Rancho Murieta Community Services District

Integrated Water Master Plan

October 2024

ADKINS
ENGINEERING & SURVEYING

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Executive Summary

Introduction

The purpose of this Integrated Water Master Plan (IWMP) is to perform a comprehensive analysis of the Rancho Murieta Community Services District's (District) water system, to identify system deficiencies, to determine future water system supply requirements, and to develop water system facility improvements that correct existing deficiencies and that provide for future system expansion. The District's existing IWMP was completed in 2010. This new IWMP meets the requirement for the District to maintain a current IWMP.

Existing and Future Demands

Demand projections are estimated for existing and buildout scenarios. Demands are based on existing and future land uses and demand factors for each lot type in the District. Maps showing existing and future lot types in the District were prepared using information from the District and its developers and reviewed with the District for accuracy. Demand factors are derived from historic usage data and reflect current consumption patterns, with adjustments made for anticipated changes in usage behavior and the effects of climate change.

The current average day demand is around 1.5 MGD and maximum day demand is around 2.8 MGD, based on production and consumption records. At buildout, the average day demand is projected to increase to 3.0 MGD and maximum day demand to 5.5 MGD.

The areas of anticipated future growth are the Rancho North Villages A through G, Riverview, Residences East and West, the Retreats, and new commercial developments in Murieta Gardens. The buildout timeline of these developments is unknown at this time, and depends on many factors.

System Evaluation

Water system evaluations determined the adequacy of the existing system to meet existing and future demands. The evaluations included raw water sources, raw water storage reservoirs, water treatment plants, booster pumps, treated water transmission pipelines, treated water storage tanks, water distribution networks, reclaimed water treatment facilities, reclaimed water booster pumps, reclaimed water storage, and reclaimed water distribution networks.

Based on the evaluation results, required improvements were formulated to address identified deficiencies at the existing and buildout timeframes. Hydraulic models were created for the District's domestic water and reclaimed water systems, for both the existing conditions and projected buildout conditions. These were used to assist in the water system analysis. The alternatives consider buildout needs to ensure that facility upgrades will be adequately sized to avoid future upsizing projects.

Future growth areas will be served by extending the existing distribution system. Future growth within the existing pressure zones will be served through new waterline extensions. Additional supply, pumping, and storage capacity will be required for these new areas. Improvements to existing pipelines will also be needed to provide adequate hydraulic capacity to convey supply from storage facilities to new customers.

The alternatives developed in this IWMP may differ from the projects that the District ultimately selects. There could be other project options that would meet the same performance goals as the alternatives in this IWMP aim to meet.

Summary of Improvement Alternatives

The Capital Improvement Program (CIP) includes the costs of improvements required for all major facilities, including improvements to existing pipelines but excluding pipeline extensions to future areas. The CIP does not include the cost of new pipeline extensions to areas that are currently undeveloped and not served by an existing pipeline. These pipeline extensions will be constructed by developers as part of the new developments. Developers may also be required to contribute to the cost for new water production, storage and pumping facilities as required by District standards.

Types of improvements included in the CIP are:

- New groundwater wells to provide supply resiliency

- Upgrades to allow for the use of Clementia Reservoir for domestic system storage
- New domestic treated water tanks
- Improvements to existing pipelines to improve fire protection capabilities
- A new potable water booster station to provide pressure to new developments
- Improvements to the Wastewater Reclamation Plant (WWRP) to increase capacity
- Improvements to reclaimed water transmission pipelines and pump station

Figure ES-1-1 below shows water system improvement alternatives to meet existing and future needs. Table ES-1 summarizes required capacities and costs. CIP projects are staged by timeframe needed as a guideline for District staff in determining specific priorities and timing for project implementation based on future development schedules and overall District needs. The recommended timeframe for each improvement group is also included in Table ES-1. There also may be other project options and timelines that will allow the District to meet performance goals.

