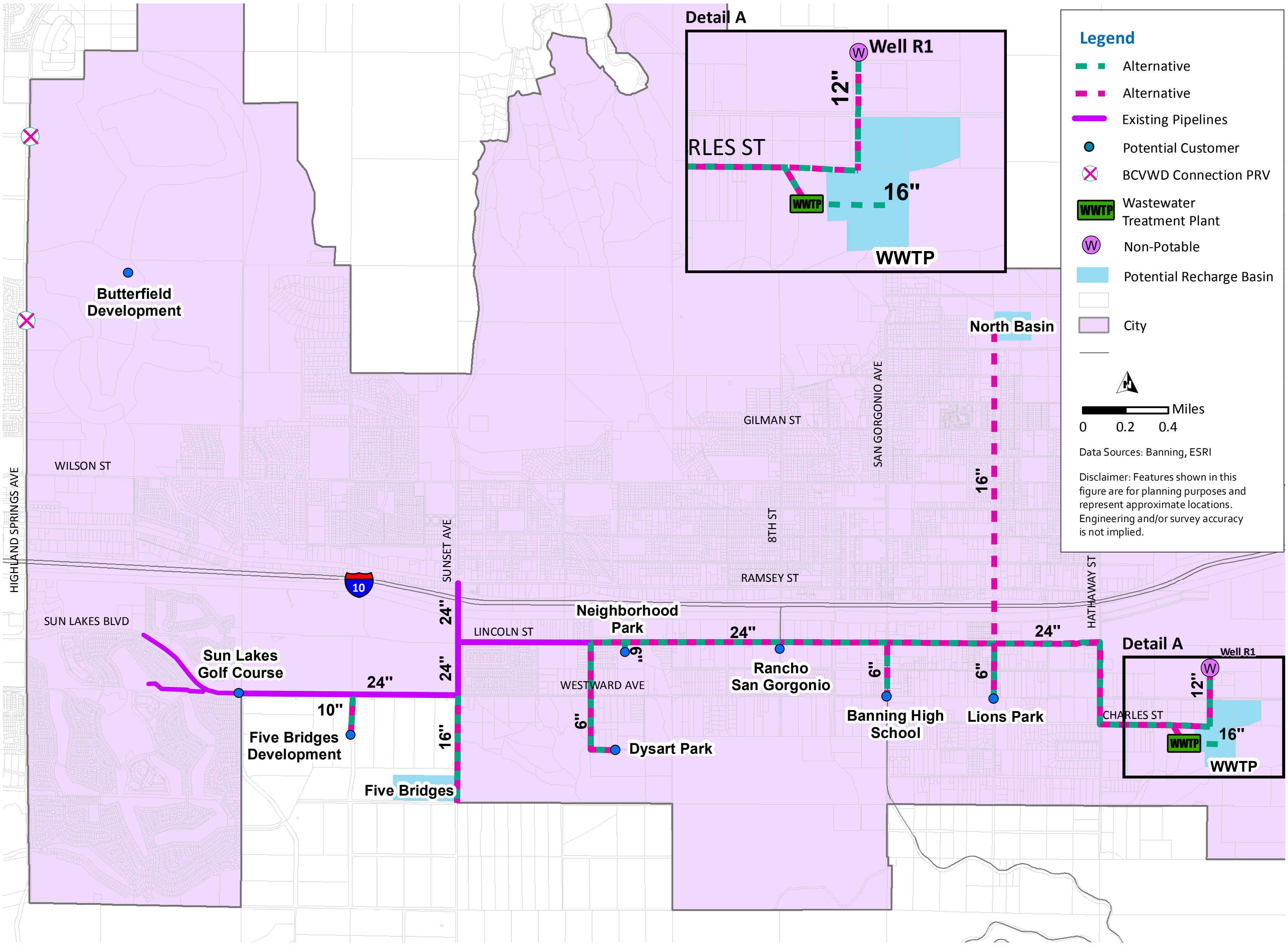

Figure 8.4 Hybrid Non-Potable Reuse and Indirect Potable Reuse Alternatives

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8.2.3.1 Alternative 5 - NPR with IPR at WWTP Ponds and Five Bridges Basin

Alternative 5 is a combination of Alternatives 1, 2, and 4. This alternative includes NPR with a total annual demand of 1,666 afy. The remaining recycled water supply from the WWTP will be used for IPR at either the WWTP Basin or the Five Bridges Recharge Basin. Since the City will be able to maximize the recycled water supply from both the WWTP and Well R-1, the estimated net yield is 5,747 afy.

Alternative 5 requires 30,000 feet (or 5.7 miles) of pipeline ranging in diameter from 6- to 24-inches. As mentioned in Section 8.2.2, minimal site improvements would be required at the WWTP Basin and a new basin would be constructed at Five Bridges. This alternative includes a 700 hp pump and equipping Well R-1. In addition, this alternative includes a Well R-1 forebay at the WWTP with associated transmission mains, which includes approximately 2,500 feet of 12 inch diameter pipeline. The City will also need monitoring wells and lysimeters.

Alternative 5 allows the City to maximize recycled water throughout the year by recharging recycled water during low irrigation demand periods. With connections to two different recharge basins, the City will have operational flexibility and can take one offline for maintenance when needed. However, as mentioned in Alternative 2, the WWTP Basin does not have groundwater wells downstream of the recharge basin. Thus, the City may not be able to benefit from the water that is recharged into the WWTP basin. Similarly, the City does not have any extraction wells downgradient of the Five Bridges Basin. However, Wells C5 and C6 may potentially benefit from recharge at the Five Bridges Basin since they are located downgradient within the same hydrologic unit (Banning Unit).

8.2.3.2 Alternative 6 - NPR with IPR at North Basin and Five Bridges Basin

An additional alternative was evaluated using the North Basin location and the WWTP Basin. Alternative 6 is a combination of Alternatives 1, 3, and 4. This alternative includes NPR with a total annual demand of 1,666 afy. The remaining recycled water supply will be used for IPR at either the North Basin or the Five Bridges Basin. Since the City will be able to maximize the recycled water supply from both the WWTP and Well R-1, the estimated net yield is 5,747 afy.

Alternative 6 requires 36,500 feet (or 6.9 miles) of pipeline ranging in diameter from 6- to 24-inches. As mentioned in Section 8.2.2, site improvements will be required at the North Basin and a new basin will be constructed at Five Bridges. This alternative includes an 800 hp pump and equipping of Well R-1. In addition, this alternative includes a Well R-1 forebay at the WWTP with associated transmission mains, which includes approximately 2,500 feet of 12-inch diameter pipeline. The City will also need monitoring wells and lysimeters.

Alternative 6 allows the City to maximize recycled water throughout the year by recharging recycled water during low irrigation demand periods. With connections to two different recharge basins, the City will have operational flexibility and can take one basin offline for maintenance when needed. Unlike Alternative 5, this alternative gives the opportunity for the City to capture the water that is recharged at the North Basin assuming that travel time requirements are met.

8.2.3.3 Preliminary Cost Estimates

The preliminary cost estimates for Alternatives 5 and 6 are summarized in Table 8.8, while details on the calculations are in Appendix F.2.

Table 8.8 Preliminary Cost Estimates for Alternatives 5 and 6

Alternative	Estimated Annual Yield (afy)	Capital Cost(1)	O&M Cost(2)	Annual Cost(3) ($/year)	Unit Cost ($/af)
5	5,747	$38,766,000	$813,000	$4,152,000	$723
6	5,747	$46,488,000	$1,050,000	$4,648,000	$809

Notes:
(1) Capital cost includes a construction contingency of 20 percent and additional markups for engineering and administrative costs of 27.5 percent. Cost estimates and cost assumptions are provided in detail in Appendix G.
(2) O&M costs assume 0.5 percent of initial capital cost for pipelines, 2 percent of initial capital cost for pump stations, and 1 percent of initial capital costs for storage tanks. O&M cost also assumes a pump station energy cost of $0.12 per kWh and a recycled water treatment cost of $400 per af.
(3) Annual cost assumes a useful life of 30 years for pump stations, 50 years for storage tanks and 80 years for pipelines, and 3.0 percent interest.

As listed in Table 8.8, the estimated capital cost ranges from approximately $38.8 million to $46.5 million. Since the WWTP Basin location (Alternative 5) is much closer to the WWTP, less piping and pumping is required, resulting in a lower capital cost and a lower unit cost of $723/af. The North Basin (Alternative 6) requires more pumping due to the higher elevation and a longer pipeline, resulting in an estimated capital cost of nearly $46.5 million and a higher unit cost of approximately $809/af.

8.2.4 Satellite Treatment Plant Alternative

The Butterfield Specific Plan identifies a satellite treatment plant as a potential alternative for serving recycled water to the development. As mentioned in Chapter 7, the satellite treatment plant would be located at the southeast corner of the Butterfield Development at Wilson Street and Highland Home Road. This satellite treatment plant would treat the wastewater from the Butterfield Development to tertiary standards for irrigation use within the development. Since the development's wastewater would be treated at the satellite plant, this alternative would result in a reduction in wastewater flows and recycled water production at the main WWTP. This scenario was evaluated as a minimum recycled water demand scenario in which demands are limited by the reduced sewer flows.

Table 8.9 WWTP Recycled Water Availability with Satellite Plant

Planning Year	Recycled Water Production Capacity w/o Satellite Plant(1) (MGD)	Recycled Water Production Capacity w/ Satellite Plant(2,3)	
		(MGD)	(afy)
Existing (2017)	1.6	1.6	1,823
Near-term (2025)	2.4	2.1	2,313
Long-term (2040)	3.5	2.9	3,237
Build-out	5.1	4.5	5,071

Notes:
(1) Values from Table 8.2.
(2) See Chapter 7 for assumptions and methodology.
(3) Assumes 10 percent losses to treat to secondary standards and an additional 10 percent losses to treat to tertiary standards.

As mentioned in Chapter 7, approximately 93 percent of Butterfield's ADWF would be routed to the satellite plant, while the remaining flow and solids would be conveyed to the City's wastewater collection system. The recycled water production from the WWTP with only

7 percent of Butterfield's ADWF entering the plant is summarized in Table 8.10, while calculations details can be found in Appendix F.4.

As listed in Table 8.9, the estimated recycled water production from the WWTP in the near-term would be reduced from 2.4 MGD (or 1,823 afy) to 2.1 MGD (or 2,313 afy) if wastewater flows from the Butterfield Development would be routed to a satellite plant. By the end of the planning period of this IMP (year 2040), the estimated recycled water production from the WWTP would be reduced from 3.5 MGD (or 3,892 afy) to 2.9 MGD (or 3,237 afy). At build-out, the estimated recycled water production from the WWTP would be reduced from 5.1 MGD (or 5,761 afy) to 4.5 MGD (or 5,040 afy). With the decrease in recycled water supply availability, the customer phasing would be shifted slightly. The projected recycled water demands within each phase are summarized in Table 8.10, while details are presented in Appendix F.1.

Table 8.10 Lower WWTP Recycled Water Availability Scenario

Phase	Pipeline Length (ft)	Potable Water Conversion Demands (afy)	New Demands (afy)	**Total NPR Demands(1) (afy)**
Near-term	24,500	1,151	257	1,408
Long-term	0	0	0	0
Build-out	1,000	0	258	258
Total	**25,500**	**1,151**	**480**	**1,666**

Note:
(1) Available supply must meet MDD conditions. Detailed calculations in Appendix F.1.

As listed in Table 8.10, the total annual recycled water demand is the same as Alternative 1. However, the phasing of Neighborhood Park is moved to the build-out phase due to insufficient supply availability. In addition, excess supply is not available for IPR and the City would not be able to combine NPR with IPR.

Since the treated wastewater flows from the Butterfield Development may not be sufficient to supply the projected recycled water demands for that new development, the demands listed in Table 8.10 may need to be reduced unless additional wastewater flows are diverted from other areas within the City to the satellite treatment plant.

8.2.4.1 Butterfield Satellite Plant Supply and Demand Balance

To evaluate whether additional wastewater flows will need to be diverted to supplement the satellite treatment plant, a recycled water supply and demand balance was performed. Similar to Alternative 1, it was assumed that the satellite plant effluent flows must be able to meet MDD conditions within the Butterfield Development. The recycled water supply and demand within the Butterfield development are summarized by phase in Table 8.11. Detailed calculations are presented in Appendix F.4.

Table 8.11 Butterfield Development Recycled Water Supply and Demand by Phase

Description	Near-Term	Long-Term	Build-out
Satellite Plant Recycled Water Supply(1) (MGD)	0.2	0.5	0.6
MDD(2) (MGD)	0.4	2.3	2.2
Deficit (MGD)	**(0.2)**	**(1.8)**	**(1.6)**

Notes:
(1) Satellite Plant supply assumes influent is 93 percent of Butterfield's wastewater flow, 10 percent losses to secondary standards, and 10 percent losses to tertiary standards.
(2) Demand breakdown by phase provided by Pardee. Assumed MDD peaking factor of 2.8.

As listed in Table 8.11, the satellite plant is estimated to have insufficient recycled water supply availability to meet MDD conditions. If the developer and City choose to build a satellite plant instead of utilizing the BCVWD co-owned wells, the developer would need to identify other options to meet MDD conditions, such as purchasing imported water from SGPWA or diverting additional wastewater flow from nearby homes to supplement influent flows to the satellite plant. Alternatively, the Butterfield Development can augment supplies from the BCVWD co-owned wells or potable water system during MDD conditions. A feasibility analysis is recommended to further evaluate this alternative.

8.3 Summary of Recommendations

The overall objective of the future recycled water system is to maximize the usage of recycled water within the service area by reaching the customers that are within a reasonable distance of the existing or planned recycled water distribution system. Along with improving local supply reliability, a primary goal is to develop a system that is less costly than imported water.

A summary of alternatives analyzed and the recommended system improvements are discussed in the proceeding sections.

8.3.1 Alternative Analysis Recommendation

A total of six alternatives were identified assuming that the Butterfield Development would be served with the BCVWD co-owned non-potable wells. In the event that the wells are converted to potable water use, the Butterfield Development recycled water demand will need to be served by alternative sources, such as recycled water from BCVWD, potable sources, or imported surface water. The annual yield and estimated costs for the alternatives are summarized in Table 8.12.

Table 8.12 Summary of Alternatives

Alternative	Annual Yield (afy)	Capital Cost ($M)	Amortized Cost with O&M ($/year)	Unit Cost ($/af)
1	1,666	$29,874,000	$2,082,000	$1,200
2	3,900	$3,205,000	$1,982,000	$508
3	3,900	$16,440,000	$2,822,000	$724
4	3,900	$20,616,000	$2,798,000	$717
5	5,747	$38,766,000	$4,152,000	$723
6	5,747	$46,488,000	$4,648,000	$809

As listed in Table 8.12, the unit costs of the alternatives range from $508/af to $1,200/af. Based on Metropolitan Water District of Southern California's (MWD) forecast, the current cost of untreated Tier 2 imported water is estimated at $781/af and is projected to increase to $1,030/af by year 2025. However, based on discussions with City staff, the cost of imported water in this region from SGPWA is higher than purchasing directly from MWD and was recently already $1,300/af. Therefore, all alternatives are estimated to be more cost effective than purchasing imported water. Moreover, there is a value of providing supply reliability within the region.

Based on the alternative analysis performed, the recommended alternative is Alternative 5, which is presented on Figure 8.5. With the combination of both NPR and IPR, the City would have seasonal flexibility and would be able to maximize the use of recycled water throughout the year. In addition, using two basins for recharge would provide the City with increased operational flexibility. Although, the City does not have any extraction wells downgradient of the Five Bridges Basin, Wells C5 and C6 may potentially benefit from recharge at the Five Bridges Basin since they are located downgradient within the Banning Unit. In addition, the City would be able to use the main backbone system for NPR to convey water to the Five Bridges Basin. The unit cost of $723/af for Alternative 5 is estimated to be more cost effective when compared to the cost of imported water in 2018 ($781/af).

