DECEMBER 2022

WATER SYSTEMS MASTER PLAN

Municipal Utilities and Engineering Department

City of Redlands
35 Cajon Street
Redlands, CA 92373

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ABBREVIATIONS

2020 UWMP	2020 Urban Water Management Plan
2021 WSCP	2021 Water Shortage Contingency Plan
2022 WSMP	2022 Water System Master Plan
AACE	Association for the Advancement of Cost Engineering
AF	Acre-Feet
Acre-ft/d	Acre-Feet per Day
Acre-ft/yr.	Acre-Feet per Year
ADD	Average Day Demand
AMI	Advanced Metering Infrastructure
AMR	Automated Meter Reader
ASL	Above Sea Level
AWWA	American Water Works Association
BOD	Biological Oxygen Demand
Brady	Richard Brady and Associates, Inc.
BVMWC	Bear Valley Mutual Water Company
CalOSHA	California Division of Occupational Safety and Health Agency
CalWARN	California's Water/Wastewater Agency Response Network
CCR	Consumer Confidence Report
CIP	Capital Improvement Program
City	City of Redlands Municipal Utilities and Engineering Department
CWC	Crafton Water Company
DU	Dwelling Unit
EPA	Environmental Protection Agency
EPS	Extended Period Simulation
ERNIE	Emergency Response Network of the Inland Empire
ES	Equalizing Storage
Fps	Feet Per Second
FSS	Fire Suppression Storage
FY	Fiscal Year
GIS	Geographical Information System
Gpcd	Gallons per capita day
GPD	Gallons per day
GPM	Gallons per minute
HAA5	Haloacedic Acid
Hinckley	Horace P. Hinckley Water Treatment Plant
HP	Horsepower
I-10	Interstate 10
I-210	Interstate 210
MCL	Maximum Contaminate Level
MDD	Maximum Day Demand

MG	Million Gallons
MGD	Million Gallons per day
mg/L	Milligrams per Liter
MUED	Municipal Utilities and Engineering Department
NPW	Non-Potable Water
NPWMP	2005 Non-Potable Water System Master Plan
NTU	Nephelometric Turbidity Unit
O&M	Operations and Maintenance
OPC	Opinion of Probable Cost
OS	Operational Storage
PHD	Peak Hour Demand
PPM	Parts Per Million
PRS	Pressure Reducing Stations
PSI	Pounds per Square Inch
PW	Potable Water
PWMP	1998 Potable Water System Master Plan
RW	Recycled Water
SAR	Santa Ana River
SB	Stand-by Storage
SBBA	San Bernardino Basin Area
SCADA	Supervisor Control and Data Acquisition
SCE	Southern California Edison
SOC	Synthetic Organic Compounds
SOP	Standard Operating Procedure
SWP	State Water Project
SWRCB-DDW	State Water Resources Control Board – Division of Drinking Water
Tate	Henry Tate Water Treatment Plant
TBD	To be determined
TDS	Total Dissolved Solids
TSS	Total Suspended Solids
TTHM	Trihalomethanes
U.S.	United States
VOC	Volatile Organic Compounds
WD	Water Distribution
WP	Water Production
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plan

EXECUTIVE SUMMARY

Introduction

In 2021, the City of Redlands (City) initiated to prepare an update to the 1998 Potable Water System Master Plan (PWMP) and the 2005 Non-Potable Water System Master Plan (NPWMP). Both the PWMP and NPWMP, as well as other relevant documents were reviewed and referenced to prepare the 2022 Water Systems Master Plan (2022 WSMP). The 2022 WSMP includes the City's three (3) water systems: Non-Potable Water (NPW), Potable Water (PW), and Recycled Water (RW) Systems. The most critical outcome of the 2022 WSMP is the development of comprehensive five-year and twenty-year capital improvement program (CIP) recommendations to improve distribution efficiency, reduce non-revenue water, and accommodate growth within the City water service areas. A hydraulic model using Innovyze InfoWater software was developed to analyze existing systems, as well as the impacts of various system improvements. The City's Water Distribution and Water Production staff supplemented these efforts with decades of relative historic knowledge.

The ultimate goal for all three (3) water production and distribution systems is to ensure long-range sustainability for each of these water resources. The City intends to use recommendations developed and identified through this Master Planning process to reduce non-revenue water in each system, maximize the use of non-potable wells to extract contaminants from groundwater resources, and eventually hopes to serve all significant users with non-potable and/or recycled water for landscape irrigation and industrial uses.

The 2022 WSMP is divided into ten (10) Sections, each of which is summarized below.

Section 1 – General Information

This Section provides general information about the City and its water service area.

Section 2 - Existing Infrastructure

This Section details the City's existing NPW, PW, and RW Systems.

Section 3 - Historic Water Use

This Section quantifies and characterizes annual and seasonal water use patterns based on demand data included in the City's 2020 Urban Water Management Plan (2020 UWMP). This information was used to calibrate each hydraulic model, with particular emphasis on maintaining minimum fire protection pressure and flow under several modeling scenarios. It was also used to validate the positive impact of water conservation programs. Water demand projections through the 2045 planning horizon are provided as well.

Section 4 - Water Supply

This Section describes and quantifies the water sources for each system, and identifies various threats and production challenges associated with each system.

Section 5 - Design Criteria

This Section identifies specific water infrastructure planning and design criteria used to evaluate existing infrastructure and to develop CIP project recommendations. These include minimum storage requirements, pumping capacity, and system hydraulics for various demand scenarios.

Section 6 - Existing System Analysis

This Section summarizes development, calibration, and use of the hydraulic model to analyze the existing systems. Scenarios including historic Average Day Demands (ADD), Maximum Day Demands (MDD) with and without fire flow, and Peak Hour Demands (PHD) were modeled to identify deficiencies within each system.

Section 7 - Future System

This Section summarizes additional hydraulic modeling to identify deficiencies as project demands within each system are applied through the planning horizon. The results were used to develop CIP project recommendations for each system.

Section 8 - Water Quality Analysis

This Section summarizes regulatory requirements relative to each system, and identifies potential contamination sources, groundwater basin characteristics, and imported water quality. A distribution system water age analysis was hydraulically modeled to evaluate the potential for taste and odor complaints related to the reduction of disinfection residual over time.

Section 9 - Water System Operation and Maintenance

This Section summarizes findings of an Operation and Maintenance (O&M) practices analysis for each system, and includes recommended improvements.

Section 10 - Capital Improvement Program

This Section identifies recommended short-term (5-Years) and long-term (20-Years) CIP projects based on discussions with City staff and 2022 WSMP analyses. An Opinion of Probable Cost (OPC), in current year dollars, is provided for each CIP project recommendation. Each OPC is developed based on estimated construction quantities, actual construction costs for regional projects of similar scope and size, and vendor quotes. Finally, the OPC for each CIP project was adjusted to account for specific site conditions.

1 GENERAL INFORMATION

1.1 GENERAL DESCRIPTION

The City of Redlands is located in Southern California in the southwestern portion of the County of San Bernardino, and includes approximately 37.5 square miles of land area. The City is approximately sixty (60) miles east of the City of Los Angeles, adjacent to the eastern borders of the cities of San Bernardino and Loma Linda, adjacent to the southern border of the City of Highlands, and adjacent to the western border of the City of Yucaipa. The unincorporated areas of Mentone and Crafton, located just east of the City. The City limits are shown in Figure 1-1.

Several major transportation corridors transverse the City: Interstate 210 (I-210) crosses the western portion of the City, Interstate 10 (I-10) transverses east/west through the middle of the City, and State Route 38 begins within the City limits and transverses east through the City. The Santa Ana River and Mill Creek watersheds are located in the northern and eastern sections of the City, respectively. These watersheds are the source of approximately half of the City's total water supply.

The topography of the City generally slopes downward in the northwesterly direction. The San Bernardino Mountain range is situated to the northeast, Zanja Peak to the east, and the San Jacinto Mountain range to the south. Elevations within the City limits varies between 1,100 feet above sea level (ASL) in the northwest, to 3,300 feet ASL in the east. The City's water service area reaches 2,500 feet ASL. This broad variation in elevation requires multiple pressure zones to distribute water within accepted industry standards throughout each system.

1.2 LAND USES

Although the City is home to large employers such as ESRI, Redlands Community Hospital, and the University of Redlands, the south and east portions of the City are primarily residential communities, with undeveloped land and recreational areas in the San Jacinto foothills. The City's northwestern area is primarily comprised of industrial, commercial, and office developments. The center of the City is primarily residential communities, with small agricultural areas to the east. Most undeveloped areas are in the southern portion of the City and south of the Santa Ana River.

City's current General Plan (2017) was developed to responsibly manage residential, industrial, and commercial growth. The General Plan also considers the City's “Sphere of Influence,” which includes the unincorporated areas of Crafton and Mentone, including large Rural Living and agricultural tracts. Of the two (2), Mentone is more heavily developed, with residential communities and light industrial uses in the west, and agricultural regions in the northwest. It should be noted that the City’s sphere of influence includes small portions in the cities of San Bernardino, Loma Linda, Yucaipa, and the County of San Bernardino and are within Redlands' water service area.

The City provides potable and non-potable water to a retail sales area, commonly referred to as the “Donut Hole”, northwest of the intersection of I-10 and I-210. This area is not within the City of Redlands jurisdiction, but is an unincorporated part of the County of San Bernardino. This area is primarily developed for commercial and industrial use.

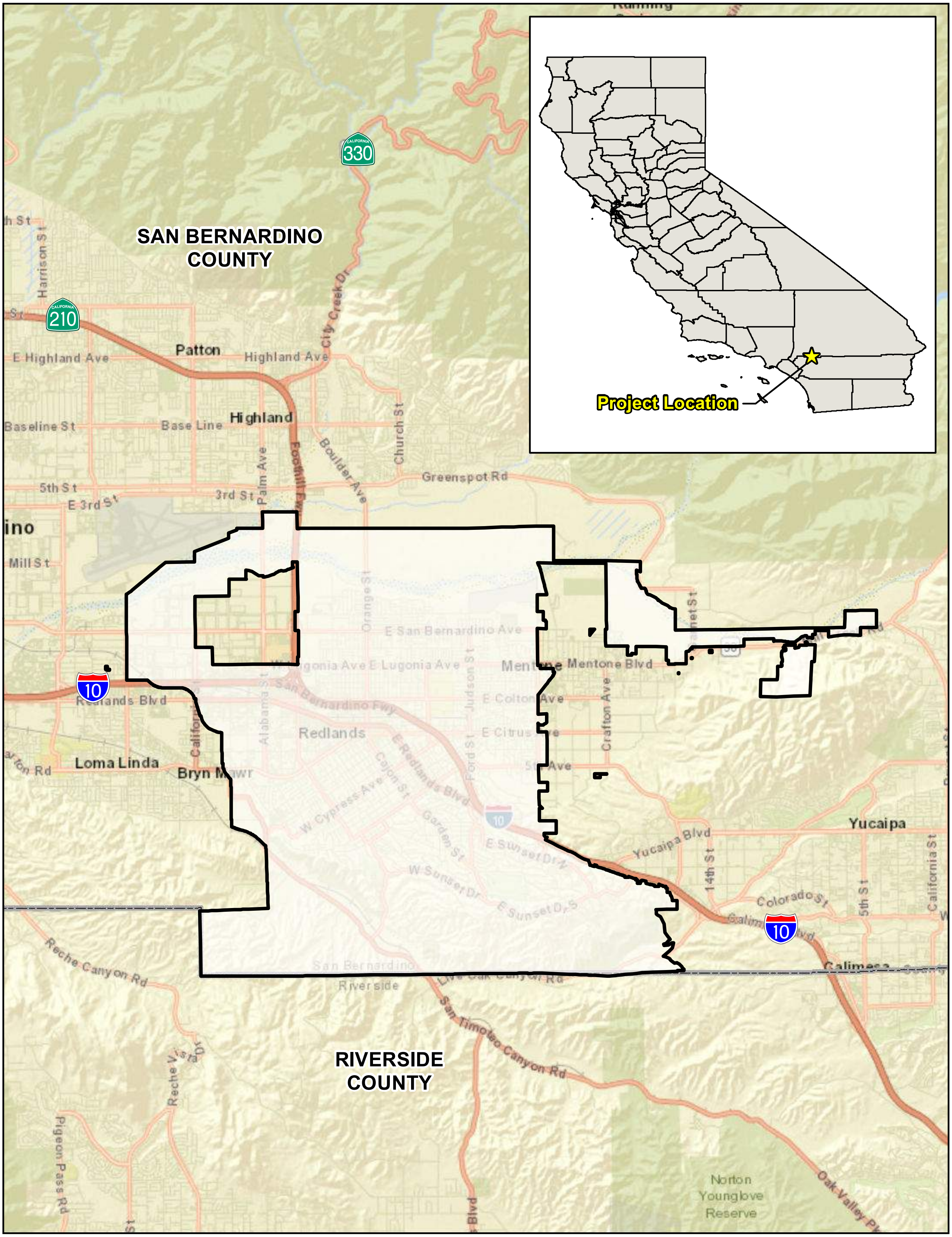

Figure 1-1
City of Redlands City Limits

Table 1-1 shows land use designations by area as identified in the General Plan. The comparative use of each category is shown graphically in Chart 1-1. The land use from the City's General Plan is shown in Figure 1-2.

Table 1-1: City of Redlands Land Use

Land Use	City of Redlands (Acres)	Sphere of Influence (Acres)	Total
Residential	6,343	4,042	10,385
Rural Living	9	2,115	2,124
Very Low Density Residential	2,694	861	3,555
Low Density Residential	2,643	574	3,217
Low Medium Density Residential	63	469	532
Medium Density Residential	520	23	543
High-Density Residential	414	-	414
Office, Commercial, Industrial	2,626	147	2,773
Office	206	-	206
Commercial	866	55	921
Commercial/Industrial	1,249	-	1,249
Light Industrial	305	92	397
Agricultural and Hillside	5,122	1,322	6,444
Agricultural	308	220	528
Hillside Conservation	23	1,102	1,125
Resource Preservation	4,791	-	4,791
Public/Institutional	1,271	130	1,401
Open Space	5,111	510	5,621
Parks & Golf Courses	600	-	600
Open Space	4,511	510	5,021
City ROW	2,881	418	3,299
Total	42,556	12,590	55,146
Note: Data from 2017 General Plan			

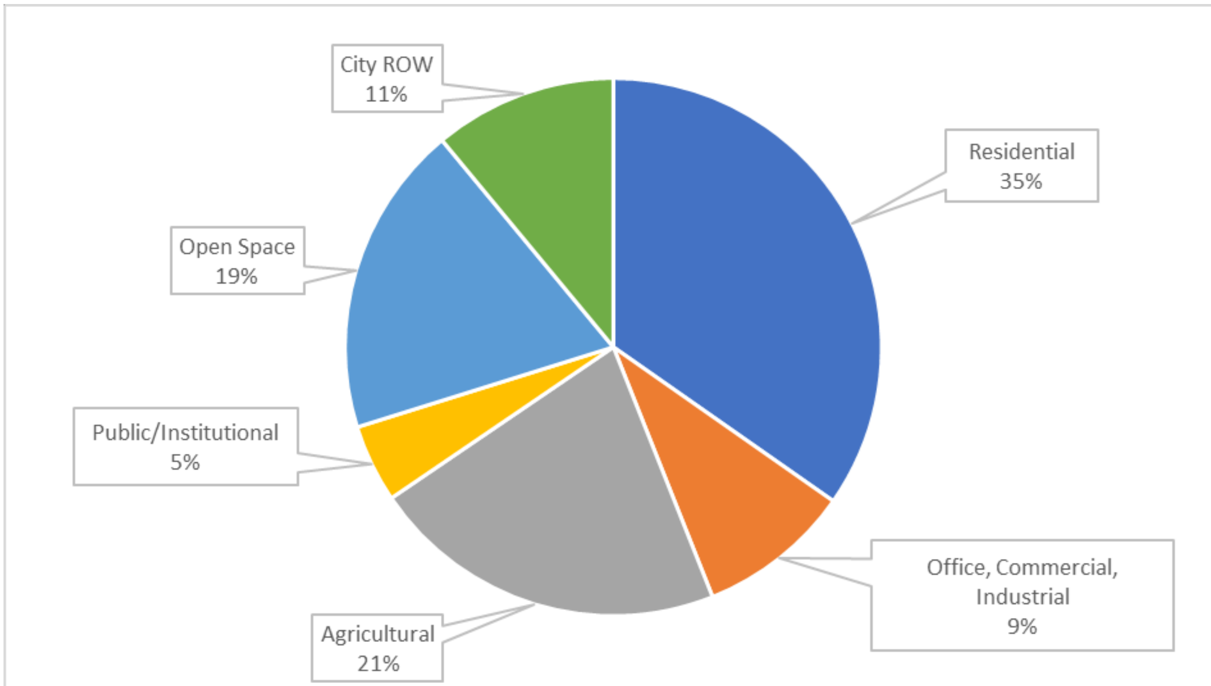

Chart 1-1: Land Use Areas

Residential development areas comprise approximately thirty-five percent (35%) of the City's total land use, and is divided into six (6) sub-categories: Rural Living, Very Low-Density Residential, Low-Density Residential, Low-Medium Density Residential, Medium-Density Residential, and High-Density Residential. Most Rural Living areas are within Crafton and allow a maximum of one (1) dwelling unit (DU) per five (5) acres of land area. These areas offer an opportunity to expand non-potable water service for agricultural uses, thereby reducing demand for potable water resources.

Office, Commercial and Industrial development areas are consolidated into a single category, and comprise approximately nine percent (9%) of the City's total land use. The General Plan considers maintenance and growth of these areas as essential to striking a balance between residential and employment areas. Office land uses include traditional office spaces and medical offices. Commercial land uses include neighborhood-serving convenience stores, retail centers, and commercial recreational areas. Industrial land use ranges from light industries such as automotive services, to research and development and heavy industry manufacturing facilities. These areas offer an opportunity to expand non-potable and recycled water service, thereby reducing demand for potable water resources.

Agricultural land uses include traditional agricultural areas, as well as areas designed for preservation and conservation of natural resources. This category comprises approximately twenty-one percent (21%) of the City's total land use. Most of this category is in mountain areas and is designated as Resource Preservation, which includes wildlife preservation and activities such as water conservation, open space recreation, and agriculture. These areas offer an opportunity to expand non-potable water service for agricultural uses, thereby reducing demand for potable water resources.

Public/Institutional land uses include developments used for public services, schools, government facilities, airports, public utilities, and facilities used or owned by the City. This category comprises approximately five percent (5%) of the City's total land use.

Open Space land uses comprise approximately nineteen percent (19%) of the City's total land use. This category is divided into parks or miscellaneous open spaces that are primarily unimproved with no immediate plans for development. These areas offer an opportunity to expand non-potable and recycled water service for outdoor irrigation, thereby reducing demand for potable water resources.

City Rights-of-Way (ROW) category is predominantly public streets, and comprises approximately eleven percent (11%) of the City's total land use.

It is important to note that there appear to be some exceptions to the generalization of the categories. For example, the City owns and maintains approximately 184 acres of Citrus Groves. These groves are surrounded by industrial and residential areas and appear to be considered in the nearby corresponding category and are not included in the Open Space category. Additionally, most of the Rural Living sub-categorized land is used for agriculture, but is classified as residential.

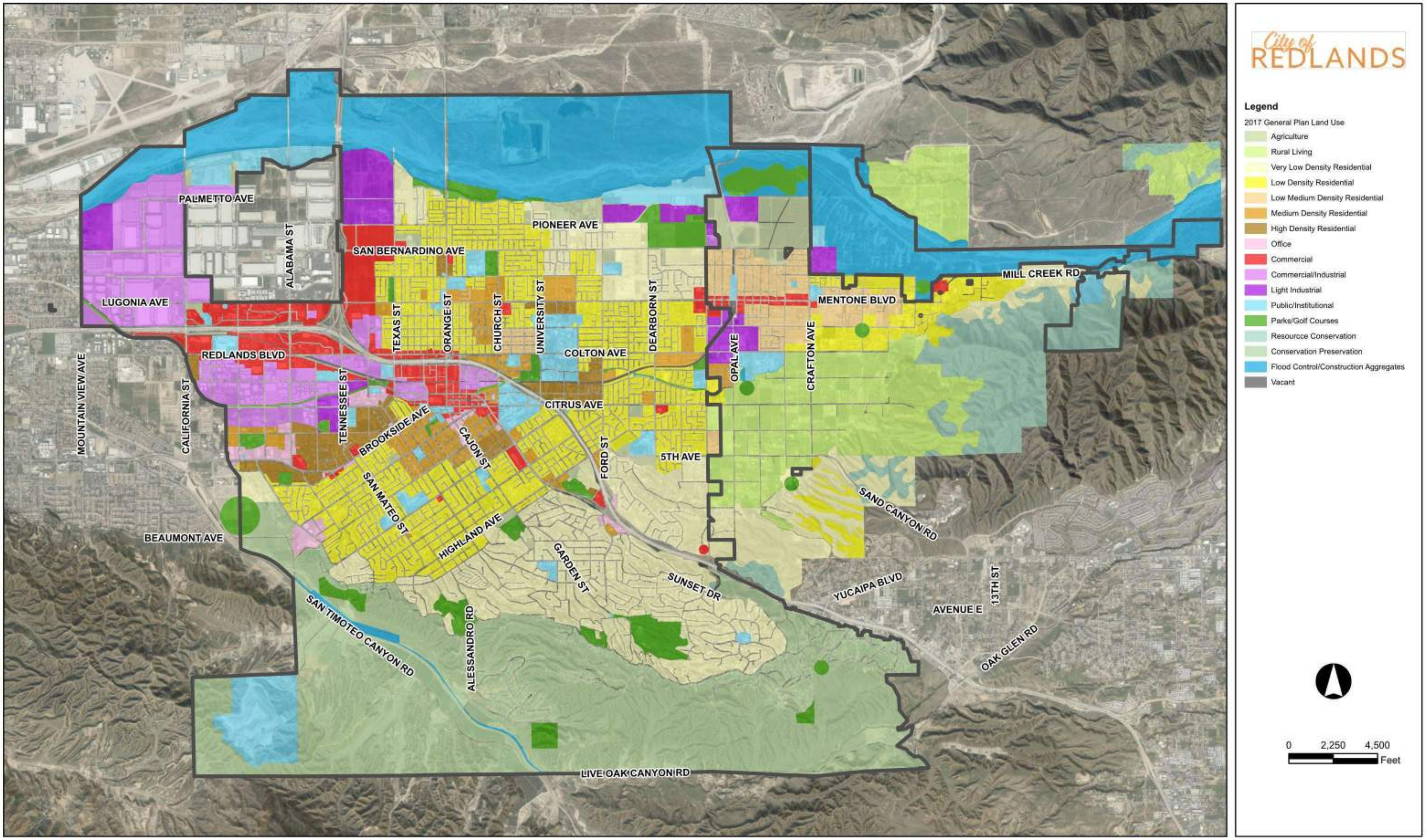

Figure 1-2
City of Redlands City Limits

1.3 WATER SERVICE AREA

The City's water system serves multiple areas within and beyond the City limits, which includes eastern portions of the cities of San Bernardino and Loma Linda, western portions of the City of Yucaipa, and the unincorporated areas of Mentone, Crafton, and the "Donut Hole". The City distributes potable water within seven (7) pressure zones, non-potable water within three (3) pressure zones, and recycled water within one (1) pressure zone to provide appropriate water pressure throughout each system. These pressure zones are detailed in Section 2. Although the recycled water distribution system is currently limited to a small area in the northwest portion of the City, the non-potable water distribution provides service to a much larger portion of the City and is separated into eight (8) detached systems.

1.4 POPULATION

The City's population has grown at a consistent pace for the last twenty (20) years. In 2020, the City population was 73,168 (2020 Unites States Census data). The General Plan predicts the population will rise to 79,000 by 2035, when the City expects to reach built-out conditions. Table 1-2 provides reported population data and growth rates since 1960.

Table 1-2: Redlands Population Growth (1960-2045)

Year	Population	Annual Growth Rate
1960	27,000	--
1970	35,000	3.0%
1980	42,000	2.0%
1990	61,000	4.5%
2000	63,591	0.4%
2005	66,342	0.9%
2010	68,747	0.7%
2015	70,112	0.4%
2020	73,168	0.9%

Note: Based on Chart 3-1 from the 2017 General Plan

The water service area extends beyond the City limits to serve residents and businesses within the municipalities and unincorporated areas described in Section 1.3. The 2020 UWMP estimates that the portions of the water service area beyond the City limits serve an additional 10,000 residents currently, and is anticipated to increase to 16,000 by 2045 when these areas are built out. At that time, the City will serve a total population of 95,000. This projection anticipates uniform population growth. Table 1-3 shows the projected population growth of the total water service area through 2045.

Table 1-3: Water Service Area Predicted Population Growth

Year	2020	2025	2030	2035	2040	2045
Population Served	83,000	85,400	87,900	90,300	92,700	95,153
Note: From Table 4-1 in the 2020 Urban Water Management Plan						

2 EXISTING INFRASTRUCTURE

2.1 GENERAL DESCRIPTION

2.1.1 POTABLE WATER SYSTEM

The City receives surface water and groundwater from several sources. Two (2) surface water treatment plants, the Horace P. Hinckley Water Treatment Plant (Hinckley) and Henry Tate Water Treatment Plant (Tate), receive surface water primarily from the Santa Ana River (Hinckley) and Mill Creek (Tate). Both sources can also be supplemented by the State Water Project (SWP). Additionally, seventeen (17) active groundwater wells that include four (4) active wellhead treatment systems are strategically located throughout the service area to diversify the City's potable water production sources. Historically, surface water treatment accounts for approximately fifty-one percent (51%) of total annual potable water production, and groundwater treatment accounts for approximately forty-nine percent (49%) of total annual potable water production.

Potable water is stored in eighteen (18) reservoirs, and is distributed through approximately 450 miles of pipelines within seven (7) pressure zones to provide uniform service pressures. Thirty-eight (38) booster pumps and thirty (30) zone transfer control valves distribute water throughout the system while maintaining each zone's desired hydraulic grade line.

2.1.2 NON-POTABLE WATER SYSTEM

The non-potable water distribution system is limited in service area, and provides untreated groundwater primarily for outdoor landscape irrigation.

2.1.3 RECYCLED WATER SYSTEM

The recycled water distribution system is limited in service area, and provides treated effluent from the City Wastewater Treatment Plant (WWTP) to customers within a small area in the northwest portion of the City. The largest recycled water customer is the Southern California Edison (SCE) Mountain View Power Plant, which uses recycled water through a "Take-or-Pay" agreement to cool equipment. The recycled water volume in this agreement is 3,000 AF/Year.

2.2 PRESSURE ZONES

POTABLE WATER PRESSURE ZONES

The City's potable water pressure zone distribution system was first developed in 1975 and later updated in 1981. Figure 2-1 shows the seven potable water pressure zones and the locations of existing reservoirs, wells, booster pumps, and water treatment plants. The overall system hydraulic profile is shown in Figure 2-2. Each pressure zone is described in detail below.

2.2.1 ZONE 1350

Pressure Zone 1350 serves the western portion of the City's water service area. It is located west of Alabama Street, north and south of the I-10 Freeway, and west of New Jersey Street. The terrain slopes downward northwesterly, and its elevation ranges from 1,050 feet to 1,250 feet. Two (2) reservoirs are used to service this zone: Texas Street and Texas Grove Reservoirs. In addition, two (2) potable wells supply water to this pressure zone: Orange Street 1 and Orange Street 2. These wells provide groundwater directly into the Texas Street and Texas Grove Reservoirs. In addition, four (4) booster pumps labeled

1550, 1551, 1552, and 1553 are located at the Texas Street reservoir site to lift water from Zone 1350 to Zone 1570.

2.2.2 ZONE 1570

Pressure Zone 1570 is bounded by the I-10 Freeway between Alabama Street and University Street and south of I-10 Freeway between New Jersey Street and Cypress Avenue. The terrain slopes downward northwesterly, and its elevation ranges from 1,190 feet to 1,470 feet. Three (3) reservoirs are used to service this zone: Dearborn, Highland Avenue, and Smiley Heights Reservoirs. Six (6) groundwater wells supply water to this pressure zone: Well No. 10, Well No. 13, Well No. 38, Well No. 39, Church Street Well, and Orange Street Well. Well No. 10 and Well No. 13 are able to supply groundwater directly into the Highland Avenue Reservoir. However, both have been idle over the last five (5) years. Ten (10) booster pumps are used to service this zone. Two (2) booster pumps, 1783 and 1784, are located at the Smiley Reservoir and lift water into Zone 1750. Pumps 1761 and 1931 are located at the Dearborn Reservoir, where Pump 1761 lifts water to Zone 1750 and Pump 1931 lifts water to Zone 1900. Booster pumps 1720, 1721, 1722, 2174, 2176, and 2177 are located at the Highland Reservoir and lift water to Zones 1750 and 2100.

2.2.3 ZONE 1750

Pressure Zone 1750 serves the downtown area of the City of Redlands. It is bounded on the southwest by Cypress Avenue, on the northwest by University Street, and on the northeast by Bear Valley Canal and Wabash Avenue. Highland Avenue and adjoining streets cover the southeast boundary of this pressure zone. The terrain slopes downward in a northwesterly direction, and its elevation ranges from 1,390 feet to 1,661 feet. Three (3) reservoirs are used to service this zone: Agate Avenue, South Avenue, and Arroyo Reservoirs. The Hinckley Water Treatment Plant (WTP) supplies water directly into the Agate Reservoir. In addition, six (6) groundwater wells provide water to this zone: Agate 2 Well, Airport 1 Well, Airport 2 Well, Mentone Acres 2 Well, Rees Well, and Muni Well. There are ten (10) booster pumps that are located within this zone. Situated at South Reservoir, booster pumps 1927 and 1928 lift water to Zone 1900 and booster pumps 2124, 2125, and 2126 lift water to Zone 2100. Located at Agate Reservoir, booster pumps 1951, 1952, and 1953 lift water to Zone 1900. Booster pumps 1723 and 1724 are used to move water within this zone.

2.2.4 ZONE 1900

Pressure Zone 1900 is bounded to the west by Highland Avenue and Wabash Avenue, King Street south of Colton Avenue, and Crafton Avenue north of Colton Avenue on the north side of the I-10 Freeway. South of the I-10 Freeway, the zone is bordered by Highland Avenue, Sunset Drive, Center Street, Elizabeth Street, Sunridge Way, Lynne Court, and Ford Street. The terrain slopes downward in a northwesterly direction, and its elevation ranges from 1,470 feet to 1,800 feet. Two (2) reservoirs are used to service this zone: Fifth Avenue and Margarita Reservoirs. In addition, one (1) groundwater well, the Madeira Well, supplies water to this pressure zone. Booster pumps 2131, 2132, 2310, and 2311 are located at the Fifth Avenue Reservoir. Pumps 2131 and 2132 are used to lift water to Zone 2100. Pumps 2310 and 2311 are used to lift water to Zone 2340.

2.2.5 ZONE 2100

Pressure Zone 2100 is bounded by King Street south of Colton Avenue, Crafton Avenue north of Colton Avenue, Reservoir Road, and Highland Avenue, and extends southwesterly towards the I-10 Freeway. South of the I-10 Freeway, the zone is bordered by Center Street, Elizabeth Street, Sunridge Way, Lynne

Court, Ford Street, Wabash Avenue, and the Redlands Country Club. The terrain slopes downward northwesterly, and its elevation ranges from 1,640 feet to 2,060 feet. Three (3) reservoirs are used to service this zone: County Club 1, Country Club 2, and Ward Way Reservoirs. Three (3) wells supply groundwater to this pressure zone: Lugonia 3 Well, Lugonia 6 Well, and Maguet 2 Well. Booster pumps 2384, 2385, 2386, and 2387 are located at the Country Club Reservoir and lift water to Zone 2340. Booster pumps 2381 and 2382 are situated at the Ward Way Reservoir and lift water to Zone 2600.

2.2.6 ZONE 2340

Pressure Zone 2340 is bounded by Sunset Drive, the Redlands Country Club, and Wabash Avenue, south of I-10 Freeway. North of the I-10 Freeway, Zone 2340 extends southeast along Sand Canyon Road from Crafton Avenue to Colorado Street and extends along Mill Creek Road, east of Orange Lane. The terrain slopes downward northwesterly, ranging from about 1,890 feet to 2,340 feet in elevation. Two (2) reservoirs are used to service this zone: Sand Canyon and Sunset Reservoirs. The Tate WTP is located within this zone and supplies water into the Ward Way Reservoir in Zone 2100. Pumps 2610 and 2611 are located at the Sand Canyon Reservoir and lift water to Zone 2600.

2.2.7 ZONE 2600

Pressure Zone 2600 extends along Mill Creek Road, east of the Mill Creek Reservoir and Crafton Hills Reservoir, to Crafton Hills College. The terrain slopes downward northwesterly, and its elevation ranges from 2,270 feet to 2,480 feet. The Crafton, Mill Creek 1, and Mill Creek 2 Reservoirs are located in this pressure zone. Booster Pumps 2510 and 2511 A/B are situated at the Mill Creek Reservoir and transfer water within Zone 2600.

NON-POTABLE WATER SYSTEMS

2.2.8 SYSTEM 1

This non-potable water system is located in the 1350 pressure zone. The system receives recycled water from the WWTP and untreated groundwater from the California Street Well hydro pneumatic system.

2.2.9 SYSTEM 2

This non-potable water system is located in the 1570 pressure zone. Well No. 30A, Well No. 31A, and Well No. 32 provide untreated groundwater to this system. System pressure is maintained by two (2) package skid booster pumps.

2.2.10 SYSTEM 3

This non-potable water system is a gravity fed located in the 1570 pressure zone. Well No. 41 and New York Street Well provide untreated groundwater to this system. This system is a weir box gravity fed system also known as the "B" Contract system.

2.2.11 SYSTEM 4

This non-potable water system is located in the 1570 pressure zone. Well No. 11 provides untreated groundwater for landscape irrigation to Ford Park. System pressure is maintained by a hydro pneumatic tank.

2.2.12 SYSTEM 5

This non-potable water system is located in the 1570 pressure zone. Well No. 16 is a gravity fed system that provides untreated groundwater to Bear Valley Mutual Water Company (BVMWC) for non-potable deliveries within their distribution network.

2.2.13 SYSTEM 6

This non-potable water system is located in the 1750 pressure zone. Agate Well No. 1 and Crafton Well is a gravity fed system that provides untreated groundwater to BVMWC for non-potable deliveries within their distribution network.

2.2.14 SYSTEM 7

This non-potable water system is located in the 1900 pressure zone. Redland Heights Well is a gravity fed system that provides untreated groundwater for landscape irrigation to Redlands Country Club.

2.2.15 SYSTEM 8

This non-potable water system is located in the 1900 pressure zone. Well No. 36 is a closed loop system that provides untreated groundwater for landscape irrigation to Hillside Memorial Park.

RECYCLED WATER PRESSURE ZONES

2.2.16 ZONE 1

This recycled water pressure zone is bounded by the Santa Ana River to the north, Mountain View Avenue to the west, Citrus Avenue and I-10 to the south, and Alabama Street and New Jersey Street to the east. Treated effluent from the City WWTP is provided to customers within this small area in the northwest portion of the City. The largest recycled water customer is the SCE Mountain View Power Plant, which uses recycled water through a “Take-or-Pay” agreement to cool equipment. Although this system is capable of blending flow with untreated groundwater provided by the California Street Well, only recycled water can serve the SCE Mountain View Avenue power plant. Mountain View Power Plant has their own well that can make up to fifty percent (50%) of their cooling water needs.

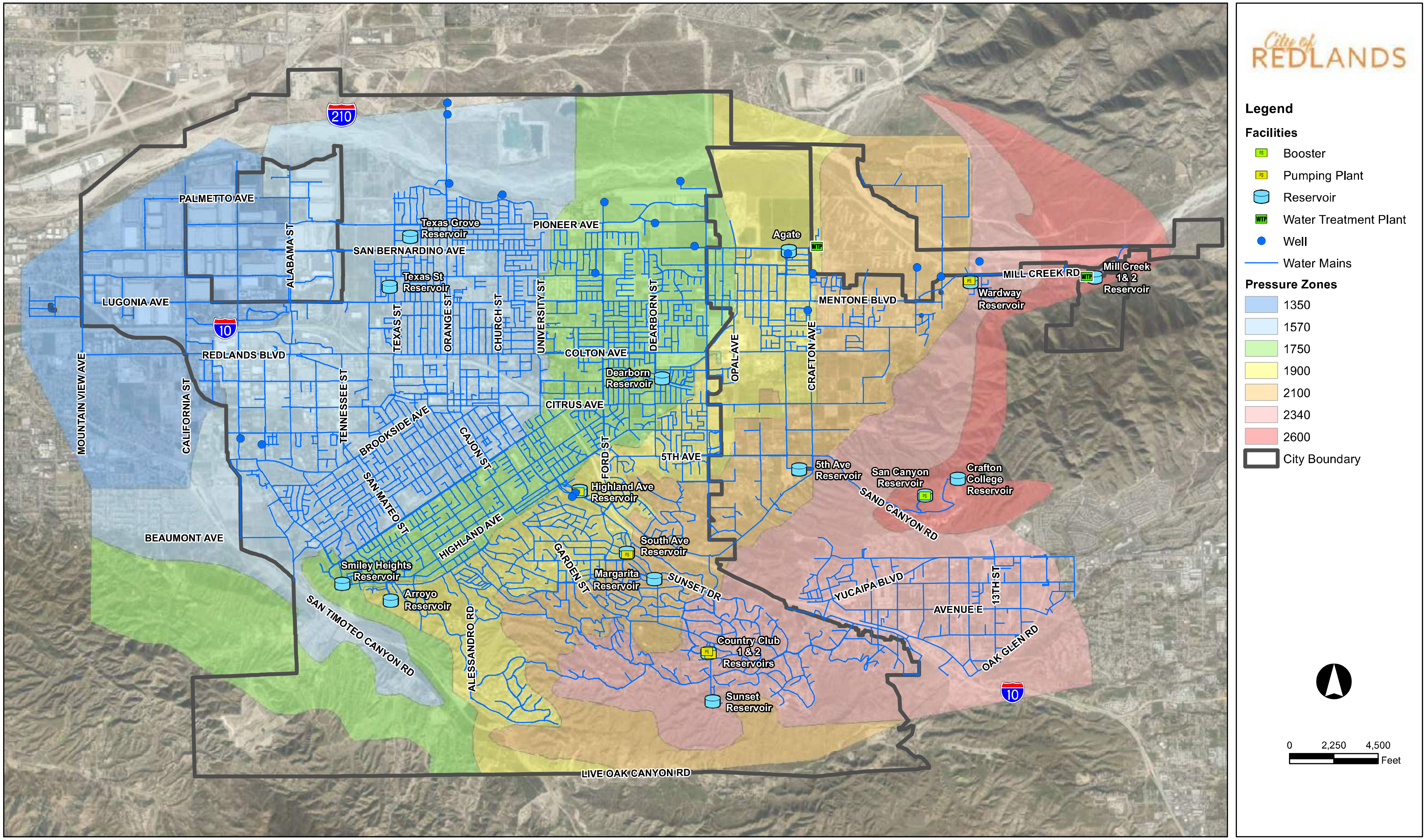

Figure 2-1
City of Redlands City Limits

2.3 STORAGE RESERVOIRS

POTABLE WATER SYSTEM

The current storage system includes eighteen (18) potable reservoirs. Treated surface water from Hinckley and Tate is supplemented by treated groundwater to supply the potable water reservoirs through a series of booster pumps. These reservoirs can store a cumulative maximum of 54.3 million gallons (MG). Table 2-1 provides the storage capacity, pressure zone served, material type, construction year, minimum water surface elevation, and maximum water surface elevation for each potable water reservoir.

Table 2-1: Existing Potable Water System Storage Reservoirs

No.	Designation	Capacity (MG) (1)	Primary Zone Served (2)	Type of Construction (2)	Year Installed (2)	Min. Water Elev. (FT) (2)	Max. Water Elev. (FT) (2)
1	Texas Grove	3.9	1350	Steel	2004	1331	1350
2	Texas Street	1	1350	Steel	1956	1315.25	1350
	Zone 1350 Total	*4.9*					
3	Dearborn	10.6	1570	Concrete	1972	1552.5	1578
4	Highland	10	1570	Concrete	1976	1556.9	1584.83
5	Smiley	3	1570	Steel	1964	1538	1570
	Zone 1570 Total	*23.6*					
6	Agate	3	1750	Steel	1968	1725	1746.83
7	Arroyo	0.5	1750	Steel	1965	1710	1750
8	South	2	1750	Steel	1964	1724	1750
	Zone 1750 Total	*5.5*					
9	Fifth Avenue	5	1900	Concrete	1974	1882.5	1905
10	Margarita	2.4	1900	Concrete	1964	1878.62	1895
	Zone 1900 Total	*7.4*					
11	Country Club 1	1	2100	Steel Inside Concrete	2010	2096.67	2111
12	Country Club 2	2	2100	Concrete	1969	2101.37	2120
13	Ward Way	2	2100	Steel	1958	2068.5	2100
	Zone 2100 Total	*5*					
14	Sand Canyon	3.5	2340	Steel	1973	2314	2353.62
15	Sunset	3	2340	Steel	1967	2277	2340
	Zone 2340 Total	*6.5*					
16	Mill Creek 1	0.2	2600	Steel	1962	2375	2390
17	Mill Creek 2	0.2	2600	Steel	1987	2375	2390
18	Crafton	1	2600	Steel	1970	2560.25	2590
	Zone 2600 Total	*1.4*					
	Total Storage Capacity	**54.3**					

Note: (1) Reservoir Data Information 2018-2021; (2) Facilities Data Information 2018-2021

NON-POTABLE WATER SYSTEM

The current storage system includes two (2) small poly tanks and three (3) cement lined open air reservoir throughout the distribution system.

2.3.1 SYSTEM 1

This non-potable system provides no storage.

2.3.2 SYSTEM 2

This non-potable system utilizes two (2) 12,150-gallon poly storage tanks located at the Texas St. Reservoir site used to supply the on-site booster pump station.

2.3.3 SYSTEM 3

This non-potable system provides no storage.

2.3.4 SYSTEM 4

This non-potable system utilizes two (2) cement lined open air reservoir used for recreation within Ford Park.

2.3.5 SYSTEM 5

This non-potable system provides no storage.

2.3.6 SYSTEM 6

This non-potable system provides no storage.

2.3.7 SYSTEM 7

This non-potable system utilizes one (1) cement lined, open air reservoir used as a water feature within the Redlands Country Club golf course.

2.3.8 SYSTEM 8

This non-potable system provides no storage.

RECYCLED WATER SYSTEM

Currently, no recycled water storage reservoirs exist, which prevents the City from expanding the recycled water distribution system beyond the existing customer base. However, two (2) 1.5 MG recycled water storage reservoirs are currently being engineered for construction in the future at the City WWTP.

2.4 WELLS

POTABLE WATER SYSTEM WELLS

The City operates seventeen (17) active groundwater wells that supply treated water to the potable water distribution system. Five (5) of these wells operate seasonally during peak demand periods. Well No. 10 and Well No. 13 have not been used for several years due to elevated levels of Nitrate, Perchlorate and 1,2 Dibromo-3chloropropane. Both wells are scheduled for rehabilitation within the next three (3) years.

Table 2-2 provides the discharge zone, capacity, ground elevation, and water surface elevation for each potable water system well.

Table 2-2: Potable Water System Wells

No.	Well Name (1)	Discharge to Zone (1)	Capacity (GPM) (1)	Ground Elev. (FT) (1)	Water Surface Elev. (FT) (1)	Efficiency (%) (1)	Testing Date (1)
1	North Orange Street 1	1350	2900	3050	293	72%	4/24/2018
2	North Orange Street 2	1350	2900	3105	288	68%	4/24/2018
3	10	1570	1400	1650	80	N/A	N/A
4	13	1570	3000	1630	62	N/A	N/A
5	38	1570	1600	1634	520	75%	6/12/2018
6	39	1570	1250	1255	535	72%	9/19/2018
7	Church Street	1570	2000	2136	485	N/A	N/A
8	Orange Street	1570	1500	1165	459	68%	4/24/2018
9	Airport 1	1750	1500	1316	498	73%	4/30/2018
10	Airport 2	1750	1000	119	545	78%	4/9/2018
11	Mentone Acres 2	1750	1600	1787	491	72%	6/12/2018
12	Rees	1750	550	1964	544	76%	4/30/2018
13	Muni	1750	2200	1570	335	N/A	N/A
14	Madeira	1900	600	697	399	57	4/30/2018
15	Lugonia 3	2100	250	500	35	58%	9/19/2018
16	Lugonia 6	2100	250	1970	59	58%	9/19/2018
17	Maguet 2	2100	400	249	253	50%	10/2/2018
Note: (1) Facilities Data Information from SCE pump efficiency tests dated 2018 - 2020							

NON-POTABLE WATER SYSTEM WELLS

The City operates twelve (12) wells that supply untreated groundwater to the non-potable water distribution systems, although the Agate 1 Well and Well No. 41 have not been used for several years. Additionally, untreated groundwater from the Crafton Well can be used to supplement the non-potable water system. Table 2-3 provides the discharge pressure system, flow rate, head, pumping efficiency, and most recent pump efficiency testing date for each non-potable well.

Table 2-3: Non-Potable Water System Wells

No.	Name	Discharge System	Flow (GPM)	Head (ft)	Efficiency (%)	Testing Date
1	California	1	1,344	462.8	52%	3/4/2016

No.	Name	Discharge System	Flow (GPM)	Head (ft)	Efficiency (%)	Testing Date
2	30A	2	1,296	295.4	57%	5/18/2018
3	31A	2	878	258.6	32%	4/20/2016
4	32	2	1,555	285.7	55%	5/18/2018
5	New York Street	3	575	230	57%	3/4/2016
6	41	3	1,575	N/A	N/A	N/A
7	11	4	345	176.7	54%	6/25/2018
8	16	5	680	84.7	39%	6/11/2018
9	Agate 1	6	1,075	160.1	78%	5/18/2018
10	Crafton	6	1,841	254.6	60%	10/30/2018
11	Redlands Heights	7	468	394.6	57%	6/18/2018
12	36	8	648	N/A	N/A	N/A
Note: Information from 2016-2018 Testing Data						

2.5 BOOSTER PUMPS

POTABLE WATER SYSTEM

The potable water distribution system includes twelve (12) pump stations with thirty-eight (38) individual booster pumps that transfer water between pressure zones. Table 2-4 provides the pump name, pump station, approximate elevation, pump design head, and pump design flow, and source (suction) and destination (discharge) pressure zones for each booster pump.

Table 2-4: Booster Pump Data

No.	Pump Name (1)	Pump Station (1)	Pump Elevation (FT) (1)	Design Head (FT) (1)	Design Flow (GPM) (1)	Zone (1)		Efficiency (%) (1)	Testing Date (1)
						Source	Destination		
1	1550	Texas	1320	325	2000	1350	1570	28%	5/8/2018
2	1551	Texas	1320	325	2000	1350	1570	23%	5/8/2018
3	1552	Texas	1320	280	1800	1350	1570	28%	5/8/2018
4	1553	Texas	1320	320	2000	1350	1570	75%	12/4/2018
5	1761	Dearborn	1570	150	1200	1570	1750	108%	4/25/2018
6	1931	Dearborn	1570	447	860	1570	1900	77%	4/25/2018
7	2174	HAWC	1572	810	700	1570	2100	74%	4/13/2018
8	2176	HAWC	1572	575	740	1570	2100	66%	4/13/2018
9	2177	HAWC	1572	550	700	1570	2100	58%	4/13/2018

No.	Pump Name (1)	Pump Station (1)	Pump Elevation (FT) (1)	Design Head (FT) (1)	Design Flow (GPM) (1)	Zone (1)		Efficiency (%) (1)	Testing Date (1)
						Source	Destination		
10	1720	HAWC	1572	230	170	1570	1750	45%	4/13/2018
11	1721	HAWC	1572	150	1250	1570	1750	73%	4/13/2018
12	1722	HAWC	1572	230	2300	1570	1750	96%	4/13/2018
13	1783	Smiley Heights	1548	230	300	1570	1750	51%	9/4/2018
14	1784	Smiley Heights	1552	245	420	1570	1750	53%	9/4/2018
15	1927	South	1748	185	2050	1750	1900	51%	6/19/2018
16	1928	South	1748	196	1750	1750	1900	51%	6/19/2018
17	2124	South	1748	550	980	1750	2100	48%	6/19/2018
18	2125	South	1749	410	1050	1750	2100	N/A	N/A
19	2126	South	1749	425	766	1750	2100	N/A	N/A
20	1951	Agate	1732	180	1010	1750	1900	68%	4/9/2018
21	1952	Agate	1732	193	960	1750	1900	70%	5/9/2018
22	1953	Agate	1732	191	1623	1750	1900	71%	4/9/2018
23	1724	Ford Park	1555	N/A	N/A	1750	1750	N/A	N/A
24	1723	Ford Park	1555	N/A	N/A	1750	1750	53%	12/4/2018
25	2131	Fifth Avenue	1905	250	890	1900	2100	60%	8/31/2018
26	2132	Fifth Avenue	1905	246	850	1900	2100	60%	9/4/2018
27	2310	Fifth Avenue	1909	257	463	2100	2340	52%	8/31/2018
28	2311	Fifth Avenue	1909	260	1580	2100	2340	68%	8/31/2018
29	2384	Country Club	2100	231	1038	2100	2340	N/A	N/A
30	2385	Country Club	2100	340	800	2100	2340	65%	10/2/2018
31	2386	Country Club	2105	290	357	2100	2340	68%	10/2/2018
32	2387	Country Club	2105	241	1232	2100	2340	67%	10/2/2018
33	2381	Ward Way	2076	394	150	2100	2340	58%	9/11/2018
34	2382	Ward Way	2076	420	160	2100	2340	66%	9/11/2018
35	2610	Sand Canyon	2332	284	513	2340	2600	N/A	N/A
36	2611	Sand Canyon	2332	260	1093	2340	2600	N/A	N/A
37	2510	Mill Creek	2375	N/A	N/A	2600	2600	N/A	N/A

No.	Pump Name (1)	Pump Station (1)	Pump Elevation (FT) (1)	Design Head (FT) (1)	Design Flow (GPM) (1)	Zone (1)		Efficiency (%) (1)	Testing Date (1)
						Source	Destination		
38	2511	Mill Creek	2375	N/A	N/A	2600	2600	N/A	N/A
Note: (1) Booster Pump Data: Information from SCE pump efficiency tests dated 2018 to 2020									

NON-POTABLE WATER SYSTEM

Non-potable system 2 utilizes two (2) small package skid booster pump systems to maintain system pressure within the zone.

RECYCLED WATER SYSTEM

The recycled water system is treated water through the City's WWTP Membrane Bioreactor (MBR) System and is permitted for six (6) million gallons per day (MGD) with future plans in the WWTP Phase 2 improvement project to take the total to 9.1 MGD. This system has limits imposed on phosphate levels for the SCE Mountain View Power Plant and has to maintain a minimum of 0.5 parts per million (ppm) chlorine residual. There are three (3) booster pumps that pump from the chlorine contact basin at the WWTP. The pumps are seventy-five (75) horsepower (hp) with a flow of 1,500 gallon per minute (GPM) each at 154 feet head that operate on a variable frequency drive based on demand.

2.6 WATER TREATMENT PLANTS

Hinckley and Tate WTP are both conventional treatment plants. Tate treats water from Mill Creek, while Hinckley treats water primarily from the Santa Ana River. Both can blend treated effluent with water from potable groundwater wells when necessary to meet drinking water standards and system demands. Tate is permitted for maximum daily potable water production of twenty (20) MGD, while Hinckley is permitted for maximum daily potable water production of 14.5 MGD. A condition assessment was prepared by evaluating O&M practices and identifying infrastructure needs for both plants. A summary information is provided below, and a detailed assessment report with the associated photos is included in Appendix A.

2.6.1 HORACE P. HINCKLEY WATER TREATMENT PLANT

Hinckley is a conventional WTP utilizing continuous rapid mix flocculation, sedimentation, filtration, and disinfection. The nominal plant treatment capacity is twelve (12) MGD, which is sustainable even with one (1) filter out of service. The peak flow rate through the plant is limited to 14.5 MGD, as permitted by California State Water Resource Control Board Division of Drinking Water (SWRCB-DDW). The treatment plant capacity is expandable to a maximum ultimate capacity of thirty-six (36) MGD. Hinckley primarily treats raw water from the Santa Ana River (SAR) and is capable of receiving and treating water from the SWP. Up to 100 percent (100%) of the plant's raw water can be supplied from the SAR, when available, during the winter. During hot-weather periods in the summer, twenty to forty percent (20%-40%) of the Hinckley's raw water can be supplemented with water from the SWP.

2.6.2 HENRY TATE WATER TREATMENT PLANT

The Tate WTP is a conventional WTP utilizing continuous rapid mix flocculation, sedimentation, filtration, and disinfection. The nominal plant treatment capacity is twenty (20) MGD. The peak flow rate through the plant is limited to twenty (20) MGD, as permitted by SWRCB-DDW. Tate was initially commissioned in 1967 to treat surface water from Mill Creek, and has received several process upgrades, the latest of which installed upgrades to the chemical feed applications, clarifier, filter, backwash and sludge processes (2005). Historically, Tate received raw water exclusively from Mill Creek. However, the Mill Creek source is not typically reliable during drought periods and high turbidity events. Therefore, to increase the raw water supply reliability, the City obtained a permit amendment from the SWRCB-DDW to treat SWP and SAR source-water at Tate.

2.6.3 WASTEWATER TREATMENT PLANT

Approximately 4,480 AF of recycled water is produced annually at the City WWTP, however approximately 1,835 AF of recycled water is delivered to customers annually. This facility was constructed in 1960 to produce 9.5 MGD of secondary wastewater treatment, and was upgraded with Membrane Bioreactor (MBR) technology in early 2000 to produce six (6) MGD of high-quality recycled water. WWTP influent is screened, clarified, and settled before being divided between the MBR and the conventional treatment systems. The treated effluent typically contains less than five (5) milligrams per liter (mg/l) of biological oxygen demand (BOD), less than five (5) mg/l of total suspended solids (TSS), less than ten (10) mg/l of total nitrogen, and less than 0.2 nephelometric turbidity unit (NTU) for turbidity. Table 2-5 provides the typical tertiary treated wastewater result.

Table 2-5: City of Redlands Typical Tertiary Treated Title 22 Recycled Water

Parameter	Influent	Effluent
BOD (mg/L)	160	<5
TSS (mg/L)	130	<5
Total Nitrogen (mg/L)	24	<10
Turbidity (NTU)	NA	<0.2

Note: Data from Water Recycling/Power Generation Reuse Project

2.7 FACILITY SEISMIC ANALYSIS

Due to the potential for seismic activity in southern California, it is essential for water facilities to be designed, equipped, and prepared for potential seismic movement. Richard Brady and Associates, Inc. (Brady) completed a Condition, Seismic, and Structural Assessment of all essential water facilities to identify potential facility risks, located in Appendix F.

City of Redlands Water System Hydraulic Schematic

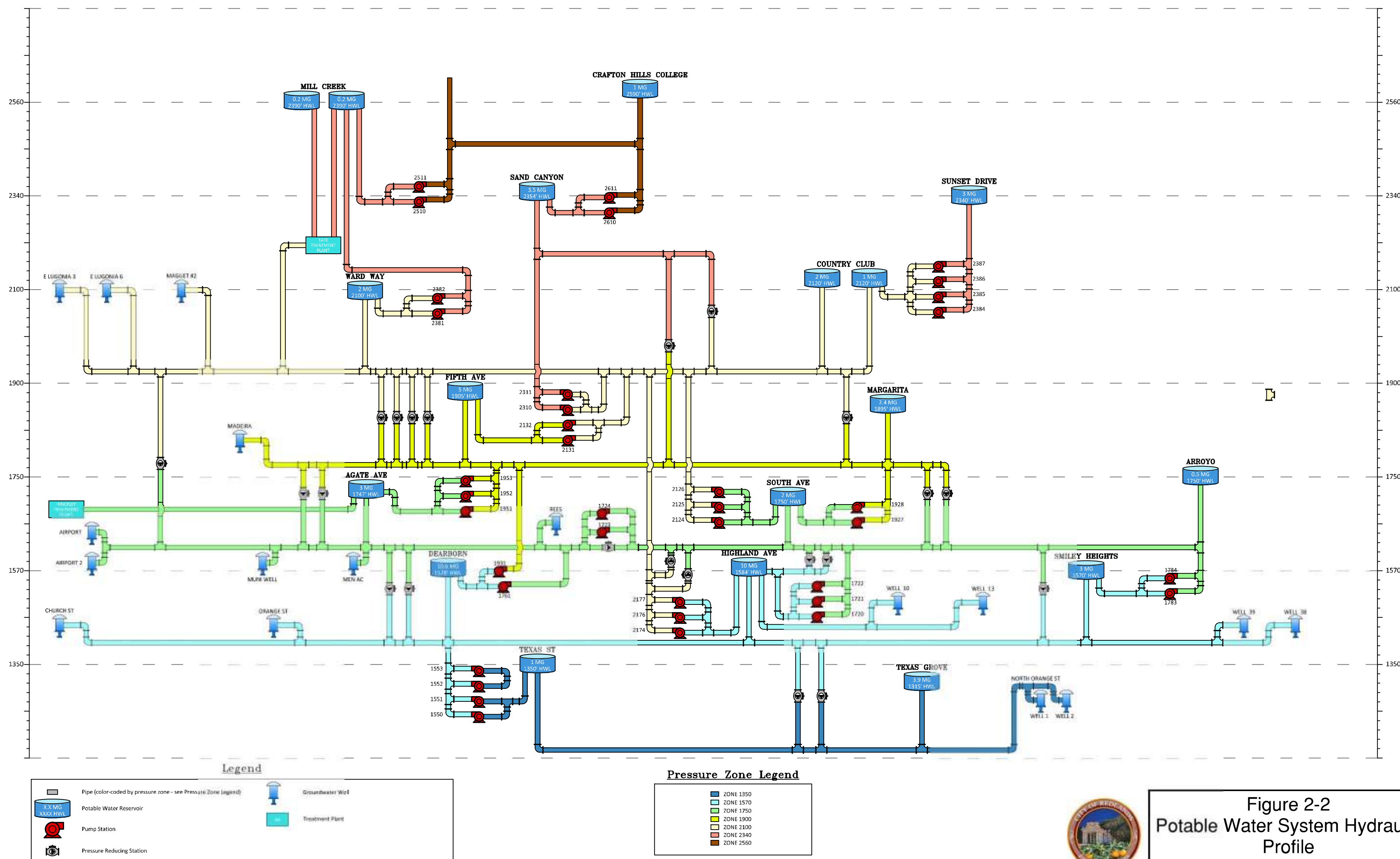

Figure 2-2
Potable Water System Hydraulic Profile

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3 WATER USE

3.1 GENERAL DESCRIPTION

This section reviews the water use characteristics for the City's water systems. The information presented in this section was used in the hydraulic models to evaluate system reliability and the ability of each system to provide sustainable water delivery service to customers. This information was also used to project future water demands and requirements to predict future operational needs.

3.2 WATER DEMAND

POTABLE WATER SYSTEM

From 2016 to 2020, the City produced an average of 22,821 AF of potable water and delivered an average of 20,770 AF of potable water each year (2020 UWMP). Table 3-1 provides annual and summary demand data within several use categories. In the table below, if the meter service does not fit in any of the general categories, it is defined as "Other". Average annual system water loss, calculated as the percentage of water delivered to water produced, was relatively high (nearly 9%) during this period. This is likely caused by system leaks and meter inaccuracies. Approximately seventy-three percent (73%) of the total demand was from residential customers, and the average per capita water usage was approximately 222.9 gallons per capita per day (gpcd) (18.5 MGD/83,000 capita). This demand data was used for water accounting and calibration of the hydraulic model, which was used to project future potable water demands through the planning horizon. The comparative use of each category is shown graphically in Chart 3-1.

Table 3-1: Historical Water Production and Demand (AF/Year)

Category	2016	2017	2018	2019	2020	Average
Single Family Residential	11,340	12,275	12,866	11,624	12,949	12,211
Multi-Family Residential	2,835	2,913	2,934	2,750	2,901	2,867
Commercial/Industrial	3,180	3,142	3,159	2,705	2,640	2,965
Landscape Irrigation	1,924	2,155	2,340	2,228	2,220	2,173
Agriculture	556	387	326	283	276	366
Other	183	253	179	174	151	188
Total Demand	20,018	21,125	21,804	19,764	21,137	20,770
Total Production	20,919	23,303	23,442	21,975	24,464	22,821
Water Loss (AF)	837	2178	1638	2211	3227	2051
Water Loss (%)	4.0%	9.3%	7.0%	10.1%	13.6%	9.0%

Note: Based on Table 4-3 in 2020 Urban Water Management Plan

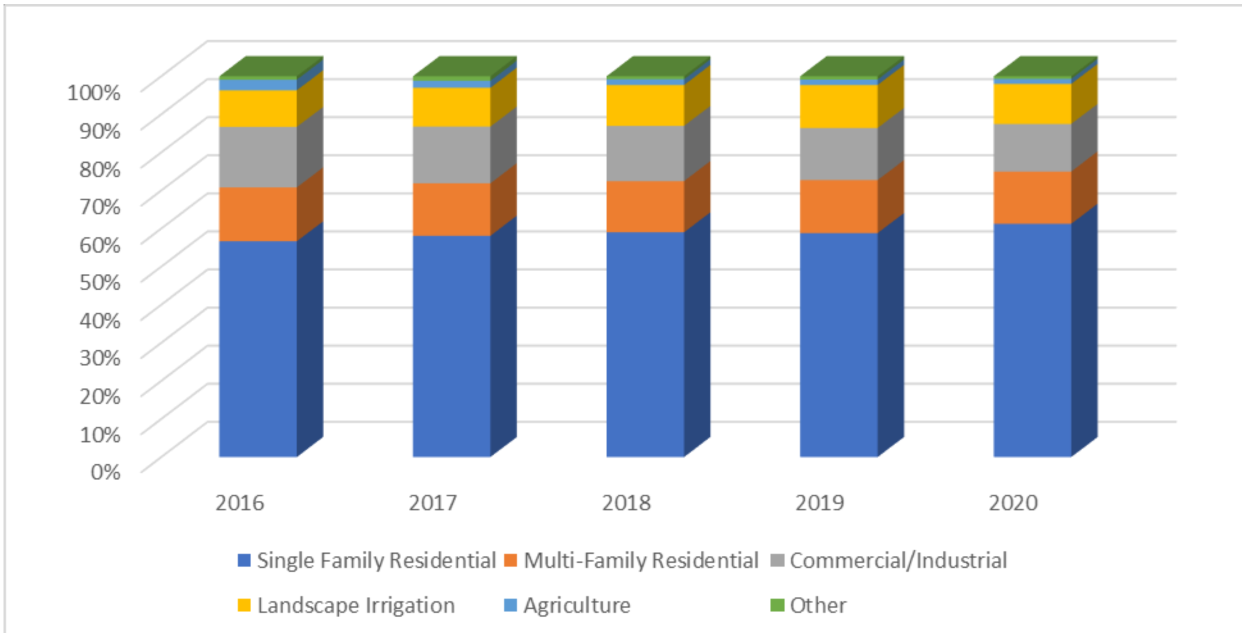

Chart 3-1: Historical Potable Water Use by Category

NON-POTABLE WATER SYSTEM

The City of Redlands produces and distributes approximately 1,868 AF of non-potable water annually. Table 3-2 provides non-potable water demands for the previous four (4) years. The non-potable water system primarily serves warehouses and commercial facilities in the northwest portion of the City. Table 3-3 provides non-potable water demand for several larger customers. Water demand for Ford Park and Hillside Memorial Park are provided as municipal use references.

Table 3-2: Historical Non-Potable Water Demand (AF/Year)

	2017	2018	2019	2020	Average
Commercial/Industrial	1,456.5	2,512.9	1,051.6	2,212.2	1,808.3
Landscape Irrigation	81.2	179.4	89.7	185.1	133.9
Agriculture	193.2	16.1	74.5	3.7	71.9
Non-Potable Demand	1,730.9	2,708.4	1,215.8	2,400.8	2,014.0

Table 3-3: 2021 Non-Potable Water Users

Users	GPD
Warehouses	396,731
Commercial Plazas	96,850
Crafton College	67,397
Park Irrigation	44,324

Users	GPD
Redlands Country Club	35,893
Agricultural	31,479
Kaiser Permanente Medical Center	15,258
Ford Park	2,167
Hillside Memorial Park	40
Note: Data Analyzed with Meter Information Provided by the City	

RECYCLED WATER SYSTEM

Recycled water is treated effluent from the City WWTP and is primarily used for equipment cooling at the SCE Mountain View Power Plant, dust control at the City landfill, and for landscape irrigation customers. The power plant generates approximately 1,056 megawatts of power, and uses a 50:50 blend of recycled and non-potable water produced by their own non-potable well. This blending is necessary because of a "Take-or-Pay" agreement that requires the power plant to use or pay for 3,000 AF of water. Currently, the City WWTP is only capable of producing approximately 4,480 AF of recycled water. Actual power plant demand varies with energy production, and averaged approximately one (1) MGD annually between 2017 and 2019. A small amount of recycled water is also used for dust control at the landfill site adjacent to the WWTP. It is anticipated that the WWTP will be able to produce approximately 6,720 AF recycled water when plant improvements are constructed in the near future and be permitted to treat up to 10,193 AF. Annual recycled water demand from 2017 through 2020 was 2,427.5 AF, 1,976 AF, 1,905.2 AF, and 1,806 AF, with an annual average of 2,028.7 AF.

3.3 FIRE FLOW

The Fire Marshall establishes minimum fire protection water requirements, including storage, pressure, and flow, within the City. These requirements were used as minimum standards when analyzing the potable water distribution system. Table 3-4 shows the minimum fire flow requirements for water delivery pressure and duration based on a Type V building construction type (wood frame). These requirements vary based on the total size of the facility and the building construction type, as defined by the California Fire Code. The storage for the highest single fire event is expected to be two (2) MG. This storage volume assumption was used to develop design criteria for the City's storage requirements.

Table 3-4: Fire Flow Criteria

Land Use	Criteria	Minimum Fire Flow (GPM)	Minimum Pressure (PSI)	Duration (HR)
Residential	Less than 3600 SqFt	1,000	20	2
Residential	Greater than 3600 SqFt	1,500	20	2
Commercial/ Industrial	Less than 11,300	1,500 – 2,750	20	2
Commercial/ Industrial	11,301 SqFt - 20,601 SqFt	3,000 – 3,750	20	4
Commercial/ Industrial	20,601 SqFt – 85,100 SqFt	4,000 – 7,750	20	4

Land Use	Criteria	Minimum Fire Flow (GPM)	Minimum Pressure (PSI)	Duration (HR)
Commercial/ Industrial	Greater than 85,101 SqFt	8,000	20	4
Note: Data provided in 2021 by the City of Redlands Fire Marshal for Type V Construction				

3.4 CONSERVATION MEASURES

The 2021 Water Shortage Contingency Plan (2021 WSCP) was developed to ensure long term sustainability of water resources.

3.4.1 WATER SHORTAGE CONTINGENCY PLAN

The City encourages and incentivizes year-round water conservation practices. Voluntary and mandatory water conservation measures for various water shortage conditions, which may be caused by climate change, infrastructure failure, source contamination, or other supply issues, are identified in 2021 WSCP, and comply with minimum California Water Code requirements. The 2021 WSCP is codified by ordinance, and includes six (6) stages, each corresponding to a specific supply shortage condition. These stages are:

Stage I: Voluntary Conservation Measures

Issued when a slight decrease in the water supply is expected

Stage II: Mandatory Compliance; Water Alert

Issued when a moderate decrease in the water supply is expected

Stage III: Mandatory Compliance; Water Warning

Issued when a significant decrease in the water supply is expected

Stage IV: Mandatory Compliance; Water Emergency

Issued when a forty percent (40%) decrease in the water supply is expected

Stage V: Mandatory Compliance; Water Emergency

Issued when a fifty percent (50%) decrease in the water supply is expected

Stage VI: Mandatory Compliance; Water Emergency

Issued when water supplies are in danger of being depleted to a point where uses such as human consumption, sanitation, and fire protection would be endangered. This would be in response to a more than fifty percent (50%) decrease in supply, most likely associated with a natural disaster.

Implementation of each stage is declared by the City Council when water demands cannot be satisfied without depleting the water supply for consumption, sanitation, and fire protection. If the City Council is unable to meet, the City Manager or his/her designee can implement the plan on an emergency basis. This implementation will be reviewed and ratified or revoked by the City Council at its next scheduled meeting. In case of a catastrophic interruption, such as an earthquake, fire, and other emergency, the City Municipal Utilities and Engineering Department (MUED) will implement an existing emergency plan, which requires designated personnel to meet at a predefined reporting time and location for task assignments. Those who cannot reach their designated area are to offer their services to other local water providers if they are also experiencing an emergency.

3.4.2 DEMAND MANAGEMENT MEASURES

The City has achieved its 2020 water use reduction targets, and will continue efforts to reduce water waste. To minimize water waste, the City has implemented water waste prevention programs, metering programs, conservative pricing, assessed and managed distribution losses, and has increased both public education and outreach. In addition to the 2021 WSCP water use restrictions for various water supply shortage conditions, the City manages several other water conservation programs.

The City's water systems are metered to ensure accurate demand measurement. Meters are maintained routinely, and a replacement schedule was developed in 2008. Meters smaller than two inches (2") are replaced every fifteen (15) to twenty (20) years, while larger meters are periodically calibrated to ensure accuracy. In 2021, the first year of a five (5) year project to replace all water meters was initiated.

The City prices water using a tiered rate structure. This tiered system includes two (2) pricing components. The first component is a fixed service charge based on the meter size, and the second is a variable commodity charge based on the amount of water delivered. The commodity charge unit rate increases at specific use thresholds.

The City also manages comprehensive public education and outreach programs, focusing on customer accountability while incentivizing water conservation practices. Examples include:

1. Water Efficiency Rebate Program Financial Incentives:
    a. Weather-Based Irrigation Controllers
    b. Drought Tolerant Lawn Conversions
    c. Synthetic Turf Replacement
    d. Water Efficient Clothes Washers
    e. High-Efficiency Sprinkler Nozzles
    f. Low Flow Toilets
2. Design and Construction of four (4) demonstration gardens;
3. Participation in regional marketing campaign;
4. Educational outreach events;
5. Offering free water-saving products including hose nozzles, toilet leak detection tablets, lawn/plant moisture meters, low water use plants, shower timers, faucet aerators, and water efficiency educational materials.

3.5 FUTURE WATER USE

POTABLE WATER SYSTEM

The 2020 UWMP projected future potable water demand, based on expected population growth, land use development, and new connections, to be 21.53 MGD in 2040 and 22.17 MGD in 2045. These demand increases are expected to be primarily from new development within the unincorporated portions of the City water service area. Table 3-5 provides potable water demand projections through 2045 for each land use category. Potable water demand is projected to increase by three to four percent (3%-4%) annually through 2045.

Table 3-5: Potable Water Demand Projection (AF/Year)

Land Use	2025	2030	2035	2040	2045
Single Family Residential	12,943	13,470	13,997	14,461	14,925
Multi-Family Residential	3,036	3,160	3,284	3,393	3,501
Commercial/Industrial	3,081	3,145	3,209	3,265	3,321
Landscape Irrigation	2,292	2,385	2,478	2,560	2,643
Agriculture	206	206	206	206	206
Other	206	214	223	230	238
Total Demand	21,764	22,580	23,397	24,115	24,834
Total Demand (MGD)	19.42	20.14	20.87	21.51	22.16
Demand Increase (%), 2020 base	3.0%	3.7%	3.6%	3.1%	3.0%
Note: Data in this table is from the 2020 Urban Water Management Plan table 4.5. Data for the water demand was linearly scaled for 2022 - 2042.					

NON-POTABLE WATER/RECYCLED WATER SYSTEMS

The most likely opportunity to expand the non-potable water and recycled water systems is to consolidate them into a single system capable of putting approximately four (4) MGD of excess recycled water to beneficial use. This would require the construction of additional storage reservoirs and booster pumps to convey water efficiently throughout the system. Approximately forty-five (45) water meters within pressure system 1 that include commercial areas, parks, agricultural areas, schools, and residential areas with heavy irrigation use could be connected to this expanded system. Another 200-300 water meters within pressure system 2 could also be connected to this expanded system.

Table 3-6 provides use data for various water meter billing classifications, and shows the potential for relieving pressure from the potable water system by transitioning customers to the expanded non-potable/recycled water system where possible. The residential classification includes rural agricultural areas within Crafton. Transitioning these areas to the expanded system would be difficult due to the distance from the expanded system. It is recommended that the City focuses on commercial and industrial customers clustered in pressure systems 1 and 2 to maximize the benefit of the expanded system. Selective parks and City-owned citrus groves with significant water use should also be prioritized for connection to the expanded system.

Table 3-6: Potential Non-Potable Water Users

Land Use	Water Use (GPD)
Residential	638,636
Commercial	416,113

Land Use	Water Use (GPD)
Parks	281,336
Agricultural	172,025
Public Institutions	145,143
Other	55,604
Note: Data Analyzed with Meter Information Provided by the City	

Table 3-7 provides typical water use within each system. System 1 and 2 are the most developed and are located close to the wastewater treatment facility, and transitioning customers within pressure system 2 offers the greatest opportunity to relieve pressure from the potable water system. Most of these customers are located north of Fern Avenue and south of I-10.

Table 3-7: Potential Non-Potable Water Users

System	Water Use (GPD)	Number of Meters
1	79,865	114
2	762,176	211
3	346,777	77
5 through 8	520,039	91
Total	1,708,857	413
Note: Data Analyzed with Meter Information Provided by the City		

It is likely that water demand within the expanded system will nearly double from 2.2 MGD to 3.9 MGD. The demand can be supplied from the WWTP recycled water system and non-potable water groundwater wells when customers are transitioned from the potable water system, and non-potable groundwater could supplement demand during peak periods. Expanding and improving this system could reduce potable water use by as much as 2,606 AF each year, with annual cost savings of $1M-$2M. Future developments within these pressure zones could be served by this system as well.

3.5.1 AREA-BASED DEMAND FACTORS

Table 3-8 provides the 5-year average water demand and projected potable water demand through 2045 by land use category. Assuming the demand remains constant, the City can anticipate total annual demand of approximately 25,547 AFY (22.7 MGD).

Table 3-8: Future Water Use Projections by Land Use (AF/Year)

Land Use	5-year avg (AFY)	Area in 2017 (Acre)	AFY/Acre	Area in 2045 (Acre)	2045 Water Demand
Single Family Residential	12,211	8,332	1.466	9,428	13,821

Land Use	5-year avg (AFY)	Area in 2017 (Acre)	AFY/Acre	Area in 2045 (Acre)	2045 Water Demand
Multi-Family Residential	2,867	681	4.210	958	4,033
Commercial/Industrial	2,965	2,017	1.470	2,773	4,076
Landscape Irrigation	2,173	4,138	0.525	5,622	2,952
Agriculture	366	2,180	0.168	2,180	366
Other	188	2,004	0.094	2,441	229
Total	20,770	19,352	8	23,402	25,477
Note: Information based on 2020 Urban Water Management Plan & 2017 General Plan					

3.5.2 CONNECTION-BASED DEMAND FACTORS

The City's potable water system currently includes approximately 23,545 connections. Table 3-9 provides potable water demand by connections in each land use category, and projects total system demand in year 2045. Landscape irrigation and agriculture are the most intensive uses, with 4.077 AFY per connection and 21.53 AFY per connection, respectively. The 2045 potable water system demand is projected based on linear growth projections identified in the 2020 UWMP and 2017 General Plan, and is estimated to be 23,996 AFY (21.4 MGD).

Table 3-9: Future Water Projections by Connection

Land Use	5-Year Average (AFY)	2020 Connections	Connection Demand (AFY)	2045 Connections	2045 Demand (AFY)
Single Family Residential	12,211	19,922	0.613	22,922	14,050
Multi-Family Residential	2,867	980	2.926	1,180	3,452
Commercial/Industrial	2,965	1,397	2.122	1,647	3,496
Landscape Irrigation	2,173	533	4.077	573	2,336
Agriculture	366	17	21.53	17	366
Other	188	696	0.270	1,096	296
Total	20,770	23,545	32	27,435	23,996
Note: Information based on 2020 Urban Water Management Plan & 2017 General Plan					

4 WATER SUPPLY

4.1 GENERAL DESCRIPTION

POTABLE WATER SYSTEM

The primary sources of potable water supply to the City of Redlands are groundwater (48.8%) and treated surface water (51.1%). The City may also receive raw water (0.1%) from the SWP, when it is available, and blend it with SAR and Mill Creek surface water if additional raw water is needed to meet demands. The City operates two (2) surface treatment plants that could be expanded to meet future demands. Current annual potable water production from all sources is 22,907 AF (20.45 MGD). Potable water production during the summer months is approximately three (3) times higher than in the winter months.

Potable water demand was calculated using Supervisor Control and Data Acquisition (SCADA) system data to calibrate the water model and provide a basis for future planning. The MDD is 1.7 times the ADD, and the PHD is 2.75 times the ADD. Currently, the average demand of the City's potable water system is 18.5 MGD, with a peak hour demand of almost fifty-two (52) MGD.

NON-POTABLE WATER SYSTEM

The primary non-potable water source is groundwater extracted from the Bunker Hill Basin.

RECYCLED WATER SYSTEM

The City WWTP produces approximately six (6) MGD of treated effluent through a MBR system that supplies the recycled water system.

4.2 WATER PRODUCTION

POTABLE WATER SYSTEM

Table 4-1 provides potable water production by source from 2017 to 2020. The comparative production of each source is shown graphically in Chart 4-1.