PROPOSED IMPROVEMENTS

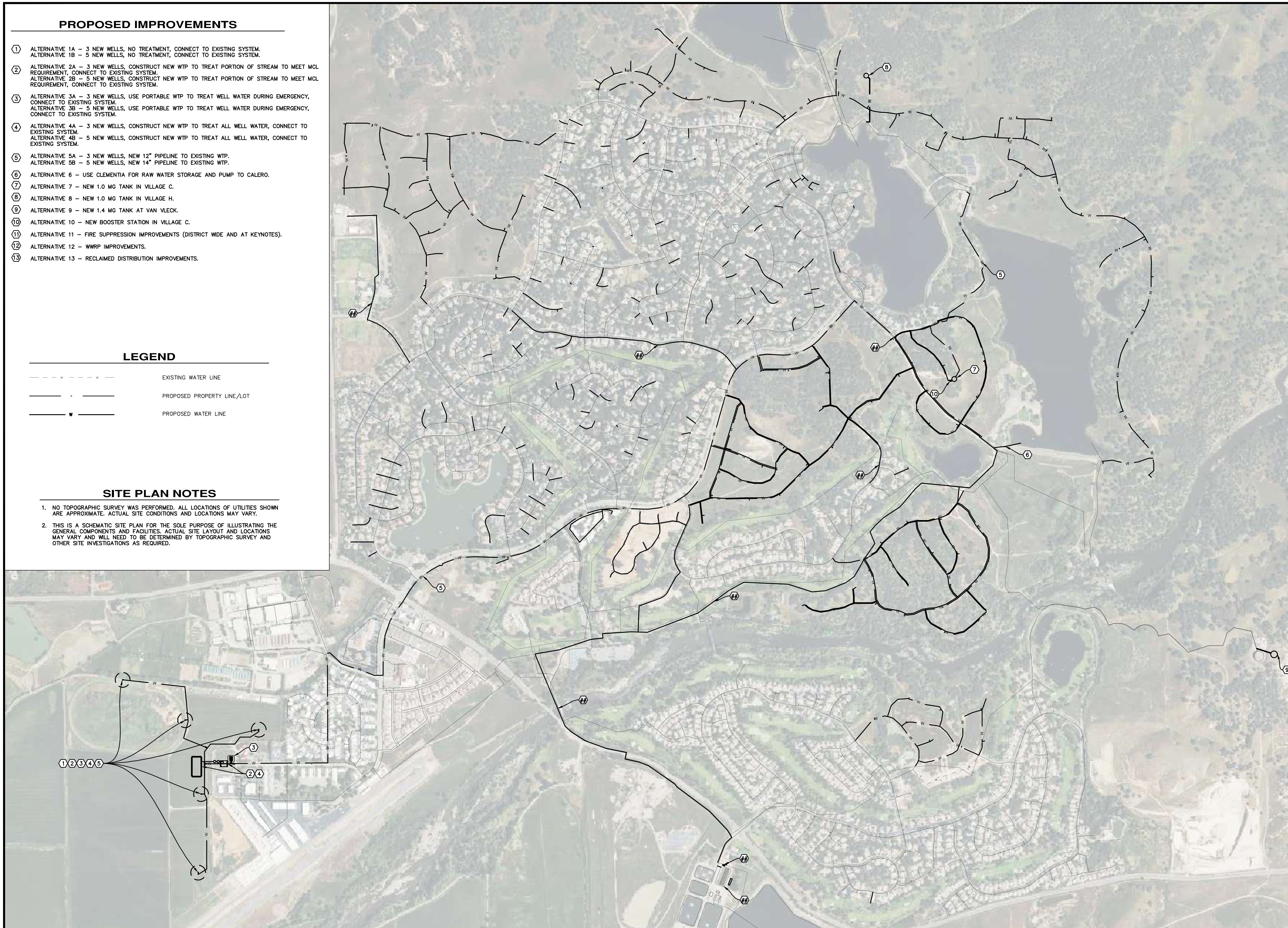
1. ALTERNATIVE 1A – 3 NEW WELLS, NO TREATMENT, CONNECT TO EXISTING SYSTEM.
   ALTERNATIVE 1B – 5 NEW WELLS, NO TREATMENT, CONNECT TO EXISTING SYSTEM.
2. ALTERNATIVE 2A – 3 NEW WELLS, CONSTRUCT NEW WTP TO TREAT PORTION OF STREAM TO MEET MCL REQUIREMENT, CONNECT TO EXISTING SYSTEM.
   ALTERNATIVE 2B – 5 NEW WELLS, CONSTRUCT NEW WTP TO TREAT PORTION OF STREAM TO MEET MCL REQUIREMENT, CONNECT TO EXISTING SYSTEM.
3. ALTERNATIVE 3A – 3 NEW WELLS, USE PORTABLE WTP TO TREAT WELL WATER DURING EMERGENCY, CONNECT TO EXISTING SYSTEM.
   ALTERNATIVE 3B – 5 NEW WELLS, USE PORTABLE WTP TO TREAT WELL WATER DURING EMERGENCY, CONNECT TO EXISTING SYSTEM.
4. ALTERNATIVE 4A – 3 NEW WELLS, CONSTRUCT NEW WTP TO TREAT ALL WELL WATER, CONNECT TO EXISTING SYSTEM.
   ALTERNATIVE 4B – 5 NEW WELLS, CONSTRUCT NEW WTP TO TREAT ALL WELL WATER, CONNECT TO EXISTING SYSTEM.
5. ALTERNATIVE 5A – 3 NEW WELLS, NEW 12" PIPELINE TO EXISTING WTP.
   ALTERNATIVE 5B – 5 NEW WELLS, NEW 14" PIPELINE TO EXISTING WTP.
6. ALTERNATIVE 6 – USE CLEMENTIA FOR RAW WATER STORAGE AND PUMP TO CALERO.
7. ALTERNATIVE 7 – NEW 1.0 MG TANK IN VILLAGE C.
8. ALTERNATIVE 8 – NEW 1.0 MG TANK IN VILLAGE H.
9. ALTERNATIVE 9 – NEW 1.4 MG TANK AT VAN VLECK.
10. ALTERNATIVE 10 – NEW BOOSTER STATION IN VILLAGE C.
11. ALTERNATIVE 11 – FIRE SUPPRESSION IMPROVEMENTS (DISTRICT WIDE AND AT KEYNOTES).
12. ALTERNATIVE 12 – WWRP IMPROVEMENTS.
13. ALTERNATIVE 13 – RECLAIMED DISTRIBUTION IMPROVEMENTS.