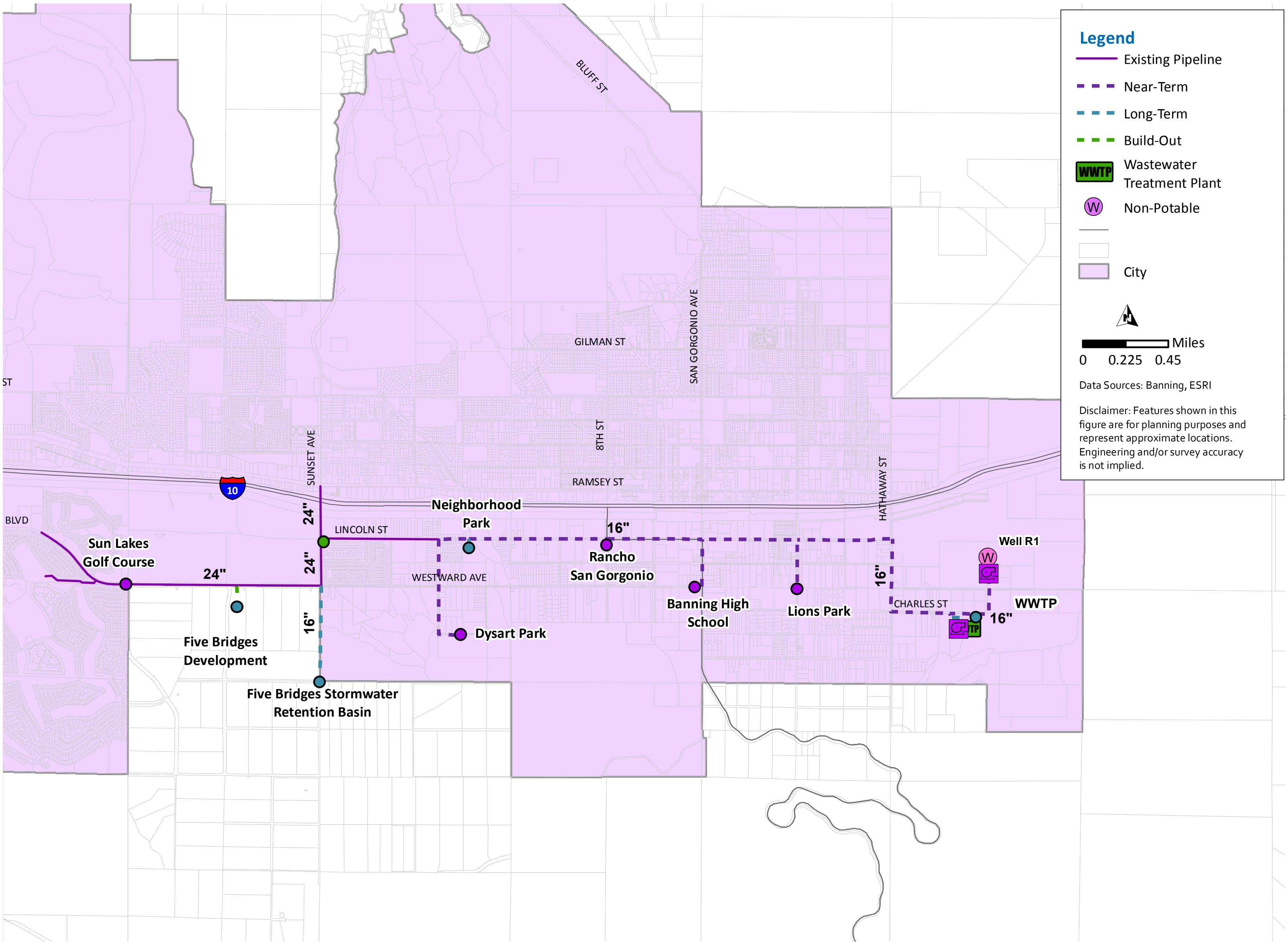
Due to additional hydrogeological studies required to evaluate the Five Bridges Basin and WWTP Basin Alternative, the implementation of Alternative 6 is recommended to occur in four phases. Phases 1 through 3 would occur within the near-term and Phase 4 would occur within the long-term. For CIP planning purposes, the following activities are anticipated to occur during each phase:

- Phase 1: The NPR system would be constructed, starting with equipping Well R-1 and connecting Well R-1 to Lions Park and Banning High School.
- Phase 2: The backbone pipeline would be extended to the RSG development. The WWTP expansion with the necessary treatment upgrades would be completed, along with the construction of the WWTP recycled water pump station.
- Phase 3: The backbone pipeline would be extended to connect to the existing pipelines in Lincoln Street and connect Dysart Park to the main recycled water system.
- Phase 4: The City would begin the construction of the pipelines to the recharge basins for IPR use.

The detailed costs for Alternative 5 are presented in Chapter 9 of the IMP and Appendix F.2.

8.3.2 Satellite Treatment Plant Recommendation

A new satellite plant at the Butterfield development would decrease the flows at the WWTP, resulting in less flow in the City's recycled water system. The flows at the Butterfield Development are not sufficient to meet recycled water demands and will need to be supplemented with additional flows from nearby neighborhoods or potable water. A satellite plant would also add a second treatment plant for City staff to operate and maintain, which increases operational cost and requires additional staff. Thus, it is not recommended to build a satellite plant at the Butterfield Development. Instead, a more cost effective solution would be to serve the recycled water demand with the BCVWD co-owned non-potable wells. In the event that the wells are converted to potable water use, the Butterfield Development recycled water demand will need to be served by alternative sources, such as recycled water from BCVWD if supplies are available, potable sources, or imported surface water. This would require a new pipeline from the terminus of their system in Cherry Valley.


Legend
Existing Pipeline
Near-Term
Long-Term
Build-Out
WWTP
Wastewater Treatment Plant
W
Non-Potable
City
Miles
0
0.225
0.45
Data Sources: Banning, ESRI
Disclaimer: Features shown in this figure are for planning purposes and represent approximate locations. Engineering and/or survey accuracy is not implied.
BLUFF ST
GILMAN ST
SAN GORGONIO AVE
8TH ST
RAMSEY ST
SUNSET AVE
HATHAWAY ST
LINCOLN ST
WESTWARD AVE
CHARLES ST
ST
BLVD
10
Neighborhood Park
Sun Lakes Golf Course
24"
24"
24"
16"
16"
16"
16"
Rancho San Gorgonio
Banning High School
Lions Park
Dysart Park
Five Bridges Development
Five Bridges Stormwater Retention Basin
Well R1
WWTP

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Chapter 9

CAPITAL IMPROVEMENT PLAN

This chapter presents the recommended capital improvement plan (CIP) for the potable water, wastewater, and recycled water systems. The proposed CIP presents improvement projects based on the water, wastewater, and recycled water system evaluations described in Chapters 6, 7, and 8 of this Integrated Master Plan (IMP). The planning horizon of this master plan is year 2040. The CIP is divided into a near-term, long-term, and build-out phases. The near-term CIP includes the years 2018 through 2025, the long-term CIP includes the years 2026 through 2040, and build-out occurs outside of the planning horizon and includes years 2041 and beyond.

This chapter starts with a summary of the cost-estimating assumptions. Subsequently, the potable water, wastewater, and recycled water CIPs are presented with a summary of recommendations on project prioritization. This chapter is concluded with a combined CIP that presents the total estimated cost of all three systems.

9.1 Cost Estimating Assumptions

The cost estimates presented in this IMP are opinions developed from bid tabulations, cost curves, information obtained from previous studies, and Carollo's experience on other similar projects. The costs are based on an *Engineering News Record* Construction Cost Index (ENR CCI) 11936 (Greater LA Index, October 2017).

The construction costs are representative of system facilities under normal construction conditions and schedules. Costs have been estimated for public works construction.

9.1.1 Cost Estimating Accuracy

The cost estimates presented in the CIP have been prepared for general master-planning purposes and for guidance in project evaluation and implementation. Final costs of a project will depend on actual labor and material costs, competitive market conditions, final project scope, implementation schedule, and other variable factors such as preliminary alignment generation, investigation of alternative routings, and detailed utility and topography surveys.

The Association for the Advancement of Cost Engineering (AACE) defines an Order-of-Magnitude Estimate, deemed appropriate for master plan studies, as an approximate estimate made without detailed engineering data. It is normally expected that an estimate of this type would be accurate within plus 50 percent to minus 30 percent. This section presents the assumptions used in developing order-of-magnitude cost estimates for the recommended facilities. As projects proceed into the preliminary design and design stages, estimates are refined when conditions become known.

9.1.2 Capital Cost Development

Capital costs developed for this IMP are estimated by multiplying the estimated construction cost with various markups. The various cost components used in the development of capital cost estimates are described below.

The cost estimates presented in the CIP have been prepared for general master-planning purposes

9.1.2.1 Baseline Construction

This is the total estimated construction cost, in dollars, of the proposed improvement projects. Baseline construction costs were calculated by multiplying the estimated number of units by the unit cost, such as length of pipeline times the average cost per lineal foot of pipeline. The majority of unit construction costs used for this IMP are presented in Section 9.1.3.

9.1.2.2 Estimated Construction Cost

Contingency costs must be reviewed on a case-by-case basis because they will vary considerably with each project. Consequently, it is appropriate to allow for uncertainties associated with the preliminary layout of a project. Such factors as unexpected construction conditions, the need for unforeseen mechanical items, and variations in final quantities are a few of the items that can increase project costs for which it is wise to make allowances in preliminary estimates. To assist the City in making financial decisions for these future construction projects, contingency costs will be added to the planning budget as percentages of the total construction cost, divided into two categories: Estimated Construction Cost and Capital Improvement Cost.

Since knowledge about site-specific conditions of each proposed project is limited at the master-planning stage, a 30-percent contingency was applied to the Baseline Construction Cost to account for unforeseen events and unknown conditions. This contingency accounts for unknown site conditions such as poor soil, unforeseen conditions, environmental mitigations, and other unknowns and is typical for master planning projects. The Estimated Construction Cost for the proposed potable, wastewater, and recycled water system improvements consists of the Baseline Construction Cost plus the 30-percent construction contingency.

9.1.2.3 Capital Improvement Cost

Other project contingency costs include costs associated with engineering, construction-phase professional services, and project administration. Engineering services associated with new facilities include preliminary investigations and reports, right-of-way (ROW) acquisition, foundation explorations, preparation of drawings and specifications during construction, surveying and staking, sampling of testing material, and start-up services. Construction-phase professional services cover such items as construction management, engineering services, materials testing, and inspection during construction. Finally, there are project administration costs, which cover such items as legal fees, environmental/California Environmental Quality Act (CEQA) compliance requirements, financing expenses, administrative costs, and interest during construction.

The cost of these items can vary, but, for the purpose of this study, it is assumed that the other project contingency costs will equal approximately 27.5 percent of the Estimated Construction Cost.

As shown in the following sample calculation of the capital improvement cost, the total cost of all project construction contingencies (construction, engineering services, construction management, and project administration) is 65.8 percent of the baseline construction cost. Calculation of the 65.8 percent is the overall markup on the baseline construction cost to arrive at the capital improvement cost. It is not an additional contingency.

Example:

Baseline Construction Cost	**$1,000,000**
Construction Contingency (30%)	$300,000
Estimated Construction Cost	**$1,300,000**
Engineering Cost (10%)	130,000
Construction Management (10%)	130,000
Project Administration (7.5%)	$97,500
Capital Improvement Cost	**$1,657,500**

9.1.3 Unit Construction Cost

Due to the large number of types of projects presented in this IMP, there are many unit construction costs utilized. The following unit construction costs are presented below:

- Pipeline Cost (see Table 9.1)
- Pump Station Cost (see Table 9.2)
- Pressure-Reducing Stations (see Table 9.3)
- Reservoir Cost (see Table 9.4)

It should be noted that these unit costs, along with some project-specific unit costs, are listed in the detailed summary CIP tables presented at the end of this chapter. A summary of miscellaneous unit cost assumptions is presented in Table 9.5. Consistent with typical master-planning cost estimating, pipeline materials are not specified at this time. Although pipeline materials are not specified in the IMP, the City currently utilizes ductile iron pipe (DIP) for the potable and recycled water systems and extra strength vitrified clay pipe (VCP) for the sewer system. Storage reservoirs are assumed to be steel cylindrical tanks, as concrete reservoirs are typically more costly. Pump stations costs are based on total horsepower. For conservative planning purposes, no differentiation is made between new pump stations or pump station upgrades, as the condition of existing pump stations that can require upgrades can vary greatly.

Table 9.1 Unit Construction Costs - Pipelines

Pipe Size (inches)	Unit Construction Cost[1] ($/LF)
Potable and Recycled Water Mains[2]	
6"	$175
8"	$190
10"	$240
12"	$250
14"	$330
16"	$330
18"	$375
20"	$420
24"	$475
30"	$500
36"	$595
Sewer Gravity Main[3]	
8"	$175
10"	$180
12"	$190
14"	$205
15"	$210
16"	$210
18"	$225
20"	$275
21"	$285
24"	$310
27"	$350
30"	$385
33"	$435
36"	$485
Sewer Force Mains	
6"	$175
8"	$175
12"	$195

Notes:
(1) ENR CCI 11936 (Los Angeles, October 2017).
(2) The unit costs may be reduced in locations with fewer utility conflicts and unpaved roads. This will be determined at the preliminary design level of the project.
(3) The Sewer Gravity Mains at interstate crossings increases to the following unit costs: 12/24" is $515/LF, 15/30" is $550/LF, 18/30" is 595/LF, and 21/42" is $765/LF.

Table 9.2 Unit Construction Costs - Pump Stations

Station Size (HP)	Unit Construction Cost(1) ($/HP)
100 hp and smaller	$5,500
150 to 500 hp	$3,500

Note:
(1) ENR CCI 11936 (Los Angeles, October 2017).

Table 9.3 Unit Construction Costs - Pressure Reducing Valves

Type	Unit Construction Cost(1) ($/PRV)
Small (1-2 valves <8")	$103,000
Medium (2-3 valves 8" and up)	$205,500
Large (3-4 valves 12" and up)	$308,000

Note:
(1) ENR CCI 11936 (Los Angeles, October 2017).

Table 9.4 Unit Construction Costs - Reservoir Storage

Type (MG)	Unit Construction Cost(1) ($/gallon)
<1	$2.75
1 to 3	$2.25
3 to 5	$2.00
5 to 10	$1.75

Note:
(1) ENR CCI 11936 (Los Angeles, October 2017).

Table 9.5 Unit Construction Costs - Miscellaneous Items

Type	Unit Construction Cost(1) ($/each)
Well Rehabilitation (per Well)	$260,000
Equipping Well	$1,000,000
New Well(2)	$2,565,000
Monitoring Well & Lysimeter	$310,000
Variable Frequency Drive (VFD)	$100,000
Backup Power Generator (per PS)	$260,000

Notes:
(1) Based on estimates from previous planning and construction projects.
(2) Does not include pipeline.

9.1.4 CIP Phasing

The proposed capital improvements are prioritized based on their urgency to mitigate existing deficiencies, condition issues, and providing service for future growth. As previously mentioned, there are two implementation phases within the planning horizon of the IMP. The near-term phase extends from the years 2018 through 2025 and the long-term phase extends from the years 2026 through 2040. Projects outside of the planning horizon were placed in the build-out phase, which occurs in 2041 and beyond.

It should be noted that several projects have been pushed into the long-term planning period (2026 to 2040) or build-out (2041 and beyond) due to funding constraints. It should be noted that the current water rates will make it difficult to fund the projects listed within the near-term planning period. Therefore, the CIP will need to be revised periodically to push projects out to later years. Other select projects may also be moved at the discretion of City staff. Future rate increases to raise capital funds, additional contributions from developers, and grant funding can potentially accelerate projects to the near-term planning phase.