Table 4-1: Potable Water Production Sources (AF/Year)

Potable Water Source	2017 (AFY)	2018 (AFY)	2019 (AFY)	2020 (AFY)	Average (AFY)
Groundwater	11,214	12,468	9,900	12,088	11,418
Santa Ana River	4,634	6,367	5,038	5,796	5,459
Mill Creek	7,455	4,607	7,003	6,045	6,278
State Water Project	-	-	35	535	285
Total Production	23,303	23,442	21,976	24,464	23,296
Note: Data from 2017-2020 Annual Production Report					

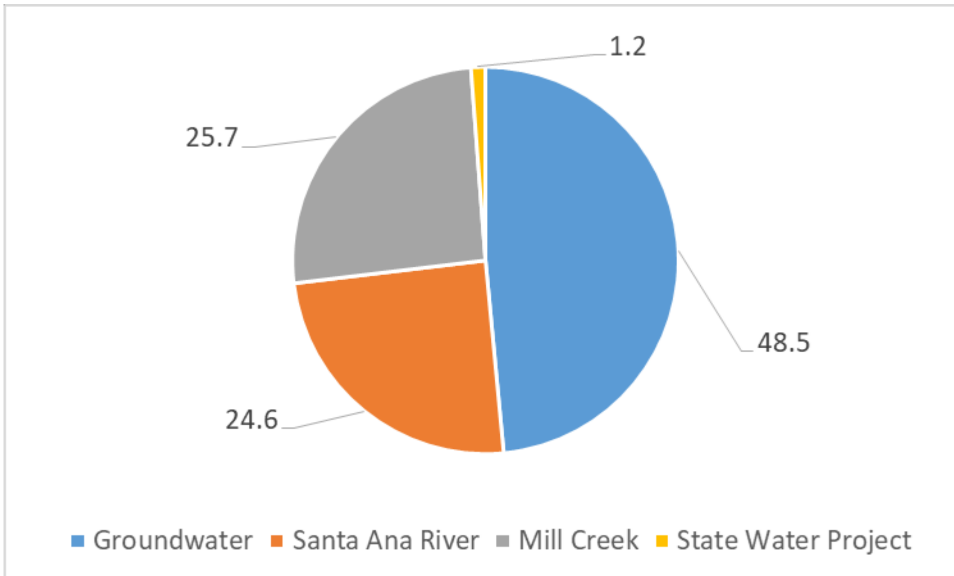

Chart 4-1: Water Production by Source

Table 4-2 provides monthly potable water production data from the 2017-2020 Annual Production Reports. The comparative production of each month and year is shown graphically in Chart 4-2. During that period, production averaged approximately 20.8 MGD from all sources. Monthly production varies seasonally, with summer season production of approximately thirty (30) MGD, and winter season production of approximately eleven (11) MGD. Annual potable water demand averages approximately 18.5 MGD. Reducing potable water system losses by replacing aging and leaking pipelines, replacing and repairing aging water meters, and implementing other improvements is a City priority.

Table 4-2: Monthly Potable Water Production

Month	2017 (MGD)	2018 (MGD)	2019 (MGD)	2020 (MGD)	Average (MGD)
January	8.2	14.0	11.4	13.1	11.7
February	8.6	16.1	8.6	16.8	12.5
March	14.7	11.8	10.4	11.4	12.1
April	21.8	20.2	20.7	13.4	19.0
May	22.9	21.4	17.9	24.6	21.7
June	26.8	25.9	25.1	27.2	26.3
July	29.4	29.7	28.8	29.8	29.4
August	27.4	29.9	29.8	31.2	29.6
September	26.0	27.5	28.4	29.4	27.8
October	24.5	22.1	24.5	26.2	24.3
November	20.0	19.9	19.4	20.1	19.9
December	18.5	12.6	9.8	18.0	14.7
Average	20.7	20.9	19.6	21.8	20.8
Note: Data from the 2017-2020 Annual Production Report					

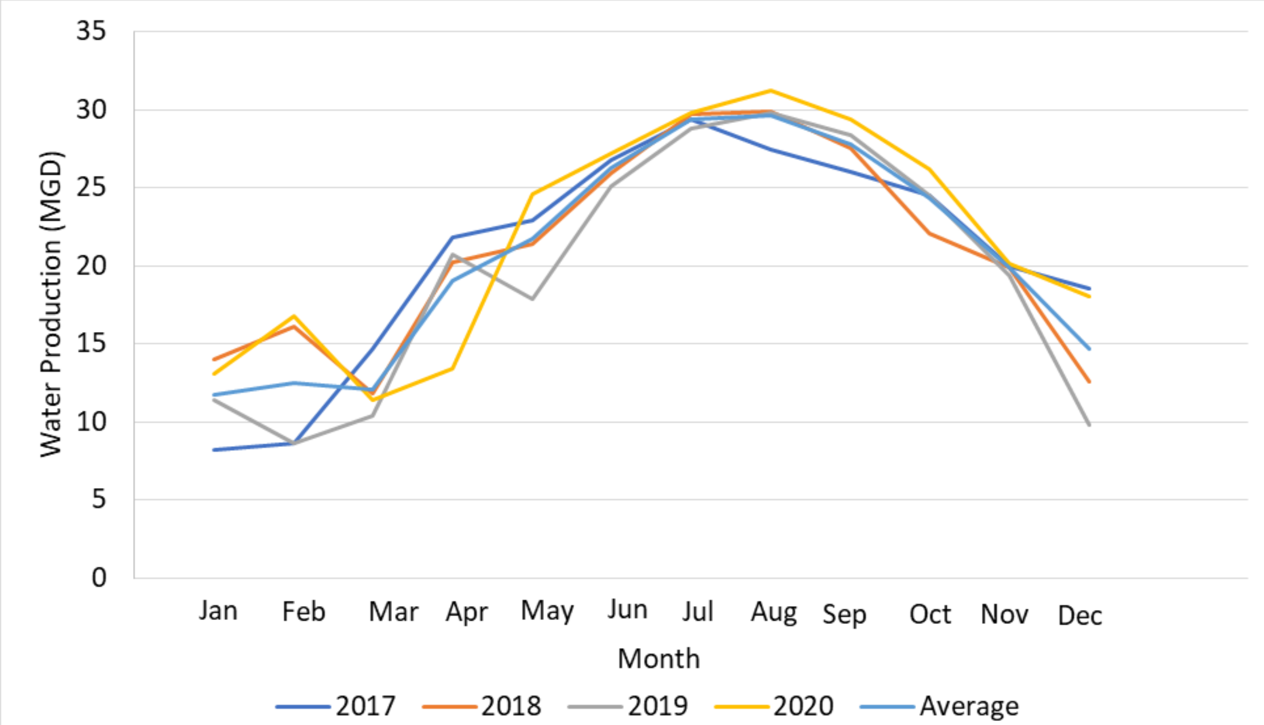

Chart 4-2: Historic Potable Water Production

NON-POTABLE WATER SYSTEM & RECYCLED WATER SYSTEM

Table 4-3 provides monthly non-potable water and recycled water production in 2020. Although the WWTP produces approximately 4.5 – 5.0 MGD of recycled water, only 1.6 MGD is being sent to customers, which could be tripled with system improvements. Similar to potable water production, non-potable water production varies significantly during the summer season (2.57 MGD) and winter season (1.11 MGD). The combined annual production average is approximately three (3) MGD, and varies seasonally from 1.5 MGD in January to 4.5 MGD in July.

Table 4-3: Monthly Non-Potable Water & Recycled Water Production/Distribution (MGD)

Month	2017-2020 Average Well Production (MGD)	Average Recycled Water Production/Distributed to Customers (MGD)	TOTAL Average Production (MGD)
January	0.5	1.0	1.5
February	0.6	1.2	1.8
March	0.6	1.4	2.0
April	1.6	1.3	2.9
May	1.8	1.2	2.0
June	2.6	1.3	3.9
July	2.7	1.8	4.5
August	2.6	1.8	4.4
September	2.4	1.6	4.0
October	2.2	1.7	3.9
November	1.5	1.3	2.8
December	0.8	1.0	1.8

Month	2017-2020 Average Well Production (MGD)	Average Recycled Water Production/Distributed to Customers (MGD)	TOTAL Average Production (MGD)
Average	1.7	1.4	3.0

Table 4-4 provides annual average production from each non-potable groundwater well from 2017-2020. The City's non-potable groundwater wells produce a combined average of 7.3 MGD. The two (2) highest producing wells, Well No. 30A and the New York Street Well serve Systems 2 and 3. These systems account for approximately thirty percent (30%) of total groundwater extractions.

Table 4-4: Non-Potable Groundwater Well Production (MGD)

Groundwater Well	Average Water Production (GPD)	Average Water Production (MGD)
Agate 1	0	0
California	85,301	0.085
Crafton	51,779	0.052
New York	282,329.9	0.282
Redlands Heights	32,792.7	0.033
Well 11	92,912.2	0.093
Well 16	65,593.3	0.066
Well 30 A	572,337.2	0.572
Well 31A	0	0
Well 32	11,025	0.011
Well 36	134,380.1	0.134
Well 41	21,961.5	0.022
Total Non-Potable	1,350,412	1.350
Note: Data from the 2017 – 2020 Annual Production Reports		

This production data was compared to demand data for 2017-2020 to identify significant water losses of approximately forty percent (40%) within the non-potable water system. Table 4-5 summarizes that analysis.

Table 4-5: Non-Potable Water System Losses

	2017	2018	2019	2020	Average
Average Production (MGD)	4.2	3.6	3.2	3.2	3.6

	2017	2018	2019	2020	Average
Average Demand (MGD)	1.6	2.4	1.1	2.1	2.1
Water Loss (MGD)	2.6	1.2	2.1	1.1	1.4
Water Loss (%)	62%	33%	66%	34%	39%

The design on the City's non-potable water system does not alleviate the water loss issue. The non-potable water wells operate on pressure using a ClaValve. Since the non-potable water system does not include storage tanks, the excess water releases into the storm drain system when the pressure exceeds a set point. The need for the system to be operated this way results in loss of water.

It is recommended that the City investigate other potential causes of this water loss, which may include a review of record drawings and specifications, analysis of operations, site visits, and interviews with the maintenance staff before CIP projects are implemented. The City should target system water losses of approximately four to six percent (4%-6%) annually. It is estimated that reducing water losses within the non-potable water system could increase revenues by approximately $750,000 to $ 1.5M annually.

The non-potable water meters are in the process of being replaced, which is expected to be completed by June 30, 2022. Resolving this issue will further reduce water loss and contribute to the targeted system water loss.

4.3 DEMAND VARIATION

4.3.1 AVERAGE DAY DEMAND

POTABLE WATER SYSTEM

Based on 2021 meter-data provided by the City, the potable water system ADD is approximately 18.5 MGD. When accounting for losses in the system, the production needed to supply this demand is approximately 20.4 MGD.

NON-POTABLE WATER SYSTEM

The ADD from non-potable groundwater wells is approximately 2.1 MGD. Approximately ninety percent (90%) of this demand is for outdoor landscape irrigation. The remainder is used for agricultural irrigation and commercial/industrial uses.

RECYCLED WATER SYSTEM

The ADD for the recycled water system, which is exclusively used by the SCE Mountain View Power Plant, for landfill dust control adjacent to the WWTP, and irrigation customers is 1.64 MGD, and can increase periodically to 2.2 MGD. Unused recycled water is blended with non-potable water produced by the California Street Well, to supplement the non-potable water distribution system.

4.3.2 MAXIMUM DAY DEMAND

POTABLE WATER SYSTEM

The potable water system MDD was determined using 2019 SCADA information provided by the City, and was found to be 1.7 times the ADD, with a peak of approximately 2.75 times the ADD. Chart 4-3 shows the diurnal curve for the MDD case. The maximum day typically occurs in mid-August, with peak hour demands occurring at approximately 5:30 a.m. and 11:00 p.m.

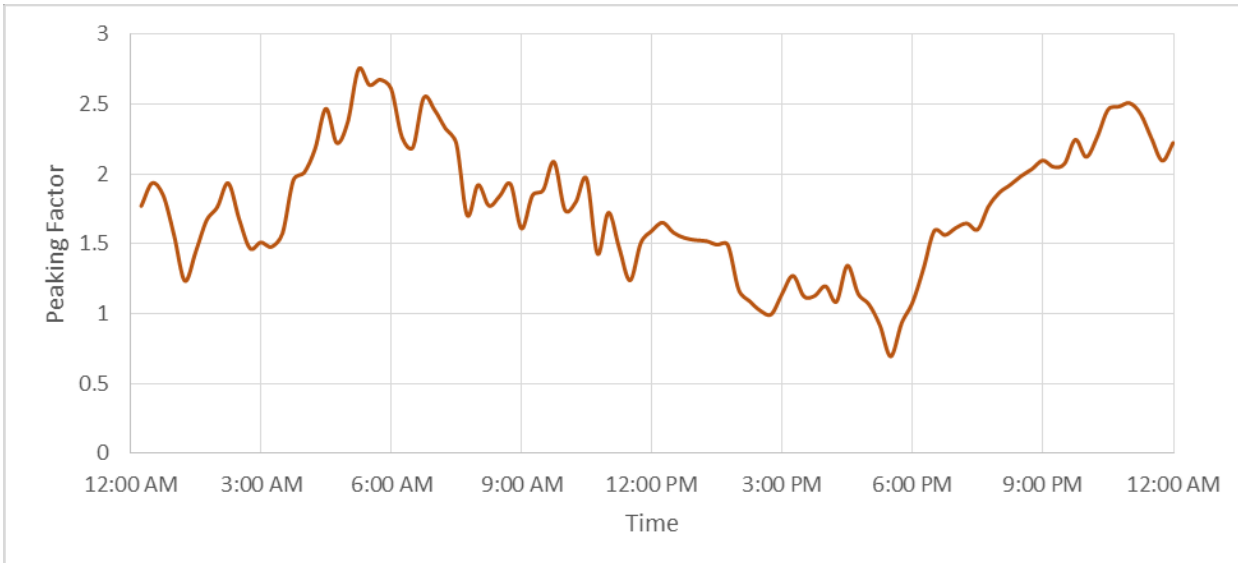

Chart 4-3: MDD Diurnal Curve for Potable Water, August 2019

NON-POTABLE WATER SYSTEM

Based on 2021 meter-data provided by the City, the non-potable water system ADD is approximately 2.1 MGD. When accounting for losses in the system, the production needed to supply this demand is approximately 3.2 MGD. The maximum day typically occurs between June and October, with peak hour demands occurring at approximately 4:30 a.m. to 7:30 a.m. when irrigation is typically applied.

RECYCLED WATER SYSTEM

Based on 2021 meter-data provided by the City, the recycled water system ADD is approximately 1.67 MGD. The maximum day of 2.2 MGD, typically occurs between June and October, with peak hour demands occurring at approximately 4:30 a.m. to 7:30 a.m. when irrigation is typically applied.

4.3.3 PEAK HOUR DEMAND

POTABLE WATER SYSTEM

Understanding the PHD is critical for sizing water mains and other facilities. During PHD, the system experiences high velocities and low service pressures in areas with undersized mains or areas that lack looped distribution pipelines. The PHD was determined using the peak water use during the peak hour of the MDD, and was found to be approximately 2.75 times the ADD, or approximately 50.9 MGD. Hinckley and Tate have a combined maximum treatment capacity of 34.5 MGD. In addition, the City's fourteen (14) active groundwater wells can produce approximately 18,350 GPM (26.4 MGD) of potable water. Therefore, the maximum potable water production capacity from all sources is approximately 58.4 MGD, which exceeds the PHD. However, existing potable water production facilities are not capable of meeting the projected 2045 PHD of sixty-one (61) MGD. It is likely that this will not become an issue until the early 2030s, and the City is currently rehabilitating several potable groundwater wells, including six (6) that are inactive due to water quality issues. Bringing these wells back into service will increase potable water production capacity to 67.2 MGD, which will provide sufficient water to meet the projected 2045 PHD and provide additional capacity as reserve production. The inactive wells include Agate 2, Lugonia 4, Well No. 10, Well No. 13, and the Crafton Well. Table 4-6 provides the maximum potable water production capacity for each facility.

Table 4-6: Active Potable Water Production Facility Capacity

Well	Capacity (GPM)	Capacity (MGD)
Airport 1	1500	2.2
Airport 2	1000	1.4
Church Street	2000	2.9
Lugonia 3	250	0.4
Lugonia 6	250	0.4
Madeira	900	1.3
Maguet 2	325	0.5
Mentone Acres 2	1600	2.3
Muni	1700	2,4
North Orange Street 1	2900	4.2
North Orange Street 2	2900	4.2
Orange Street	1500	2.2
Rees	1200	1.7
38	1500	2.2
39	1250	1.8
10	1400	2.0
13	3300	4.8
Hinckley WTP	10,070	14.5
Tate WTP	13,888	20
Total	49,158	70.8

NON-POTABLE WATER SYSTEM

Understanding the PHD is critical for sizing water mains and other facilities. During PHD, the system experiences high velocities and low service pressures in areas with undersized mains or areas that lack looped distribution pipelines. The PHD was determined using the peak water use during the peak hour of the MDD, and was found to be approximately 5.3 times the ADD, or approximately 11.2 MGD.

RECYCLED WATER SYSTEM

Understanding the PHD is critical for sizing water mains and other facilities. During PHD, the system experiences high velocities and low service pressures in areas with undersized mains or areas that lack looped distribution pipelines. The PHD was determined using the peak water use during the peak hour of the MDD, and was found to be approximately 6.1 times the ADD, or approximately 10.2 MGD.

4.4 EMERGENCY CONNECTIONS

The City maintains emergency water connections with the City of Loma Linda and Western Heights Water Company. In addition, the City is a member of the Emergency Response Network of the Inland Empire (ERNIE) and California's Water/Wastewater Agency Response Network (CalWARN). The intent of these programs and connections is to ensure that the City is able to provide water service to its customers during emergencies.

4.5 PRODUCTION-DEMAND PROJECTION

4.5.1 PROJECTION

POTABLE WATER SYSTEM

Table 4-7 provides projected potable water system demands (ADD, MDD, and PHD) through year 2042. The current peaking factors identified in Section 4.3.3 are assumed to remain constant through the planning horizon. The ADD is expected to increase by 3.2 MG to 21.7 MGD, MDD will increase by approximately 5.5 MGD to thirty-seven (37) MGD, and the PHD will increase by 9.3 MGD to 60.2 MGD. The projected daily production is determined by the daily demand and anticipated system water loss, which was 12.8% in 2020. This is relatively high for a typical water distribution system. Reducing water loss within the distribution system will reduce the difference between water produced and water distributed, resulting in significant cost savings, estimated to be approximately $2M-$3M each year. This savings over the planning horizon will significantly offset CIP expenditures.

Table 4-7: Potable Water Demand Projections (MGD)

	2022	2027	2032	2037	2042
Projected ADD	18.5	19.2	20	20.8	21.7
Projected MDD	31.5	33.5	34.8	35.9	37.0
Projected PHD	50.9	54.2	56.3	58.1	60.2
Expected Inefficiencies	12.8%	8%	6%	4%	4%
Projected Average Daily Production Demand	20.7	20.7	21.2	21.6	22.5
Note: Projected ADD was interpolated using Table 3.3: Projected Demand for Potable Water. 2022 potable water demand was assumed to be equal to the 2020 potable water demand.					

NON-POTABLE WATER SYSTEM

As demand increases, the City will need to expand, and perhaps consolidate, the non-potable water and recycled water systems. Transitioning potable water connections to this expanded system will reduce potable water system demands by approximately 1.7 MGD. Table 4-8 shows how the decreasing water loss within the system over time reduces production necessary to meet increasing demands. This table assumes expansion and consolidation of the non-potable water and recycled water systems.

Table 4-8: Projected Production Demands Assuming Optional Expansion

	2022	2027	2032	2042
Average Projected Demand (MGD)	2.2	2.5	2.8	3.1
Assumed Water Loss (%)	40%	28%	16%	5%

	2022	2027	2032	2042
Projected Average Production (MGD)	3.7	3.5	3.3	3.3

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5 DESIGN CRITERIA

5.1 GENERAL DESCRIPTION

This section presents industry-standard guidelines for water infrastructure used to determine replacement or repair needs for the City's water distribution systems. The design criteria also outlines recommendations for the design and construction of infrastructure to ensure reliability, safety, and functionality. In addition, the criteria establish conditions that include but are not limited to water supply, treatment facilities, storage capacity, pressure, pipe velocity, and other hydraulic parameters. These criteria served as a benchmark to evaluate the City's existing infrastructure and identify potential projects for the City's future CIP projects. Data from previous sections including demand factors, water supply, existing infrastructure, and industry standards were evaluated against the design criteria to identify recommended infrastructure upgrades in the CIP. System pressures, pipeline flow velocity, and age were primary criteria of concern.

5.2 DESIGN CRITERIA

5.2.1 DEMAND

POTABLE WATER SYSTEM

Three important demand factors were used when evaluating the City's water system: ADD, MDD, and PHD. The City's potable water ADD is approximately 18.5 MGD, or approximately 222.9 gpcd, which is typical for this region. Due to very effective conservation measures, the current consumption per capita has been reduced from approximately 285 to 222.9 gpcd in the past ten (10) years. The MDD is estimated to be 1.7 times the ADD, or 31.5 MGD. The PHD is estimated to be 2.75 times ADD, or 50.9 MGD. The 1.7 MDD and 2.75 PHD peaking factors are calculated from the City's 2017-2021 SCADA information.

5.2.2 SUPPLY

POTABLE WATER SYSTEM

The City relies on groundwater wells and surface WTP for potable water production. The City supplies approximately half of its demand through groundwater wells. The other half is supplied through its surface WTP. Collectively, these facilities can meet the PHD. However, demand is expected to increase over the next few decades due to growth and further development. As the service areas and demands grow, the City may need an additional five to six (5 – 6) MGD of supply capacity to meet sixty-one (61) MGD of peak hour demand in 2045.

NON-POTABLE WATER SYSTEM & RECYCLED WATER SYSTEM

The City relies on groundwater wells and it's WWTP to supply non-potable and recycled water. However, the City does not utilize the entirety of its WWTP effluent. Therefore, any expansion of the recycled/non-potable water system should use this excess water. The groundwater wells are also used to supply non-potable water to small, detached systems throughout the City. These areas are primarily large parks and open spaces within the City.

5.2.3 STORAGE VOLUME

POTABLE WATER SYSTEM

Fire flow during peak demands was modeled, and water storage was found to be adequate for existing and future hydraulic conditions. Based on the EPA's Effect of Water Age on Distribution System Water Quality Report, Office of Water (4601M), the water distribution system requires enough operational storage (OS) for variation in demand, equalizing storage (ES), stand by and/or fire suppression storage (SB and FSS) and dead storage. The emergency outage storage is evaluated based on the MDD and fire flow requirements. The currently available storage of 54.3 MG is higher than the 2022 storage requirement of 38.9 MG and 2042 storage requirement of forty-five (45) MG. Since current storage is larger than the current and future emergency demand, fire protection and diurnal fluctuations can be met during the emergencies.

For operational variation during emergencies, it is recommended that the City have enough storage to store the MDD (31.5 MG) to allow fluctuating demands throughout the day. Each zone will also need to maintain fire flow capacity to fight fires. California's Fire Code Fire Flow Requirements table was used to estimate the fire flow requirements. The City's maximum fire demand is 8,000 GPM for 4 hours for one zone. This is equivalent to 1.92 MG of storage capacity. Due to the commercial/industrial areas in Zones 1350 and 1570, the required fire flow storage demand is 1.92 MG. Zones 1750, 1900, and 2100 have several large commercial sites that would require approximately 4,000 GPM for 4 hours (0.96 MG). The remaining zones are primarily residential areas with some commercial zoning. These zones require 3,000 GPM for 3 hours (0.54 MG). Emergency Storage will vary by region as it depends on the possible disaster to each water agency. Table 5-1 shows the operation and fire flow and/or emergency storage required for each zone. The overall required storage is sufficient for the City's current production.

Table 5-1: Storage Volume by Zone

	Storage Volume (MG)	2022 Operational Storage (MG)	Fire Flow Required (MG)
Zone 1350	4.9	2.46	1.92
Zone 1570	23.6	12.14	1.92
Zone 1750	5.5	7.24	0.96
Zone 1900	7.4	5.40	0.96
Zone 2100	5.0	3.14	0.96
Zone 2340	6.5	1.02	0.54
Zone 2600	1.4	0.10	0.54
Total	54.3	31.50	7.38

Also, it is essential to note that during emergencies, water could be brought from the WTP, wells, emergency connections with other systems, allowable supply that can be taken from nearby lakes and canals during the fire, and fire trucks from the Fire Department supply. Therefore, it may be difficult to estimate all emergency supply sources accurately. However, with the two (2) emergency interconnections, two (2) WTP, and natural water storage reservoirs, the 54.3 MG of total reservoir capacity is sufficient storage capacity for the planning horizon.

NON-POTABLE WATER SYSTEM

The current storage system includes two (2) small poly tanks and three (3) cement lined open air reservoir throughout the distribution system.

RECYCLED WATER SYSTEM

The City does not have any storage capacity for recycled water, but is currently engineering two (2) 1.5 MG reservoirs to be constructed at the WWTP in the future. This will allow the WWTP to store water until needed, instead of the demand-based system operation that is currently in place. The capacity of the reservoirs will be needed to maintain the operational variations that will exist as demand increases. The recycled water system only services two (2) fire hydrants located at the SCE Mountain View Power Plant and at California Street Well. Since the recycled water system does not service any other fire hydrants for firefighting or require emergency storage in case of a natural disaster, the City will only need storage capacity for its operational flow. The storage capacity is recommended to be equal to the ADD of the recycled water system, which is approximately 2.1 MG for the existing system and three (3) MG for future storage. This excludes the recycled water storage needed for the Mountain View Power Plant.

5.2.4 PUMP STATIONS

POTABLE WATER SYSTEM

Pump Stations are an essential component of the City's potable water distribution system and are required to lift water and maintain the hydraulic grade. The City must maintain emergency power equipment, capacity, and redundancy for the pump stations within the system. The list of booster stations can be found in Table 2-4. The City must maintain a flow capacity equivalent or greater than the MDD, with a three (3)-day fire flow recharge for its system. It is also recommended that the system's redundancy be able to supply the PHD when a pump station has its largest pump offline. In case of a power outage, it is recommended that there is standby power for all pumping stations.

NON-POTABLE WATER SYSTEM

Non-potable system 2 utilizes two (2) small package skid booster pump systems to maintain system pressure within the zone.

RECYCLED WATER SYSTEM

Booster pumps pump recycled water from the wastewater treatment plant's chlorine contact basin into the recycled water system. There are no booster pumps in the City's recycled water systems. The WWTP is constructed in the lowest area of the City in Zone 1. If the City expands and consolidates the non-potable water and recycled water systems, booster pumps will be necessary to transfer water to higher pressure zones. Each pump stations should provide the MDD for the zone it serves. Backup pumps must replace the single largest pump within a pump station facility. Backup power is suggested for each pump station.

5.2.5 WATER TREATMENT

Potable water must meet maximum contaminate level (MCL) standards mandated by the United States (U.S.) Environmental Protection Agency (EPA) and California's Department of Water Resources. The surface water the City utilizes needs to be treated for potential contaminants at the City's treatment plants. Groundwater wells meeting the same MCL standards can also supply potable water. Otherwise, the groundwater must be treated through onsite treatment facilities, or by blending the water with water from other sources to meet the MCL standard. Specific requirements for WTP are dependent on the purpose of the facility. The City owns and operates two surface WTP, Hinckley and Tate. The field report attached in Appendix A provides a detailed description of these facilities.

5.2.6 SYSTEM PRESSURE

POTABLE WATER SYSTEM

Based on American Water Works Association (AWWA) standards, the pressure for the water system is recommended to be between forty (40) and 150 pounds per square inch (psi). The variation within a pressure zone should be no more than twenty (20) psi, as a significant variation in pressure could lead to fatigue within the system due to repeated cycles in hydraulic stress. For example, the residual pressure for a fire hydrant is required to be at twenty (20) psi for effective firefighting. Pressure-reducing valves are required between pressure zones to maintain reliable pressure between each zone.

NON-POTABLE WATER SYSTEM & RECYCLED WATER SYSTEM

Pressure within the non-potable water system should remain between forty (40) psi and 150 psi with a target range between forty five (45) psi to eighty (80) psi. The variation within each pressure zone should not vary by more than twenty (20) psi.

5.2.7 PIPE VELOCITY

Based on AWWA standards, the pipe velocity for the water system should not exceed ten (10) feet per second (fps) under all conditions with a desirable velocity of five (5) fps during normal operations. Pipelines should be sized to provide head losses that do not exceed 3.5 feet per 1,000 feet of pipeline under PHD or five (5) feet per 1,000 of pipeline under MDD conditions. This criteria applies to all three (3) water systems.

5.2.8 PIPELINE REDUNDANCY

Pipeline redundancy involves both avoiding dead-end pipelines and avoiding disruption in the entire system if a pipeline segment needs to be shut down. Avoiding dead ends also helps prevent water stagnation, deterioration, corrosion, and may improve water quality. In some cases, dead ends may be the best or only option to service some areas due to the cost of constructing and maintaining redundant pipelines. Therefore, it is recommended that dead-ends be looped whenever possible, or flushed annually. New water mains must be installed ten feet (10’) horizontally and one foot (1’) vertically from treated and untreated sewage, and recycled water, as outlined in the State of California Code of Regulations: Title 22, Division 4, Chapter 16 Article 4 Section § 64572, Water Main Separation. The City can request an exemption to this standard if a condition meets specific requirements outlined in the same sections. Distribution system pipelines should be a minimum diameter of eight inches (8”) and sized appropriately. Fire hydrant laterals should be six inches (6”) in diameter. This criteria applies to all three (3) water systems.

5.2.9 SUMMARY OF DESIGN CRITERIA

POTABLE WATER SYSTEM

A summary of the potable water system design criteria is provided in Table 5-2, including typical industry standards and current City standards.

Table 5-2: Design Criteria Summary – Potable Water System

Demand	Typical Industry Standards	City Standard
Maximum Day Demand[1,2]	Typical 1.5-2 times ADD	1.7 times ADD
Peak Hour Demand[1,2]	Typical 2-4 times ADD	2.75 times ADD

Demand	Typical Industry Standards	City Standard
Storage		
Reservoir Capacity[1]	Combined Operational, Fire, and Emergency Storage	Combined Operational, Fire, and Emergency Storage
Operational Capacity[2]	Maintain operation during peak demands through the day 50% of MDD	50% of MDD
Fire Storage[1]	Largest Single Fire Flow event	Largest Single Fire Flow event
Emergency Storage[1]	Dependent on Region, Typically 0.5 to 2 times the MDD	50% of MDD, with emergency connection the City maintains
Pump Station		
Pump Station Capacity[1]	Pump Station must supply MDD Booster Pumps must supply MDD and Fire Flow	
Pump Station Configuration[1]	Stand-by pump equal in size to the largest duty pump	
Pump Station Backup Power[1]	Previsions for emergency Power at all Stations	
Pipelines	Industry Standards	
Minimum Diameter Pipe	N/A	8" Diameter for Mainline 6" for hydrant laterals.
Maximum Pipe Velocities[1]	10 fps under all Conditions	10 fps under all Conditions
Maximum Pipe Head Loss[1]	3.5 ft per 1000 ft at PHD 5 ft per 1000 ft at MDD	
System Pressure		
Minimum Static Pressure[1]	40 psi at PHD	40 psi at PHD
Maximum Static Pressure[1]	150 psi at PHD	150 psi at PHD
Minimum Fire Hydrant Residual Pressure[1]	20 psi	20 psi
Maximum Dynamic Pressure Variation[1]	20 psi	20 psi

Note: [1] Industry Standards are based on the AWWA manuals for water distribution systems that include but are not limited to M22 Sizing Water Service Lines and Meters, M31 Distribution System Requirements for Fire Protection and M42 Steel Water Storage Tanks

[2] Analysis of Eastern Municipal Water District: Water System Planning & Design, and Western Municipal Water District: Design Criteria for Water Distribution Systems were used for comparison to City Standards, due to limited information provided in AWWA manuals

NON-POTABLE WATER SYSTEM & RECYCLED WATER SYSTEM

A summary of the non-potable water system and recycled water system design criteria is provided in Table 5-3, including recommended City standards for these water systems.

Table 5-3: Design Criteria Summary - Non-potable & Recycled Water System

Storage	Recommended City Standards
Operational Storage	Equal to ADD
Fire Flow Storage	N/A
Emergency Storage	N/A
Pump Station	**Recommended City Standards**
Pump Station Capacity	Pump Station must supply MDD
Pump Station Configuration	Stand-by pump equal in size to the largest duty pump
Pump Station Backup Power	Previsions for emergency Power at all Stations
Pipelines	**Recommended City Standards**
Maximum Velocity	10 fps peak
Preferred Maximum Velocity	5 fps under ADD and MDD
Maximum Distribution Pipe Velocity	5 fps under all Conditions
System Pressure	**Recommended City Standards**
Minimum Static Pressure	40 psi at PHD
Maximum Static Pressure	150 psi at ADD
Maximum Dynamic Pressure Variation	20 psi
Note: Industry Standards are based on the AWWA manuals for water distribution systems that include but are not limited to M22 Sizing Water Service Lines and Meters, M31 Distribution System Requirements for Fire Protection and M42 Steel Water Storage Tanks	

5.3 PLANNING CRITERIA

5.3.1 METHODOLOGY

The design criteria outlined in this section will be used in determining what facilities need to be repaired, replaced, or constructed. The CIP also considers the age of the City's facilities and includes those facilities that exceed the average service life. Age is not necessarily an indication of current performance issues, but is an indicator that the asset's future performance is expected to deteriorate.

5.3.2 AVERAGE SERVICE LIFE

The age of each pipeline and facility was considered when evaluating service life replacement schedules. The typical service life for various water facilities and pipeline materials are provided in 5-4 and Table 5-5. This information was used to develop CIP recommendations for each water system. If a structure exceeds its average service life, it is expected to deteriorate and increase the chance of failure. The State of California Water Board recommends replacing mainline pipelines every forty (40) years. However, pipeline service life can vary depending on the pipeline material.

Table 5-4: Water Facility Average Service Life

Equipment	State of California Water Board [1]	NMT Asset Management [2]
Source of Supply		
Wells (including appurtenances)	25 – 35	
Intake Structures	35 – 45	
Transmission Mains	35 – 40	65 - 95
Pumping Plants		
Pumping Equipment	10 – 15	15 - 25
Structures	30 – 60	50 - 100
Treatment Plants		
Chlorination Equipment	10 – 15	
Equipment	10 – 15	15 - 25
Structures	30 – 60	60 - 70
Transmission/Distribution		
Structures	30 – 60	50
Reservoirs and Tanks	30 – 60	50 - 80
Main & Distribution Pipes	35 – 40	65 - 95
Services	30 – 50	
Valves	35 – 40	
Backflow Prevention Valves	35 – 40	
Blow-off Valves	35 – 40	
Meters	10 – 15	
Hydrants	40 – 60	
General		
Structures	30 – 40	50
Electrical Systems	7 – 10	15 - 25
Equipment	10 – 15	15 - 25
Transportation Equipment	10	
Computers	5	5 - 10
Store Equipment	10	
Lab/Monitoring Equipment	5 – 7	
Tools and Shop Equipment	10 – 15	
Landscaping/Grading	40 – 60	
Power Operated Equipment	10 – 15	
Communication Equipment	10	

Note: (1) California State Water Resources: Table 1: Typical Equipment Life Expectancy; (2) Environmental Finance Center NMT-Asset management guide for water and wastewater
The life expectancy of mechanical equipment and pumps is assumed to be 20 years, and the Electrical equipment 15 years for CIP purposes

Table 5-5: Water Pipeline Average Service Life

Standard Abbreviation	Material	Service Life (Years)
ACP	Asbestos Concrete Pipe	80
CIP	Cast Iron Pipe	120
CMLC	Cement Mortar Lined and Steel Coated	100
CON	Concrete	100
DIP	Ductile Iron Pipe	100
DW	Dipped and Wrapped Steel	40
PVC	Polyvinyl Chloride	70
STL	Steel	70
Note: Based on MASC Life Expectancy of Water Distribution Lines		

5.3.3 PERFORMANCE INDICATORS

The primary performance indicators used for pipelines are the flow velocities and pressures predicted by the hydraulic model. The updated hydraulic model was used to identify pipe segments and conveyance facilities that require upgrades to meet the performance metrics presented in Section 5.2.9. Additional repair and replacement considerations include the age of water facilities and whether the City has indicated a performance issue with a specific facility.

6 EXISTING SYSTEM ANALYSIS

6.1 GENERAL DESCRIPTION

This Section discusses the hydraulic model development and calibration for the existing water systems to meet the current and future needs of the City's service area. The hydraulic model runs inside of a computer Geographic Information System (GIS) that manages and maps the individual components of the City's water infrastructure. The physical components of the hydraulic model include water pipelines, valves, storage tanks, pumping facilities, source water supplies, and water demands.

The model was calibrated with operational data used to set boundary conditions. Extended period simulations were performed using demand and supply data, along with demand patterns developed in previous sections. Settings were adjusted to calibrate the model to a reasonable representation of the system performance, and the model was considered calibrated when its output matched the collected instrumentation data.

Once calibrated, several scenarios were analyzed to evaluate operating pressure and pipeline flow velocity. Each scenario was chosen to represent the different operational conditions of the system. These scenarios included:

- Existing System operating under Average Day Demands
- Existing System operating under Maximum Day Demands with and without Fire Flow
- Existing System operating under Peak Hour Demands

Because the size of the non-potable water system and recycled water system is small, with only a well pump into the distribution pipe, the existing system will not be modeled. However, since the City is planning to construct recycled water reservoirs at the WWTP, the current system was modeled with these future reservoirs.

The results of these scenario analyses are presented in subsection 6.4. The results are summarized in color-coded maps to identify out-of-range pressure nodes and/or pipeline segments quickly. In addition, the results of these analyses are used to identify improvements to the system that will become recommended capital improvement projects.

6.2 METHODOLOGY

A detailed hydraulic model is a valuable tool used to analyze the complex operation of a water system. The general steps of a model formulation are:

1. Inputting the system's physical data in GIS format
2. Obtaining meter data to set boundary conditions in the model
3. Translating the physical data into a network of nodes and links
4. Inputting accurate water demands
5. Calibrating the model to simulate actual field conditions and system performance
6. Performing model runs based on current and future system conditions to predict performance.

The physical data required for a hydraulic model includes the geographic network of pipes, nodes, tanks, pump stations, valves, and supply sources representing the City's potable water system. The connectivity of the pipes and nodes in GIS allows the system components in the model to be hydraulically linked. Pipe information includes the pipe diameter, length, pipe material, and associated roughness coefficient. The roughness coefficient function, known as the Hazen-Williams "C" factor (when the Hazen-Williams head loss formula is used), estimates friction losses in the system. The "C" factor is assigned based on the diameter, material, and when known, pipe age. However, "C" factors are subjective, and also based on industry best practices and operations input. Node information contains the node elevation and water demand or supply at that point in the system.

Initial hydraulic boundary conditions must be entered into the model database. Of particular importance is the initial water level for tanks and the initial open/closed setting for control valves. City water supply sources, such as pumped sources from groundwater wells and treatment plants, can be modeled as either varied or constant supplies into the water system. Understanding and adequately simulating these boundary conditions is critical to the successful calibration of the model.

Determining accurate water demands is crucial to developing an accurate hydraulic model. Metered demands, water supplies, pumped flows, and changes in tank volumes are reviewed over a given period to determine actual daily demand patterns. Annual consumption by metered account provides a spatial distribution of demand and average system usage.

Node elevations were updated using current topographic maps. Where available, as-built information was used to update the model to match existing conditions. Storage tanks were annotated with ground elevation, diameter, and height. Operational settings in the model were verified during workshops with the City staff and through a detailed review of SCADA operational data. These settings were updated in the hydraulic model. The locations of normally closed valves were also confirmed and identified in the model.

The current operational status and functionality for the City's potable water system pressure reducing stations (PRS) were obtained from City staff and updated in the hydraulic model. Settings provided by City staff represent typical conditions and may vary depending on the season, system demands, and storage conditions. For example, operations staff may change the settings to allow more water into a particular system to fill a tank or less water to turn over the tank.

6.3 HYDRAULIC MODEL CALIBRATION

The final step in developing a reliable hydraulic model is calibration. For the 2021 Master Plan, SCADA information was evaluated to provide the City with a reliable and accurate overview of its potable water system. This was necessary to analyze the water distribution system correctly. A properly calibrated model provides the confidence needed to make significant capital planning decisions and delivers a planning tool to guide operational decisions.

Macro-level calibration procedures use continuous monitoring to obtain data points to simulate system operations over an extended period. Actual field data can be obtained using SCADA records or by placing monitoring equipment in the system. For the City's potable model calibration, a week of data from August 2019 was obtained from SCADA for tank levels, pump station flows and/or status, and pressures, where available. The data was used to establish boundary conditions for the calibration period.

The hydraulic model was calibrated for an extended period simulation (EPS). EPS calibration was performed to ensure the model accurately reflects how the overall system operates over time concerning transmission mains, pumps, tanks, and reservoir operations under normal operating conditions. A preliminary review of the model data was conducted before EPS calibration. It was believed to provide a reasonably accurate representation of actual system characteristics in water main geometry, spatial demand allocation, and pipe roughness. Precise duplication of the data recorded at all locations within the water distribution system during extended period calibration is not realistic due to many factors influencing the results. Model calibration aims to minimize the error between the SCADA and the model simulations and create a "best fit" at all locations. Some error between the SCADA and model simulations is expected; however, limits to the amount of allowable error must be made to ensure the calibrated model accurately represents the existing potable water distribution system. Based upon the size and number of facilities in the developed model, the desired accuracies of the extended period calibration for the hydraulic model are:

1. Minimum of twenty-four (24) hours is performed.
2 Tank levels must be within five feet (5') between field data and model simulations at least eighty percent (80%) of the time.
3. Tank levels must be within eight feet (8') between field data and model simulations the entire time.

A composite time-of-day demand curve was determined for each pressure zone within the potable water system for extended period calibration based on available SCADA data and plant production rates. The time-of-day diurnal demand curve is a series of 24-hour demand factors that define how water usage varies over a day. Each demand factor is defined as the ratio of the hourly demand to the daily average. The composite time-of-day calibration demand curve corresponding to each potable water pressure zone is provided in Chart 6-1.

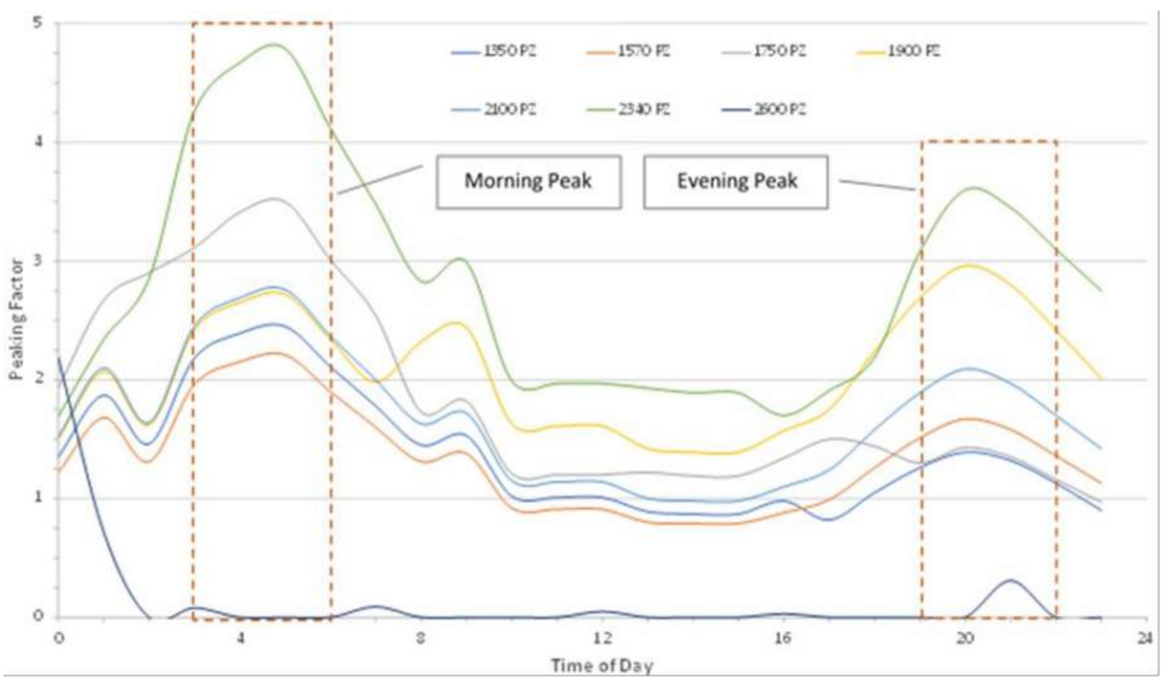

Chart 6-1: Composite Time-of-Day Demand Curve

The diurnal curves developed for model calibration closely resemble the traditional "double hump" pattern of water use throughout the day, with morning and evening peak demands. The morning peak hour demand, which occurs from 3:00 - 6:00 a.m., was 2.2 - 4.8 times the average use for the day, which represents the peak water use for the day for most of the system, with the exception for Pressure Zone 1900, where peak water use occurs during the evening.

Extended period simulations were performed on the potable water system using the demand curves developed above. In general, the tanks and SCADA points exhibited similar trending patterns in the model compared to the field data collected. Tank and pump station trending graphs resulting from the extended period calibration are included in Appendix B.

Examples of the calibration results are shown in Chart 6-2 and Chart 6-3, illustrating some of the variations between the field data and model simulations. The calibration results for these two tanks are examples of the level of accuracy between actual tank water levels observed and the model predictions throughout the system. In summary, the extended period simulation satisfies the calibration goals discussed with City staff.

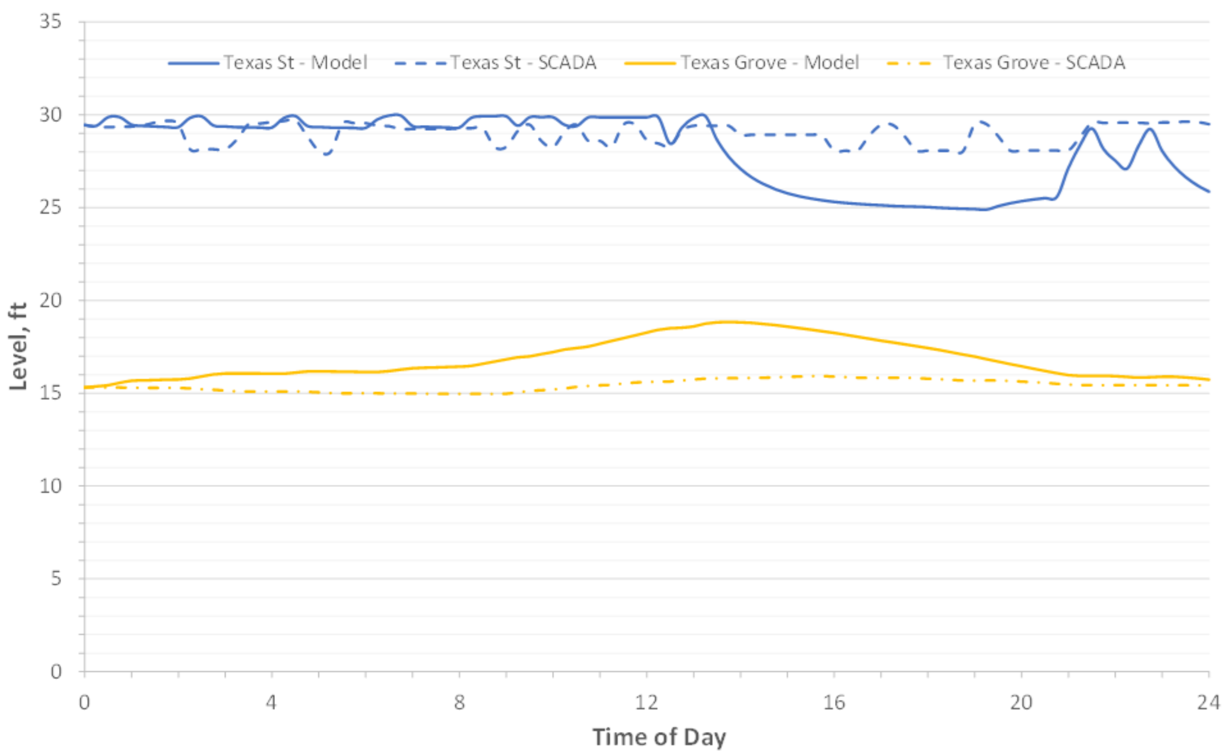

Chart 6-2: Calibration Results for 1350 PZ Tank Levels – 1

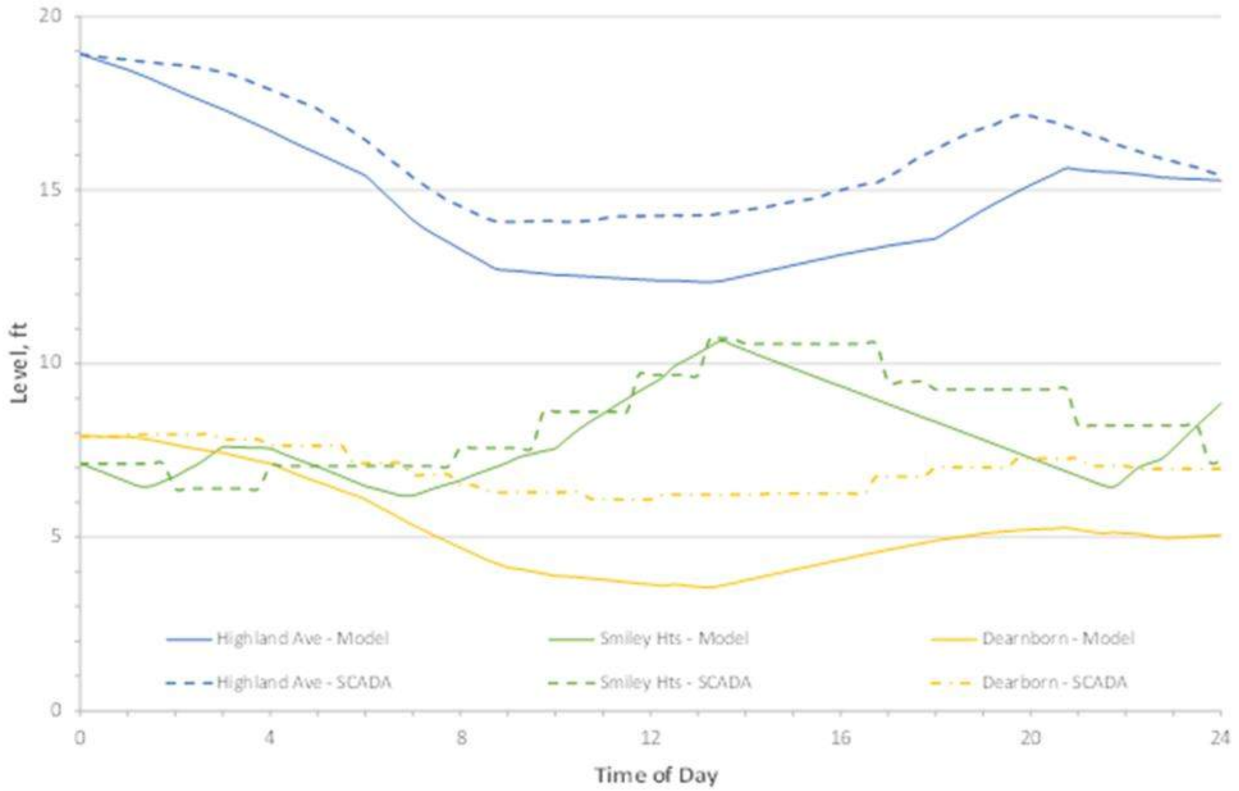

Chart 6-3: Calibration Results for 1350 PZ Tank Levels – 2

During the extended period model calibration, adjusting valve and pump settings slightly to simulate accurate tank levels was necessary. Table 6-1 includes the calibration results after localized adjustments to improve the model's accuracy. Typically, this information would include pumped flows and discharge pressures. Actual pump flows were estimated based on pump status for all sites except the Country Club Booster pump station, which was not available. The modeled flows are within ten percent (10%) of the total anticipated flow presented in SCADA. Discharge pressures were only provided for this site as well. While they are consistently less than SCADA (approximately 7 – 10%), this could be due to an elevation discrepancy, pump losses, location of the gage, and other factors.

Table 6-1: Model Calibration Accuracy

Parameter	Allowable Deviation	Minimum Acceptance Required	Acceptance Level Achieved
Tank Level Differential between field and model	2 feet	80%	70%
Tank Level Differential between field and model	5 feet	100%	97%

6.4 EXISTING SUPPLY ANALYSIS – POTABLE WATER SYSTEM

6.4.1 SCENARIO 1: EXISTING SYSTEM, ADD, EPS, MAX PRESSURE OVER 24 HOURS

This scenario models the average daily demand for the system, primarily looking at the pressure within the system. As described in the design criteria, the preferred pressures will be between forty (40) psi and 150 psi. Figure 6-1 shows the results of the analysis. Most high-pressure areas that exceed the 150 psi pressure limit are in the western portions of each pressure Zone 1570, 1700, 1900, 2100, and 2340 near the zone boundaries, which can be expected. In addition, higher pressure was also identified downstream of booster stations P-2381 and P-2382, which pump water from Ward Way Reservoirs to higher pressure zones.

6.4.2 SCENARIO 2: EXISTING SYSTEM, MDD, EPS, MAX PRESSURE OVER 24 HOURS

This scenario models the maximum daily demand for the system, primarily looking at the pressure within the system. The model as calibrated used the peak day on the maximum month for the year and is therefore considered to be the MDD. The pressure range, less than forty (40) psi and greater than 150 psi, was used to locate areas are of concern. Figure 6-2 shows the results of this analysis, with similar results as Scenario 1—a few areas with high pressure in the western portion of Zone 1900.

6.4.3 SCENARIO 3: EXISTING SYSTEM, MDD, EPS, MAX VELOCITY OVER 24 HOURS

The MDD Scenario was used to analyze the pipe velocities within the system. The maximum velocity design criteria was used to identify pipeline projects for the CIP to alleviate high system velocities. Figure 6-3 shows that very few areas exceed the maximum recommended velocity. The areas with high velocities are near pumping stations, which is expected.

6.4.4 SCENARIO 4: EXISTING SYSTEM, MDD, FIRE FLOW, RESIDUAL PRESSURE DURING FIRE EVENT (Steady State)

This scenario analyzed the residual pressure at fire hydrants during a fire flow event. To provide adequate fire flow pressure, the residual pressure should not fall below twenty (20) psi. The minimum required fire flow was generated utilizing information supplied by the City. Figure 6-4 shows the residual pressures from the fire flow scenario.

6.4.5 SCENARIO 5: EXISTING SYSTEM, PHD, STEADY-STATE SCENARIO, MIN PRESSURE

This scenario analyzed the system pressures under peak hour demand. The maximum day demand and peaking factors were used for the simulation. The pressure range used to determine adequate pressure was between forty (40) psi and 150 psi. Figure 6-5 shows the results of this analysis. The results are similar to results from Scenarios 1 and 2. The high-pressure area is located in the western portion of Zone 1900 and the west portion of the City's service area.

6.4.6 SCENARIO 6: EXISTING SYSTEM, PHD, STEADY-STATE SCENARIO, MAX VELOCITY

This scenario analyzed the velocity conditions under peak hour demand. Although a few isolated areas exceed the maximum recommended velocity, there are no significant areas with excess velocity. Generally, locations with high velocity are near pumping stations which is expected. The results are shown in Figure 6-6.

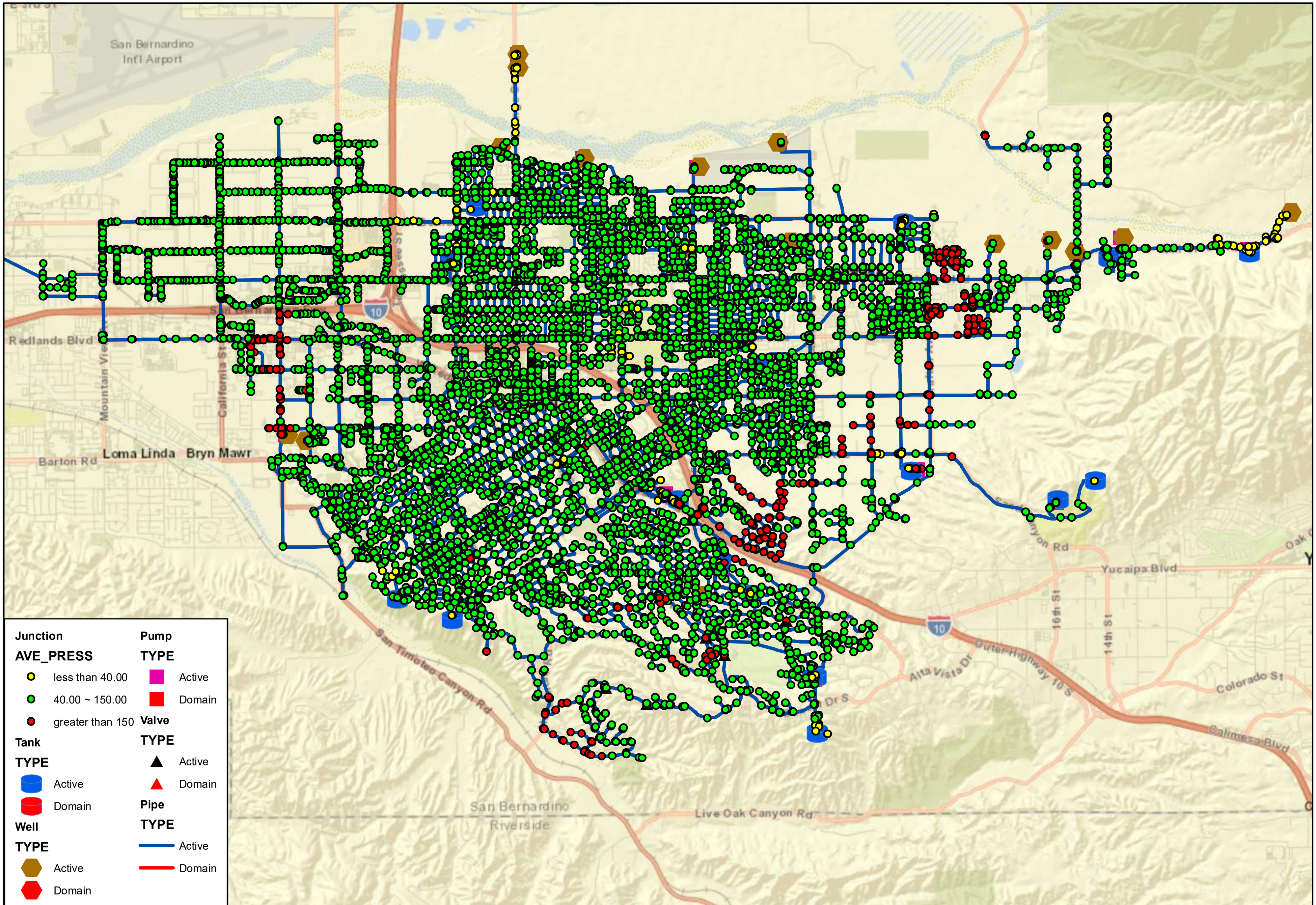

Figure 6-1
Hydraulic Capacity Evaluation
Existing System, ADD, EPS, Maximum Pressure Over 24 Hours

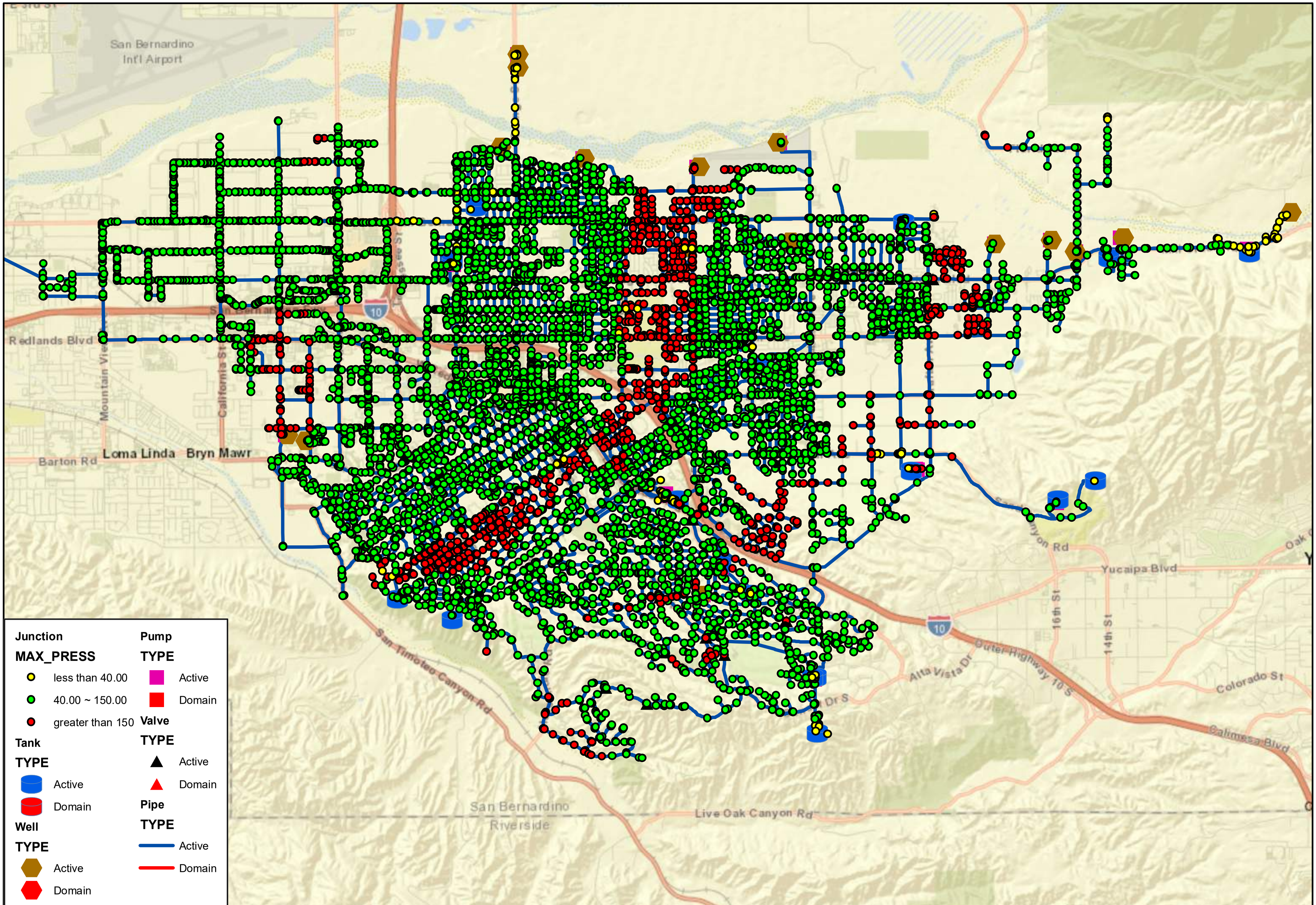

Figure 6-2
Hydraulic Capacity Evaluation
Existing System, MDD, EPS, Maximum Pressure Over 24 Hours

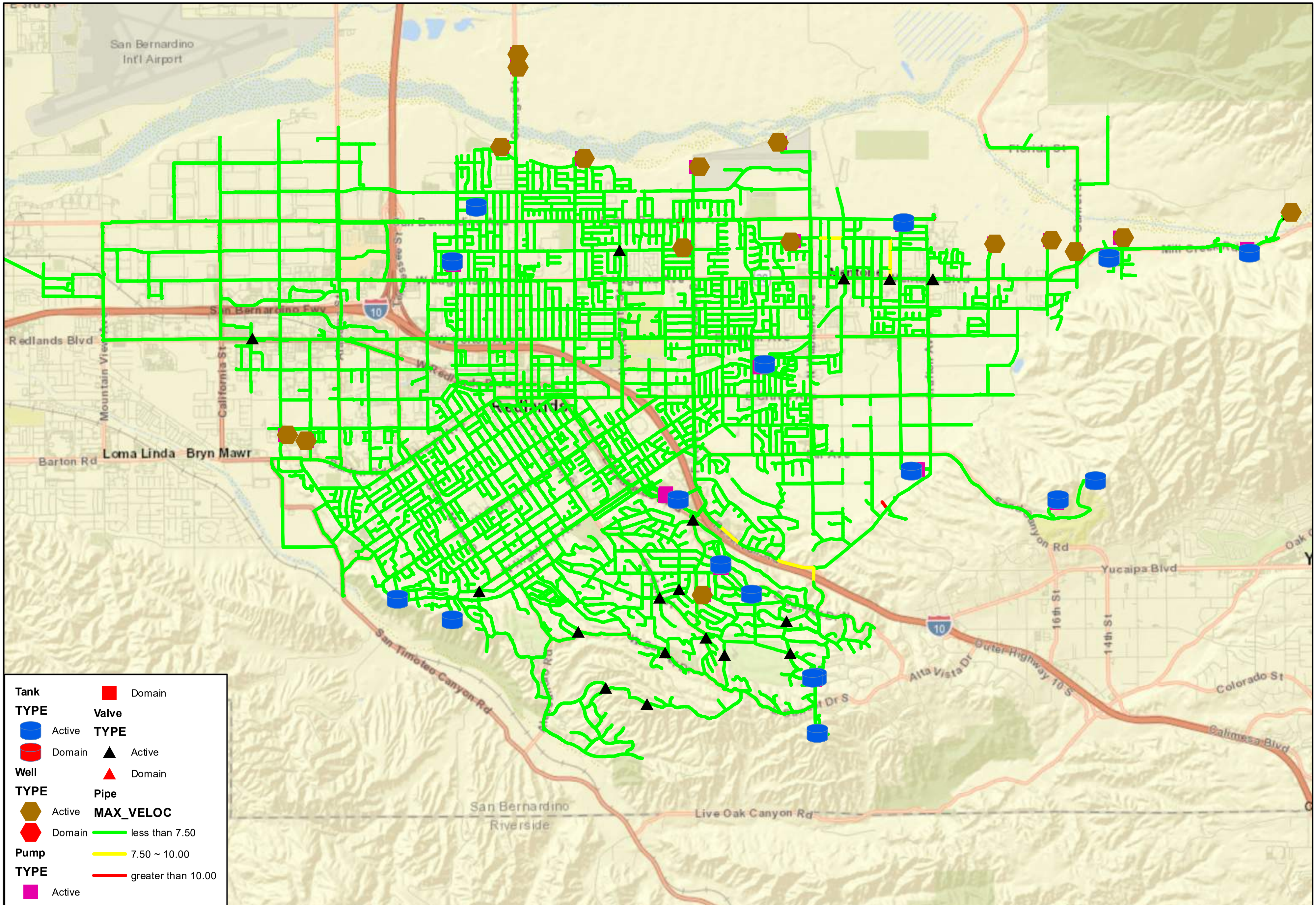

Figure 6-3
Hydraulic Capacity Evaluation
Existing System, MDD, EPS, Maximum Pipe Velocity Over 24 Hours

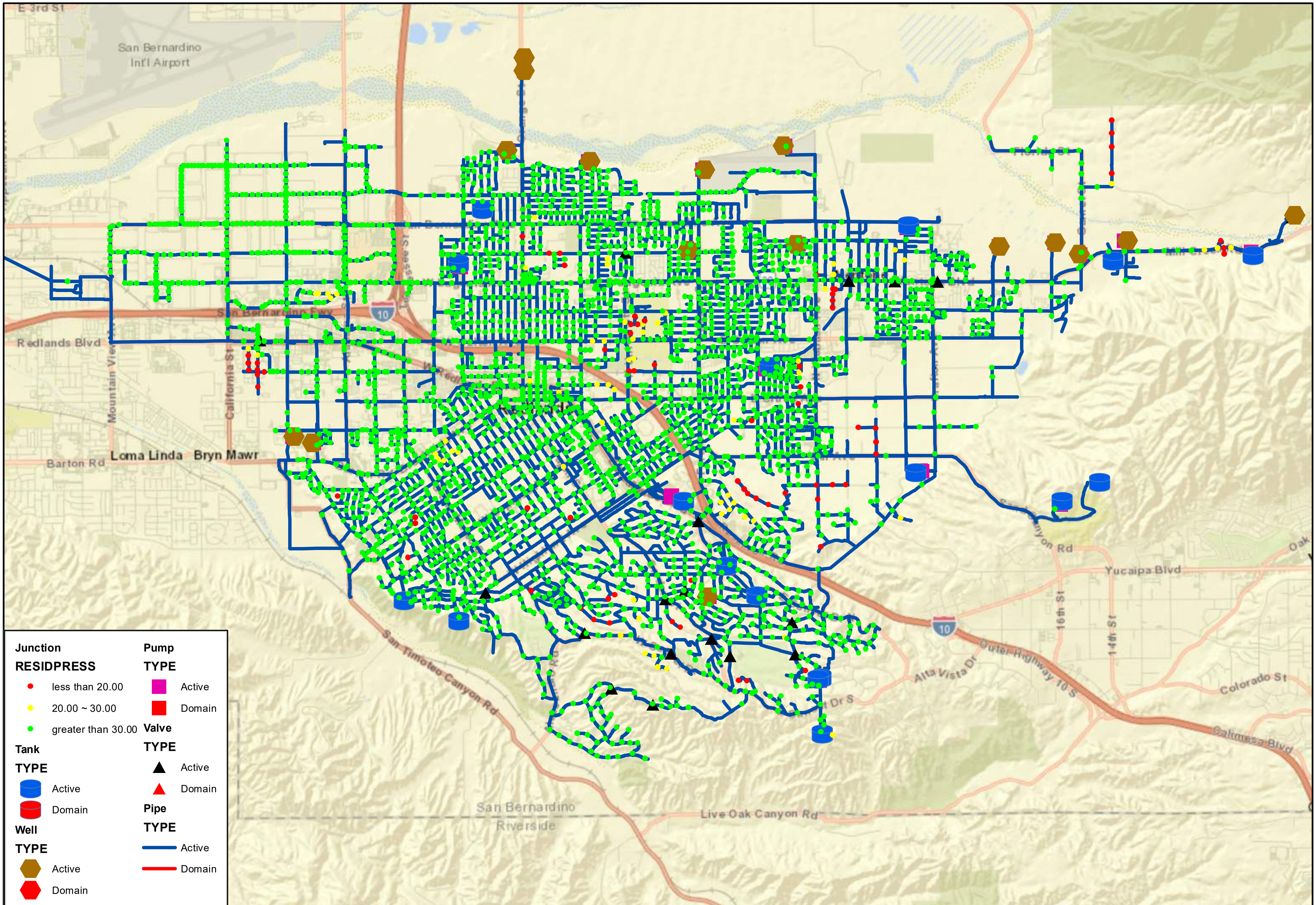

Figure 6-4
Hydraulic Capacity Evaluation
Existing System, MDD, Fireflow Scenario, Residual Pressure During Fire Event

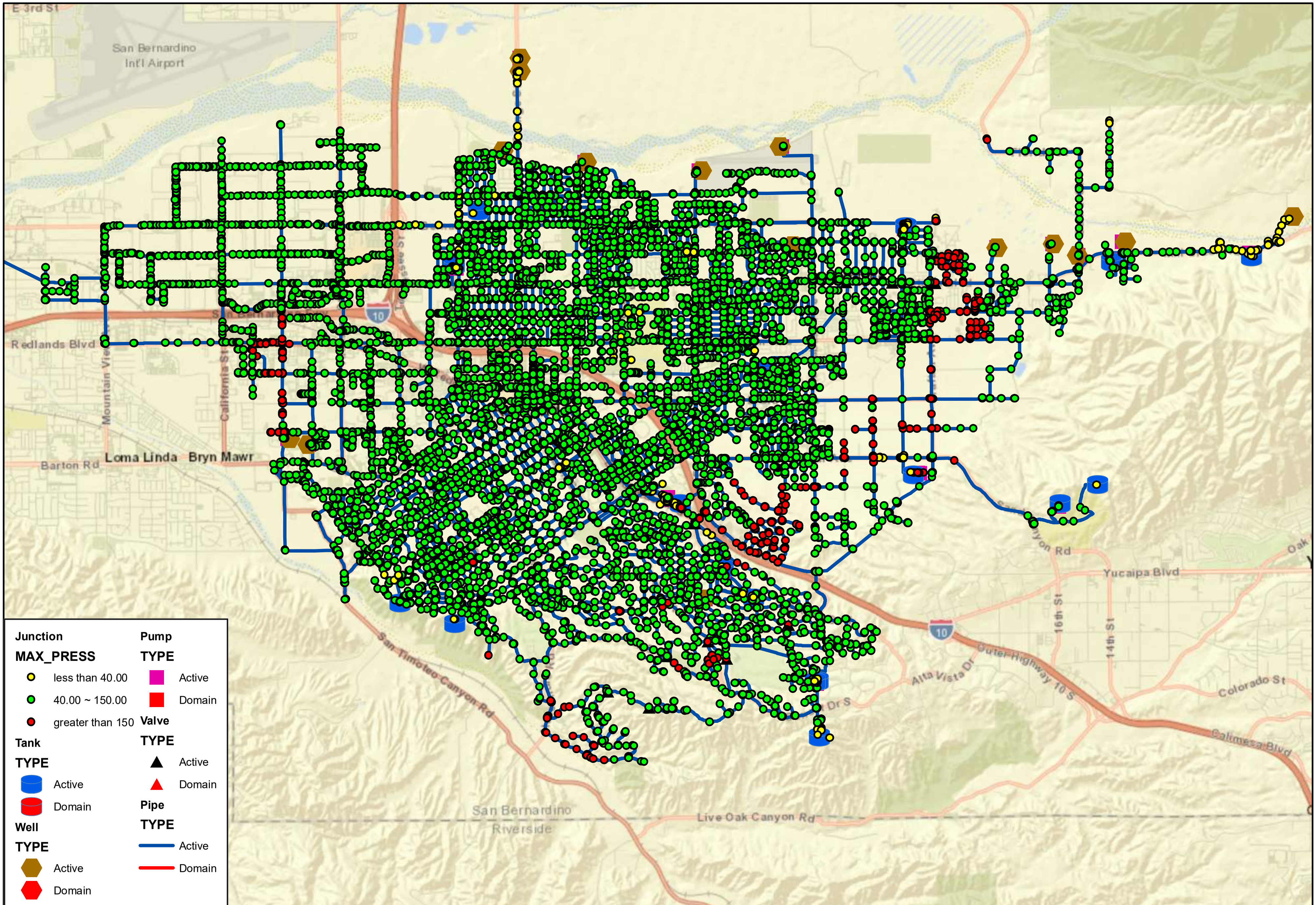

Figure 6-5
Hydraulic Capacity Evaluation
Existing System, PHD, Steady State Scenario, Maximum Pressure

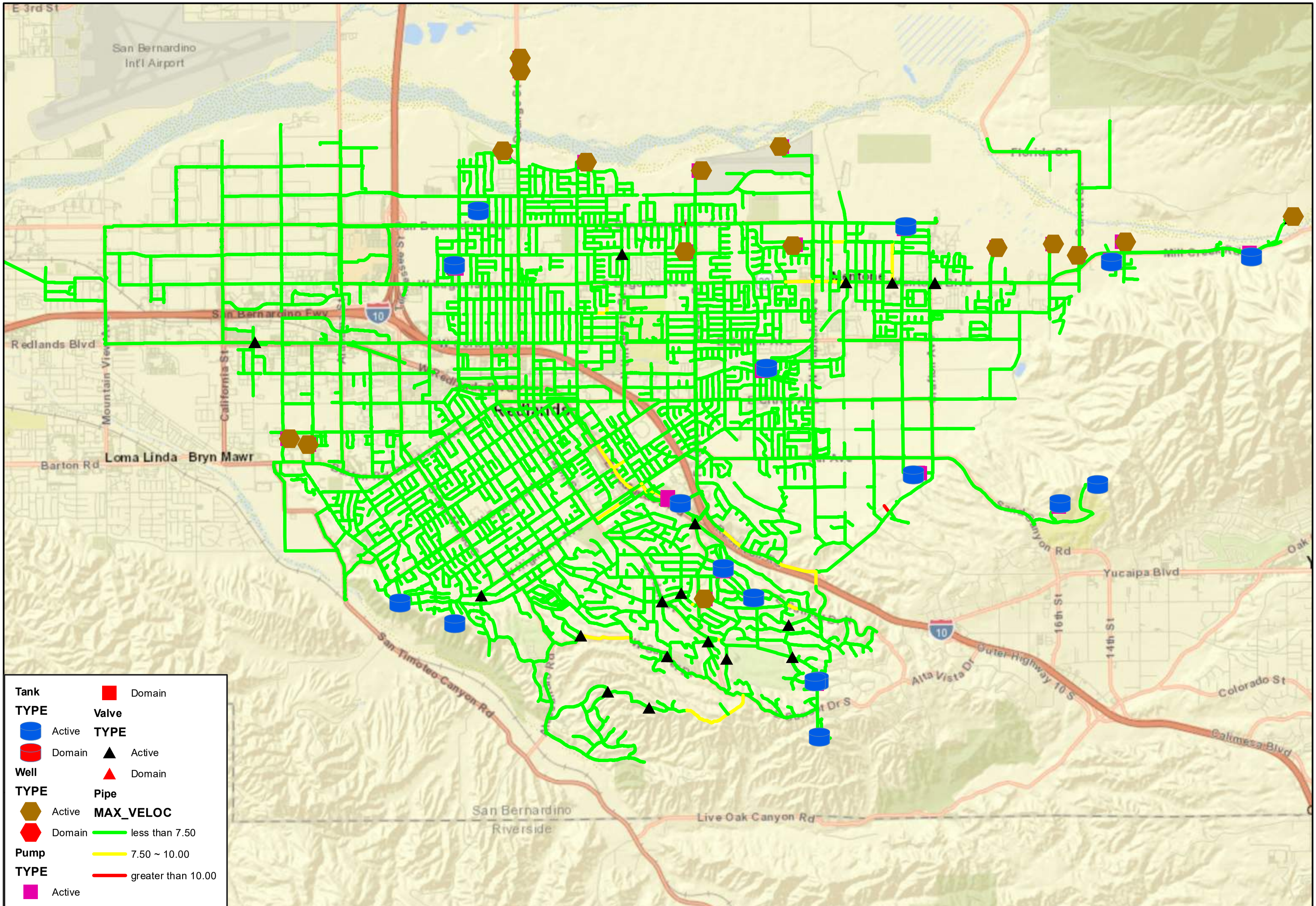

Figure 6-6
Hydraulic Capacity Evaluation
Existing System, PHD, Steady State Scenario, Maximum Velocity

6.5 EXISTING SUPPLY ANALYSIS – NON-POTABLE WATER SYSTEM & RECYCLED WATER SYSTEM

6.5.1 ZONE 1 SUPPLY

Zone 1 currently contains a single well and the WWTP that primarily produces water for the SCE Mountain View Power Plant. However, some of this water is blended with the California Street Well to service Zone 1. Typically, the recycled water from the WWTP supplies Zone 1. However, when the power plant needs more water or the WWTP decreases its water production, the California Street Well increases its water production to supply this zone. It should be noted that the system does not have any storage capacity and requires the pumps to turn on and off depending on the current demand of the system. The City is currently engineering two (2) recycled water reservoirs for construction at the WWTP in the future to resolve this issue.