LEGEND

— — w — — w —	EXISTING WATER LINE
——— - ———	PROPOSED PROPERTY LINE/LOT
——— W ———	PROPOSED WATER LINE

SITE PLAN NOTES

1. NO TOPOGRAPHIC SURVEY WAS PERFORMED. ALL LOCATIONS OF UTILITIES SHOWN ARE APPROXIMATE. ACTUAL SITE CONDITIONS AND LOCATIONS MAY VARY.
2. THIS IS A SCHEMATIC SITE PLAN FOR THE SOLE PURPOSE OF ILLUSTRATING THE GENERAL COMPONENTS AND FACILITIES. ACTUAL SITE LAYOUT AND LOCATIONS MAY VARY AND WILL NEED TO BE DETERMINED BY TOPOGRAPHIC SURVEY AND OTHER SITE INVESTIGATIONS AS REQUIRED.



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DISTRIBUTION AND STORAGE ALTERNATIVES
FOR
RANCHO MURIETA CSD
ALL PROPOSED IMPROVEMENTS

SCALE	SHOWN
DATE	08/08/2024
DRAWN BY	BAG
PROJ. NO.	3971-02

FIGURE
ES-1

Table ES-1: Summary of Alternatives and Cost Estimates

Summary of Alternatives and Cost Estimates			
#	Description	**Existing/Buildout**	**Estimated Cost**
1A	3 New Wells, No Treatment	Existing	$6,349,000
1B	5 New Wells , No Treatment	Buildout	$10,455,000
2A	3 New Wells, Partial Treatment	Existing	$12,533,000
2B	5 New Wells, Partial Treatment	Buildout	$21,284,000
3A	3 New Wells, Portable Treatment	Existing	$6,349,000
3B	5 New Wells, Portable Treatment	Buildout	$10,455,000
4A	3 New Wells, Full Treatment	Existing	$17,184,000
4B	5 New Wells, Full Treatment	Buildout	$29,579,000
5A	3 New Wells, Treat at 3 New Wells WTP	Existing	$11,987,000
5B	5 New Wells, Treat at Existing WTP	Buildout	$16,855,000
6	Use Clementia for Domestic Storage	Buildout	n/a[1]
7	New Tank in Village C	Buildout	$3,272,000
8	New Tank in Village H	Buildout	$3,438,000
9	New Tank at Van Vleck	Buildout	$4,254,000
10	Village C Booster Station	Buildout	$1,678,000
11	New Hydrants and Pipeline Upsizing	Existing	$8,397,000
12	WWRP Improvements	Existing	$376,000
13	Reclaimed Transmission Improvements	Buildout	$5,547,000
[1]Since the cost for this alternative is primarily for pump rental, the capital cost is not comparable and is not included in this table.			

Implementation Considerations

Sizing, location, and estimated costs of master plan projects are at a conceptual level. Project implementation will require predesign studies, including specific routing and siting studies, environmental review, and detailed design of specific projects. Timing for specific projects will be determined based on development needs, coordination with other construction projects, such as those for other utilities and street improvements, or for other District needs.

CHAPTER 1. Introduction

This section describes the purpose, organization, and scope of the IWMP, identifies acronyms and abbreviations used in the report, and lists acknowledgements.

1-1. Purpose

The District prepared this master plan update to ensure adequate water system capacity for existing and future customers, and to plan for water system improvements in developing areas. The study area for this master plan update encompasses all lands within the District boundary.

Since the last IWMP in 2010, significant changes have transpired in the District's plans for development, resulting in the need for an updated IWMP. This IWMP includes current and future development information to more accurately reflect current levels of development and enable District staff to respond effectively to new water system demands. An up-to-date IWMP enables the District to proactively set appropriate developer requirements and fees to address improvements needed for new development as it occurs.

The planning timeframe extends to buildout within the District boundary. Water demands projections were based on current planning information regarding future land uses during the planning horizon. Due to the long-range nature of buildout conditions, the buildout scenario will be re-evaluated in future master plan updates as more information becomes available.

The California Water Code requires all urban water suppliers that provide water for municipal purposes either directly or indirectly to more than 3,000 customers (or supply more than 3,000 acre-feet of water annually) to prepare an Urban Water Management Plan (UWMP) and Water Shortage Contingency Plan (WSCP). While the District does not qualify under either of those criteria, it is projected to have significantly more than 3,000 customers at buildout. This IWMP will support an UWMP at the time when it becomes required.

1-2. Organization of the IWMP Report

Chapters 2 through 4 of the IWMP report describe the existing water system facilities, water system performance objectives, and water demand projections. Chapter 5 describes the water system analysis conducted to determine required supply, treatment, storage, pipeline, and reclaimed system capacities for existing and future demands. Chapter 6 develops improvement alternatives to meet existing and future water system needs, including estimated costs and phasing. Chapter 7 summarizes and concludes the report.

1-3. Scope of Services

The District retained Adkins Engineering & Surveying, Inc (Adkins) and Maddaus Water Management, Inc (MWM) to prepare the IWMP. The following major elements comprise the scope of work for the IWMP:

- Existing and Future Demand Analysis – Study area features and land use assumptions have been compiled for use in the overall IWMP effort. Water demand projections have been developed based on development projections provided by the District. Demand factors and peaking factors have been derived from historic usage data.
- Existing Water System Features; Performance Objectives – Information on existing water system facilities has been updated to use as a basis for the system analysis. Performance objectives have been established to define levels of service for the water system evaluation.
- Hydraulic Model Development and Calibration – The District did not have a model of its domestic water system prior to this IWMP. This effort included developing a EPANet2.2 model of the District's domestic water system and calibrating it using system operating data. Models including future demands and recommended improvements were also developed. This effort also included developing existing and buildout models for the reclaimed water system.