9.2 Potable Water System CIP

The improvement projects included in the potable water CIP are a compilation of the recommendations made in Chapter 6 of this IMP. The water system CIP includes the following project categories:

- Capacity and Reliability Improvements
  - Pipelines
  - Fire Flow Improvements
  - Booster Pumping Stations
  - Storage Reservoirs
  - Wells
  - Valves
- Repair and Rehabilitation (R&R) Improvements
  - Pipelines
  - Storage Reservoirs
  - Wells
  - Valves
  - Multi-Site Projects
- Other Projects

A detailed list of potable water CIP projects with project descriptions, sizing, and cost estimating information is provided at the end of this chapter in Table 9.10 and project locations are shown on Figure 9.9 with the exception of build-out projects that were triggered outside of the planning horizon of this master plan. The key project phasing assumptions and cost summarizes are presented below.

9.2.1 Potable Water CIP by Phase

The potable water system CIP is summarized by improvement category and phase in Table 9.6, while phasing is graphically shown on Figure 9.1.

Table 9.6 Summary of Potable Water Improvement Costs by Project Category

Project Category	Near-Term 2018-2025 ($ Million)	Long-Term 2026-2040 ($ Million)	Build-Out 2041 &Beyond ($ Million)(2)	Total ($ Million)
Capacity & Reliability	$59.6	$44.8	$69.5	$173.8
R&R Improvements	$10.4	$93.0	$254.3	$357.7
Other	$38.7	$-	$-	$38.7
Grand Total	**$108.7**	**$137.8**	**$323.8**	**$570.2**
Number of Years	8	15	N/A	N/A
Total Annual Cost ($/year)	**$13.6**	**$9.2**	**N/A**	**N/A**
Anticipated Developer Funding	$33.6	$32.6	$69.5	$135.7
City Funded CIP	$75.0	$105.2	$254.3	$434.5
City Annual Cost ($/year)	**$9.4**	**$7.0**	**N/A**	**N/A**

(3) Notes:
(1) Numbers may vary slightly due to rounding.
(2) The costs per year do not include build-out since the implementation timeline is unknown and may be outside of the 2040 planning horizon.

As listed in Table 9.6 and on Figure 9.1, the potable water CIP through the year 2040 is $246.5 million, which is approximately 43 percent of the total CIP (or $570.2 million) through build-out. The near-term projects account for about $108.7 million, which equates to roughly $13.6 million per year through year 2025. The long-term projects account for about $137.8 million, which equates to roughly $9.2 million per year from 2026 through 2040. The average estimated capital cost for the 23-year planning horizon of this IMP is $10.7 million per year, which excludes the build-out improvement projects that equate to approximately $323.8 million (or 57 percent) of the total CIP. Since the timing of the build-out projects is unknown, the costs are not included in the average annual expenditures.

The vast majority of the improvement projects ($434.5 million) are associated with City funded CIP projects that occur within the build-out phase, which is outside of the planning horizon of this IMP. It is anticipated that approximately $135.7 million in developer funding will be provided for future growth within the City. The developer funding equates to approximately 26 percent (or $66.2 million) of the CIP through the year 2040. With developer funding, the City's anticipated average annual expenditures equate to $9.4 million in the near-term phase and $7.0 million in the long-term phase, or an overall average of $7.8 million within the 23-year planning horizon of this IMP.

In addition, as shown on Figure 9.2, the majority of the proposed improvements consist of R&R projects, which equate to approximately 62.7 percent of the total CIP cost. Capacity and reliability (C&R) improvements account for approximately 30.5 percent and other projects account for approximately 6.8 percent of the total CIP cost.

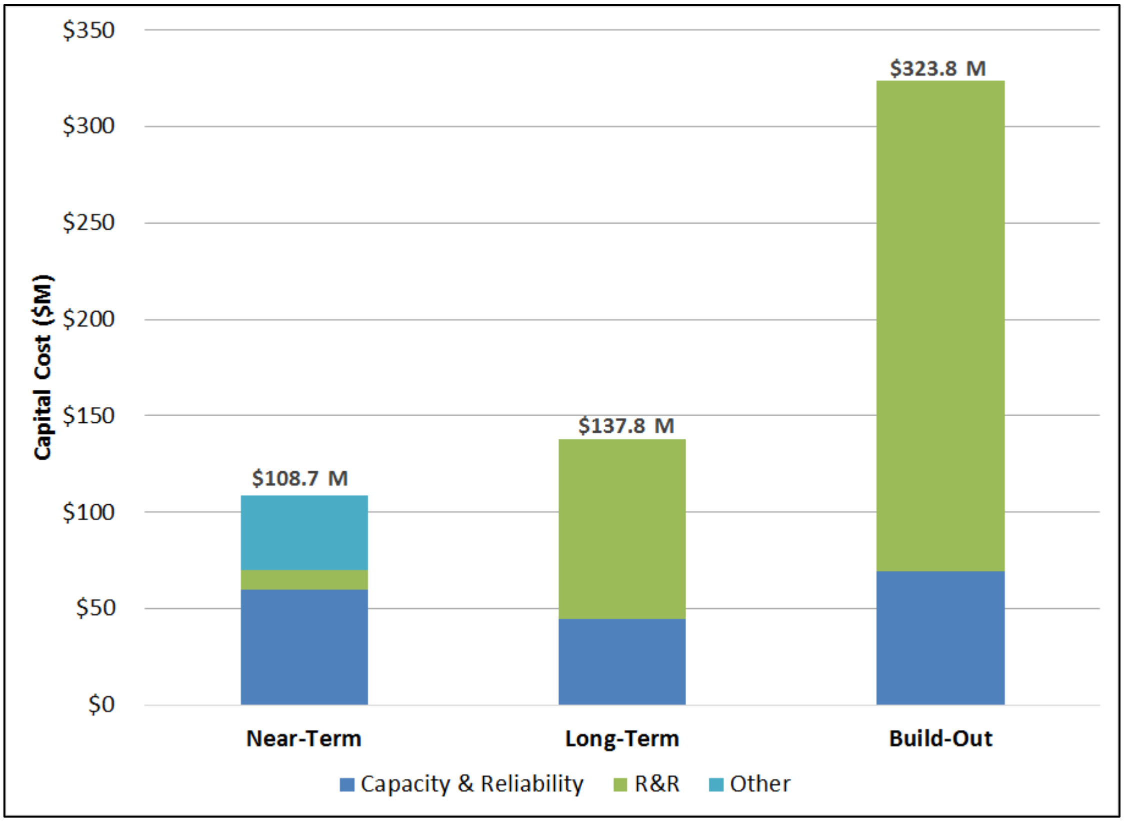

Figure 9.1 Potable Water CIP by Improvement Category and Phase

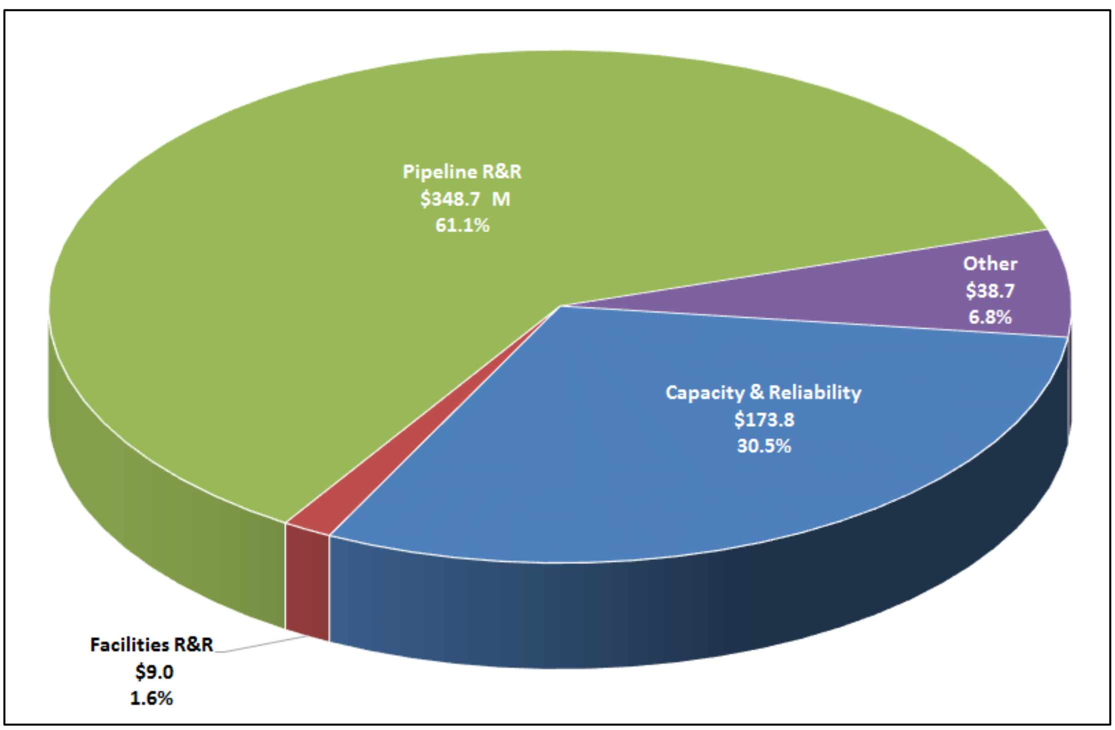

Figure 9.2 Potable Water CIP by Project Type

9.2.1.1 Near-Term Projects

As summarized in Table 9.6 and shown on Figure 9.1, the cost for the near-term projects is approximately $108.7 million, which includes $59.6 million for capacity and reliability improvements, $10.4 million for R&R, and $38.7 million for other CIP projects. The vast majority (or 69 percent) of the projects within the near-term mitigate existing capacity deficiencies and include site improvements throughout the City's potable water system. Some of existing system deficiencies were triggered due to changes in criteria and requirements that occurred after the projects were constructed. The remaining projects (or 31 percent) are affiliated to future growth within the City. Individual project details and the allocation of existing and future user benefits for each of these projects are listed in Table 9.10.

A majority of the recommended improvements within the near-term are capacity and reliability projects, which equate to nearly 55 percent of the near-term CIP. The projects include approximately 8.8 miles of new transmission mains, twenty-three (23) fire flow projects, two (2) pump station projects with a total horsepower of 260, two (2) storage reservoirs with a total capacity of 5.5 MG, one (1) new well with total capacity of 1,400 gpm, the conversion of well M-7 to potable water, and three (3) PRV projects. New altitude valves have also been included, which are in the City's existing CIP.

The R&R improvement projects equate to approximately 10 percent of the near-term CIP and include site improvements at various locations throughout the City's distribution system, which were identified as part of the Condition Assessment Technical Memorandum (TM) in Appendix D. The site improvements vary in complexity and include items such as seismic evaluations and upgrades, site security, emergency power, and safety improvements. In addition, the repair and replacement of pipelines throughout the City's distribution system have been included as part of the R&R projects. The City has already identified three (3) pipeline replacements, which are currently included in the City's existing CIP.

Other projects equate to approximately 36 percent of the near-term CIP and include a pipeline rehabilitation asset study and various projects that are currently included in the City's existing CIP. The purpose of the pipeline rehabilitation asset study is to prepare a pipeline replacement plan based on field testing and existing maintenance records, which would assist in refining the pipeline replacement R&R cost estimates. In addition, one of the City's existing key projects that may be implemented within the near-term phase is the Chromium 6 treatment pilot study, design, and construction. This project is pending based on potential changes to regulatory guidelines.

9.2.1.2 Long-Term Projects

As summarized in Table 9.6 and shown on Figure 9.1, the cost for the long-term projects is approximately $137.4 million, which includes $44.4 million for capacity and reliability improvements and $93.0 million for R&R projects. The vast majority (or 68 percent) of the projects within the long-term include R&R improvements at various sites within the City's distribution system and pipeline replacements. The detail for each of these projects is listed in Table 9.10.

The capacity and reliability improvement projects equate to approximately 32 percent of the long-term CIP. The capacity and reliability improvements are attributed to future growth and supply reliability. Projects include approximately 3.6 miles of new transmission mains, demolishing the existing Mountain pump station, the addition of VFDs to Wells C6 and C8, two (2) pump station projects with a total horsepower of 200, three (3) storage reservoirs with a total capacity of 6.5 MG, one (1) new well with total capacity of 1,800 gpm, the conversion of well M-12 to potable water, and new pressure reducing valves for re-zoning.

The R&R improvement projects equate to approximately 68 percent of the long-term CIP and include site improvements at various locations throughout the City's distribution system, which were identified as part of the Condition Assessment Technical Memorandum (TM) in Appendix D. The site improvements vary in complexity and include items such as seismic evaluations and upgrades, site security, emergency power, and safety improvements. In addition, the repair and replacement of pipelines throughout the City's distribution system have been included as part of the R&R projects. It should be noted that the timing of the pipeline replacements may change upon completion of the pipeline rehabilitation asset study, which is included as part of the near-term projects.

9.2.1.3 Build-Out Projects

As summarized in Table 9.6 and shown on Figure 9.1, the cost for build-out projects is approximately $323.8 million, which includes $69.5 million for capacity and reliability improvements and $254.3 million for R&R projects. The vast majority (or 73 percent) of the projects within the build-out phase are pipeline replacements. The detail for each of these projects is listed in Table 9.10.

The capacity and reliability improvement projects equate to approximately 21 percent of the built-out phase of the CIP. The capacity and reliability improvements are attributed to future growth and supply reliability. Projects include approximately 4.2 miles of new transmission mains, two (2) new pump stations with a total horsepower of 160, four (4) storage reservoirs with a total capacity of 13.0 MG, and three (3) new wells with total capacity of 5,400 gpm.

The R&R improvement projects equate to approximately 79 percent of the build-out phase of the CIP and include pipeline replacements. It should be noted that the timing of the pipeline replacements may change upon completion of the pipeline rehabilitation asset study, which is included as part of the near-term projects.

9.3 Wastewater System CIP

The improvement projects included in the wastewater CIP are a compilation of the recommendations made in Chapter 7 of this IMP. The wastewater system CIP includes the following project categories:

- Capacity Improvements
  - Gravity Mains
  - Force Mains
  - Lift Stations
- R&R Improvements
  - Gravity Mains
  - Force Mains
- Treatment Plant Related Improvements
- Other Projects

A detailed list of wastewater CIP projects with project descriptions, sizing, and cost estimating information is provided at the end of this chapter in Table 9.11 and project locations are shown on Figure 9.10. The key project phasing assumptions and cost summarizes are presented below.

9.3.1 Wastewater CIP by Phase

The wastewater system CIP is summarized by improvement category and phase in Table 9.7, while phasing is graphically shown on Figure 9.3.

Table 9.7 Summary of Wastewater Improvement Costs by Project Category

Project Category	Near-Term 2018-2025 ($ Million)	Long-Term 2026-2040 ($ Million)	Build-Out 2041 &Beyond ($ Million)(2)	Total ($ Million)
Capacity	$19.6	$8.4	$34.9	$62.8
R&R Improvements	$1.4	$2.1	$0.1	$3.5
Treatment Plant	$27.3	$-	$-	$27.3
Other	$0.1	$1.6	$3.9	$5.6
Grand Total	**$48.3**	**$12.0**	**$38.9**	**$99.2**
Number of Years	8	15	N/A	N/A
Total Annual Cost ($/year)	**$6.0**	**$0.8**	**N/A**	**N/A**
Anticipated Developer Funding	$33.1	$7.2	$33.6	$73.8
City Funded CIP	$15.2	$4.9	$5.3	$25.4
City Annual Cost ($/year)	**$1.9**	**$0.3**	**N/A**	**N/A**

Notes:
(1) Numbers may vary slightly due to rounding.
(2) The costs per year do not include build-out since the implementation timeline is unknown and may be outside of the 2040 planning horizon.