6.5.2 SYSTEM 2 SUPPLY

System 2 is currently supplied by five (5) wells: Well No. 30A, Well No. 31, Well No. 32, Well No. 41, and the New York Street Well, although Well No. 30A and the New York Street Well are the primary supply wells. Well No. 30A is typically the primary yearly source. The New York Street Well supplements production during spring and summer months and decreases production during autumn. It should be noted that the system does not have any storage capacity and requires the pumps to turn on and off depending on the current demand of the system. In addition to on and off control of pumps the pressure is regulated with the use of Cla-Valves. There will be instances where water will be discharged to the storm drain while maintaining the system pressure for end users.

6.5.3 SYSTEM 3 SUPPLY

There are no existing non-potable sources in this system. However, BVMWC does supply some parts of the northern part of this system, which include the University of Redlands and Redlands Sports Park. This is provided through the open-air BVMWC Reservoir at Agate.

6.5.4 DETACHED SYSTEMS

The detached systems typically include a well, a single pipeline, and the end-user. The purpose of these wells is to provide groundwater to a specific single end-user, such as a park or golf course.

6.5.5 SCENARIO 1, EXISTING SYSTEM, ADD, EPS, MAX PRESSURE OVER 24 HOURS

This scenario looks at the pressure of the existing system and the future reservoirs. As Figure 6-7 shows, the existing system operates with some low-pressure areas in the east part of Zone 1. These areas are primarily due to low demand within the system.

6.5.6 SCENARIO 2, EXISTING SYSTEM, MDD, EPS, MAX PIPE VELOCITY OVER 24 HOURS

This scenario looks at the MDD for the existing system modeled with the future reservoirs to check for excess velocity. Velocity below ten (10) fps is acceptable, with a preferred velocity of five (5) fps. As Figure 6-8 shows, four (4) areas exceed ten (10) fps and should be upsized. These projects are shown and explained further in Table 6-2.

Table 6-2: Undersized Non-Potable Water System Pipelines

	Street	From	To	Existing Material	Length (ft)	Existing Diameter (in)	Proposed Diameter (in)	New Material
NP-CIP-1	Pioneer Ave	Nevada St	Alabama St	DIP	2608	6	12	DIP
	Alabama St	Pioneer Ave	625' South of Pioneer St	DIP/PVC	624	6	12	DIP
Subtotal					**3232**			
NP-CIP-2	Orange Tree Ln	California St	Oregon St	PVC	1900	6	8	DIP
	Orange Tree Ln	Oregon St	240' East of Plum Ln	PVC	1468	6	10	DIP
Subtotal					**3368**			
NP-CIP-3	Texonia Park	N//A	N//A	DIP	1006	10 & 12	16	DIP
	W Lugonia Ave	Texas St	Lawton St	STL/PVC	1200	6,8, & 10	12	DIP
Subtotal					**2206**			

6.5.7 OTHER AGENCIES

Currently, three (3) other agencies operate non-potable water sources within the City water service area and its sphere of influence. The Crafton Water Company (CWC), BVMWC, and Western Height Water agencies. These water agencies primarily irrigate City-owned agricultural and landscape areas within the City and its sphere of influence. The City has partial ownership of these and several other local water companies. Additional water stock information is available on the City website. The City owns 184 acres of citrus groves divided into twenty-three (23) separate groves. BVMWC provides irrigation water for seven (7) of these groves and some nearby parks. CWC provides irrigation water for two (2) groves and parts of Crafton. These groves and the water providers are listed in Table 6-3.

Table 6-3: City-Owned Citrus Groves

No.	Name[1]	Water Provider[2]	Acres[1]
1	Mullin	Bear Valley	8.5
2	Judson	Bear Valley	13.1
3	Lugonia	Bear Valley	18
4	Pioneer & Judson East	Bear Valley	2.7

No.	Name[1]	Water Provider[2]	Acres[1]
5	Pioneer & Judson West	Bear Valley	4.3
6	Dearborn & Pioneer	Bear Valley	8.6
7	Granite	Bear Valley	2.7
	Subtotal		57.9
8	Fifth Ave	C.W.C.	10.6
9	Jacinto Memorial	C.W.C.	4.2
	Subtotal		14.8
10	Riverview	Groves on Wells	5.9
11	University Grove	Groves on Wells	23.5
	Subtotal		29.4
12	Ramirez	Recycled Water	4.6
13	Daniels	Recycled Water	4.9
	Subtotal		9.5
14	Beal Park	City	0.4
15	California	City	4.8
16	Fire Station 262	City	0.04
17	Mountain View	City	13.9
18	Olive	City	3.7
19	Prospect	City	25
20	Texas	City	12.5
21	Wabash	City	1.4
22	West Redlands Gateway	City	6.4
23	West Riverview	City	4.3
	Subtotal		72.44
	Total		**184.04**

Note: (1) From City Owned Citrus Grove Map provided by the City; (2) Provided by the City.

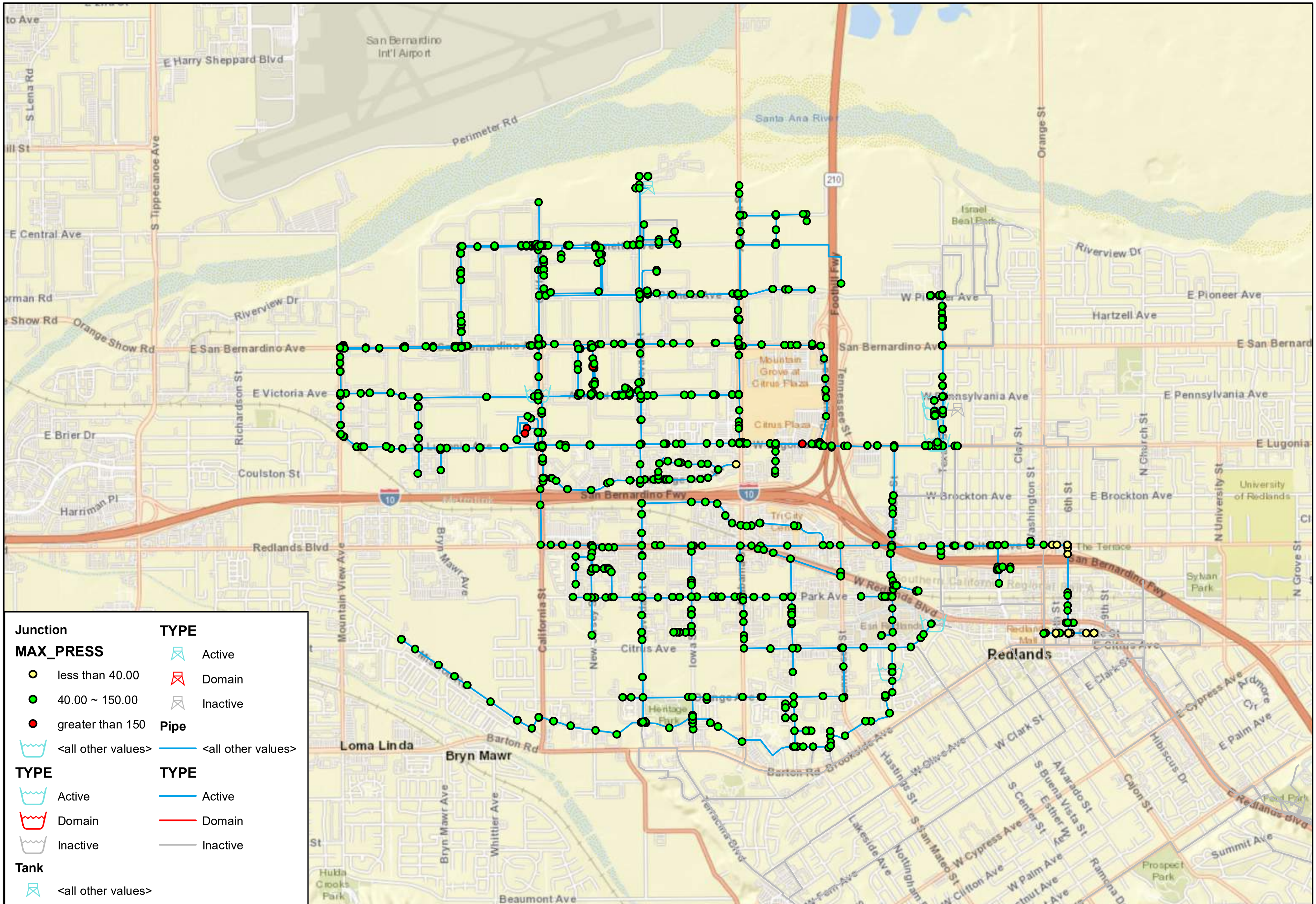

Figure 6-7
Hydraulic Capacity Evaluation
Existing System, ADD, EPS, Maximum Pressure Over 24 Hours

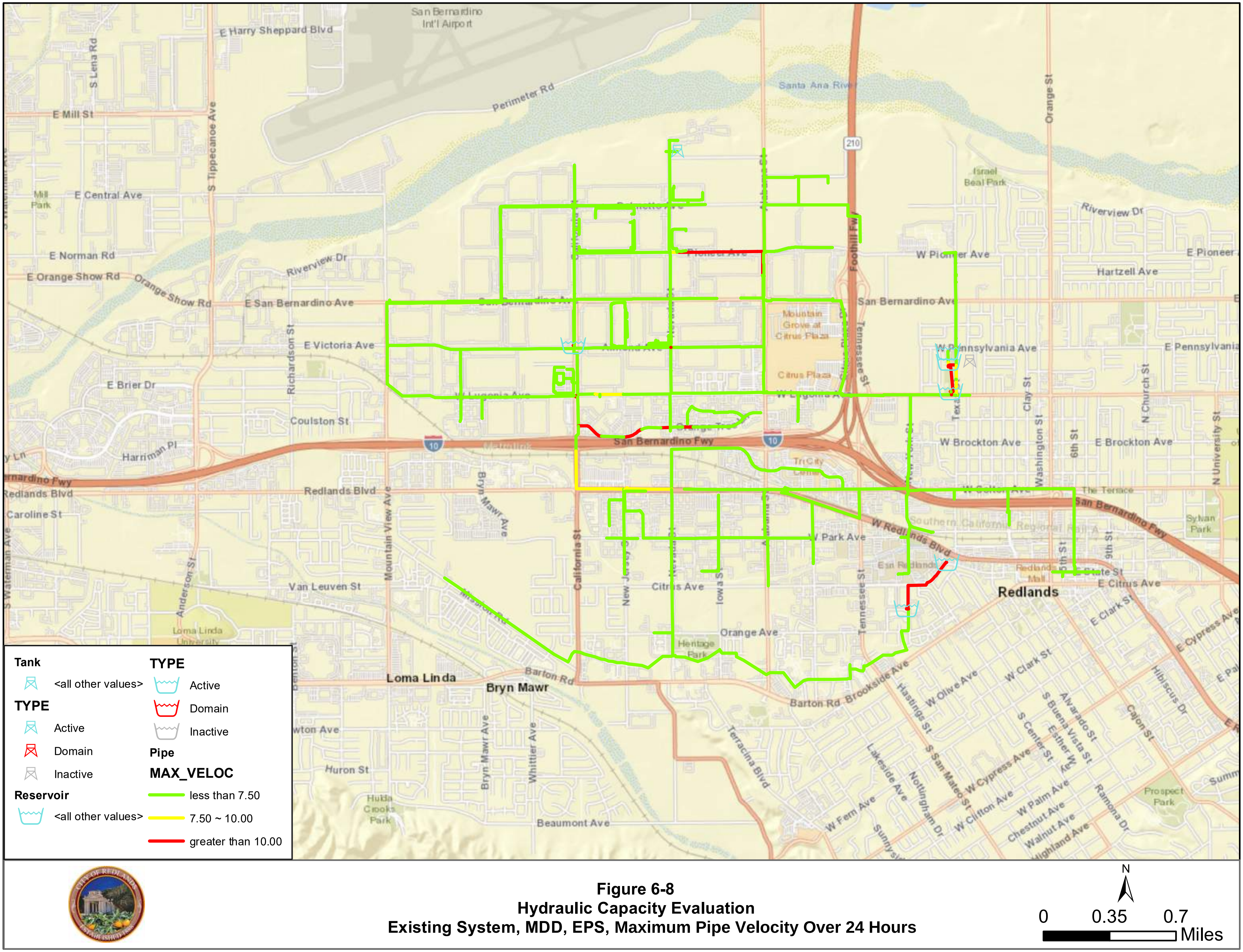

Figure 6-8
Hydraulic Capacity Evaluation
Existing System, MDD, EPS, Maximum Pipe Velocity Over 24 Hours

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7 FUTURE SYSTEM

7.1 GENERAL DESCRIPTION

Several additional potable water system scenarios were analyzed to evaluate future operating conditions and evaluate pressure and pipeline flow velocity under these conditions. The future conditions were selected to represent 2042 projected water demands, assuming that all improvements identified in Section 6 are completed. Five (5) scenarios were analyzed, including ADD, MDD, PHD, and fire flow conditions. The results are summarized in color-coded maps to quickly identify out-of-range pressure nodes and/or undersized pipeline segments. Additional CIP project recommendations were developed based on this analysis.

The City's existing non-potable water system was analyzed to develop a future water system to decrease potable water use and better serve the City's existing recycled/non-potable water system. Scenarios were modeled to evaluate system pressure and velocity under various conditions, and evaluated against the design criteria. Utility billing data was used to identify potential customers for expansion of the system. When possible, meters providing agricultural or landscape irrigation water were read to supplement the utility billing data. New pipelines are assumed to be eight inches (8”) in diameter, and the hydraulic model assumed construction of the WWTP recycled water storage reservoirs was complete.

7.2 FUTURE SUPPLY ANALYSIS – POTABLE WATER SYSTEM

7.2.1 SCENARIO 1 FUTURE DEMAND, EXISTING SYSTEM, MDD, FIRE FLOW SCENARIO, RESIDUAL PRESSURE DURING FIRE HOURS.

This scenario used the predicted future MDD of approximately thirty-seven (37) MGD to analyze fire flow and used the minimum of twenty (20) psi of residual pressure for fire hydrants. Figure 7-1 shows areas of concern that appear similar to the existing demand analysis. The pipe sections with deficiencies are included in the CIP list in Table 7-1.

7.2.2 SCENARIO 2 FUTURE DEMAND, EXISTING SYSTEM, MDD, EPS SCENARIO, MAX PRESSURE OVER 24 HOURS.

This scenario used the existing system under the future MDD of approximately thirty-seven (37) MGD to predict the distribution of pressure within the system. Figure 7-2 shows the results of this analysis. Areas of high pressure were identified primarily in the west sized of Zone 1570, portions of Zone 1750, and in the western portions of Zone 2100.

7.2.3 SCENARIO 3 FUTURE DEMAND, ADD, EPS SCENARIO, MAX PRESSURE OVER 24 HOURS

This scenario analyzed the future ADD and the maximum pressure in the system. The simulation identified pressures higher than 150 psi within the western portion of Pressure Zones 1570, 1900, and 2340. The results are shown in Figure 7-3.

7.2.4 SCENARIO 4 FUTURE DEMAND, MDD, EPS SCENARIO, MIN PRESSURE OVER 24 HOURS

This scenario analyzed the future MDD to locate minimum pressures. It shows that the low-pressure system area is primarily in the western portion of Pressure Zone 2100. The western parts of Pressure Zone 1570 and 2340 still have high pressures. The results are shown in Figure 7-4.

7.2.5 SCENARIO 5 FUTURE DEMAND, MDD, EPS SCENARIO, MAX VELOCITY OVER 24 HOURS

This scenario analyzed the future MDD velocities within the existing system. It shows isolated areas where velocities are above 10 feet per second. These pipelines are included in the CIP list in Table 7-1. The results are shown in Figure 7-5.

7.2.6 SCENARIO 6 FUTURE DEMAND, FUTURE SYSTEM, MDD, FIRE FLOW SCENARIO, RESIDUAL PRESSURE DURING FIRE HOURS.

This scenario analyzed the future MDD of the system during a fire flow event. The low residual pressures are included in the CIP list in Table 7-1. The results are shown in Figure 7-6.

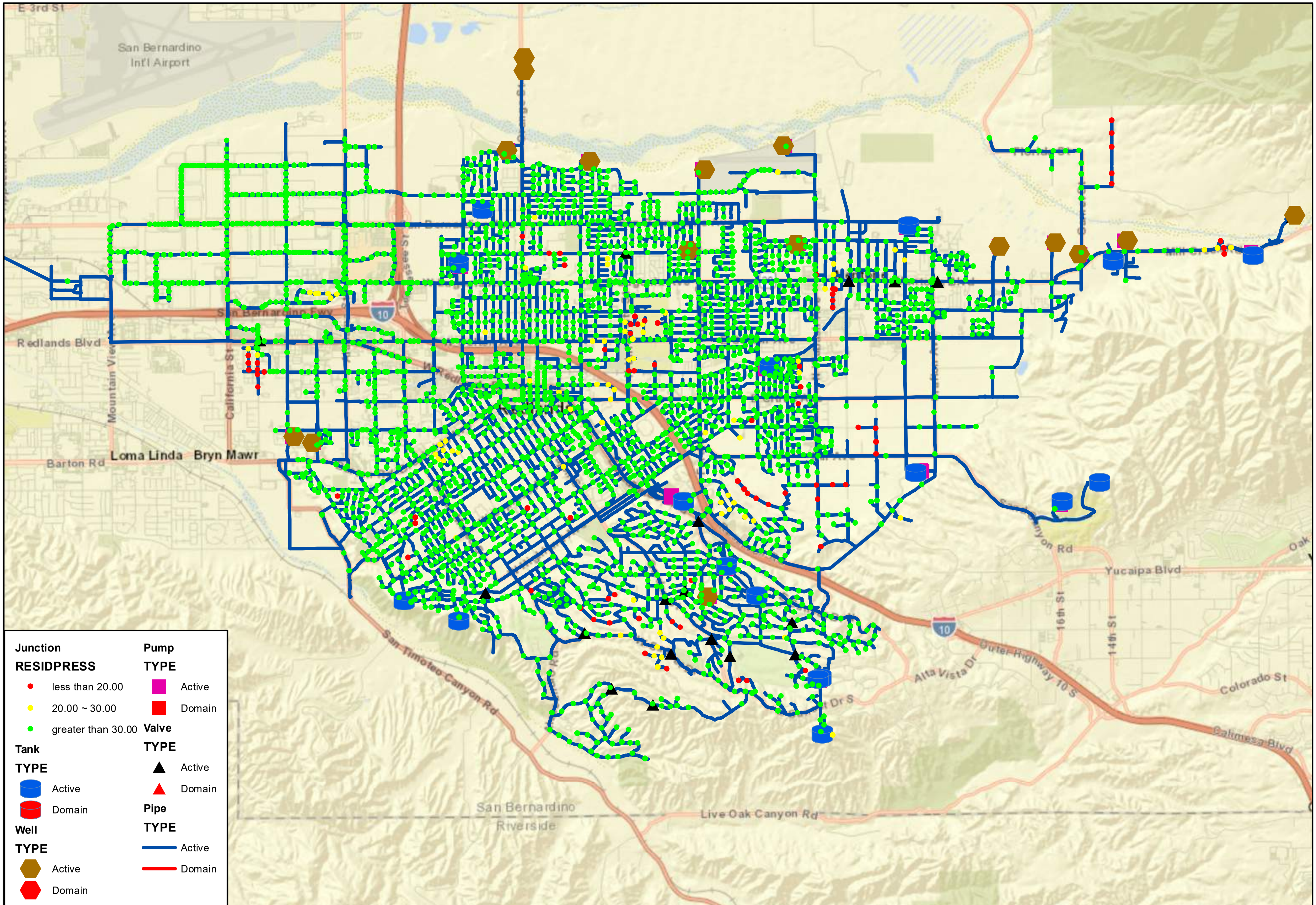

Figure 7-1
Hydraulic Capacity Evaluation
Future Demand, Existing System, MDD, Fireflow Scenario, Residual Pressure During Fire Event

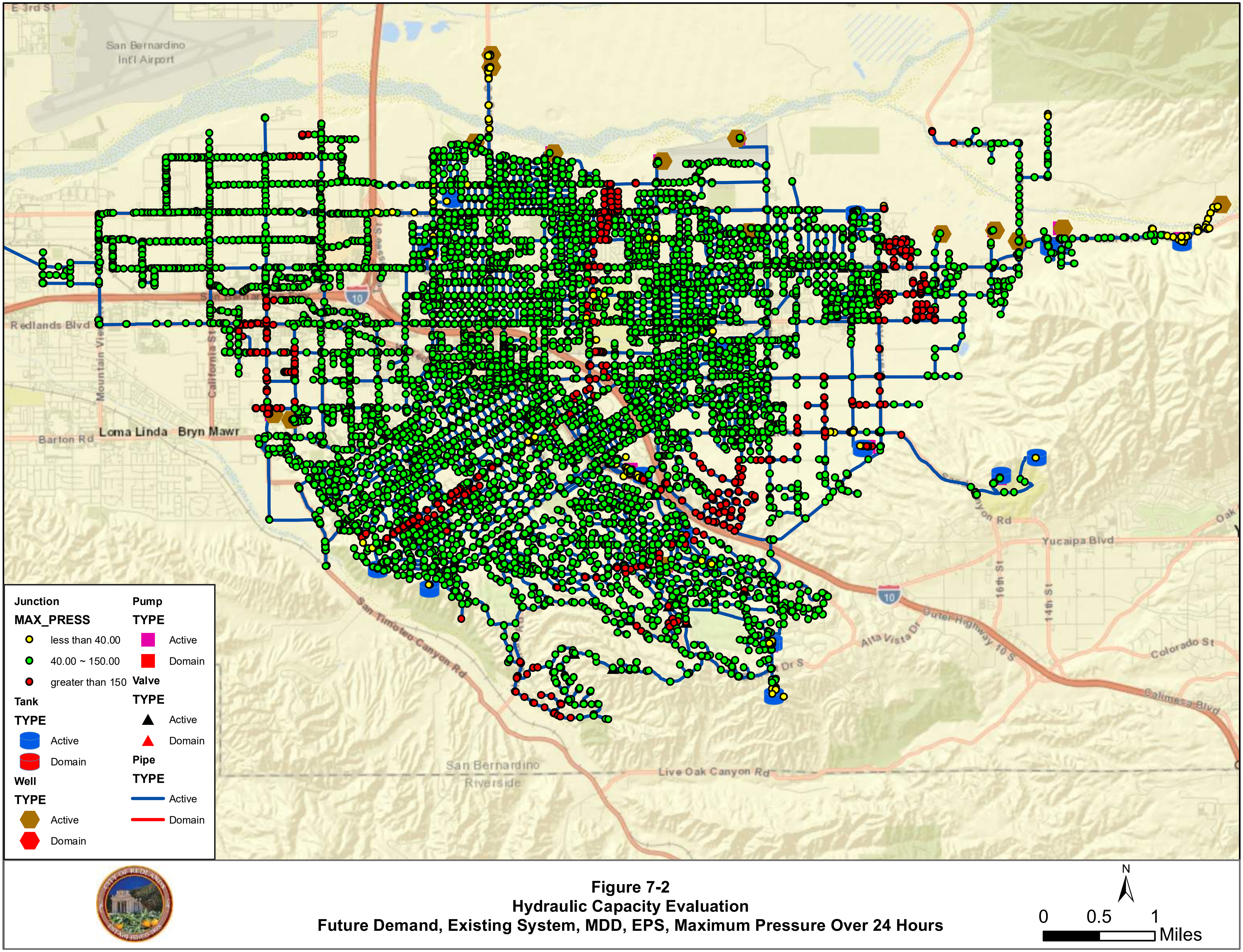

Figure 7-2
Hydraulic Capacity Evaluation
Future Demand, Existing System, MDD, EPS, Maximum Pressure Over 24 Hours

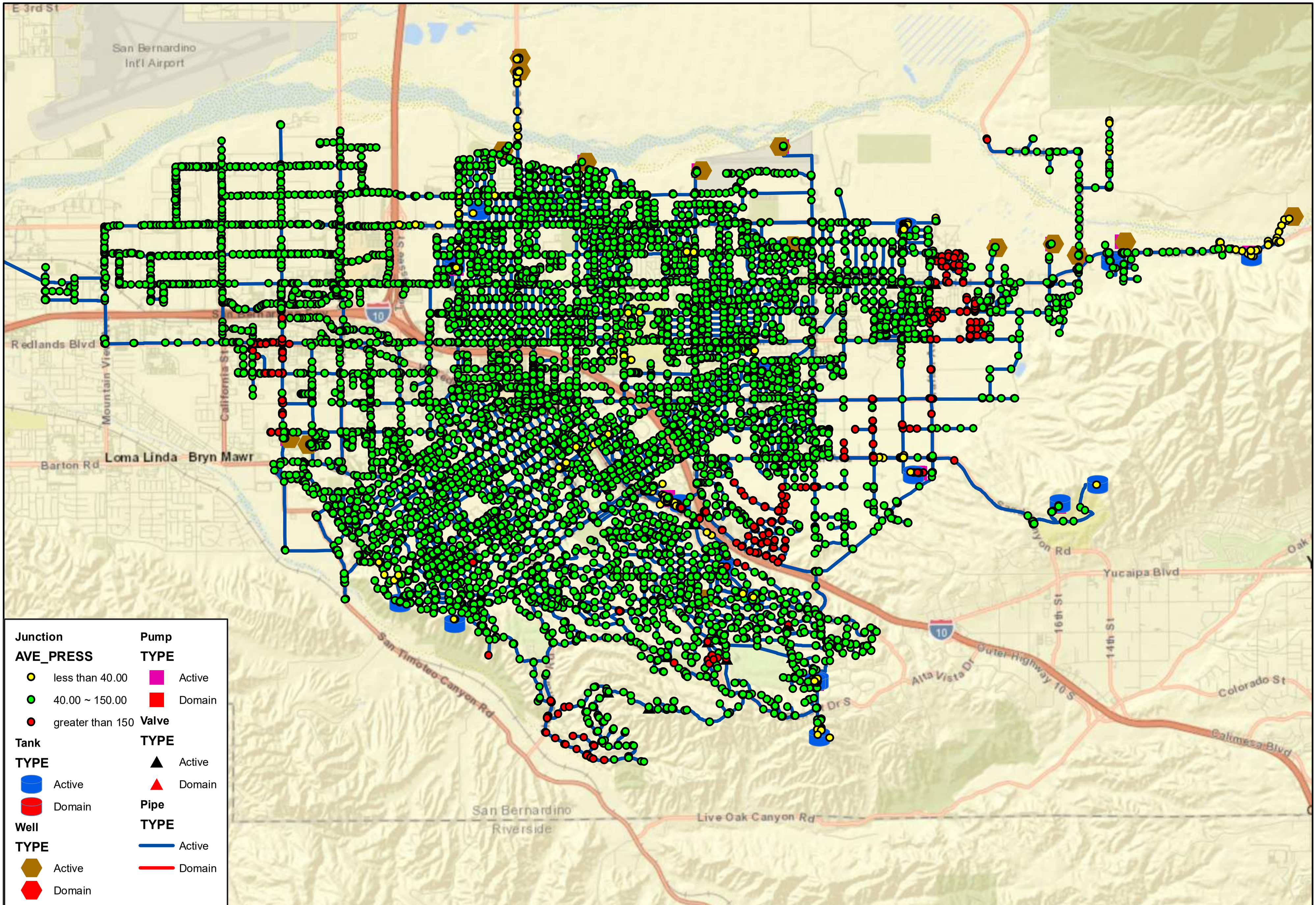

Figure 7-3
Hydraulic Capacity Evaluation
Future Demand, Future System, ADD, EPS, Maximum Pressure Over 24 Hours

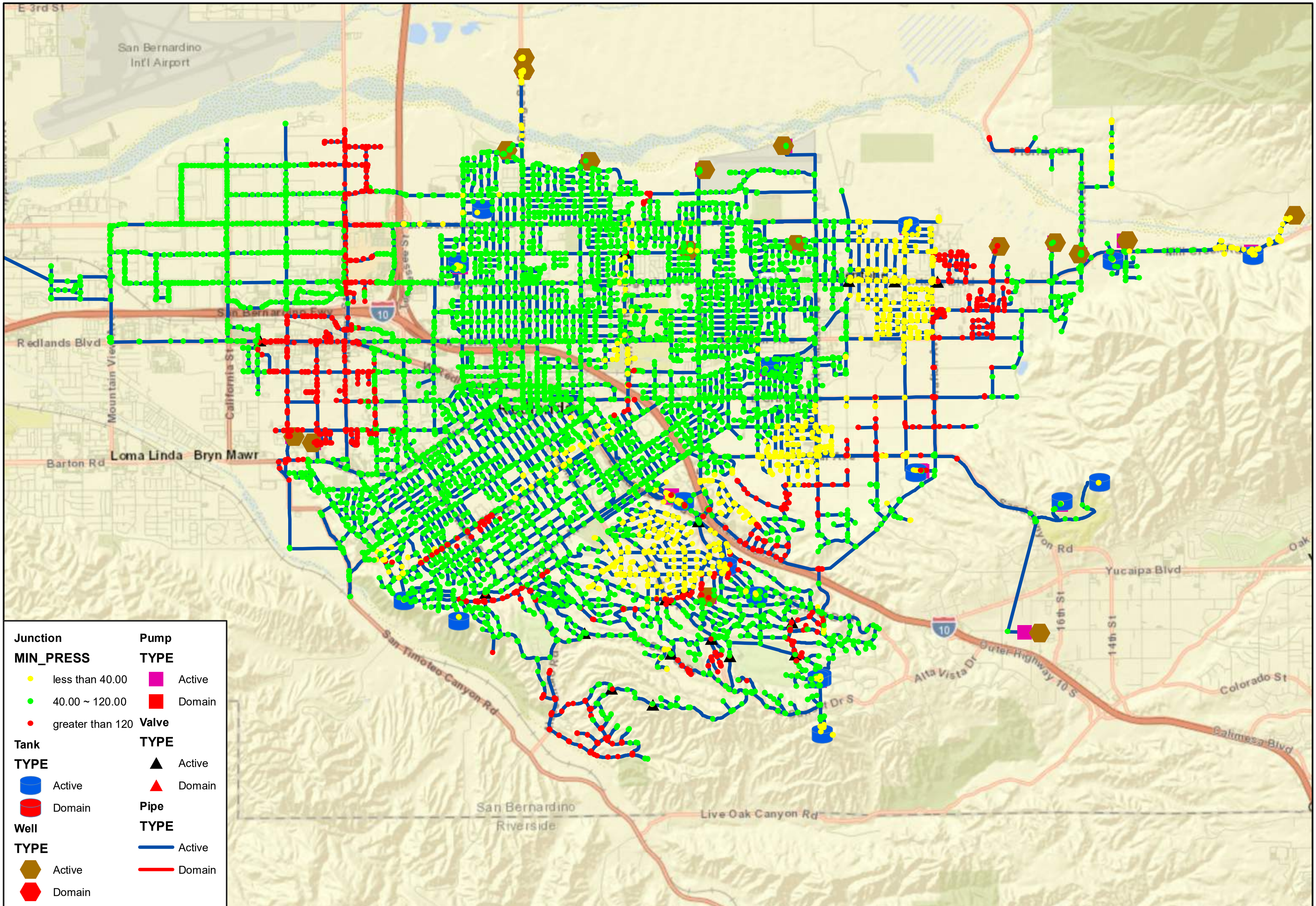

Figure 7-4
Hydraulic Capacity Evaluation
Future Demand, Future System, MDD, EPS Scenario, Minimum Pressure Over 24 Hours

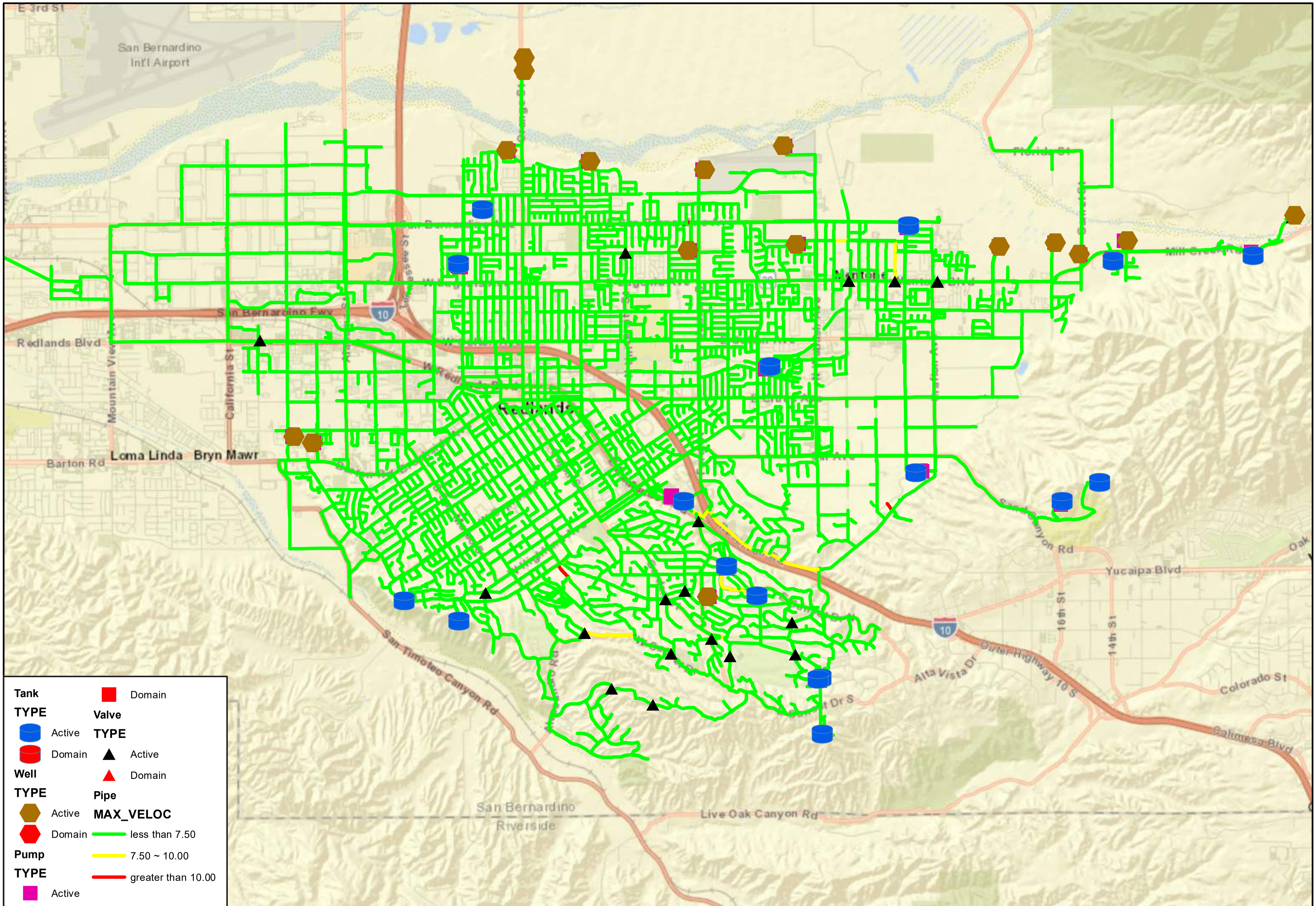

Figure 7-5
Hydraulic Capacity Evaluation
Future System, MDD, EPS Scenario, Maximum Velocity Over 24 Hours

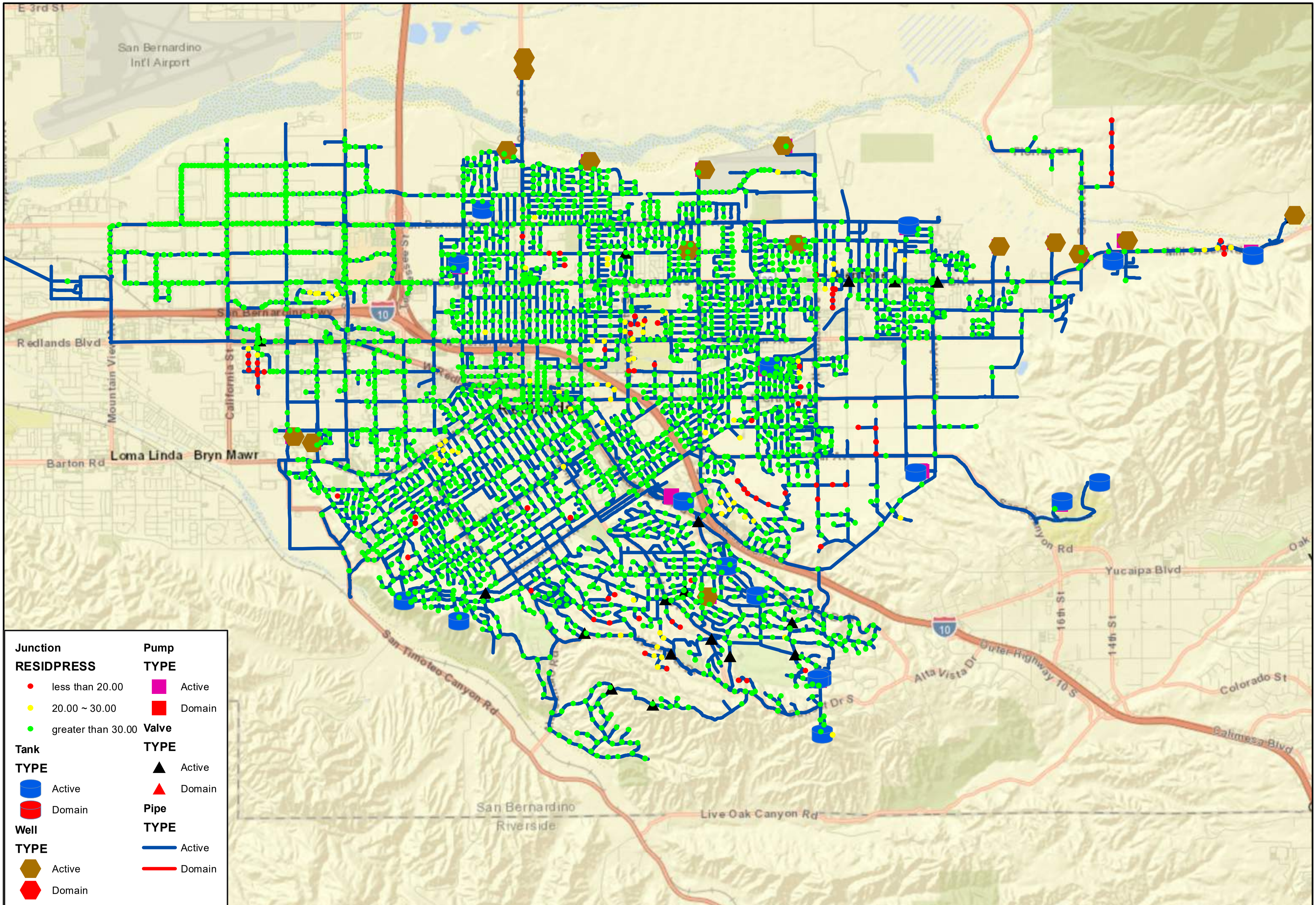

Figure 7-6
Hydraulic Capacity Evaluation
Future System, Future System, MDD, Fireflow Scenario, Residual Pressure During Fire Hours

7.3 FUTURE SUPPLY ANALYSIS – NON-POTABLE WATER SYSTEM & RECYCLED WATER SYSTEM

7.3.1 SCENARIO 1, FUTURE SYSTEM, ADD, EPS, MAX PRESSURE OVER 24 HOURS

As Figure 7-7 shows, the expanded system is within the forty (40) psi and 150 psi pressure design criteria. This will prevent loud noise and fatigue due to high water pressure, and will provide adequate water pressure to the City's customers.

7.3.2 SCENARIO 2, FUTURE SYSTEM, ADD, EPS, MAX PIPE VELOCITY OVER 24 HOURS

This scenario looks at the ADD for the expanded system to check for high velocities. As Figure 7-8 shows, the developed system will result in a few pipelines that will exceed the maximum acceptable velocity of ten (10) fps.

7.3.3 SCENARIO 3, FUTURE SYSTEM, MDD, EPS, MAX PRESSURE OVER 24 HOURS

The maximum pressure scenario was run to determine the system's pressure during the MDD. Figure 7-9 shows that most of the system will have adequate demand, with a few areas that slightly exceed the recommended pressure range of forty (40) psi and 150 psi.

7.3.4 SCENARIO 4, FUTURE SYSTEM, MDD, EPS, MAX PIPE VELOCITY OVER 24 HOURS

This scenario looks at the PHD for the expanded system to check for high velocities, and assumes a MDD peaking factor of 2.7. As Figure 7-10 shows, the developed system will result in a few pipelines that will exceed the maximum acceptable velocity of ten (10) fps. However, these pipelines are the same as those in the existing system analysis.

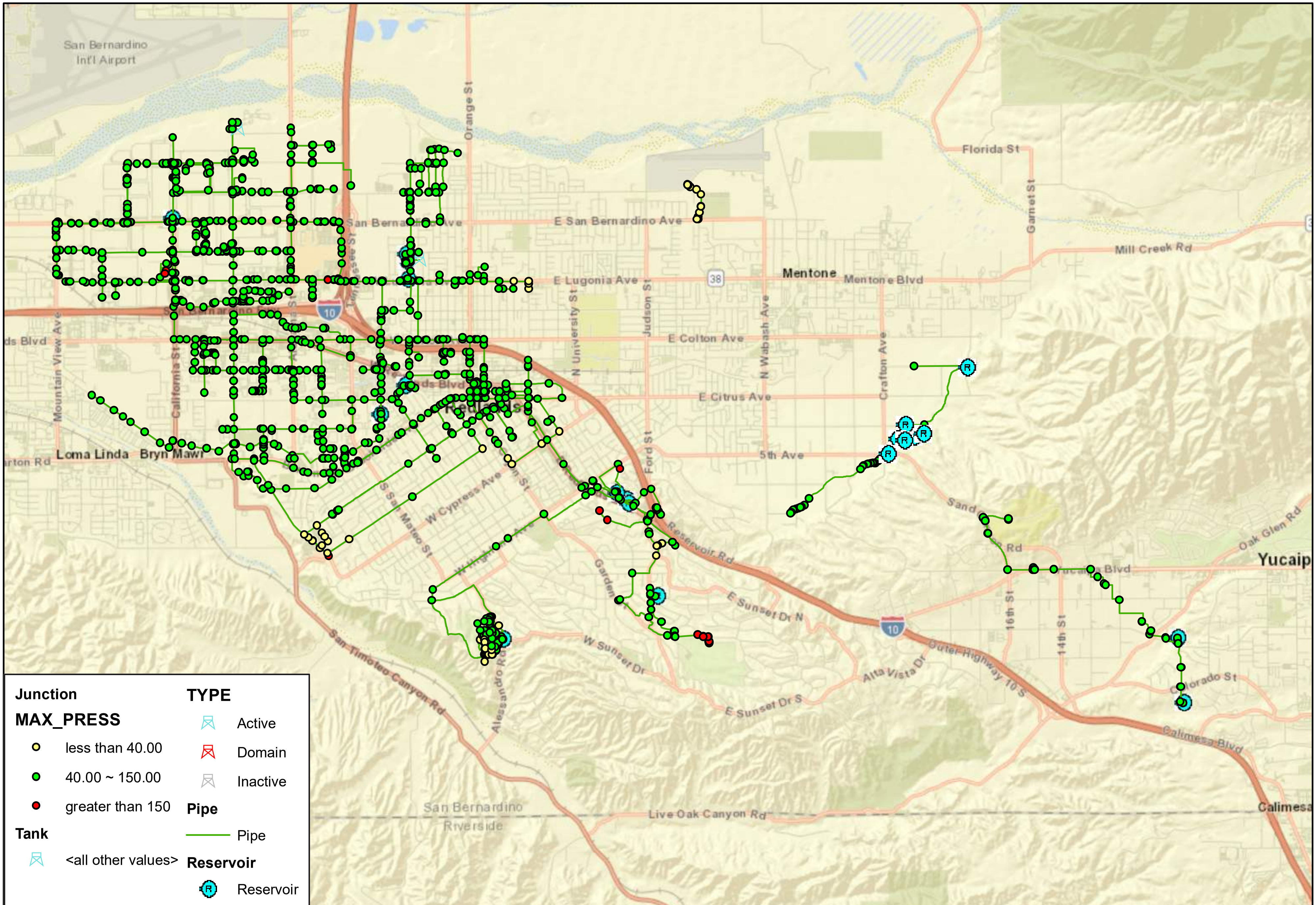

Figure 7-7
Hydraulic Capacity Evaluation
Proposed Future System, ADD, EPS, Maximum Pressure Over 24 Hours

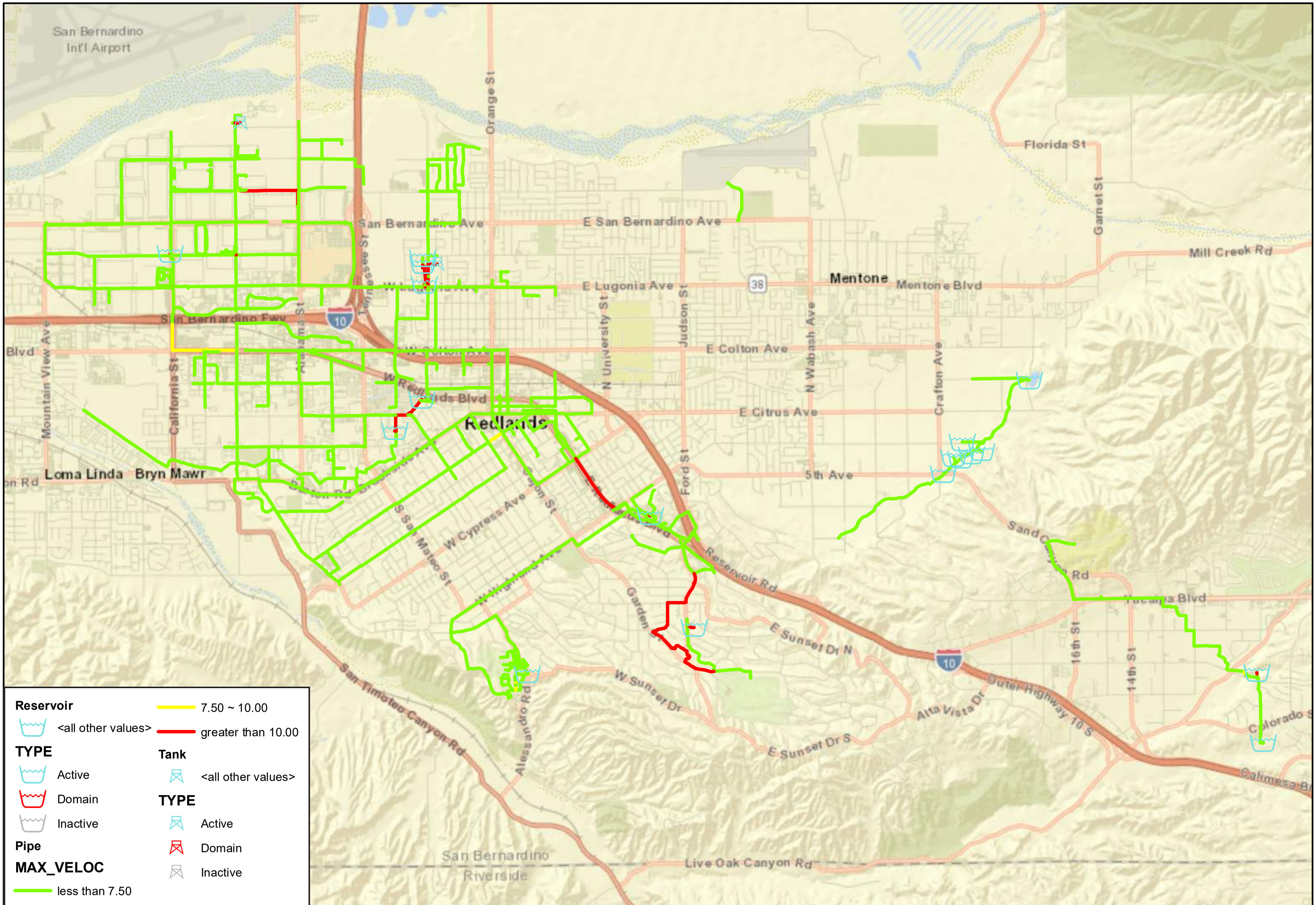

Figure 7-8
Hydraulic Capacity Evaluation
Proposed Future System, ADD, EPS, Maximum Pipe Velocity Over 24 Hours

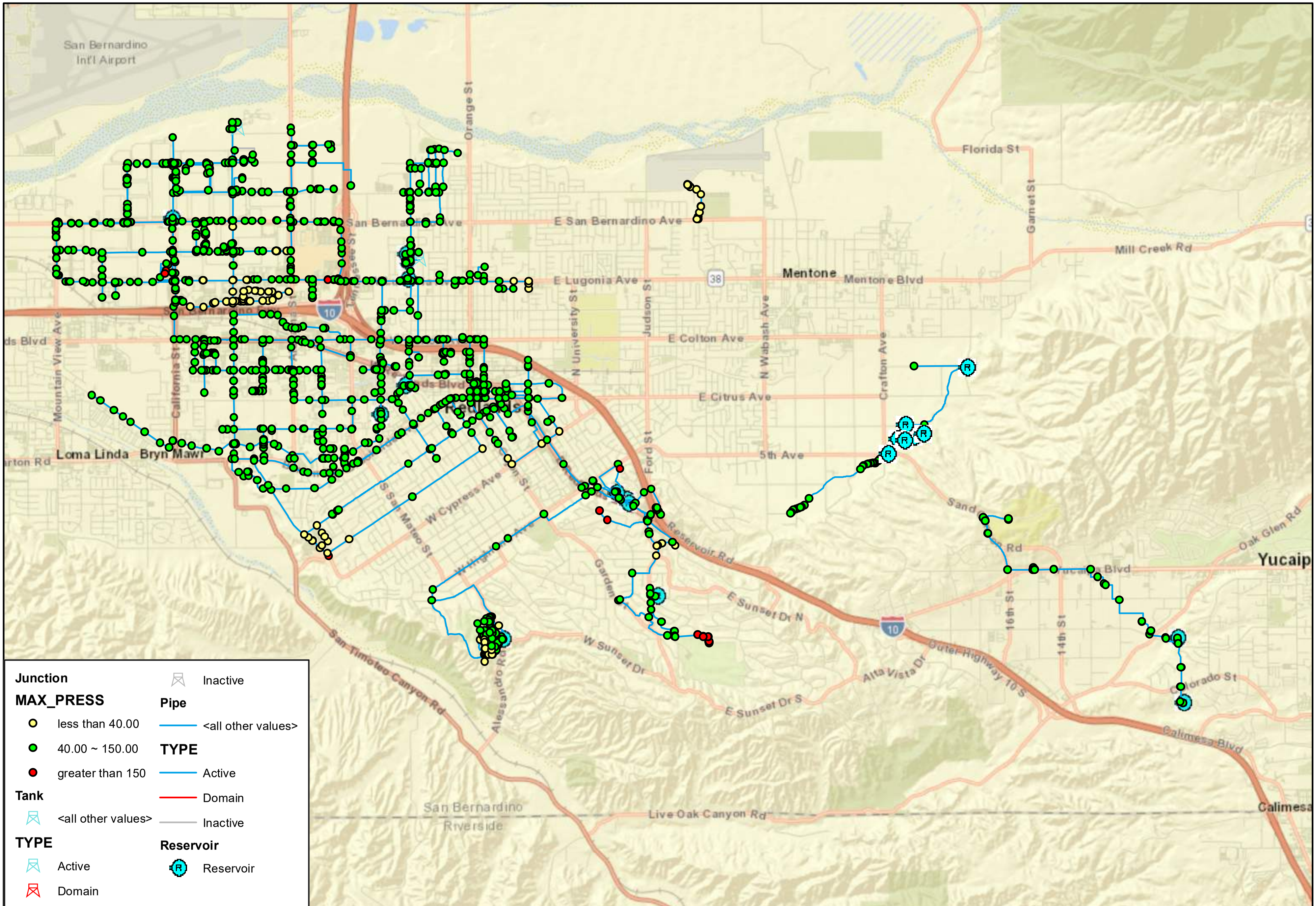

Figure 7-9
Hydraulic Capacity Evaluation
Proposed Future System, MDD, EPS, Maximum Pressure Over 24 Hours

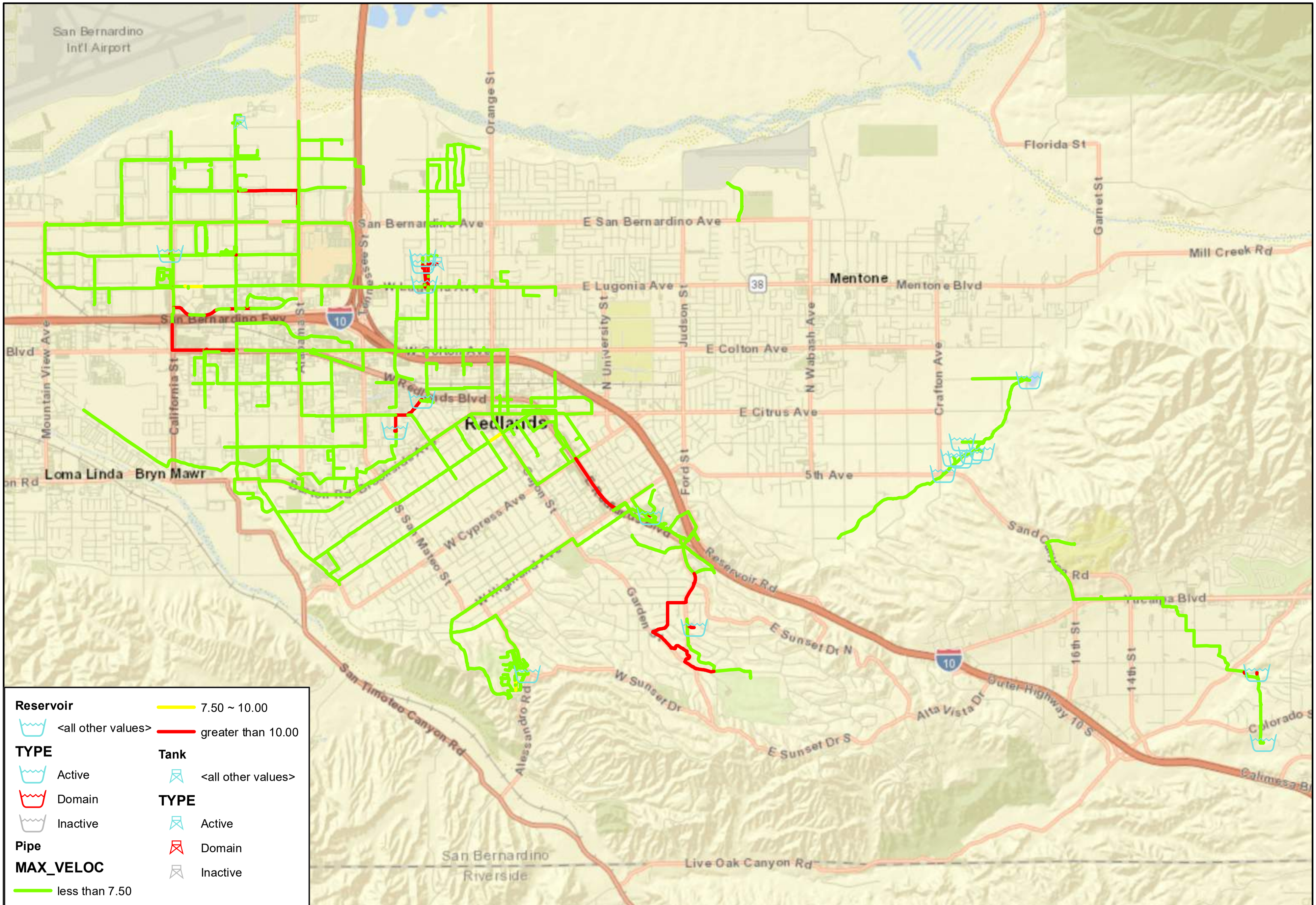

Figure 7-10
Hydraulic Capacity Evaluation
Proposed Future System, MDD, EPS, Maximum Pipe Velocity Over 24 Hours

7.4 SUMMARY OF DEFICIENCIES

7.4.1 POTABLE WATER SYSTEM

Twelve (12) potable water system deficiencies were identified, and each were recommended a CIP project to resolve the issue. These deficiencies are related to high velocities, low fire flow pressure, and undersized pipelines. The list of deficiencies discovered is shown in Table 7-1.

Table 7-1: Potable Water System Deficiencies

CIP	Location	Reason	Comments
CIP-1	San Bernardino Ave from Agate Ave to Crafton Ave	Maximum velocity > 10 fps	Replace with larger pipe
CIP-2	Mill Creek Rd near Mill Creek Reservoir	Maximum velocity > 10 fps	Dead ends removed from CIP due to no demand. Extended Pipeline replacement to Mill Creek Rd.
CIP-3	Jasper Ave from Mentone Blvd to Nice Ave	Fire Flow Pressure Issues	Looping dead-end to Nice St to fix issue.
	Opal Way from Naples Ave to Mentone Blvd		
	Naples Ave from Jaspar Ave to Opal Way		
CIP-4	Opal Ave from 6th Ave to end of Pipeline	Fire Flow Pressure Issues	Complex Project, Changing contingency of project from 30% to 40%.
	6th Ave from Opal Ave to Marion Rd		
	Marion Rd/Marvin Ave from Camelot Dr to Pleasantview Dr		
	Wabash Ave from 6th Ave to end of Pipeline		
CIP-5	Pennsylvania Ave from Lassen St to Church St	Fire Flow Pressure Issues	Replace with larger pipe
CIP-6	University Station/ Orange Blossom Trail from University St to Grove St	Pipe Diameter Size too small	Replace with larger pipe
	Cook St from University Station to Sylvan Blvd		
CIP-7	End of Pipeline from St Catherine St to Valencia Drive	Maximum velocity > 10 fps	Assumed to be a Lateral due to the pipe being a dead-end and small diameter
CIP-8	San Bernardino Ave from Nelson St to Judson St	Maximum velocity > 10 fps	Small Diameter Size causing high velocity, assumed to be error
CIP-9	Park Ave from City limits to Essex Ct	Fire Flow Pressure Issues	Fire flow was determined to be lower than assumed

CIP	Location	Reason	Comments
	Amigos Dr from Park Ave to Rancho Dr		
	Rancho Dr from end of Pipeline to New Jersey St		
	New Jersey St from Park Ave to Redlands Blvd		
CIP-10	Emerald Ave from Newport Ave to Tres Lagos St	Fire Flow Pressure Issues	Low pressure due to elevation. A subzone may be needed to fix issue
	Newport Ave from Garnet St to Emerald Ave		
CIP-11	Fairmont Dr from Sunset Dr to end of Pipeline	Fire Flow Pressure Issues	Low pressure due to elevation. A subzone may be needed to fix issue
	Manzanita Rd from end of Pipeline to Fairmont Dr		

7.4.2 NON-POTABLE WATER SYSTEM & RECYCLED WATER SYSTEM

Currently, no storage reservoirs exist within the recycled water system. The City is currently engineering two (2) 1.5 MG recycled water reservoirs for future construction at the WWTP. Additional system storage will be necessary as demand increases. Also, the detached non-potable water systems are not efficient and should be connected in the future. Doing so allows additional connections to be transitioned to the non-potable water system, which will reduce potable water system demand.

7.5 MITIGATION OF DEFICIENCIES

7.5.1 METHODOLOGY

The primary method of correcting high velocity or low pressure associated with fire flow demand is to increase the pipe diameter or loop dead ends, where possible. The issues found in the potable water system modeling can be corrected by increasing pipe diameters or looping in some areas. The non-potable water system and recycled water system can be improved by adding storage capacity and joining disconnected systems.

7.5.2 PROJECTS FOR FUTURE CONDITIONS – POTABLE WATER SYSTEM

Deficient pipeline segments are listed in Table 7-2 along with the recommended replacement pipeline diameter, the segment length, and the estimated cost for each. The cost estimate includes labor, equipment, materials, for each project, and a budget for unanticipated construction issues. The approach to developing cost estimates is explained in Section 10. The total length of all projects is five (5) miles and is estimated to cost approximately \$3.7M. Figure 7-11 shows the locations of each deficient pipeline segment. Figure 7-12 and Figure 7-13 show the anticipated system hydraulic improvements if all deficiencies are resolved.

Table 7-2: Recommended CIP to Correct Deficiencies

CIP	New Diameter (in)	Replacement Material	Length (LF)	Cost Estimate
CIP-1	24	DIP	2046	$654,720

CIP	New Diameter (in)	Replacement Material	Length (LF)	Cost Estimate
CIP-2	8	DIP	203	$33,000
CIP-3	8	DIP	649	$104,000
CIP-4	12	DIP	11979	$2,395,800
CIP-5	8	DIP	1142	$183,000
CIP-6	8	DIP	1569	$251,000
CIP-7	N/A	DIP	TBD	TBD
CIP-8	N/A	DIP	TBD	TBD
CIP-9	N/A	DIP	TBD	TBD
CIP-10	N/A	DIP	TBD	TBD
CIP-11	N/A	DIP	TBD	TBD
Totals			**27,193**	**$ 3,621,520**

7.5.3 PROJECTS FOR FUTURE CONDITIONS – NON-POTABLE WATER SYSTEM & RECYCLED WATER SYSTEM

Currently, no storage reservoirs exist within the recycled water system. The City is currently engineering two (2) 1.5 MG recycled water reservoirs for future construction at the WWTP. It is recommended that another reservoir be constructed in Zone 1. The specific location for this reservoir requires additional analysis. It is recommended that the minimum storage capacity for the expanded system be three (3) MG.

It is also recommended that detached systems be connected where practical. Connecting all detached systems would require approximately sixty (60) miles of eight inch (8”) diameter pipelines, and a small amount of twelve inch (12”) pipelines, throughout the west and southwest portions of the City. Connecting the Redlands Country Club, Hillside Memorial Park, and Redlands Dog Park systems would allow additional connection transitions to the Pressure System 2 of the non-potable water system, potentially reducing potable water system demand by 0.9 MGD.

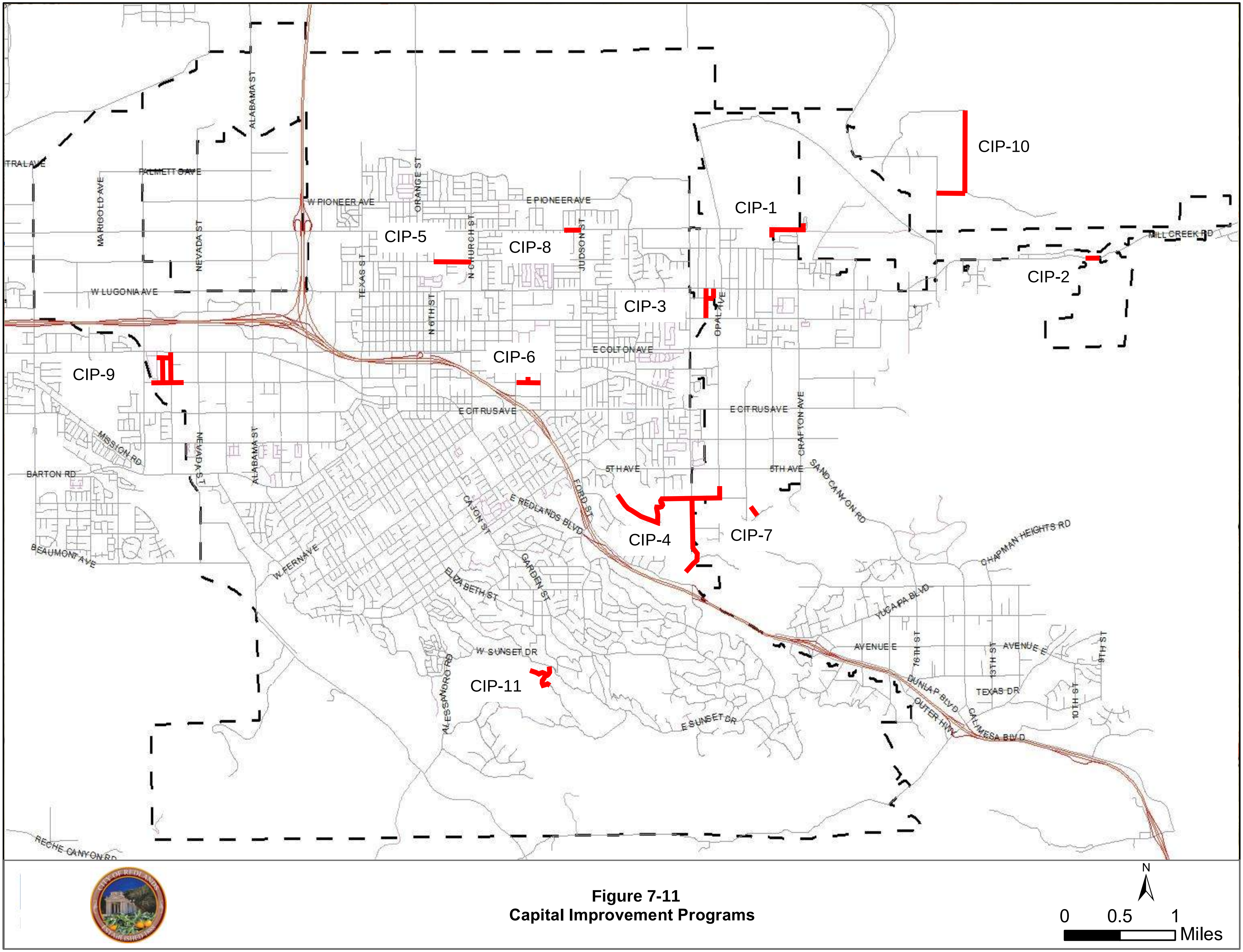

Figure 7-11
Capital Improvement Programs

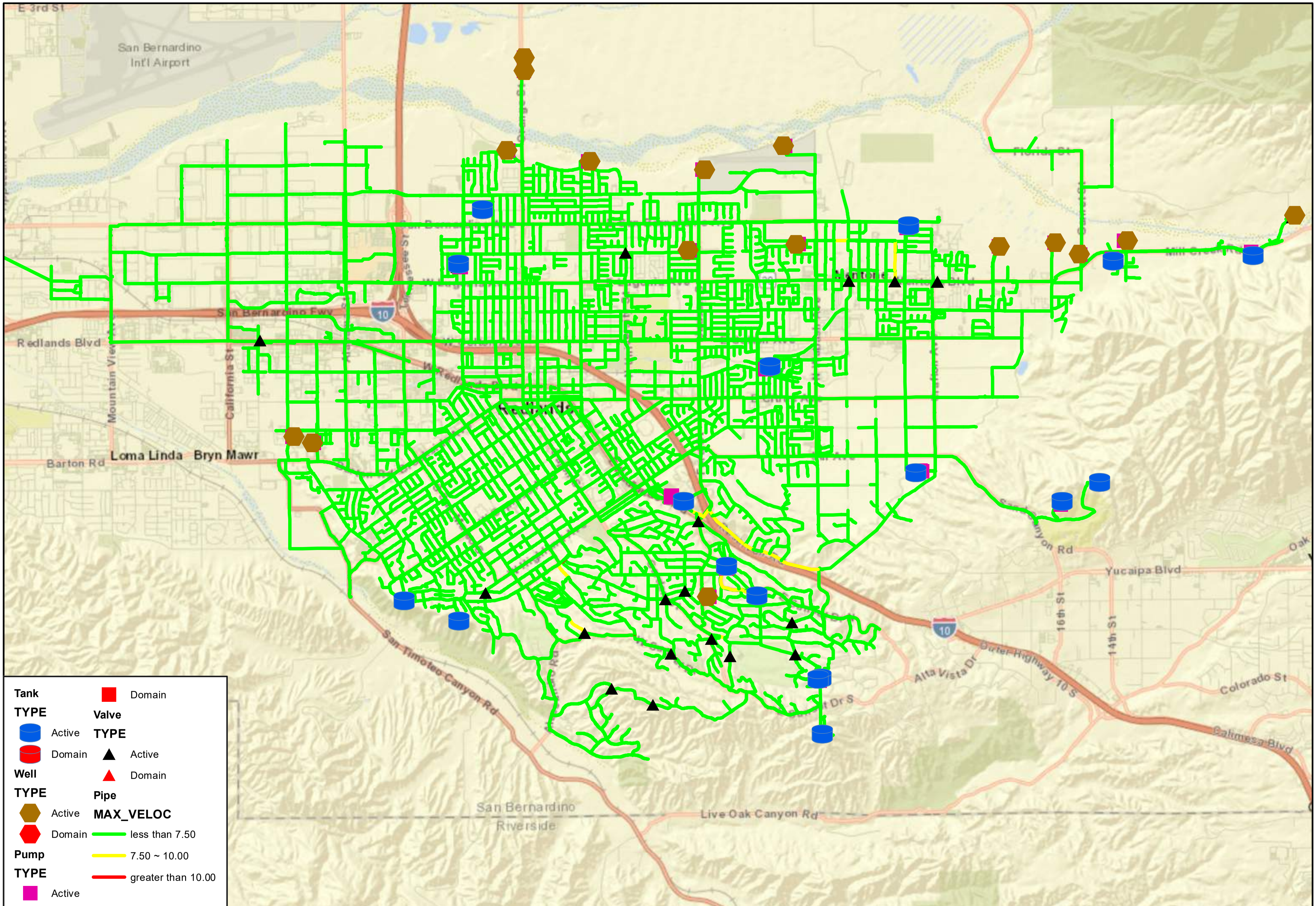

Figure 7-12
Capital Improvement Programs
Future System, MDD, EPS Scenario, Maximum Velocity Over 24 Hours

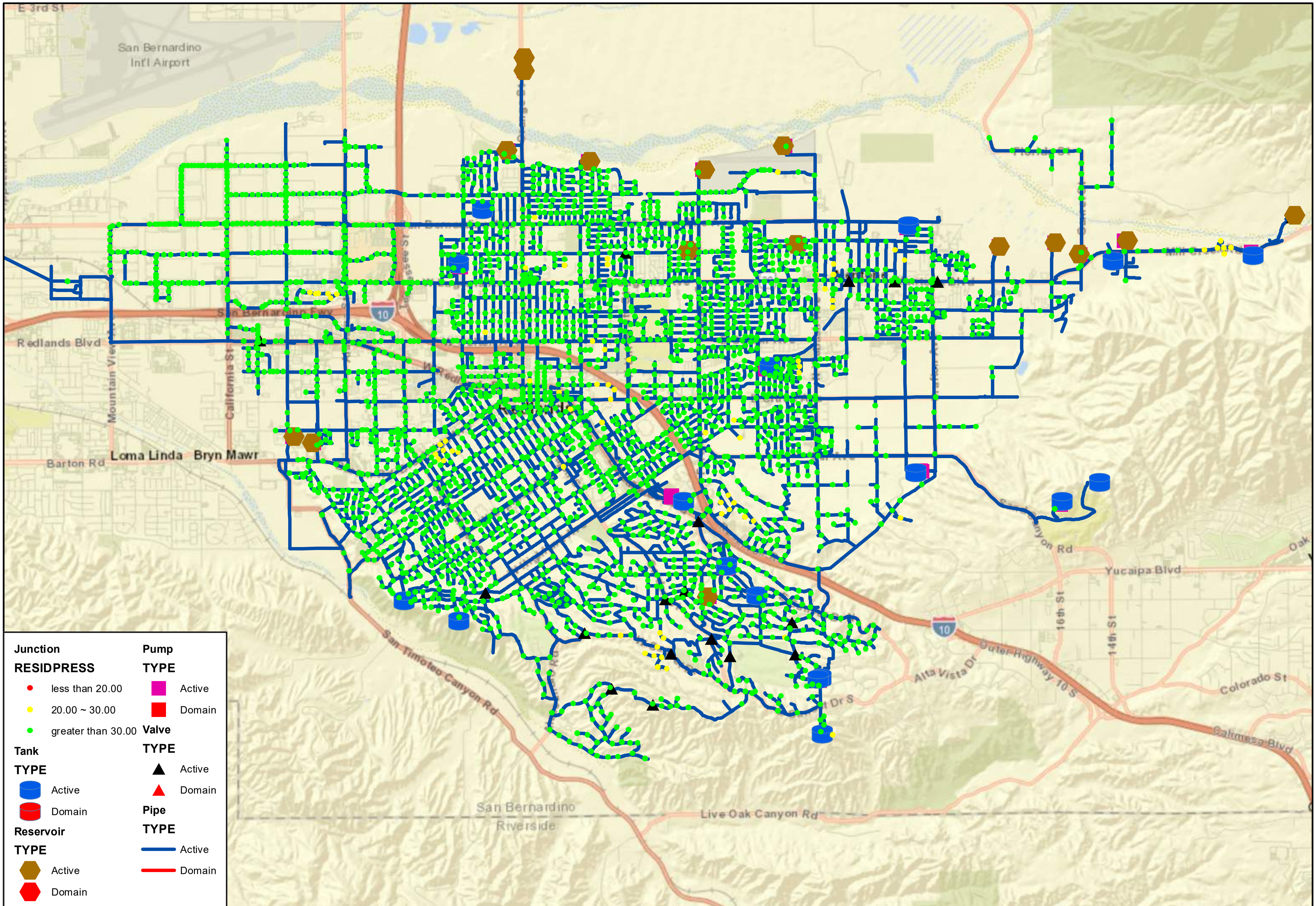

Figure 7-13
Capital Improvement Programs,
Future System, Future System, MDD, Fireflow Scenario, Residual Pressure During Fire Hours

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8 WATER QUALITY

8.1 METHODOLOGY

A common water quality concern in potable water distribution systems is water age. Water quality degrades over time and the loss of disinfection residual often leads to customer taste and odor complaints. Though not as strict as the requirements for potable water, the non-potable and recycled water systems must meet specific state and federal regulatory requirements for irrigation, agricultural, and commercial/industrial uses. Due to the lack of storage reservoirs in the existing non-potable water system and recycled water system, water age is not a good indicator of water quality, and should be analyzed when reservoirs are added to the systems.

8.2 REGULATORY REQUIREMENTS

The State of California and the U.S. EPA require potable water to meet rigid water quality standards prior to distribution. The City obtains water from surface and groundwater sources. Groundwater sources typically contain more microbial contaminants, inorganic contaminants, pesticides, herbicides, and industrial chemical byproducts. As a result, wells are closely monitored to make sure that the water being pumped from the Bunker Hill groundwater basin meets all regulatory requirements.

Recycled water regulations are identified in the California Code of Regulations 2018 Title 22 – Division 4 Chapter 3. Because the WWTP process includes tertiary treatment and disinfection, the following recycled water common uses are allowed:

1. Irrigation of food crops, including all edible root crops, where the recycled water comes into contact with the edible portion of the crop;
2. Irrigation of parks and playgrounds;
3. Irrigation of school yards;
4. Irrigation of residential landscaping;
5. Irrigation of golf courses;
6. Any other irrigation uses not specified in this section and not prohibited by different areas of the California Code of Regulations.

Tertiary water is also allowed for industrial and commercial cooling, and is used at the SCE Mountain View Power Plant. Groundwater extracted by the non-potable water wells is typically classified as un-disinfected secondary water, and can be used for surface irrigation. However, more rigid water quality standards may apply when this groundwater may be reasonably expected to contact the edible portion of food crops.

8.3 GOALS AND PREFERENCES

The City must not exceed potable water MCL established by the state and federal regulatory agencies for coliform bacteria, turbidity, metals like aluminum and lead, nitrates, fluoride, and other contaminants. State and federal regulatory agencies also establish Primary Drinking Water Standards to regulate other pollutants. Finally, the U.S. EPA has established secondary standards for drinking water aesthetics. The City has not violated any of these regulations or standards. Disinfected tertiary recycled water standards are identified in the State of California Code of Regulations.