- Water System Analysis and Recommended Improvements – Water system evaluations have been conducted to determine adequacy of capacity of existing supply, treatment, transmission, storage, and distribution facilities for both domestic and reclaimed water systems. Based on the analysis results, improvement recommendations have been formulated to address identified deficiencies.
- IWMP Report – This report has been prepared to document the key assumptions, findings, and recommendations of the IWMP analyses.

1-4. Distribution of Work

This masterplan was completed by the combined efforts of Adkins and MWM. In general, Adkins was responsible for the development of hydraulic models, evaluation of physical infrastructure, and development of alternatives. In general, MWM was responsible for evaluating the reliability of water supplies, developing demand projections, and modeling water supply availability under different future scenarios.

Michael Moser, P.E. of Adkins is responsible for the following sections and sub-sections:

- Executive Summary - all
- Chapter 1: Introduction – all
- Chapter 2: Existing Facilities – all
- Chapter 3: Performance Objectives – all
- Chapter 4: Water Demands– sections 4-4 and 4-5
- Chapter 5: System Analysis – section 5-1, and sections 5-3 through 5-6.
- Chapter 6: Improvement Alternatives – all
- Chapter 7: Summary, Recommendations, and Conclusions – all

Lisa Maddaus, P.E. of MWM is responsible for the following sections and sub-sections:

- Chapter 4: Water Demands – sections 4-1 through 4-3
- Chapter 5: System Analysis – section 5-2

1-5. Acknowledgements

This report would not be possible without the valuable assistance and participation of the following District staff:

Travis Bohannon	Interim Director of Operations
Ron Greenfield	Utilities Supervisor
Michael Fritschi	Former Director of Operations
Mimi Morris	General Manager

1-6. Acronyms and Abbreviations

Below are abbreviations and acronyms used in this report.

AACE	American Association of Cost Engineering
ACP	asbestos cement pipe
ADD	average day demand
Adkins	Adkins Engineering & Surveying, Inc.
ADU	accessory dwelling unit
ADWF	average dry weather flow
AF	acre-feet
AFY	acre-feet per year
ASR	aquifer storage and recovery
AWWA	American Water Works Association
CCB	chlorine contact basin
CCP	chlorine contact pipe
CCR	California Code of Regulations
cfs	cubic feet per second
CHW	Hazen-Williams coefficient
CIP	Capital Improvement Plan
CT	contact time
DAF	dissolved air floatation
DE	Dunn Environmental, Inc.
District	Rancho Murieta Community Services District
DO	dissolved oxygen
DWP	Drinking Water Program
DWR	California Department of Water Resources

EDU	equivalent dwelling unit
ELAP	Environmental Laboratory Accreditation Program
fps	feet per second
FSA	financing and services agreement
ft	feet
gal	gallons
gpcd	gallons per capita per day
GPDA	gallons per day per account
GPM	gallons per minute
HP	horsepower
I/I	infiltration and inflow
IFC	International Fire Code
in	inches
IWMP	integrated water master plan
LF	linear feet
MCL	maximum contaminant level
MDD	maximum day demand
MG	million gallons
MGD	million gallons per day
MMD	maximum month demand
MPN	most probable number
MWM	Maddaus Water Management, Inc.
NCPS	North Course Pump Station
NPDWR	National Primary Drinking Water Regulations
NPV	net present value
NRW	non-revenue water
NSDWR	National Secondary Drinking Water Regulations
NTU	nephelometric turbidity unit
O&M	operation and maintenance
PHD	peak hour demand

PIP	plastic irrigation pipe
pph	persons per household
psi	pounds per square inch
PVC	polyvinyl chloride
RII	rainfall induced infiltration
RMCC	Rancho Murieta Country Club
RWQCB	Regional Water Quality Control Board
SCADA	supervisory control and data acquisition
sf	square feet
SVM	shared vision model
SWTR	surface water treatment rules
TDH	total dynamic head
UWMP	Urban Water Management Plan
USGS	United States Geological Survey
VFD	variable frequency drive
WSCP	water shortage contingency plan
WTP	water treatment plant
WWRP	wastewater reclamation plant

CHAPTER 2. Existing Facilities

2-1. Overview

This chapter describes the study area, history, and present conditions of the District's water systems, both domestic and reclaimed, which serves as a baseline for planning and analysis. This chapter outlines the domestic water system and the reclaimed water system, including:

1. Study Area
2. System History
3. Raw water sources
4. Water rights
5. Water treatment facilities
6. Treated water storage facilities
7. Treated water distribution facilities
8. Wastewater reclamation facilities
9. Reclaimed water transmission and distribution facilities

These parameters are incorporated into design criteria, modeling, and analysis of existing and buildout conditions, described more in following chapters.

2-2. Study Area

Figure 2-1 shows the general location of Rancho Murieta. It is located on the eastern boundary of Sacramento County, with Amador County to the east. It is approximately 23 miles southeast of the City of Sacramento along Highway 16.