As listed in Table 9.7 and on Figure 9.3, the wastewater CIP through the year 2040 is $60.3 million, which is approximately 61 percent of the total CIP (or $99.2 million) through build-out. The near-term projects account for about $48.3 million, which equates to roughly $6.0 million per year through year 2025. The long-term projects account for about $12.0 million, which equates to roughly $0.8 million per year from 2026 through 2040. The average estimated capital cost for the 23-year planning horizon of this IMP is $2.6 million per year, which excludes the build-out improvement projects that equate to approximately 38.9 million (or 39 percent) of the total CIP. Since the timing of the build-out projects is unknown, the costs are not included in the average annual expenditures.

The vast majority of the improvement projects are associated with developer funded CIP projects that occur within the near-term and built-out phase. It is anticipated that approximately $73.8 million in developer funding will be provided for future growth within the City, which includes the expansion of the existing treatment plant. The developer funding equates to approximately 67 percent (or $40.2 million) of the CIP through the year 2040. With developer funding, the City's anticipated average annual expenditures equate to $1.9 million in the near-term phase and $0.3 million in the long-term phase, or an overall average of $0.9 million within the 23-year planning horizon of this IMP.

In addition, as shown on Figure 9.4, the majority of the proposed improvements consist of capacity projects, which equate to approximately 63.3 percent of the total CIP cost. R&R improvements account for approximately 3.5 percent, treatment plant related improvements account for approximately 27.5 percent, and other projects account for approximately 5.6 percent of the total CIP. The treatment plant improvements were based on the City's CIP. Since a capacity analysis of the treatment plant was not included in this IMP, an additional analyses is recommended to further refine the cost estimates and phasing based on future flow projections. Therefore, the near-term costs may be reduced and included within the long-term and build-out phases.

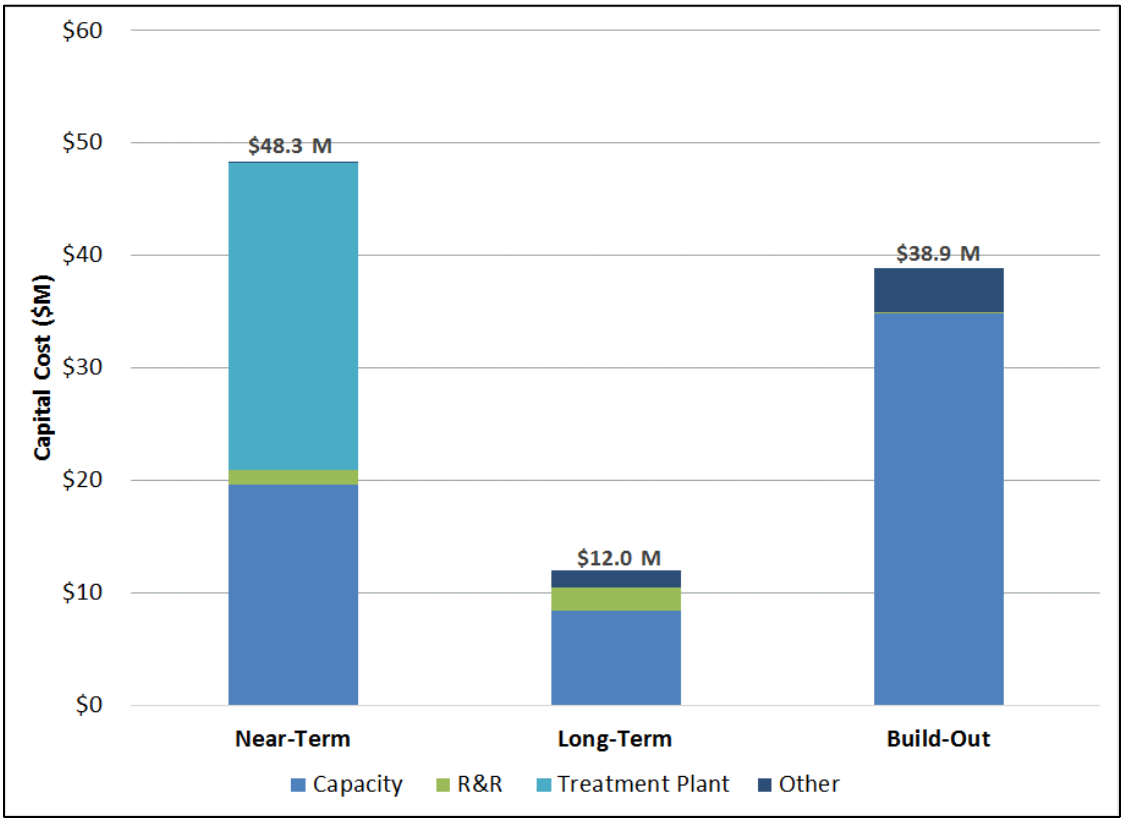

Figure 9.3 Wastewater CIP by Improvement Category and Phase

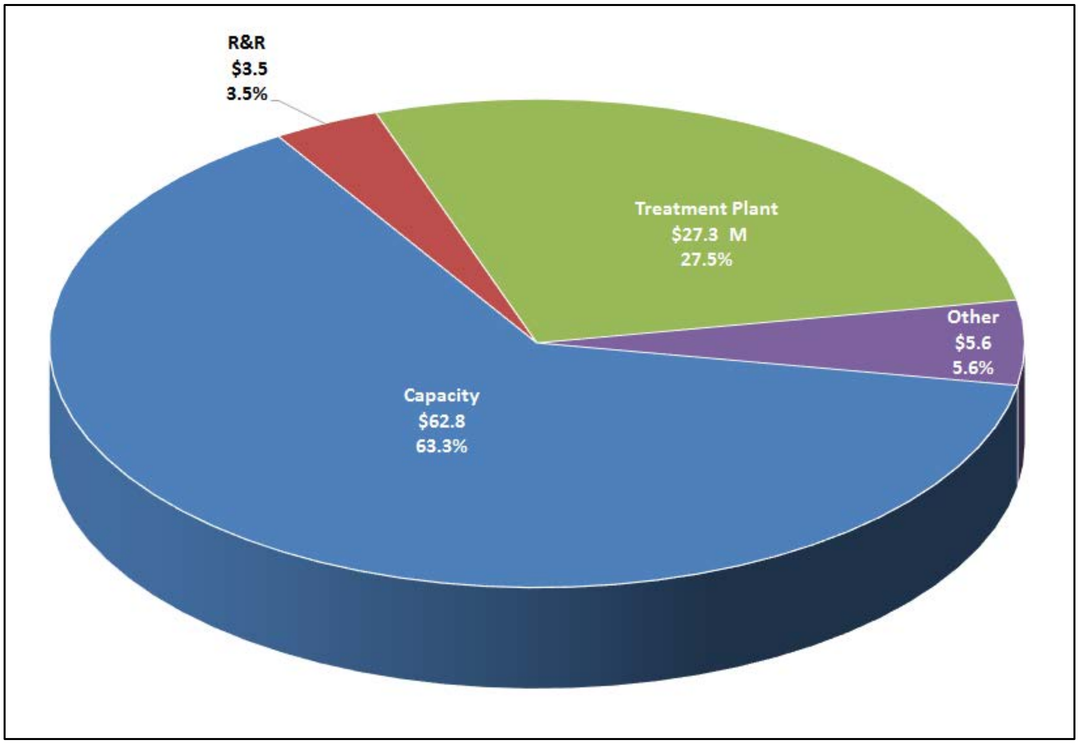

Figure 9.4 Wastewater CIP by Project Type

9.3.1.1 Near-Term Projects

As summarized in Table 9.7 and shown on Figure 9.3, the cost for the near-term projects is approximately $48.3 million, which includes $19.6 million for capacity improvements, $1.4 million for R&R, $27.3 million for treatment plant related projects, and $0.1 million for other CIP projects. The vast majority (or 56 percent) of the projects within the near-term are related to treatment plant improvements. The detail for each of these projects is listed in Table 9.11.

The capacity improvements are attributed to both future growth and mitigating existing deficiencies throughout the City's sewer system. Some of existing system deficiencies were triggered due to changes in criteria and requirements that occurred after the projects were constructed. Projects include approximately 1.7 miles of gravity main replacements and 5.0 miles of new gravity mains to accommodate new growth within the City, mainly the Butterfield and RSG master planned communities.

The R&R improvement projects equate to approximately 3 percent of the near-term CIP and include annual sewer replacements and site improvements at one (1) lift station, which was identified as part of the Condition Assessment Technical Memorandum (TM) in Appendix D.

The majority of the recommended improvements within the near-term are treatment plant related projects, which equate to over 56 percent of the near-term CIP. The key treatment plant project is the expansion of the City's existing wastewater plant, which will accommodate additional sewer flows related to future growth within the City and produce Title 22 quality recycled water.

Other projects equate to less than one percent of the near-term CIP and include a lift station telemetry study to review the condition of existing lift stations.

9.3.1.2 Long-Term Projects

As summarized in Table 9.7 and shown on Figure 9.3, the cost for the long-term projects is approximately $12.0 million, which includes $8.4 million for capacity improvements, $2.1 million for R&R, and $1.6 million for other CIP projects. The vast majority (or 70 percent) of the projects within the long-term are capacity related improvements. The detail for each of these projects is listed in Table 9.11.

The capacity improvement projects equate to approximately 70 percent of the long-term. The capacity improvements are attributed to future growth and mitigating future deficiencies throughout the City's sewer system. Projects include approximately 0.3 miles of gravity main replacements and 1.4 miles of new gravity mains, 0.8 miles of new force mains, and two (2) new lift stations with a total capacity of 2.5 mgd to accommodate new growth within the City.

The R&R improvement projects equate to approximately 17 percent of the long-term CIP and include annual sewer replacements.

Other projects equate to approximately 13 percent of the long-term CIP and include septic removal for residential, commercial, and industrial users and the connection to the City's wastewater collection system.

9.3.1.3 Build-Out Projects

As summarized in Table 9.7 and shown on Figure 9.3, the cost for the built-out projects is approximately $38.9 million, which includes $34.9 million for capacity improvements, $0.1 million for R&R, and $4.9 million for other CIP projects. The vast majority (or 90 percent) of the projects within the build-out phase are capacity improvements. The detail for each of these projects is listed in Table 9.11.

The capacity improvement projects equate to approximately 90 percent of the build-out phase. The capacity improvements are attributed to future growth and mitigating future deficiencies throughout the City's sewer system. Projects include approximately 1.7 miles of gravity main replacements and 15.3 miles of new gravity mains, 1.2 miles of new force mains, and three (3) new lift stations with a total capacity of 0.9 mgd to accommodate new growth within the City.

The R&R improvement projects equate to less than one percent of the build-out phase and include site improvements at one (1) lift station, which was identified as part of the Condition Assessment Technical Memorandum (TM) in Appendix D.

Other projects equate to approximately 13 percent of the build-out phase and include septic removal for residential, commercial, and industrial users and the connection to the City's wastewater collection system.

9.4 Recycled Water CIP

The improvement projects included in the recycled water CIP are a compilation of the recommendations made in Chapter 8 of this IMP. The recycled water system CIP includes the following project categories:

- Capacity Improvements
  - Pipelines

- Booster Pump Stations
- Wells
- Other Projects

A detailed list of recycled water CIP projects with project descriptions, sizing, and cost estimating information is provided at the end of this chapter in Table 9.12 and project locations are shown on Figure 9.11. The key project phasing assumptions and cost summarizes are presented below.

9.4.1 Recycled Water CIP by Phase

The recycled water system CIP is summarized by project type and phase in Table 9.8, while phasing is graphically shown on Figure 9.5.

Table 9.8 Summary of Recycled Water Improvement Costs by Project Type

Project Type	Near-Term 2018-2025 ($ Million)	Long-Term 2026-2040 ($ Million)	Build-Out 2041 &Beyond ($ Million)(2)	Total ($ Million)
Pipelines	$17.3	$2.3	$0.2	$19.8
Pump Stations	$5.8	$-	$-	$5.8
Wells	$1.7	$-	$-	$1.7
Storage	$3.7	$-	$-	$3.7
Valves	$0.7	$-	$-	$0.7
Other	$4.1	$3.0	$-	$7.1
Grand Total	**$33.3**	**$5.3**	**$0.2**	**$38.8**
Number of Years	8	15	N/A	N/A
Total Annual Cost ($/year)	**$4.2**	**$0.4**	**N/A**	**N/A**
Anticipated Developer Funding	$18.1	$1.3	$0.2	$19.2
City Funded CIP	$15.2	$4.0	$-	$18.4
City Annual Cost ($/year)	**$1.9**	**$0.3**	**N/A**	**N/A**

Notes:
(1) Numbers may vary slightly due to rounding.
(2) The costs per year do not include build-out since the implementation timeline is unknown and may be outside of the 2040 planning horizon.

As listed in Table 9.8 and on Figure 9.5, the recycled water CIP through the year 2040 is estimated to be $38.6 million, which is approximately 99 percent of the total CIP (or $38.8 million) through build-out. The near-term projects account for about $33.3 million, which equates to roughly $4.2 million per year through year 2025. The long-term projects account for about $5.3 million, which equates to roughly $0.4 million per year from 2026 through 2040. The average estimated capital cost for the 23-year planning horizon of this IMP is $1.7 million per year, which excludes the build-out improvement projects that equate to approximately

$0.2 million (or less than one percent) of the total CIP. Since the timing of the build-out projects is unknown, the costs are not included in the average annual expenditures.

The City funded and developer funded improvement projects are nearly equivalent. It is anticipated that approximately $19.6 million in developer funding will be provided for future growth within the City. The developer funding equates to approximately 50 percent (or $19.4 million) of the CIP through the year 2040. With developer funding, the City's anticipated average annual expenditures equate to $1.9 million in the near-term phase and $0.3 million in the long-term phase, or an overall average of $0.8 million within the 23-year planning horizon of this IMP.

In addition, as shown on Figure 9.6, the majority of the proposed improvements consist of pipeline projects, which equate to approximately 51.1 percent of the total CIP cost. Pump station improvements account for approximately 14.9 percent, storage improvements account for 9.6 percent, valves account for 1.8 percent, wells account for approximately 4.4 percent, and other projects account for approximately 18.2 percent of the total CIP cost.