8.4 ASBESTOS

The City water distribution systems include approximately 200 miles of asbestos cement pipelines, which can release asbestos fibers into the water supply if the pipeline has deteriorated. Asbestos is a known carcinogen that can cause breathing problems, lung cancer, Mesothelioma, and a host of other health problems when certain exposure conditions exist. State regulatory agencies have established a limit of seven (7) million asbestos fibers per liter of water. It is recommended that these pipelines are evaluated for replacement if they exceed their service life or are found to have integrity issues that could cause asbestos to enter the water system.

8.5 GROUNDWATER QUALITY

The Upper Santa Ana Valley Groundwater Basin is an alluvial groundwater basin fed by multiple tributaries, including the Santa Ana River and Mill Creek, which are located within the City's water service area. The Bunker Hill Sub-basin, also known as the San Bernardino Basin Area (SBBA), lies within a portion of the Upper Santa Ana Valley Groundwater Basin and has a surface area of approximately 89,600 acres and a groundwater storage capacity of 5,976,000 acre-feet.

8.5.1 BUNKER HILL SUBBASIN/SBB

Based on the State's Department of Water Resources, *Upper Santa Ana Valley Groundwater Basin, Bunker Hill Sub-basin Report*, the SBBA is part of the northeastern portion of the Upper Santa Ana Valley Basin. The Santa Ana River, Mill Creek, and Lytle Creek are the main tributaries for this sub-basin, which is bordered by the San Gabriel Mountains, San Bernardino Mountains, and Crafton Hills. The Banning fault, Redlands Fault, San Andrea Fault, San Jacinto Fault, and the Glen Helen fault are located within the SBBA as well. The total dissolved solids (TDS) of the SBBA groundwater is typically 155 mg/l - 1,140 mg/l, and some portions of the SBBA frequently exceed MCL standards for nitrates, perchlorates, volatile organic compounds (VOC), and Synthetic Organic Compounds (SOC).

The City is located in the southern portion of the SBBA, and extracts SBBA groundwater for potable water and non-potable water production. SBBA water meeting state and federal water quality standards does not need additional treatment or disinfection for potable use. Alternatively, extracted groundwater exceeding nitrate or perchlorate MCLs may be distributed for non-potable use.

8.6 SOURCE WATER QUALITY

When available, the Hinckley and Tate raw water influent may be supplemented with SWP water for treatment and potable water distribution. The City has ownership in various private and mutual water companies to supply water to the City's Tate and Hinckley WTP. SWP water is not always available, and is only used as a last resort. SWP water often includes organics and sediment that are difficult to treat, so it is typically blended with raw water influent from other sources prior to treatment.

The quality of non-potable water varies depending on the extraction location. Groundwater extracted from one (1) location may be blended with groundwater extracted from another location to improve water quality within the non-potable water distribution system.

The City WWTP typically produces recycled water with less than five (5) mg/l BOD, five (5) mg/l TSS, ten (10) mg/l total nitrogen, and 0.2 NTU turbidity. This water can be blended with non-potable water to further improve water quality. Constructing recycled water storage reservoirs will create additional opportunities to improve water quality.

8.7 DISINFECTION

The City disinfects water with Sodium Hypochlorite and chlorine gas to prevent microbial contaminant migration into the potable water system. The disinfectant residual typically ranges from 0.8 mg/l -1.2 mg/l.

8.8 WATER AGE

The water age in water supply systems depends primarily on maintaining an efficient demand and supply balance through the use of SCADA, providing adequate water storage volume, and operation and maintenance practices. The potable water system is designed to provide storage reserve to support emergency relief efforts such as fire suppression and earthquake damage.

The Water Industry Database (AWWA and AWWA RF 1992) indicates an average potable water distribution system retention time of 1.3 days and a maximum retention time of twenty-four (24) days based on a survey of more than 800 U.S. utilities. In addition, the literature cites examples of both "short" (less than three days) and "long" (greater than three days) water ages. Several water age recommendations published in the literature are summarized below in Table 8-1.

Table 8-1: Water Age Recommendations

Population Served	Water Main Length (mi)	Water Ages (Days)	Method of Determination
800,000	2,750	3 to 7	Hydraulic Model
300,000	1,100	1 to 3	Fluoride tracer
80,000	358	More than 16	Chloramine Conversion
24,000	86	12 to 24	Hydraulic Model
Note: Source is USEPA Effects of Water Age on Distribution System Water Quality, August 15, 2002			

Several smaller cities with populations between 50,000 - 100,000 report water age of three (3) to sixteen (16).

8.9 WATER AGE POTENTIAL HEALTH IMPACTS

Potential health impacts associated with the water age-related chemical and biological issues are identified in Table 8-2.

Table 8-2: Potential Water Age Issues

Chemical Issues	Biological Issues	Physical Issues
Disinfection By-Product Formation	Disinfection By-Product Biodegradation	Temperature Increase
Disinfectant Decay	Nitrification	Sediment Deposition
Corrosion Control Effectiveness	Microbial Regrowth/Recovery/Shielding	Color
Taste and Odor	Taste and Odor	
Note: (1) The source is USEPA Effects of Water Age on Distribution System Water Quality, August 15, 2002; (2) The Chemical Health Effects Tables (U.S. EPA, 2002a) summarizes potential adverse health effects from high/long-term exposure to hazardous chemicals in drinking water.		

The Microbial Health Effects Tables (U.S. EPA, 2002b) summarizes potential health effects from exposure to waterborne pathogens, the most concerning of which are the formation of disinfection by-products, presence of haloacedic acid (HAA5) and trihalomethanes (TTHM), and nitrification and microbial regrowth after disinfectant depletion.

8.10 WATER AGE DETERMINATION

Water age can be predicted by conducting hydraulic modeling and analysis, mathematical modeling and fluid dynamics computations, and tracer studies using fluoride, sodium chloride, calcium chloride, lithium chloride, pulsed chlorine, and coagulants. Mathematical modeling is arguably the least accurate of these methods.

8.11 CITY OF REDLANDS WATER SUPPLY SYSTEM AGE

An ADD potable water age analysis was conducted using the hydraulic model, as shown in Figure 8-1, and indicated typical water age is three (3) days, and increases to twelve (12) days at the edges of the water system and in the potable water storage reservoirs. More specifically, water age within the potable water system distribution system piping is typically one to three (1 – 3) days, and storage reservoir water age is typically seven to twelve (7 – 12) days.

Water age is primarily a function of the system size, operation, SCADA automation, and design. As water demand increases, the amount of residence time in the distribution system decreases. Demand is related to land use patterns, types of commercial-industrial activity present in a community, weather, the general living conditions (pandemic, work from home, etc.), and community water use habits (water conservation practices, reuse practices, etc.). Cities with effective water conservation programs typically experience greater water age when all other factors are constant, due to reduced demands.

The water age analysis, water quality data review, and site visits did not indicate water quality issues within the City potable water supply system. This finding is consistent with the annual Consumer Confidence Report (CCR) prepared and distributed to all potable water customers to provide specific water quality characteristics. The CCR complies with state and federal regulatory agency requirements. The installation of mechanical mixers in potable water storage reservoirs is recommended to prevent water quality issues related to longer retention times. Table 8-3 summarizes the storage reservoir hydraulic model water age evaluation.

Table 8-3: Storage Reservoir Water Age

Reservoir	Percent Full (%)	Level (ft)	Water Age (days)
5th Ave	53	11.92	11.3
Agate	72	18.04	7.0
Arroyo	59	23.74	11.0
Country Club Reservoirs	66	12.6	11.1
Crafton	19	5.63	11.7
Dearborn	31	7.9	11.7
Highland	68	18.91	11.3
Margarita	85	11.96	11.7
Mill Creek	58	11.57	11.2
Sand Canyon	29	11.37	11.7
Smiley	32	7.12	10.1
South Ave	75	19.53	10.7
Sunset	39	24.5	11.6

Reservoir	Percent Full (%)	Level (ft)	Water Age (days)
Texas Grove	77	15.32	11.4
Texas St	84	29.45	11.5
Ward Way Reservoirs	51	15.92	11.7

8.12 INDICATORS OF HIGH WATER AGE

Aesthetic issues such as discoloration, poor taste, and noxious odors may be caused by water age, deteriorating pipeline materials, treatment and disinfection practices, and turbidity. No aesthetic issues were noted during the site visits and contacts with the Operations staff.

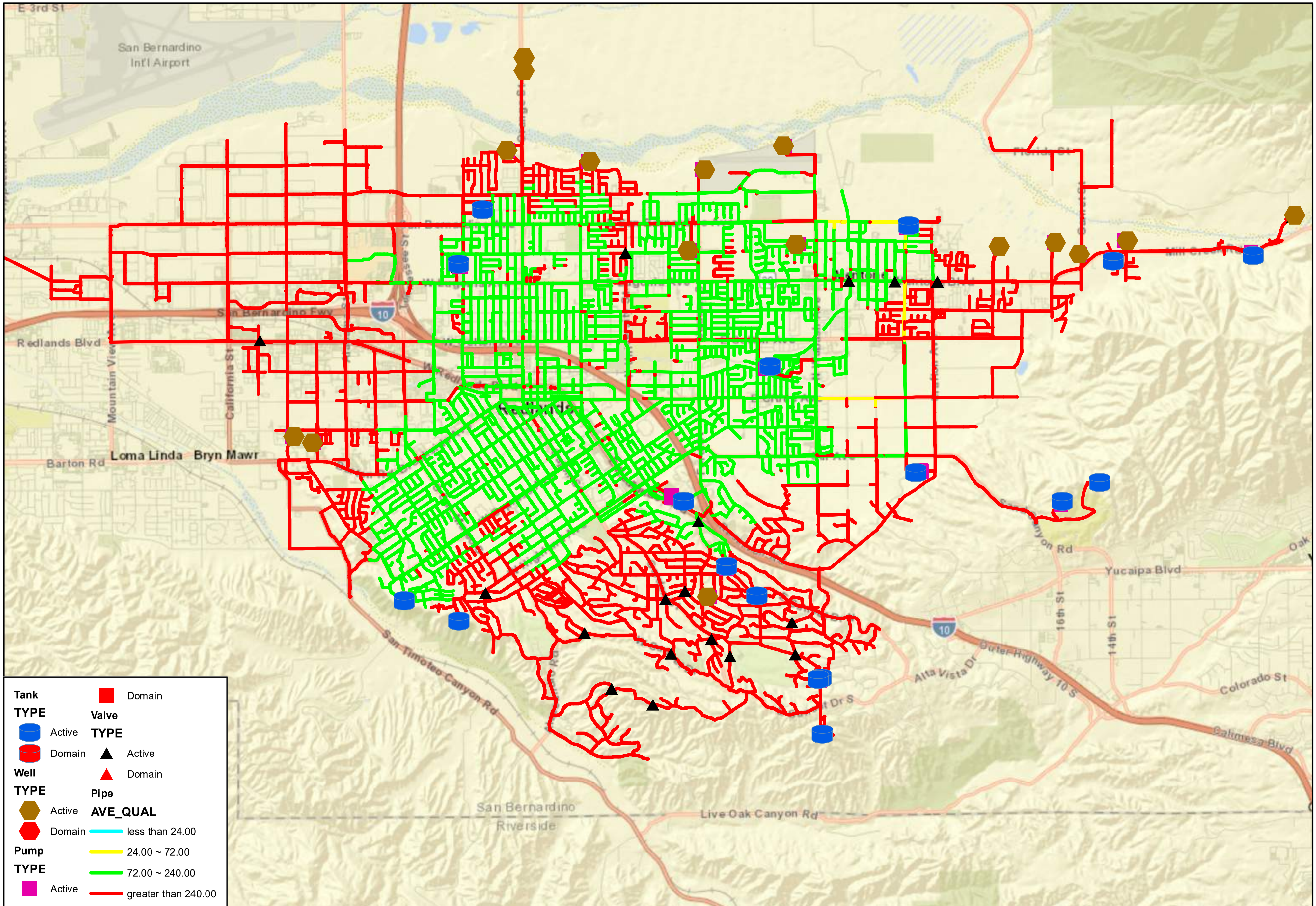

Figure 8-1
Hydraulic Capacity Evaluation
Existing System, ADD, Water Age Analysis Scenario WQ- A

9 WATER SYSTEM OPERATION AND MAINTENANCE

9.1 GENERAL DESCRIPTION

Operations and maintenance practices for all water systems were reviewed to develop improvement recommendations. This included a review of available records and operating procedures during several site visits and MUED staff workshops.

9.2 SCADA

Water Production Operators monitor and operate the water system through a comprehensive SCADA system. SCADA is used to gather and analyze real-time or near real-time sensor data, which is used for monitoring treatment plants, transmission, and distribution processes. The system monitors pressure and flow, storage reservoir levels, and treatment processes. SCADA is also useful for collecting regulatory reporting data, obtaining operational information for planning purposes, optimizing energy and chemical use, identifying water loss, and improving various maintenance practices. The City is developing Standard Operating Procedures (SOP) for the operation, security, and long-term maintenance of the SCADA system.

9.3 WORK ORDER PROCESS

The City is currently implementing an electronic asset management system to improve work order processing efficiency. This system populates a database that will be used to accurately determine operation and maintenance resource needs and associated costs, including labor, equipment, materials, price, date, and location tracking.

9.4 INSPECTION AND MAINTENANCE

The MUED staff inspects water production and distribution facilities routinely and has inspection and maintenance protocols in place for treatment plants, storage reservoirs, wells, booster pumps, and other equipment. Preventive maintenance activities are conducted in accordance with manufacturer recommendations. The CIP project recommendations include regularly scheduled mechanical and electrical equipment maintenance and replacement, storage reservoir inspection and maintenance, and other routine practices to ensure the water systems continue to function efficiently.

9.5 WATER SYSTEM STAFF AND MANAGEMENT

The MUED organization includes Water Distribution (WD), Water Production (WP), and WWTP Divisions to operate and maintain the water systems. The Utilities Operations Manager oversees all water and wastewater production, distribution, operation, and maintenance practices, and is supported by a Regulatory Compliance Officer and an Administrative Assistant.

The WD Division includes Water Distribution System Operators and Field Technicians. The Field Technicians are primarily responsible for servicing and reading water meters for water utility billing. Most water meters are read manually or with an Automated Meter Reader (AMR) system, both of which are labor intensive. This group includes the following staff positions:

- Senior Customer Service Field Technician (1)
- Customer Service Field Technician (1)
- Meter Readers (3)

The Water Distribution Operators maintain the water distribution systems. This group includes the following staff positions:

- Water Distribution Superintendent (1)
- Cross Connection Control Inspector (1)
- Water Distribution Supervisor (1)
- Water Distribution Crew Leaders (4)
- Water Distribution Operators (12)

The Water Production Division operates and maintains all water production facilities, including WTP, storage reservoirs, groundwater wells, pressure reducing valves, and booster pumps. This group includes the following staff positions:

- Water Production Superintendent (1)
- Administrative Assistant (1)
- Water Production Supervisor (1)
- Water Maintenance Supervisor (1)
- Maintenance Foreperson (1)
- Water Treatment Operator (8)
- Water Quality Technician (3)
- Plant Mechanic (3)
- Maintenance Worker (3)
- Electrical & Instrumentation Technician (2)

The WWTP Division is responsible for recycled water production from the City WWTP effluent. This group includes the following staff positions:

- Wastewater Operations Superintendent (1)
- Administrative Assistant (1)
- Wastewater Operations Supervisor (1)
- Wastewater Collections Supervisor (1)
- Maintenance Foreperson (1)
- Wastewater Facilities Operator (6)
- Plant Mechanic (3)
- Maintenance Worker (2)
- Line Maintenance Worker (7)
- Laboratory Manager (1)
- Laboratory Analyst (4)

MUED staffing for operation and maintenance of the potable water system, non-potable water system, and recycled water system is appropriate for the size and complexity of each system, and meets general guidance provided in the U.S. EPA Public Water System Classification and Staffing Requirements and AWWA M5 manual.

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10 CAPITAL IMPROVEMENT PROGRAM

10.1 GENERAL DESCRIPTION

This section presents the proposed capital improvements for the next five (5) years and a general description of the improvements recommended through 2042. This section also recommends recurring annual capital expenditures to repair or replace aging and outdated infrastructure, including meter and pipeline replacements. The water systems analysis described in sections 6 and 7 was used as the basis for identifying CIP project recommendations. The CIP project recommendations are based on a review of the existing City CIP, as well as deficiencies predicted by the hydraulic model in the existing infrastructure for current conditions and projected future growth conditions.

The CIP projects were evaluated based on the following:

- Hydraulic analysis of existing and projected water supply systems requirements;
- Condition assessment based on the visual inspection of the Hinckley and Tate WTP, reservoir sites with booster pumps, and associated electrical and mechanical equipment;
- Risk-based asset register database evaluation.

The hierarchy of projects is established using several methodologies. The first methodology employed is a hydraulic analysis of existing and future water supply systems. The highest priority projects are those with hydraulic deficiencies and/or regulatory compliance issues. These are planned for the first five-year period. The following criticality methodology relies on the well and booster pump efficiency analysis and the distribution systems age. The third methodology uses condition assessment based on the site visits.

10.1.1 APPROACH TO PLANNING LEVEL COSTS

Each CIP project recommendation includes an OPC based on planning level quantity estimates. Unit costs are based on vendor quotes and actual bid prices for projects of similar scope and size. In some cases, the OPC was adjusted to account for specific site conditions. Each OPC includes standard mark-ups for contingencies, engineering fees, legal fees, administrative costs, and other soft costs to provide a complete budget level prediction of project costs. Table 10-1 summarizes OPC assumptions for each CIP project recommendation.

Table 10-1: CIP Project Recommendation OPC Assumptions

OPC Item	Markup Assumption
Construction Contingency	10%-25% [1]
Engineering	15%
Legal & Administration	5%
TOTAL	1.30x-1.45x Construction Cost [1]
Note: (1) Varies based on the complexity of the project.	

Many of the recommended projects consist of replacing water pipelines, rehabilitation of existing well sites and equipment, booster pump repair or replacement, or rehabilitation of reservoir facilities. Modifications and upgrades to existing pumping facilities tend to be more complex than pipeline replacement projects, and higher contingency and soft cost factors were used for these facilities. All cost opinions are shown in 2022 dollar values, and should be escalated for project implementation in future years. Each OPC is considered a Class 5 Construction Cost Opinion as defined by the Association for the Advancement of Cost Engineering (AACE) International Recommended Practice No. 18R-97. Table 10-2 below defines the various cost opinion classes defined by AACE International.

Table 10-2: AACE Construction Cost Classes

COST OPINION CLASS	Primary Characteristic	Secondary Characteristic			
	Level of Project Definition	End Usage	Methodology	Expected Accuracy Range	Preparation Effort
	Expressed as % of Complete Definition	Typical Purpose of Cost Opinion	Typical Estimating Method	Typical Variation in Low and High ranges (a)	Typical Degree of effort relative To Least Cost Index of 1(b)
Class 5	0% to 2%	Concept Screening	Capacity Factored. Parametric Models. Judgment or Analogy	L: -20% to -50% H: +30% to +100%	1
Class 4	1% to 15%	Study or Feasibility	Equipment Factored or Parametric Model	L: -15% to -30% H: +20% to +50%	2 to 4
Class 3	10% to 40%	Budget Authorization, or Control	Semi-Detailed Unit Costs with Assembly Level Line Items	L: -10% to -20% H: +10% to +30%	3 to 10
Class 2	30% to 70%	Control or Bid/ Tender	Detailed Unit Cost with Forced Detailed Take-Off	L: -5% to -15% H: +5% to +20%	4 to 20
Class 1	50% to 100%	Check Estimate or Bid/Tender	Detailed Unit Cost with Detailed Take-Off	L: -3% to -10% H: +3% to +15%	5 to 100

Note: (1) The state of process technology and applicable reference cost data availability affects the range markedly. The +/- value represents typical percentage variation of actual costs from the cost estimate after applying contingency (typically at a 50% level of confidence) for a given scope.
(2) If the range index value of "1" represents 0.005% of project costs, then an index value of 100 represents 0.5%. Estimate preparation effort depends on the size of the Project and the quality of estimating data and tools.

10.2 OPINION OF PROBABLE COST

The CIP projects consist of specific project recommendations to repair, replace, or upgrade existing facilities. Recurring consumable replacement and preventive maintenance requirements are generally excluded and not considered a capital improvement. The CIP projects for potable water have been divided into six (6) broad categories, sorted by the facility type. Each category is described below.

10.2.1 CIP ITEM W1: POTABLE WATER DISTRIBUTION SYSTEM

CIP item W1 includes projects that are part of the potable water distribution system, including pipelines, pipeline appurtenances, and metering infrastructure.

The potable water system elements are proposed for replacement because of the 12.8% water loss reported in 2020. The losses include meter inaccuracies and other possible contributors such as unaccounted connections. Reductions in water loss after successfully implementing the proposed CIP could significantly offset the initial investment. The City has replaced aging pipelines annually since 2010, totaling to more than eighty-nine (89) miles of replaced pipeline. In 2015, the City developed a funding plan that included revenue increases to continue this practice annually. Metering of the distribution system is also included in the W1 project list. The potable water distribution system is fully metered and the City has a meter replacement and maintenance plan in place. Meters smaller than 2" are replaced every fifteen to twenty (15 – 20) years, and all meters over two inches (2") are calibrated and repaired if necessary to ensure accuracy. In 2021, the City began a five year project to replace all water meters within its service area.

Additionally, from 2014 to 2015, MUED staff conducted an audit on all commercial properties/accounts to ensure all connections were accounted for in the City's billing system. This allowed the City to decrease unaccounted for water loss and the associated loss in revenue. The City is currently implementing Advanced Metering Infrastructure (AMI) system to improve efficiency in the meter readings.

10.2.2 CIP ITEM W2: HINCKLEY TREATMENT PLANT

CIP item W2 includes projects located at the Hinckley WTP. As Hinckley ages, equipment will need to be repaired, replaced, and upgraded. CIP project recommendations are related to the following:

- Sludge Presses
- Generators
- MCC Installations
- Aging Mechanical Equipment

10.2.3 CIP ITEM W3: TATE TREATMENT PLANT

CIP item W3 includes projects identified at the Tate WTP. As Tate ages, equipment will need to be repaired, replaced, and upgraded. CIP project recommendations are related to the following:

- Raw Water Influent Line
- Clarifier Coating Systems
- Clarifier Covers
- Influent Flash Mixer

- NaOCl Disinfection System
- EIMCO Settlers
- MCC Installations
- Aging Mechanical Equipment

10.2.4 CIP ITEM W4: POTABLE WATER BOOSTER PUMP STATIONS

CIP item W4 includes projects located at the City's potable water booster pump stations. These projects primarily focus on station refurbishment or rehabilitation but may require other capital improvements. Maintaining and improving these booster stations can increase energy and water treatment efficiency. Booster pump stations will require rehabilitation as equipment ages and efficiency declines. CIP project recommendations assume a typical service life of twenty (20) years. Projects should be scheduled on a rotating basis to distribute the financial impact over the planning horizon. Brady completed a condition, seismic, and structural assessment of all water facilities, which should be used to supplement these CIP project recommendations. The executive summary of the assessment can be found in Appendix F.

10.2.5 CIP ITEM W5: POTABLE WATER STORAGE RESERVOIRS

CIP item W5 includes projects related to both the concrete and steel potable water storage reservoirs, including refurbishment and upgrades to meet current standards. Also included are projects to increase facility reliability, correct known deficiencies, and improve water quality within the tank. Brady completed a condition, seismic, and structural assessment of all water facilities, which should be used to supplement these CIP project recommendations. The executive summary of the assessment can be found in Appendix F.

Corrosion is visible on some equipment and steel storage reservoir exterior walls. Although the cathodic protection cabinets at some reservoirs are showing signs of aging, they are still functional and in working condition but are nearing the end of their service life. Steel storage reservoirs should be inspected, maintained, and retrofitted with corrosion protection or replaced. Steel storage reservoirs typically require recoating and corrosion repairs every twenty (20) years. After recoating, the steel tanks must be dewatered, inspected, and minor coating repairs made at regular intervals during the coating life. Concrete storage reservoirs typically require less frequent replacement and maintenance.

10.2.6 CIP ITEM W6: GROUNDWATER WELLS

CIP item W6 includes project recommendations related to groundwater wells. As groundwater wells age, they need to be repaired, replaced, and upgraded. In addition to physical wear, some wells may require improvements to resolve groundwater contaminant issues. Perchlorate has been detected at some groundwater wells, and the City will need to assess the need for perchlorate treatment at several well sites. This evaluation will determine the level and type of treatment necessary at each site. Currently Perchlorate levels at Well No. 38 and Well No. 39 will require design of a wellhead treatment system beginning in 2022.

By 2030 – 2035, additional potable water supply will be needed to meet demands. The use of groundwater wells will be less costly than the expansion of surface WTP. It is recommended that inactive well sites be assessed for rehabilitation, including measures to mitigate groundwater contamination where necessary, and returned to active status to increase potable water supply.

10.2.7 FUTURE WATER SUPPLY PLANNING

The City may need to increase water treatment capacity at Hinckley and/or Tate to meet future demands unless potable water use is decreased, or groundwater wells are refurbished and reactivated. However, to increase capacity at the Tate WTP will require an assessment and improvement to the distribution system to eliminate the bottleneck presently existing at the discharge from the Tate WTP. The extent of these capacity upgrades will be determined based on the expansion of the recycled water system and future demands. The need to increase treatment capacity will also be affected by the future availability of surface water and the number of active groundwater sources. CIP project recommendations that affect water supply and production should be scheduled and implemented before planning year 2035, when potable water demand could exceed available supply.

10.2.8 CIP NP1: NON-POTABLE WATER SYSTEM IMPROVEMENTS

CIP item NP1 includes project recommendations related to the repair, replacement, and improvement of non-potable wells, non-potable meter replacements, and general system deficiencies. Non-potable groundwater well sites should be inspected and maintained regularly, and rehabilitated every ten (10) years. Meters should be inspected and maintained regularly, including periodic accuracy verification, and replaced in accordance with manufacturer recommendations. The non-potable water system could be improved by implementing an AMI system to reduce operating costs. Three (3) CIP project recommendations will improve system hydraulics. These projects are recommended for completion within the next five (5) years.

Also included in NP1 are recommendations to expand and consolidate the non-potable water system at the City’s discretion. This would include construction of new pipelines, along with associated booster stations and other necessary equipment.

10.2.9 CIP NP2: RECYCLED WATER SYSTEM IMPROVEMENTS

CIP NP2 includes project recommendations related to the repair, replacement, improvement, and expansion of the recycled water system. This includes the construction of two (2) storage reservoirs and several pipeline additions and replacements, and assumes all pipelines will be replaced within seventy (70) years

Also included in NP2 are recommendations to expand and consolidate the recycled water system at the City’s discretion. This would include construction of new pipelines, including a transmission line from the WWTP’s proposed storage reservoirs located in System 1 to System 2 at the Texas St. non-potable pumping station. It will also be necessary to construct a new non-potable reservoir or repurpose the existing potable water reservoir at this site. Improvements are also needed for other pipelines, associated booster stations, and other necessary equipment.

10.3 PROJECTS AND COSTS

10.3.1 POTABLE WATER SYSTEM CIP PROJECT RECOMMENDATIONS

Potable water system 5-year CIP project recommendations and estimated costs are provided in Table 10-3, and summarized by category in Chart 10-1.

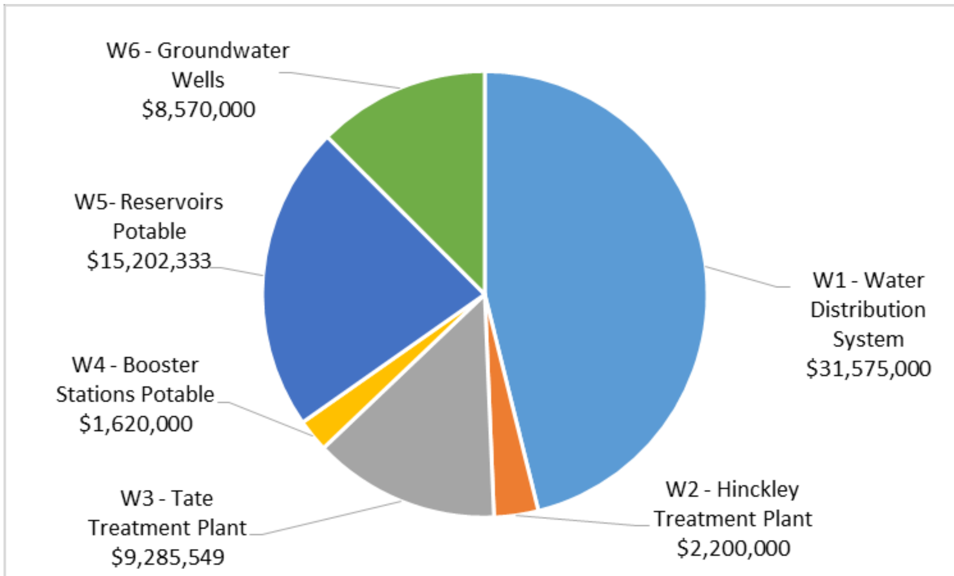

Chart 10-1: Potable Water System 5-Year CIP Cost Summary

Table 10-3: 5-Year Potable Water CIP Recommendations

W1	Water Distribution System	FY 2022-23	FY 2023-24	FY 2024-25	FY 2025-26	FY 2026-27	5-year Total
W1-1	Pipeline Replacement	$4,500,000	$4,500,000	$4,500,000	$4,500,000	$4,500,000	$22,500,000
W1-2	Water Meter Replacements	$1,815,000	$1,815,000	$1,815,000	$1,815,000	$1,815,000	$9,075,000
CIP W1 Subtotal		**$6,315,000**	**$6,315,000**	**$6,315,000**	**$6,315,000**	**$6,315,000**	**$31,575,000**

W2	Hinckley Treatment Plant	FY 2022-23	FY 2023-24	FY 2024-25	FY 2025-26	FY 2026-27	5-year Total
W2-1	Hinckley Sludge Press	$355,000	$345,000	$0	$0	$0	$700,000
W2-2	Replace Aging Mechanical & MCC Equipment	$0	$90,000	$470,000	$470,000	$470,000	$1,500,000
CIP W2 Subtotal		**$355,000**	**$435,000**	**$470,000**	**$470,000**	**$470,000**	**$2,200,000**

W3	Tate Treatment Plant	FY 2022-23	FY 2023-24	FY 2024-25	FY 2025-26	FY 2026-27	5-year Total
W3-1	Tate Transmission Line Replacement	$835,549	$1,900,000	$1,900,000	$0	$0	$4,635,549
W3-2	Tate Clarifier Recoating	$0	$1,000,000	$0	$0	$0	$1,000,000
W3-3	Tate Clarifier Covers	$0	$1,560,000	$0	$0	$0	$1,560,000
W3-4	Tate Influent Flash Mixer	$0	$0	$180,000	$0	$0	$180,000
W3-5	Tate NaOCl Disinfection System	$0	$0	$360,000	$0	$0	$360,000
W3-6	Replace Aging Mechanical & MCC Equipment	$0	$90,000	$470,000	$470,000	$520,000	$1,550,000
CIP W3 Subtotal		**$835,549**	**$4,550,000**	**$2,910,000**	**$470,000**	**$520,000**	**$9,285,549**

W4	Booster Stations	FY 2022-23	FY 2023-24	FY 2024-25	FY 2025-26	FY 2026-27	5-year Total
W4-1	1750 Blend Manifold Replacement	$120,000	$0	$0	$0	$0	$120,000
W4-2	Booster Pump Station Rehabilitation	$300,000	$300,000	$300,000	$300,000	$300,000	$1,500,000
CIP W4 Subtotal		**$420,000**	**$300,000**	**$300,000**	**$300,000**	**$300,000**	**$1,620,000**

W5	Reservoirs - Potable	FY 2022-23	FY 2023-24	FY 2024-25	FY 2025-26	FY 2026-27	5-year Total
W5-1	Reservoir Sites Fixed Generators	$0	$750,000	$300,000	$300,000	$300,000	$1,650,000
W5-2	Sunset Reservoir Replacement	$2,000,000	$6,000,000	$0	$0	$0	$8,000,000

W5-4	Seismic Assessment Improvements	$1,000,000	$2,891,000	$0	$1,571,333	$0	$5,462,333
W5-5	Texas Grove Reservoir Stair Installation	$0	$0	$0	$90,000	$0	$90,000
CIP W5 Subtotal		**$3,000,000**	**$9,641,000**	**$300,000**	**$1,961,333**	**$300,000**	**$15,202,333**

W6	Groundwater Wells	FY 2022-23	FY 2023-24	FY 2024-25	FY 2025-26	FY 2026-27	5-year Total
W6-1	Groundwater Well Equipping Rehabilitation	$514,000	$506,000	$600,000	$600,000	$600,000	$2,820,000
W6-2	East Lugonia Well 3 Replacement	$0	$0	$3,000,000	$0	$0	$3,000,000
W6-3	Groundwater Contamination Mitigation	$575,000	$575,000	$1,000,000	$0	$0	$2,150,000
W6-4	Entrained Air Treatment Assessment	$0	$0	$600,000	$0	$0	$600,000
CIP W6 Subtotal		**$1,089,000**	**$1,081,000**	**$5,200,000**	**$600,000**	**$600,000**	**$8,570,000**

Potable water system 20-year CIP project recommendations and estimated costs are provided in Table 10-4, and summarized by category in Chart 10-2. Annually recurring projects are assumed to have the same costs each year. Figure 10-1 provides a map depicting the remaining service life of the potable water system facilities.

Table 10-4: 20-Year Potable Water CIP Recommendations

Project	2022-2026	2027-2031	2032-2036	2037-2041	20-year Total
W1 - Water Distribution System	$31,575,000	$31,575,000	$31,575,000	$31,575,000	$126,300,000
W2 - Hinckley Treatment Plant[1]	$2,200,00	N/A	N/A	N/A	$2,200,00
W3 - Tate Treatment Plant[1]	$9,285,549	N/A	N/A	N/A	$9,285,549
W4 - Booster Stations - Potable	$1,620,000	$3,883,690	$1,620,000	$1,620,000	$8,743,690
W5 - Reservoirs - Potable	$15,202,333	$5,198,000	$180,000	$180,000	$20,760,333
W6 - Groundwater Wells[2]	$8,570,000	$7,912,500	$5,487,500	$6,750,000	$28,720,000
Total	**$68,452,882**	**$48,569,190**	**$38,862,500**	**$40,125,000**	**$196,009,572**

Note: (1) A condition assessment was performed in 2021 on the Hinckley and Tate Treatment Plants based on current conditions. Hinckley is 36-year-old, and Tate is 55 years old. The mechanical and electrical equipment may need to be replaced in the future, but the information is not included and is labeled as Optional. The cost opinion for replacing mechanical and electrical equipment was based on the 2022 economic indices. The potential cost for each plant's electrical and mechanical replacement is estimated at 1-2 million dollars per year in the CIP planning horizon. Therefore, a detailed conditional mechanical and electrical equipment assessment is needed to determine accurate mechanical and electrical CIP elements. (2) The 20-year cost for W6 - Wellhead Treatment is dependent on the Wellhead Evaluation.

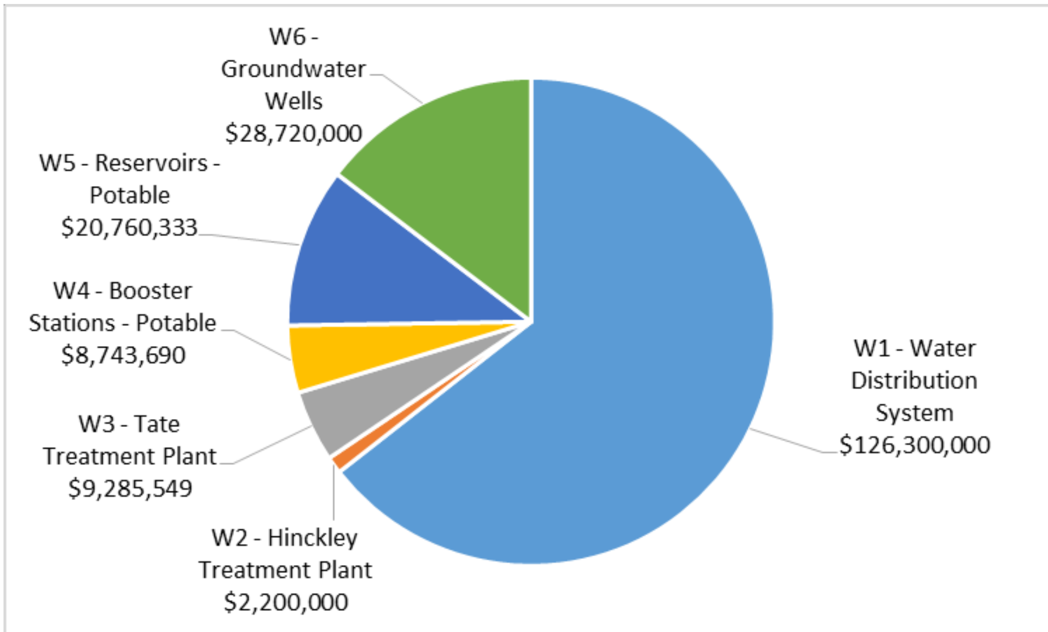

Chart 10-2: Potable Water System 20-Year CIP Cost Summary

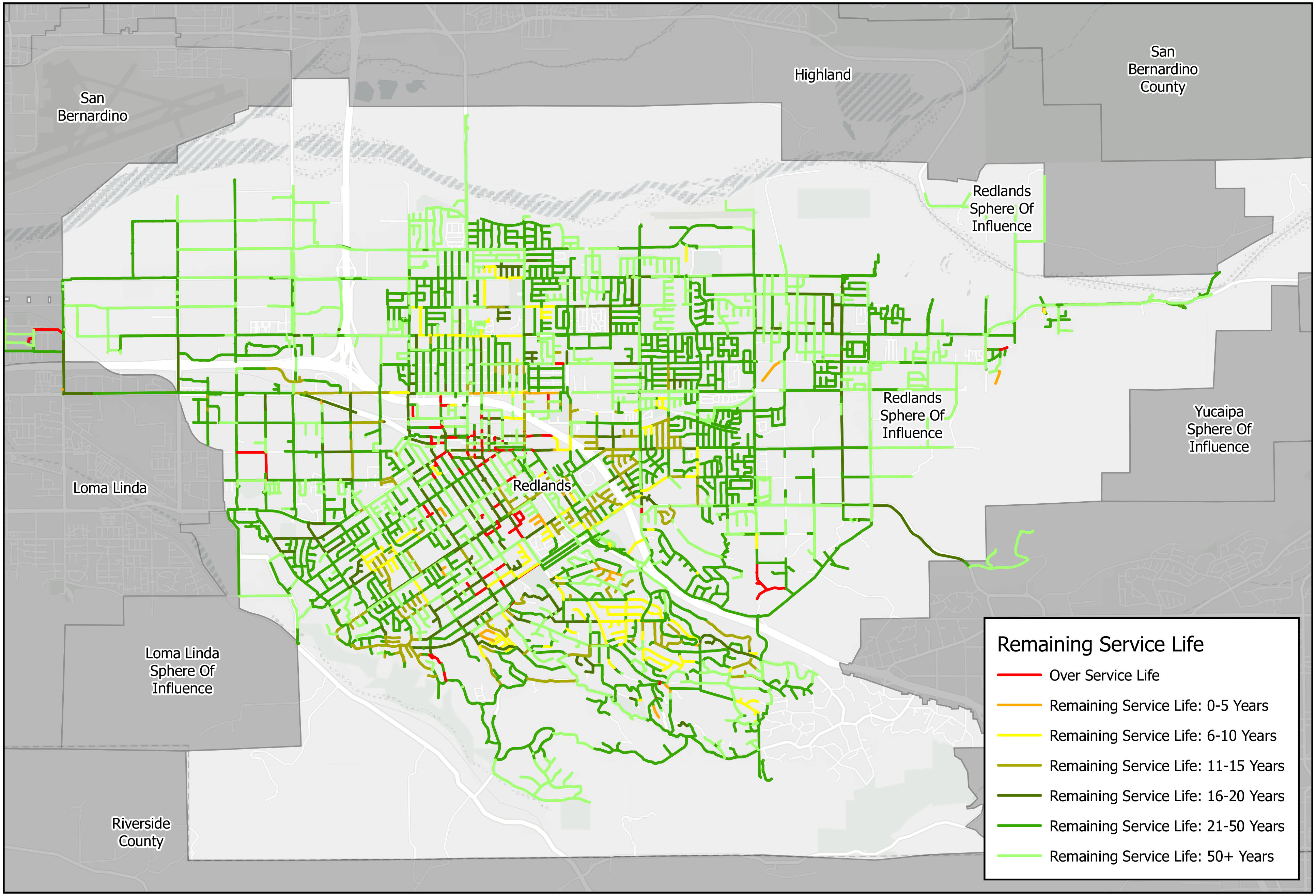

Figure 10-1
Redlands Potable Water Pipeline Remaining Service Life

Source: ESRI, USGS, City of Redlands

10.3.2 NON-POTABLE WATER SYSTEM CIP PROJECT RECOMMENDATIONS

Non-potable water system 5-year CIP project recommendations and estimated costs are provided in Table 10-5.

Table 10-5: 5-Year Non-Potable Water CIP Recommendations

NP 1	Non-potable Water Improvements	FY 2022-23	FY 2023-24	FY 2024-25	FY 2025-26	FY 2026-27	5-year Total
NP 1.1	Pipeline Replacement and Expansion	$0	$0	TBD	TBD	TBD	TBD
NP 1.2	Groundwater Well Equipment Rehabilitation	$267,000	$136,000	$375,000	$375,000	$375,000	$1,528,000
CIP NP 1 Subtotal		**$267,000**	**$136,000**	**$375,000**	**$375,000**	**$375,000**	**$1,528,000**

NP 2	Recycled Water Improvements	FY 2022-23	FY 2023-24	FY 2024-25	FY 2025-26	FY 2026-27	5-year Total
NP 2.1	Recycle Water Reservoirs - design two & build in phases	$734,839	$0	$3,000,000	$0	$3,000,000	$6,734,839
NP 2.2	Pipeline Replacement and Expansion	$0	$0	$750,000	$750,000	$3,000,000	$1,500,000
CIP NP 2 Subtotal		**$734,839**	**$0**	**$3,750,000**	**$750,000**	**$3,000,000**	**$8,234,839**

Non-potable water system 20-year CIP project recommendations and estimated costs are provided in Table 10-6. Annually recurring projects are assumed to have the same costs each year. Figure 10-2 provides a map depicting the remaining service life of the non-potable and recycled water systems facilities.

Table 10-6: 20-Year Non-Potable Water CIP Recommendations

NP 1	Non-potable Water Improvements	2022-2026	2027-2031	2032-2036	2037-2041	20-year Total
NP 1.1	Pipeline Replacement and Expansion	TBD	$1,000,000	$1,000,000	$1,000,000	$3,000,000
NP 1.2	Groundwater Well Equipment Rehabilitation	$1,528,000	1,007,800	$1,108,600	$1,219,500	$4,863,900
NP 1.3	Meter Replacement	$0	$0	$55,000	$55,000	$110,000
CIP NP 1 Subtotal		**$1,528,000**	**$2,007,800**	**$2,163,600**	**$2,274,500**	**$7,973,900**

NP 2	Recycled Water Improvements	2022-2026	2027-2031	2032-2036	2037-2041	20-year Total
NP 2.1	Recycle Water Reservoirs - design two & build in phases	$6,734,839	$0	$0	$0	$6,734,839
NP 2.2	Pipeline Replacement and Expansion	$1,500,000	$1,000,000	$1,000,000	$1,000,000	$4,500,000
CIP NP 2 Subtotal		**$8,234,839**	**$1,000,000**	**$1,000,000**	**$1,000,000**	**$11,234,839**
CIP NP Total		**$9,762,839**	**$3,007,800**	**$3,163,600**	**$3,274,500**	**$19,208,739**

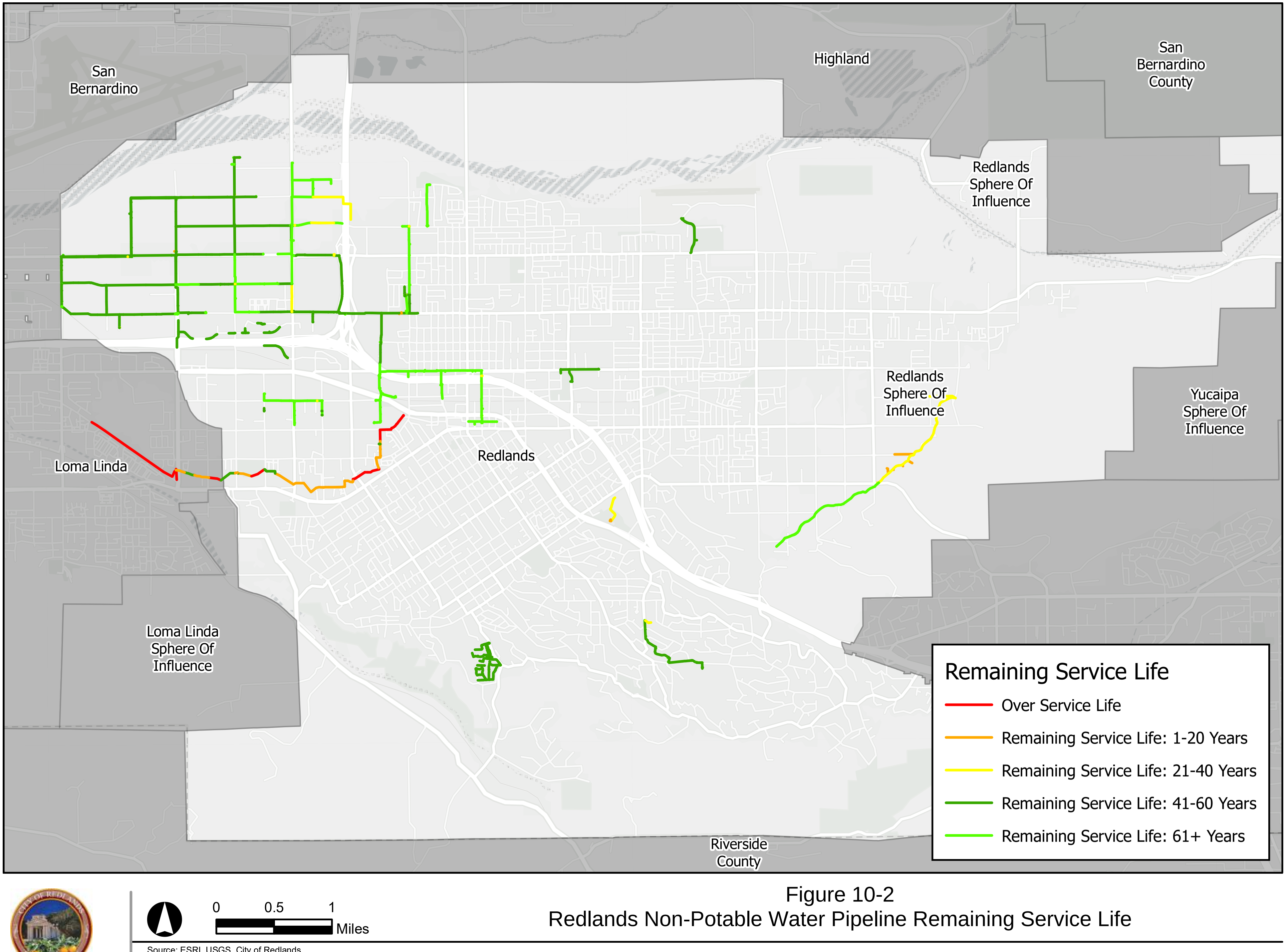

Figure 10-2
Redlands Non-Potable Water Pipeline Remaining Service Life

10.4 THE CAPITAL IMPROVEMENT PROJECTS

The CIP project recommendations assume that projects that meet regulatory compliance requirements and resolve hydraulic deficiencies will be constructed first. Project priorities are likely to change as economic conditions and community demographics change. A comprehensive analysis of each project is necessary prior to implementation. Cost estimates are provided in 2022 dollars, and assume no property acquisition is necessary.

10.4.1 POTABLE WATER SYSTEM CIP PROJECT DESCRIPTIONS

CIP W1-1: Water Pipeline Replacements - Hydraulics

5-Year Budget Allocation: $22,500,000

Priority: High

It is recommended that the City consider an annual project to replace pipelines to resolve hydraulic deficiencies as provided in Table 10-7. The first three (3) years should focus on replacing existing pipes with deficiencies that create high velocities and/or low pressures within the system. This project replaces approximately 27,193 LF of existing undersized pipelines with new eight inch (8”), twelve inch (12”), and twenty-four inch (24”) diameter pipelines at twelve (12) locations.

Table 10-7: Pipeline Replacement

CIP	New Diameter (in)	Length (ft)	Location	Issue	Budget	Year
CIP-1	24	2046	San Bernardino Ave/Agate Ave	High Velocity	$654,720	2024
CIP-2	8	203	Mill Creek Rd	High Velocity	$33,000	2022
CIP-3	8	649	Naples Ave/Jasper Ave	Fire Flow	$104,000	2023
CIP-4	12	11979	Wabash Ave/6th Ave	Fire Flow	$2,395,800	2024
CIP-5	8	1142	Pennsylvania Ave/De Anza St	Fire Flow	$183,000	2022
CIP-6	8	1569	Park Ave/Cook St	Under-Sized	$251,000	2023
CIP-7	N/A	TBD	Valencia Dr	High Velocity	TBD	TBD
CIP-8	N/A	TBD	San Bernardino Ave	High Velocity	TBD	TBD
CIP-9	N/A	TBD	Park Ave/New Jersey St	Fire Flow	TBD	TBD
CIP-10	N/A	TBD	Emerald Ave/Newport Ave	Fire Flow	TBD	TBD
CIP-11	N/A	TBD	Sunset Dr/Fairmont Dr	Fire Flow	TBD	TBD

Approximately 34,320 LF (6.5 miles) of the City's 382 miles of water distribution pipelines are currently beyond the expected service life of the pipe material. It is recommended that the City continue to proactively replace aging pipelines once the material service life is reached. Chart 10-3 shows the service life distribution remaining in the City's pipelines.

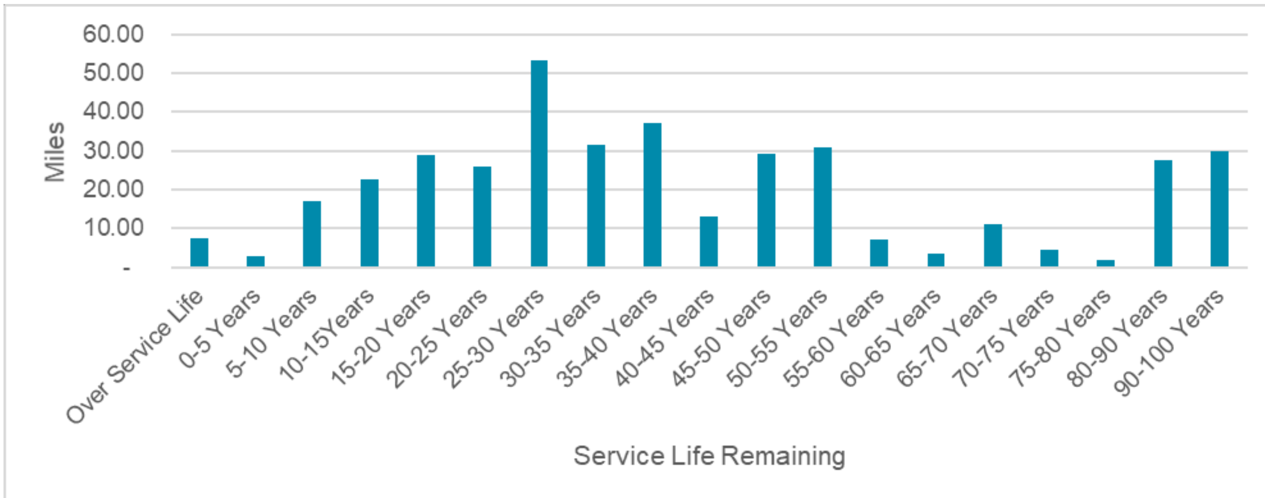

Chart 10-3: Potable Water Pipeline Remaining Service Life

Most of the City's pipelines have twenty to fifty (20-50) years of remaining service life, with about seventy (70) miles of pipeline service life ending in the next twenty (20) years. The City should continue to replace a portion of these lines each year, with a replacement goal of approximately 28,000 LF each year, which will replace all pipelines within seventy (70) years. Pipeline segments with hydraulic deficiencies are recommended to be replaced first, followed by aging asbestos-cement pipelines. All new pipelines should a minimum of eight inches (8”) in diameter. Replacement pipeline segments should be selected yearly based on age and maintenance history, and coordinated with other projects, such as scheduled street rehabilitation projects.

CIP W1-2: Water Meter Replacement

5-year Budget Allocation: $9,075,000

Priority: Medium

In 2021, the City began a five (5) year annual project to replace all water meters. Table 10-8 provides the meter size and number of meters being replaced and Table 10-9 provides the number of meter size and number of meters being retro-fitted. It is recommended that this project be continued to reduce water loss within the system, and significantly increase revenue. After completion of the project, water meter accuracy testing should be conducted annually, meters will be replaced based on the test results, not on a predetermined length of time.

In 2022, the City began to update its meters to Advanced Metering Infrastructure (AMI) to maintain accurate billing and analysis data. AMI will encourage water conservation and reap benefits for business and residential customers by allowing them the ability of real time water consumption.

Table 10-8: Meter Replacement

Meter Size (in)	Number of Meters
5/8"	117
3/4"	4,682
1"	7,415
1 1/2"	521

Meter Size (in)	Number of Meters
2"	497
3"	50
4"	30
6"	13
8"	8
12"	1
Total	**13,334**

Table 10-9: Meter Retro-Fit

Meter Size (in)	Number of Meters
5/8"	60
3/4"	3,417
1"	4,346
1 1/2"	233
2"	259
3"	27
4"	22
6"	14
8"	6
Total	**8,384**

CIP W2-1: Hinckley Sludge Press

5-Year Budget Allocation: $700,000

Priority: Medium

This project engineers and installs a sludge press at Hinckley to reduce labor, equipment, and disposal costs associated with processing sludge residual to the treatment process.

CIP W2-2: Replace Aging Mechanical & MCC Equipment

5-Year Budget Allocation: $1,500,000

Priority: Medium

This project replaces and upgrades electrical and mechanical equipment at Hinckley as necessary. A comprehensive condition assessment should be completed annually to identify the remaining service life of equipment in order to prepare for replacement.

CIP W3-1: Tate Transmission Line Replacement

5-Year Budget Allocation: $4,635,549

Priority: High

A Condition, Structural, & Seismic Assessment completed by Brady identified this as a top-priority project. The City selected a consultant to engineer the replacement of this raw water influent line, with the goal of constructing the project in the near-future.

CIP W3-2: Tate Clarifier Recoating

5-Year Budget Allocation: $1,000,000

Priority: Medium

The Tate Clarifier Recoating project is on the City's CIP list. Therefore, it is recommended that this project remains in the CIP. The recommended budget for this CIP is $1,000,000.

CIP W3-3: Tate Clarifier Covers

5-Year Budget Allocation: $1,560,000

Priority: Medium

The Tate Clarifier Covers project is on the City's CIP list. Therefore, it is recommended that this project remains in the CIP. The recommended budget for this CIP is $1,560,000.

CIP W3-4: Tate Influent Flash Mixers

5-Year Budget Allocation: $180,000

Priority: Low

The Tate Influent Flash Mixers project is on the City's CIP list. Therefore, it is recommended that this project remains in the CIP. The recommended budget for this CIP is $180,000.

CIP W3-5: Tate NaOCl Disinfection System

5-Year Budget Allocation: $360,000

Priority: Low

The Tate NaOCl Disinfection System project is on the City's CIP list. Therefore, it is recommended that this project remains in the CIP. The recommended budget for this CIP is $360,000.

CIP W3-6: Replace Aging Mechanical & MCC Equipment

5-Year Budget Allocation: $1,550,000

Priority: Medium

This project replaces and upgrades electrical and mechanical equipment at Tate, as necessary. A comprehensive condition assessment should be completed annually to identify the remaining service life of equipment in order to prepare for replacement.

Coating loss and corrosion are present on the EIMCO settlers at the plant. To maintain the equipment from metal loss and eventual failure from corrosion, the ferrous surfaces should be abrasively blasted and recoated.

CIP W4-1: 1750 Blend Manifold Replacement

5-Year Budget Allocation: $120,000

Priority: High

The 1750 Blend Manifold Replacement project is on the City's CIP list. Therefore, it is recommended that this project remains in the CIP. The recommended budget for this CIP is $120,000.

CIP W4-2: Booster Pump Station Rehabilitation

5-Year Budget Allocation: $1,500,000

Priority: Medium

The booster pump stations will need rehabilitation and refurbishment once they approach the end of their service life. Their typical service life is based on 20-year equipment life. It is recommended the City rehabilitate a single pump station each year, and the station refurbishments can be placed on a 15-year cycle. It is recommended the station priority be based on efficiency, condition, and age. Table 10-10 provides a list prioritized on efficiency from SCE pump tests.

Table 10-10: Booster Pump Stations Rehabilitation

Name	Total Num. Pumps	Cumulative HP	Average Efficiency
South	5	800	49.8
Fifth Avenue	4	450	59.2
HAWC	6	975	66.6
Sand Canyon	2	200	66.7
Country Club	4	550	68.1
Agate	3	300	68.7
Dearborn	2	300	69.9
Texas	4	1000	73.5
Smiley Heights	2	80	52.0
Ford Park	2	N/A	N/A
Ward Way	2	100	56
Mill Creek	2	125	56.0

CIP W5-1: Reservoir Sites Fixed Generators

5-Year Budget Allocation: $1,650,000

Priority: High

The Reservoir Sites Fixed Generators project is on the City's CIP list. Therefore, it is recommended that this project remains in the CIP. The recommended budget for this CIP is $1,650,000

CIP W5-2: Sunset Reservoir Replacement

5-Year Budget Allocation: $8,000,000

Priority: High

This project engineers and replaces a three (3) MG potable water storage reservoir that is seismically deficient.

CIP W5-4: Seismic Assessment Improvements

5-Year Budget Allocation: $5,462,333

Priority: Medium

The City currently owns and operates eighteen (18) storage tanks ranging from 0.2 to 10.6 MG, provided in Table 10-11. The total budget varies every year based on the recommendations from Brady.

In the first year, the City should conduct a preliminary inspection of each reservoir to assess the interior and exterior coating conditions and determine if additional upgrades are required to meet current seismic design standards and California Division of Occupational Safety and Health Agency (CalOSHA) requirements. The priority of the projects should be based on the recommendations from Brady.

Table 10-11: Storage Tank Reservoirs

Priority	Name	Capacity (MG)	Type	Year Constructed
1	Texas Grove	3.9	Steel	2004
2	Ward Way	2	Steel	1958
3	Mill Creek 1	0.2	Steel	1962
4	Smiley	3	Steel	1964
5	South	2	Steel	1964
6	Arroyo	0.5	Steel	1965
7	Sunset	3	Steel	1967
8	Agate	3	Steel	1968
9	Crafton	3.5	Steel	1970
10	Sand Canyon	3.5	Steel	1973
11	Mill Creek 2	0.2	Steel	1987
12	Texas Street	1	Steel	1956
13	Highland	10	Concrete	1976
14	Country Club 1	1	Concrete	1924
15	Country Club 2	2	Concrete	1969
16	Margarita	2.4	Concrete	1964
17	Dearborn	10.6	Concrete	1974
18	Fifth Avenue	5	Concrete	1974

Table 10-12 provides the reservoirs Brady proposes need replacing.

Table 10-12: Replacement per Seismic Study

Priority	Name	Capacity (MG)	Type	Year Constructed	Seismic Replacement Budget	Replacement Year
1	Sunset	3	Steel	1967	$8,000,000	2023

CIP W5-5: Texas Grove Reservoir Stair Installation

5-Year Budget Allocation: $90,000

Priority: TBD

The Texas Grove Reservoir Stair project is on the City's CIP list and scheduled for 2022. Therefore, it is recommended that this project remains in the CIP, and the cost be escalated twenty percent (20%) to bring the prices to the year 2022 dollars. The recommended budget in this CIP is $90,000.

CIP W6-1: Groundwater Well Equipping Rehabilitation

5-Year Budget Allocation: $2,820,000

Priority: High

The groundwater well pumping equipment will need rehabilitation and refurbishment once they approach the end of their service life. Their typical service life is based on 20-year equipment life. It is recommended the City rehabilitate one or more groundwater well-pumping equipment each year, provided in Table 10-13. Once all wells are rehabilitated, the City should evaluate the oldest wells to determine when the next rehabilitation should start. It is recommended the station priority be based on efficiency, condition, and age. The list below is prioritized on efficiency from SCE pump tests. The costs below represent above-ground equipment only and do not include subsurface improvements.

Table 10-13: Groundwater Wells

Name	Capacity (GPM)	Status in 2019	Efficiency	HP	Rehab Budget	Rehab Year
Well 10	1400	IDLE	0.27	75	$120,000	2025
Maguet 2	400	IN USE	0.35	25	$50,000	2025
Rees	550	IN USE	0.43	250	$112,640	2024
Lugonia 3	250	IN USE	0.43	25	$35,010	2022
Madeira	600	IN USE	0.62	150	$119,058	2023
Crafton	1700	IDLE	0.65	200	$280,000	2026
Airport 2	1000	IN USE	0.68	300	$133,760	2024
Airport 1	1500	IN USE	0.70	350	$124,907	2022
Mentone Acres 2	1600	IN USE	0.71	300	$138,502	2023
Orange Street	1500	IN USE	0.74	300	$280,000	2027
North Orange Street 1	2900	IN USE	0.77	350	$128,460	2022
Church Street	2000	IN USE	0.77	400	$139,304	2024
Well 39	1250	IN USE	0.78	250	$133,030	2023
North Orange Street 2	2900	IN USE	0.78	350	$275,000	2031

Name	Capacity (GPM)	Status in 2019	Efficiency	HP	Rehab Budget	Rehab Year
Well 38	1600	IN USE	0.78	300	$123,230	2023
Agate 2	1750	IDLE	0.79	200	$175,000	2023
Lugonia 6	1300	IDLE	UNK	75	$67,850	2022
Muni	2200	IN USE	UNK	300	$314,000	2022
Well 13	3300	IDLE	UNK	300	$120,000	2025
Mill Creek 2 - Surface Water	850	IN USE	0.64	75	$225,000	2025
Mill Creek 2A - Surface Water	600	IN USE	.49	50	$67,200	2024
Note: (1) UNK-Un known; (2) Mill Creek 2 and 2A are detached potable wells used to deliver water to the Mill Creek Mutual Water System located on Highway 38.						

CIP W6-2: East Lugonia Well 3 Replacement

5-Year Budget Allocation: $3,000,000

Priority: Low

The East Lugonia Well 3 Replacement project is on the City's CIP list. Therefore, it is recommended that this project remains in the CIP. The recommended budget for this $2,600,000.

CIP W6-3: Groundwater Contaminate Mitigation

5-Year Budget Allocation: $2,150,000

Priority: Medium

The City currently has wells with contaminates that exceed or are close to exceeding mandated MCLs. Although these wells are not currently used for potable water production, they will eventually be needed. It is recommended that the City implement a study to determine the water quality and the most appropriate treatment process for the specific contaminants identified, provided in Table 10-14. The project's first phase would be implementing a study, followed by treatment implementation for one set of wells each year. This approach would complete all projects by 2035 to meet future demands. The costs assume blending treatment could not be used to meet MCLs. Typically, SWRCB-DDW no longer permits blending as an alternative, but requires best available technology to achieve MCL regulation limits. Furthermore, many emerging constituents of concern could most likely become regulated as a MCL. Therefore, the budget costs should be revisited once the study is complete.

Table 10-14: Well Study and Treatment

Study and Evaluation	Cost
Wellhead Treatment Design for Wells 38, 39, Church St, and Orange	$575,000
Wellhead Treatment Design for Agate 1 and 2, Crafton, Wells 10 and 13	$575,000
Wellhead Treatment and Implementation for Wells 38 and 39	$1,000,000

CIP W6-4: Entrained Air Treatment Assessment

5-Year Budget Allocation: $600,000

Priority: Low

The Entrained Air Treatment Assessment project is on the City's CIP list. Therefore, it is recommended that this project remains in the CIP. The recommended budget for this $600,000. It is recommended that the City elaborates the entrained air issue further to capture the urgency and cost estimate better.

10.4.2 NON-POTABLE WATER SYSTEM CIP PROJECT DESCRIPTIONS

CIP NP1-1 Pipeline Replacement and Expansion

5-Year Budget Allocation: TBD

Priority: Low

It is recommended that the City budget for hydraulically deficient pipeline replacements, provided in Table 10-15. The NPW CIP should be focused heavily on replacing existing pipes with deficiencies that create high velocities or pressures as predicted by the hydraulic model performed for this Master Plan. The replacement of this pipeline is divided into 4 separate areas, grouped by segments in the exact physical location.

Table 10-15: Non-Potable Pipeline Replacement

CIP	New Diameter (in)	Length (ft)	Street	Issue	Budget
NP-CIP-1	12	2,608	Pioneer Ave	High Flow Velocity	$610,000
	12	624	Alabama St	High Flow Velocity	$146,000
Subtotal		**3,232**			**$756,000**
NP-CIP-2	8	1,900	Orange Tree Ln	High Flow Velocity	$296,000
	10	1,468	Orange Tree Ln	High Flow Velocity	$286,000
Subtotal		**3,368**			**$582,000**
NP-CIP-3	16	1,006	Texonia Park	High Flow Velocity	$314,000
	12	1,200	W Lugonia Ave	High Flow Velocity	$281,000
Subtotal		**2,206**			**$595,000**
Total		**8,806**			**$1,933,000**

This NPW CIP includes the expansion of the City's non-potable water system. This would include expanding non-potable water in System 2 and the connection of the Hillside Memorial Park, Ford Park, and Redlands Country Club to the system. This will also leave the potential for the City to connect to the Bear Valley non-potable water system if the City chooses.

CIP NP1-2 Groundwater Well Equipping Rehabilitation

5-Year Budget Allocation: $1,528,000

Priority: Medium

The City has eleven (11) non-potable wells. These wells will need rehabilitation and refurbishment once they approach the end of their service life, provided in Table 10-16. Their typical service life is based on 20-year equipment life. It is recommended the City rehabilitate at least one pump each year. Once all wells are rehabilitated, the City should evaluate the oldest wells to determine when the subsequent rehabilitation should start. It is recommended the stations priority be based on efficiency, condition, and age. The list below is prioritized on efficiency from SCE pumps tests. The cost below represents above-ground equipment only and does not include subsurface improvements.

Table 10-16: Ground Water Well CIP list with the Cost Opinion

Name	Capacity (gpm)	Status in 2019	Efficiency	HP	Rehab Budget	NPW CIP Year
Well 31A	850	Idle	31.9%	450	$195,230	2022
Well 11	300	In Use	47.9%	60	$64,180	2023
California	500	In Use	52.0%	100	$100,000	2024
Well 16	1500	In Use	53.2%	150	$280,000	2025
New York Street	1500	In Use	54.9%	150	$111,050	2023
Well 41	800	In Use	Unknown	100	$280,000	2027
Well 14	2200	In Use	Unknown	125	$280,000	2028/2029
Crafton	1750	idle	59.8%	200	$280,000	2030/2031
Well 32	1850	In Use	60.5%	200	$119,122	2022
Well 30A	1500	In Use	61.2%	150	$280,000	2034/2035
Agate 1	1800	Idle	77.5%	200	$350,000	2038

CIP NP2-1: Recycle Water Reservoirs

5-Year Budget Allocation: $6,734,839

Priority: Medium

This NPW CIP was included in the City's NPW CIP list and focuses on designing two steel reservoirs to improve the management and operation of the City's recycled water system. In addition, the City plans to construct one reservoir near the WWTP to store reclaimed water from the treatment facility. These reservoirs could significantly increase the efficiency of the recycled/non-potable water system.

CIP NP2-2: Pipeline Replacement and Expansion

5-Year Budget Allocation: $1,500,000

Priority: Medium

Approximately 10,000 LF of the City's recycled/non-potable pipelines are over the service life. Another 9,200 LF of the pipeline will reach the end of their service life within 20 years. There is also another 31,300 LF of pipeline that material or installation data is unknown and may need to be replaced. The figure below shows the remaining service life based on the material and age of the pipe. Chart 10-4 shows the service life distribution remaining in the City's pipelines.

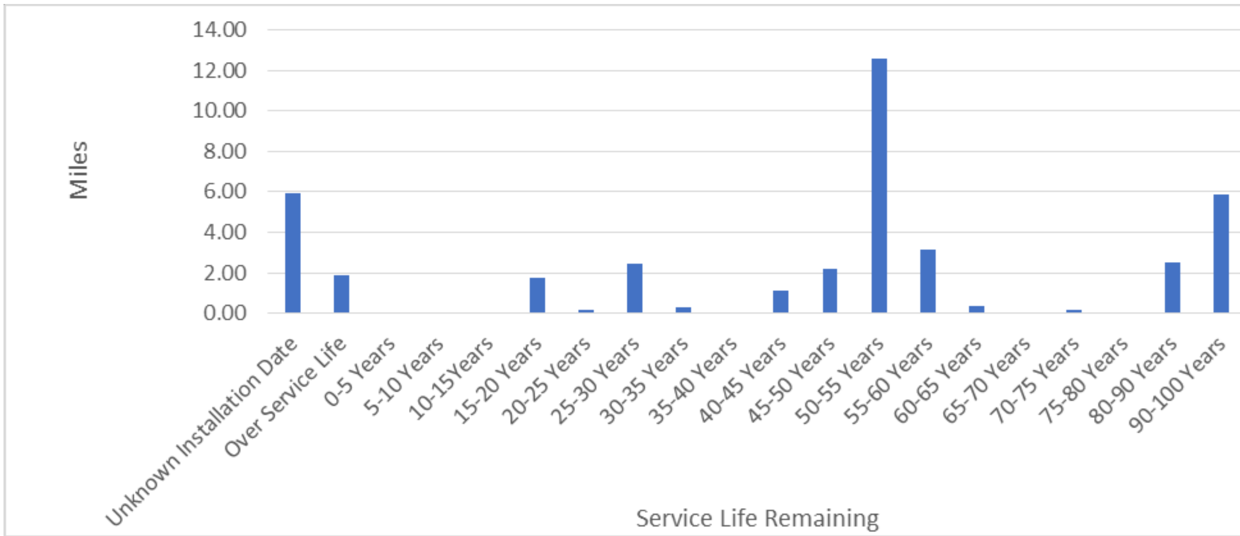

Chart 10-4: Recycled/Non-potable Water Pipeline Remaining Service Life

Most of the City's recycled/non-potable pipelines have a remaining service life of forty to sixty (40-60) years left. To replace all pipelines in 70 years, the City would need to replace approximately 3,100 LF of pipelines. However, due to the number of pipelines with unknown service age, a pipeline over the service life, and pipeline found during the hydraulic modeling, the first five years increase funding to $1,500,000 to replace these pipelines. For the following 15 years, the funding needed is $2,000,000. It is recommended that the City first replace the short pipelines identified in CIP NP1-1 and then prioritize replacing aging pipelines. Pipelines less than 8 inches in diameter should be replaced with 8-inch diameter pipelines. Replacement of pipeline segments should be selected yearly based on age breakage history and coordinated with other CIP projects, such as street improvements or pavement replacement. As a result, the City can benefit from upcoming pavement replacement or avoid replacement in freshly paved streets.

This NPW CIP includes the expansion of the City's recycled water system. The construction of the new pipelines in the first five-year period would primarily include the 8-inch pipelines, with a few pipeline segments more significant than 8 inches where needed.

11 LIST OF APPENDICES

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City of Redlands' Hinckley and Tate Water Treatment Plants, Agate and Sunset reservoirs and site booster pumps, wells and ancillary equipment Condition Assessment

APPENDIX B

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Condition, Seismic, and Structural Assessment Executive Summary

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CONDITION ASSESSMENT

City of Redland's Water Treatment Plants

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Introduction

The City of Redlands Water System utilizes two (2) surface water treatment plants.

The total combined capacity of these facilities is approximately 32 million gallons per day (MGD). Below is a plant process summary based on the site visit with an outline of identified CIP projects:

2 The Horace P. Hinckley Water Treatment Plant

The Horace P. Hinckley Water Treatment Plant (Hinckley WTP) is a conventional water

treatment plant (WTP) utilizing rapid mix flocculation, sedimentation, filtration, and disinfection continuously. The nominal plant capacity is 12 MGD. The peak flow rate is limited to 14.5 MGD based on a maximum filtration rate of 6 gallons per minute per square foot (gpm/sq ft) based on the California Department of Public Health (CDPH). The "firm" capacity is 12 MGD with one filter out-of-service. The treatment plant capacity is expandable to a maximum ultimate capacity of 36 MGD.

The Hinckley WTP treats raw water from the Santa Ana River (SAR), Mill Creek, and State Water Project (SWP). Santa Ana River is the primary source for the plant. Up to 100 percent of the plant's raw water can be supplied from the Santa Ana River during the winter. During hot-weather periods in the summer, the source of 20 to 40 percent of the plant's raw water can be SWP.

2.1 Treatment Process

<u>The first step</u> in the conventional water treatment is the pre-treatment. In this stage, the three chemicals injected into the static mixer's first stage are chlorine, aluminum sulfate, and cationic polymer—the aluminum sulfate and cationic polymer act as a coagulant to help the creation of the floc. Chemical dosages are selected based on historical data (water quality versus required chemical dose), operator experience, and jar test data. However, suppose the raw water quality should change to that for which no historical chemical dosage data is available. In that case, the primary coagulant, coagulant aid, and filter aid dosages should be determined by performing jar and filterability index tests on the raw water. The chlorine dosing will start the disinfection process, prevent the bio growth on the filter and pipes, and assist with any taste and odor issues that the plant could have. The influent flow is conveyed and controlled through the off-site piping and valving influent flowmeter, control valve, and influent static mixer. Though part of the influent meter and mixing process, the flow splitter box is not used, as Hinckley WTP has not been expanded. The primary coagulant aluminum chlorohydrate (ACH), coagulant aid cationic polymer (C308), and disinfectant sodium hypochlorite (NaOCl) may be added to the raw water upstream and downstream of the inline mechanical and static mixer before the flocculation/sedimentation basins.

The second step is coagulation, and flocculation is when suspended solids get together to form larger particles called the "floc." Flocculation is achieved through two flocculation basins, each designed for a flow of 9 mgd. Each flocculation basin has three stages and contains vertical shaft flocculation rotated by an external gear reducer powered by electric motors. The electric motors can run at various speeds using six variable frequency drives (VFDs). Cationic polymer C308 can be added to the third-stage flocculation basin, as needed, to increase the efficiency of the flocculation process.

The third step is sedimentation. In this step, larger particulates settle to the floor of the sedimentation basin. The chemicals added in the first step assist the sedimentation process in helping the particulates settle at the floor of the settling tank. Solids separation is achieved through the sedimentation basins located close to the flocculation basins. Each sedimentation basin is designed for a flow of 9 mgd and includes inclined plate settlers and an aluminum-supported fabric cover to reduce the disinfection dose being affected by sunlight.

The fourth step is filtration. The media filtration system consists of three media filters (anthracite over sand and gravel) equipped with an automated backwash system. The individual filters are designed for a 2.4 mgd (6 gpm per sq. ft) flow and are equal-loading, declining-rate, self-backwashing types with an air scour system and a filter-to-waste system. However, usually, backwashes are manually initiated by the operators.

The fifth step is post-disinfection. We add chlorine to treated water to complete disinfection before it is discharged into the distribution system.

The sixth step is the backwash. The particulates and polymer in the filters are attached to media and then clog them up. The backwash process starts by draining the filters to pre-set point and then air scouring for approximately ten minutes, breaking down any particulates stuck in the system. Finally, backwashing is performed to clean the filters to keep their continuous operation and prevent water from overflowing. The recycled wash water system consists of a waste wash water basin, recycle pump station, recycle wash water treatment plant, and a polymer feed room. The recycled wash water system receives flow primarily from the waste wash water discharged during filter backwashing. A smaller fraction of flow is obtained intermittently from the flocculation and sedimentation basins drainage and overflows lines, from chemical feed room drainage lines, sludge decant pond, and water quality sample pump discharge (grab samples from the laboratory sink). Spent filter backwash wash water flows by gravity from each filter to the waste wash water recovery basin located at the west end of the plant site. The waste wash water basin has a concrete bottom with concrete and soil cement sidewall and a capacity of 0.15 MG. The basin can hold wash water from three consecutive filter backwashes (based on a volume per backwash of 46,500 gallons). A sump on the basin floor located at the north end removes the sludge that settles in the basin. The sump pump in the floor of the basin discharges into the sludge drying ponds. Sludge is removed from the wash water recovery basin as needed, and no less than annually, to prevent large amounts of sludge buildup.

The operation of the filtration process determines flows into the wash water basin. When a filter is backwashed, the basin receives the backwash flow. Decanted supernatant water from the sludge lagoons is pumped from the decant structures to the wash water recovery basin. On occasions when the wash water package treatment plant is out of service, the wash water is sent to a series of percolation ponds located to the north of the sludge drying ponds.