The study area is comprised of rolling terrain. Ground elevations in the District range from about 140 feet in the southwestern portion to 350 feet along the east side of Calero Reservoir.

The climate is classified as Mediterranean-Hot Summer. Rainfall averages 21 inches annually. In July, the average daily temperature ranges from a high of about 97 degrees Fahrenheit to a low of about 61 degrees. In January, the average daily temperature ranges from a high of about 58 degrees to a low of about 40 degrees.

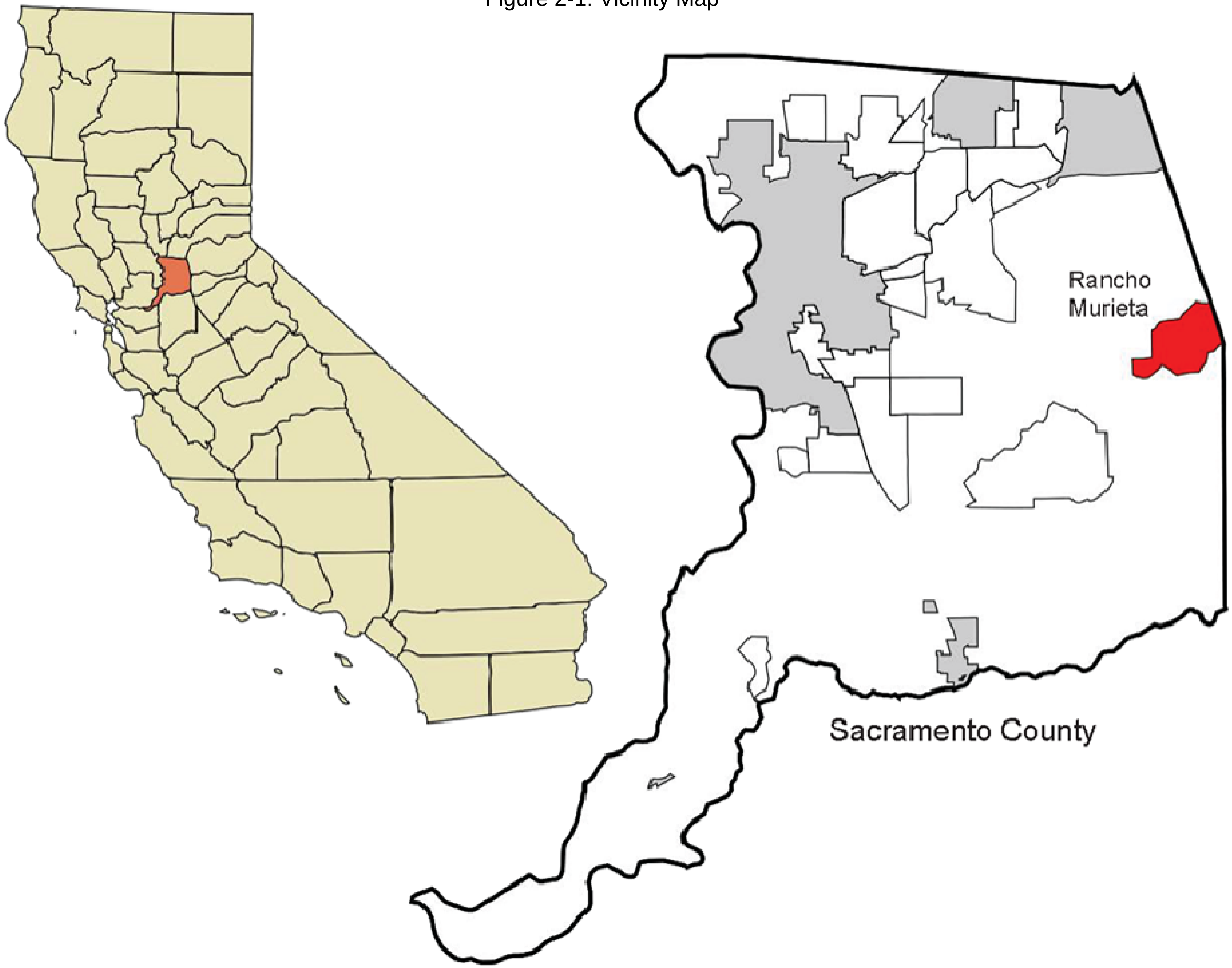
The current population in the study area is about 6,900 residents. At buildout, the population is projected to be about 10,500 residents. Figure 2-2 shows the study area for this IWMP, as defined by the District boundary. The study area includes all lands within the District boundary.

The study area is comprised of rolling terrain. Ground elevations in the District range from about 140 feet in the southwestern portion to 350 feet along the east side of Calero Reservoir.

The climate is classified as Mediterranean-Hot Summer. Rainfall averages 21 inches annually. In July, the average daily temperature ranges from a high of about 97 degrees Fahrenheit to a low of about 61 degrees. In January, the average daily temperature ranges from a high of about 58 degrees to a low of about 40 degrees.

The current population in the study area is about 6,900 residents. At buildout, the population is projected to be about 10,500 residents.