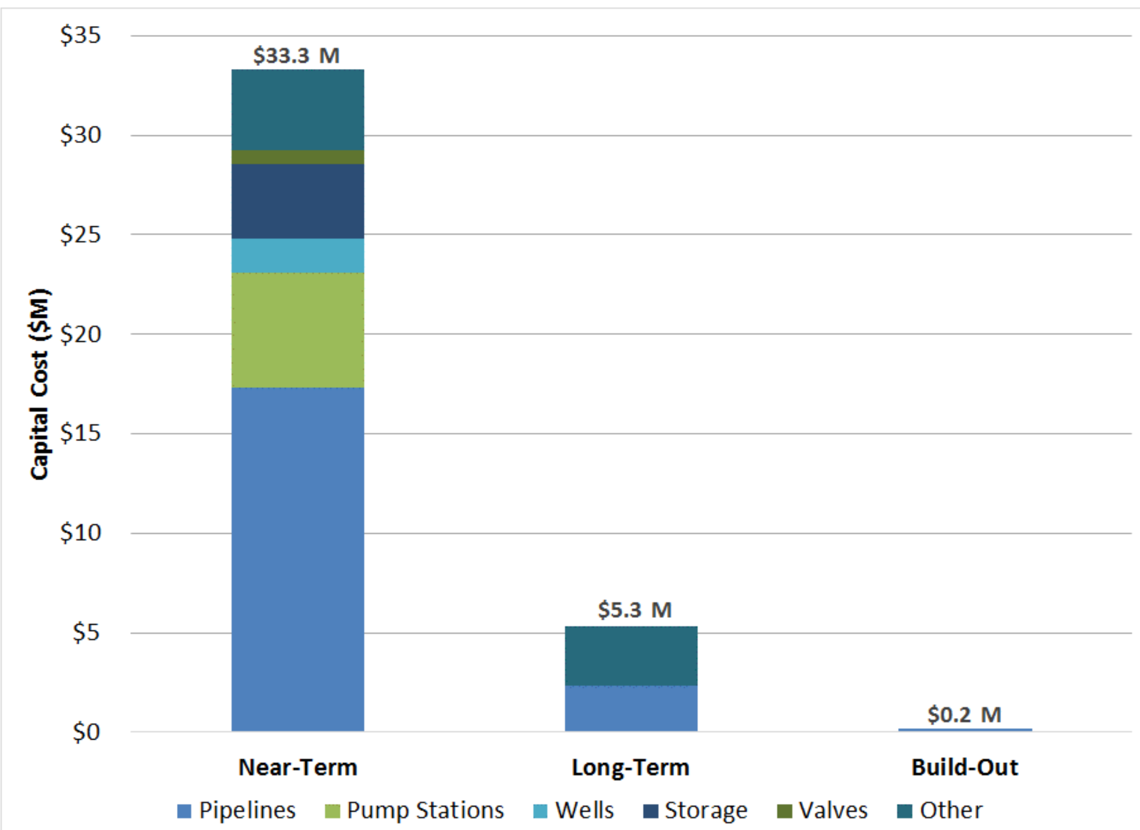

Figure 9.5 Recycled Water CIP by Project Type and Phase

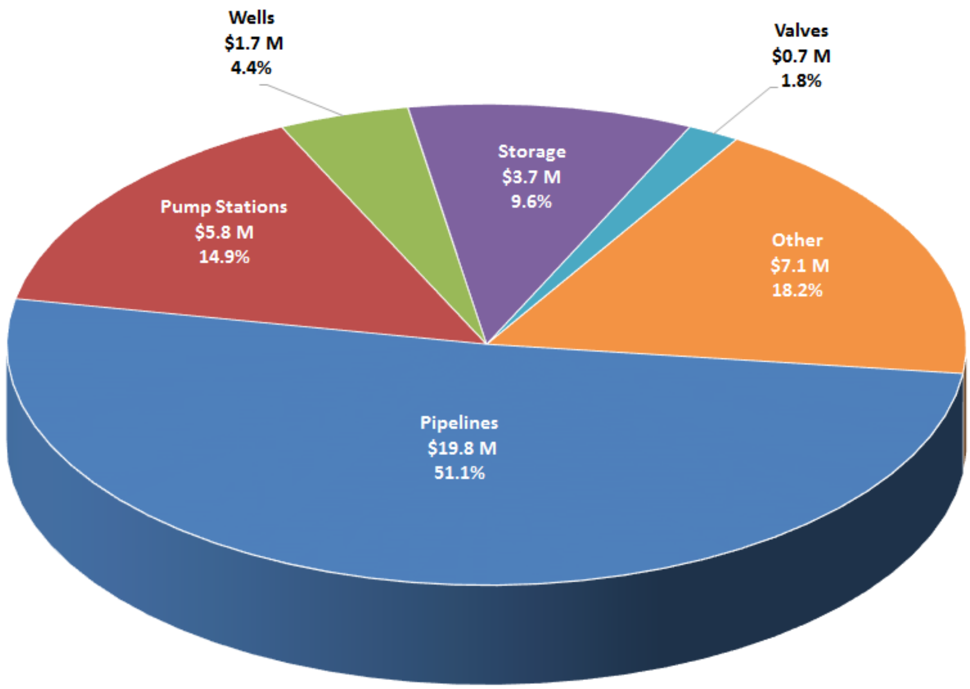

Figure 9.6 Recycled Water CIP by Project Type

9.4.1.1 Near-Term Projects

As summarized in Table 9.8 and shown on Figure 9.4, the cost for the near-term projects is approximately $33.3 million, which includes $29.2 million for capacity improvements and $4.1 million for other CIP projects. The vast majority (or 88 percent) of the projects within the near-term are related to capacity improvements. The detail for each of these projects is listed in Table 9.12.

The capacity improvements include projects to develop the City's backbone recycled water system and connect irrigation customers to the system. Projects include approximately 5.2 miles of pipelines, one (1) new pump station with a total horsepower of 1,400, one (1) storage tank with a capacity of 1 MG, the equipping of Well R-1, and valves to connect the Beaumont Cherry Valley Water District (BCVWD) wells into the City's system.

Other projects equate to less than one percent of the near-term CIP and include a hydrogeological study to review the conditions of potential recharge sites. In addition, site improvements at the Five Bridges and the WWTP basin were included within this category as well as 404 permitting and an update to the Recycled Water Master Plan.

9.4.1.2 Long-Term Projects

As summarized in Table 9.8 and shown on Figure 9.4, the cost for the long-term projects is approximately $5.3 million, which includes $2.3 million for capacity improvements and $3.0 million for other CIP projects. The detail for each of these projects is listed in Table 9.12.

The improvements within the long-term are nearly equally split between capacity related projects and other projects. The capacity improvements include projects to connect additional irrigation sites and extend pipelines to recharge basins. Projects include approximately 0.9 miles of pipelines. The other projects include monitoring wells and lysimeters, which is a requirement prior to initiating groundwater recharge with recycled water.

9.4.1.3 Build-Out Projects

As summarized in Table 9.8 and shown on Figure 9.4, the cost for the build-out phase is approximately $0.2 million. The sole project within the build-out phase is related to capacity improvements for new growth within the City. The detail for each of this project is listed in Table 9.12.

9.5 Integrated Systems CIP

The integrated systems CIP for the City's water, wastewater, and recycled water systems is summarized in Table 9.9 and graphically depicted on Figure 9.7. As shown in Table 9.9, the combined CIP costs for all three systems through planning year 2040 is estimated to be about $345.4 million, respectively.

Table 9.9 Integrated CIP by System and Phase

Project Type	Near-Term 2018-2025 ($ Million)	Long-Term 2026-2040 ($ Million)	Build-Out 2041 &Beyond ($ Million)(2)	Total ($ Million)
Potable Water System(1)	$108.7	$137.8	$323.8	$570.2
Wastewater System(2)	$48.3	$12.0	$38.9	$99.2
Recycled Water System(3)	$33.3	$5.3	$0.2	$38.8
Grand Total	**$190.3**	**$155.1**	**$362.9**	**$708.3**
Number of Years	8	15	N/A	N/A
Total Annual Cost ($/year)	**$23.8**	**$10.3**	**N/A**	**N/A**
Anticipated Developer Funding	$84.8	$41.1	$103.3	$229.1
City Funded CIP	$105.5	$114.1	$259.6	$479.1
City Annual Cost ($/year)	**$13.2**	**$7.6**	**N/A**	**N/A**

Notes:
(1) See Table 9.10.
(2) See Table 9.11.
(3) See Table 9.12.
(4) The costs per year do not include build-out since the implementation timeline is unknown and may be outside of the 2040 planning horizon.
(5) Numbers may vary slightly due to rounding.

As shown on Figure 9.8, the potable water system CIP comprises the largest portion of cost with $570.2 million (80 percent) of the total combined CIP, while the wastewater system CIP represents the second largest cost with $99.2 million (14 percent).

The phasing of the integrated CIP by system is depicted on Figure 9.7. As shown on this figure, about $190.3 million of project costs are included in the near-term phase and $155.1 million are scheduled for the long-term phase. Nearly 51 percent (or $362.9) of the improvement projects are anticipated to occur in the build-out phase, which is outside of the planning horizon of this IMP.

It is anticipated that a combined total of approximately $84.8 million in developer funding will be provided within the near-term and $41.1 million within the long-term planning phases. With consideration of developer funding, the City's anticipated remaining average annual expenditures equate to approximately $13.2 million in the near-term phase and $7.6 million in the long-term phase, or an overall average of $9.5 million per year within the 23-year planning horizon of this IMP.

As mentioned in Section 9.1.4, the current water rates will make it difficult to fund all the projects recommended within the near-term planning phase. Therefore, the CIP will need to be revised periodically to adjust the project phasing based on system needs and available funding. The phasing of other select projects may also be adjusted at the discretion of City staff. Future rate increases to raise capital funds, additional contributions from developers, and grant funding can potentially accelerate projects to the near-term planning phase. The dynamic CIP planning tool can be utilized by City staff to make adjustments to cost estimating assumption and phasing. The tool is designed so that changes on master sheets, such as the unit cost assumptions, will ripple throughout individual project sheets and the summary CIP table. This will allow City staff to efficiently make updates when needed.