The recycle treatment plant system can treat 1,000 gpm (1.5 mgd) if the influent flow rate permits. The recycle treatment plant system consists of a single-impeller flash mixer first stage and a twin-impeller second stage flocculation mixer followed by an incline plate settler to enhance sludge removal. Anionic polymer A6320 (A6320) is used as the primary coagulant. Anionic polymer A210 (A210) is used for the dewatering of sludge. Anionic polymer is stored and prepared in the utility building. Jar tests are performed to set the optimum dose. Typical dose rates of 0.5 to 2.0 milligrams per liter (mg/L) for the primary coagulant A6320. The SCADA system sends an alarm to the operator when effluent turbidity from the recycle wash water treatment plant is 2.0 nephelometric turbidity units (NTU) or higher. Flow and turbidity are monitored and recorded continuously.

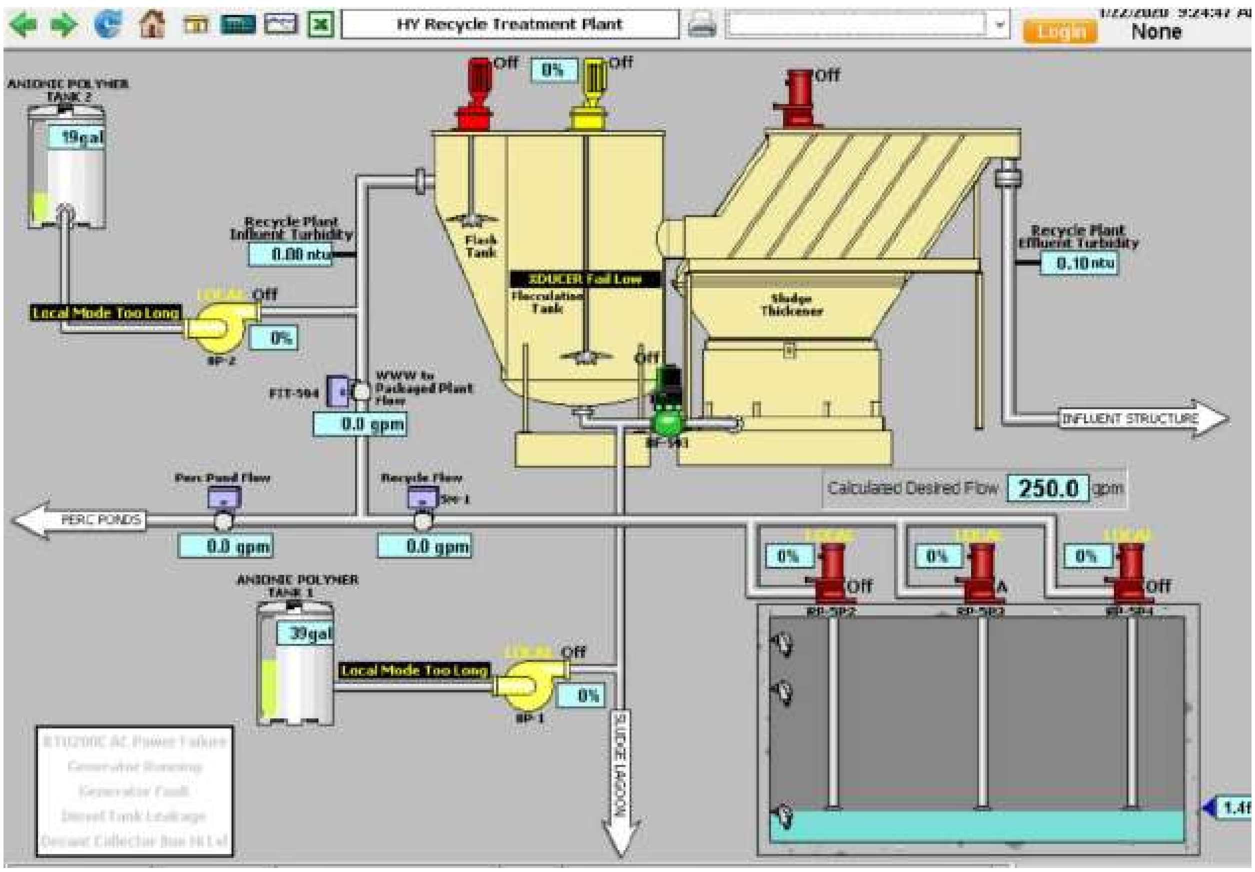

Figure 1 The Wash Water Recycled Water Treatment Plant

The recycle treatment plant effluent is continually monitored by SCADA and grab samples are pulled daily for comparison. The effluent of the recycling plant is returned to the influent structure ahead of the chemical feed application. The recycle treatment plant is shown in the above Figure. The SCADA system enables the operator to:

- Adjust the operating condition of the three washwater return pumps.
- Control and monitor the recycle flow rate.
- Control and monitor the flocculator (mixer) drive.
- Control and monitor the polymer feed system.
- Control and monitor the waste discharge valve and related instrumentation monitoring
- and online analysis results.

The chlorine is injected after combined filter effluent before going to the Agate storage reservoir. Solids removed from the sedimentation basin flow by gravity to four sludge lagoons located at the northwest end of the plant site. Each lagoon has a capacity of 40,000 cubic feet and provides one hundred thirty-one days of sludge storage at a nominal plant capacity of 12 MGD. Sludge production at a plant flow rate of 12 MGD is estimated to be approximately 305 cu ft of dry sludge per day. On a monthly basis, the sludge in each lagoon is dried and removed by mechanical means for off-site disposal to a landfill. The supernatant from the sludge ponds flows by gravity to a collection box.

From the collection box, the supernatant is pumped to the wash water basin influent. A process schematic of the Hinckley WTP is presented in ***Figure 2***.

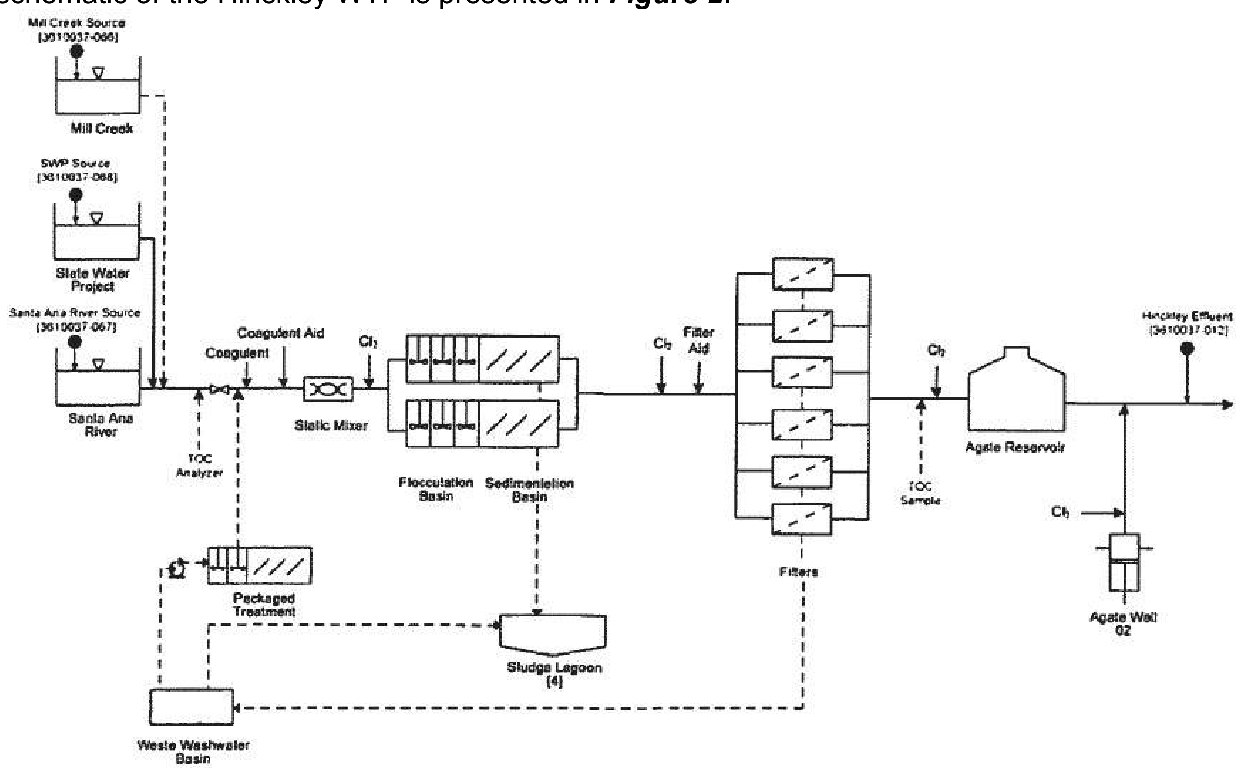

Figure 2 Hinckley WTP

2.2 CIP Recommendations

2.2.1 CIP 1

The package wash water treatment plant is installed to treat the filter wash water effluent. The operator shared the concern with the age of installed equipment, the installed capacity, and operation complexity. The replacement unit (s) may be furnished with the new technology to provide adequate capacity. The operator thought that a dissolved air flotation or filter technology might be applicable. The cost of the proposed replacement technology could be $2M for the 1.5 mgd capacity. The final capacity would be 4.5 mgd with a refurbishment price tag of $6 M.

2.2.2 CIP 2

Capacity upgrade. The current plant capacity is sufficient for the current demand. However, with the population increase in the future, the demand increase would require additional plant capacity. Therefore, the capacity could be increased in 12 mgd increments in phases 1 and Phase 2. The Phase 1 increase could cost $25M, with a similar estimate for Phase 2.

2.2.3 CIP 3

Remove provisional support of seismic flex tend at the reservoir and provide permanent support. The cost of this upgrade maybe $25,000.

2.2.4 CIP 4

Upgrade MCC and replace dated electrical equipment at the cost of $500,000.

2.2.5 CIP 5

Replace dated mechanical equipment at the cost of $1M.

2.3 Hinckley Water Treatment Plant Facilities

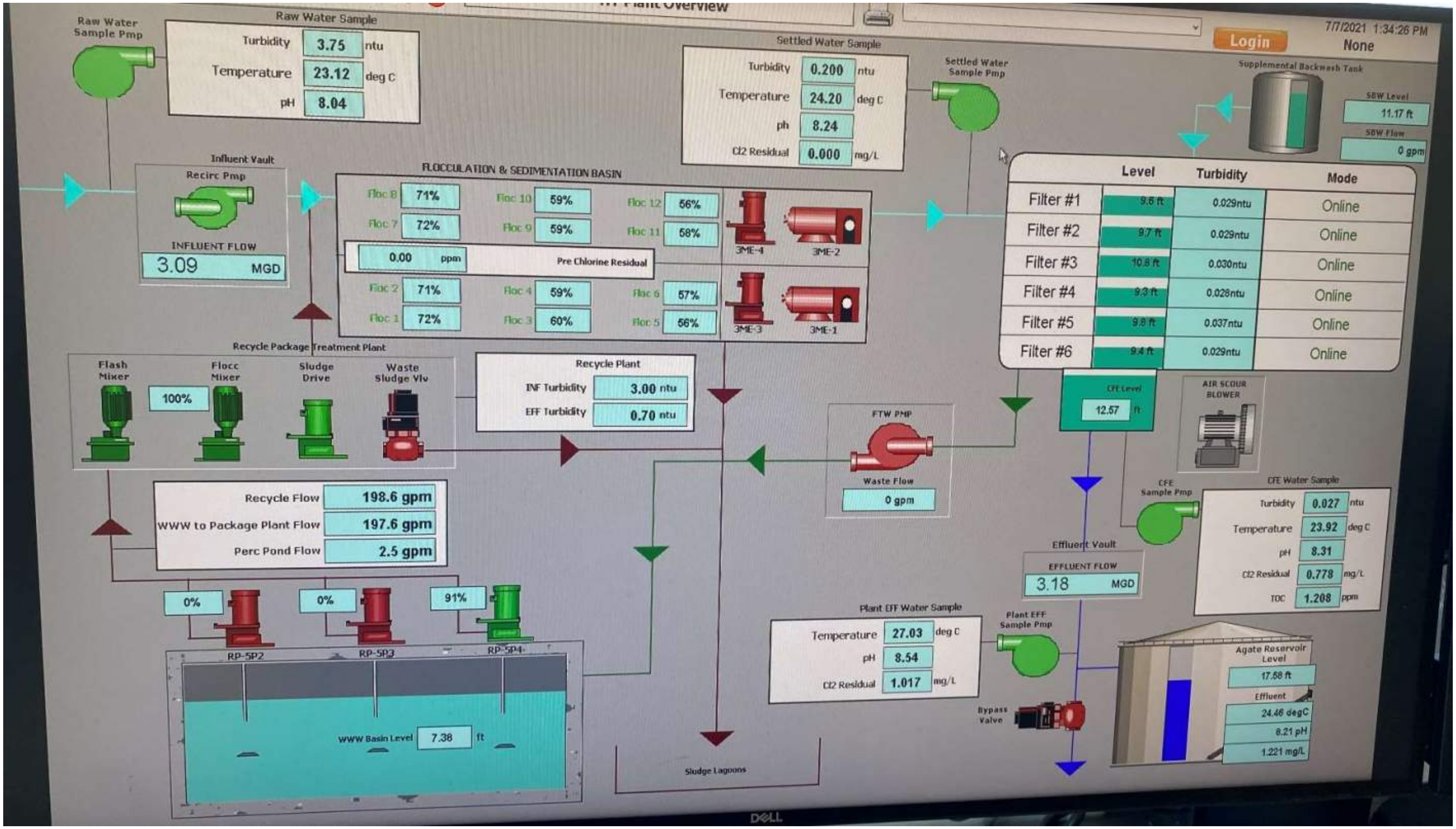

Figure 3 The City of Redlands Hinckley WTP SCADA

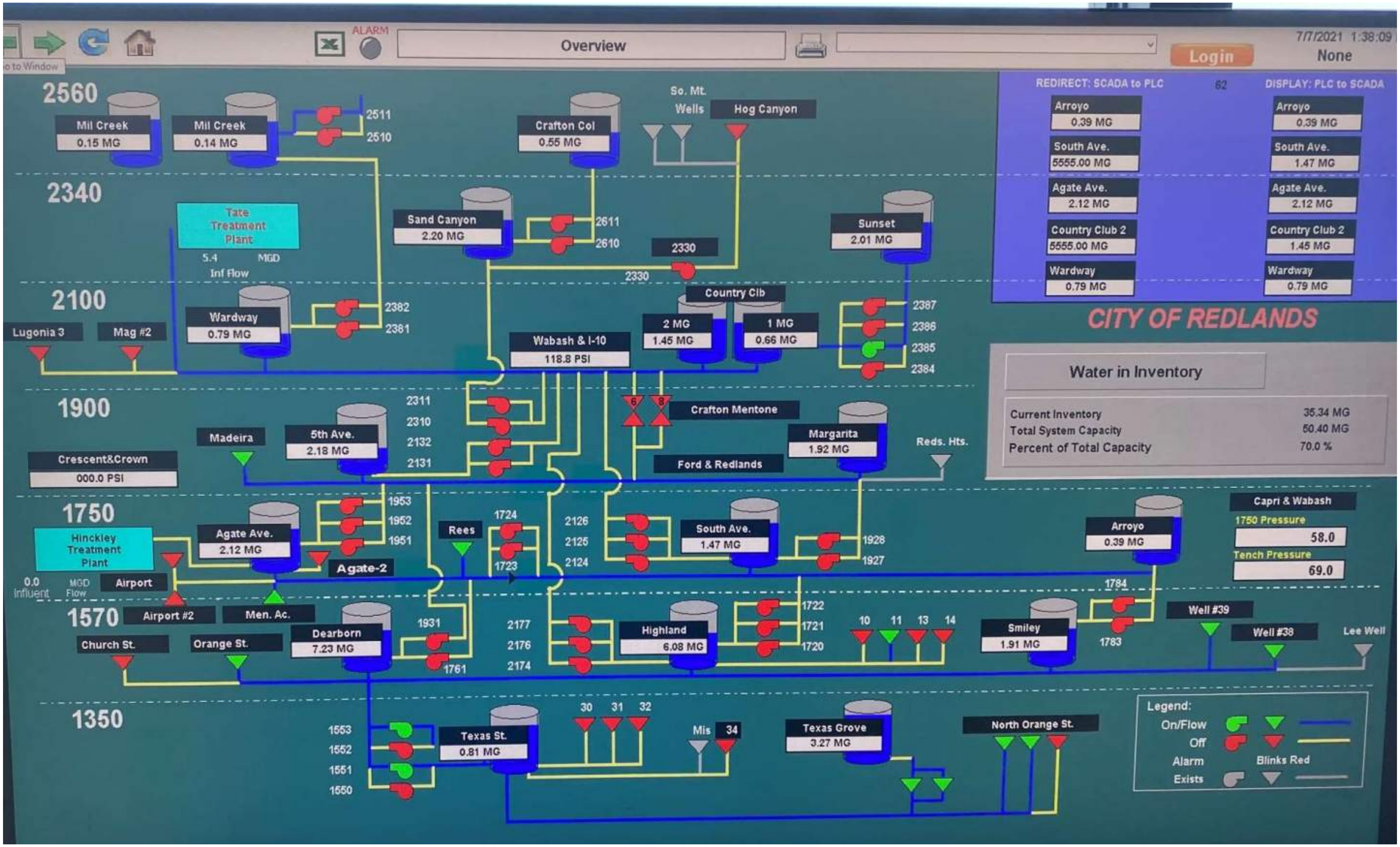

Figure 4 The City of Redlands Water Supply System Supervisory and Data Control System

Figure 5 Hinckley WTP Flocculation and Sedimentation Basin Inlet

Figure 6 Hinckley WTP Filters

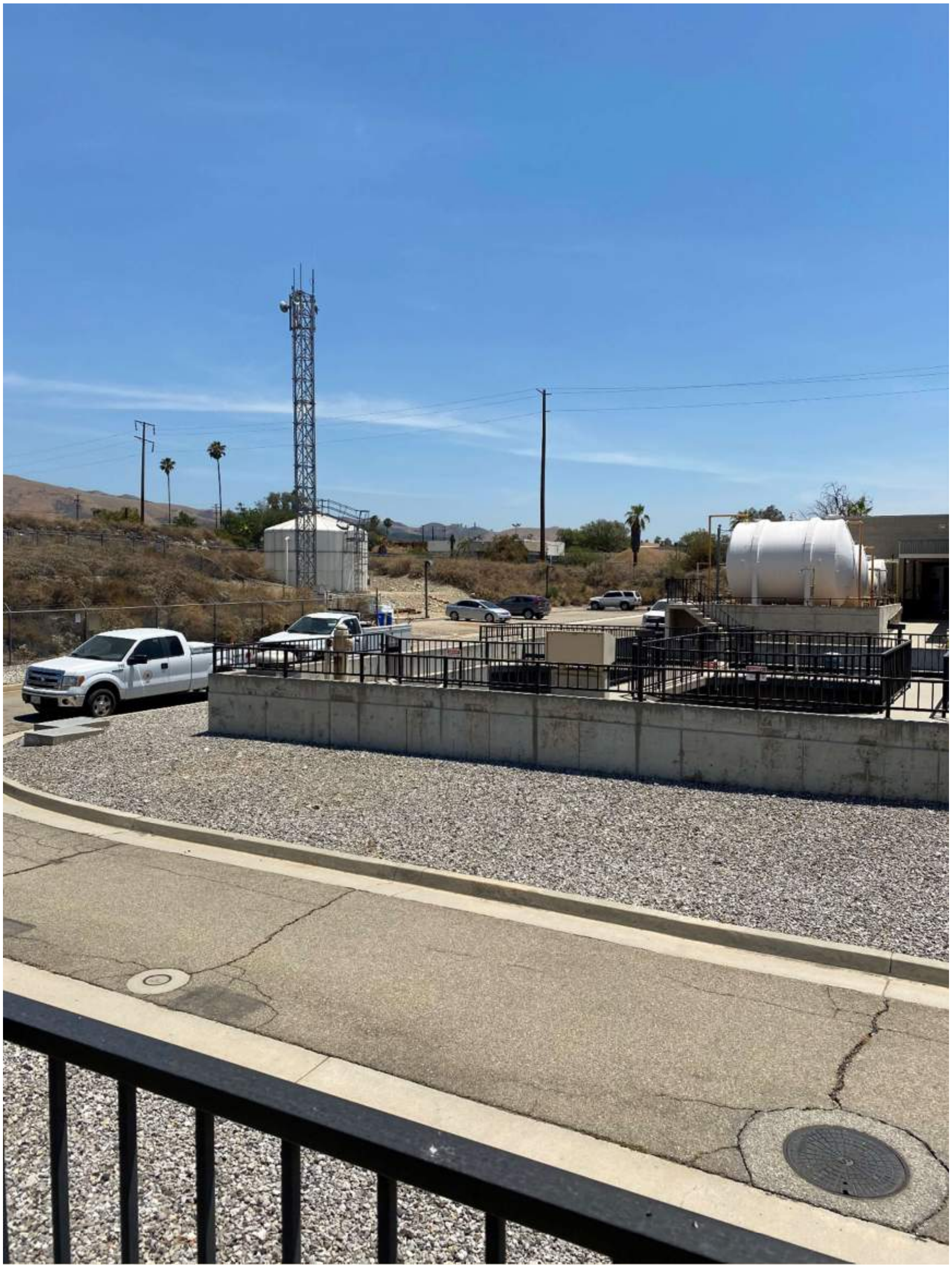

Figure 7 Hinckley WTP Filters Layout

Figure 8 Hinckley WTP Flocculation/Sedimentation/Settling

Figure 9 Hinckley WTP Flocculation/Sedimentation/Settling

Figure 10 Settler Sludge Lagoons

Figure 11 Hinckley WTP Filter Backwash Basin

Figure 12 Hinckley WTP Generator Building

2.3.1 Agate Reservoir

Filtered water is discharged to the 3-million-gallon (MG) Agate Reservoir before entering the utility distribution system. The Agate Reservoir includes a baffle system to increase detention time. An engineered blending plan was added to blend three wells' water with the Hinckley WTP effluent. These blending wells are Agate No. 1, Agate No. 2, and the Crafton, well as combining sources. Agate reservoir is a 3 mg steel tank with an average detention time of 7.2 hrs, a nominal detention time of 5 hours, and an ultimate detention time of 3.6 hrs.

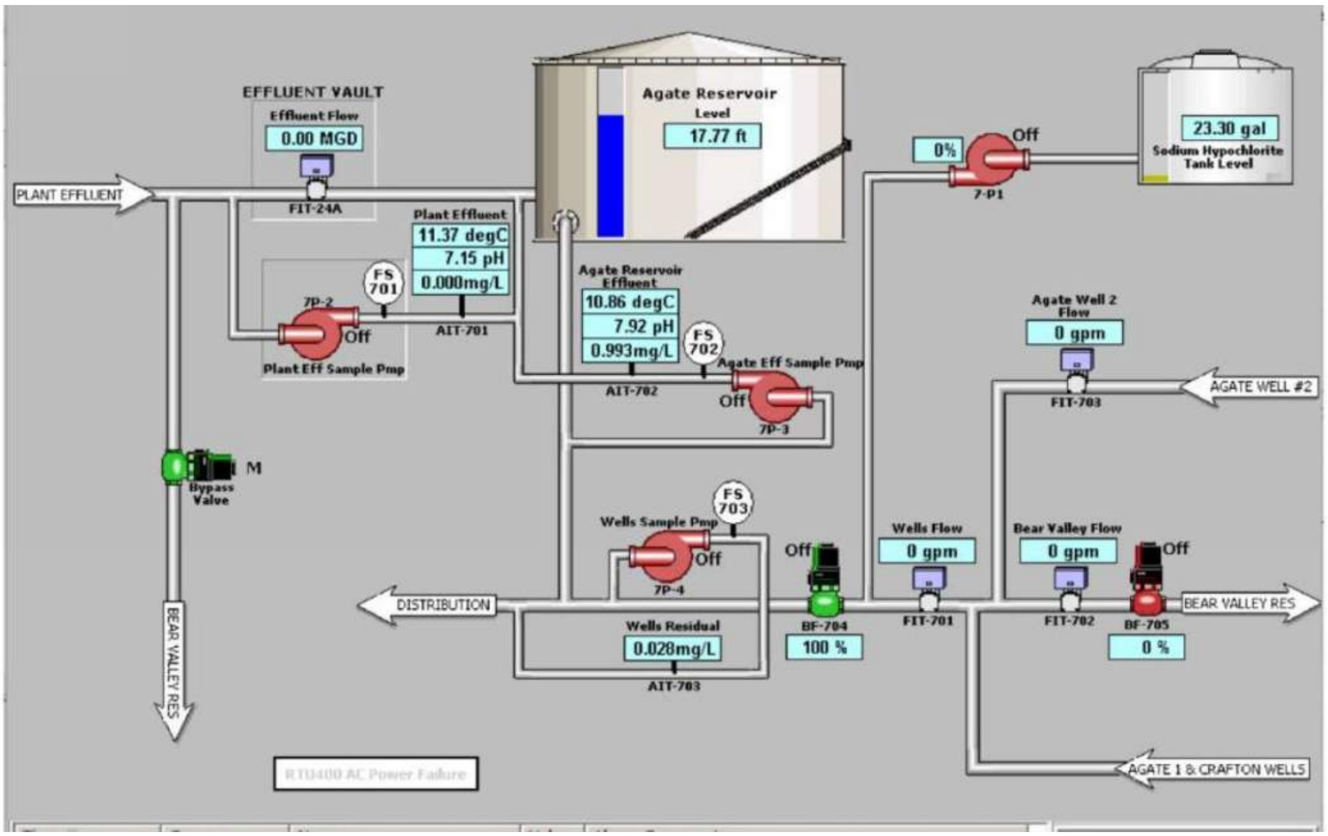

Figure 13 Agate Reservoir SCADA Layout

Figure 14 Agate Reservoir and Booster PS

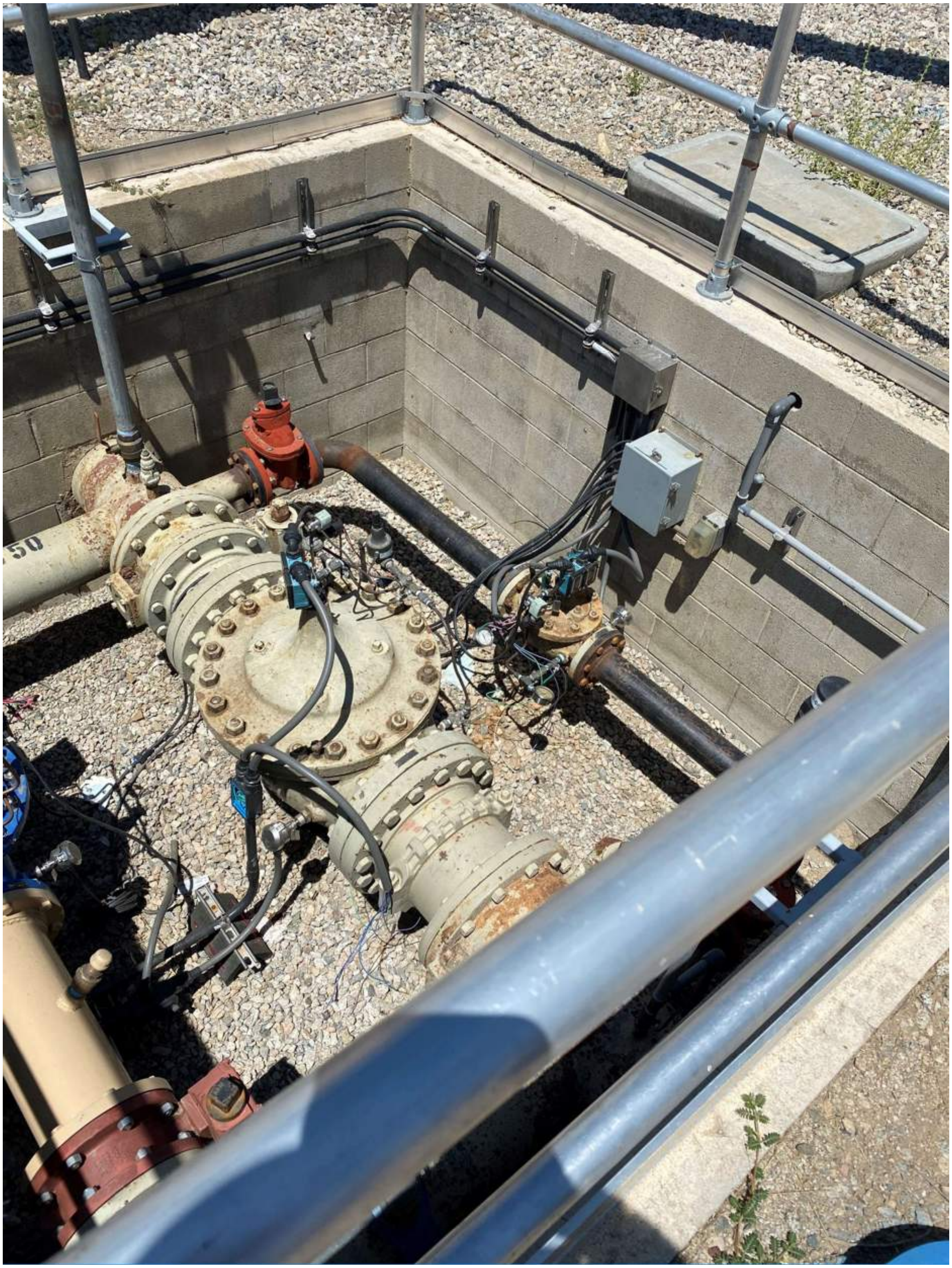

Figure 15 Cla-Valve Flow Control Valve at Agate Reservoir

Figure 16 Agate Reservoir Seismic Expansion Unit Supported Off the Concrete Slab

Figure 17 Agate Reservoir Roof

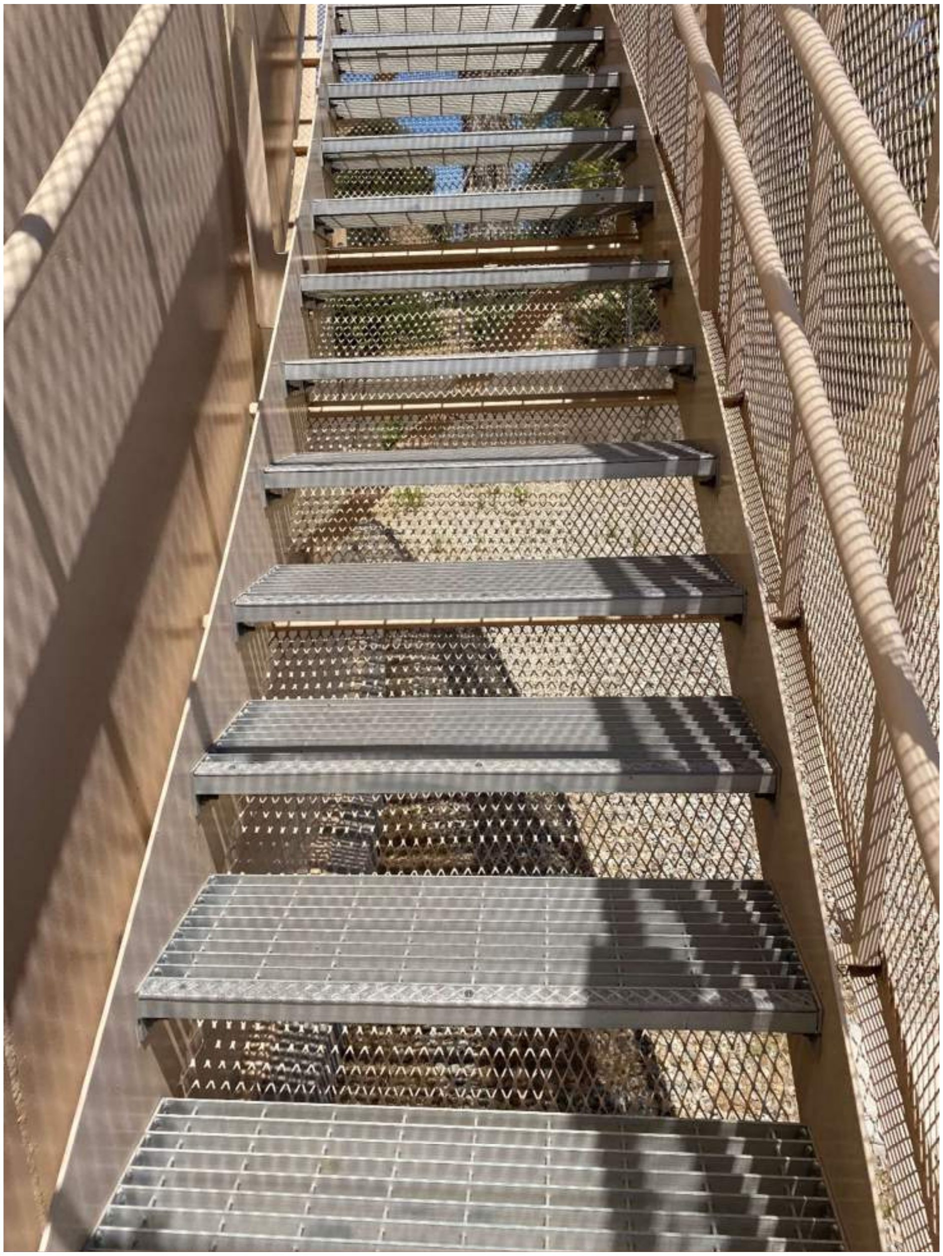

Figure 18 Agate Access Stairway with Partial Enclosure Safety Cage

Figure 19 MCC Center for Agate Reservoir PS

Figure 20 Agate Reservoir Pre-Treatment

Figure 21 Disinfectant Storage at Agate Site

Figure 22 Chemical Storage

Figure 23 Booster PS Discharge Valves and Pressure Gauges

3 The Tate Water Treatment Plant

The Tate Water Treatment Plant (Tate WTP) was initially commissioned in 1967 to treat surface water from Mill Creek. Since then, several process upgrades have been implemented, with the latest being completed in June 2005. Tate WTP consists of chemical treatment, chemical mixing through an inline static mixer, flocculation, and sedimentation through two EIMCO reactor clarifiers (each equipped with four adjustable speed, vertical turbine flocculation), filtration with four dual media gravity filters (anthracite over sand), and chlorine disinfection. The maximum plant flow rate is 20 MGD with all filters online and 14.9 MGD with one filter offline for backwashing.

A process schematic of the Tate WTP is presented on a SCADA screen in the Figure below.

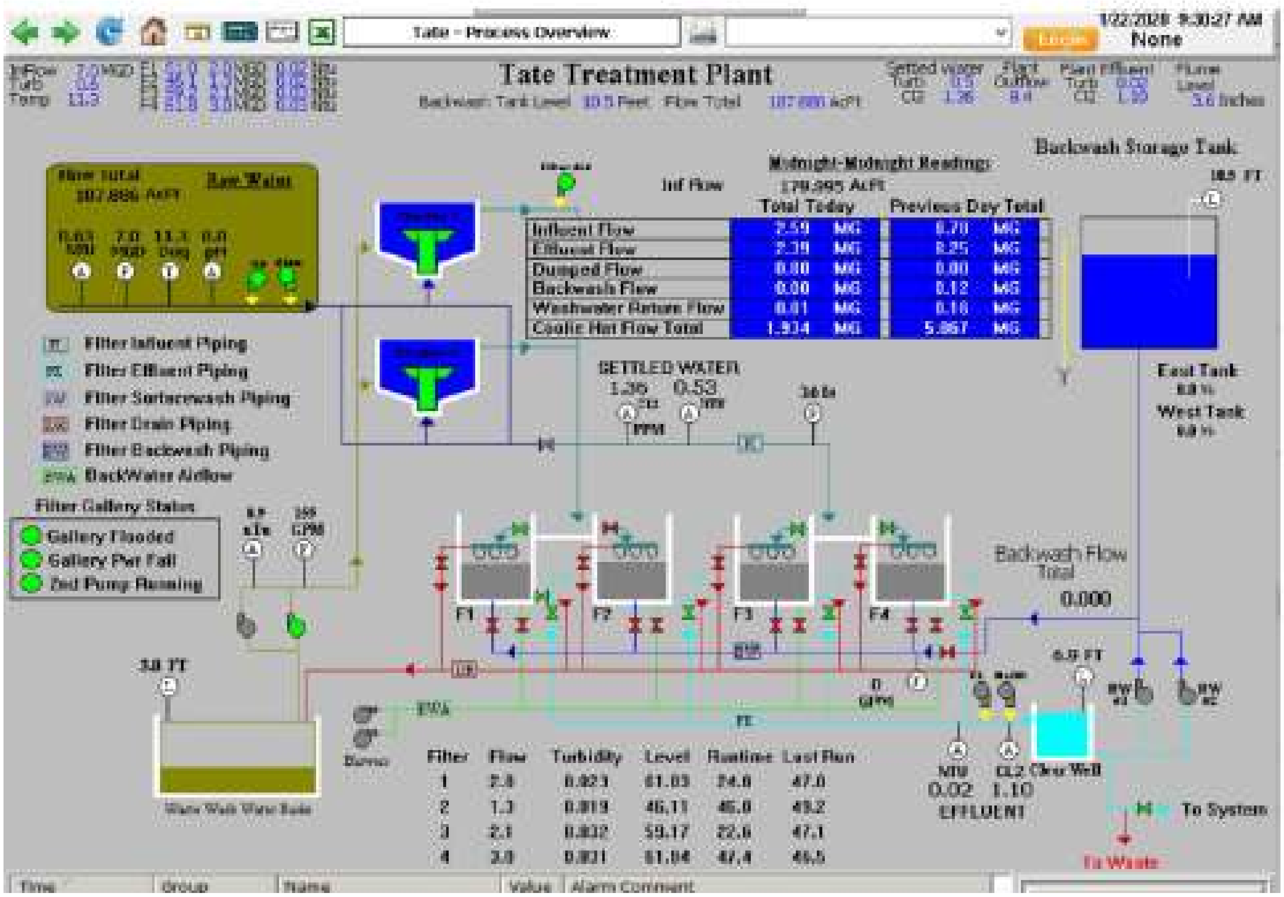

Figure 24 Tate WTP SCADA Screen

Historically, Tate WTP had received its raw water supply exclusively from Mill Creek. However, the availability of the Mill Creek supply had been reduced during drought periods and is subject to interruptions during high turbidity events. To increase the raw water supply reliability, the City obtained a permit amendment from the CDPH to treat State Water Project (SWP) and Santa Ana River (SAR) water at Tate WTP.

3.1 Treatment Process

The first step in the conventional water treatment is the pre-treatment. Flow is conveyed and controlled through off-site piping and valving, influent flowmeter and control valve, influent sampling system, influent static mixer, and flow splitter box. Chemical mixing of primary coagulant, aluminum chlorohydrate (ACH), and coagulant aid (C-308P) are achieved by the static mixer before reactor clarifiers. Chlorine is also added for disinfection.

The second step is coagulation, and flocculation is when suspended solids get together to form larger particles called the "floc." Flocculation and sedimentation are achieved through two EIMCO reactor clarifiers, each equipped with four adjustable speed vertical turbine flocculation that operates in a continuous operation mode.

The third step is sedimentation. In this step, larger particulates settle to the floor of the sedimentation basin. The chemicals added in the first step assist the sedimentation process in helping the particulates settle at the floor of the settling tank. Solids separation is achieved through the sedimentation basins located close to the flocculation basins. There are two settlers, ten mgd in capacity each, 106 ft in dia, four flocculation, vertical turbine type, adjustable frequency drive, 5 HP motor, 30 min detention time, 50 ft in dia with 3.2 hr detention time.

The fourth step is filtration. The media filtration system consists of dual media filters (anthracite over sand), a filter backwash system, and an air scour system operated continuously. During normal operations, the total flow rate is equally distributed across the four online filters, and filters are only taken offline for backwash and maintenance. The filtered air scours system and backwash system operate during the filter backwash process. ClariFloc A-6320 anionic polymer (filter aid) is added upstream of dual media filters and chlorine for disinfection as needed. There are four filters, each five mgd in capacity, with a six mgd design loading rate, 13.75x41.8 ft with 9.5 side water depth. Media surface area is 575 sqft, with 30 inches of anthracite and 8 inches of sand depth.

The fifth step is post-disinfection. Again, the chlorine is added to treated water to complete disinfection before discharge into the distribution system.

The sixth step is the backwash. The particulates and polymer in the filters are attached to media and then clog them up. The backwash process starts by draining the filters to pre-set point and then air scouring for approximately ten minutes, breaking down any particulates stuck in the system. Finally, backwashing is performed to clean the filters to keep their continuous operation and prevent water from overflowing. The recycled wash water system consists of a waste wash water basin, recycle pump station, recycle wash water treatment plant, and a polymer feed room. The recycled wash water system receives flow primarily from the waste wash water discharged during filter backwashing.

3.2 CIP Recommendations

3.2.1 CIP 1

EIMCO settlers could be refurbished, and corrosion of the mechanical equipment could be addressed. Just sandblasting and repainting both using would be appx $50,000. The replacement cost would be $1M.

3.2.2 CIP 2

Capacity upgrade. The current plant capacity is sufficient for the current demand. However, with the population increase in the future, the demand increase would require additional plant capacity. Therefore, the capacity could be increased in 10 mgd increments in phases 1 and Phase 2. The Phase 1 increase could cost $20M, with a similar estimate for Phase 2.

3.3 Tate Water Treatment Plant Facilities

Figure 25 Historical Artifact Wooden Pipeline Used at the District in the 19th Century

Figure 26 MCC and Communications Equipment

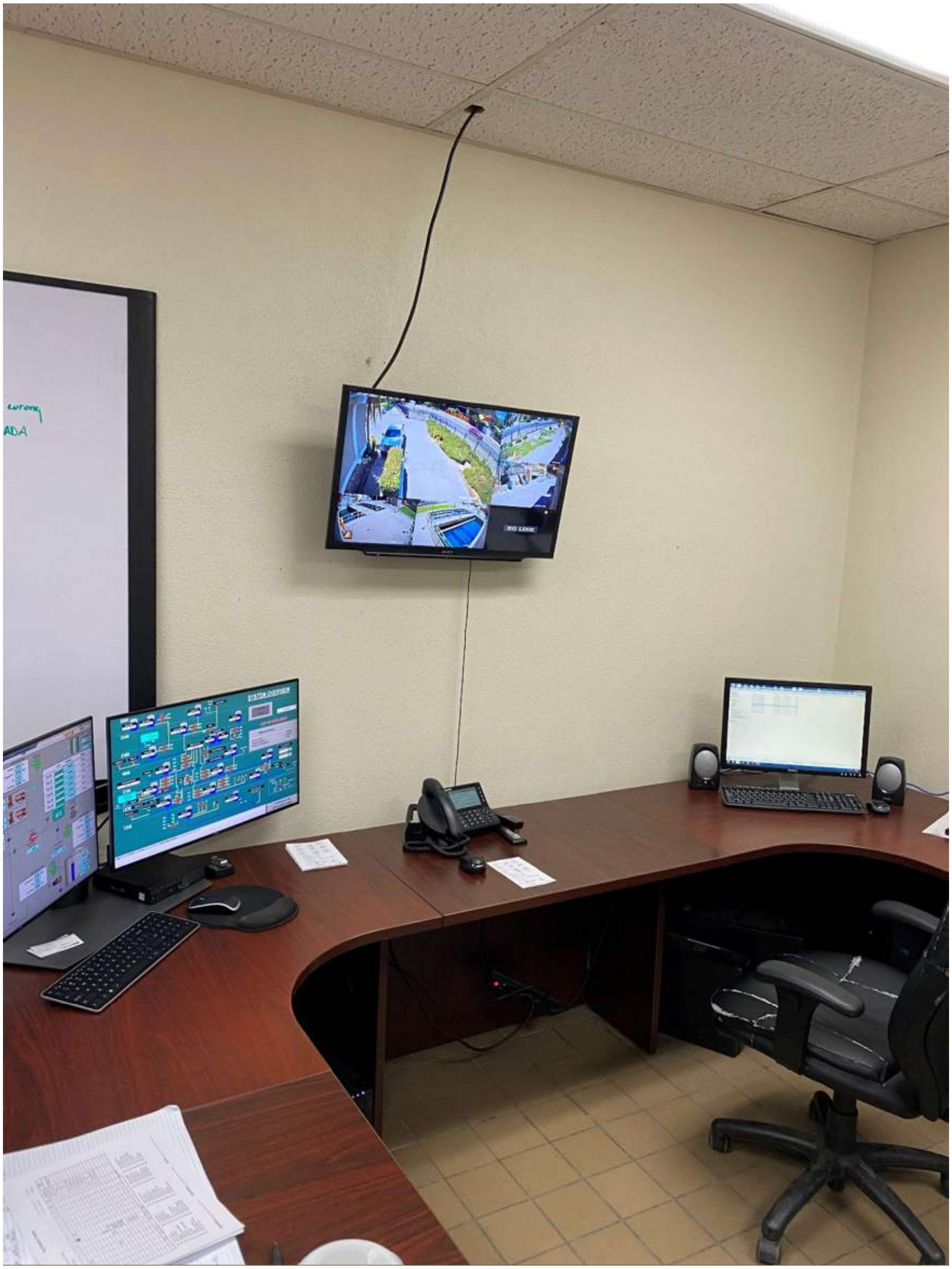

Figure 27 Control Room-Excellent Working Condition

Figure 28 Onsite Lab-In Excellent Working Condition

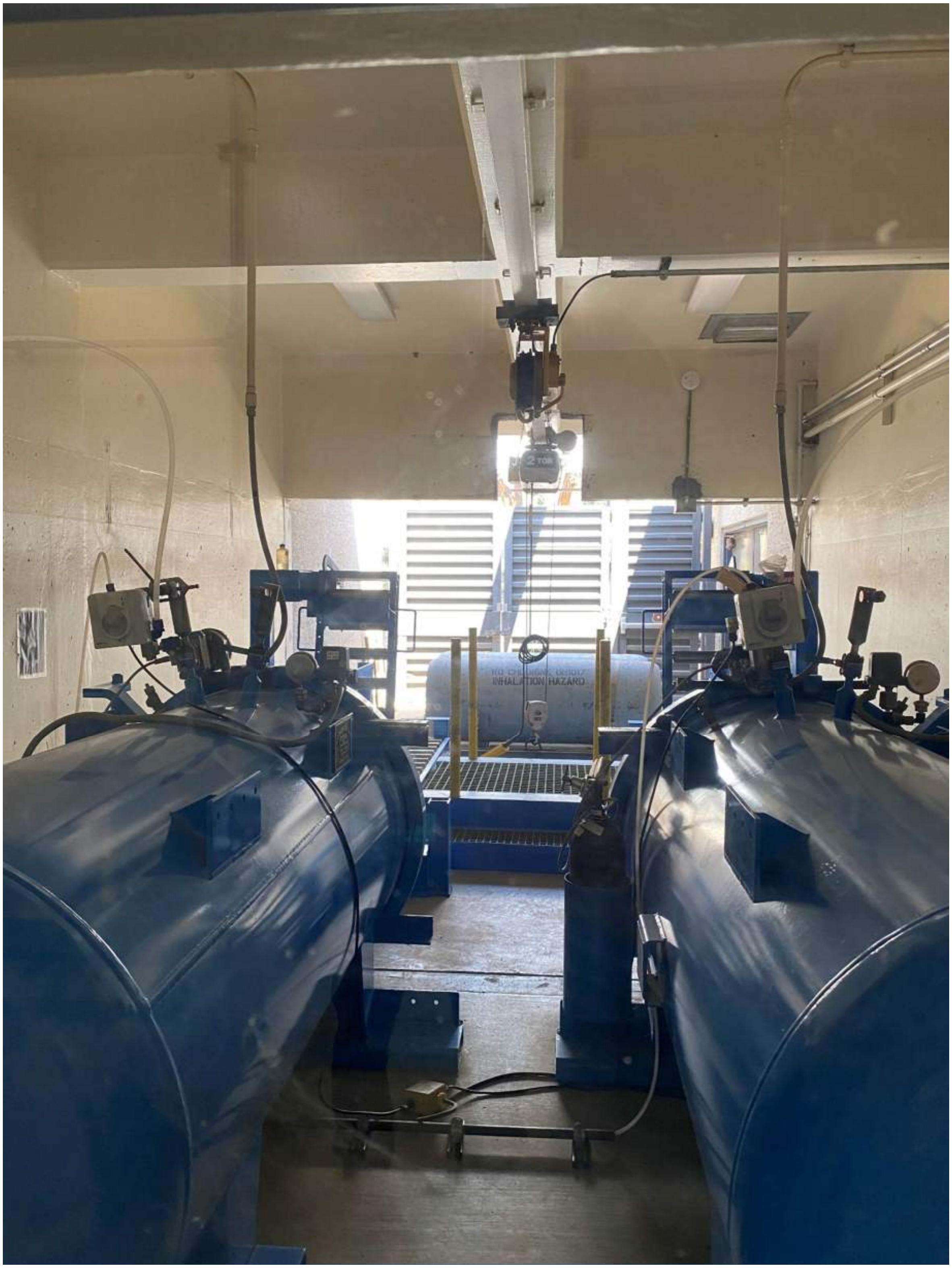

Figure 29 Two Chlorinators in Duty Mode, With Safety Enclosure

Figure 30 Safety Enclosure of the Chlorinators

Figure 31 Chlorine Cylinders Stored Onsite

Figure 32 Safety Cylinder Operation

Figure 33 Standby Diesel Generator Set

Figure 34 Effluent Reservoir Onsite

Figure 35 Chemical Storage Area with Concrete Containment-In Good Working Condition

Figure 36 Eimco Settler (Signs of Aging and Corrosion of Mechanical Equipment)

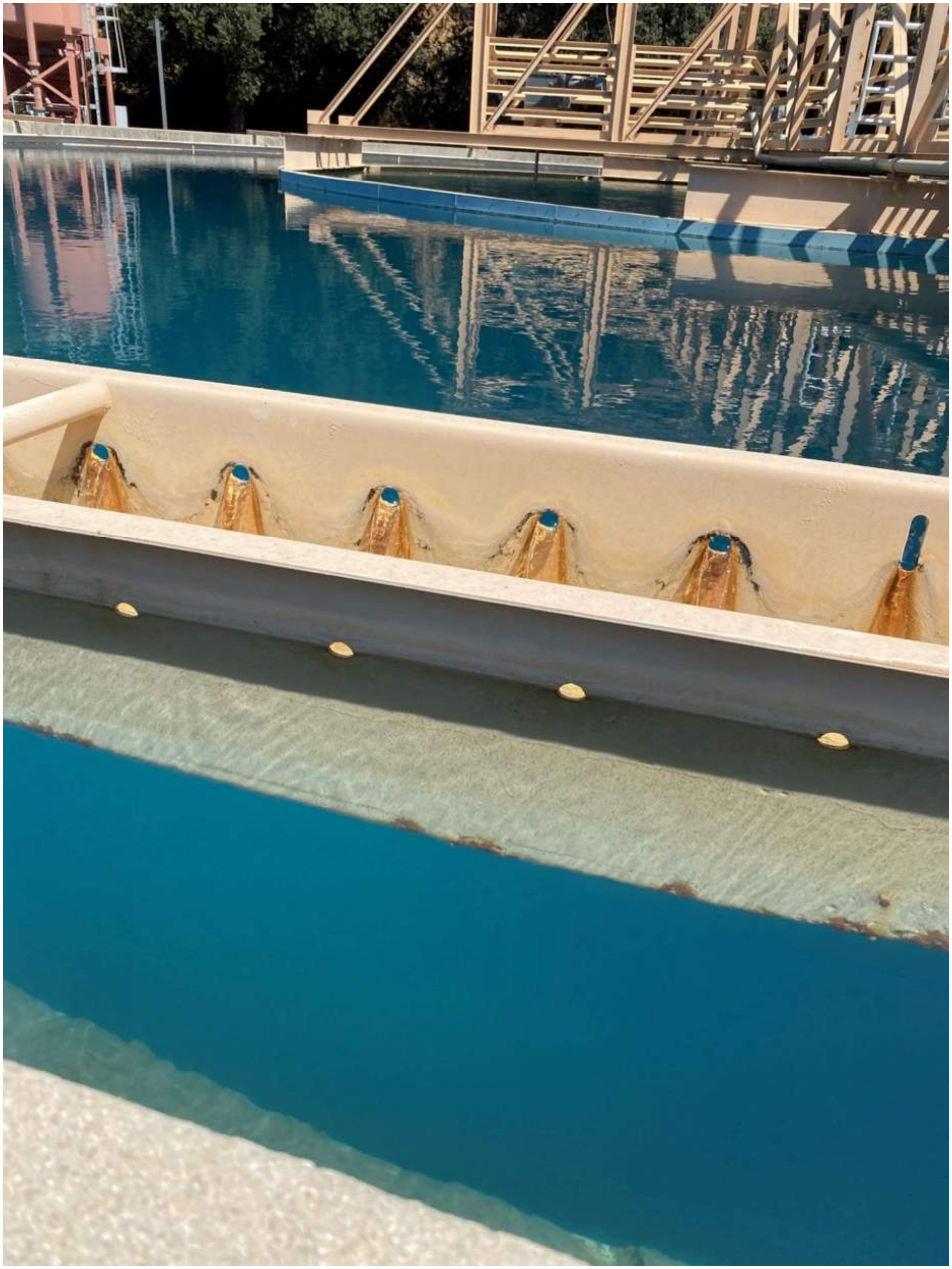

Figure 37 Eimco Settler (Signs of Aging and Corrosion of Mechanical Equipment)

Figure 38 Filter Equipment (Seems to be an Excellent Working Condition)

Figure 39 Filter Equipment (Seems to be in Excellent Working Condition)

Figure 40 Washwater Reservoir (seems to be in excellent working condition)

4 Sunset reservoir

The reservoir is located in a remote location on the southwest side of the City. The reservoir does not have a backup, and it has not been Inspected since it was erected in the 1970s. The reservoir is of welded steel construction; it is 60 ft tall and 90 ft in diameter and has 3 mg capacity. The seismic safety provisions are not visible and likely not provided. Also, corrosion is visible on equipment and reservoir shells. The corrosion protection cabinet is antiquated but is functional and in working condition. The inspection of the tank inside is not possible since the reservoir does not have a backup capacity. However, the diver could inspect by completing the videotaping and establishing the condition baseline without the reservoir operation interruption.

4.1 CIP Recommendations

4.1.1 CIP 1

Increase water supply system reliability by providing reservoirs with redundancy. Depending on the system hydraulic analysis, the pool may be replaced with a reservoir of the same volume at the exact location or similar location. The replacement cost is estimated at $6M.

Figure 41 Signs of Aging and Corrosion are Evident

Figure 42 Communication Antenna of Service Provider Erected Onsite

Figure 43 Steel Shell is Spot Corroded-Attached Ladders are Rusted Beyond Repair

Figure 44 Reservoir Corrosion Protection Cabinet (seems antiquated)

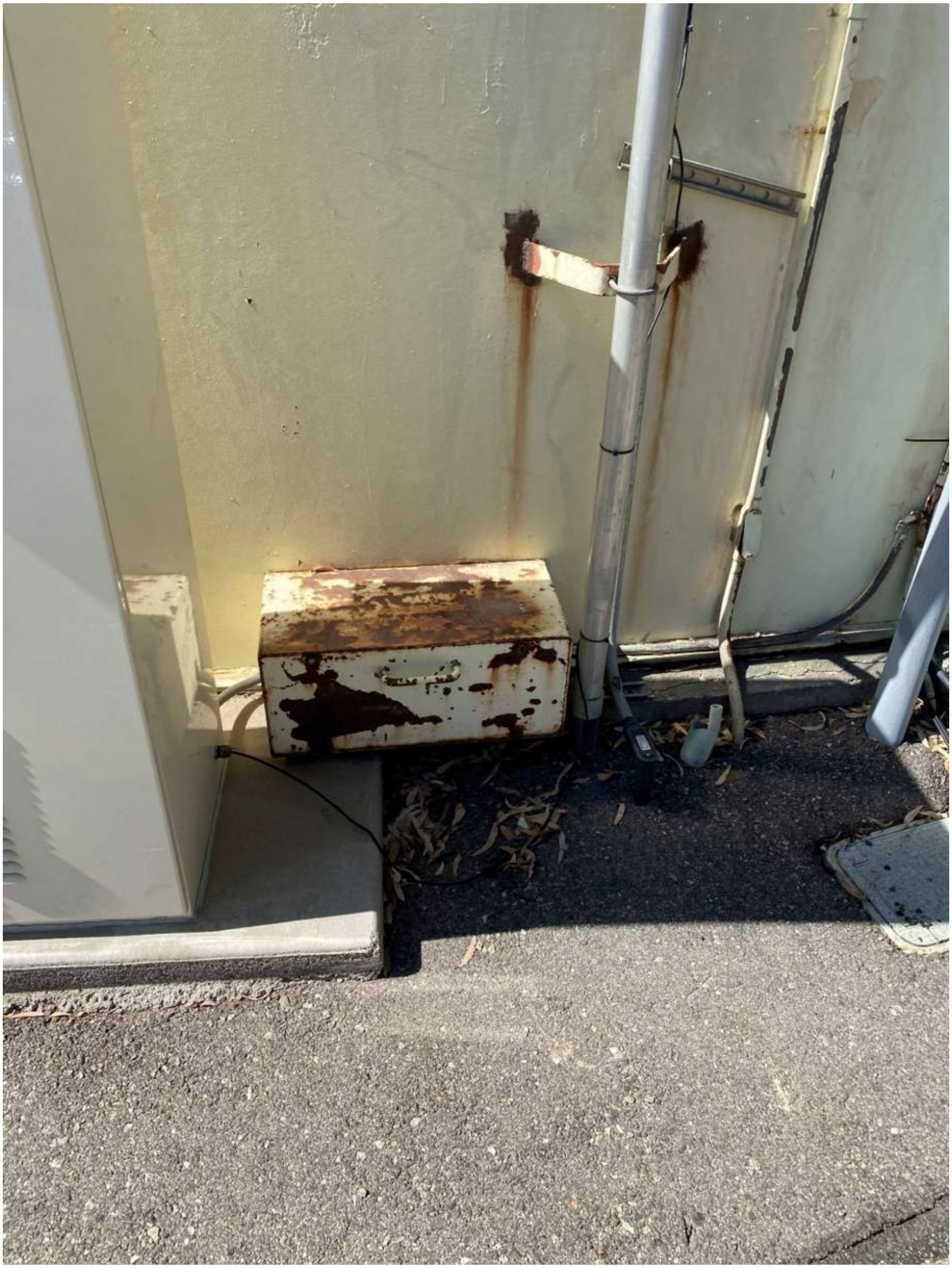

Figure 45 Close-Up of Corrosion Detail

Figure 46 Reservoir Base (surrounding asphalt is cracking and evidence of deterioration)

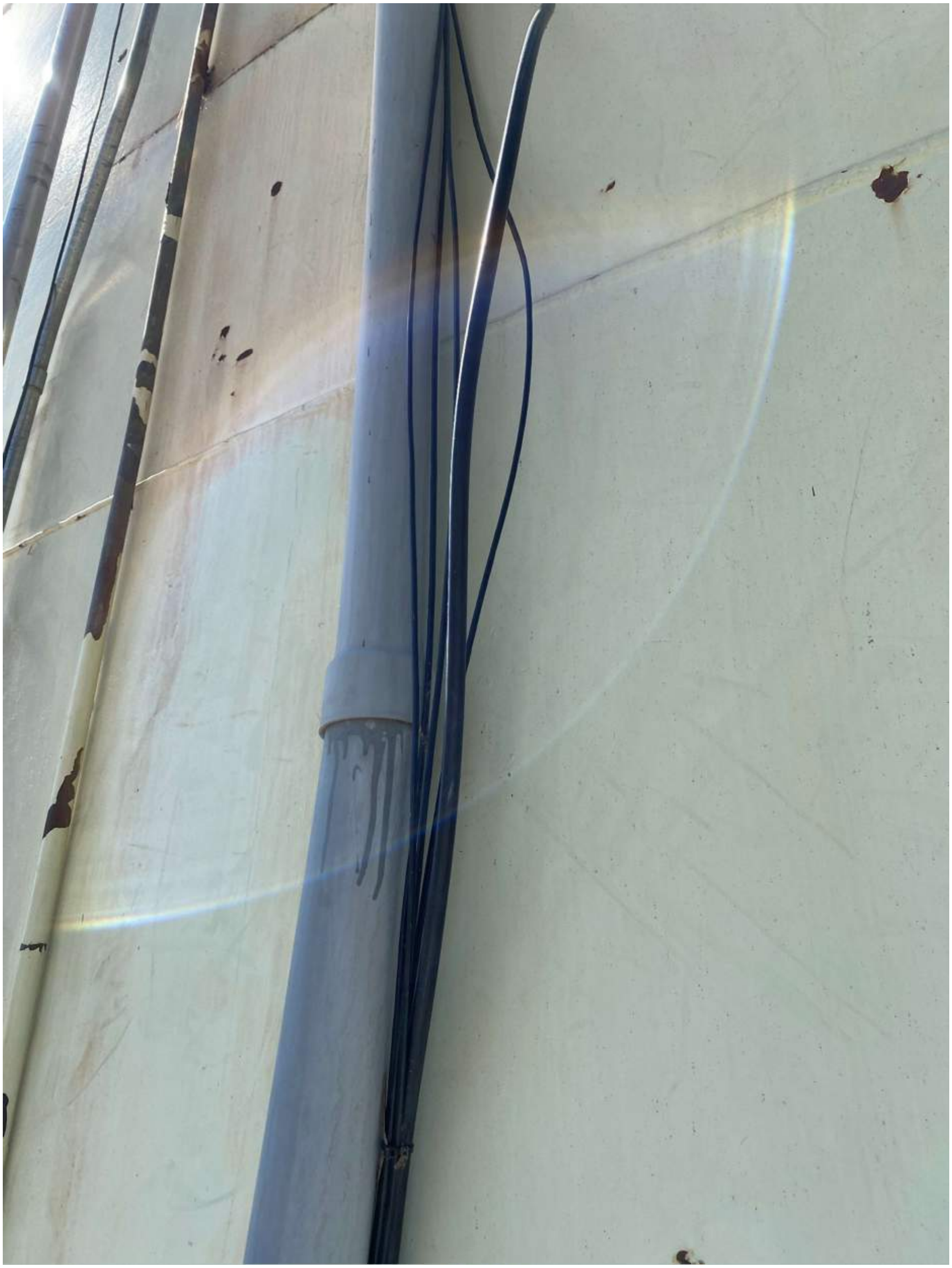

Figure 47 Reservoir Control Wires are Dangling on the Side of the Reservoir

Figure 48 Access Manhole Valve is Corroded

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APPENDIX B

Hydraulic Model Calibration Results

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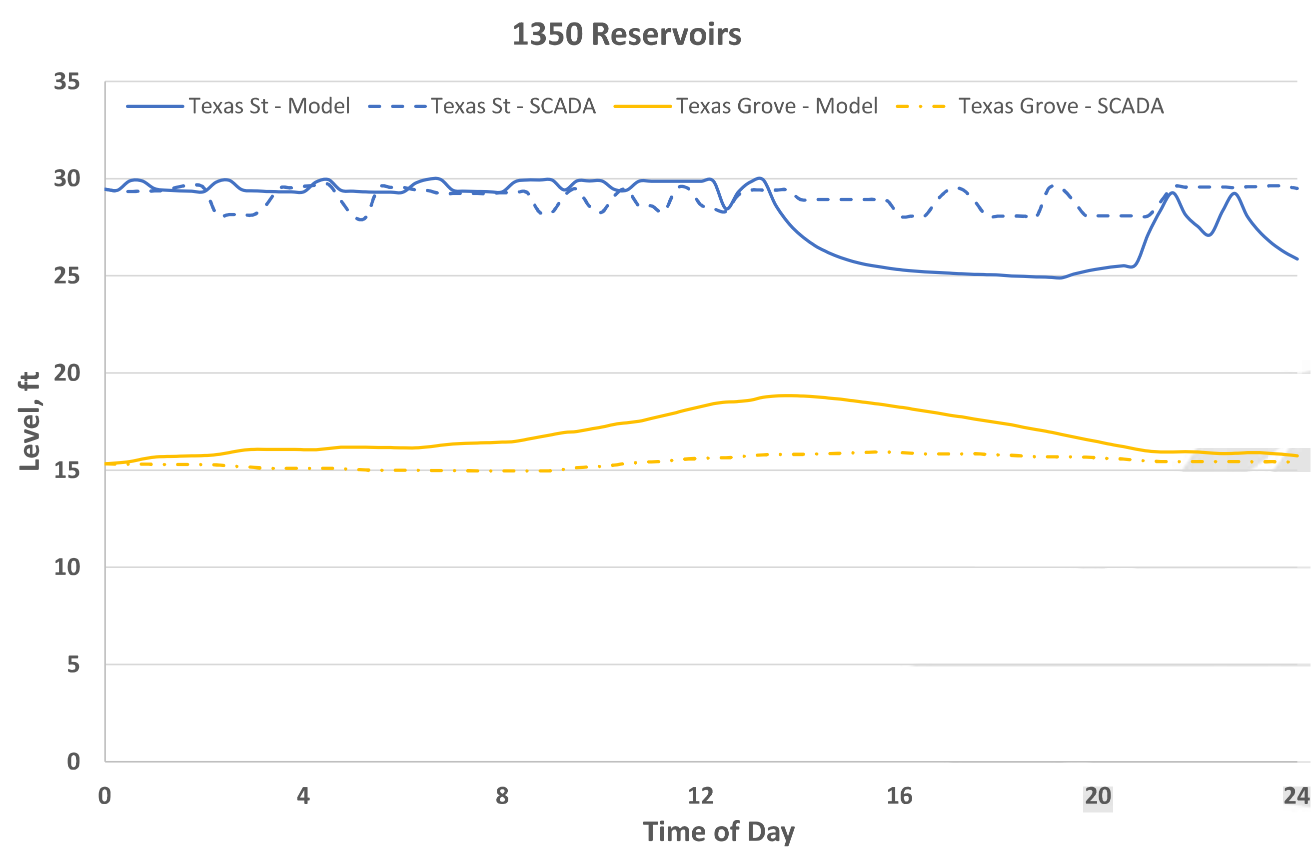
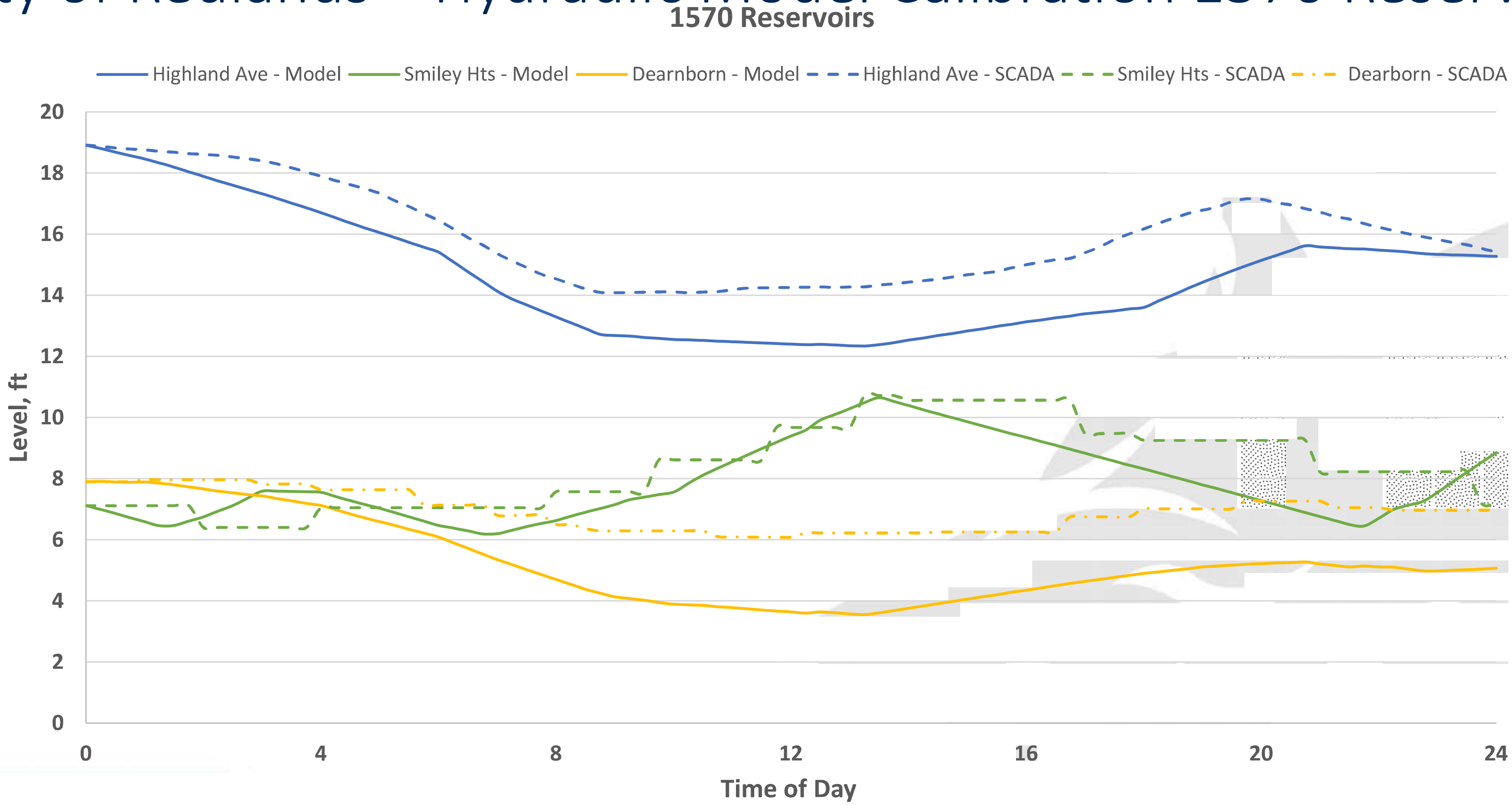
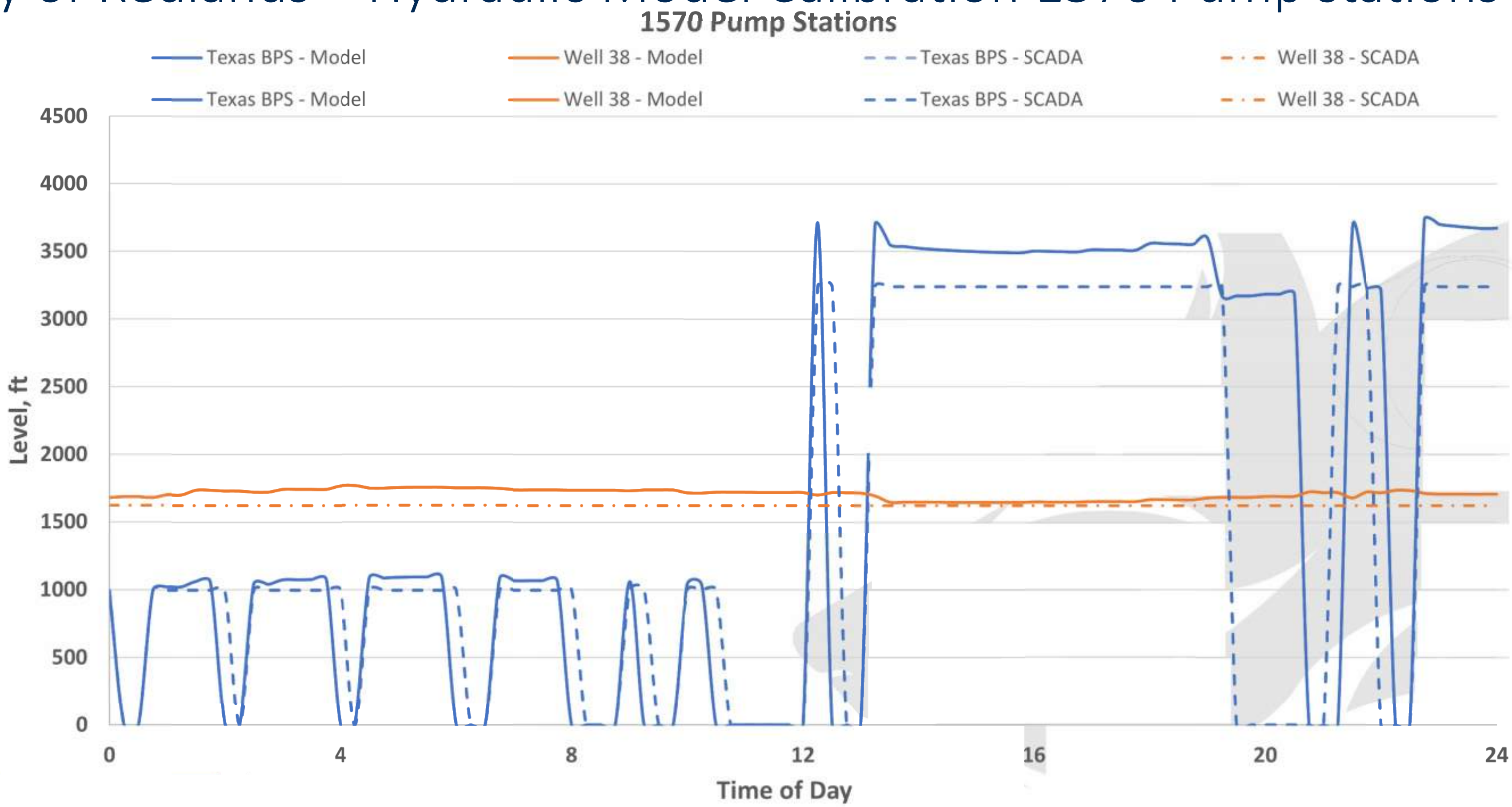
City of Redlands – Hydraulic Model Calibration 1350 reservoirs