Figure 2-1: Vicinity Map



ADKINS ENGINEERING & SURVEYING

RANCHO MURIETA RECLAIMED WATER CONVEYANCE SYSTEM

FIGURE 2-2

H:\3971-02 2023 Integrated Water Master Plan\GIS\3971-02 Rancho Murieta Vic.mxd

2-3. System History

2-3.1. Domestic Water System History

The District was formed in 1982 to provide water supply, wastewater, storm drainage and flood control services to the master-planned community of Rancho Murieta. The area served by the District encompasses approximately 3,500 acres. Land uses within this service area include the development of approximately 2,000 acres for single-family residences, townhouses, apartments, duplexes and manufactured homes, in addition to two golf courses and light commercial. The Cosumnes River is the primary source of water for the District, from which water is seasonally diverted to three storage reservoirs (Calero, Chesbro, and Clementia).

The Rancho Murieta Master Plan (1984) specifies that "the reservoirs shall be maintained as integral parts of the water supply system, the drainage system or the wastewater system as established in the project water budget." The water budget described in the 1984 Master Plan follows the “One Water” approach, a nationally recognized approach that envisions managing all water in an integrated, inclusive, and sustainable manner. Rancho Murieta has long embraced the concept of “One Water” to optimize their available water resources, including using their off-stream storage reservoirs and reuse of reclaimed water for irrigation.

2-3.2. Reclaimed Water System History

The District owns and operates the Wastewater Reclamation Plant (WWRP) which receives domestic wastewater from the community of Rancho Murieta and currently provides secondary- and tertiary-level treatment to reclaim water for irrigation. Throughout the history of the WWRP, it has provided water for irrigation to the two golf courses in the District, as well as to the Van Vleck ranch south of the District.

2-4. System Inventory

2-4.1. Sources

The District's potable water supply consists of surface water diversions from the Cosumnes River, along with a small amount of precipitation runoff that naturally flows into the reservoirs. These diversions are seasonal and dictated by water rights permit 16762, which allows for diversions between the dates of November 1st and May 31st into the District's three storage reservoirs: Calero, Chesbro, and Clementia.

The Cosumnes River watershed encompasses nearly 1,300 square miles. The watershed begins at the western slopes of the Sierra Nevada mountains at an elevation of nearly 8,000 feet. The Cosumnes River drops to 130 feet in elevation as it passes through Rancho Murieta. Only 4% of the watershed upstream from Rancho Murieta is controlled by dams or reservoirs.

The Cosumnes River is an 80-mile-long river with relatively natural, unregulated stream flows that vary from higher winter-spring flood flows to reduced or intermittent summer flows. The upper reaches of the Cosumnes River are in the Eldorado National Forest, while the lower reaches, on its way to the confluence with the Mokelumne River and the San Joaquin Delta, flow through one of the most biologically rich regions in California's Central Valley, consisting of riparian forests, wetlands, vernal pool-dotted grasslands, and blue oak woodlands as well as productive row-crop agriculture, pasture lands, and rural homes and businesses. See Figure 2-5 for a map of the Cosumnes watershed.

The diversion from the Cosumnes River is located at the Granlees Dam and includes a diversion structure and three pumps. Two of these pumps are 125 horsepower (HP), and the third is 500 HP. The third pump is only operable when flows in the Cosumnes exceed 175 cfs. Raw water is conveyed to Calero or Chesbro via a 33-inch pipeline, or to Clementia via a 21-inch pipeline. Clementia's water level is maintained independently of Calero and Chesbro. Calero is at the highest elevation of the three reservoirs and is the first to be drawn from for use. Raw water is delivered to the Water Treatment Plant (WTP) through a 30-inch siphon between Calero and Chesbro, and a 36-inch supply line from Chesbro to the WTP. These reservoirs and their storage capacities are summarized in Table 2-1. Adkins performed a bathymetric survey of Chesbro and Calero Reservoirs in 2023 to develop depth-to-volume relationships, or stage-storage curves. Figure 2-3 shows the volume curve for Calero Reservoir and Figure 2-4 shows the volume curve for Chesbro Reservoir.