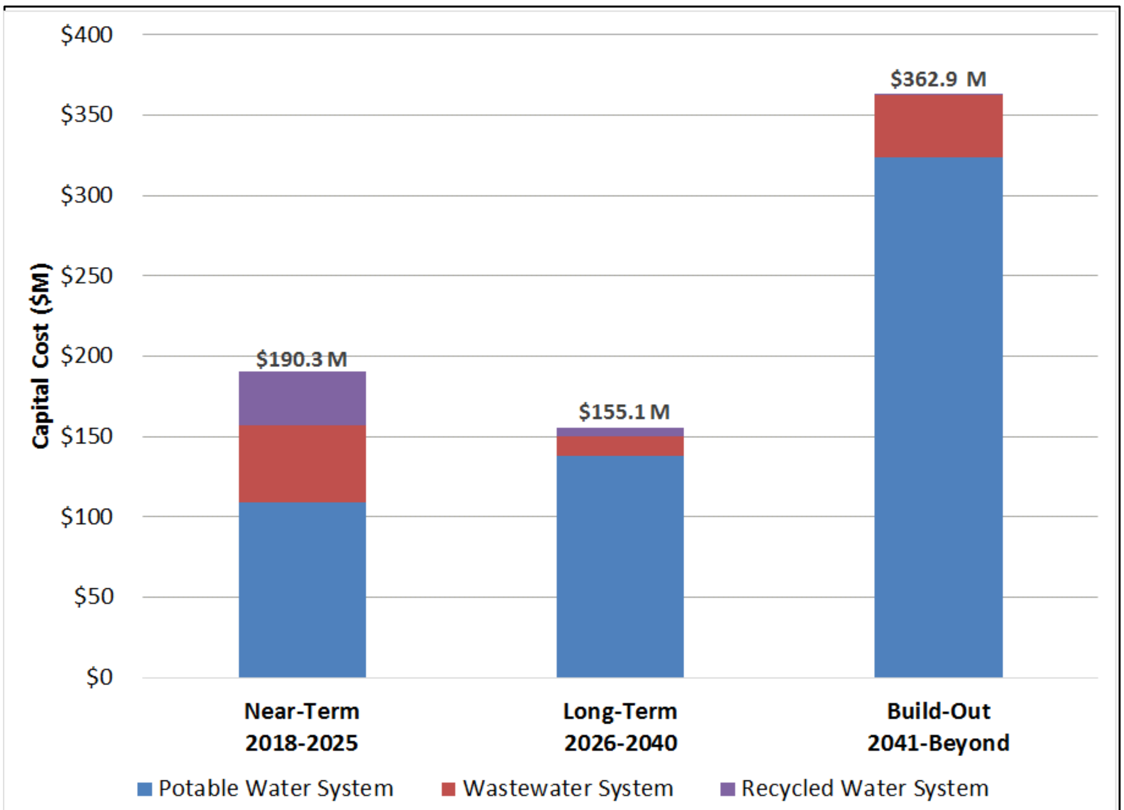

Figure 9.7 Integrated Systems CIP by Phase

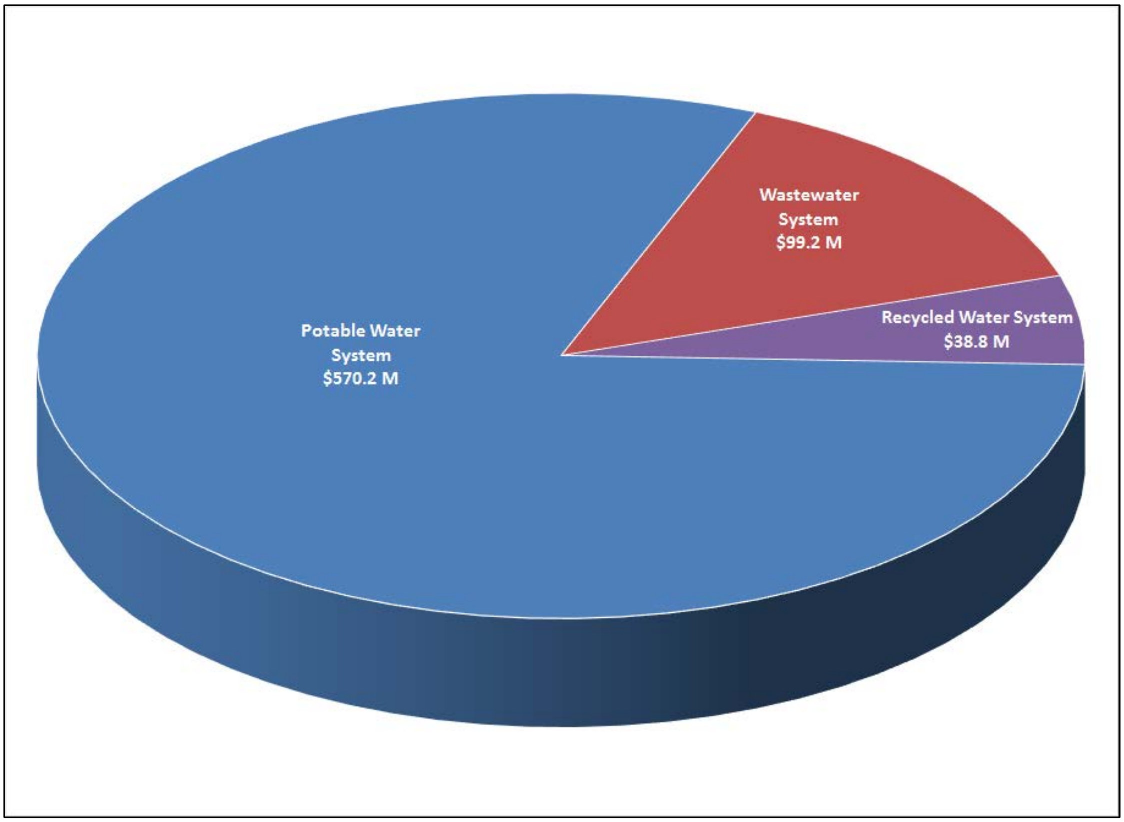

Figure 9.8 Integrated Systems CIP by Costs

Table 9.10 Potable Water Capital Improvement Plan Summary

Project		Proposed Size/Diameter	CIP Cost Estimate	City Cost	Developer Cost	CIP Phasing									
						Near-Term								Long-Term	Build-out
						2018	2019	2020	2021	2022	2023	2024	2025	2026-2040	2041 & beyond
Capacity and Reliability Improvements			$ 173,849,000	$ 38,060,000	$ 135,789,000	$ 1,601,000	$ 3,837,000	$ 7,153,000	$ 21,094,000	$ 17,115,000	$ 4,093,000	$ 1,512,000	$ 3,158,000	$ 44,756,000	$ 69,530,000
Pipelines		**Diameter (in)**													
PWP-1	New Transmission Main for Proposed Lower Main Well C-8	12	$ 414,000	$ -	$ 414,000	$ -	$ -	$ 414,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWP-2	New Transmission Main for Upper Main Reservoir 1 (RSG)	24	$ 5,118,000	$ 4,043,000	$ 1,075,000	$ -	$ -	$ 512,000	$ 2,559,000	$ 2,047,000	$ -	$ -	$ -	$ -	$ -
PWP-3	New Transmission Main for Proposed Development in Foothill West Zone (Butterfield)	12	$ 3,522,000	$ -	$ 3,522,000	$ -	$ -	$ 352,000	$ 1,761,000	$ 1,409,000	$ -	$ -	$ -	$ -	$ -
PWP-4	New Transmission Main for Proposed Development in Main Zone (RSG)	12	$ 8,288,000	$ -	$ 8,288,000	$ -	$ -	$ 829,000	$ 4,144,000	$ 3,315,000	$ -	$ -	$ -	$ -	$ -
PWP-5	New Transmission Main for Foothill West Reservoir 1 & PS (Butterfield)	18	$ 3,730,000	$ -	$ 3,730,000	$ -	$ -	$ 373,000	$ 1,865,000	$ 1,492,000	$ -	$ -	$ -	$ -	$ -
PWP-6	New Transmission Main from Mountain Booster PS to Existing Mountain North (Butterfield)	12	$ 1,450,000	$ -	$ 1,450,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,450,000	$ -
PWP-7	New Transmission Main for Proposed Development in Mountain North Zone (Butterfield)	12	$ 1,865,000	$ -	$ 1,865,000	$ -	$ -	$ -	$ -	$ -	$ 1,865,000	$ -	$ -	$ -	$ -
PWP-8	New Transmission Main for Proposed Upper Main Well C-9	12	$ 414,000	$ -	$ 414,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 414,000	$ -
PWP-9	New Transmission Main for Mountain North Reservoir 1 & PS (Butterfield)	18	$ 4,040,000	$ 1,939,000	$ 2,101,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 4,040,000	$ -
PWP-10	New Transmission Main for Upper Main Reservoir 2	24	$ 394,000	$ -	$ 394,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 394,000	$ -
PWP-11	New Transmission Main for Proposed Development in Upper Butterfield Zone (Butterfield)	12	$ 414,000	$ -	$ 414,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 414,000	$ -
PWP-12	New Transmission Main for Proposed Upper Butterfield Reservoir (Butterfield)	12	$ 1,865,000	$ -	$ 1,865,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,865,000	$ -
PWP-13	Water Canyon Pipe Phase 2 (City's Existing CIP)	n/a	$ 3,250,000	$ 3,250,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 3,250,000	$ -
PWP-14	New Transmission Main for Proposed Upper Main Well C-10	12	$ 829,000	$ -	$ 829,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 829,000	$ -
PWP-15	New Transmission Main for Proposed Foothill West Well C-11	12	$ 414,000	$ -	$ 414,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 414,000
PWP-16	New Transmission Main for Proposed Upper Main Well C-12	12	$ 414,000	$ -	$ 414,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 414,000
PWP-17	New Transmission Main for Foothill West Reservior 2	18	$ 3,108,000	$ -	$ 3,108,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 3,108,000
PWP-18	New Transmission Main for Upper Main Reservoir 3	30	$ 4,144,000	$ -	$ 4,144,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 4,144,000
PWP-19	New Transmission Main for Black Bench Reservoir 1 & PS	18	$ 3,108,000	$ -	$ 3,108,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 3,108,000
PWP-20	New Transmission Main for Loma Linda Reservoir 1 & PS	18	$ 3,108,000	$ -	$ 3,108,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 3,108,000
Fire Flow Improvements		**Diameter (in)**													
PWFF-1	Fire Flow Improvement 1	8	$ 126,000	$ 126,000	$ -	$ 126,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWFF-2	Fire Flow Improvement 2	8	$ 31,000	$ 31,000	$ -	$ 31,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWFF-3	Fire Flow Improvement 3 (Includes PRV & Check Valves)	n/a	$ 341,000	$ 341,000	$ -	$ 341,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWFF-3	Fire Flow Improvement 3 (Includes PRV & Check Valves)	n/a	$ 511,000	$ 511,000	$ -	$ 511,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWFF-3	Fire Flow Improvement 3 (Includes PRV & Check Valves)	8	$ 567,000	$ 567,000	$ -	$ -	$ 402,000	$ 165,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWFF-3	Fire Flow Improvement 3 (Includes PRV & Check Valves)	12	$ 2,145,000	$ 2,145,000	$ -	$ -	$ 995,000	$ 1,150,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWFF-4	Fire Flow Improvement 4	8	$ 31,000	$ 31,000	$ -	$ -	$ -	$ -	$ 31,000	$ -	$ -	$ -	$ -	$ -	$ -
PWFF-5	Fire Flow Improvement 5	8	$ 220,000	$ 220,000	$ -	$ -	$ -	$ -	$ 220,000	$ -	$ -	$ -	$ -	$ -	$ -
PWFF-6	Fire Flow Improvement 6	8	$ 157,000	$ 157,000	$ -	$ -	$ -	$ -	$ 157,000	$ -	$ -	$ -	$ -	$ -	$ -
PWFF-7	Fire Flow Improvement 7	8	$ 31,000	$ 31,000	$ -	$ -	$ -	$ -	$ 31,000	$ -	$ -	$ -	$ -	$ -	$ -
PWFF-8	Fire Flow Improvement 8	8	$ 31,000	$ 31,000	$ -	$ -	$ -	$ -	$ 31,000	$ -	$ -	$ -	$ -	$ -	$ -
PWFF-9	Fire Flow Improvement 9	8	$ 189,000	$ 189,000	$ -	$ -	$ -	$ -	$ 189,000	$ -	$ -	$ -	$ -	$ -	$ -
PWFF-10	Fire Flow Improvement 10	8	$ 157,000	$ 157,000	$ -	$ -	$ -	$ -	$ 157,000	$ -	$ -	$ -	$ -	$ -	$ -
PWFF-11	Fire Flow Improvement 11	8	$ 63,000	$ 63,000	$ -	$ -	$ -	$ -	$ -	$ 63,000	$ -	$ -	$ -	$ -	$ -
PWFF-11	Fire Flow Improvement 11	12	$ 829,000	$ 829,000	$ -	$ -	$ -	$ -	$ -	$ 829,000	$ -	$ -	$ -	$ -	$ -
PWFF-12	Fire Flow Improvement 12	8	$ 409,000	$ 409,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 409,000	$ -	$ -	$ -	$ -
PWFF-12	Fire Flow Improvement 12	12	$ 622,000	$ 622,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 622,000	$ -	$ -	$ -	$ -
PWFF-13	Fire Flow Improvement 13	8	$ 315,000	$ 315,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 315,000	$ -	$ -	$ -	$ -
PWFF-14	Fire Flow Improvement 14	8	$ 441,000	$ 441,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 441,000	$ -	$ -	$ -	$ -
PWFF-15	Fire Flow Improvement 15	8	$ 441,000	$ 441,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 441,000	$ -	$ -	$ -	$ -
PWFF-16	Fire Flow Improvement 16	8	$ 1,512,000	$ 1,512,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,512,000	$ -	$ -	$ -
PWFF-17	Fire Flow Improvement 17	8	$ 63,000	$ 63,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 63,000	$ -	$ -
PWFF-18	Fire Flow Improvement 18	8	$ 472,000	$ 472,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 472,000	$ -	$ -
PWFF-19	Fire Flow Improvement 19	8	$ 31,000	$ 31,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 31,000	$ -	$ -
PWFF-20	Fire Flow Improvement 20	8	$ 94,000	$ 94,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 94,000	$ -	$ -
PWFF-21	Fire Flow Improvement 21	8	$ 94,000	$ 94,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 94,000	$ -	$ -
PWFF-22	Fire Flow Improvement 22	8	$ 283,000	$ 283,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 283,000	$ -	$ -
PWFF-23	Fire Flow Improvement 23	8	$ 227,000	$ 227,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 227,000	$ -	$ -
Booster Pump Stations		**Quantity (hp)**													
PWPU-1a	Upgrade Existing Mountain Booster Pump Station	80	$ 729,000	$ 729,000	$ -	$ -	$ 729,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWPU-1b	Demolish Existing Mountain Booster Pump Station	n/a	$ 166,000	$ 166,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 166,000	$ -
PWPU-2	New Foothill West Pump Station	180	$ 1,044,000	$ -	$ 1,044,000	$ -	$ -	$ 104,000	$ 522,000	$ 418,000	$ -	$ -	$ -	$ -	$ -
PWPU-3	New Mountain 2 Booster Pump Station	120	$ 696,000	$ 334,000	$ 362,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 696,000	$ -
PWPU-4	Add VFD to Well C-6	50	$ 166,000	$ 166,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 166,000	$ -
PWPU-5	Add VFD to Well C-8	80	$ 166,000	$ 166,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 166,000	$ -
PWPU-6	New Upper Butterfield Zone Pump Station	80	$ 456,000	$ -	$ 456,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 456,000	$ -
PWPU-7	New Loma Linda Pump Station	80	$ 729,000	$ -	$ 729,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 729,000
PWPU-8	New Black Bench Pump Station	80	$ 729,000	$ -	$ 729,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 729,000
Storage		**Quantity (MG)**													
PWS-1	Proposed Upper Main Reservoir 1	4.0	$ 13,260,000	$ 10,475,000	$ 2,785,000	$ -	$ -	$ 1,326,000	$ 6,630,000	$ 5,304,000	$ -	$ -	$ -	$ -	$ -
PWS-2	Proposed Foothill West Reservoir 1	1.5	$ 5,594,000	$ -	$ 5,594,000	$ -	$ -	$ 559,000	$ 2,797,000	$ 2,238,000	$ -	$ -	$ -	$ -	$ -
PWS-3	Proposed Mountain North Reservoir 1	1.0	$ 5,594,000	$ 2,685,000	$ 2,909,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 5,594,000	$ -
PWS-4	Proposed Upper Main Reservoir 2	4.0	$ 13,260,000	$ -	$ 13,260,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 13,260,000	$ -
PWS-5	Proposed Upper Butterfield Reservoir	1.0	$ 3,729,000	$ -	$ 3,729,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 3,729,000	$ -
PWS-6	Proposed Foothill West Reservoir 2	1.5	$ 5,594,000	$ -	$ 5,594,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 5,594,000
PWS-7	Proposed Upper Main Reservoir 3	9.0	$ 26,106,000	$ -	$ 26,106,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 26,106,000
PWS-8	Proposed Black Bench Reservoir 1	1.5	$ 5,594,000	$ -	$ 5,594,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 5,594,000
PWS-9	Proposed Loma Linda Reservoir 1	1.0	$ 3,729,000	$ -	$ 3,729,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 3,729,000

Table 9.10 Potable Water Capital Improvement Plan Summary

	Project	Proposed Size/Diameter	CIP Cost Estimate	City Cost	Developer Cost	CIP Phasing									
						Near-Term								Long-Term	Build-out
						2018	2019	2020	2021	2022	2023	2024	2025	2026-2040	2041 & beyond
Wells		**Quantity (gpm)**													
PWW-1	Proposed Main Zone Well C-8	1,400	$ 3,422,000	$ -	$ 3,422,000	$ 342,000	$ 1,711,000	$ 1,369,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWW-2	Convert Well M-7 to Supply the Upper Main Pressure Zone	500	$ 191,000	$ -	$ 191,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 191,000	$ -	$ -
PWW-3	Convert Well M-12 to Supply the Upper Main Pressure Zone	1,100	$ 191,000	$ -	$ 191,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 191,000	$ -
PWW-4	Proposed Upper Main Well C-9	1,800	$ 4,252,000	$ -	$ 4,252,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 4,252,000	$ -
PWW-5	Proposed Upper Main Well C-10	1,800	$ 4,251,000	$ -	$ 4,251,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 4,251,000
PWW-6	Proposed Foothill West Well C-11	1,800	$ 4,251,000	$ -	$ 4,251,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 4,251,000
PWW-7	Proposed Upper Main Well C-12	1,800	$ 4,251,000	$ -	$ 4,251,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 4,251,000
Valves															
PWV-1	Altitude Valves (City's Existing CIP)	n/a	$ 250,000	$ 250,000	$ -	$ 250,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWV-2	New Pressure Reducing Valve for Rancho San Gorgonio	n/a	$ 341,000	$ -	$ 341,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 341,000	$ -	$ -
PWV-3	Foothill West to Upper Main Zone Pressure Reducing Station	n/a	$ 681,000	$ -	$ 681,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 681,000	$ -	$ -
PWV-4	C2 PRVs 1 & 2	n/a	$ 681,000	$ -	$ 681,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 681,000	$ -	$ -
PWRZ-1	New Pressure Reducing Valves for Re-Zoning	n/a	$ 3,424,000	$ 3,424,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 3,424,000	$ -
Repair and Rehabilitation Projects			$ 357,746,000	$ 357,746,000	$ -	$ 1,467,000	$ 1,330,000	$ 1,242,000	$ 1,231,000	$ 1,316,000	$ 1,354,000	$ 1,703,000	$ 748,000	$ 93,094,000	$ 254,261,000
Pipelines		**Quantity (mi)**													
PWRR-1	Pipeline Age Replacement Program	133	$ 346,826,000	$ 346,826,000	$ -	$ -	$ -	$ -	$ 100,000	$ 800,000	$ 100,000	$ 800,000	$ 100,000	$ 90,665,000	$ 254,261,000
PWRR-2	Water Line Replacement Locations #2 (City's Existing CIP)	n/a	$ 650,000	$ 650,000	$ -	$ 650,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWRR-3	Water Line Replacement Locations #3 (City's Existing CIP)	n/a	$ 650,000	$ 650,000	$ -	$ -	$ 650,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWRR-4	Water Line Repalcement: Jacinto View/Chevy (City's Existing CIP)	n/a	$ 580,000	$ 580,000	$ -	$ -	$ 30,000	$ 550,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Storage															
PWRR-5	San Gorgonio Reservoir Site R&R	n/a	$ 767,000	$ 767,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 375,000	$ 375,000	$ 17,000	$ -
PWRR-6	Southwest Reservoir Site R&R	n/a	$ 41,000	$ 41,000	$ -	$ -	$ -	$ 28,000	$ -	$ -	$ -	$ -	$ -	$ 13,000	$ -
PWRR-7	Mountain Reservoir Site R&R	n/a	$ 788,000	$ 788,000	$ -	$ -	$ 161,000	$ 161,000	$ 214,000	$ -	$ -	$ -	$ -	$ 252,000	$ -
PWRR-8	High Valley Reservoir Site R&R	n/a	$ 839,000	$ 839,000	$ -	$ 181,000	$ 181,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 477,000	$ -
PWRR-9	Sunset Reservoir Site R&R	n/a	$ 642,000	$ 642,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 582,000	$ -	$ -	$ 60,000	$ -
Valves															
PWRR-10	Foothill East PRV R&R	n/a	$ 28,000	$ 28,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 28,000	$ -
PWRR-11	Hargrave & John PRV R&R	n/a	$ 71,000	$ 71,000	$ -	$ -	$ 35,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 36,000	$ -
Wells															
PWRR-12	Well 1 Site R&R	n/a	$ 879,000	$ 879,000	$ -	$ 133,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 746,000	$ -
PWRR-13	Well 3 Site R&R	n/a	$ 30,000	$ 30,000	$ -	$ -	$ -	$ -	$ 17,000	$ -	$ -	$ -	$ -	$ 13,000	$ -
PWRR-14	Well C-2 Site R&R	n/a	$ 776,000	$ 776,000	$ -	$ -	$ -	$ -	$ 627,000	$ -	$ -	$ -	$ -	$ 149,000	$ -
PWRR-15	Well C-5 Site R&R	n/a	$ 477,000	$ 477,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 386,000	$ -	$ -	$ 91,000	$ -
PWRR-16	Well C-6 Site R&R	n/a	$ 26,000	$ 26,000	$ -	$ -	$ -	$ -	$ -	$ 13,000	$ -	$ -	$ -	$ 13,000	$ -
PWRR-17	Well M-3 Site R&R	n/a	$ 235,000	$ 235,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 13,000	$ -	$ -	$ 222,000	$ -
PWRR-18	Well M-11 Site R&R	n/a	$ 66,000	$ 66,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 25,000	$ -	$ 41,000	$ -
PWRR-19	Well M-12 Site R&R	n/a	$ 41,000	$ 41,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 41,000	$ -
PWRR-20	Well Enclosures (City's Existing CIP)	n/a	$ 400,000	$ 400,000	$ -	$ 80,000	$ -	$ 80,000	$ -	$ 80,000	$ -	$ 80,000	$ -	$ 80,000	$ -
PWRR-21	Well Rehabilitation (City's Existing CIP)	n/a	$ 750,000	$ 750,000	$ -	$ 150,000	$ -	$ 150,000	$ -	$ 150,000	$ -	$ 150,000	$ -	$ 150,000	$ -
Multi-Site Projects															
PWRR-22	Multi-Site Projects (Emergency Power & Safety Retrofits)	n/a	$ 2,184,000	$ 2,184,000	$ -	$ 273,000	$ 273,000	$ 273,000	$ 273,000	$ 273,000	$ 273,000	$ 273,000	$ 273,000	$ -	$ -
Other Projects			$ 38,716,000	$ 38,716,000	$ -	$ 2,299,000	$ 3,180,000	$ 2,290,000	$ 5,790,000	$ 25,000,000	$ -	$ -	$ 157,000	$ -	$ -
PWO-1	Pipeline Rehabilitation Asset Study	n/a	$ 216,000	$ 216,000	$ -	$ 216,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWO-2	Security Cameras at Water Yard (City's Existing CIP)	n/a	$ 33,000	$ 33,000	$ -	$ 33,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWO-3	Replace SCADA Computer Hardware/Software (City's Existing CIP)	n/a	$ 750,000	$ 750,000	$ -	$ 250,000	$ 250,000	$ 250,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWO-4	Work Truck (City's Existing CIP)	n/a	$ 80,000	$ 80,000	$ -	$ -	$ -	$ 40,000	$ 40,000	$ -	$ -	$ -	$ -	$ -	$ -
PWO-5	Automatic Meter Reading (AMR) (City's Existing CIP)	n/a	$ 3,800,000	$ 3,800,000	$ -	$ 1,800,000	$ 2,000,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWO-6	Advanced Metering Infrastructure (AMI) (City's Existing CIP)	n/a	$ 750,000	$ 750,000	$ -	$ -	$ 750,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
PWO-7	Computer Information System/ERP (City's Existing CIP)	n/a	$ 1,500,000	$ 1,500,000	$ -	$ -	$ -	$ 750,000	$ 750,000	$ -	$ -	$ -	$ -	$ -	$ -
PWO-8	Chromium 6 Treatment Pilot Study, Design, and Construction (City's Existing CIP)	n/a	$ 31,430,000	$ 31,430,000	$ -	$ -	$ 180,000	$ 1,250,000	$ 5,000,000	$ 25,000,000	$ -	$ -	$ -	$ -	$ -
PWO-9	Water Master Plan Update (City's Existing CIP)	n/a	$ 157,000	$ 157,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 157,000	$ -	$ -
CIP Total			**$ 570,311,000**	**$ 434,522,000**	**$ 135,789,000**	**$ 5,367,000**	**$ 8,347,000**	**$ 10,685,000**	**$ 28,115,000**	**$ 43,431,000**	**$ 5,447,000**	**$ 3,215,000**	**$ 4,063,000**	**$ 137,850,000**	**$ 323,791,000**
Annual Cost			**N/A**	**N/A**	**N/A**	**$ 5,367,000**	**$ 8,347,000**	**$ 10,685,000**	**$ 28,115,000**	**$ 43,431,000**	**$ 5,447,000**	**$ 3,215,000**	**$ 4,063,000**	**$ 9,190,000**	**N/A**