City of Redlands – Hydraulic Model Calibration 1570 Reservoirs

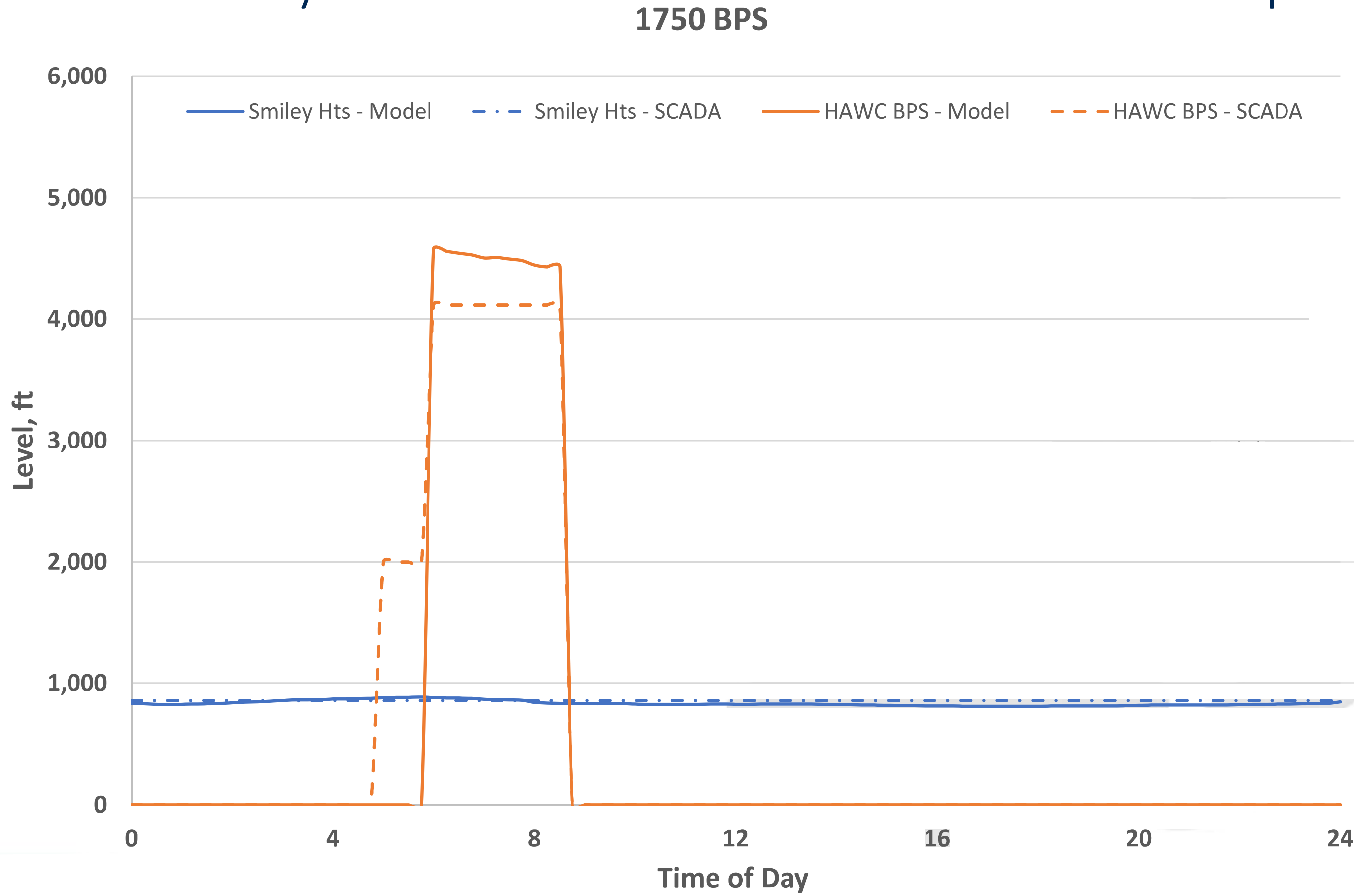


City of Redlands – Hydraulic Model Calibration 1570 Pump stations

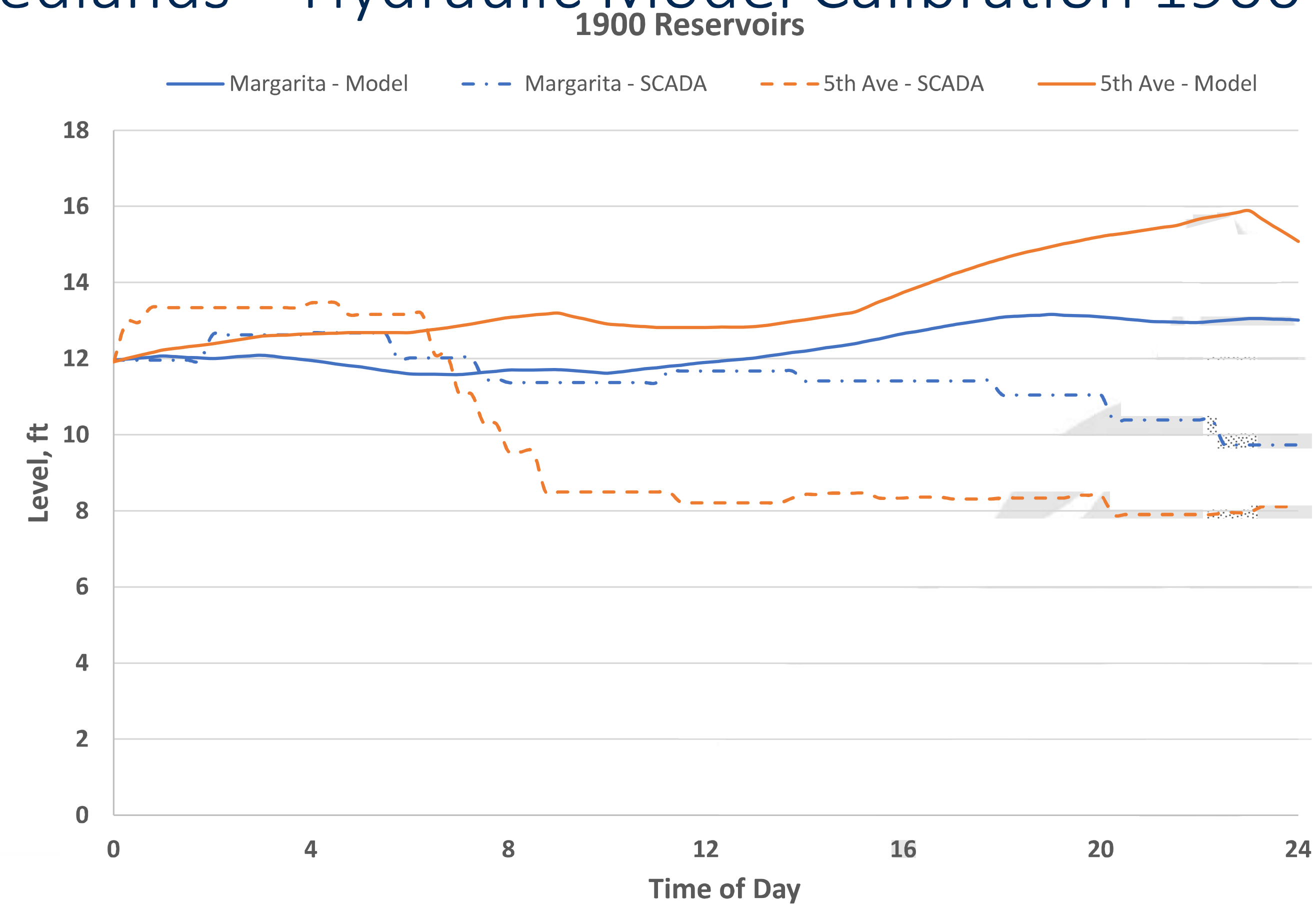


City of Redlands – Hydraulic Model Calibration 1750 reservoirs

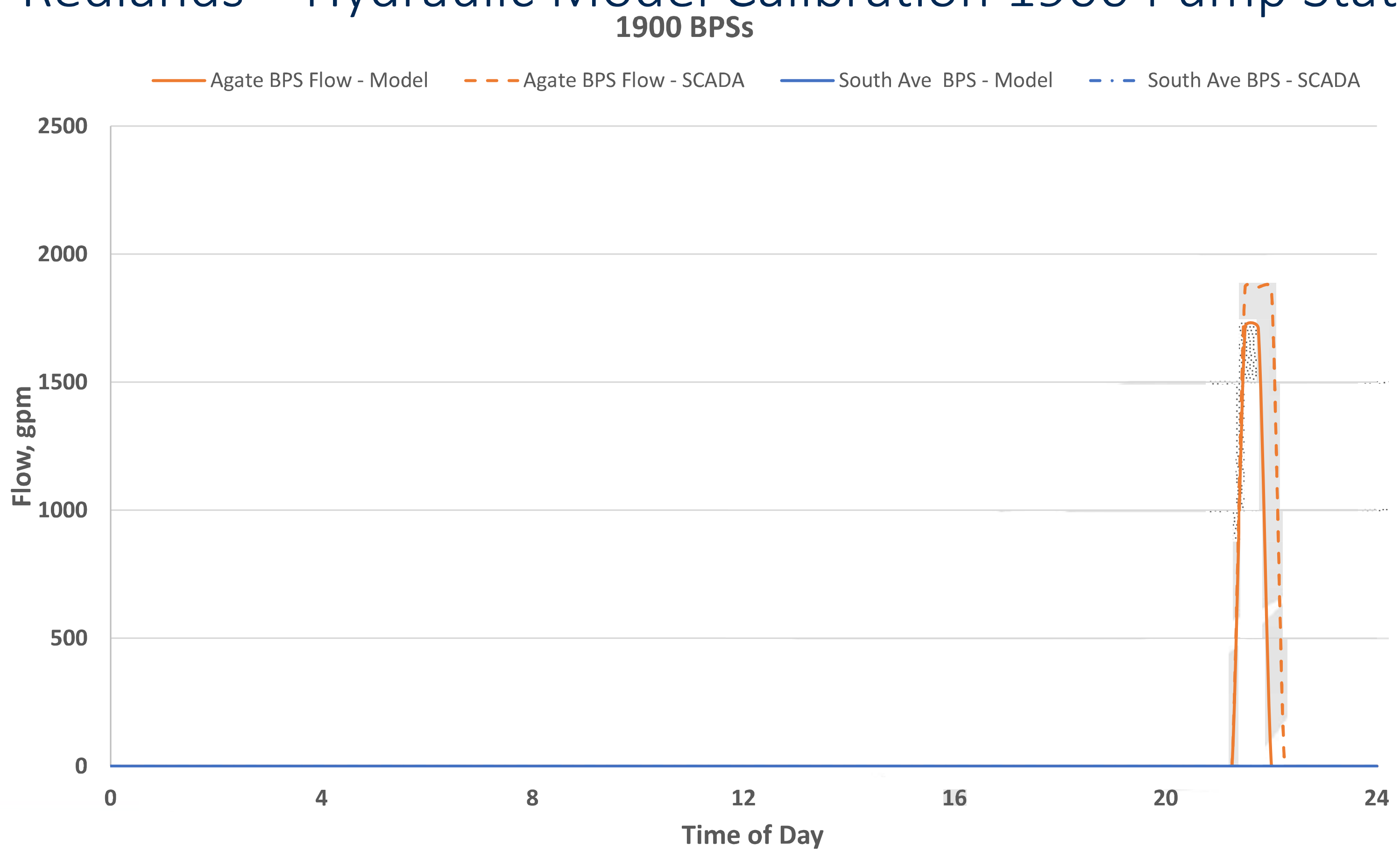
City of Redlands – Hydraulic Model Calibration 1750 Pump Stations



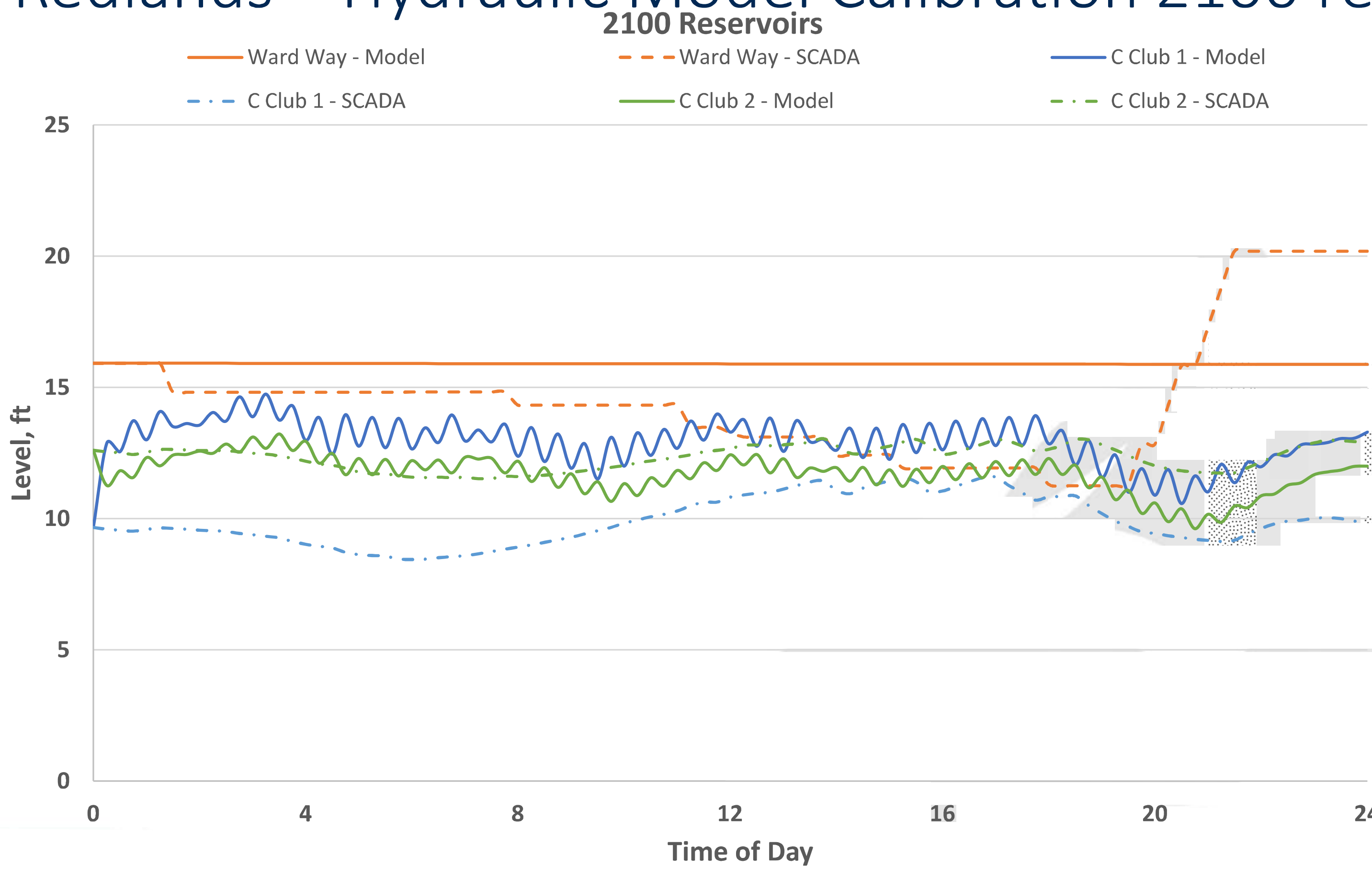
City of Redlands – Hydraulic Model Calibration 1900 reservoirs



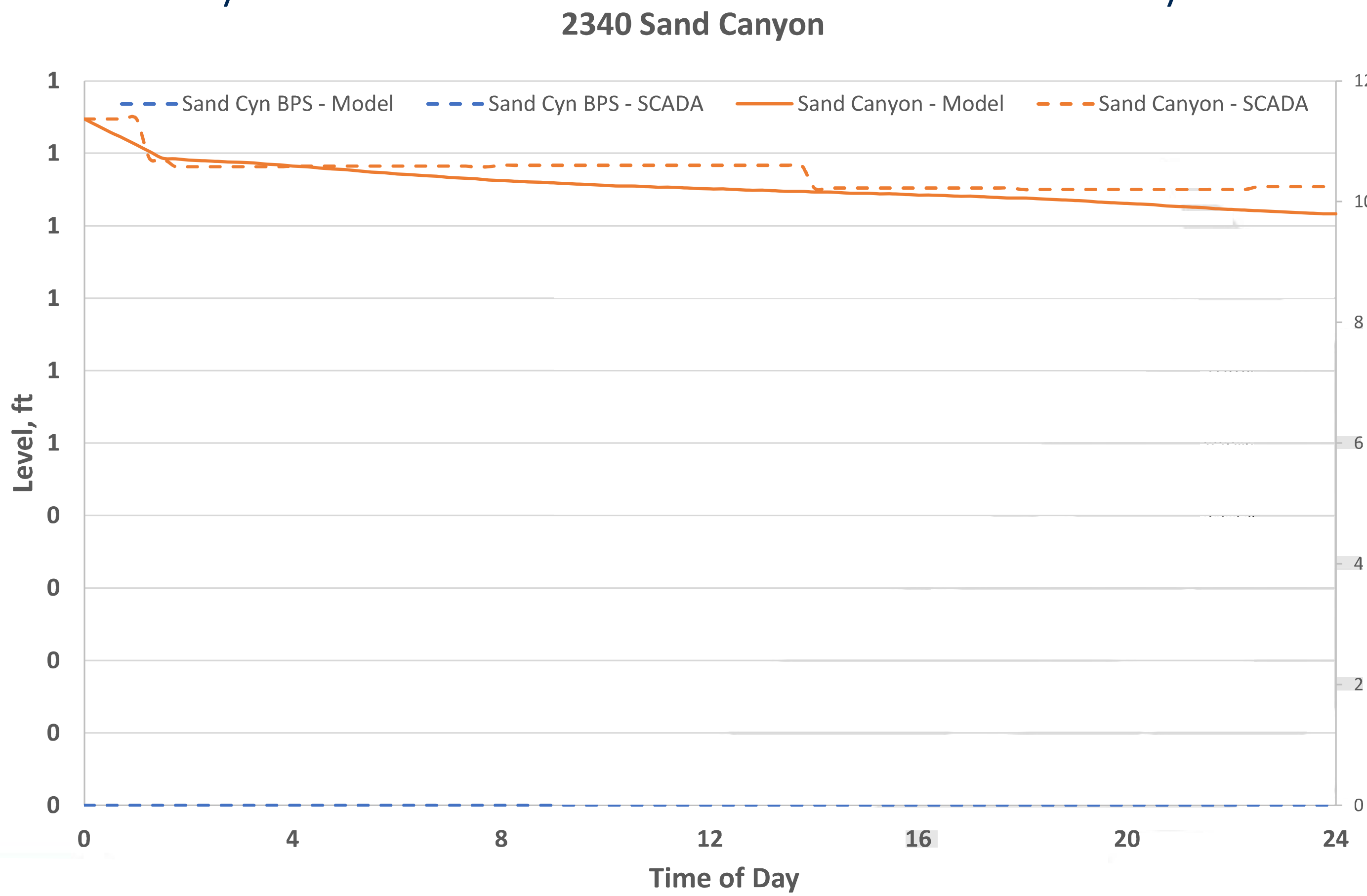
City of Redlands – Hydraulic Model Calibration 1900 Pump Stations



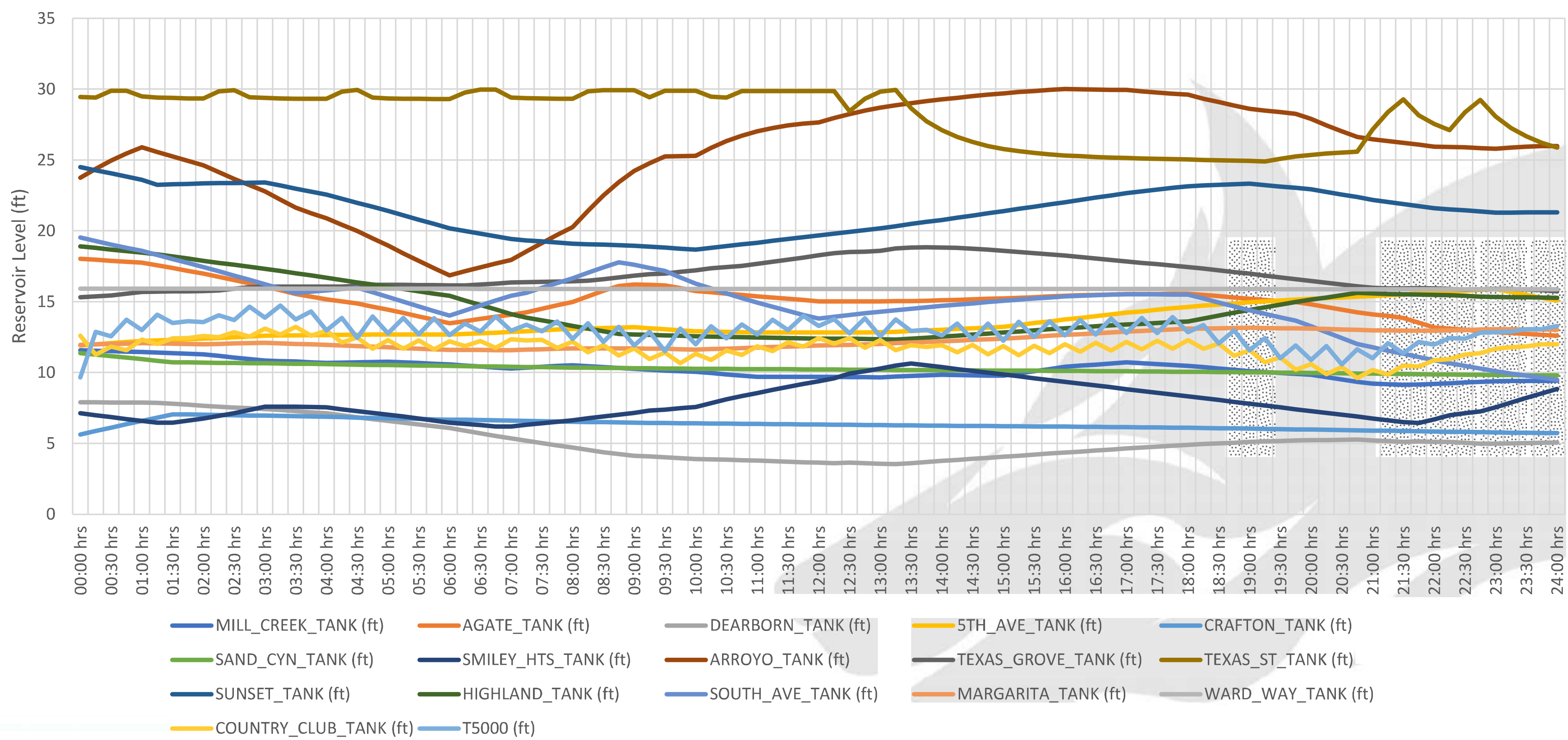
City of Redlands – Hydraulic Model Calibration 2100 reservoirs



City of Redlands – Hydraulic Model Calibration 2340 Sand Canyon reservoirs



Model Tanks Water Level Fluctuations (ft)



APPENDIX C

CIP Cost Tables

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REDLANDS CIP ROLLUP CALCULATION SUMMARY

		5-Year CIP						20-year CIP				
W1	Water Distribution System	2022	2023	2024	2025	2026	5-year Total	2022-2026	2027-2031	2032-2036	2037-2041	20-year Total
W1-1	Water Pipeline Replacement Due to Hydraulic D	$2,158,000	$1,122,000	$3,019,000	$0	$0	$6,299,000	$ 6,299,000	$ -	$ -	$ -	$ 6,299,000
W1-2	Reoccuring Water Pipeline Replacement Over Li	$3,942,000	$4,978,000	$3,081,000	$6,100,000	$6,100,000	$24,201,000	$ 24,201,000	$ 30,500,000	$ 30,500,000	$ 30,500,000	$ 115,701,000
W1-3	Citywide Pavement Repair for Water Facilities	$500,000	$500,000	$500,000	$500,000	$500,000	$2,500,000	$ 2,500,000	$ 2,500,000	$ 2,500,000	$ 2,500,000	$ 10,000,000
W1-4	2-inch and Smaller Meter Replacement	$645,000	$645,000	$645,000	$645,000	$645,000	$3,225,000	$ 3,225,000	$ 3,225,000	$ 3,225,000	$ 3,225,000	$ 12,900,000
W1-5	Greater than 2-inch Meter Refurbishment	$55,000	$55,000	$55,000	$55,000	$55,000	$275,000	$ 275,000	$ 275,000	$ 275,000	$ 275,000	$ 1,100,000
W1-6	Dead End Bypass and Hydrant Flushing	$0	$0	$0	$0	$0	$0	$ -	$ -	$ -	$ -	$ -
W1-7	Automated Metering Infrastructure (AMI) Optio	$0	$0	$0	$0	$0	$0	$ -	$ -	$ -	$ -	$ -
	CIP W1 Subtotal	$7,300,000	$7,300,000	$7,300,000	$7,300,000	$7,300,000	$36,500,000	$ 36,500,000	$ 36,500,000	$ 36,500,000	$ 36,500,000	$ 146,000,000
W2	Hinckley Treatment Plant	2022	2023	2024	2025	2026	5-year Total	2022-2026	2027-2031	2032-2036	2037-2041	
W2-1	Hinckley Transmission Line Replacement	$0	$4,800,000	$0	$0	$0	$4,800,000	$ 4,800,000				$ 4,800,000
W2-2	Hinckley/Tate Roof Repair	$0	$0	$0	$0	$0	$0	$ -				$ -
W2-3	Hinckley WTP Safety Fencing	$0	$0	$0	$0	$0	$0	$ -				$ -
W2-4	Hinckley Sludge Press	$360,000	$0	$0	$0	$0	$360,000	$ 360,000				$ 360,000
W2-5	Hinckley Generator Replacement	$0	$0	$0	$0	$0	$0	$ -				$ -
W2-6	Hinckley WTP Paving	$0	$180,000	$0	$0	$0	$180,000	$ 180,000				$ 180,000
W2-7	Wash Water treatment plant replacement	$0	$0	$0	$6,000,000	$0	$6,000,000	$ 6,000,000				$ 6,000,000
W2-8	Flexible joint support replacement	$0	$0	$0	$50,000	$0	$50,000	$ 50,000				$ 50,000
W2-9	Upgrade MCC and replace dated electrical insta	$0	$50,000	$150,000	$150,000	$150,000	$500,000	$ 500,000				$ 500,000
W2-10	Replace aging mechanical equipment	$0	$40,000	$320,000	$320,000	$320,000	$1,000,000	$ 1,000,000				$ 1,000,000
W2-11	Capacity upgrade 12MGD+12MGD - Not Include	$0	$0	$0	$0	$0	$0	$ -				$ -
	CIP W2 Subtotal	$360,000	$5,070,000	$470,000	$6,520,000	$470,000	$12,890,000	$ 12,890,000	$ -	$ -	$ -	$ 12,890,000
W3	Tate Treatment Plant	2022	2023	2024	2025	2026	5-year Total	2022-2026	2027-2031	2032-2036	2037-2041	
W3-1	Tate Transmission Line Assessment	$0	$0	$4,800,000	$0	$0	$4,800,000	$ 4,800,000				$ 4,800,000
W3-2	Tate ACH Tank Replacement	$0	$0	$0	$0	$0	$0	$ -				$ -
W3-3	Tate Clarifier Recoating	$0	$0	$0	$0	$0	$0	$ -				$ -
W3-4	Tate Clarifier Covers	$1,560,000	$0	$0	$0	$0	$1,560,000	$ 1,560,000				$ 1,560,000
W3-5	Tate Influent Static Mixer	$0	$0	$180,000	$0	$0	$180,000	$ 180,000				$ 180,000
W3-6	Tate NaOCl Disinfection System	$360,000	$0	$0	$0	$0	$360,000	$ 360,000				$ 360,000
W3-7	Tate PLC Replacement	$0	$0	$0	$0	$0	$0	$ -				$ -
W3-8	PRV Station Replacement (Redlands Blvd. & Nev	$120,000	$300,000	$0	$0	$0	$420,000	$ 420,000				$ 420,000
W3-9	Settler equipment corrosion refurbishment	$0	$0	$0	$0	$50,000	$50,000	$ 50,000				$ 50,000
W3-10	Upgrade MCC and replace dated electrical insta	$0	$50,000	$150,000	$150,000	$150,000	$500,000	$ 500,000				$ 500,000
W3-11	Replace aging mechanical equipment	$0	$40,000	$320,000	$320,000	$320,000	$1,000,000	$ 1,000,000				$ 1,000,000
W3-12	Capacity upgrade 12 mgd - Not Included in CIP	$0	$0	$0	$0	$0	$0	$ -				$ -
	CIP W3 Subtotal	$2,040,000	$390,000	$5,450,000	$470,000	$520,000	$8,870,000	$ 8,870,000	$ -	$ -	$ -	$ 8,870,000
W4	Booster Stations - Potable	2022	2023	2024	2025	2026	5-year Total	2022-2026	2027-2031	2032-2036	2037-2041	
W4-1	1750 Blend Manifold Replacement	$120,000	$0	$0	$0	$0	$120,000	$ 120,000	$ -		$ -	$ 120,000
W4-2	Booster Pump Station Rehabilitation	$3,270,000	$1,960,000	$2,125,000	$2,125,000	$870,000	$10,350,000	$ 10,350,000	$ 9,580,000	$ 5,900,000	$ 10,350,000	$ 36,180,000
	CIP W4 Subtotal	$3,390,000	$1,960,000	$2,125,000	$2,125,000	$870,000	$10,470,000	$ 10,470,000	$ 9,580,000	$ 5,900,000	$ 10,350,000	$ 36,300,000
W5	Reservoirs - Potable	2022	2023	2024	2025	2026	5-year Total	2022-2026	2027-2031	2032-2036	2037-2041	
W5-1	Reservoir Sites Fixed Generators	$0	$0	$0	$0	$0	$0	$ -	$ -	$ -	$ -	$ -
W5-2	Sunset Reservoir Seismic Rehab	$3,000,000	$3,000,000	$0	$0	$0	$6,000,000	$ 6,000,000	$ -	$ -	$ -	$ 6,000,000
W5-3	Texas St. Reservoir & booster station	$0	$0	$0	$0	$0	$0	$ -	$ -	$ -	$ -	$ -
W5-4	Crafton Hills & Property-One Reservoir	$0	$0	$0	$0	$0	$0	$ -	$ -	$ -	$ -	$ -
W5-5	Agate Reservoir TTHM Treatment System	$900,000	$0	$0	$0	$0	$900,000	$ 900,000	$ -	$ -	$ -	$ 900,000
W5-6	Reservoir Mixing System						$0	$ -	$ -	$ -	$ -	$ -
W5-7	Texas Grove Reservoir stair installation	$90,000	$0	$0	$0	$0	$90,000	$ 90,000	$ -	$ -	$ -	$ 90,000
W5-8	Emerald Main Line Isolation	$0	$0	$0	$0	$0	$0	$ -				$ -
W5-9	Steel Reservoir Rehabilitation	$100,000	$2,891,000	$1,568,000	$1,225,000	$1,470,000	$7,254,000	$ 7,254,000	$ 4,998,000			$ 12,252,000
W5-10	Steel Reservoir Inspection and Minor Repairs	$0	$0	$0	$0	$0	$0	$ -	$ 110,000	$ 180,000	$ 180,000	$ 470,000
W5-11	Concrete Reservoir Inspection and Minor Repair	$44,000	$45,000	$34,000	$56,000	$50,000	$229,000	$ 229,000	$ 90,000			$ 319,000
	CIP W5 Subtotal	$4,134,000	$5,936,000	$1,602,000	$1,281,000	$1,520,000	$14,473,000	$ 14,473,000	$ 5,198,000	$ 180,000	$ 180,000	$ 20,031,000
W6	Groundwater Wells	2022	2023	2024	2025	2026	5-year Total	2022-2026	2027-2031	2032-2036	2037-2041	
W6-1	Groundwater Well Equipping Rehabilitation	$1,350,000	$1,050,000	$1,275,000	$1,875,000	$1,200,000	$6,750,000	$ 6,750,000	$ 7,912,500	$ 5,487,500	$ 6,750,000	$ 26,900,000
W6-2	East Lugonia Well 3 Repalcement	$0	$0	$3,000,000	$0	$0	$3,000,000	$ 3,000,000	$ -	$ -	$ -	$ 3,000,000
W6-3	Groundwater Contamination Mitigation	$150,000	$575,000	$575,000	$575,000	$575,000	$2,450,000	$ 2,450,000	$ -	$ -	$ -	$ 2,450,000
W6-4	Entrained Air Treatment Assessment	$0	$0	$600,000	$0	$0	$600,000	$ 600,000				
	CIP W7 Subtotal	$1,500,000	$1,625,000	$5,450,000	$2,450,000	$1,775,000	$12,800,000	$ 12,200,000	$ 7,912,500	$ 5,487,500	$ 6,750,000	$ 32,350,000
W7	Water Use Improvements	2022	2023	2024	2025	2026	5-year Total	2022-2026	2027-2031	2032-2036	2037-2041	

W7-1	Brookside Median-Water Efficient Landscape Im	$0	$0	$0	$0	$0	$0	$ -				$ -
W7-2	WBIC/Smart Irrigation Controllers-City facilities	$0	$0	$0	$0	$0	$0	$ -				$ -
W7-3	Water Efficiency Upgrades- City Facility Landsca	$300,000	$300,000	$300,000	$300,000	$300,000	$1,500,000	$ 1,500,000				$ 1,500,000
	CIP W8 Subtotal	$300,000	$300,000	$300,000	$300,000	$300,000	$1,500,000	$ 1,500,000	$ -	$ -	$ -	$ 1,500,000
		$ 19,024,000	$ 22,581,000	$ 22,697,000	$ 20,446,000	$ 12,755,000	$ 97,503,000					

5 year CIP Totals

W1 - Water Distribution System	$ 36,500,000
W2 - Hinckley Treatment Plant	$ 12,890,000
W3 - Tate Treatment Plant	$ 8,870,000
W4 - Booster Stations - Potable	$ 10,470,000
W5 - Reservoirs - Potable	$ 14,473,000
W6 - Groundwater Wells	$ 12,800,000
W7 - Water Use Improvements	$ 1,500,000

20 year CIP Totals

Projec	2022-2026	2027-2031	2032-2036	2037-2041	20-year Total
W1 -	$36,500,000	$36,500,000	$36,500,000	$36,500,000	$146,000,000
W2 -	$12,890,000	$0	$0	$0	$12,890,000
W3 -	$8,870,000	$0	$0	$0	$8,870,000
W4 -	$10,470,000	$9,580,000	$5,900,000	$10,350,000	$36,300,000
W5 -	$14,473,000	$5,198,000	$180,000	$180,000	$20,031,000
W6 -	$12,200,000	$7,912,500	$5,487,500	$6,750,000	$32,350,000
W7 -	$1,500,000	$0	$0	$0	$1,500,000
Total	$96,903,000	$59,190,500	$48,067,500	$53,780,000	$257,941,000

Note: 1) the 20-year cost for W6 - Wellhead Treatment is dependant on the Wellhead Evaluation

W1-1 CALCUALTION SUMMARY TABLES

Materials	CIP	Replacement Diameter (in)	Length (ft)	Probable Cost Calculation	Issue	Location	Year
DIP	CIP-1		0	$0	High Flow Velocity	Center St. & Crecent Ave.	
DIP	CIP-2		0	$0	High Flow Velocity	Valencia Dr.	
DIP	CIP-3	24	2,046	$982,195	High Flow Velocity	San Bernardino Ave. & Agate Ave.	2022
DIP	CIP-4	8	203.37	$32,539	High Flow Velocity	Mill Creek Rd.	2022
DIP	CIP-5	0	0	$0	High Flow Velocity	San Bernaridino Ave	
DIP	CIP-6	0	0	$0	Fire Flow Defecient	Park Ave. & New Jersey ST.	
DIP	CIP-7	8	242	$38,723	Fire Flow Defecient	University St. & Colton Ave	2023
DIP	CIP-8	8	1,267	$202,776	Fire Flow Defecient	Highland Ave. & Walnut St.	2022
DIP	CIP-9	0	0	$0	Fire Flow Defecient	Emerald Ave & Newport Ave.	
DIP	CIP-10	8	649	$103,814	Fire Flow Defecient	Naples Ave. & Jasper Ave.	2023
DIP	CIP-11	12	11,979	$3,018,731	Fire Flow Defecient	Wabash Ave. & 6th Ave.	2024
DIP	CIP-12	0	0	$0	Fire Flow Defecient	Sunset Dr. Fairmont Dr.	
DIP	CIP-13	8	503	$80,530	Fire Flow Defecient	Sierra Vista Ave. & Escondido Rd.	2022
DIP	CIP-14	8	4,217	$674,698	Fire Flow Defecient	Pacific St. ,Center Crest Dr. & Benita Marie Crest	2022
DIP	CIP-15	8	1,142	$182,685	Fire Flow Defecient	Pennsylvania Ave. De Anza St	2022
DIP	CIP-16	0	0	$0	Small Diameter	Monterey Ave	
DIP	CIP-17	8	1,569	$250,998	Small Diameter	Park Ave & Cook ST.	2023
DIP	CIP-18	8	1,045	$167,250	Small Diameter	Madeira Ave. & Agate Ave	2023
DIP	CIP-19	12	2,330	$559,140	Small Diameter	Sunset Dr.	2023
			27,193	6,294,079			

REPORT FORMAT

	CIP	Replacement Diameter (in)	Length (ft)	Location	Issue	Budget Cost Calculation	Year
	CIP-1	N/A	-	Center St. & Crecent Ave.	High Flow Velocity	N/A	N/A
	CIP-2	N/A	-	Valencia Dr.	High Flow Velocity	N/A	N/A
	CIP-3	24	2,046	San Bernardino Ave. & Agate Ave.	High Flow Velocity	$983,000	2022
	CIP-4	8	203	Mill Creek Rd.	High Flow Velocity	$33,000	2022
	CIP-5	N/A	-	San Bernaridino Ave	High Flow Velocity	N/A	N/A
	CIP-6	N/A	-	Park Ave. & New Jersey ST.	Fire Flow Defecient	N/A	N/A
	CIP-7	8	242	University St. & Colton Ave	Fire Flow Defecient	$39,000	2023
	CIP-8	8	1,267	Highland Ave. & Walnut St.	Fire Flow Defecient	$203,000	2022
	CIP-9	N/A	-	Emerald Ave & Newport Ave.	Fire Flow Defecient	N/A	N/A
	CIP-10	8	649	Naples Ave. & Jasper Ave.	Fire Flow Defecient	$104,000	2023
	CIP-11	12	11,979	Wabash Ave. & 6th Ave.	Fire Flow Defecient	$3,019,000	2024
	CIP-12	N/A	-	Sunset Dr. Fairmont Dr.	Fire Flow Defecient	N/A	N/A
	CIP-13	8	503	Sierra Vista Ave. & Escondido Rd.	Fire Flow Defecient	$81,000	2022
	CIP-14	8	4,217	Pacific St. ,Center Crest Dr. & Benita Marie Crest	Fire Flow Defecient	$675,000	2022
	CIP-15	8	1,142	Pennsylvania Ave. De Anza St	Fire Flow Defecient	$183,000	2022
	CIP-16	N/A	-	Monterey Ave	Small Diameter	N/A	N/A
	CIP-17	8	1,569	Park Ave & Cook ST.	Small Diameter	$251,000	2023
	CIP-18	8	1,045	Madeira Ave. & Agate Ave	Small Diameter	$168,000	2023
	CIP-19	12	2,330	Sunset Dr.	Small Diameter	$560,000	2023

W1-2 CALCUALTION SUMMARY TABLES

LF of pipe, diameter, and when it reaches end service life

			Length (ft) (columns below)																		
Diameter ---> This row acrosss	ERROR CHECK	TOTAL LF	1	1.5	2	3	4	4.5	6	8	10	12	14	15	16	18	20	24	30	32	36 Unknown
Error (lower than -100)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Less than 0 Greater or equal to -100	34,567	34,567	1,317	2,696	2,748	184	7,869	-	5,602	7,991	-	4,064	-	89	1,994	-	14	-	-	-	-	-
Less than or equal to 5 and greater than or equal to 0	14,734	14,734	-	-	3,630	-	862	-	3,500	4,979	-	1,546	-	-	216	-	-	-	-	-	-	-
Greater than 5 less than or equal to 10	89,411	89,411	-	-	2,971	317	2,418	-	37,925	21,435	1,758	21,431	-	-	748	220	188	-	-	-	-	-
Greater than 10 less than or equal to 15	118,997	118,997	-	-	-	-	3,326	-	48,932	50,107	-	16,632	-	-	-	-	-	-	-	-	-	-
Greater than 15 less than or equal to 20	151,992	151,992	-	215	1,082	-	1,981	1,809	20,295	58,570	3,920	46,596	-	-	5,080	-	-	12,444	-	-	-	-
Greater than 20 less than or equal to 25	137,680	137,680	215	-	3,050	-	3,823	-	44,735	63,839	438	15,651	-	-	5,212	718	-	-	-	-	-	-
Greater than 25 less than or equal to 30	277,802	277,802	291	944	3,118	2,165	6,830	-	86,603	112,749	2,873	57,560	-	-	4,670	-	-	-	-	-	-	-
Greater than 30 less than or equal to 35	165,927	165,927	511	-	1,407	-	8,477	-	44,321	80,313	1,067	29,258	-	-	456	-	102	15	-	-	-	-
Greater than 35 less than or equal to 40	195,763	195,763	-	629	3,965	-	5,509	-	13,206	93,760	2,322	41,717	-	4,761	18,186	-	10,815	43	850	-	-	-
Greater than 40 less than or equal to 45	68,593	68,593	-	-	434	-	215	-	9,950	12,056	44	12,626	-	-	11,462	-	13,458	5,175	2,306	867	-	-
Greater than 45 less than or equal to 50	155,063	155,063	21	778	3,904	-	1,162	-	3,808	45,871	60	56,532	5,195	360	17,147	-	6,462	9,845	2,744	47	1,125	-
Greater than 50 less than or equal to 55	163,129	163,129	-	-	1,082	-	691	-	2,471	103,090	-	44,865	-	-	616	-	3,321	1,162	5,831	-	-	-
Greater than 55 less than or equal to 60	38,506	38,506	-	-	2,137	-	203	-	1,876	13,952	-	6,891	-	-	156	-	443	12,849	-	-	-	-
Greater than 60 less than or equal to 65	18,343	18,343	413	-	-	-	1,298	-	147	3,023	-	2,684	-	-	3,236	-	6,845	140	-	-	556	-
Greater than 65 less than or equal to 70	58,093	58,093	-	-	77	214	9	-	1,201	3,594	749	14,380	63	-	1,315	-	4,825	16,835	14,833	-	-	-
Greater than 70 less than or equal to 75	21,516	21,516	-	-	-	-	-	-	354	7,153	-	11,804	-	-	2,204	-	-	-	-	-	-	-
Greater than 75 less than or equal to 80	9,277	9,277	-	-	12	2	2,334	-	649	2,386	132	3,752	-	-	12	-	-	-	-	-	-	-
Greater than 80 less than or equal to 90	137,929	137,929	-	-	70	-	132	-	991	53,709	165	38,937	5,536	-	17,079	197	-	8,520	12,592	-	-	-
Greater than 90 less than or equal to 100	158,822	158,822	-	-	73	12	16	-	1,337	116,632	-	18,559	-	-	3,622	-	23	17,121	-	-	-	1,428
Error (greater than 100)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Total	2,016,144	2,016,144	2,768	5,262	29,761	2,893	47,154	1,809	327,903	855,208	13,528	445,486	10,794	5,211	93,410	1,135	46,494	84,149	39,155	914	1,682	1,428
Error Check	-		2,768	8,030	37,790	40,683	87,838	89,647	417,550	1,272,758	1,286,286	1,731,772	1,742,566	1,747,777	1,841,187	1,842,322	1,888,816	1,972,965	2,012,120	2,013,034	2,014,716	2,016,144

Asbestos Concrete Pipe			Length (ft)																		
Pipe Diameter	LENGTH (LF)	TOTALCOST	1	1.5	2	3	4	4.5	6	8	10	12	14	15	16	18	20	24	30	32	36 Unknown
Pipe Replacement Diameter			8	8	8	8	8	8	8	8	10	12	14	15	16	18	20	24	30	32	36	16
Error (lower than -100)	-	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Less than 0 Greater or equal to -100	34,567	$ 4,643,000.25	$ 157,982	$ 323,559	$ 329,768	$ 22,025	$ 944,301	$ -	$ 672,260	$ 958,915	$ -	$ 731,606	$ -	$ 20,038	$ 478,443	$ -	$ 4,104	$ -	$ -	$ -	$ -	$ -
Less than or equal to 5 and greater than or equal to 0	14,734	$ 1,886,804.31	$ -	$ -	$ 435,633	$ -	$ 103,403	$ -	$ 419,958	$ 597,523	$ -	$ 278,361	$ -	$ -	$ 51,927	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Greater than 5 less than or equal to 10	89,411	$ 12,224,515.88	$ -	$ -	$ 356,525	$ 38,007	$ 290,168	$ -	$ 4,551,030	$ 2,572,159	$ 263,719	$ 3,857,635	$ -	$ -	$ 179,572	$ 59,369	$ 56,332	$ -	$ -	$ -	$ -	$ -
Greater than 10 less than or equal to 15	118,997	$ 15,277,503.62	$ -	$ -	$ -	$ -	$ 399,153	$ -	$ 5,871,840	$ 6,012,834	$ -	$ 2,993,677	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Greater than 15 less than or equal to 20	151,992	$ 24,748,544.16	$ -	$ 25,811	$ 129,869	$ -	$ 237,661	$ 217,108	$ 2,435,436	$ 7,028,436	$ 587,977	$ 8,387,227	$ -	$ -	$ 1,219,206	$ -	$ -	$ 4,479,813	$ -	$ -	$ -	$ -
Greater than 20 less than or equal to 25	137,680	$ 18,206,819.05	$ 25,797	$ -	$ 365,982	$ -	$ 458,802	$ -	$ 5,368,229	$ 7,660,676	$ 65,633	$ 2,817,167	$ -	$ -	$ 1,250,765	$ 193,768	$ -	$ -	$ -	$ -	$ -	$ -
Greater than 25 less than or equal to 30	277,802	$ 37,436,442.00	$ 34,954	$ 113,224	$ 374,203	$ 259,749	$ 819,604	$ -	$ 10,392,408	$ 13,529,862	$ 430,910	$ 10,360,749	$ -	$ -	$ 1,120,779	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Greater than 30 less than or equal to 35	165,927	$ 21,775,328.54	$ 61,362	$ -	$ 168,882	$ -	$ 1,017,214	$ -	$ 5,318,479	$ 9,637,604	$ 160,009	$ 5,266,415	$ -	$ -	$ 109,485	$ -	$ 30,450	$ 5,429	$ -	$ -	$ -	$ -
Greater than 35 less than or equal to 40	195,763	$ 30,983,881.69	$ -	$ 75,507	$ 475,793	$ -	$ 661,136	$ -	$ 1,584,731	$ 11,251,141	$ 348,234	$ 7,509,098	$ -	$ 1,071,277	$ 4,364,551	$ -	$ 3,244,370	$ 15,587	$ 382,456	$ -	$ -	$ -
Greater than 40 less than or equal to 45	68,593	$ 15,103,151.17	$ -	$ -	$ 52,057	$ -	$ 25,829	$ -	$ 1,193,979	$ 1,446,701	$ 6,624	$ 2,272,621	$ -	$ -	$ 2,750,993	$ -	$ 4,037,341	$ 1,863,163	$ 1,037,590	$ 416,253	$ -	$ -
Greater than 45 less than or equal to 50	155,063	$ 29,485,391.72	$ 2,508	$ 93,355	$ 468,439	$ -	$ 139,447	$ -	$ 456,946	$ 5,504,539	$ 9,066	$ 10,175,831	$ 1,091,027	$ 81,070	$ 4,115,350	$ -	$ 1,938,609	$ 3,544,328	$ 1,234,599	$ 22,616	$ 607,661	$ -
Greater than 50 less than or equal to 55	163,129	$ 25,142,080.18	$ -	$ -	$ 129,895	$ -	$ 82,917	$ -	$ 296,523	$ 12,370,819	$ -	$ 8,075,667	$ -	$ -	$ 147,789	$ -	$ 996,251	$ 418,256	$ 2,623,963	$ -	$ -	$ -
Greater than 55 less than or equal to 60	38,506	$ 8,216,305.91	$ -	$ -	$ 256,479	$ -	$ 24,301	$ -	$ 225,114	$ 1,674,189	$ -	$ 1,240,419	$ -	$ -	$ 37,378	$ -	$ 132,890	$ 4,625,536	$ -	$ -	$ -	$ -
Greater than 60 less than or equal to 65	18,343	$ 4,250,010.14	$ 49,511	$ -	$ -	$ -	$ 155,796	$ -	$ 17,661	$ 362,784	$ -	$ 483,209	$ -	$ -	$ 776,569	$ -	$ 2,053,560	$ 50,526	$ -	$ -	$ 300,394	$ -
Greater than 65 less than or equal to 70	58,093	$ 17,823,429.06	$ -	$ -	$ 9,202	$ 25,645	$ 1,080	$ -	$ 144,066	$ 431,265	$ 112,415	$ 2,588,384	$ 13,156	$ -	$ 315,485	$ -	$ 1,447,546	$ 6,060,462	$ 6,674,723	$ -	$ -	$ -
Greater than 70 less than or equal to 75	21,516	$ 3,554,639.30	$ -	$ -	$ -	$ -	$ -	$ -	$ 42,482	$ 858,356	$ -	$ 2,124,760	$ -	$ -	$ 529,042	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Greater than 75 less than or equal to 80	9,277	$ 1,343,722.34	$ -	$ -	$ 1,399	$ 240	$ 280,026	$ -	$ 77,836	$ 286,325	$ 19,812	$ 675,311	$ -	$ -	$ 2,772	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Greater than 80 less than or equal to 90	137,929	$ 27,670,155.40	$ -	$ -	$ 8,431	$ -	$ 15,804	$ -	$ 118,875	$ 6,445,081	$ 24,800	$ 7,008,650	$ 1,162,608	$ -	$ 4,099,050	$ 53,243	$ -	$ 3,067,247	$ 5,666,366	$ -	$ -	$ -
Greater than 90 less than or equal to 100	158,822	$ 24,869,779.95	$ -	$ -	$ 8,718	$ 1,484	$ 1,880	$ -	$ 160,471	$ 13,995,795	$ -	$ 3,340,667	$ -	$ -	$ 869,276	$ -	$ 6,805	$ 6,163,453	$ -	$ -	$ -	$ 321,230.51
Error (greater than 100)	-	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Total	2,016,144	$ 324,641,504.68	$ 332,113.70	$ 631,456.41	$ 3,571,275.52	$ 347,150.07	$ 5,658,522.08	$ 217,107.78	$ 39,348,323.17	$ 102,625,005.48	$ 2,029,199.41	$ 80,187,453.47	$ 2,266,790.54	$ 1,172,384.89	$ 22,418,430.32	$ 306,380.26	$ 13,948,258.95	$ 30,293,800.93	$ 17,619,696.51	$ 438,869.63	$ 908,055.04	$ 321,230.51
Error Check	-																					

	Material Average Service Life	SERVICE LIFE (YEARS)
ACP	Asbestos Concrete Pipe	70
CIP	Cast Iron Pipe	120
CMLC	Cement Mortar Lined and Coated	100
CMLDIP	Cement Mortar Lined Ductile Iron Pipe	100
CMLSTL	Cement Mortar Lined Steel Pipe	100
CON	Concrete	100
COP	Copper	50
DIP	Ductile Iron Pipe	100
DW	Dipped and Wrapped	40
HDPE	High Density Polyethylene	50
OTH	Other	40
PVC	Polyvinyl Chloride	70
RCP	Reincforced Concrete Pipe	100
STL	Steel	100
Unk	Unknown	40
VCP	Vitrified Clay Pipe	50
	Total	

W1-4 & W1-5 CALCUALTION SUMMARY TABLES

Assume they want to repalce all meters less than 2" and including 2" over 20 years

METER REPALCEMENT — Meter Repalce Unit Cost

Qty of meters looked up in GIS — Rounded

Size (in)	Number of meters	Unit Cost	Total Cost
5/8"	169	$ 428.47	72,411
3/4"	7,938	$ 459.51	3,647,598
1"	12,493	$ 545.00	6,808,673
1 1/2"	833	$ 1,298.10	1,081,319
2"	859	$ 1,495.60	1,284,721
Total	22,292		12,894,722

Yearly Cost over 20 years

644,736.10 per year when done over 20 years

645000 Assume $515,000 when rounded up

1,114.60 About 1,100 meter per year

586.36 approx unit cost per meter

Assume a Recalibrate Every 10 Years

METER CALIBRATION — Cost taken from City

Size (in)	Number of meters	Rehab Costs	Total
2.5"	1		
3"	66	550	36,300
4"	182	550	100,100
6"	154	550	84,700
8"	174	550	95,700
10"	115	550	63,250
Unknown	265	550	145,750
Total	957		525,800.00

Yearly Cost

52,580.00 Per year when done over 10 years

55000 Assume $55,000 when rounded up

95.70 About 100 meters per year

REFERENCE INFORMAIOTN ON METER REPLACEMENT COSTS

Meter Replacement Costs from City			
	Total Cost	Qty	Unit Cost
5/8"	$ 4,614.26	14	$ 428.47
3/4"	$ 182,743.99	517	$ 459.51
1"	$ 206,680.39	493	$ 545.00
1 1/2"	$ 108,840.86	109	$ 1,298.10
2"	$ 197,879.51	172	$ 1,495.60

Meter Replacement Costs from City Not used			
3	13	21585.33	1660.41
3	59	197879.51	3353.89
4	13	26670.41	2051.57
4	17	69042.1	4061.3
6	1	4446.69	4446.69
6	15	114734.25	7648.95
8	1	6258.35	6258.35
6x8	0	12064.93	12064.93
4	0	11635.46	11635.46
6	0	16221.78	16221.78
8	8	157530	19691.25
10	0	22987.57	22987.57

REFERENCE INFORMATION ON METER REHAB COSTS FROM CITY BIDS

Cost to rehab, form the City					
Meter Size	Total Cost	Qty	Unit Cost	Assumed Unit Cost	
3"	7997.1	19	420.9	550	
4"	12627	30	420.9	550	
6"	7997.1	19	420.9	550	
8"	2104.5	5	420.9	550	

W4-2 CALCUALTION SUMMARY TABLES

Station Count (13 Stations)	Pump Count (39 Pumps)	Pump Station (1)	Pump Name (1)	Test #	HP	Head	Capacity	Overall Eff	Improved Eff	Total KwH	Kw Input	Cost per Kwh	Average cost per Acre Ft	Estimated cost of Rehab ($1,000 per HP)	Total per Station for Mechanical	For Elec. and civil/coatings	Instalaltion Labor	Soft Costs	Total
1	1	Texas	1550	1	250	252	2805	74		175200	180	0.16	55.76	$250,000					
	2	Texas	1551	1	250	387.4	1780	72.6		522096	178.8	0.16	87.28	$250,000					
				2	250	335	2260	73.5											
				3	250	231.5	2706	62											
	3	Texas	1552											$250,000					
	4	Texas	1553	1	250	267.3	3143	80.7		608388	196	0.17	57.57	$250,000					
				2	250	249.2	3240	78.4							$1,000,000	$1,000,000.00	$1,000,000.00	$1,350,000.00	$4,350,000
		Texas	4		1000			73.53333333											
2	5	Dearborn	1761	1	100	161.2	1490	54.2	70	65052	83.5	0.2	59.96	$100,000					
	6	Dearborn	1931	1	200	373.8	1516	76.5		108588	139.4	0.2	98.38	$200,000					
				2	200	397.1	1380	75.2											
				3	200	420	1244	73.7							$300,000	$300,000.00	$300,000.00	$405,000.00	$1,305,000
		Dearborn	2		300			69.9											
3	7	HAWC	2174	1	200	544.2	1103	73.6		44088	153.6	0.16	118.74	$200,000					
	8	HAWC	2176	1	200	539.6	875	65.8		35796	135.1	0.16	161.65	$200,000					
	9	HAWC	2177	1	150	534.8	691	58.1	70	106776	119.7	0.16	147.7	$150,000					
	10	HAWC	1720	1	75	166.3	525	44.9	65	30120	36.6	0.16	59.44	$75,000					
	11	HAWC	1721	1	150	185.5	2130	73.4		143640	101.3	0.16	40.55	$150,000					
				2	150	205.4	2000	78											
	12	HAWC	1722	1	200	183.4	2045	69		76368	102.3	0.16	42.65	$200,000					
				2	200	179.7	2115	69.6							$975,000	$975,000.00	$975,000.00	$1,316,250.00	$4,241,250
		HAWC	6		975			66.55											
4	13	Smiley Heights	1783											$150,000					
	14	Smiley Heights	1784											$150,000	$300,000	$300,000.00	$300,000.00	$405,000.00	$1,305,000
		Smiley Heights	2		UNK			UNK											
5	15	South	1927	1	100	200.5	1103	50.7	70	57336	82.2	0.18	73.66	$100,000					
	16	South	1928	1	150	163.5	1780	51.2	70	177876	107	0.18	59.42	$150,000					
	17	South	2124	1	200	389.9	980	47.6	70	36540	151.2	0.18	152.5	$200,000					
	18	South	2125		200									$150,000					
	19	South	2126		150									$150,000	$750,000	$750,000.00	$750,000.00	$1,012,500.00	$3,262,500
		South	5		800			49.83333333											
6	20	Agate	1951	1	100	155.2	1780	67.6		29532	77	0.16	36.41	$100,000					
				2	100	186.6	1519	68.3											
				3	100	204	1335	65.3											
	21	Agate	1952	1	100	157.1	1870	70.4		171948	78.6	0.16	35.38	$100,000					
				2	100	176.7	1684	70.9											
				3	100	213.7	1305	66.3											
	22	Agate	1953	1	100	153.6	1905	71		55440	77.6	0.16	34.29	$100,000					
				2	100	181.6	1623	71.1											
				3	100	205.4	1362	67.6							$300,000	$300,000.00	$300,000.00	$405,000.00	$1,305,000
		Agate	3		300			68.72222222											
7	23	Rees	1724											$150,000					
	24	Rees	1723											$150,000	$300,000	$300,000.00	$300,000.00	$405,000.00	$1,305,000
		Rees	2		UNK			UNK											
8	25	Fifth Avenue	2131	1	125	225.1	1018	59.9	72	148644	72	0.21	79.48	$125,000					
	26	Fifth Avenue	2132	1	125	236.7	1020	61.9	72	43369	110.97	0.24	93.79	$125,000					
				2	125	263.3	775	54.1											
	27	Fifth Avenue	2310	1	50	204.2	520	51.9	65	39240	38.5	0.21	83.2	$50,000					
				2	50	224.5	465	55.4											
				3	50	238.4	401	52.8											
	28	Fifth Avenue	2311	1	150	239.5	2337	68		21096	155	0.21	74.53	$150,000					
				2	150	262.6	2129	69.7							$450,000	$450,000.00	$450,000.00	$607,500.00	$1,957,500
		Fifth Avenue	4		450			59.2125											
9	29	Country Club	2384		100									$150,000					
	30	Country Club	2385	1	100	214.8	1134	65.4		42636	70.1	0.15	51.59	$100,000					
				2	100	234.5	1071	68.3											
				3	100	259.2	956	69.3											
	31	Country Club	2386	1	150	233.5	1777	68.1		244224	114.7	0.15	53.94	$150,000					
				2	150	259.9	1603	69.9											
	32	Country Club	2387	1	200	258.3	2264	67.4		57972	163.4	0.15	60.24	$200,000					
				2	200	281.6	2109	68.4											
				3	200	306.6	1932	68							$600,000	$600,000.00	$600,000.00	$810,000.00	$2,610,000
		Counrty Club	4		550			68.1											
10	33	Ward Way	2381											$150,000					
	34	Ward Way	2382											$150,000	$300,000	$300,000.00	$300,000.00	$405,000.00	$1,305,000
		Ward Way	2		UNK			UNK											
11	35	Sand Canyon	2610	1	50	253.9	539	66.2						$50,000					
	36	Sand Canyon	2611	1	150	318.5	1362	67.2						$150,000	$200,000	$200,000.00	$200,000.00	$270,000.00	$870,000
		Sand Canyon	2		200			66.7											
12	37	Yucaipa	2330											$150,000	$150,000	$150,000.00	$150,000.00	$202,500.00	$652,500
		Yucaipa	1		UNK			UNK											
13	38	Mill Creek	2510											$150,000					
	39	Mill Creek	2511											$150,000	$300,000	$300,000.00	$300,000.00	$405,000.00	$1,305,000
		Mill Creek	2		UNK			UNK											

	Name	Num. Pum	Total HP	Eff.	Rehab Cost	Rounded	Year
1	South	5	800	49.8	$3,262,500	$3,270,000	2022
2	Fifth Avenue	4	450	59.2	$1,957,500	$1,960,000	2023
3	HAWC	6	975	66.6	$4,241,250	$4,250,000	2024/2025
4	Sand Canyon	2	200	66.7	$870,000	$870,000	2026
5	Counrty Club	4	550	68.1	$2,610,000	$2,610,000	2027
6	Agate	3	300	68.7	$1,305,000	$1,310,000	2028
7	Dearborn	2	300	69.9	$1,305,000	$1,310,000	2029'
8	Texas	4	1000	73.5	$4,350,000	$4,350,000	2030/2031
9	Smiley Heights	2	UNK	UNK	$1,305,000	$1,310,000	2032
10	Rees	2	UNK	UNK	$1,305,000	$1,310,000	2033
11	Ward Way	2	UNK	UNK	$1,305,000	$1,310,000	2034
12	Yucaipa	1	UNK	UNK	$652,500	$660,000	2035
13	Mill Creek	2	UNK	UNK	$1,305,000	$1,310,000	2036

W5-6 CALCUALTION SUMMARY TABLES

Name	Capacity (MG)	Type	Year	Budget	
Texas Grove	3.9	Steel	1956	$ 30,000	
Agate	3	Steel	1968	$ 30,000	
Arroyo	0.5	Steel	1965	$ 30,000	
South	2	Steel	1964	$ 30,000	
Ward Way	2	Steel	1958	$ 30,000	
Sunset	3	Steel	1967	$ 30,000	
Mill Creek 1	0.2	Steel	1962	$ 30,000	
Mill Creek 2	0.2	Steel	1987	$ 30,000	
Crafton	3.5	Steel	1970	$ 30,000	
Texas Street	1	TBD	TBD	$ 30,000	
Dearborn	10	Concrete	1972	$ 30,000	
Highland	10	Concrete	1976	$ 30,000	
Country Club 1	1	Concrete	1969	$ 30,000	
Country Club 2	2	Concrete	1924	$ 30,000	

Total	$ 420,000	
over 5 year	$ 84,000	per year

Smiley	3	Steel	1964	Complete 2021
Sand Canyon	3.5	Steel	1973	Complete 2021
Fifth Avenue	5	Concrete	1974	Complete 2021
Margarita	2.4	Concrete	1964	Complete 2021

W5-9, W5-10, W5-11 CALCUALTION SUMMARY TABLES

Priority	ID	Name	Capacity (MG)	Type	Year	Rehab Budget	Rehab Year	Rehab Year	Inspection Budget	Inspection Year	Inspection Year	Inspection Year	Rehab Again Year
1	1	Texas Grove	3.9	Steel	1956	$ 1,911,000	2023	2043	$ 20,000	2028	2033	2038	2043
2	13	Ward Way	2	Steel	1958	$ 980,000	2023	2043	$ 15,000	2028	2033	2038	2043
3	16	Mill Creek 1	0.2	Steel	1962	$ 98,000	2024	2044	$ 10,000	2029	2034	2039	2044
4	5	Smiley	3	Steel	1964	$ 1,470,000	2024	2044	$ 20,000	2029	2034	2039	2044
5	8	South	2	Steel	1964	$ 980,000	2025	2045	$ 15,000	2030	2035	2040	2045
6	7	Arroyo	0.5	Steel	1965	$ 245,000	2025	2045	$ 10,000	2030	2035	2040	2045
7	15	Sunset	3	Steel	1967	$ 1,470,000	2026	2046	$ 20,000	2031	2036	2041	2046
8	6	Agate	3	Steel	1968	$ 1,470,000	2027	2047	$ 20,000	2032	2037	2042	2047
9	18	Crafton	3.5	Steel	1970	$ 1,715,000	2028	2048	$ 20,000	2033	2038	2043	2048
10	14	Sand Canyon	3.5	Steel	1973	$ 1,715,000	2029	2049	$ 20,000	2034	2039	2044	2049
11	17	Mill Creek 2	0.2	Steel	1987	$ 98,000	2029	2049	$ 10,000	2034	2039	2044	2049

Tank Rehabilitation

CIP Budget per year	2022	$0	1
	2023	$ 2,891,000	2
	2024	$ 1,568,000	3
	2025	$ 1,225,000	4
	2026	$ 1,470,000	5
	2027	$ 1,470,000	6
	2028	$ 1,715,000	7
	2029	$ 1,813,000	8

CONCRETE TANKS		Designation	Capacity (MG)	Type	Year	Cost
1		Country Club 2	2	Concrete	1924	$ 44,000
2		Margarita	2.4	Concrete	1964	$ 45,000
3		Country Club 1	1	Concrete	1969	$ 34,000
4		Dearborn	10	Concrete	1972	$ 56,000
5		Fifth Avenue	5	Concrete	1974	$ 50,000
6		Highland	10	Concrete	1976	$ 56,000
7		Texas Street	1	TBD		$ 34,000
					Total	$ 319,000

W6-1 CALCUALTION SUMMARY TABLES

	LIST OF WELLS FROM REPORT TABLE				
#	Name	Discharge to Z	Capacity (GPM)	Ground Elev. (FT)	Water Surface Elev. (FT)
1	North Orange Street 1	1350	2900	3050	293
2	North Orange Street 2	1350	2900	3105	288
3	10	1570	1500	1650	80
4	13	1570	3000	3006	171
5	38	1570	1500	1634	520
6	39	1570	1250	1255	535
7	Church Street	1570	2000	2136	485
8	Orange Street	1570	1500	1165	459
9	Agate 2	1750	1500	1646	299
10	Airport 1	1750	1700	1316	498
11	Airport 2	1750	1100	119	545
12	Mentone Acres 2	1750	1900	1787	491
13	Rees	1750	1200	1964	544
14	Muni	1750	N/A	N/A	N/A
15	Madeira	1900	750	697	399
16	Lugonia 3	2100	250	500	N/A
17	Lugonia 4 - INACTIVE?	2100	1300	UNK	UNK
18	Lugonia 6	2100	250	N/A	N/A
19	Maguet 2	2100	300	249	253
20	Crafton	2100	1600	N/A	N/A
The City of Redlands Facilities Data Information					

	THIS IS THE TABLE FOR THE CIP - WELL REHABILITATION						
Index	LIST OF WELLS THAT DANIEL SENT	Zone?	Capacity GPM	Efficiency	HP	Rehab Budget	Rehab Year
18	Well 10	1570	1400	0.27	75	$1,050,000	2022
12	Maguet 2	2100	400	0.35	25	$300,000	2022
17	Rees	1750	550	0.43	250	$412,500	2023
9	Lugonia 3	2100	250	0.43	25	$0	2023
10	Lugonia 6	2100	250	0.60	30	$187,500	2023
11	Madeira	1900	600	0.62	150	$450,000	2023
3	Crafton	2100	1700	0.65	200	$1,275,000	2024
7	Airport 2	1750	1000	0.68	300	$750,000	2025
6	Airport 1	1750	1500	0.70	350	$1,125,000	2025
13	Mentone Acres 2	1750	1600	0.71	300	$1,200,000	2026
5	Orange Street	1570	1500	0.74	300	$1,125,000	2027
1	North Orange Street 1	1350	2900	0.77	350	$2,175,000	2028
8	Church Street	1570	2000	0.77	400	$1,500,000	2029
21	Well 39	1570	1250	0.78	250	$937,500	2030
4	North Orange Street 2	1350	2900	0.78	350	$2,175,000	2031
20	Well 38	1570	1600	0.78	300	$1,200,000	2032
2	Agate 2	1750	1750	0.79	200	$1,312,500	2033
20	Lugonia 4		1300	UNK	UNK	$975,000	2034
16	Muni	1750	UNK	UNK	UNK	$1,000,000	2035
19	Well 13	1570	UNK	UNK	UNK	$1,000,000	2036
14	Mill Creek 2 - Surface Water		UNK	UNK	UNK	$1,000,000	2037
15	Mill Cereek 2A - Surface Water		UNK	UNK	UNK	$1,000,000	2038

NP 1	Non-potable Water Improvements	2022	2023	2024	2025	2026	5-year Total		2022-2026	2027-2031	2032-2036	2037-2041	20-year Total
NP 1.1	Pipeline Replacement for Future System	$ 756,000	$ 991,000	$ 1,177,000			$ 2,924,000		$ 2,924,000	$ -	$ -	$ -	$ 2,924,000
NP 1.2	Groundwater Well Equipment Rehabilitation	$ 637,500	$ 225,000	$ 375,000	$ 1,125,000	$ 1,125,000	$ 3,487,500		$ 3,487,500	$ 3,562,500	$ 2,962,500	$ 2,062,500	$ 12,075,000
NP 1.3	Meter replacement and calibration	$ 11,000	$ 11,000	$ 11,000	$ 11,000	$ 11,000	$ 55,000		$ 55,000	$ 55,000	$ 55,000	$ 55,000	$ 220,000
CIP NP 1	Subtotal	$ 1,404,500	$ 1,227,000	$ 1,563,000	$ 1,136,000	$ 1,136,000	$ 6,466,500		$ 6,466,500	$ 3,617,500	$ 3,017,500	$ 2,117,500	$ 15,219,000
NP 2	Recycled Water Improvements	2022	2023	2024	2025	2026	5-year Total		2022-2026	2027-2031	2032-2036	2037-2041	20-year Total
NP 2.1	Recycle Water Reservoirs - design two & build in	$ 200,000	$ 1,400,000	$ -	$ 200,000	$ 1,400,000	$ 3,200,000		$ 3,200,000				$ 3,200,000
CIP NP 2	Subtotal	$ 844,000	$ 1,809,000	$ 223,000	$ 1,600,000	$ 2,800,000	$ 7,276,000		$ 7,276,000	$ 3,700,000	$ 3,700,000	$ 3,700,000	$ 18,376,000
	Optional Expansion	2022	2023	2024	2025	2026	5-year Total		2022-2026	2027-2031	2032-2036	2037-2041	20-year Total
	Expansion of the Recycled/Non-pootable Water	$ 2,500,000	$ 2,500,000	$ 2,500,000	$ 2,500,000	$ 2,500,000	$ 12,500,000		$ 12,500,000	$ 12,500,000	$ 12,500,000	$ 12,500,000	$ 50,000,000
CIP Optio	Subtotal	$ 2,500,000	$ 2,500,000	$ 2,500,000	$ 2,500,000	$ 2,500,000	$ 12,500,000		$ 12,500,000	$ 12,500,000	$ 12,500,000	$ 12,500,000	$ 50,000,000

Proj	2022-2026	2027-2031	2032-2036	2037-2041	20-year Total CIP Cost
NP 1	$ 6,466,500	$ 3,617,500	$ 3,017,500	$ 2,117,500	$ 15,219,000
NP 2	$ 7,276,000	$ 3,700,000	$ 3,700,000	$ 3,700,000	$ 18,376,000
NP 3	$ 12,500,000	$ 12,500,000	$ 12,500,000	$ 12,500,000	$ 50,000,000
Tota	$ 13,742,500	$ 7,317,500	$ 6,717,500	$ 5,817,500	$ 33,595,000

CIP NP 1-2 Groundwater Well Equipment Rehabilitation

	Name	Discharge to Zone	**Test #**	HP	**Head**	**Capacity**	**Overall Eff**	**Improved Eff**	**Total KwH**	**Kw Input**	**Cost per Kwh**	**Average cost per Acre Ft**
1	California	1	1	100	462.80	519	52.00%	68.00%	215784	87	0.14	129.09
4	New York Street	2	1	150	226.1	1429	57.00%	70.00%	536952	106.8	0.13	51.59
			2	150	235.30	1558	52.80%					
7	30A	2	1	150	254.00	1566	60.90%	70.00%	118356	123	0.16	66.41
			2	150	276.50	1463	61.40%					
7	31A	2	1	450	258.60	878	31.90%	72.00%	475248	134	0.13	107.75
7	32	2	1	200	269.80	1854	60.50%	Half of the data in this file is for pump 36				
9	41	2	1	100		825			74940	73.7	0.25	122.5
6	11	detached systems	1	60	176.80	311	47.90%	65.00%	39804	21.6	0.17	64.12
6	14	detached systems	1	125	125.20	2261						
			2	125	138.60	2176						
			3	125	149.30	2152						
6	16	detached systems	1	150	103.80	1202	50.50%	71.00%	99744	46.5	0.15	31.62
			2	150	143.80	1879	55.90%					
10	Agate 1	detached systems	1	200	307.10	1816	77.50%		177240	135.5	0.16	62.81
2	Chicken Hill	detached systems	1	75	233.60	653	61.50%		5496	46.7	0.5	193.74
			2	75	252.80	618	63.20%					
			3	75	272.50	578	64.20%					
3	Crafton	detached systems	1	200	254.60	1754	59.80%	70.00%	42830	120.11	0.37	161.68
11	Hog Canyon	detached systems	1	100	282.70	950	66.40%		121	76.2	15.23	6641.61
									These values do not make sense.			