Figure 2-3: Calero Stage-Storage Curve

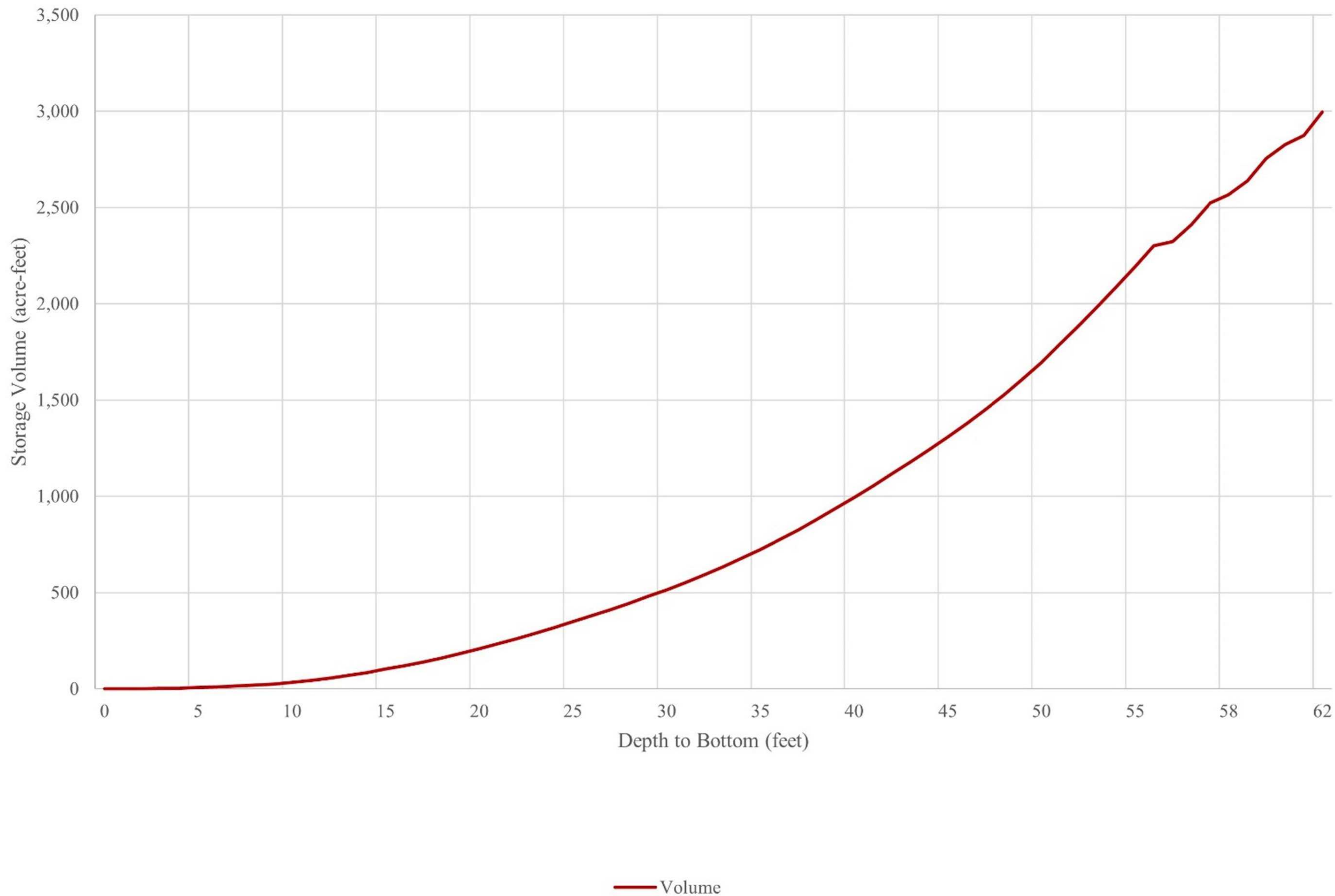
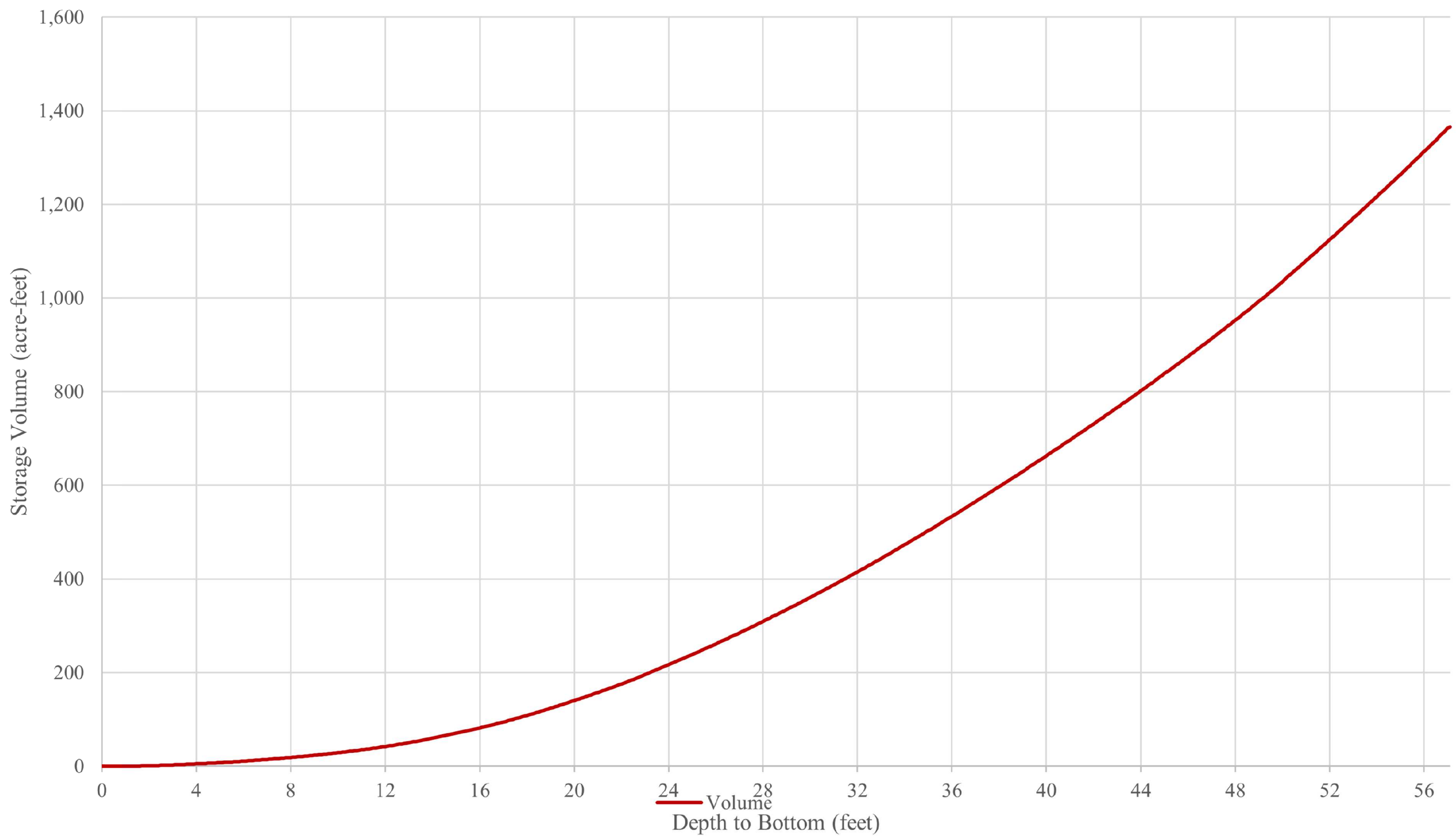