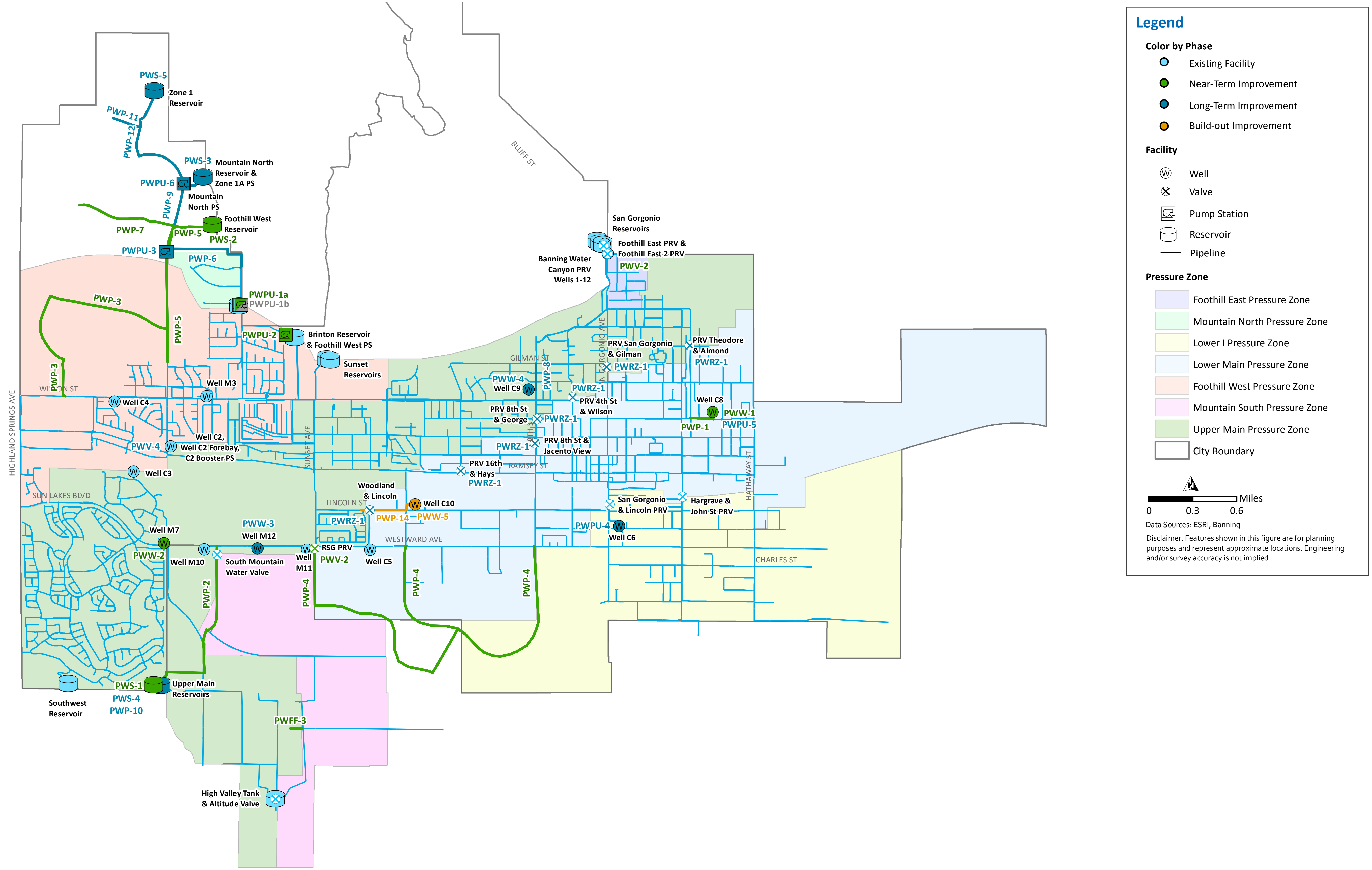

Figure 9.9 Potable Water CIP Map

Table 9.11 Wastewater Capital Improvement Plan Summary

Project	Proposed Size/Diameter	CIP Cost Estimate	City Cost	Developer Cost	CIP Phasing									
					Near-Term								Long-Term	Build-Out
					2018	2019	2020	2021	2022	2023	2024	2025	2026-2040	2041 & beyond
Capacity Related Improvements		$ 14,289,000	$ 10,890,000	$ 3,399,000	$ 472,000	$ 1,044,000	$ 456,000	$ 315,000	$ 5,871,000	$ 315,000	$ 315,000	$ 157,000	$ 791,000	$ 4,553,000
Gravity Mains	**Diameter (in)**	**$ 8,716,000**	**$ 5,317,000**	**$ 3,399,000**	**$ 472,000**	**$ 1,044,000**	**$ 456,000**	**$ 315,000**	**$ 298,000**	**$ 315,000**	**$ 315,000**	**$ 157,000**	**$ 791,000**	**$ 4,553,000**
WWGM-1 Gravity Main along Williams Street	10	$ 298,000	$ 298,000	$ -	$ -	$ -	$ -	$ -	$ 298,000	$ -	$ -	$ -	$ -	$ -
WWGM-2 Northern Segment of Gravity Main along Hathaway Street	12	$ 315,000	$ 315,000	$ -	$ -	$ -	$ -	$ 315,000	$ -	$ -	$ -	$ -	$ -	$ -
WWGM-3A Casing Under I-10	15/30	$ 456,000	$ 456,000	$ -	$ -	$ -	$ 456,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -
WWGM-3B Gravity Main along Hathaway Street	15	$ 1,044,000	$ 1,044,000	$ -	$ -	$ 1,044,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
WWGM-4 Gravity Main along Ramsey Street	12	$ 315,000	$ 315,000	$ -	$ -	$ -	$ -	$ -	$ -	$ 315,000	$ -	$ -	$ -	$ -
WWGM-5 Gravity Main along Charles Street	21	$ 472,000	$ 472,000	$ -	$ 472,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
WWGM-6 Gravity Main along Livingston Street	12	$ 315,000	$ 315,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 315,000	$ -	$ -	$ -
WWGM-7 Gravity Main along Fourth Street	12	$ 157,000	$ 157,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 157,000	$ -	$ -
WWGM-8 Gravity Main along Charles Street	21	$ 472,000	$ 340,000	$ 132,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 472,000	$ -
WWGM-9 Gravity Main along Porter Street	30	$ 319,000	$ 128,000	$ 191,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 319,000	$ -
WWGM-10 Gravity Main along Porter Street	24	$ 2,631,000	$ 789,000	$ 1,842,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 2,631,000
WWGM-11 Gravity Main,Porter Street to WWTP	24	$ 1,541,000	$ 478,000	$ 1,063,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,541,000
WWGM-12 Gravity Main south of Charles Street to WWTP	21	$ 236,000	$ 90,000	$ 146,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 236,000
WWGM-13 Gravity Main along Wilson Street	8	$ 145,000	$ 120,000	$ 25,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 145,000
Force Mains	**Diameter (in)**	**$ 485,000**	**$ 485,000**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ 485,000**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**
WWFM-1 Interim Westward Lift Station Force Main Upgrade	12	$ 485,000	$ 485,000	$ -	$ -	$ -	$ -	$ -	$ 485,000	$ -	$ -	$ -	$ -	$ -
Lift Stations	**Quantity (mgd)**	**$ 5,088,000**	**$ 5,088,000**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ 5,088,000**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**
WWLS-1 Interim Westward Lift Station Upgrade	4.40 mgd	$ 5,088,000	$ 5,088,000	$ -	$ -	$ -	$ -	$ -	$ 5,088,000	$ -	$ -	$ -	$ -	$ -
New Service Related Improvements		$ 48,526,000	$ 321,000	$ 48,205,000	$ -	$ -	$ -	$ 2,611,000	$ 2,411,000	$ 3,543,000	$ 1,492,000	$ 580,000	$ 7,587,000	$ 30,302,000
Gravity Mains	**Diameter (in)**	**$ 37,848,000**	**$ 321,000**	**$ 37,527,000**	**$ -**	**$ -**	**$ -**	**$ 2,611,000**	**$ 2,411,000**	**$ 3,543,000**	**$ 1,492,000**	**$ 580,000**	**$ 2,370,000**	**$ 24,841,000**
WWGM-14 Butterfield Offsite Trunk	15	$ 2,611,000	$ -	$ 2,611,000	$ -	$ -	$ -	$ 2,611,000	$ -	$ -	$ -	$ -	$ -	$ -
WWGM-15 Butterfield-Loma Linda Offsite Trunk	15	$ 870,000	$ -	$ 870,000	$ -	$ -	$ -	$ -	$ 870,000	$ -	$ -	$ -	$ -	$ -
WWGM-16 Westward Lift Station Bypass	18	$ 746,000	$ 321,000	$ 425,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 746,000	$ -
WWGM-17 RSG Main Trunk	18	$ 6,576,000	$ -	$ 6,576,000	$ -	$ -	$ -	$ -	$ 1,541,000	$ 3,543,000	$ 1,492,000	$ -	$ -	$ -
WWGM-18 Gravity Main along Wilson Street	8	$ 580,000	$ -	$ 580,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 580,000	$ -	$ -
WWGM-19 Gravity Main for RMG	8	$ 435,000	$ -	$ 435,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 435,000	$ -
WWGM-20 Gravity Main along Lincoln Street	8	$ 29,000	$ -	$ 29,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 29,000	$ -
WWGM-21 Gravity Main along Cottonwood Road	8	$ 1,160,000	$ -	$ 1,160,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,160,000	$ -
WWGM-22 Gravity Main along Fountain Street	8	$ 1,595,000	$ -	$ 1,595,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,595,000
WWGM-23 Gravity Main along Longhorn Road	8	$ 5,801,000	$ -	$ 5,801,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 5,801,000
WWGM-24 Gravity Main along Bobcat Road	12	$ 2,204,000	$ -	$ 2,204,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 2,204,000
WWGM-25 Gravity Main along Sunset Avenue	12	$ 7,716,000	$ -	$ 7,716,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 7,716,000
WWGM-26 Gravity Main along Westward Avenue	8	$ 870,000	$ -	$ 870,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 870,000
WWGM-27 Gravity Main along Mias Canyon Road and Bluff Street	8	$ 3,626,000	$ -	$ 3,626,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 3,626,000
WWGM-28 Gravity Main along Florida Street	8	$ 435,000	$ -	$ 435,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 435,000
WWGM-29 Gravity Main along Almond and Blanchard Street	8	$ 435,000	$ -	$ 435,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 435,000
WWGM-30 Casing for Gravity Main Crossing I-10	12/24	$ 854,000	$ -	$ 854,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 854,000
WWGM-31 Gravity Main along Lincoln Street	8	$ 870,000	$ -	$ 870,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 870,000
WWGM-32 Gravity Main along Ramsey Street	8	$ 435,000	$ -	$ 435,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 435,000
Force Mains	**Diameter (in)**	**$ 3,045,000**	**$ -**	**$ 3,045,000**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ 1,160,000**	**$ 1,885,000**
WWFM-2 Force Main along Westward Avenue	8	$ 1,160,000	$ -	$ 1,160,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,160,000	$ -
WWFM-3 Force Main along Porter Street	6	$ 1,305,000	$ -	$ 1,305,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,305,000
WWFM-4 Force Main along Roadrunner Trail	6	$ 290,000	$ -	$ 290,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 290,000
WWFM-5 Force Main Creek Crossing	6	$ 290,000	$ -	$ 290,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 290,000
Lift Stations	**Quantity (mgd)**	**$ 7,633,000**	**$ -**	**$ 7,633,000**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ 4,057,000**	**$ 3,576,000**
WWLS-2 Distribution Center Lift Station	1.90 mgd	$ 2,596,000	$ -	$ 2,596,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 2,596,000	$ -
WWLS-3 Business Park Lift Station	0.62 mgd	$ 1,461,000	$ -	$ 1,461,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,461,000	$ -
WWLS-4 Porter Street Lift Station	0.16 mgd	$ 1,076,000	$ -	$ 1,076,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,076,000
WWLS-5 Roadrunner Trail Lift Station	0.34 mgd	$ 1,225,000	$ -	$ 1,225,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,225,000
WWLS-6 Bluff Street Lift Station	0.40 mgd	$ 1,275,000	$ -	$ 1,275,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,275,000
Rehabilitation and Replacement Projects		$ 3,514,000	$ 3,406,000	$ 108,000	$ 146,000	$ 350,000	$ -	$ 100,000	$ 350,000	$ -	$ 60,000	$ 350,000	$ 2,050,000	$ 108,000
Gravity Mains		**$ 3,280,000**	**$ 3,280,000**	**$ -**	**$ 60,000**	**$ 350,000**	**$ -**	**$ 60,000**	**$ 350,000**	**$ -**	**$ 60,000**	**$ 350,000**	**$ 2,050,000**	**$ -**
WWRR-1 Annual Sewer Replacement	NA	$ 3,280,000	$ 3,280,000	$ -	$ 60,000	$ 350,000	$ -	$ 60,000	$ 350,000	$ -	$ 60,000	$ 350,000	$ 2,050,000	$ -
Lift Stations		**$ 234,000**	**$ 126,000**	**$ 108,000**	**$ 86,000**	**$ -**	**$ -**	**$ 40,000**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ 108,000**
WWRR-2 Caltrans Lift Station Site Improvements	N/A	$ 148,000	$ 40,000	$ 108,000	$ -	$ -	$ -	$ 40,000	$ -	$ -	$ -	$ -	$ -	$ 108,000
WWRR-3 Westward Lift Station Site Improvements	N/A	$ 86,000	$ 86,000	$ -	$ 86,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Treatment Plant Related Improvements		$ 27,320,000	$ 5,320,000	$ 22,000,000	$ 280,000	$ 1,890,000	$ 15,150,000	$ 10,000,000	$ -	$ -	$ -	$ -	$ -	$ -
WWTP-1 Digestor Cleaning	N/A	$ 150,000	$ 150,000	$ -	$ -	$ -	$ 150,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -
WWTP-2 Heat Exchanger Repairs	N/A	$ 60,000	$ 60,000	$ -	$ 10,000	$ 50,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -

Table 9.11 Wastewater Capital Improvement Plan Summary

Project		Proposed Size/Diameter	CIP Cost Estimate	City Cost	Developer Cost	CIP Phasing									
						Near-Term								Long-Term	Build-Out
						2018	2019	2020	2021	2022	2023	2024	2025	2026-2040	2041 & beyond
WWTP-3	Boiler Gas Control Valves	N/A	$ 80,000	$ 80,000	$ -	$ 15,000	$ 65,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
WWTP-4	Digestor Gas Pipeline	N/A	$ 30,000	$ 30,000	$ -	$ 5,000	$ 25,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
WWTP-5	WWTP Upgrade	N/A	$ 27,000,000	$ 5,000,000	$ 22,000,000	$ 250,000	$ 1,750,000	$ 15,000,000	$ 10,000,000	$ -	$ -	$ -	$ -	$ -	$ -
Other Projects			**$ 5,561,000**	**$ 5,561,000**	**$ -**	**$ -**	**$ 50,000**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ 1,595,000**	**$ 3,916,000**
WWO-1	Septic Removal	8	$ 5,511,000	$ 5,511,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,595,000	$ 3,916,000
WWO-2	Lift Station Telemetry	N/A	$ 50,000	$ 50,000	$ -	$ -	$ 50,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
CIP Total			**$ 99,210,000**	**$ 25,498,000**	**$ 73,712,000**	**$ 898,000**	**$ 3,334,000**	**$ 15,606,000**	**$ 13,026,000**	**$ 8,632,000**	**$ 3,858,000**	**$ 1,867,000**	**$ 1,087,000**	**$ 12,023,000**	**$ 38,879,000**
Annual Cost			**N/A**	**N/A**	**N/A**	**$ 898,000**	**$ 3,334,000**	**$ 15,606,000**	**$ 13,026,000**	**$ 8,632,000**	**$ 3,858,000**	**$ 1,867,000**	**$ 1,087,000**	**$ 802,000**	**N/A**

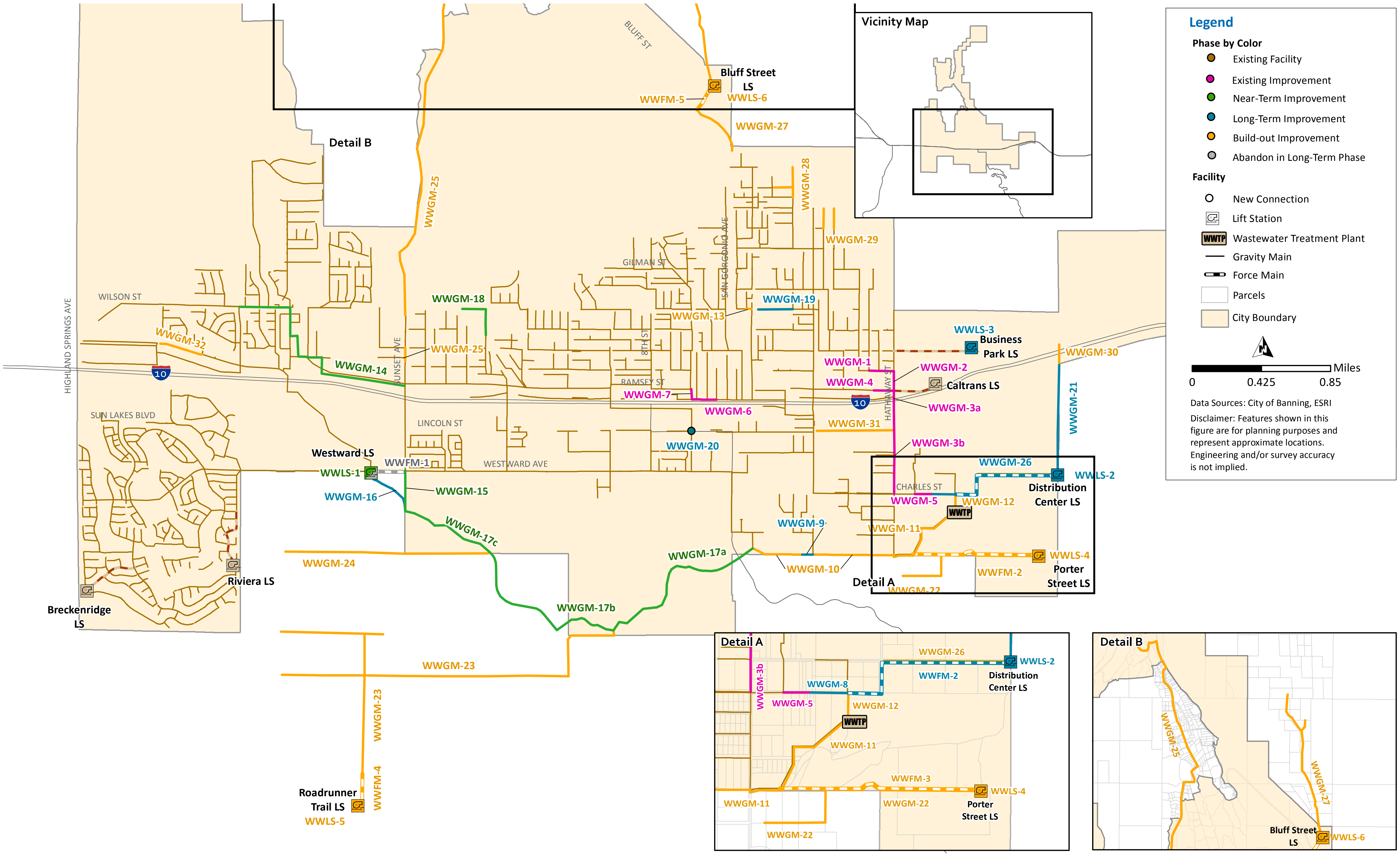

Figure 9.10 Wastewater CIP Map

Table 9.12 Recycled Water Capital Improvement Plan Summary

Project	Proposed Size/Diameter	CIP Cost Estimate	City Cost	Developer Cost	CIP Phasing									
					Near-Term								Long-Term	Build-out
					2018	2019	2020	2021	2022	2023	2024	2025	2026-2040	2041 & beyond
Capacity and Reliability Improvements		$ 31,750,000	$ 12,116,000	$ 19,634,000	$ 12,812,000	$ 4,330,000	$ 4,144,000	$ 5,801,000	$ 1,116,000	$ 1,015,000	$ -	$ -	$ 2,333,000	$ 199,000
Pipelines	**Diameter (in)**													
RWP-1 Recycled Water Backbone System	24	$ 14,172,000	$ 6,378,000	$ 7,794,000	$ 5,905,000	$ 4,330,000	$ 3,937,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -
RWP-2 Lion's Park Lateral	6	$ 435,000	$ -	$ 435,000	$ 435,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
RWP-3 Banning High School Lateral	6	$ 435,000	$ -	$ 435,000	$ -	$ -	$ -	$ -	$ 435,000	$ -	$ -	$ -	$ -	$ -
RWP-4 Rancho San Gorgonio Lateral	12	$ 207,000	$ -	$ 207,000	$ -	$ -	$ 207,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -
RWP-5 Neighborhood Park Lateral	6	$ 145,000	$ -	$ 145,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 145,000	$ -
RWP-6 Dysart Park Lateral	6	$ 1,015,000	$ -	$ 1,015,000	$ -	$ -	$ -	$ -	$ -	$ 1,015,000	$ -	$ -	$ -	$ -
RWP-7 Five Bridges Development Lateral	10	$ 199,000	$ -	$ 199,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 199,000
RWP-8 Well R-1 Pipeline	12	$ 1,036,000	$ 466,000	$ 570,000	$ 1,036,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
RWP-9 Five Bridges Basin Pipeline	16	$ 1,641,000	$ 738,000	$ 903,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 1,641,000	$ -
RWP-10 WWTP Basin Pipeline	16	$ 547,000	$ 246,000	$ 301,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 547,000	$ -
Booster Pump Stations	**Quantity (hp)**													
RWPS-1 WWTP Recycled Water Pump	1400	$ 5,801,000	$ 2,610,000	$ 3,191,000	$ -	$ -	$ -	$ 5,801,000	$ -	$ -	$ -	$ -	$ -	$ -
Wells	**Quantity (well)**													
RWW-1 Equip Well R-1	1	$ 1,707,000	$ -	$ 1,707,000	$ 1,707,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Storage	**Quantity (tank)**													
RWS-1 Well R-1 Forebay	1	$ 3,729,000	$ 1,678,000	$ 2,051,000	$ 3,729,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Valves	**Quantity (PRV)**													
RWV-1 BCVWD Co-owned Wells PRV	2	$ 681,000	$ -	$ 681,000	$ -	$ -	$ -	$ -	$ 681,000	$ -	$ -	$ -	$ -	$ -
Other Projects		$ 7,072,000	$ 7,072,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 150,000	$ 3,938,000	$ 2,984,000	$ -
RWO-1 Five Bridges Site Improvements	n/a	$ 3,194,000	$ 3,194,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 3,194,000	$ -	$ -
RWO-2 WWTP Basin Site Improvements	n/a	$ 411,000	$ 411,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 411,000	$ -	$ -
RWO-3 Hydrogeological Study	n/a	$ 150,000	$ 150,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 150,000	$ -	$ -	$ -
RWO-4 Monitoring Wells and Lysimeters	n/a	$ 2,984,000	$ 2,984,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 2,984,000	$ -
RWO-5 404 Permitting	n/a	$ 200,000	$ 200,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 200,000	$ -	$ -
RWO-6 Recycled Water Master Plan Update	n/a	$ 133,000	$ 133,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 133,000	$ -	$ -
CIP Total		**$ 38,822,000**	**$ 19,188,000**	**$ 19,634,000**	**$ 12,812,000**	**$ 4,330,000**	**$ 4,144,000**	**$ 5,801,000**	**$ 1,116,000**	**$ 1,015,000**	**$ 150,000**	**$ 3,938,000**	**$ 5,317,000**	**$ 199,000**
Annual Cost		**N/A**	**N/A**	**N/A**	**$ 12,812,000**	**$ 4,330,000**	**$ 4,144,000**	**$ 5,801,000**	**$ 1,116,000**	**$ 1,015,000**	**$ 150,000**	**$ 3,938,000**	**$ 354,000**	**N/A**

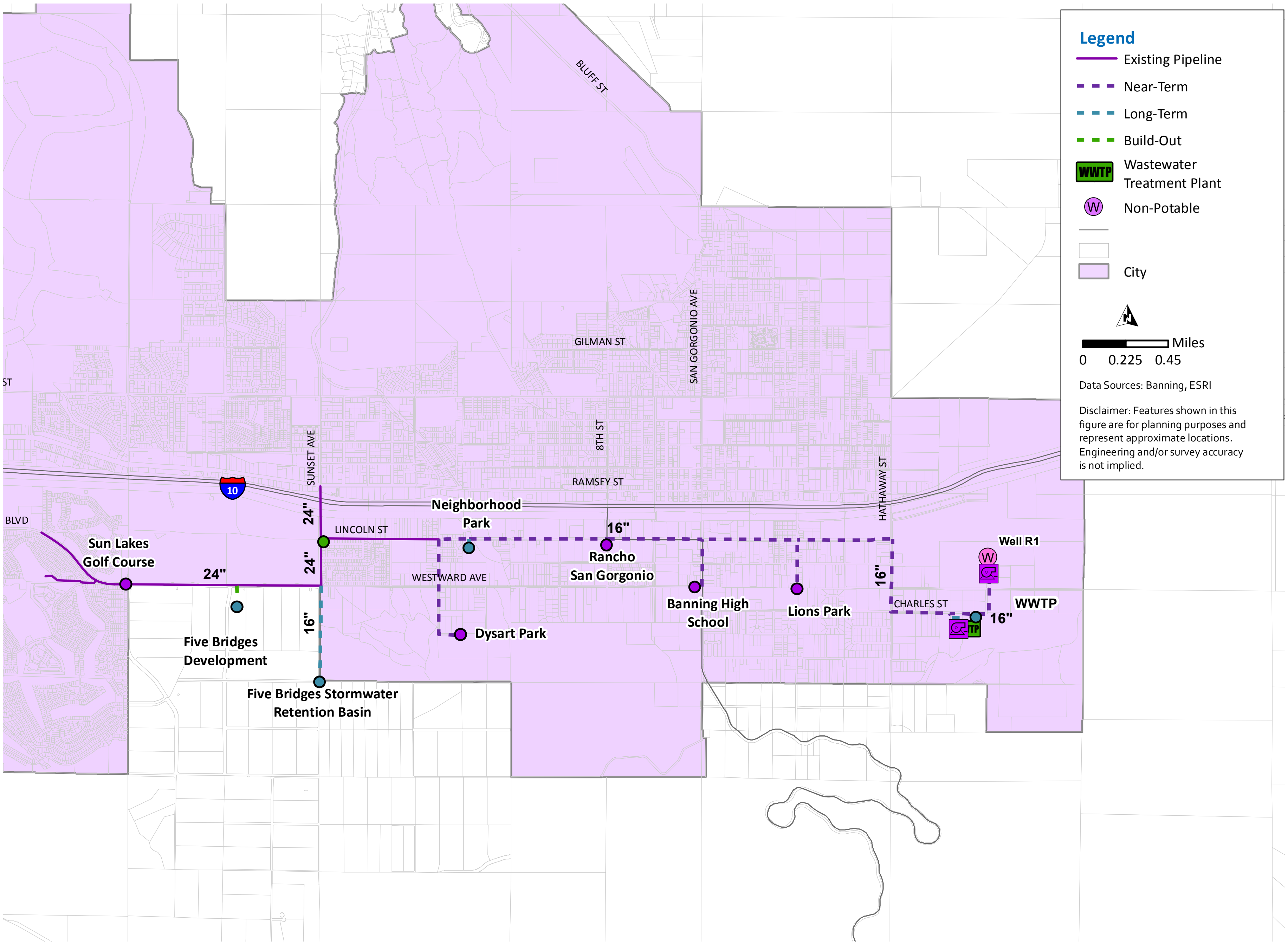

Figure 9.11 Recycled Water CIP Map

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