Name	Capacity	Status in 2019	Efficiency	HP	Rehab Budget	CIP Year
31A	850	idle	31.9%	450	$637,500	2022
11	300	In Use	47.9%	60	$225,000	2023
California	500	In Use	52.0%	100	$375,000	2024
16	1500	In Use	53.2%	150	$1,125,000	2025
New York Street	1500	In Use	54.9%	150	$1,125,000	2026
41	800	In Use	N/A	100	$600,000	2027
14	2200	In Use	N/A	125	$1,650,000	2028/2029
Crafton	1750	idle	59.8%	200	$1,312,500	2030/2031
32	1850	In Use	60.5%	200	$1,387,500	2032/2033
30A	1500	In Use	61.2%	150	$1,125,000	2034/2035
Chicken Hill	600	In Use	63.0%	75	$450,000	2036
Hog Canyon	950	Idle	66.4%	100	$712,500	2037
Agate 1	1800	Idle	77.5%	200	$1,350,000	2038
					$12,075,000	

CIP NP 1-3 Meter Replacement

Size (in)	r of meters	Cost per m	Total Cost	Meter Cost	Strainer	Lid	Install
3/4""	8	$ 350	$ 2,800	229.14		45	79.36
1""	22	$ 420	$ 9,240	294.9		45	79.36
1 1/2""	27	$ 1,000	$ 27,000	503.55		100	396.77
2""	117	$ 1,140	$ 133,380	646.5		100	396.77
4""	4	$ 3,050	$ 12,200	2000	450		595.15
6""	2	$ 7,650	$ 15,300	4900	770		1984
Unknown	4	$ 2,500	$ 10,000	1000	500		1000
Total	184						1.03
				Meter Calibration			
4""	4	$ 380	$ 1,520	300	80		
6""	2	$ 380	$ 760	300	80		
			$ 212,200				

Non-potable /Recycled Water Pipeline Analysis

Material Average Service Life		
Abv.	Material	Service Life
ACP	Asbestos Concrete Pipe	70
CIP	Cast Iron Pipe	120
CMLC	Cement Mortar Lined and Coated	100
CMLDIP	Cement Mortar Lined Ductile Iron Pipe	100
CMLSTL	Cement Mortar Lined Steel Pipe	100
CON	Concrete	100
COP	Copper	50
DIP	Ductile Iron Pipe	100
DW	Dipped and Wrapped	40
HDPE	High Density Polyethylene	50
OTH	Other	40
PVC	Polyvinyl Chloride	70
RCP	Reincforced Concrete Pipe	100
STL	Steel	100
Unk	Unknown	40
VCP	Vitrified Clay Pipe	50

Age	Length																	
Diameter (in)	Error Check		1	1.5	2	3	4	6	8	10	12	14	15	16	18	20	24	UNK
Error (lower than -100)	31324.65896	31324.65896	170.0532008	0	7650.993724	5.960794973	129.7816831	0	1503.606909	2838.241313	1635.244562	0	4479.447815	7375.907811	0	0	0	5535.421146
Less than 0 Greater or equal to -100	9941.277878	9941.277878	0	0	0	0	38.47763056	0	0	0	0	0	0	528.9785146	678.2402007	8695.581532	0	0
Less than or equal to 5 and greater than or equal to 0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Greater than 5 less than or equal to 10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Greater than 10 less than or equal to 15	32.99984171	32.99984171	0	0	32.99984171	0	0	0	0	0	0	0	0	0	0	0	0	0
Greater than 15 less than or equal to 20	9145.788628	9145.788628	0	0	0	0	145.6934138	347.4828578	1153.894925	0	66.26587706	0	1074.792121	0	792.4039506	5564.24105	0	1.014433667
Greater than 20 less than or equal to 25	724.4704218	724.4704218	0	0	11.41977579	0	0	0	586.0381374	10.08172033	0	0	0	0	0	116.9307883	0	0
Greater than 25 less than or equal to 30	13025.334	13025.334	0	0	115.1688049	0	1272.288579	2737.98753	702.37107	0	924.2124762	7273.305542	0	0	0	0	0	0
Greater than 30 less than or equal to 35	1501.374678	1501.374678	0	0	1198.626349	0	0	10.27101633	292.4773135	0	0	0	0	0	0	0	0	0
Greater than 35 less than or equal to 40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Greater than 40 less than or equal to 45	5913.238597	5913.238597	0	0	0	0	2116.021845	0	328.2039271	0	0	0	0	3016.261109	310.5901137	142.1616023	0	0
Greater than 45 less than or equal to 50	11713.52697	11713.52697	0	0	0	0	0	10926.02914	0	0	0	0	0	754.1529482	0	33.34487264	0	0
Greater than 50 less than or equal to 55	66463.12701	66463.12701	0	0	6573.923512	0	127.1116537	22100.41276	5988.10909	1660.127735	12660.13005	0	470.0535481	3565.89249	0	609.0151693	12708.351	0
Greater than 55 less than or equal to 60	16751.61504	16751.61504	0	0	0	0	0	10603.10546	1178.507472	0	3415.056614	0	0	1554.945491	0	0	0	0
Greater than 60 less than or equal to 65	1893.355577	1893.355577	0	0	178.0873316	0	20.85264859	0	0	0	1694.415597	0	0	0	0	0	0	0
Greater than 65 less than or equal to 70	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Greater than 70 less than or equal to 75	972.2601155	972.2601155	0	0	0	0	0	39.99993194	0	0	932.2601835	0	0	0	0	0	0	0
Greater than 75 less than or equal to 80	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Greater than 80 less than or equal to 90	13262.0683	13262.0683	0	0	413.138807	0	330.6263726	7587.508824	425.2161629	1357.952259	12.9633912	0	0	3125.475221	0	0	9.187265354	0
Greater than 90 less than or equal to 100	30975.44527	30975.44527	126.6871248	11.26999086	981.7236462	0	33.61925957	3826.239269	13558.25638	0	12437.6496	0	0	0	0	0	0	0
Error (greater than 100)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Total	213640.5413	213640.5413	296.7403256	11.26999086	17156.08179	5.960794973	4214.473086	58179.03679	25716.68138	5866.403027	33778.19836	7273.305542	6024.293484	19921.61358	1781.234265	15161.27501	12717.53826	5536.43558
Total (miles)	40.46222373	3052.007733																

Cost ($/in-ft)
19.5

Age	Length																	
Diameter (in)			8	8	8	8	8	8	8	10	12	14	15	16	18	20	24	30
Error (lower than -100)	31324.65896	$ 9,261,669.20	$ 26,528.30	$ -	$ 1,193,555.02	$ 929.88	$ 20,245.94	$ -	$ 234,562.68	$ 553,457.06	$ 382,647.23	$ -	$ 1,310,238.49	$ 2,301,283.24	$ -	$ -	$ -	$ 3,238,221.37
Less than 0 Greater or equal to -100	9941.277878	$ 3,800,382.91	$ -	$ -	$ -	$ -	$ 6,002.51	$ -	$ -	$ -	$ -	$ -	$ -	$ 165,041.30	$ 238,062.31	$ 3,391,276.80	$ -	$ -
Less than or equal to 5 and greater than or equal to 0	0	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Greater than 5 less than or equal to 10	0	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Greater than 10 less than or equal to 15	32.99984171	$ 5,147.98	$ -	$ -	$ 5,147.98	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Greater than 15 less than or equal to 20	9145.788628	$ 3,035,607.26	$ -	$ -	$ -	$ -	$ 22,728.17	$ 54,207.33	$ 180,007.61	$ -	$ 15,506.22	$ -	$ 314,376.70	$ -	$ 278,133.79	$ 2,170,054.01	$ -	$ 593.44
Greater than 20 less than or equal to 25	724.4704218	$ 140,772.38	$ -	$ -	$ 1,781.49	$ -	$ -	$ -	$ 91,421.95	$ 1,965.94	$ -	$ -	$ -	$ -	$ -	$ 45,603.01	$ -	$ -
Greater than 25 less than or equal to 30	13025.334	$ 2,955,017.43	$ -	$ -	$ 17,966.33	$ -	$ 198,477.02	$ 427,126.05	$ 109,569.89	$ -	$ 216,265.72	$ 1,985,612.41	$ -	$ -	$ -	$ -	$ -	$ -
Greater than 30 less than or equal to 35	1501.374678	$ 234,214.45	$ -	$ -	$ 186,985.71	$ -	$ -	$ 1,602.28	$ 45,626.46	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Greater than 35 less than or equal to 40	0	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Greater than 40 less than or equal to 45	5913.238597	$ 1,486,832.84	$ -	$ -	$ -	$ -	$ 330,099.41	$ -	$ 51,199.81	$ -	$ -	$ -	$ -	$ 941,073.47	$ 109,017.13	$ 55,443.02	$ -	$ -
Greater than 45 less than or equal to 50	11713.52697	$ 1,952,760.77	$ -	$ -	$ -	$ -	$ -	$ 1,704,460.55	$ -	$ -	$ -	$ -	$ -	$ 235,295.72	$ -	$ 13,004.50	$ -	$ -
Greater than 50 less than or equal to 55	66463.12701	$ 16,148,439.54	$ -	$ -	$ 1,025,532.07	$ -	$ 19,829.42	$ 3,447,664.39	$ 934,145.02	$ 323,724.91	$ 2,962,470.43	$ -	$ 137,490.66	$ 1,112,558.46	$ -	$ 237,515.92	$ 5,947,508.27	$ -
Greater than 55 less than or equal to 60	16751.61504	$ 3,122,197.86	$ -	$ -	$ -	$ -	$ -	$ 1,654,084.45	$ 183,847.17	$ -	$ 799,123.25	$ -	$ -	$ 485,142.99	$ -	$ -	$ -	$ -
Greater than 60 less than or equal to 65	1893.355577	$ 427,527.89	$ -	$ -	$ 27,781.62	$ -	$ 3,253.01	$ -	$ -	$ -	$ 396,493.25	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Greater than 65 less than or equal to 70	0	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Greater than 70 less than or equal to 75	972.2601155	$ 224,388.87	$ -	$ -	$ -	$ -	$ -	$ 6,239.99	$ -	$ -	$ 218,148.88	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Greater than 75 less than or equal to 80	0	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Greater than 80 less than or equal to 90	13262.0683	$ 2,613,294.50	$ -	$ -	$ 64,449.65	$ -	$ 51,577.71	$ 1,183,651.38	$ 66,333.72	$ 264,800.69	$ 3,033.43	$ -	$ -	$ 975,148.27	$ -	$ -	$ 4,299.64	$ -
Greater than 90 less than or equal to 100	30975.44527	$ 5,802,306.13	$ 19,763.19	$ 1,758.12	$ 153,148.89	$ -	$ 5,244.60	$ 596,893.33	$ 2,115,087.99	$ -	$ 2,910,410.01	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Error (greater than 100)	0	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Total	213640.5413	$ 51,210,559.99	$ 46,291.49	$ 1,758.12	$ 2,676,348.76	$ 929.88	$ 657,457.80	$ 9,075,929.74	$ 4,011,802.30	$ 1,143,948.59	$ 7,904,098.42	$ 1,985,612.41	$ 1,762,105.84	$ 6,215,543.44	$ 625,213.23	$ 5,912,897.26	$ 5,951,807.91	$ 3,238,814.81

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APPENDIX D

Water System Inefficiency Cost Opinion

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	Interpulation									
2020	2022	2025	2027	2030	2032	2035	2037	2040	2042	2045
18.90	19.10	19.40	19.72	20.20	20.48	20.90	21.14	21.50	21.78	22.20

	2022*	2027	2032	2037	2042
Projected ADD	18.90	19.72	20.48	21.14	21.78
Projected MDD	32.13	33.52	34.82	35.94	37.03
Projected PHD	51.98	54.23	56.32	58.14	59.90
Expected Inefficiencies (%)	**12.8**	**8**	**6**	**4**	**3**
Projected ADD production	21.32	21.30	21.71	21.99	22.43
Delta inefficiencies	2.42	1.58	1.23	0.85	0.65
Delta inefficiencies (afy)	2,710	1,767	1,376	947	732
inEfficiencies Monetization					
Delta inefficiencies Lost Billing Charges ($)	2,333,439	1,521,674	1,185,239	815,623	630,237
Delta inefficiencies Lost Purchase Charges ($), based on $500 $/(afy)well water price	1,354,752	883,456	688,128	473,536	365,904
Delta inefficiencies Power Use ($) 60% of Billing charges	1,400,063	913,004	711,143	489,374	378,142
Delta inefficiencies O&M ($) 30% of B illing charges	700,032	456,502	355,572	244,687	189,071
Delta inefficiencies New Capacity Planning ($) 30% of B illing charges	700,032	456,502	355,572	244,687	189,071
TOTAL Monatized Inefficienices	6,488,318	4,231,139	3,295,654	2,267,907	1,752,425
AVERAGE Monatized Inefficienices	3,607,089				
Reduction in inefficiencies ($)	-	2,257,179	3,192,664	4,220,411	4,735,893
AVERAGE Monatized Reduction in Inefficienices	**$ 2,881,229**				

Where

* 2020 Urban water management plan water supply data was used to approximate the 2022 base line level

1,000,000 gpd 1,120 afyr

City of Redlands 2021 Water use rate

($)

1 mg

1.46	100	cft	748	gal	1,952
1.78			748		2,380
2.69			748		3,596
Average					2,643

City of Redlands 2021 MP Data	ADD	PDD
	(mgd)	(mgd)
2022	18.9	32.13
2042	21.78	37.026

The City of Redlands
CIP implementation and Water Supply Inefficiencies reduction 2022 to 2042

	2022*	**2027**	**2032**	**2037**	**2042**
Projected ADD	18.90	19.72	20.48	21.14	21.78
Projected MDD	32.13	33.52	34.82	35.94	37.03
Projected PHD	51.98	54.23	56.32	58.14	59.90
Expected Inefficiencies (%)	**12.8**	**10**	**8**	**6**	**4**
Projected ADD production	21.32	21.69	22.12	22.41	22.65
Delta inefficiencies	2.42	1.97	1.64	1.27	0.87
Delta inefficiencies (afy)	2,710	2,209	1,835	1,421	976
inEfficiencies Monetization					
Delta inefficiencies Lost Billing Charges ($)	2,333,439	1,902,092	1,580,319	1,223,435	840,316
Delta inefficiencies Lost Purchase Charges ($), based on $500 $/(afy)well water price	1,354,752	1,104,320	917,504	710,304	487,872
Delta inefficiencies Power Use ($) 60% of Billing charges	1,400,063	1,141,255	948,191	734,061	504,190
Delta inefficiencies O&M ($) 30% of B illing charges	700,032	570,628	474,096	367,031	252,095
Delta inefficiencies New Capacity Planning ($) 30% of B illing charges	700,032	570,628	474,096	367,031	252,095
TOTAL Monatized Inefficienices	6,488,318	5,288,923	4,394,205	3,401,861	2,336,567
AVERAGE Monatized Inefficienices	4,381,975				
Reduction in inefficiencies ($)	-	1,199,395	2,094,113	3,086,457	4,151,751
AVERAGE Monatized Reduction in Inefficienices	**$ 2,106,343**				

Where
* 2020 urban water management plan water supply data was used to approximate the 2022 base line level

1,000,000 gpd   1,120 afyr

City of Redlands 2021 Water use rate   1 mg
($)

	1.46	100	cft	748	gal	1,952
	1.78			748		2,380
	2.69			748		3,596
Average						2,643

	ADD	PDD
	(mgd)	(mgd)
2022	18.9	32.13
2042	22.17	37.689

APPENDIX E

CIP Projects Remaining After Quality Control

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City of Redlands

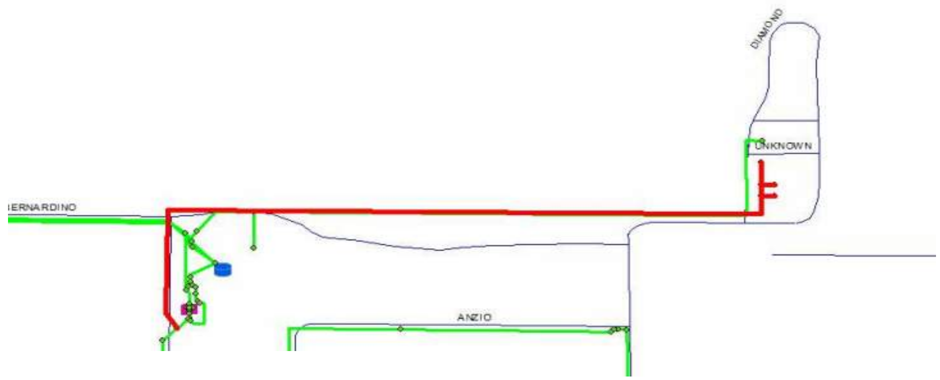
CIP project list based on QA/QC feedback review

CIP_1_Removed

CIP_2_Removed

CIP_3

MOID	MATERIAL	LENGTH	DIAMETER
14486	STL	1877.16	24
14487	STL	62.16	24
14488	STL	29.21	24
14490	STL	31.13	24
14491	STL	5.00	24
14492	STL	31.13	24
14493	STL	5.00	24
14485	STL	5.45	24



CIP_4

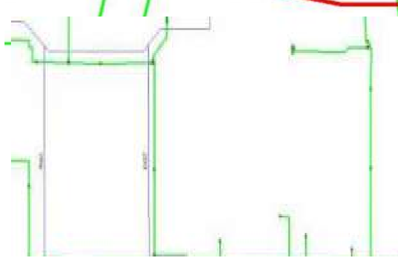
MOID	MATERIAL	LENGTH	DIAMETER
18405	STL	112.16	6
18406	ACP	85.52	8
23657	DIP	2.43	6
22928	DIP	3.26	6

CIP_5_Removed

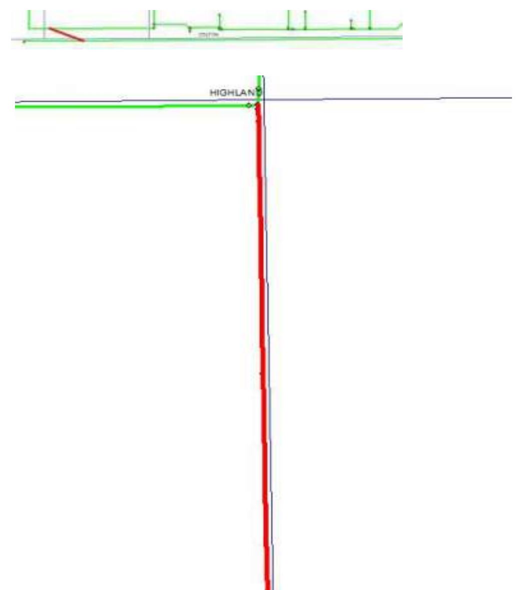
CIP_6_Removed

CIP_7

MOID	MATERIAL	LENGTH	DIAMETER
P839	PVC	76.03	8
P841	DIP	165.99	8

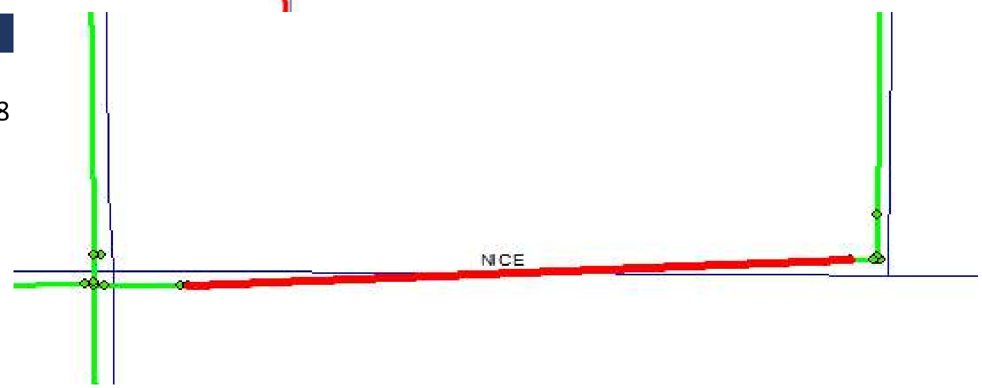


CIP_8			
MOID	MATERIAL	LENGTH	DIAMETER
404	ACP	10.23	8
3647	STL	48.24	8
3648	STL	15.52	8
3940	ACP	537.53	8
3946	ACP	620.80	8
3948	ACP	30.02	8
8150	STL	5.01	8

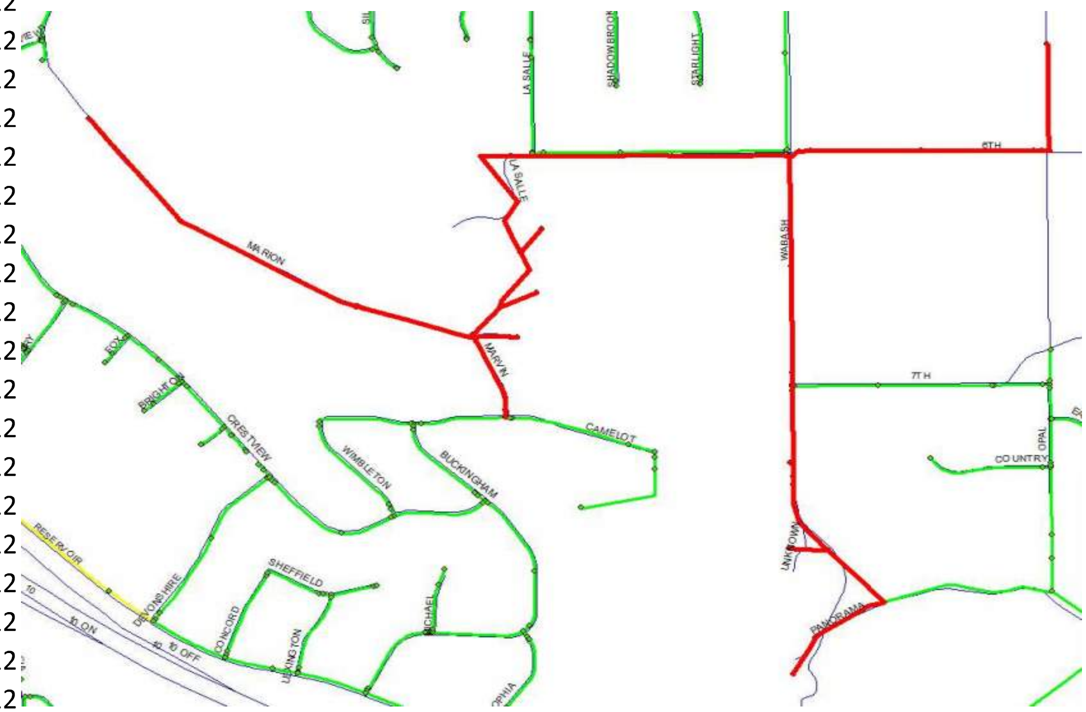


CIP_9_Removed

CIP_10			
MOID	MATERIAL	LENGTH	DIAMETER
P843	ACP	648.84	8



CIP_11			
MOID	MATERIAL	LENGTH	DIAMETER
290	ACP	20.11	12
292	ACP	10.00	12
1557	STL	374.44	12
3535	ACP	241.62	12
3536	ACP	418.31	12
3539	ACP	403.12	12
3646	ACP	41.92	12
3656	ACP	577.24	12
7463	ACP	43.91	12
7473	ACP	425.62	12
7474	ACP	28.04	12
7476	ACP	221.41	12
7477	STL	202.35	12
7478	STL	175.55	12
7510	STL	217.72	12
7539	STL	212.98	12
7540	ACP	4.72	12
7541	ACP	3.94	12
10016	ACP	211.18	12
10017	ACP	352.80	12
10018	ACP	569.41	12
10019	PVC	56.86	12
10021	PVC	5.00	12
10022	PVC	829.56	12
10023	STL	584.89	12
10024	PVC	3.33	12
13970	ACP	28.03	12
13975	ACP	424.89	12
13976	ACP	17.74	12
13978	ACP	65.80	12

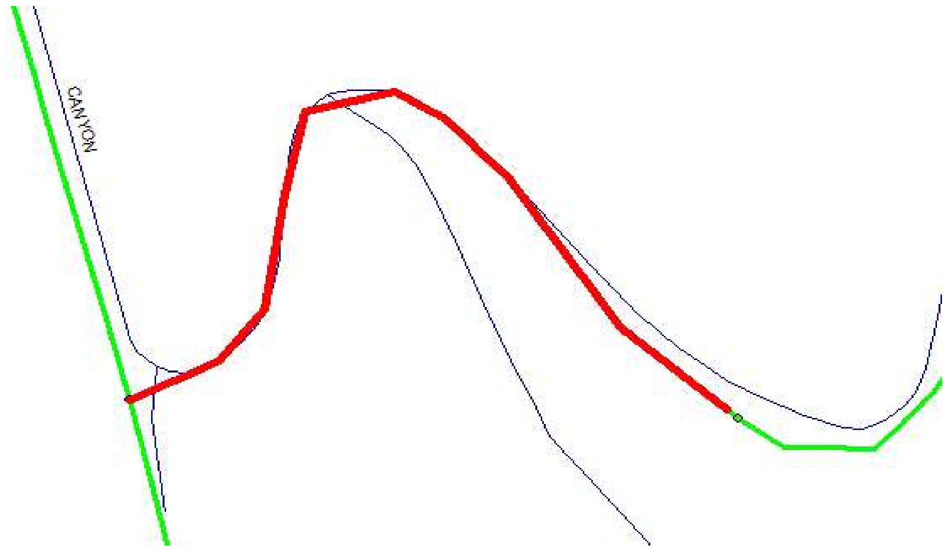


13980	ACP	431.04	12
13983	STL	612.84	12
13984	PVC	57.76	12
13985	PVC	6.31	12
13986	ACP	38.85	12
13987	PVC	4.16	12
13988	PVC	3.73	12
13989	PVC	3.64	12
13990	ACP	626.46	12
14057	ACP	182.44	12
18275	STL	69.05	12
18276	STL	5.94	12
22955	DIP	11.79	12
22958	DIP	478.79	12
22961	DIP	260.12	12
22964	DIP	438.68	12
22967	DIP	603.16	12
22968	DIP	2.69	12
22969	DIP	1.13	12
22970	DIP	11.14	12
22973	DIP	611.55	12
22974	DIP	3.56	12
22975	DIP	3.38	12
22976	DIP	5.38	12
22977	DIP	9.43	12
22978	DIP	1.32	12
22979	DIP	6.13	12
24762	CMLC	33.77	12
24755	CMLC	4.95	12
24756	CMLC	5.03	12
24757	CMLC	1.32	12
24761	CMLC	602.56	12
24758	CMLC	2.40	12
P523	ACP	49.32	12

P525	ACP	14.37	12
P745	DIP	2.41	12

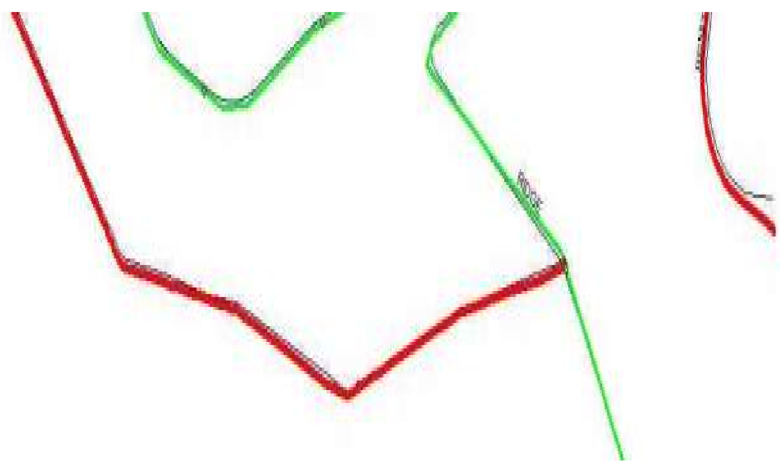
CIP_12_Removed

CIP_13



MOID	MATERIAL	LENGTH	DIAMETER
P845	DIP	503.31	6

CIP_14

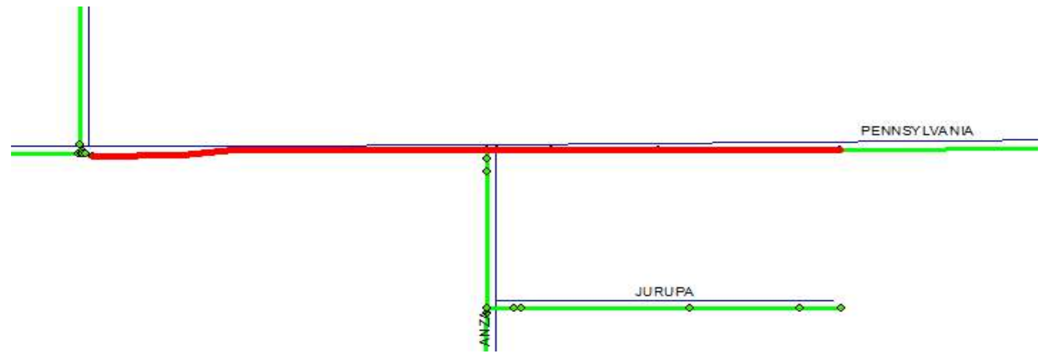


MOID	MATERIAL	LENGTH	DIAMETER
2104	ACP	490.32	8
2106	CMLC	262.70	8
2170	ACP	461.71	8
2190	ACP	28.35	8
7739	CMLC	7.01	8
7745	CMLC	5.66	6
7907	ACP	59.19	8
7908	ACP	288.88	8
7915	ACP	28.03	8
7916	ACP	19.93	8
7945	ACP	647.21	8
9325	ACP	552.38	8

9647	ACP	757.98	8
9651	STL	607.51	8

CIP_15

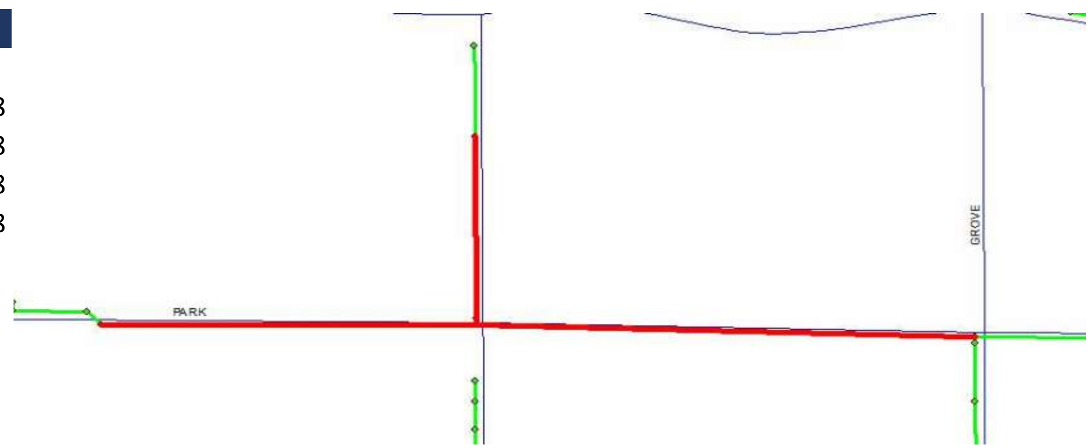
MOID	MATERIAL	LENGTH	DIAMETER
11929	ACP	275.74	8
11930	ACP	398.94	8
11932	ACP	204.19	8
11996	ACP	84.25	8
11997	ACP	163.35	8
23545	ACP	15.31	8



CIP_16_Removed

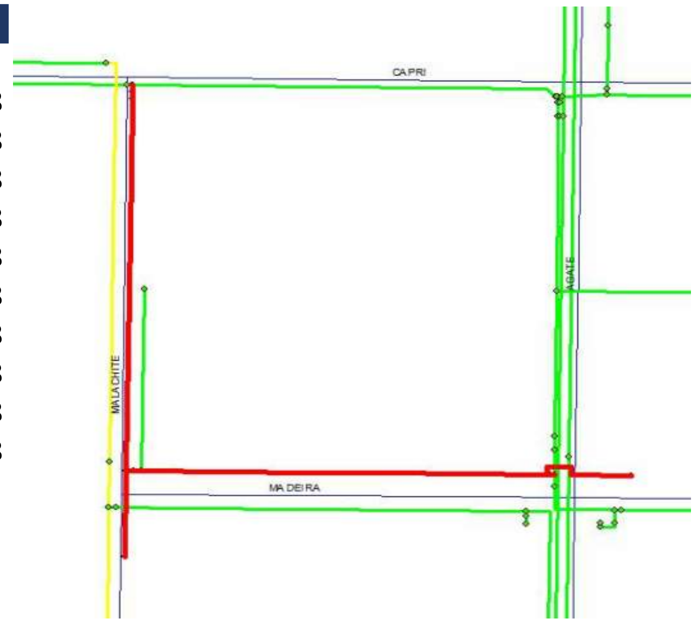
CIP_17

MOID	MATERIAL	LENGTH	DIAMETER
12966	STL	723.06	8
12950	STL	291.39	8
12953	STL	11.43	8
12962	STL	542.86	8



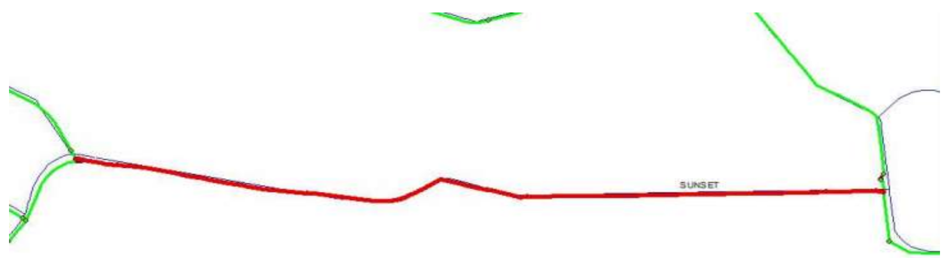
CIP_18

MOID	MATERIAL	LENGTH	DIAMETER
900	STL	7.50	8
1040	STL	7.51	8
5768	STL	94.55	8
5783	STL	7.48	8
5784	STL	408.92	8
18315	PVC	8.00	8
18221	STL	402.48	8
18225	STL	3.56	8
18226	STL	3.56	8
18229	STL	101.75	8



CIP_19

MOID	MATERIAL	LENGTH	DIAMETER
2192	ACP	481.89	12
7905	ACP	116.79	12
7906	ACP	369.41	12
7951	ACP	141.08	12
10728	ACP	11.73	12
16321	ACP	543.18	12
16323	ACP	96.76	12
16327	ACP	536.67	12
21695	DIP	21.53	12
P133	ACP	10.71	12



NP- CIP_1

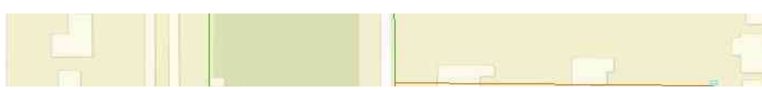
MOID	MATERIAL	LENGTH	DIAMETER
644	PVC	187.23	6
645	PVC	463.27	6
646	PVC	658.44	6
780	PVC	108.75	6
781	PVC	282.83	6
782	PVC	835.55	6
783	DIP	15.00	6
784	DIP	21.00	6
785	DIP	15.00	6
786	DIP	21.00	6
788	PVC	389.33	6
789	PVC	234.42	6

NP- CIP_2

MOID	MATERIAL	LENGTH	DIAMETER
268	PVC	20.91	6
269	PVC	273.46	6
758	PVC	144.27	6
759	PVC	39.67	6
760	PVC	1.52	6
761	PVC	4.00	6
762	PVC	95.47	6
763	PVC	5.00	6
768	PVC	478.41	6
770	PVC	75.26	6
834	PVC	22.59	6
839	PVC	43.17	6
840	PVC	178.15	6
841	PVC	10.00	6
848	PVC	113.93	6
849	PVC	393.77	6
1850	<Null>	617.72	8
1851	<Null>	532.52	8
1859	<Null>	102.09	8
1860	<Null>	216.25	8

NP- CIP_3

MOID	MATERIAL	LENGTH	DIAMETER
104	PVC	5.00	12
112	PVC	171.06	10
114	PVC	4.99	10
399	PVC	436.19	12
400	PVC	389.27	12
701	ACP	115.23	20



707	STL	93.05	12
708	STL	5.00	12
709	UNK	5.08	10
710	STL	4.29	12
712	STL	5.64	12
715	STL	11.10	12
716	STL	16.00	12
717	STL	781.62	12
720	STL	12.37	12
721	STL	2.97	12
723	STL	4.00	6
724	STL	11.00	6
725	STL	5.50	6
726	STL	5.50	6
727	STL	7.00	8
728	STL	6.44	8
729	STL	6.50	8
730	STL	8.96	8
731	STL	8.96	8
732	STL	6.00	8
733	PVC	8.20	8
734	STL	6.00	8
735	STL	16.14	8
736	STL	16.14	8
737	PVC	19.66	10
739	PVC	2.97	8
740	PVC	8.58	10

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APPENDIX F

Condition, Seismic, and Structural Assessment Executive Summary

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CONDITION, SEISMIC, AND STRUCTURAL ASSESSMENTS FOR THE WATER FACILITIES INFRASTRUCTURE

City of Redlands

Executive Summary

June 30, 2022

BRADY

Water • Power • Transportation

FINAL

Executive Summary

From: **Richard Brady, P.E., BCEE, CEO, Richard Brady & Associates, Inc.**
Project Manager

To: **Veronica Medina, EIT, Project Manager**
Paul Mariscal, Water Production Operations Superintendent

Date: June 30, 2022

Subject: Condition Assessment Summary and Recommended Action Plan

EXECUTIVE SUMMARY

<u>Scope of Work</u>

BRADY's Scope of Work was to provide professional engineering services to perform condition, seismic, and structural assessments for the City of Redlands water facilities infrastructure. The infrastructure evaluated included 18 reservoirs, 9 pump stations, and two pipelines.

<u>Recommended Capital Improvement Projects</u>

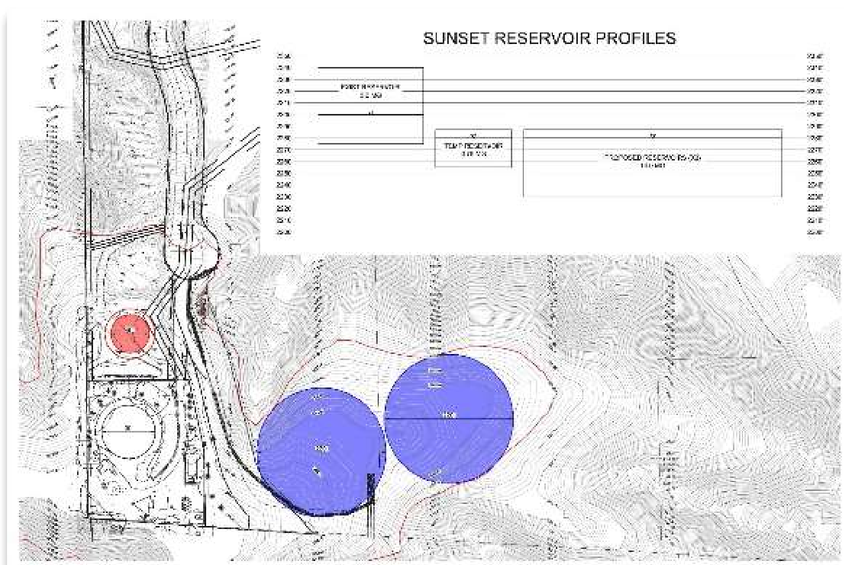
In summation, in order of priority, the recommended capital improvement projects that require the City's action to improve water security in a future seismic event are as follows:

1. **New Sunset Reservoir(s)**. The Sunset Reservoir dates to 1967. It is in extremely poor condition with no seismic restraint and has a lead paint coated interior. The soil below the reservoir is fractured granite and potentially unstable. A new reservoir in this location would require the top ten feet of soil to be removed. Large Verizon cell towers have been installed adjacent to the reservoir. In a major seismic event, the Sunset Reservoir is in danger of either tipping over into the canyon below, or in the other direction, into the cell towers that would affect cell phone communications for the region, causing critical problems if the towers were damaged. The recommendation is broken into two separate phases.

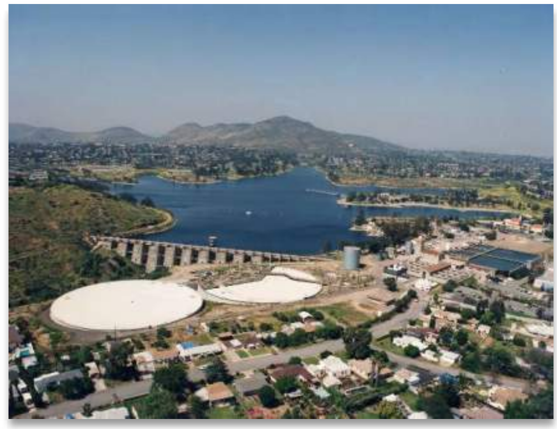
Given the lack of seismic anchorage and likely overturning in a major seismic event, the first phase is to erect a temporary 750,000-gallon bolted steel tank on the parcel north of existing reservoir. The City can purchase this temporary tank, which can be disassembled, stored, and re-used in the future where needed in the system, possibly as a reclaimed water tank in the Crafton Hills college area. This action is necessary because the Sunset Reservoir has no back-up reservoir to service the 2340 service Zone in this area. The Sand Canyon Reservoir also services the 2340 Zone, but it not hydraulically connected to the Sunset service area; they operate independently of one another.

We have studied an option to construct a new partially buried prestressed concrete reservoir in an adjacent parcel that the City may purchase assuming a successful negotiation is concluded. If the land purchase is not successful, then the recommendation is to construct a new 3 MG prestressed concrete reservoir on the existing tank site.

If the negotiation is successful, the second phase is to construct a new partially buried 10 MG prestressed concrete reservoir in the City's parcel adjacent to the existing Sunset site. The property has 4 separate but adjoining parcels, 22 acres in total. The new reservoir can comfortably be constructed in this 22-acre parcel, with room for a possible second 10 million gallon (MG) reservoir at some future date. These are the largest reservoirs that can be constructed at this site at the right hydraulic elevation. The existing Sunset Reservoir is only 3 MG but the additional 7 MG at this location will account for the 6 MG of lost storage in the City due to reduced high-water levels (HWL) to meet current seismic codes. The other major benefit will be to significantly improve the City's water reliability and resiliency by having a large water volume that can serve the City by gravity in the instance that power is lost in a seismic event. The additional 6 MG of storage is a placeholder for more storage that will be lost if a reservoir consolidation plan is accepted by the City and other stakeholders (Municipal Utilities/Public Works Commission). Sunset Reservoir is one of the City's most valuable assets as it sits at the highest elevation in the City, allowing gravity flow to 97% of the City's



population. The recommended size is 10 MG. A proposed layout is illustrated in the image to the right. The layout shows a potential second reservoir, if needed, to meet water demands at some future date. These two reservoirs are illustrated in blue; the temporary 750,00-gallon tank is illustrated in red.

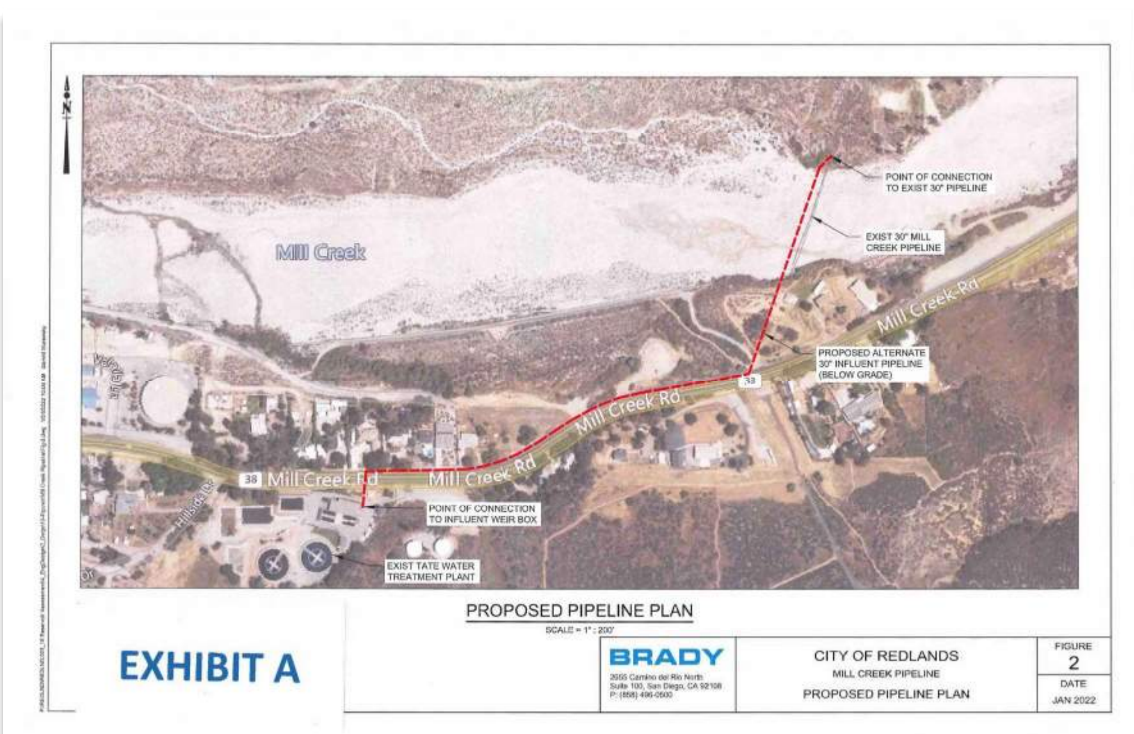


The proposed layout would be similar to a 1994 BRADY design for two new 21 MG prestressed concrete reservoirs at the Alvarado Water Treatment Plant (WTP) in the City of San Diego. The adjacent photo of the completed project will help visualize what this could look like. The reservoir on the right includes a Native Plant Demonstration Garden on the reservoir for public education purposes of native, drought tolerant landscaping. Similar joint public use can be considered for the new Sunset Reservoir site, which provides an impressive 360-degree view of the area.

2. **(a) Second Mill Creek Pipeline**. The poor condition of this pipe presents a significant concern about the extended operating life of this asset. The pipe is elevated above Mill Creek on concrete piers spaced approximately 40 feet apart. Numerous large boulders span the length and width of Mill Creek. The chance of failure of this pipe during a major seismic event is significant, as well as during a major storm event where large boulders could damage the piers. Historical boulder impacts to the existing support piers are visually evident. To improve overall reliability of this critical asset, several pipe replacement alternatives will be considered. A second pipeline is only one of these

alternatives. The Mill Creek Pipeline delivers raw water for treatment at Tate WTP. This pipeline is critically important to the City's water security because the Tate WTP can serve more than 85% of the City's entire population by gravity flow. Maintaining continuous operations of the Tate WTP is therefore essential in a future major seismic event. A possible solution is to construct a second 30-inch diameter inlet pipeline parallel to the existing pipeline **under** Mill Creek. The pipe should parallel the existing pipe in Mill Creek Road all the way to the inlet structure at the Tate WTP. A dedicated inlet structure should be constructed to receive this new pipeline, and then connect to the existing inlet box for distribution to the reactor clarifiers. The new pipeline should also have a dedicated flow meter located at the plant site. A concept sketch is included below. The City has issued a Request for Proposals (RFP) for the design of this pipeline. Proposals are due February 29, 2022. The consultant selected to predesign and design this pipeline will also be tasked with developing a project budget. Therefore, our cost table below will not include a price for the new pipeline given this cost will be developed by others at some future date.

Additionally, once the new pipeline is in operation, the existing piers should be reinforced as necessary to provide additional security against catastrophic failure. Flexible expansion joints should be provided on each side of the creek where the pipe comes out of and re-enters the ground.



Once the new pipeline is constructed below Mill Creek, and the existing pipeline is rehabilitated with improvements to protect the support piers, the Tate WTP will have two independent pipelines delivering raw water to the plant. This is the ideal outcome for a water treatment facility as important as Tate – two separate ways to keep the plant in service during a major seismic event.

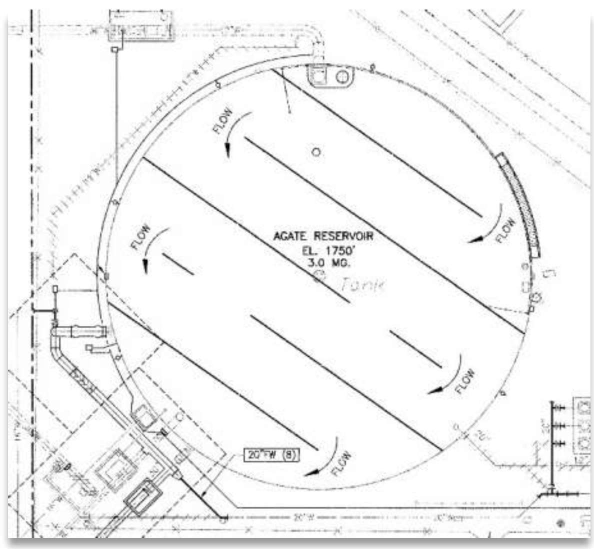
(b) Mill Creek 1 and 2 Tanks. The Mill Creek 1 and 2 tanks were erected in 2005 at 200,000 gallons each. The tanks are redundant to each other, so a minimum of 200,000 gallons of storage is needed at all times to provide the desired benefit – supplemental backwash supply to the Tate WTP.

Structural calculations to bring Mill Creek 1 and 2 into code compliance would require a significant reduction in operating volumes. Each tank would see a reduction from the design volume of 200,000 gallons to 90,420 gallons, to 45% of the original volume. The Mill Creek 1 and 2 tanks were not provided with seismic anchorage, are in danger of overturning in a seismic event, and lack sufficient freeboard to protect the tanks from damage due to sloshing.

Given the importance of the Tate WTP to the City in a major seismic event, and due to the critical importance of maintaining the assets needed to allow filter backwashing under low plant production rates, the Mill Creek 1 and 2 tanks are in need of rehabilitation. The easiest approach is to build a third Mill Creek tank, but space at the site is very limited. Though it is not a proven industry solution, serious consideration should be given to raising the reservoir roofs as needed to restore the operating volumes of each tank to 200,00 gallons. Seismic anchorage is also required. Using cost figures we recently developed for California American Water, a rehabilitation budget cost of $2.50 per gallon is the approximate cost per tank. Therefore, a budget figure of $500,000 is recommended per tank, $1M total, to resolve this problem as quickly as possible.

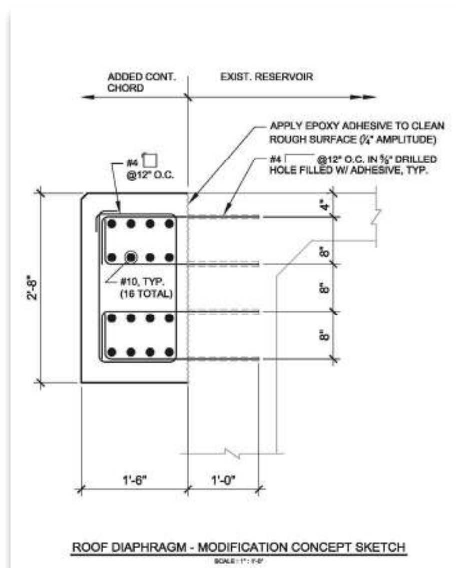
3. **Optimize Agate Reservoir and Construct New 60-inch Pipeline**. The Agate Reservoir dates to 1968 and serves as the clearwell reservoir for the Hinckley WTP. The reservoir does not meet current seismic design codes, but can be brought into compliance by reducing the operating capacity to 2,163,000 gallons, which will decrease stresses in the steel shell and increase the freeboard to allow for sloshing. However, the operating volume of the Agate Reservoir is critically important as the City achieves regulatory CT compliance (disinfectant concentration x time) inside of Agate Reservoir. The reservoir was retro-fitted in 2010. Hanging hypalon curtains were secured to the reservoir roof and floor to create a serpentine flow pattern inside reservoir to promote improved mixing for the purposes of obtaining the regulatory mandated CT credit. The City uses free chlorine as their primary and secondary disinfection method. The strength of a chemical disinfectant (chlorine) for inactivating pathogens when in contact with water can be measured by its CT value. The Hypalon curtains are shown in the figure at the right. There are many unintended consequences that have resulted from the installation of the baffle curtains. In a major seismic event, the sloshing forces of the water will act on the curtains, which in turn will collapse the roof, leading to complete reservoir failure. The curtains may also become dislodged and plug the outlet pipe serving gravity flow supply to the City's largest reservoirs – Dearborn at 10.6 MG and Highland at 10.0 MG. The possibility of this occurring will result in losing the Hinckley WTP supply as all of the plant effluent flows through Agate Reservoir. Hinckley provides gravity flow to 40% of the City's population.

Hypalon curtains shown anchored to the



A new 60-inch diameter pipe parallel to the existing 30-inch plant effluent pipeline will allow the City to achieve CT compliance in piping, not a baffled circular reservoir where the flow circulation pattern is not ideal. Together with the existing 30-inch pipeline, CT can be achieved in pipe, with no future reliance on the hypalon curtains hanging from the reservoir roof. Once the new pipe is installed, the hypalon curtains can be removed to protect the reservoir and the system so the existing Agate Reservoir can remain in service at a reduced operating level. The 60-inch pipe will also provide a supplemental storage volume of 0.22 MG.

4. **Highland Reservoir Seismic Upgrade**. The 10 MG rectangular concrete reservoir is in fair condition, but does require immediate action to correct a minor seismic deficiency. The Highland design has some unique structural features due to the presence of recreational activities on the reservoir roof. As a result of anticipating the associated live loads from these activities, the structure generally meets current seismic design codes. The problem to be solved relates to the roof diaphragm. The presence of slip dowels in the existing roof design (designed to allow for thermal expansion/contraction) do not transfer seismic loads from the roof diaphragm to the perimeter walls. In a (horizontal) seismic event, the roof slab could collapse. A structural beam



(reinforced with 16- #10 bars) around the entire perimeter will contain and transfer diaphragm loads to walls per the detail shown at the right.

5. **New 5th Ave Reservoir and Pump Station**. The 5 MG Fifth Avenue Reservoir is the only City reservoir that is near an active fault. This reservoir is a key reservoir for the Tate WTP as water transfers into the lower elevations. Water stored in Fifth Avenue is also pumped to the 2340 Zone (Sand Canyon Reservoir). The loss of Fifth Avenue for any reason would restrict operations. The site of the Fifth Avenue reservoir and pump station is located within a County Fault Zone for the Reservoir Canyon fault, which was adopted in the Countywide Policy Map in October 2020. As shown in the figures at the left and below, the edge of the County Fault Zone passes through the middle of the Fifth Avenue property. The Reservoir Canyon Fault is a normal fault that is part of the Crafton Hills Fault Zone. The fault trace is mapped about 400 feet away from the closest location to the Fifth Avenue reservoir.

The purple shaded zone on the County of San Bernardino fault map is a County Fault Zone – which is a zone of required investigation defined by the County of San Bernardino around the surface trace of an active fault that poses a risk of surface fault rupture in the future. Based on the Alquist-Priolo Act, fault zones are generally defined about 500 feet away from either side of a known fault (approximately 1,000 ft in width total) to account for the possibility that faults are not precisely identified and may occur in more than one branch in that area. The width of a fault zone can vary, for example, where faults are very well-defined at the surface, have narrow zones of deformation, and have vertical to sub-vertical inclinations, the zone can be narrower. Where faults are not well defined, contain wide zones of deformation, are segmented, or inclined at shallow angles, the fault zone may be wider to reflect this uncertainty. For the Reservoir Canyon Fault, the County Fault Zone appears to be about

1,200 feet in width (about 600 feet on either side of the inferred fault trace). The 400-foot distance described in the report is the distance between the Fifth Avenue Reservoir and the edge of the mapped fault zone, not the width of the fault zone itself.

The intent of earthquake fault zones is primarily to prevent the "location of developments and structures for human occupancy across the trace of active faults." This is primarily to eliminate the risk of surface fault rupture damage to the structure. This does not mean no development can occur in a fault zone. Instead, it means that a site-specific fault investigation must be performed to determine if an active fault crosses the site, locate it with as much precision as possible, and recommend appropriate structural setbacks from the active fault trace. The intent of the proposed exploratory fault trench is to investigate the undeveloped portion of the site that is within the County Fault Zone of required investigation. This investigation would be used to determine whether a fault is present at the site that may impact siting of a future reservoir.

Construction of a new tank at the site may be governed by the City of Redlands Development Code. However, for surface fault rupture hazards, the lead agency may be either the City of Redlands, or the County of San Bernardino, depending on who is generally tasked with reviewing site-specific fault investigations. Depending on the agency, local jurisdictions may defer to the County or State for these reviews."

Therefore, there may be potential for surface rupture, or associated off-fault deformations at the site from movement of the Reservoir Canyon fault reaching the ground surface at the Fifth Avenue site. The County of San Bernardino Development Code, Chapter 82.15.040 (b) requires that construction of any new structure used for critical facilities (such as a reservoir) be located 150 feet or farther from any active earthquake fault trace. As required by codes, new structures or improvements at the Fifth Avenue site will require a site-specific surface fault rupture investigation. We recommend excavating the fault trench location shown on the figure to the right as a part of any future reservoir improvement plan. The trench excavation will be approximately 3 feet wide and up to 10 feet deep.

Fortunately, the existing reservoir walls are completely buried with a column supported exposed concrete roof. Major structure damage that may occur in an earthquake will not result in a tidal wave of water causing damage to other structures. However, the loss of Fifth Avenue will cause an operational challenge for City staff. Replacing this reservoir with a prestressed concrete reservoir designed to AWWA D110, Type 1 (cast in place concrete walls) provides the City with the best protection in a future seismic event.

A Brief Clarification of Risk at the Fifth Avenue Reservoir Site

It should be noted that throughout the City's system, compliance with current seismic codes is clearly the goal, however, compliance with current code does not guarantee that a reservoir constructed before 1994 (Northridge earthquake, the last regional earthquake of significance) would not be seriously damaged in an earthquake. For example, most of the City's 12 steel tanks can be made code compliant by simply lowering the operating levels. Various recommendations to bring structures "up to code" only addresses the hazard of strong ground shaking. In no way does it mitigate the hazard of surface fault rupture.

As noted above, the 5th Ave reservoir is mapped in very close proximity to Reservoir Canyon Fault. If that fault were to rupture, there would potentially be feet of displacement that could occur both horizontally and vertically. The existing reservoir could not withstand that level of movement and would be severely damaged. If you want to strengthen the existing reservoir for both ground shaking and surface fault rupture, a much more intensive structural strengthening would be required. Even without design drawings to review, our structural team believes you would have to replace the foundation with a rigid mat foundation and also consider strengthening the walls. There may also be other options to consider, like placement of geofoam or some sort of compressible foam behind the walls to reduce pressures and allow some movement without damage to the walls.

The City should assume the currently mapped fault location unless an exploratory excavation proves otherwise.

In light of these facts, we recommend that the best path forward is to abandon the existing structure and replace it with a 5 MG prestressed concrete reservoir designed to withstand the likelihood of significant ground movement in this location. A new Fifth Avenue Pump Station, adjacent to the new reservoir at grade, should be designed to lift water from the Fifth Avenue Reservoir directly up to Sunset Reservoir. A proposed location of the new reservoir and is shown in the figure above.

It should be noted that any modifications to any of the reservoirs will trigger the requirement to bring the reservoir up to all codes per the California Water Resources Control Board Department of Drinking Water. The plans would need to be submitted to the Water Board for approval and permitting.

6. **Improve Margarita or replace storage elsewhere.** The 2.4 MG rectangular concrete reservoir is in poor condition and requires immediate attention to correct structural and seismic deficiencies. The roof and walls do not meet current seismic codes. The roof will likely collapse in a major seismic event. The only practical upgrade solution is to construct a "tank within a tank" by first removing the roof and then constructing new walls, footing, and floor. In this solution, the wall height can be increased to provide more storage in this footprint if desired. The new roof can either be concrete or an aluminum dome. Alternatively, pending the results of the reservoir consolidation study, it is highly likely we can eliminate this reservoir by constructing more storage at a higher hydraulic gradient (Sunset). The reservoir could then be demolished and the property sold for residential use. It is recommended that any action regarding Margarita be postponed until the consolidation study is complete and accepted by the City and other stakeholders.

7. **Lower operating levels in steel tanks and replace lost storage elsewhere.** The City has 12 steel tanks distributed throughout the system. To better understand the seismic resilience of these steel tanks, a series of calculations was performed based on AWWA D100-11 standards to check for three primary failure mechanisms within the steel tank structure: (1) adequate freeboard and sloshing, (2) seismic overturning, and (3) allowable stress development in the shell. Although all of the tanks satisfied static standards, they failed when subjected to design level earthquake standards.

   The tanks without seismic anchorage along their perimeters usually failed in both seismic overturning and stress development in the shell. Additionally, the tank shells were not constructed with sufficient freeboard to contain the calculated sloshing. Retrofitting is not required if the HWL is lowered to the recommended values. The 12 steel tanks collectively provide 20.4 MG of storage volume in their current operating mode. Lowering the elevations as recommended will result in the loss of more than 6 MG of storage. This supports the idea of consolidation, as this storage can be replaced in properly designed prestressed concrete tanks – the best seismic designed tank in the industry today – at the highest elevation possible, which makes the Sunset site the ideal location. The majority of the City's steel tanks are also very old; Texas Street dates to 1956 and only three have been erected since 1973, which makes all of the steel tanks close to 50 years old or older. The industry standard for the useful operating life of steel tanks is 50 years, so we recommend that no further investments be made in this infrastructure.

The City's recently updated hydraulic model was utilized to study the consequences of lowering the maximum high water elevations in the City's steel tanks. The primary concern is potential impacts on meeting pressure and fire code requirements. Each pressure zone was analyzed. The modeling efforts revealed deficiencies in individual tanks, but the deficiencies were offset by excess storage in other reservoirs serving the same pressure zone. For example, the Smiley Heights reservoir shows a deficiency of 3.05 million gallons, but fortunately this reservoir is in the same pressure zone with the Dearborn and Highland Reservoirs, the City's largest reservoirs at 10.6 MG and 10 MG, respectively. Sunset Reservoir also show a deficiency of 1.04 million gallons, but this deficiency will be resolved when the new Sunset Reservoir is constructed. The only two tanks needing immediate attention is Mill Creek 1 and 2. Proposed upgrades were described above as project 2(b).

8. **Replace Texas St. Reservoir in the existing footprint or abandon and replace storage elsewhere**. The reservoir is in poor condition. The reservoir could possibly be replaced in the existing reservoir footprint at a size no less than 1 MG and as large as 2.4 MG to make up for the storage deficiency at Texas Grove, due to the need to operate at a lower height to meet seismic codes. In the short term to meet current seismic codes, the reservoir high water elevation needs to be reduced by 38% (13.4 feet), resulting in a revised operating volume of 619,000 gallons. This is a significant loss of storage. Lowering the operating depth is necessary to increase freeboard for sloshing and to decrease the extreme steel shell stress under seismic conditions. It is therefore recommended that the reservoir not be operated at full capacity. The possibility for overturning, sliding, and water loss being released are high in a seismic event, if operated at full capacity. Given the size and age of this tank, retroactively equipping the tank with seismic anchorage is not recommended. Replacement or abandonment is recommended. The final decision about Texas Street should be postponed until the reservoir consolidation study is complete and accepted by the City and other stakeholders. If the reservoir is eventually abandoned, the pump station would need to be relocated to the Texas Grove Reservoir site.

9. **Maintain Dearborn and Country Club reservoirs.** The City's concrete reservoirs are in much better condition than the steel tanks. The 10 MG Dearborn Reservoir (photo at right), a rectangular concrete design, is in fair condition and requires no immediate attention or action to correct any structural or seismic deficiencies, especially since the completion of structural and seismic improvements in 2014. At that time, the original seventy-seven (77) columns were structurally enhanced and a new interior footing was placed around the entire reservoir interior, which protects the roof from collapse (unlike Highland Reservoir).

The other concrete reservoirs that appear to be code compliant without modifications in operating elevations are Country Club 1 and 2. Country Club 1 Reservoir is 20 feet high, with a diameter of 102.42 feet. The reservoir is buried with an exposed roof. The reservoir was built as a conventionally reinforced concrete tank in 1924 and is the City's oldest reservoir. An aluminum roof was added in 1980 and the tank was lined with steel plates in 2010, so technically the reservoir can now be considered a steel tank. The new steel tank rests within the old concrete structure and, overall, is in fairly good condition. However, the steel wall and old concrete wall are not structurally connected. It appears that the steel tank improvement was implemented strictly to resolve a leakage problem with the old concrete reservoir and is effectively a liner. The roof was also replaced for the second time. Due to the improvements in the reservoir that have been implemented in the past 40 years, and because the reservoir walls are completely below ground, the reservoir is in fair condition and does not require immediate attention or action. The reservoir passes all seismic requirements outlined by AWWA D100-11. Country Club 2 Reservoir dates back to 1969; it is in fair condition and does not require any maintenance at this time. Given the age and size of this reservoir, no further investment should be made at this site.

Both Country Club 1 and 2 should be maintained in service to provide additional water security to the City. The reservoirs are very high in the City's hydraulic profile and pump water up to the Sunset Reservoir area. Beyond the water treatment plants, Sunset will become the City's most valuable water asset; having at least two ways to get water up the “hill” is critical. Improvements to the Country Club pump station should be made as a necessary improvement to assure the reliability of this facility. A new permanent emergency generator is in the City’s current plans, so no further action is needed at the County Club site. Overall, the City’s concrete reservoirs have done the job they were designed to do and have long useful lives ahead.

10. **Pump Stations.** The pump stations located at the Texas Street, Ward Way, Dearborn, South, Sand Canyon, Highland, 5th Avenue, Smiley Heights and Country Club reservoirs were evaluated by a team of experts in the mechanical, electrical, civil and structural engineering disciplines. Visual and prominent operational conditions were observed, and additional structural assessments (with supporting calculations) were provided when warranted. Consideration was given to each structure with respect to fire hardening, and the presence of hazardous materials onsite was investigated and documented in the August 2021 Limited Asbestos and Lead Survey prepared by Group Delta Consultants, Inc. The only pump station needing immediate attention is at Ward Way, where new pumps have been recently installed, but the building above the pumps needs to be replaced. The photo below is from a recent pump station project that BRADY designed for the City of Fountain Valley. The proposed pump station building at the Ward Way site should look something like the structure shown in the photo. This pump station house four vertical turbine pumps and a 500 KW diesel generator.

Project Costs

Opinions of Probable Construction Cost (OPCC) are summarized in Table 1 on the following page. The detailed estimating backup for each capital project is included in a separate technical memorandum but is summarized in this Executive Summary. OPCC's are summarized in the following separate categories:

1. Base cost of the capital project based on historical cost information for similar projects, supplemented where necessary with RS Means historical cost information.
2. Variable costs such as contractor general conditions costs (10%), and overhead and profit costs (15%), as a percentage of the OPCC.
3. Engineering and administrative cost as a percentage of the OPCC (soft costs).
4. Class 5 estimating contingency (+30%).

Recommended Capital Improvement Projects

Name	Recommended Improvement	Construction Cost (1)	Soft Costs (2) 25%	Recommended Project Budget	Comments
Sunset Reservoir	New 15 MG Reservoir	$21,761,910	$5,440,477	$27,202,387	AWWA D110, Type 1 design
Mill Creek Pipeline	Construct a new 30-inch parallel pipeline under Mill Creek	Cost of be developed by the consultant selected by the City to prepare the predesign and design documents for this project			RFP has been issued for predesign and design, cost TBD
Mill Creek Steel Tanks	Raise reservoir walls to restore volume to original design	$1,000,000			Will require a detailed structural evaluation and pricing before moving forward
Agate Reservoir	Construct a new 60-inch pipeline to achieve CT in a pipe; remove hypalon curtains	$3,726,854	$931,714	$4,658,568	Parallel existing 30-inch from Hinckley to Agate site, retain existing reservoir at reduced operating level
Highland Reservoir	Construct grade beam around reservoir exterior perimeter	$1,257,067	$314,267	$1,571,333	
Fifth Avenue Reservoir	New 5 MG Reservoir (min.)	$9,484,401	$2,371,100	$11,855,502	AWWA D110, Type 1 design
Fifth Avenue Pump Station	New pump station to lift water to Sunset	$9,867,988	$2,466,997	$12,334,986	Above ground, 4 vertical turbine pumps, emergency generator
Texas Street	Replace with 1.5 MG or abandon	Decision on Texas Street should be deferred until a future reservoir consolidation study is complete			AWWA D110, Type 1 design
Ward Way Pump Station	Erect new building over existing pumps	$3,106,952	$776,738	$3,883,690	Masonry block building, concrete roof
Note 1	Cost includes no project contingency but +30% for budgeting purposes				
Note 2	Soft costs include non-City project management, design, and construction phase services at 25%				

BRADY
Water • Power • Transportation

Respectfully submitted,

Richard Brady

Richard Brady, P.E., BCEE

CEO, Richard Brady & Associates

Project Manager and Engineer of Record

Additional background information is included in the following sections.

Section 1 – Historical Background of the City's Water Assets

Section 2 – Seismic Background

Section 3 – Recommended Capital Improvement Program

Section 4 – Pump Station Assessments

Section 5 – Prestressed Concrete Reservoir Design

Appendix A – City Response to the Draft Executive Summary dated February 27, 2022

Appendix B – BRADY Response to City's Draft Executive Summary Comments dated May 3, 2022

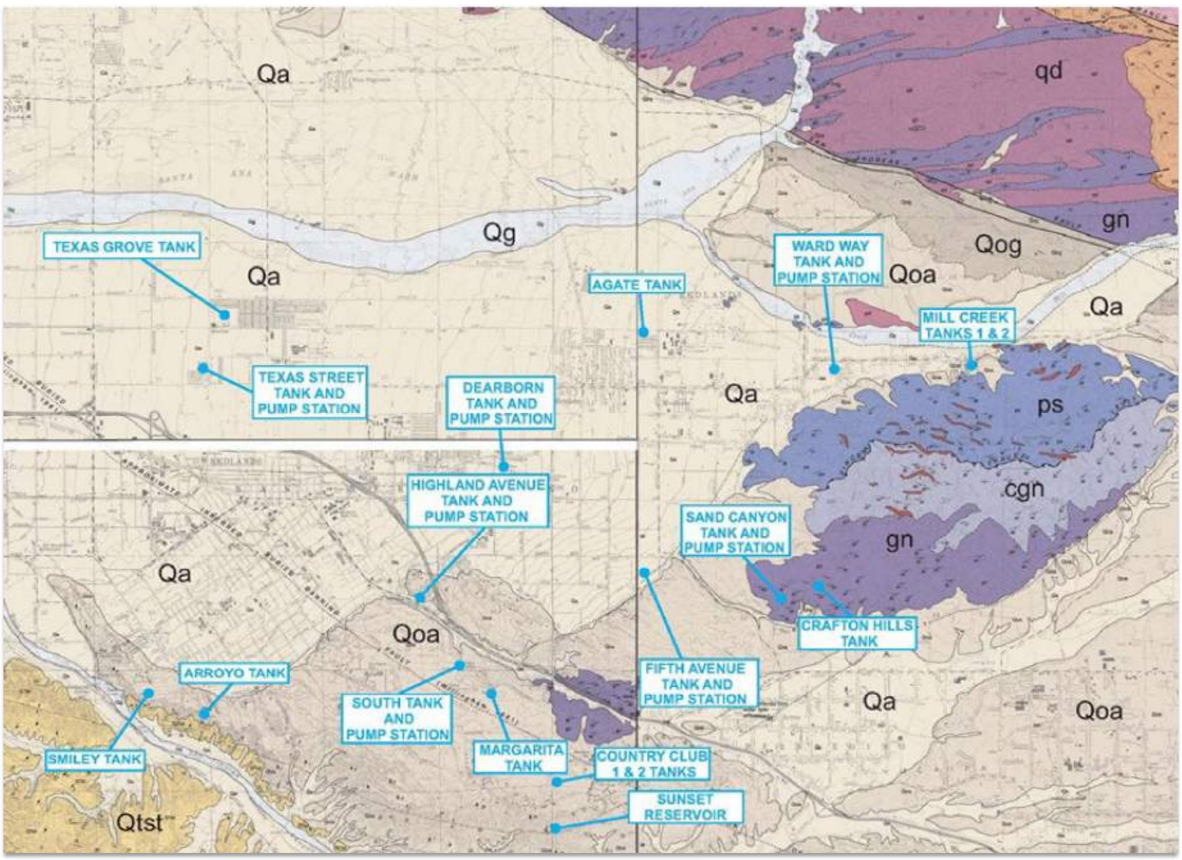
Appendix C – Steel Tank Capacity Analysis

Appendix D – Summary Table

SECTION 1 – HISTORICAL BACKGROUND OF THE CITY'S WATER ASSETS

The City's water assets evaluated in BRADY's work are summarized in the table below and figures are provided on the following pages.

Name	Type	Year	Size
Country Club 1	Concrete	1924	1
Country Club 2	Concrete	1969	2
Dearborn	Concrete	1972	10.6
Fifth Avenue	Concrete	1974	5
Highland	Concrete	1976	10
Margarita	Concrete	1964	2.4
Agate Reservoir	Steel	1968	3
Arroyo Reservoir	Steel	1965	0.5
Crafton Hills	Steel	1970	1
Mill Creek 1	Steel	2005	0.2
Mill Creek 2	Steel	2005	0.2
Sand Canyon	Steel	1973	3.5
Smiley	Steel	1964	3
South	Steel	1964	2
Sunset	Steel	1967	3
Texas Grove	Steel	2004	4
Texas Street	Steel	1956	1
Ward Way	Steel	1958	2
Hinckley Pipeline	ML&CSP*	1987	36 inches
Mill Creek Pipeline	Steel	1968	30 inches
Dearborn Pump Station			2000 gpm
Fifth Avenue Pump Station			3000 gpm
Highland Pump Station			4500 gpm
Sand Canyon Pump Station			1500 gpm
South Pump Station			4000 gpm
Texas Grove Pump Station			5800 gpm
Ward Way Pump Station			150 gpm
* Mortar lined and coated steel pipe			


TEXAS GROVE TANK
TEXAS STREET TANK AND PUMP STATION
DEARBORN TANK AND PUMP STATION
AGATE TANK
WARD WAY TANK AND PUMP STATION
MILL CREEK TANKS 1 & 2
HIGHLAND AVENUE TANK AND PUMP STATION
SAND CANYON TANK AND PUMP STATION
CRAFTON HILLS TANK
FIFTH AVENUE TANK AND PUMP STATION
ARROYO TANK
SMILEY TANK
SOUTH TANK AND PUMP STATION
MARGARITA TANK
COUNTRY CLUB 1 & 2 TANKS
SUNSET RESERVOIR
Qa
Qg
qd
gn
Qog
Qoa
ps
cgn
Qtst

Brief Assessment Summary

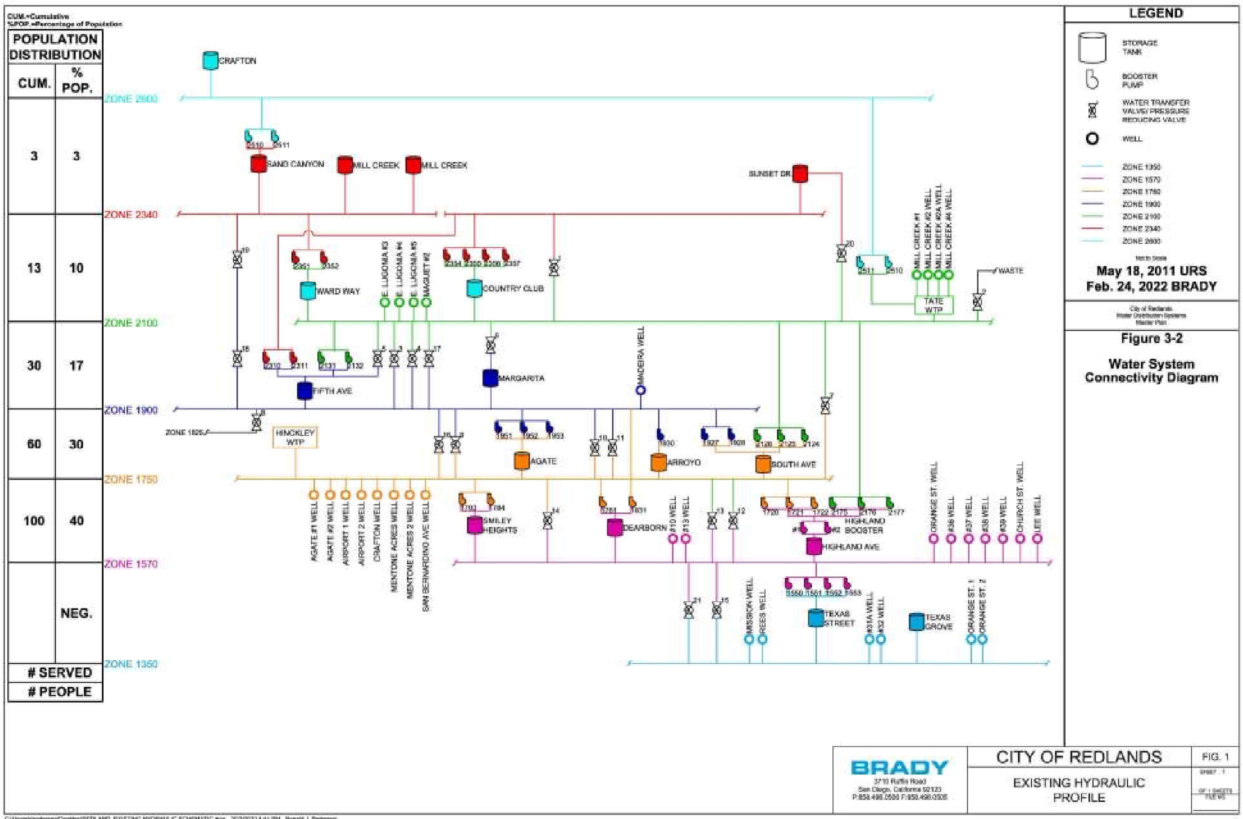
As both the table and graph below illustrate, the City's reservoirs range in age from 1924 to 2005. The majority of the City's reservoirs were constructed before the 1971 San Fernando earthquake. Therefore, it is not a surprise that only a few of the City's 18 reservoirs meet current seismic codes. However, the City is very fortunate that only one reservoir – the 5 million gallon (MG) Fifth Avenue Reservoir -- is in close proximity to an active fault that requires special care and attention, as discussed in detail herein.

Existing Hydraulic Profile

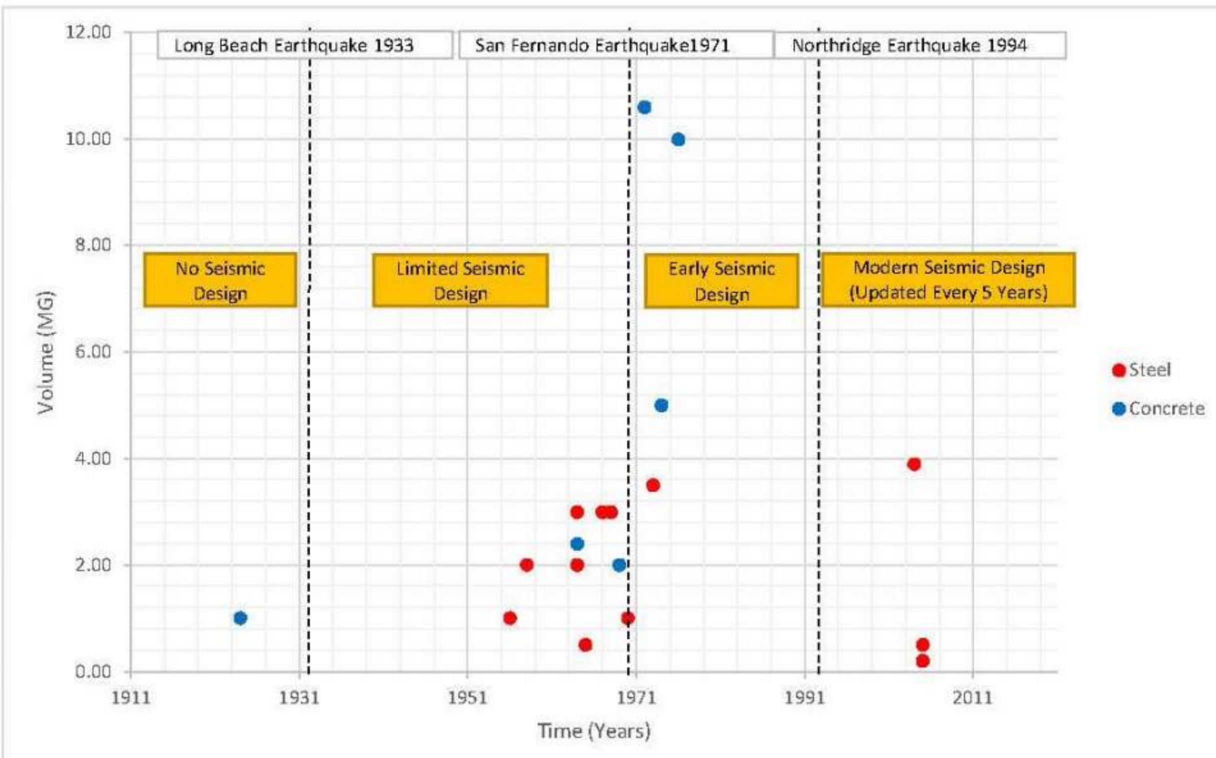
The hydraulic profile of the City's entire treatment, pumping, storage, and distribution system is graphically illustrated on the Hydraulic Profile image below. This figure was obtained from the City's last Water Facilities Master Plan (Master Plan) from 2011, a report that was not finalized. At the moment, the City has contracted with Michael Baker International to update the Master Plan. As of finalizing our efforts under our contract, the Master Plan has not yet been accepted by the City. The Master Plan should include an updated Hydraulic Profile as well as recommendations regarding the amount of storage the City should have for current needs and for at least 10 years into the future. We may revise some of our recommendations once we have an opportunity to review the recommendations of the Master Plan.

In our efforts, we spent considerable time trying to understand the hydraulic profile. As noted in the Title Block, this is truly a "Water System Connectivity Diagram" where all of the City's water assets can be review in one image. The City's system is very well interconnected with numerous ways to move water around the City in the event of disruption in service due to a major seismic event. As we studied the Hydraulic Profile and asked questions of City Operations Staff over the past year, a few discrepancies were discovered that are now reflected in our updated diagram (February 24, 2022) shown in the image below, and described herein:

1. The Sand Canyon and Sunset Reservoir both serve the highest zone in the City (2340), but are not hydraulically interconnected. Both tanks are steel with no second tank as backup. This the reason why a temporary tank is needed at the Sunset site, to allow the existing tank to be taken off-line while a new permanent reservoir is constructed on the adjacent property.
2. The Mill Creek steel tanks are not connected to Sunset as well. These are small steel tanks that are primarily used as a supplemental filter backwash source at the Tate WTP.
3. The majority of City storage is consolidated in the 1750 and 1570 pressure zones, serving 40 percent of the City's population by gravity. The City's largest reservoirs are Dearborn (10.6 MG) and Highland (10 MG), that sit at the bottom of the hydraulic profile.
4. The City has 18 reservoirs spread throughout the distribution system. Compared to other similarly size cities (population 75,000), 18 reservoirs are on the high end, with many older small steel tanks. The opportunity for consolidation of storage is promising, by aggregating more storage at the Sunset and Fifth Avenue sites.

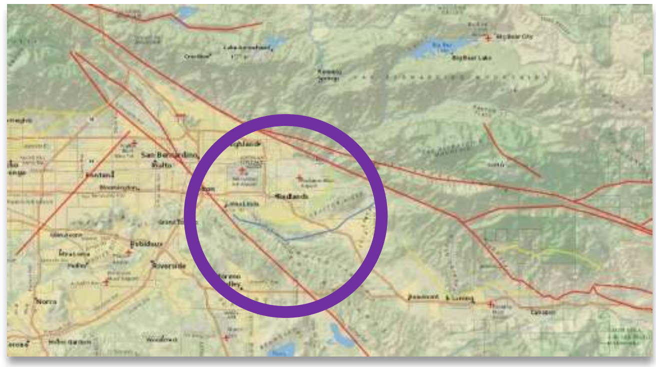

CUM.=Cumulative
%POP.=Percentage of Population
POPULATION DISTRIBUTION
CUM.
% POP.
3
3
13
10
30
17
60
30
100
40
NEG.
SERVED
PEOPLE
ZONE 2600
ZONE 2340
ZONE 2100
ZONE 1900
ZONE 1825
ZONE 1750
ZONE 1570
ZONE 1350
CRAFTON
SAND CANYON
MILL CREEK
MILL CREEK
SUNSET DR.
WARD WAY
COUNTRY CLUB
E. LUGONIA #3
E. LUGONIA #4
E. LUGONIA #5
MAGUET #2
MILL CREEK #1
MILL CREEK #2 WELL
MILL CREEK #2A WELL
MILL CREEK #4 WELL
TATE WTP
WASTE
FIFTH AVE
MARGARITA
MADEIRA WELL
HINCKLEY WTP
AGATE
ARROYO
SOUTH AVE
AGATE #1 WELL
AGATE #2 WELL
AIRPORT 1 WELL
AIRPORT 2 WELL
CRAFTON WELL
MENTONE ACRES WELL
MENTONE ACRES 2 WELL
SAN BERNARDINO AVE WELL
SMILEY HEIGHTS
DEARBORN
HIGHLAND BOOSTER
HIGHLAND AVE
ORANGE ST. WELL
#36 WELL
#37 WELL
#38 WELL
#39 WELL
CHURCH ST. WELL
LEE WELL
MISSION WELL
REES WELL
TEXAS STREET
#31A WELL
#32 WELL
TEXAS GROVE
ORANGE ST. 1
ORANGE ST. 2
LEGEND
STORAGE TANK
BOOSTER PUMP
WATER TRANSFER VALVE/ PRESSURE REDUCING VALVE
WELL
ZONE 1350
ZONE 1570
ZONE 1750
ZONE 1900
ZONE 2100
ZONE 2340
ZONE 2600
May 18, 2011 URS
Feb. 24, 2022 BRADY
Figure 3-2
Water System Connectivity Diagram
BRADY
3710 Ruffin Road
San Diego, California 92123
CITY OF REDLANDS
EXISTING HYDRAULIC PROFILE
FIG. 1

Steel			Concrete		
Tank Name	Year Built	Volume (MG)	Tank Name	Year Built	Volume (MG)
Agate Reservoir	1968	3.00	Country Club 1	1924	1.00
Arroyo Reservoir	1965	0.50	Country Club 2	1969	2.00
Crafton Hills	1970	1.00	Dearborn	1972	10.60
Mill Creek 1	2005	0.20	Fifth Avenue	1974	5.00
Mill Creek 2	2005	0.50	Highland	1976	10.00
Sand Canyon	1973	3.50	Margarita	1964	2.40
Smiley	1964	3.00			
South Avenue	1964	2.00			
Sunset	1967	3.00			
Texas Grove	2004	3.90			
Texas Street	1956	1.00			
Ward Way	1958	2.00			



Section 2 – Seismic Background

<u>The Inconvenient Truth – Significant Seismic Risk in Redlands is Real</u>



The City of Redlands is located in an area with very high seismic activity. Two of California's most highly active fault zones, the internationally known San Andreas Fault Zone and the San Jacinto Fault Zone, flank the City of Redlands. These two fault zones consist of multiple fault segments and branches that have the potential for larger earthquakes when they rupture in combination. A regional fault map is shown in the figure below, followed by additional information concerning these faults.

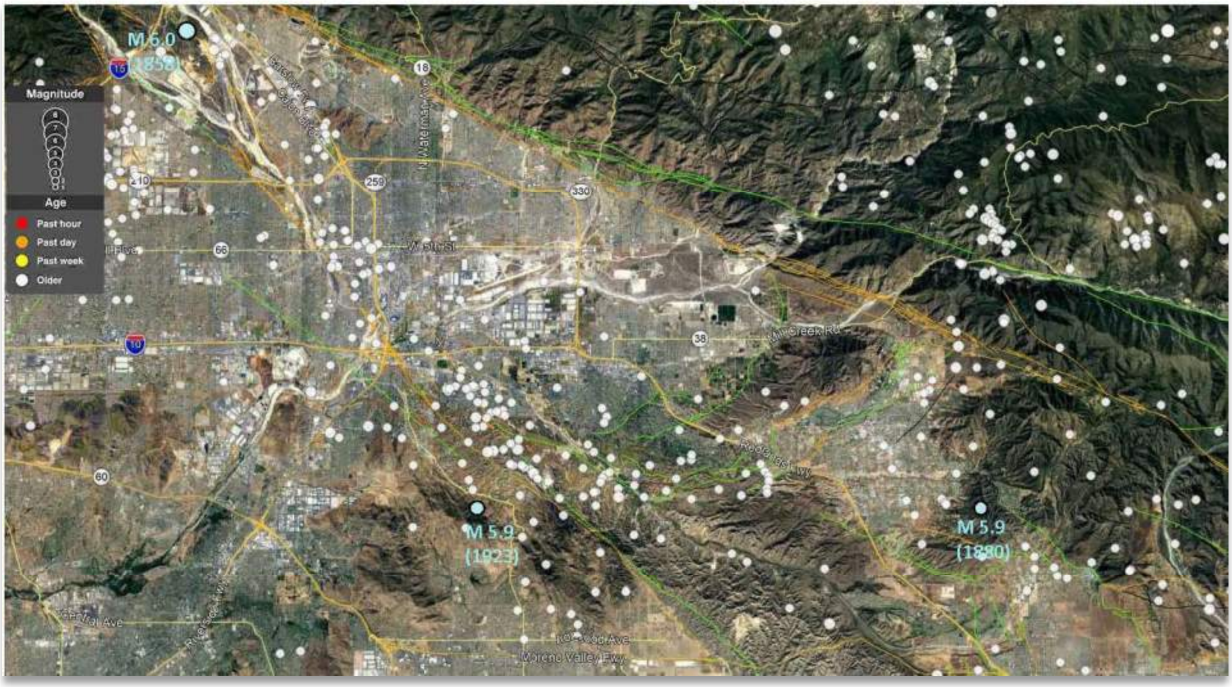
San Andreas Fault

The San Andreas Fault Zone is a right-lateral strike slip fault system that extends a total length of 315 miles through California. This fault system forms the boundary between the Pacific Plate and the North American Plate. The Southern San Andreas section of the fault system extends from Parkfield down to its termination at the Salton Sea. The Southern San Andreas section is estimated to be capable of producing earthquakes with a maximum magnitude (MW) of 8.2. In the area of Redlands and Yucaipa, the structure of the San Andreas fault becomes very complex where it has interacted with other faults over the millennia, resulting in fractured segments and discontinuous branches. Recurrence intervals between ground-rupturing earthquakes vary on the San Andreas fault system depending on location. Near Redlands this interval is estimated to be 175 to 200 years (USGS, 2017).

San Jacinto Fault

The San Jacinto Fault Zone is a right-lateral strike slip fault with a total length of about 125 miles, extending from San Bernardino down to Superstition Mountain. At the northern end of the fault, it connects with the San Andreas fault Zone. It is estimated to be capable of producing earthquakes with a maximum magnitude (MW) of 7.8, when all of the segments rupture in combination from San Bernardino to Superstition Mountain. The San Jacinto fault has a typical recurrence interval for ground-rupturing earthquakes of 100 to 300 years. Given the proximity of the sites to the San Jacinto and San Andreas fault systems, there have been a number of moderate to large earthquakes that have occurred close to the City of Redlands. A historical earthquake search was performed using the Advanced National Seismic System (ANSS) Comprehensive Earthquake Catalog (ComCat) (USGS, 2021). This search included earthquakes with magnitudes 5.0+ with epicentral distances of 100 miles of the center of the project sites in Redlands (approximately at the location of Dearborn Reservoir).

The earthquakes with epicenters closest to the City of Redlands water facilities are the 1923 magnitude 6.0 Greater Los Angeles Area earthquake south of Redlands, the 1880 magnitude 5.9 earthquake near Yucaipa, and the 1858 magnitude 6.0 earthquake near San Bernardino, northwest of Redlands. An interesting figure of historical seismic activity in the region is included below. Each white dot represents an historical seismic event.


M 6.0
(1858)
Magnitude
Age
Past hour
Past day
Past week
Older
M 5.9
(1923)
M 5.9
(1880)

Section 3 – Recommended Capital Improvement Program

Recommended program to improve the seismic resiliency of this system, in order of priority:

1. **Sunset Reservoir.** The Sunset Reservoir dates to 1967. It is in extremely poor condition

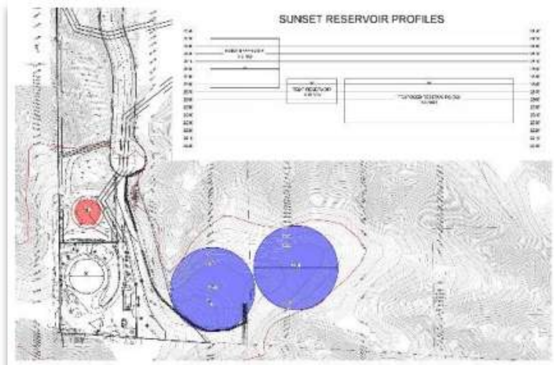
with no seismic restraint and a lead paint coated interior. The soil below the reservoir is fractured granite and potentially unstable. A new reservoir in this location would require the top ten feet of soil to be removed. Large Verizon cell towers have been installed adjacent to the reservoir. In a major seismic event, the Sunset Reservoir is in danger of

either tipping over into the canyon below, or in the other direction, into the cell towers that would affect cell phone communications, causing critical problem s if the towers were damaged. For these reasons, the reservoir should be relocated to a more suitable, safer location.

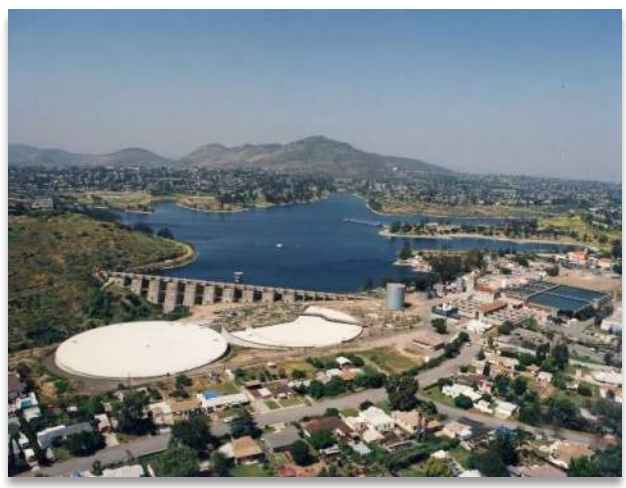
The recommendation is broken into two separate phases.

Given the lack of seismic anchorage and likely overturning in a major seismic event, the first phase is to erect a temporary 750,000-gallon bolted steel tank on the parcel north of existing reservoir. The City can purchase this temporary tank, which can be disassembled, stored, and re-used in the future where needed in the system, possibly as a reclaimed water tank in the Crafton Hills college area. This action is necessary because the Sunset Reservoir has no back-up reservoir to service the 2340 service Zone in this area. The Sand Canyon Reservoir also services the 2340 Zone, but it not hydraulically connected to the Sunset service area; they operate independently of one another.

We have studied an option to construct a new partially buried prestressed concrete reservoir in an adjacent parcel that the City may purchase assuming a successful negotiation is concluded. If the land purchase is not successful, then the recommendation is to construct a new 3 MG prestressed concrete reservoir on the existing tank site.



If the negotiation is successful, the second phase is to construct a new partially buried prestressed 10 MG concrete reservoir in the City's parcel adjacent to the existing Sunset site. The property has 4 separate but adjoining parcels, 22 acres in total. The new reservoir can comfortably be constructed in this 22-acre parcel, with room for a possible second 10 MG reservoir at some future date. These are the largest reservoirs that can be constructed at this site at the right hydraulic elevation. The existing Sunset Reservoir is only 3 MG but the additional 7 MG at this location will account for the 6 MG of lost storage in the City due to reduced high-water levels (HWL) to meet current seismic codes. The second phase is to construct a new partially buried prestressed concrete reservoir in the City's newly acquired parcel adjacent to the existing Sunset site. Sunset Reservoir is one of the City's most valuable assets as it sits at the highest elevation in the City, allowing gravity flow to 97% of the City's population. The recommended size is 10 million gallons (MG). A proposed layout is illustrated in the image to the right. The layout shows a potential second reservoir if needed to meet water demands at some future date. These two reservoirs are illustrated in blue; the temporary 750,00-gallon tank is illustrated in red.



The proposed layout would be similar to a 1994 BRADY design for two new 21 MG prestressed concrete reservoirs at the Alvarado WTP in the City of San Diego. The adjacent photo of the completed project will help visualize what this could look like. The reservoir on the right includes a Native Plant Demonstration Garden on the reservoir for public education purposes of native, drought tolerant landscaping. Similar joint public use can be considered for the new Sunset Reservoir site, which provides an impressive 360-degree view of the area.

2. **(a) Mill Creek Pipeline.** The poor condition of this pipe presents a significant concern about the extended operating life of this asset. The pipe is elevated above Mill Creek on concrete piers spaced approximately 40 feet apart. Numerous large boulders span the length and width of Mill Creek. The chance of failure of this pipe during a major seismic event is significant, as well as during a major storm event where large boulders could damage the piers. Historical boulder impacts to the existing support piers are visually evident.

To improve overall reliability of this critical asset, several pipe replacement alternatives will be considered. A second pipeline is only one of these alternatives. The Mill Creek Pipeline delivers raw water for treatment at Tate WTP. This pipeline is critically important to the City's water security because the Tate WTP can serve more than 85% of the City's entire population by gravity flow. Maintaining continuous operations of the Tate WTP is therefore essential in a future major seismic event. A possible solution is to construct a second 30-inch diameter inlet pipeline parallel to the existing pipeline under Mill Creek. The pipe should parallel the existing pipe in Mill Creek Road all the way to the inlet structure at the Tate WTP. A dedicated inlet structure should be constructed to receive this new pipeline, and then connect to the existing inlet box for distribution to the reactor clarifiers. The new pipeline should also have a dedicated flow meter located at the plant site. A concept sketch is included below. The City has issued a Request for Proposals (RFP) for the design of this pipeline. Proposals are due February 29, 2022. The consultant selected to predesign and design this pipeline will also be tasked with developing a project budget. Therefore, our cost table below will not include a price for the new pipeline given this cost will be developed by others at some future date.

Additionally, once the new pipeline is in operation, the existing piers should be reinforced as necessary to provide additional security against catastrophic failure. Flexible expansion joints should be provided on each side of the creek where the pipe comes out of and re-enters the ground.

(b) Mill Creek 1 and 2 Tanks. The Mill Creek 1 and 2 tanks were erected in 2005 at 200,000 gallons each. The tanks are redundant to each other, so a minimum of 200,000 gallons of storage is needed at all times to provide the desired benefit – supplemental backwash supply to the Tate WTP.

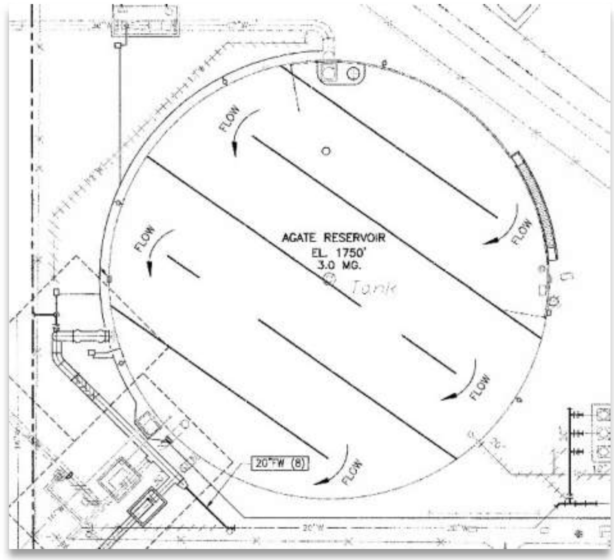
Structural calculations to bring Mill Creek 1 and 2 into code compliance would require a significant reduction in operating volumes. Each tank would see a reduction from the design volume of 200,000 gallons to 90,420 gallons, to 45% of the original volume. The Mill Creek 1 and 2 tanks were not provided with seismic anchorage, are in danger of overturning in a seismic event, and lack sufficient freeboard to protect the tanks from damage due to sloshing.

Given the importance of the Tate WTP to the City in a major seismic event, and due to the critical importance of maintaining the assets needed to allow filter backwashing under low plant production rates, the Mill Creek 1 and 2 tanks are in need of rehabilitation. The easiest approach is to build a third Mill Creek tank, but space at the site is very limited. Though it is not a proven industry solution, serious consideration should be given to raising the reservoir roofs as needed to restore the operating volumes of each tank to 200,00 gallons. Seismic anchorage is also required. A budget figure of $500,000 is recommended to resolve this problem as quickly as possible.

3. **Agate Reservoir**. The Agate Reservoir dates to 1968 and serves as the clearwell reservoir for the Hinckley Water Treatment Plant (WTP). The reservoir does not meet current seismic

design codes, but can be brought into compliance by reducing the operating capacity to 2,163,000 gallons which will decrease stresses in the steel shell and increase the freeboard to allow for sloshing. However, the operating volume of the Agate Reservoir is critically important as the City achieves regulatory CT compliance (concentration x time) inside of Agate Reservoir. The reservoir was retro-fitted in 2010. Hanging hypalon curtains were secured to the reservoir roof and floor to create a serpentine flow pattern inside reservoir to promote improved mixing for the purposes of obtaining the regulatory mandated CT credit. The City uses free chlorine as their primary and secondary disinfection method. The strength of a chemical disinfectant (chlorine) for inactivating pathogens when in contact with water can be measured by its CT value. CT values are used to evaluate the inactivation of pathogens by disinfection using a logarithmic scale, thus it is referred to as "log inactivation."

Hypalon curtains shown anchored to the



Log inactivation is simply the order of magnitude in which inactivation of unwanted organisms occurs and relates to the percentage of organisms inactivated. The Hypalon curtains are shown in the figure at the right. There are many unintended consequences that have resulted from the installation of the baffle curtains. In a major seismic event, the sloshing forces of the water will act on the curtains, which in turn will collapse the roof, leading to complete reservoir failure. The curtains may also become dislodged and plug the outlet pipe serving gravity flow supply to the City's largest reservoirs – Dearborn at 10.6 MG and Highland at 10.0 MG. The possibility of this occurring will result in losing the Hinckley WTP supply as all of the plant effluent flows through Agate Reservoir. Hinckley provides gravity flow to 40% of the City's population.

We recommend no further financial investments be made at the Agate Reservoir. A new 60-inch diameter pipe parallel to the existing 30-inch plant effluent pipeline will allow the City to achieve CT compliance in piping, not a baffled circular reservoir where the flow circulation pattern is not ideal. Ideal situations are not present in a water storage tank. Depending on the degree of short -circuiting, baffling factors can be anywhere from the ideal pipe plug flow of 1, to less than 0.1 for a poorly designed system. Higher baffling factors mean less short circuiting, a higher CT, and a better disinfection outcome. Recent CT tracer studies indicate that baffles provide a baffling factor of 0.68, but come with the risk of reservoir failure.

An extract from the USEPA Manual of Disinfection Profiling and Benchmarking (1999) is provided below and gives baffling factors for various types of tanks. As noted above, these examples show that a pipe is the ideal system and an un-baffled tank is the worst.

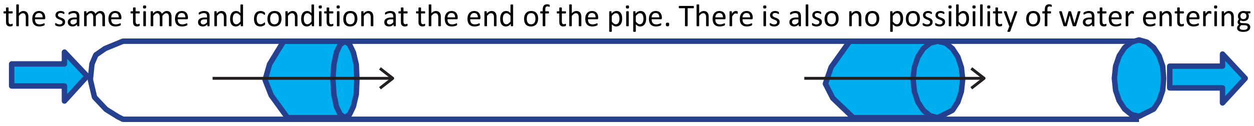
Baffling Condition	Baffling Factor	Description
Unbaffled (mixed flow)	0.1	None, agitated basin, very low length to width ratio, high inlet and outlet flow velocities. Can be approximately achieved in flask mix tank.
Poor	0.3	Single or multiple inlets and outlets, no intra-basin baffles
Average	0.5	Baffled inlet or outlet with some intra-basin baffles
Superior	0.7	Perforated inlet baffle, serpentine or perforated intra-basin baffles, outlet weir or perforated launders
Perfect (plug flow)	1	Very high length to width ratio (pipeline flow) (greater than 40:1), perforated inlet, outlet, and intra basin baffles

Unbaffling CT

Disinfection of water with chlorine requires time for the chemical to react with and kill the target microbial pathogens. Ideally, the time for the reaction is provided in a purpose-built reactor or contact tank specifically designed for this purpose and which provides a controllable process. However, the reality of how to implement an ideal solution is quite different. Most water supplies rely on disinfection to occur in tanks and reservoirs of various sizes, with various arrangements of inlet and outlet structures and varying levels of short-circuiting. The effective contact time is often less than is assumed.

An Idea that is Too Good to Ignore – Obtain CT in a Pipe

When water flows in a pipe at a constant flowrate, there is very little mixing along the length of the pipe, such that a volume of water entering the pipe at one end will exit the pipe at virtually the same time and condition at the end of the pipe. There is also no possibility of water entering



Plug flow in a pipe. An ideal situation for disinfection.

the pipe to short-circuit to the outlet any faster than the rest of the water. This 'no mixing and no short-circuiting" option is the **ideal** situation and is called **plug flow**.

As all the water passes down the pipe at the same time, the time that the first 10% leaves the pipe is very similar to the time that all the water leaves the pipe. **This means the baffling factor is 1:** the ideal contact tank.

Flow Rate (MGD) =	14.5
Flow Rate (ft^3/sec) =	26.94
Temperature (deg C) =	13.6
pH =	8.4
Residual Chlorine (mg/L) =	1
Baffling Factor =	1
Log Removal =	0.5
Contact Time (min*mg/L) =	24.2

Actual Detention Time (min) =	24.2
Theoretical Detention Time (min) =	24.2
Volume (ft^3) =	32,532
Gallons =	243,347

Pipe Diameter (in) =	60
Pipe Area (ft^2) =	19.6
Length of Pipe (ft) =	**1657**

Since the new 60-inch pipe will parallel the existing 30-inch pipe, both pipes will contribute to achieving CT compliance. The equivalent diameter of a 60-inch and 30-inch pipe is 64.04 inches. Using this larger equivalent pipe diameter, the new 60-inch minimum length is 1,454 feet. More than adequate room is available to achieve this outcome. The City will have a long-term solution where CT is always achieved, 100% of the time without a contribution from the seismically challenged Agate Reservoir. Once the new pipe is installed, the hypalon curtains can be removed to protect the reservoir and the system so the existing Agate Reservoir can remain in service at a reduced operating level. The 60-inch pipe will also provide a supplemental storage volume of 0.22 MG. The estimated construction cost of the new proposed 60-inch pipeline is approximately $3,725,000. This estimate includes a 15% estimating contingency. The City can therefore avoid the $5M plus cost to replace Agate with a new tank.

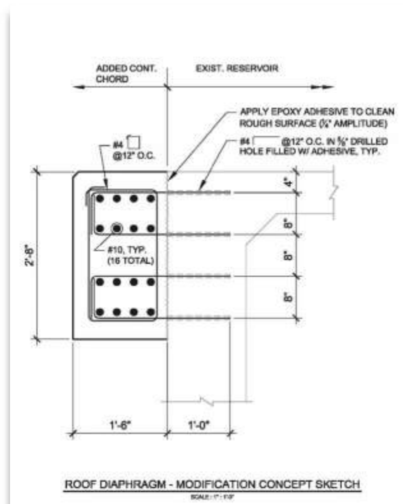
Proposed Layout

Fortunately, there is adequate space to construct a new 60-inch pipeline parallel to the existing 30-inch pipe to achieve CT between the filter effluent control structure at the Hinckley WTP and the meter located on the east side of the Agate Reservoir. The proposed layout is shown in the figure to the right.

The existing meter can be retained. A second effluent weir will be needed downstream of the meter to provide submergence for flow measuring accuracy. Two pipes will exit the weir structure and connect to existing piping for delivery to the system and the 10.6 MG Dearborn and 10 MG Highland Reservoirs down gradient.

4. Highland Reservoir. The rectangular concrete reservoir is in fair condition, but does require immediate action to correct a minor seismic deficiency. The Highland design has some unique structural features due to the presence of recreational activities on the reservoir roof. As a result of anticipating the associated live loads from these activities, the structure generally meets current seismic design codes.

The problem to be solved relates to the roof diaphragm. The presence of slip dowels in the existing roof design (although designed to allow for thermal expansion/contraction), do not transfer seismic loads from roof diaphragm to walls. In a (horizontal) seismic event, the roof slab could collapse. A structural beam (reinforced with 16- #10 bars) around the entire perimeter will contain and transfer diaphragm loads to walls per the detail shown at the right.



5. Fifth Avenue Reservoir. The only reservoir at risk is the 5 MG Fifth Avenue Reservoir. This reservoir is a key storage reservoir for the Tate WTP as water transfers into the lower system elevations. Water stored in Fifth Avenue is pumped to the 2340 Zone (Sand Canyon Reservoirs).

The loss of Fifth Avenue for any reason would restrict operations. The site of the Fifth Avenue reservoir and pump station is located within a County Fault Zone for the Reservoir Canyon fault, which was adopted in the Countywide Policy Map in October 2020. As shown in the figures at the left and below, the edge of the County Fault Zone passes through the middle of the Fifth Avenue property. The Reservoir Canyon Fault is a normal fault that is part of the Crafton Hills Fault Zone. The fault trace is mapped about 400 feet away from the closest location to the Fifth Avenue reservoir. Therefore, there may be potential for surface rupture, or associated off-fault deformations at the site from movement of the Reservoir Canyon fault reaching the ground surface at the Fifth Avenue site. The County of San Bernardino Development Code, Chapter 82.15.040 (b) requires that a structure used for critical facilities (such as a reservoir) be located 150 feet or farther from any active earthquake fault trace. Additionally, the San Bernardino County-Wide Plan (County of San Bernardino, 2020) for mitigation of natural environmental hazards states in Policy HZ-6:

"We require **new** critical and essential facilities to be located outside of hazard areas, whenever feasible."

Per Table S-2 of the County of San Bernardino General Plan, Land Use Compatibility Chart in Fault Hazard Zones, essential land uses are "restricted" in fault hazard Zones, which is defined as:

> "Restricted unless alternative sites are not available or feasible and it is demonstrated through a site investigation that, although mitigation may be difficult, hazards will be adequately mitigated."

However, the restricted use and requirements for earthquake fault Zones have generally applied to new structures or any existing structures where the alterations/additions to the structure are greater than 50 percent of the value of the structure. Since the Fifth Avenue Reservoir was constructed in 1974, and design drawings were not located in the City's records, we cannot confidently prepare design improvements to the existing structure that would be needed to satisfy the likely very restrictive design criteria.

Fifth Avenue is the only City reservoir that is near an active fault. As required by codes, new structures or improvements at the Fifth Avenue site will require a site-specific surface fault rupture investigation performed in accordance with *California Geological Survey (CGS) Note 49, Guidelines for Evaluating the Hazard of Surface Fault Rupture*. We recommend excavating the fault trench location shown on the figure to the right. The trench excavation will be approximately 3 feet wide and up to 10 feet deep.

Fortunately, the reservoir walls are completely buried with a column supported exposed concrete roof. Major structure damage that may occur in an earthquake will fortunately not result in a tidal wave of water causing damage to other structures. However, the loss of Fifth Avenue will cause an operational nightmare for City staff. Replacing this reservoir with a prestressed concrete reservoir designed to AWWA D110, Type 1 (cast in place concrete walls) provides the City with the best protection in a future seismic event. Fortunately, there is available space on the existing site for a new reservoir. Though the guidance suggests we find an alternate site for a new reservoir positioned a safe distance away from the Reservoir Canyon Fault, there do not appear to be any other suitable sites available that meet the hydraulic reality. Fifth Avenue was likely designed by James M. Montgomery (JMM), the dominant reservoir design firm of this era, and the shape and burial fit the JMM standard design in the 1970s. Dearborn and Highland also have completely buried walls and exposed concrete roofs and were designed by JMM. They were all constructed in the 1970s within a four-year span. (Dearborn 1972, Fifth Avenue 1974, and Highland 1976). Fifth Avenue sits in the ground because of the way it is designed, not because of hydraulics.

The Tate WTP serves the 2100 Zone, supplies water to Fifth Avenue by gravity to serve the 1900 Zone below Fifth Avenue. The existing reservoir is below ground, not for hydraulic reasons, but required by the type of design in 1974. Non-circular reservoir shapes are rarely constructed at grade. With the significant static head available (200 feet) the new reservoir can sit at grade, 30-40 feet higher than the existing HWL elevation in Fifth Avenue. The hydraulic model confirmed this change is possible.

The geophysical survey conducted at the site indicates that the upper 3 to 8 feet at the site are more disturbed and/or less dense (likely fill). Below 10 feet, the shear wave velocities increase with depth, indicating the soils increase in stiffness and become denser, with shear wave velocities ranging from about 900 ft/s to about 1,400 ft/s. Based on the seismic velocity measurements obtained, the resulting average VS,30 is 1,156 ft/s (352 m/s).

A Brief Clarification of Risk at the Fifth Avenue Reservoir Site

It should be noted that throughout the City's system, compliance with current seismic codes is clearly the goal, however, compliance with current code does not guarantee that a reservoir constructed before 1994 (Northridge earthquake, the last regional earthquake of significance) would not be seriously damaged in an earthquake. For example, most of the City's 12 steel tanks can be made code compliant by simply lowering the operating levels. Various recommendations to bring structures "up to code" only addresses the hazard of strong ground shaking. In no way does it mitigate the hazard of surface fault rupture.

It should be noted that any modifications to any of the reservoirs will trigger the requirement to bring the reservoir up to all codes per the California Water Resources Control Board Department of Drinking Water. The plans would need to be submitted to the Water Board for approval and permitting.

The 5th Ave reservoir is mapped in very close proximity to Reservoir Canyon Fault. If that fault were to rupture, there would potentially be feet of displacement that could occur both horizontally and vertically. The existing reservoir could not withstand that level of movement, it would be severely damaged. If you want to strengthen the existing reservoir for both ground shaking and surface fault rupture, a much more intensive structural strengthening would be required. Even without design drawings to review, our structural team believes you would have to replace the foundation with a rigid mat foundation, and also consider strengthening the walls. There may also be other options to consider, like placement of geofoam or some sort of compressible foam behind the walls, to reduce pressures and allow some movement without damage to the walls.

For some additional background - there has been some research on the topic of designing or strengthening structures to cross active faults, and observations from prior earthquakes. These studies suggest that continuous, rigid foundations perform better when spanning over a fault. There have been a couple of documented case histories from Turkey to support this, as well as numerical modeling studies. Also, UC Berkeley performed a seismic retrofit of Bowles Hall in 2008 and a portion of the building was found to span across the Hayward fault. They performed studies to identify the fault, estimate fault displacement, and then they designed the area with a new mat foundation and flexible retaining walls that could deform as part of the retrofit. At this stage of our dialogue we suggest the City considering locating the fault as soon as possible; at this moment we need to assume it is right there where it is mapped.

In light of these facts, we recommend that the best path forward is to abandon the existing structure and replace it with a 5-million-gallon prestressed concrete reservoir designed to withstand the likelihood of significant ground movement in this location. A new Fifth Avenue Pump Station, adjacent to the new reservoir at grade, should be designed to lift water from the Fifth Avenue Reservoir directly up to Sunset Reservoir. A proposed location of the new reservoir and new pipeline is shown on the figures above and below.

6. Margarita Reservoir. The rectangular concrete reservoir is in poor condition and requires immediate attention to correct structural or seismic deficiencies. The roof and walls do not meet current seismic codes. The roof will likely collapse in a major seismic event. The walls are deficient for combined hydrostatic and seismic loading. The only practical upgrade solution is to construct a "tank within a tank" by first removing the roof and then constructing new walls, footing, and floor. In this solution, the wall height can be increased to provide more storage in this footprint if desired. The new roof can either be concrete or an aluminum dome. Alternatively, pending the results of the reservoir consolidation study, it is highly likely we can eliminate this reservoir by constructing more storage at a higher hydraulic gradient (Sunset). The reservoir could then be demolished and the property sold for residential use. It is recommended that any action regarding Margarita be postponed until the consolidate study is complete and accepted by the City and other stakeholders.

7. Steel Tanks.

A Brief History of Southern California Steel Tanks Designs – Why does Redlands have so many?

Over the past century, the decision on what type of tank to design has depended primarily on the entity that is paying the initial costs. Steel was an easy early choice because most tanks built for developing cities were small, typically 1 MG or less, and had low initial capital costs. As cities continued to grow, larger tank volumes were needed that made steel less practical for sizes of 3 MG or greater. This is likely the City's history as well.

Steel was scarce during World War II; concrete was the only practical choice. Advances in the understanding of how concrete behaves in hydraulic structures led to tank size increases up to

40 MG. The explosive growth in California after World War II led to considerable urban growth. Cities and agencies began tasking residential developers with the responsibility – and costs – of developing water infrastructure. Developers typically choose the lowest cost options, thus resulting in hundreds of small steel tanks being constructed from San Diego to Santa Barbara.

Once in service, operations and maintenance responsibilities and expenses were transferred to the end utility. This is primarily why the City eventually has so many small steel tanks, with the associated high maintenance cost (corrosion control, painting). Twelve of the City's 18 storage tanks are steel, representing approximate 66% of the total storage, however after the HWL's are lowered, they will provide only 31 % (14.1 MG) of the City's total storage volume of 45.1 MG. The City's six concrete reservoirs provide 31 MG of storage.

Seismic Assessment of the City's 12 Steel Tanks

Design of the City's welded steel tanks likely followed American Water Works Association's Standard D100 – Welded Carbon Steel Tanks for Water Storage, which has a history dating back to 1941. Steel tank evaluations are fairly straightforward compared to reinforced concrete (RC) because the entire tank exterior is visible as well as the inside when drained; only the underside of the floor is hidden from view. Typical problems in older tanks erected prior to 1978 are the possible presence of lead paint and coal tar enamel coatings on the interior. Corrosion of the steel over the decades is another obvious major concern. As we have determined from our structural calculations, very few of the City's steel tanks meet current seismic design codes.

In order to better understand the seismic resilience of the City's steel tanks, a series of calculations was performed based on AWWA D100-11 standards. These calculations are used to check for three primary failure mechanisms within the steel tank structure: (1) adequate freeboard and sloshing, (2) seismic overturning, and (3) allowable stress development in the shell. A table representing these findings in both the seismic and static case is provided on the following page. Although all of the tanks satisfied static standards, they failed when subjected to design level earthquake standards. The tanks without seismic anchorage along their perimeters usually failed in both seismic overturning and stress development in the shell. Additionally, the tank shells were not constructed with sufficient freeboard to contain the calculated sloshing. If the high-water lines (HWL) of the tanks cannot be lowered sufficiently, then seismic retrofitting would be required (i.e., mechanical anchorage, increased shell thickness, or increased shell height). Retrofitting is not required if the HWL is lowered to the recommended values provided in the table below.

Tank	Date Constructed	Overall Grade	Diameter	Height of Shell	Height to Diameter Ratio
Agate Reservoir	1968	C	143'-0"	25'-6"	0.18
Arroyo	1965	D	46'-6"	40'-6"	0.87
Crafton Hills	1970	C	76'-0"	30'-6"	0.40
Mill Creek E	2005	B	47'-0"	16'-0"	0.34
Mill Creek W	2005	B	47'-0"	16'-0"	0.34
Sand Canyon	1973	C	122'-0"	40'-0"	0.33
Smiley	1964	C	127'-0"	32'-6"	0.26
South	1964	C	111'-0"	26'-6"	0.24
Texas Grove	2004	B	180'-0"	25'-0"	0.14
Texas Street	1956	D	70'-0"	35'-3"	0.50
Ward Way	1958	D	104'-0"	32'-0"	0.31

The recommended reductions in operating levels at each of the City's steel tanks, including the justification for the reduction, is summarized in the table on the following page.

Tank	Current Operating Volume	Operating Volume to Meet Code	Operational Capacity %	Reduction Justification
Agate Reservoir	3,000,000	2,163,000	72%	Extreme Stresses Developed in 1st and 2nd layer plates when subjected to seismic loading
Arroyo	500,000	194,400	39%	Seismic anchorage required to resist overturning and not enough freeboard
Crafton Hills	1,000,000	780,500	78%	Extreme stresses in multiple layers of tank shell under seismic loading and insufficient freeboard.
Mill Creek E	200,000	90,420	45%	Seismic anchorage required to resist overturning and not enough freeboard
Mill Creek W	200,000	90,420	45%	Seismic anchorage required to resist overturning and not enough freeboard
Sand Canyon	3,500,000	2,658,000	76%	Extreme stresses in multiple layers of tank shell in both seismic and static case, freeboard was also not sufficient
Smiley	3,000,000	2,037,000	68%	Extreme stress in first layer of tank shell in seismic case, freeboard was also not sufficient
South	2,000,000	1,412,000	71%	Freeboard not sufficient
Texas Grove	4,000,000	2,760,000	69%	Extreme stresses in multiple layers of tank shell in both seismic and static case, freeboard was also not sufficient
Texas Street	1,000,000	642,000	64%	Seismic anchorage required to resist overturning and not enough freeboard
Ward Way	2,000,000	1,322,000	66%	Freeboard not sufficient
Total:	20,400,000	14,149,740	69%	Entire system will be reduced 30% in order to meet seismic standards

*Seismic reductions based on AWWA D100-11 and do not consider hydraulic or fire requirements

Hydraulic and fire flow demands were not considered in this structural analysis; lowering the tank operating level may cause the tanks to be deficient in meeting those requirements. As noted in the table, the steel tanks provide 20.4 MG of storage volume in their current operating mode. Lowering the elevations as recommend will result in the loss of more than 6 MG of storage. This supports the idea of consolidation, as this storage can be replaced in properly designed prestressed concrete tanks – the best seismic designed tank in the industry today – at the highest elevation possible, which makes the Sunset site the ideal location. The steel tanks could be seismically retrofitted with mechanical anchorage; however, this would only alleviate the issues in overturning and would not satisfy the freeboard requirements. The steel tank shells could have a dual retrofit by raising the tank shells and anchoring the systems. This process would require the excavation of the existing foundations to be replaced by deeper foundations and the tank being structurally modified, which is very cost prohibitive and not recommended. The majority of the City's tanks are also very old, Texas Street dates to 1956 and only three have been erected since 1973, which makes all of the steel tanks close to 50 years old or older. The industry standard for steel tanks is 50 years, so we recommend no further investments be made in this infrastructure.

8. Texas Street Steel Tank. The reservoir is in poor condition. The reservoir could possibly be replaced in the existing reservoir footprint at a size not less than 1 MG and as large as 2.4 MG to make up for the storage deficiency at Texas Grove due to need to operate at a lower height to meet seismic codes. In the short term to meet current seismic codes, the reservoir high water elevation needs to be reduced by 38% (13.4 feet), resulting in a revised operating volume of 619,000 gallons. This is a significant loss of storage. Lowering the operating depth is necessary to increase freeboard for sloshing and to decrease the extreme steel shell stress under seismic conditions. It is therefore recommended that the reservoir not be operated at full capacity. The possibility for overturning, sliding, and water loss being released are high in a seismic event if operated at full capacity. Given the size and age of this tank, retroactively equipping the tank with seismic anchorage is not recommended. Replacement or abandonment is recommended. The final decision about Texas Street should be postponed until the reservoir consolidation study is complete and accepted by the City and other stakeholders. If the reservoir is eventually abandoned, the pump station would need to be relocated to the Texas Grove Reservoir site.

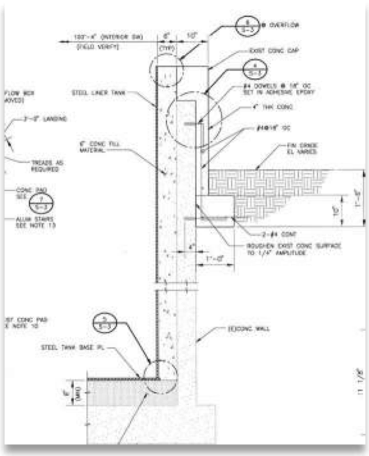
In the table below, an overall grade has been established to provide an easy means for understanding the condition of all City reservoirs. The tanks noted with a B Grade were relatively recently constructed, but still require a reduction in operating high water levels. However, due to the age of these tanks, they are in decent shape, recently painted, and in the case of Texas

Grove, seismic anchors were installed. The Mill Creek E and W tanks, with an age of less than 20 years, are, unfortunately, seismically deficient. This is important as they provide supplemental backwash storage supply for the self-backwashing filters at the Tate WTP. Sufficient storage for this purpose is still available, but both tanks must be added together to make this work. Even though there are two tanks, there is no backup tank now with the reduced volumes.

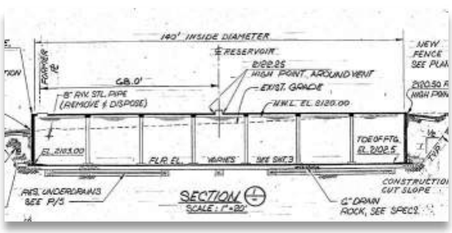
8. The Good News Regarding the City's Concrete Reservoirs. The City's concrete reservoirs are in much better condition than the steel tanks. The 10 MG Dearborn Reservoir is a rectangular concrete design, is in fair condition and requires no immediate attention or action to correct any structural or seismic deficiencies. Dearborn also has completely buried walls and exposed concrete roofs and were designed by JMM, the dominant reservoir designer of that era. Structural and seismic improvements were also implemented in 2014. The design was very conservative. The original seventy-seven (77) columns were structurally enhanced and a new interior footing was placed around the entire reservoir interior. Unlike Highland, the roof is now protected from collapse with the new interior footing. The original 14-inch square columns were enlarged to 30-inch square columns. The columns were accessed by cutting 54-inch square holes in the roof. Forms were placed around the existing columns and concrete was placed from the top. The holes in the roof were patched with concrete. The reservoir circulation design could be improved. Short-circuiting is likely occurring, as there are no barriers to stop the inlet water from flowing directly to the outlet sump. All other water outside of this "river" is not moving and will become stagnant over time. This lack of baffling, has resulted in a significant dead Zone above the inlet pipe, a layer of water that is 13.75 feet deep (centerline elevation 1564.25 feet to the high-water elevation 1578.00 feet).

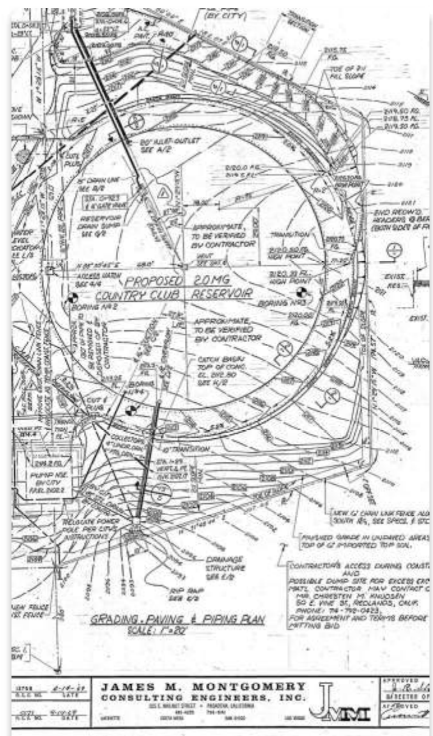
Inserting baffles is not recommended. It is unnecessarily costly and will require anchorage to the existing columns, a seismic loading condition that is not recommended. A simple reconfiguration of the inlet pipeline to include better flow distribution can be considered. A Computational Flow Dynamics (CFD) Model was developed to examine potential inlet pipe improvements. At a minimum, a simple raising of the inlet pipe discharge elevation to force a "top to bottom" flow pattern should be considered.

The other concrete reservoirs that appear to be safe are Country Club 1 and 2. Country Club 1 Reservoir is the City's oldest reservoir dating back to 1924. The reservoir is 20 feet high, with a diameter of 102.42 feet. The reservoir is buried with an exposed roof. The reservoir was built as a conventionally reinforced concrete tank in 1924. An aluminum roof was added in 1980 and the tank was lined with steel plates in 2010, so technically the



reservoir can now be considered a steel tank. The new steel tank rests within the old concrete structure and overall is in fairly good condition. However, the steel wall and old concrete wall are not structurally connected. It appears the steel tank improvement was implemented strictly to resolve a leakage problem with the old concrete reservoir and is technically a liner. The roof was also replaced for the second time. Due to the improvements in the reservoir that have been in the past 40, and because the reservoir walls are completely below ground, the reservoir is in fair condition and does not require immediate attention or action. The reservoir passes all seismic requirements outlined by AWWA D100-11. The interior of the tank had no major notable issues other than sediment build up on the tank floor.





Country Club 2 Reservoir dates back to 1969. The reservoir is a conventionally reinforced concrete structure, 16.25 feet high, with a diameter of 140 feet. The reservoir is buried with an exposed concrete roof. The reservoir was designed by James M. Montgomery (JMM). JMM was the dominant design firm in Southern California at the time of this design (1974) and the characteristics of this reservoir follows the JMM standard concrete reservoir design. The existing concrete reservoir is in fair condition and does not require any immediate maintenance at this time. Given the age and size of this reservoir, no further investment should be made at this site. Both Country 1 and 2 should be maintained in service to provide additional water security to the City. The reservoirs are very high in the City's hydraulic profile and pump water up to the Sunset Reservoir area. Beyond the water treatment plants, Sunset will become the City's most valuable water asset and having at least two ways to get water up the "hill" is critical. The other Sunset supply source will be the new reservoir and pump station at the Fifth Avenue Reservoir site. Improvements to the Country Club pump station should be made a necessary to assure the reliability of this facility. A new permanent emergency generator is in the City's current plans, so no further action is needed at the County Club site.

Overall, the City's concrete reservoirs have done the job they were designed to do, and have long useful lives ahead.

Section 4 – Pump Station Assessments

The pump stations were evaluated by a team of experts in the mechanical, electrical, civil and structural engineering disciplines. Visual and prominent operational conditions were observed and additional structural assessments (with supporting calculations) were provided when warranted. Consideration was given to each structure with respect to fire hardening, which typically resulted in the recommendation to clear adjacent landscaping to minimize any fire risk. Lack of emergency power was not identified; it was assumed that portable emergency generators are utilized as needed. The presence of hazardous materials onsite was investigated and documented in the August *2021 Limited Asbestos and Lead Survey* prepared by Group Delta Consultants, Inc.

It should be noted that because of the importance of the Country Club and Highland Reservoirs (serving as a "control hub for many other components"); the March 2011 draft *Master Plan* prepared by URS recommended particular "focus on the operations of these two most complex reservoir systems and their associated PRV and booster systems".

Following is a summary of our findings and recommendations.

1. Texas St. Pump Station. Station consists of four exposed electric driven vertical turbine pumps (circa 1960's) that are generally in good condition with a capacity to deliver 7800 gpm from Zone 1350 to Zone 1570. The March 2011 draft *Master Plan* prepared by URS recommends (pg. 92) "further examining" the idea of upsizing all four existing pumps.

The reinforced CMU structure houses electrical controls/equipment and does not meet current seismic code; additional reinforcement is required. Replacement of the main 5kV distribution switchgear, general housekeeping within the structure and minor mechanical/electrical modifications are recommended; as well as minor site improvements including paving, security, lighting and maintenance to minimize fire risk.

The pump station shares a large site with the Texas Reservoir, a potable water well and treatment equipment and an extensive filter treatment chain which is no longer in full-time service but occupies most of the site. The plant is still used to supplement non-potable water storage needs. To discourage vandalism and related problems as recently experienced, removal of equipment/facilities no longer in use is recommended. Additionally, re-evaluating and possibly re-purposing the entire site should be considered, potentially relocating/upsizing pumping equipment to the Texas Grove Reservoir site.

2. Ward Way Pump Station. Station consists of two electric driven pumps contained within a very small metal structure, with a capacity to deliver 310 gpm from Zone 2100 to Zone 2340. Although pumps appear to have been very recently replaced, the structure is severely undersized and in a state of disrepair; with an inefficient piping layout and limited accessibility for equipment maintenance/installation/removal. The roof, walls and floor are in a state of disrepair with deteriorating soundproofing the likely cause of neighbors' noise complaints. A structural assessment was not performed because the recommendation is to replace the entire structure with a larger, quieter, code complaint structure that can accommodate equipment and provide access for operations and maintenance.

Site paving improvements, lighting and security are recommended, as well as the replacement of the electrical equipment and instrumentation.

3. Dearborn Pump Station. Station consists of two exposed electric driven vertical turbine pumps with a capacity to deliver 2060gpm from Zone 1570 to Zones 1750 and 1900, a piping/valve vault, exposed electrical cabinets, and a small CMU structure that houses electrical controls/equipment.

Pumps appeared to be in satisfactory condition, with some leakage observed. No major concerns were noted. The construction of asphalt or concrete paving is recommended adjacent to the pumps and electrical equipment to protect equipment and facilitate maintenance. Site lighting and security improvements are also suggested, as well as modifications to piping appurtenances and electrical equipment for enhanced operator convenience.

Chrysotile ACM, Crocidolite AMC, lead, chromium and zinc were detected on site. The Chrysotile ACM and Crocidolite AMC were found on a piece of transite pipe that was at the site (debris on the ground) near the pump station shed. The chromium, lead and zinc were found on paint chip samples.

South, Sand Canyon, Highland and 5th Avenue Pump Stations

The pumping stations at the South, Sand Canyon, Highland and 5th Avenue reservoirs share a common piping layout and equipment design, contained within an enclosed steel moment frame structure. These structures are generally in good condition and can withstand gravity loading, but **additional lateral structural support is required to meet seismic code**. This is true for each of the four pump stations; a "benefit-cost" decision is necessary to determine if these structural improvements are worth the capital cost. It may be concluded that failure of the structure is acceptable following a seismic event, as long as the station continues pumping water.

Equipment at each of these four stations is in varying stages of replacement need, with some parts nearly obsolete. When appropriate, it is recommended that they be replaced with newer (preferably off the shelf) versions for inventory availability, flexibility and uniformity.

The structures do not typically contain combustible materials, nor are there openings for errant flying embers, however routine clearing of nearby dry vegetation will reduce fire risk. Lead, chromium and zinc were found at all locations. Chrysotile ACM was also detected at each of these sites with the exception of the Highland facility, where no evidence of asbestos was reported. The Chrysotile ACM and Crocidolite AMC were found on a piece of transite pipe that was at the site (debris on the ground) near the pump station shed. The chromium, lead and zinc were found on paint chip samples.

Additional characteristics and conclusions unique to each of these four stations are identified below.

4. South Pump Station. Station consists of five pumps (circa 1962-1968); three contained within a steel moment frame structure (circa 1964). These four-horizontal split-case and one vertical turbine pump have the capacity to deliver 5956 gpm from Zone 1750. Pumps P-1927 and P-1928 deliver to Zone 1900. Pumps P-2124, P-2125, and P-2126 deliver to Zone 2100. The March 2011 draft *Master Plan* prepared by URS states that, "Additional pumps are recommended for P-1927".

Upgrading the vertical turbine applications with premium efficient motors and heavy-duty bearings is recommended to support the significant investment of VFD controllers. Replacement of the pump starter panels for P-1927 and P-2126, is also recommended, as well as reconstruction of the failing retaining walls. Paving, security and other site improvements, as well as replacement/upgrade of miscellaneous equipment and parts are also suggested.

5. 5th Avenue Pump Station. Two horizontal split-case pumps installed in late 1969 are contained within a steel moment frame structure; two vertical turbine pumps with a barrel to accommodate a third vertical turbine pump assembly are located outside the structure. Record drawings show natural gas engine conversions to electric motors in late 1997.

The pump station, pumps and motors generally appear to be in good condition with only minor improvements suggested, however, paving improvements for the access road are recommended for improved access, particularly in an emergency situation. Interior piping appears to be in good condition with no signs of deterioration, corrosion or leakage; however significant condensation near meter was observed which might veil leakage around connections. Exterior piping shows signs of above grade spalling, potentially indicating a more serious problem below grade. Recommend improving access to valves in below grade vault for operator convenience.

6. Sand Canyon Pump Station. Station consists of two electric driven horizontal split-case pumps (circa 1972) with a capacity to deliver 3783gpm from Zone 2340 to Zone 2600.

We recommend immediate replacement of pump starter panels, additional lighting, and site paving/drainage improvements to better control offsite runoff and extend life of pavement. Minor equipment/parts replacement as well as minor mechanical/electrical modifications are also recommended to enhance operator convenience.

7. Highland Pump Station. Station consists of six electric driven horizontal split-case pumps (circa 1961-62) and two booster pumps (circa 1976) and a steel moment frame structure constructed in 1966 with a combined capacity to deliver 6600gpm from Zone 1570 to Zones 1750 and Zone 2100.

Minor maintenance and painting are recommended, as well as regular housekeeping/monitoring of miscellaneous stockpiled items to avoid interference with pumping operations. Minor site paving and drainage improvements are suggested, and minor mechanical/electrical modifications to enhance operator convenience. Extending the vertical turbine booster pump can and motor base to above grade (leaving discharge at existing elevation) will offer better protection if vault floods.

OTHER

The pumping operations at the Country Club and Smiley Heights reservoirs were also visited, although all pumping equipment is exposed and there are no structures located onsite. General observations and recommendations are provided below.

8. Country Club Pump Station. The pumping equipment at the Country Club Reservoir site consists of three electrically driven submersible pumps and one vertical turbine pump that appear to be in good working order and together have the capacity to deliver 3427gpm from Zone 2100 to Zone 2340. There also exists an abandoned underground pump station on the south side of the site, between the two reservoirs. Piping and appurtenances appear to be in good working order, with no leakage or evidence of malfunction observed. Minor recommendations regarding site drainage, lighting and security are suggested.

The existing electrical system is generally in good condition, including a 600V, 800A, 3-phase, 60HZ manual/automatic transfer switch with an external generator connection capable of connecting a 600kW portable generator; enough capacity to fully support the complete site electrical system in an emergency/stand-by power situation.

Site not tested for asbestos; lead, chromium and zinc were detected on site.

9. Smiley Heights Pump Station. The two exposed electric driven submersible pumps have the capacity to deliver 720gpm from Zone 1570 to Zone 1750 and are generally in good condition, requiring no immediate attention. Minor recommendations regarding security, lighting, landscaping and electrical and piping modifications to enhance operator convenience are suggested.

Site not tested for asbestos; lead, chromium and zinc were detected on site.

Section 5 – Prestressed Concrete Reservoir Design

Closing Words – Why is BRADY's Support for Prestressed Concrete Reservoir Design So Strong?

BRADY's corporate knowledge regarding reservoir design preferences has been shaped by the many decades key members of our design team have spent in the planning, design, and construction of reservoirs of all shapes, sizes, and materials. BRADY's structural team also includes Max Dykmans, P.E., S.E., the former President and Owner of DYK, prior to their merger with Natgun in 2011 to form what is known as DN Tanks today. A summary of this experience is included in the table below:

Name	Years of Experience	Comments
Richard Brady, P.E., BCEE	41	Design experience with all reservoir types, shapes, and size; largest tank, conventional rectangular concrete, 52 MG (Sydney, Australia)
Karl Kuebitz, P.E.	22	Former Design Manager for DYK/DN Tanks; world's largest prestressed concrete reservoirs, 4 @ 42 MG
Lee Biggers, P.E., S.E.	55	Mr. Biggers was inducted as a Fellow of the Structural Engineers Association of California in 2010; Structural Engineer of Record for more than 50 circular prestressed and conventional concrete reservoirs
Max Dykmans, P.E., S.E.	46	Former Owner and President of DYK; builder of more than 3,000 prestressed concrete reservoirs over 40 years

The professional experience of Mr. Dykmans and Mr. Kuebitz is primarily as designers of prestressed concrete reservoirs designed to AWWA D110, Type 1 spanning back to the 1970's. Mr. Brady and Mr. Biggers have experience in all reservoir types, shapes, and sizes ranging from 630,000 gallons to 52 MG. At one time Mr. Brady was the designer of the largest prestressed reservoir in California when he was the designer of 2-21 MG reservoirs at the City of San Diego's Alvarado WTP in 1994. Mr. Brady eclipsed his California record and the entire world, when his 35 MG Earl Thomas Reservoir began construction in 2001 for the City of San Diego (photo at the right). All three reservoirs are shown in the picture at the right.

Our company's Engineer Manager, Karl Kuebitz, P.E. was also the leader of our structural efforts on this project. Mr. Kuebitz, P.E. has been involved in the site development, design and construction of hundreds of Type 1 prestressed concrete tanks over 20-years, including the largest in the World at 42 Million Gallons (MG). And not just one, but four at this size.

There is no doubt we favor the prestressed design, for the reasons discussed herein., Below is a short history of what we have collectively learned after all of these years for the benefit of the City of Redlands.

<u>What Have We Learned as a Group After all These Years?</u>

1. Circular shapes are best for storing water. Very easy to design, all forces are radial and circumferential.

2. Avoid odd shapes (trapezoids) where structural design forces are difficult to analyze. A catastrophic failure of a 30 MG conventionally reinforced tank in the initial hydrostatic test in 1994 was a true "lesson learned" for the water industry (see photo, right). It didn't take long to figure out this was a design error. (Not a BRADY project.)

3. In areas of seismic concern, the premier reservoir design choice is prestressed concrete designed to AWWA D110, Type 1.

4. Avoid attempts to make something cheaper. A prestressed design concept developed in the 1960's, known in the industry as the "Pritzker System", became notorious because of a 5 MG reservoir failure in 1998. Pritzker Tanks were a proprietary system that enjoyed moderate success in the 1960's, building around two dozen tanks in the United States, most of which in Southern California. The Pritzker System relied on 8'-0" wide by 2'-8" deep "U" shaped precast panels. The "U" shape varied from 4 to 8 inches in thickness. Although relatively thin, the "U" shape was to

provide the required stiffness to span from floor to roof. The precast panels were stacked next to each other to form a circle and then an internal tendon ring beam was cast at the top and bottom of the wall. There was no continuity or reinforcing between any of the panels, instead the hoop force was entirely taken at the top and bottom of the wall in the ring beam. The 5.0 MG Westminster Reservoir that failed on September 21, 1998, at 5:45 a.m. was a Pritzker design (see photo, above). The tank failed quickly through a single wall panel, releasing its entire contents in a matter of minutes, resulting in significant property damage and several minor injuries. Fortunately, there were no fatalities. It is believed that a poor detail used in the bottom ring beam was the cause of the failure.

5. Minimize construction joints to reduce the amount of waterstop needed. Most reservoirs that experience excessive leakage is often the result of a folded waterstop, not cracks. Cracks can be fixed; folded waterstops are difficult to locate and a challenge to fix.

6. If possible, complete burial of reservoirs constructed out of concrete is preferred. A conventionally reinforced concrete reservoir should be completely buried for an equivalent comparison to a prestressed concrete reservoir. Due to inherent design advantages of a prestressed concrete reservoir (core wall is continuously kept in compression), this type of reservoir can be fully exposed, partially buried, or completely buried with no change in performance. Advantages of buried reservoirs include:

   a. Temperature conditions are stabilized. The thermal environment created by burial is beneficial to a longer structure life.

   b. Expansion and contraction problems are minimized. Less leakage should be expected.

   c. Removing the roof and walls from exposure to direct sunlight improves the life of the structure.

   d. Leakage that might occur is not visible (out of sight, out of mind).

   e. Security is improved.

   f. Chlorine residual will likely last longer due to more stable temperature conditions.

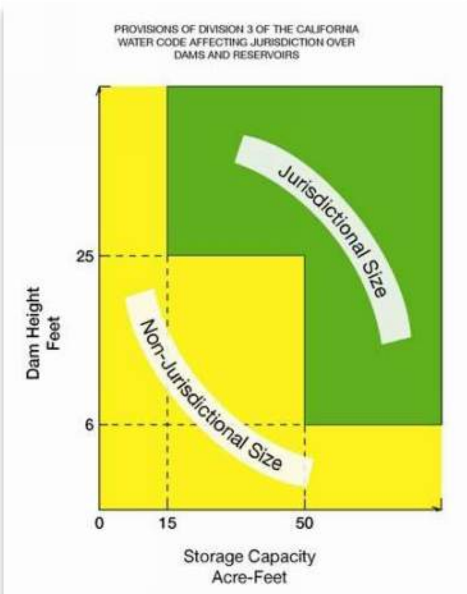
7. The region wide datum change that has occurred since 1962 should be watched carefully. It is critically important that we clarify where we are in the real world at the start, with an entire new survey at the outset. All elevations must be confirmed with an accurate land survey, not by simply referencing a historical document.

8. Before a reservoir is taken out of service for rehabilitation work, scenarios should be discussed with the City’s operations staff including "what if" failure and recovery plans, to try to foresee any potential problems, and try to uncover unintended consequences.

9. The best solution is almost never the lowest cost solution. Reservoir projects are civil engineering projects at their finest, with hydraulics being the primary and most critical problem to be solved. Structural engineering is just a discipline, not the project.

The Advantage of Circular Concrete Reservoirs

Nearly 69% of the City's storage resides in rectangular or square reinforced concrete (RC) reservoirs. The beauty of the design of RC reservoirs is that because no special forms were needed, they could be partially or completely buried, making them relatively economical; providing significant cost/gallon savings over other reservoir types, such as steel. Wall height was typically kept to a maximum of 22-feet, to keep wall thickness within reason, and columns were spaced every 20- to 25-feet. A center dividing wall with a height of 10-14 feet was a common design feature in larger tanks to create two storage compartments, allowing for flexible operation during routine maintenance. As shown on the Chart to the left; non-circular shapes with a wall height of either 25 feet or higher, or a storage capacity greater than 50 acre-feet (16 MG), falls within their jurisdiction size.



Even though all of the City's RC reservoirs have exposed roofs, the more common and current RC reservoir roofs were typically buried in two feet of soil; sloping in one direction for drainage (crown at the ridge line). For the same reason, the floor would slope in the other direction to the outlet pipe. The result was that every column from row-to-row was a different height, creating erection and quality challenges for the contractor. Columns were heavily reinforced to carry soil and live loads on the roof. If not properly supported with spacers (i.e., "dobies"), the reinforcement cage could be displaced during the placement of concrete down the cage. Floor and roof slabs were placed in a "checkerboard" fashion, with alternating pours. Walls were placed in a non-consecutive fashion, a single wall would be erected, leaving spacing between walls, which would be filled in after a 7- day curing period. Corners were placed last. The primary objective was to minimize shrinkage and resulting cracking of the concrete. The concrete mix design was carefully specified to these issues; sand content was limited to 41%; water addition was tightly controlled; and 28-day shrinkage was limited to 0.042%.

By the very nature of the material, over time many RC tanks experienced problems with cracking and chloride corrosion of the reinforcing steel. Concrete in a conventionally reinforced tank wall is allowed to develop tension. Concrete in a circular or rectangular conventionally reinforced tank wall will be placed into tension under hydrostatic, hydro-dynamic, and thermal conditions. Concrete experiencing high levels of tension can result in the yielding of steel reinforcement, cracking of concrete, and subsequent leakage. Cracking can also provide an easy avenue for the intrusion of salts, which can further corrode steel reinforcement. This corrosion creates expansion of the reinforcing which creates further tension in the concrete cover, possibly leading to delamination of concrete from areas of the tank wall. RC tanks were often buried for a reason – out of sight, out of mind. Significant leakage has even been known to occur during the initial hydrostatic test. Besides these structural issues; age of water, poor circulation, and loss of chlorine residual are other common problems of RC reservoir designs.

In prestressed concrete reservoirs, the balance in determining the optimum geometry is wall height to diameter ratio. Slabs on grade are the easiest and least expensive; however, maximizing diameter to provide a high slab on grade construction effort is offset by the most expensive element in tank construction – the roof and supporting column structure. For conventional concrete reservoir designs, the most cost-effective wall should not exceed 22 feet in height to keep wall thicknesses and reinforcing steel requirements within reason. For prestressed concrete designs, 30 feet is the historical water elevation. However, improvements to prestressing technologies over the past 30 years, heights of 40 feet and above have been economically achieved. This has allowed for diameters to be reduced and expensive roof areas to be minimized.

Reservoir Types and Shapes

For reservoir shapes, three reservoir design types are available for consideration:

1. Prestressed concrete (PC);
2. Conventional concrete, circular or rectangular (RC);
3. Steel.

For reservoirs less than 1 MG, steel is the traditional choice, primarily because of the lower cost and because reservoirs in this volume range are considered less critical to uninterrupted operations. For sizes 1-5 MG, circular prestressed concrete and rectangular conventionally reinforced concrete remain the traditional choices. Non-circular shapes are not considered to be cost effective to design or construct in sizes less than 5 MG, and ideally require complete burial. The variable loading conditions that directly result due to the non-circumferential loadings can cause excessive cracking and leakage to occur, particularly in designs that are exposed to natural sunlight. For reservoirs larger than 5 MG, concrete is the traditional choice.

Steel. For reservoirs larger than 1 MG, a steel tank option can be easily dismissed at the initial stage of evaluation for numerous reasons. A steel tank should not be considered if the tank cannot be constructed at grade; partial burial of a steel tank is not recommended. Additionally, for reservoirs critical to the City's water operation, tank types that will require future removal from service for an extended duration to allow re-painting of the interior, may not meet the City's operational needs. It should be expected that a steel tank will need interior re-painting/re-coating at least every 12-15 years, and a period of at least six months should be anticipated for this activity for larger size tanks. To further aggravate the problem with steel, strict seasonal requirements for steel tank coating system applications coincide with the peak demand summer months when a water tank is needed most. With the ever-increasing regulations regarding water and air quality, the potential for more stringent steel tank painting regulations is significant. Due to the inevitable stricter environmental regulations to contain and recollect abrasive blast material and removed coatings, it may be necessary to include tank “shrouding” in future maintenance costs. In sizes above 1 MG, a steel tank option would be an extremely poor choice.

Concrete. With regards to concrete, either rectangular conventional or prestressed concrete can be considered. Prestressed concrete reservoirs are typically constructed in the 30-35 feet wall height range but are also very economical for any water depth/wall height.

A circular prestressed concrete tank is an extremely efficient system to hold water. For a given water depth, a circular tank requires the least wall area of any shape to contain the desired capacity. A circular prestressed tank wall has the corewall in pure axial compression and the prestressing steel in pure axial tension, thereby utilizing 100 percent of both materials in their most efficient state of stress.

Rectangular versus Round. Both prestressed concrete and rectangular conventionally reinforced tanks use concrete as their primary component. However, there are vast differences in performance between the two alternatives.

The most obvious, and most significant, difference between the two types of tanks is that the concrete is allowed to go into tension in one type and continuously kept in compression in the other. A properly designed prestressed tank will have its corewall held in compression, not only under hydrostatic loading, but under thermal and seismic loadings as well. In contrast, the concrete in a rectangular conventionally reinforced tank will be placed into tension under these same conditions. Under hydrostatic, thermal, and seismic loads, reinforced concrete walls are put into a state of tension, which can result in yielding of steel reinforcement, cracking of concrete and subsequent leakage. Cracking can also provide an easy avenue for the intrusion of salts, which can corrode the steel reinforcement. Unfortunately, reinforced concrete under large liquid pressure loads behaves chaotically due to the large amount of tension in the concrete. Reinforced concrete in this condition frequently cracks even when designed to current Codes.

Even if concrete and reinforcement tensile stresses are kept low and steel reinforcement is closely spaced to minimize cracking, over long periods of time the small cracks that do develop will tend to migrate toward one another. This will take place when localized bond failures occurring along the reinforcing bars result in larger, potentially troublesome cracks.

In a rectangular conventionally reinforced concrete tank, hydrostatic loads are not resisted by uniform hoop stress, as they are in a circular prestressed concrete tank. Instead, the water must be held back by a retaining wall cantilevered off the floor. Although fixed connections between the wall and wall footing can save cost, this cantilevered wall produces large moments in the floor and wall, which promotes subsequent cracking and leakage. Since it is very difficult to fully resist the water load just by a cantilever from the floor, the wall is often also tied to the roof so it is fixed at its base and top. Although this helps the wall carry the water load, it can also promote significant problems in the roof when subjected to thermal expansion.

A major place of concern is caused by the stress concentrations that occur at the sharp corners of the tank, which are extremely difficult to accurately analyze and design for. This is one reason why you normally see pressure vessels for gas and other liquids follow the form of a circle, not a square. All these stress concentrations and moment conditions that produce tension can ultimately result in leakage. In the United States, the vast majority of large concrete tanks are built as circular prestressed concrete. In fact, in this country, it is very uncommon for large conventionally reinforced concrete tanks to be built above grade. History has shown that the allowable leakage loss rate can equal the full capacity of the tank each year.

Even though circular prestressed concrete tanks could receive short term cost benefits by using rigid connections, a Type 1 prestressed concrete tank design incorporates free connections at the base and top of the wall. Even with free moving connections, tank walls are still subject to some circumferential and vertical bending moments, caused by differential temperature and differential dryness conditions between the inside and outside of the wall. To counteract these effects, circular prestressed concrete design adds vertical prestressing plus circumferential prestressing above and beyond that required for the hydrostatic loads. This keeps the concrete crack-free in both the vertical and circumferential directions, unlike a conventionally reinforced concrete tank.

Most Type 1 prestressed concrete tank designs use a floor with a slight upward cone shape that promotes drainage and will actually help keep the floor in compression. In comparison, many rectangular conventionally reinforced concrete tanks use hopper floors or sloped floors, which are prone to cracking and leakage.

<u>Constructability</u>

Placement of RC Tank Walls:

A RC tank wall will require two or three curtains of reinforcement steel in both the horizontal and vertical directions. Because of this reinforcing steel congestion, contractors typically pour the RC tank walls by dropping and vibrating the concrete from the top of the wall. Besides losing the advantage of being able to visually inspect the concrete placement (which is possible in the PC tank walls), the dropping of concrete from the top of the wall can result in concrete segregation and inadequate consolidation. Hollow areas and voids that may develop at the bottom of the wall, which is the most critical area of the tank, can result in subsequent leakage.

RC tanks must have the slabs poured in a checkerboard fashion to manage shrinkage. Placements are scheduled one week apart to allow 7-day shrinkage, typically equal to 50% of 28-day shrinkage. It would not be practical to wait four weeks between placements.

Placement of PC Tank Walls:

A Type I PC tank wall typically has a low abundance of steel in the corewall, since circumferential prestressing is added after the wall has been poured. The concrete for PC tank walls is normally placed through pouring openings in the side of the wall forms, thereby allowing the contractor to properly vibrate and place it. Visual inspection of the concrete placement and consolidation is possible through the wall form openings. A denser, homogeneous corewall should result due to these measures.

Unlike Rectangular Conventionally Reinforced Concrete Tanks, Circular Prestressed Concrete Tanks:

- Are designed under a comprehensive national tank code.
- Allow for the design of seismic and thermal loadings.
- Use galvanized material for their principal horizontal reinforcement.
- Incorporate flexible wall-to-base and wall-to-roof connections by using neoprene pads.
- Incorporate seismic cables between the wall-to-base and the wall-to-roof.
- Incorporate waterstops in all floor, wall, and roof joints.
- Use low-chloride concrete to prevent corrosion of the reinforcing steel.
- Use exterior wall coatings to prevent the intrusion of salts into the concrete.
- Use exterior roof membranes to counteract thermal cracking in the roof.
- Do not require coating of internal concrete surfaces.

SUMMARY OF BENEFITS OF PRESTRESSED CONCRETE TANKS:

In the United States, the vast majority of concrete tanks built today are circular prestressed concrete. Conventionally reinforced tanks, whether or not they are rectangular or circular, and whether or not they are above or below ground, are not normally the first choice of municipalities. Therefore, for critically important reservoirs, prestressed concrete is by far the premium choice for the following reasons.

- Enhanced operational and seismic performance as roof and footings do not have fixed connections. Floor to wall and wall to roof are designed utilizing an “anchored flexible base.” This allows for all three components of the tank structure to move independently of each other.
- Fewer joints in floor slabs, walls, and roofs for less potential of leakage.
- Walls are virtually crack-free, since they are in compression both vertically and horizontally. This provides less likelihood of leakage and corrosion of steel reinforcing.

- Significantly improved wall pouring capabilities. Conventionally cast-in-place concrete tanks typically have significant amounts of reinforcing in the walls. With standard forms for conventional cast-in-place tanks, pours are commonly required from the top of the form. This poses a significant challenge to properly consolidating the full height of a potentially 20’ to 30’ tall wall with congested reinforcing steel. These challenges have a high likelihood of reducing the quality of the wall. With reduced need for mild reinforcing in a prestressed concrete tank wall, pour windows can be easily utilized, which greatly enhances the final product.

- Prestressed concrete tank walls are in bi-axial permanent compression. Tanks are designed to be in 200 psi residual compression, ensuring a long-lasting leak free wall.

- Prestressed concrete tanks have excellent performance with temperature and dryness stress variations. As the wall to roof also utilizes an anchored flexible connection, the slab is free to expand and contract with the thermal variations. This is opposed to a conventionally reinforced concrete tank where thermal loads in the roof can induce bending moments when rigidly connected.

- Circular prestressed concrete tanks provide enhanced water quality. The circular shape provides for natural and efficient turnover of water in the tank with no dead spots. This is opposed to square or rectangular concrete tanks which can have “dead zones” towards the corners of the tank, creating a significant water quality issue and promoting bacteria growth.

- Prestressed concrete tanks allow for efficient construction operations, with concurrent construction of floor, wall, roof and columns. This results in an optimized construction schedule, often significantly decreasing durations as compared to a reinforced concrete tank.

Life Cycle Costs

Prestressed concrete is the lowest life cycle alternative. Though the initial cost of a steel tank is traditionally lower, over the 50-year span of a steel tank’s useful life, it will require periodic replacement of the interior and exterior coating systems. For example, though the initial cost of the steel tank is only $3.6 million, when considering life cycle costs associated with maintaining the steel option over 100 years (the expected life of a prestressed concrete tank), the cost of steel is nearly $10 million. The cost for prestressed concrete is less than half this amount. Constructing a circular prestressed concrete reservoir will provide reliable storage with minimal maintenance for the next 75-100 years.

Operations and Maintenance Issues

Concrete is superior to steel with respect to operations and maintenance issues. Considerably less downtime for maintenance is needed (one day per year). Concrete does not require any protective coatings when used for storing potable water. Steel requires regular maintenance, attention to interior and exterior coatings, and cathodic protection systems. These traditional coatings have fallen out of favor in many parts of the United States due to the potential health and taste problems sometimes attributed to volatile organic compounds leaching out of the coatings and into the water.

Seismic Risk Assessment

Prestressed concrete design is by far the better choice for seismic Zone considerations. The footings, walls, and roof of a prestressed concrete tank are not adjoined with fixed connections. This allows for all three components of the tank structure to move independently of each other, should an earthquake occur. The wall sections sit on neoprene bearing pads and are secured to the footing with seismic cables. In the event of an earthquake, the wall may undergo limited displacement in any direction and return to its original location regardless of the horizontal and vertical force accelerations. This would not be the case for a steel tank design that uses fixed connections. Prestressed concrete is also more resilient against wildfires and errant gun fire compared to steel.

An important component to consider for reservoirs located in seismically active zones is their capacity to absorb energy and to sustain cyclic loading during a seismic event. The design of prestressed concrete tanks takes into consideration horizontal and vertical ground accelerations, the sloshing of the water, and the overturning moments. With the majority of new generation prestressed concreted tanks located in the Western United States in areas of high seismic activity, Type 1 prestressed designs have not experienced earthquake-induced problems in the past 50 plus years. During both the 1971 San Fernando and the 1994 Northridge earthquakes, there were numerous Type 1 tanks located in close proximity to the respective epicenters. None of these tanks exhibited any major structural damage.

Possibly the most dramatic test of a Type 1 prestressed tank was a 10 MG, 40-foot-tall tank located within a couple miles of the epicenter of the Northridge earthquake. This tank is completely above ground and was full at the time of the earthquake. In addition to this tank, there were approximately two dozen Type 1 tanks within a 20-mile radius of the epicenter that performed excellently. In contrast, many steel tanks were severely damaged. The typical problem experienced was the buckling of the steel tank walls. More than twenty steel tanks were damaged in this natural disaster.

Reliability

Prestressed concrete offers the most reliable design. As noted in this report, the prestressed concrete reservoir design is the superior choice for seismic considerations. Since no coatings are required, the time required for operations and maintenance activities are considerably reduced. Prestressed concrete is a proven design for Seismic Zone 4 in California.

In summary, a prestressed concrete reservoir is considered “Best Value” for the following reasons:

- ✓ Prestressed concrete requires minimal maintenance over the 100-year expected life cycle of the reservoir.
- ✓ Prestressed concrete is superior for seismic conditions and is considerably less susceptible to earthquake damage.
- ✓ Prestressed concrete can be fully or partially buried, thereby reducing earthwork requirements, while minimizing environmental impacts.
- ✓ Prestressed concrete provides superior reliability compared to all other options.

Overall, the City has a well-connected water delivery system operated by a knowledgeable and dedicated operations team. System seismic vulnerabilities are primarily entirely due to the age of the City’s water infrastructure assets, not because of neglect or poor attention to proactive maintenance. We have very much enjoyed this extremely challenging engineering engagement that would not have come together without the assistance and support of City engineering and operations staff.

THE END

12 REFERENCES

AAWA Distribution Systems. (2008). M31 Distribution System Requirements for Fire Protection, Fourth Edition

AAWA Distribution Systems. (2013). M42 Steel Water Storage Tanks, Revised Edition

AAWA Distribution Systems. (2014). M22 Sizing Water Service Lines and Meters, Third Edition

California State Water Resource Control Board. Table 1: Typical Equipment Life Expectancy.

California Department of Water Resources. (2004). Upper Santa Ana Valley Groundwater Basin, Bunker Hill Subbasin.

California Department of Water Resources. (2004). Upper Santa Ana Valley Groundwater Basin, Yucaipa Hill Subbasin.

California Office of Administrative Law. (2018). Code of Regulations Title 22 – Division 4 Chapter 3.

California State Water Resource Control Board. (2018). Maximum Contaminant Levels and Regulatory Dates for Drinking Water U.S. EPA vs. California.

The City of Redlands. (2017). City of Redlands General Plan 2035.

The City of Redlands. (2020). City Owned Citrus Groves

The City of Redlands. (2021). Fire Code Table, Fire Flow Requirements Table.

The City of Redlands. (2021). Consumer Confidence Report.

The City of Redlands. (2021). City of Redlands Draft Water Shortage Contingency Plan.

The City of Redlands. (2021). 2020 IRUWMP Part 2 Chapter 4 Redlands 2020 UWMP.

Eastern Municipal Water District. (2007). Water System Planning & Design.

Environmental Finance Center New Mexico Tech. (2006). Asset Management: A Guide for Water and Wastewater Systems.

MASC. (2016). Life Expectancy of Water Distribution Lines.

U.S. EPA. (2002) Effect of Water Age on Distribution System Water Quality.

Western Municipal Water District. (2011). Design Criteria for Water Distribution Systems.

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