Figure 2-4: Chesbro Stage-Storage Curve



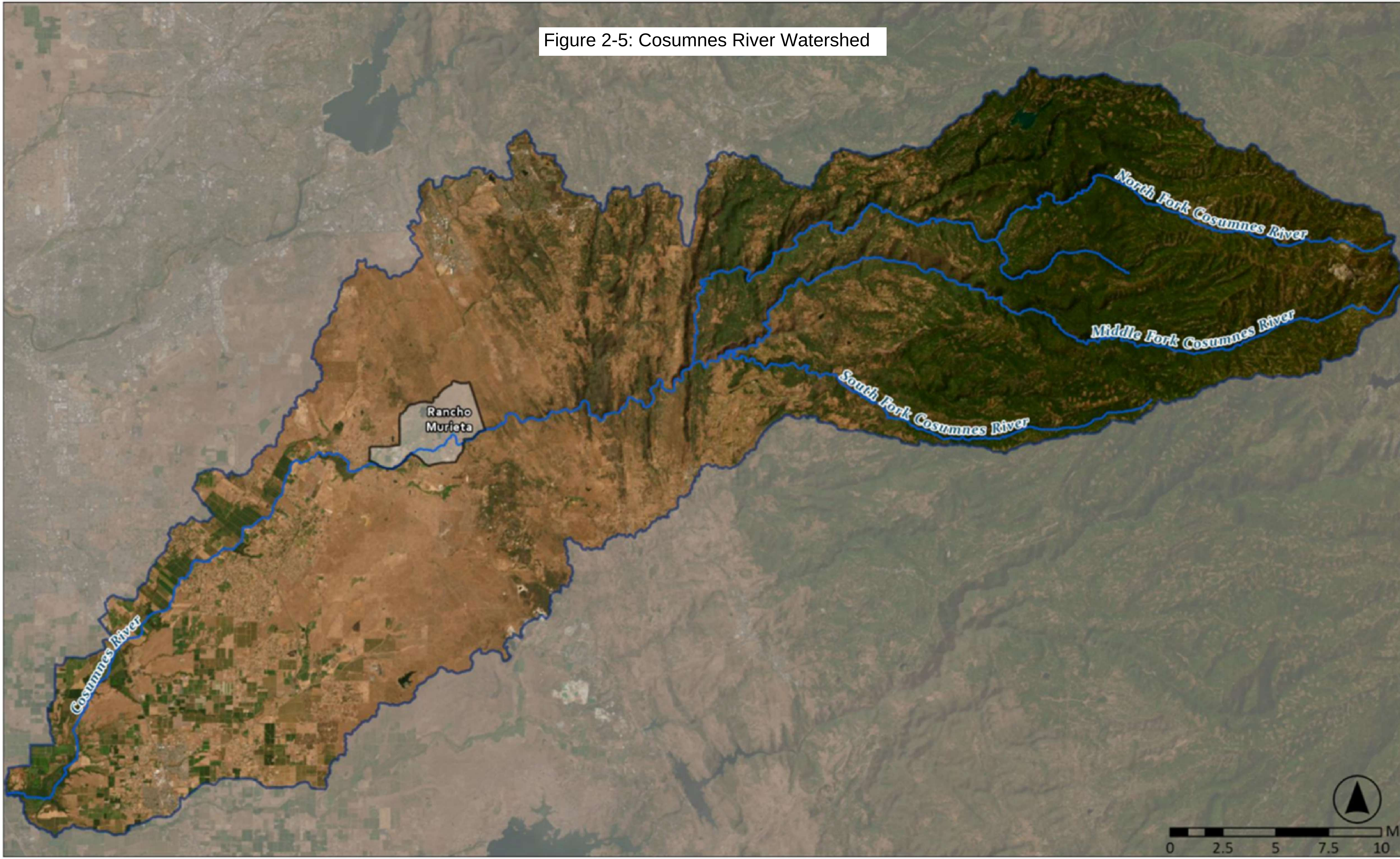


Figure 2-5: Cosumnes River Watershed
North Fork Cosumnes River
Middle Fork Cosumnes River
South Fork Cosumnes River
Rancho Murieta
Cosumnes River
Mi
0
2.5
5
7.5
10

Table 2-1: Raw Water Reservoir Capacities

Raw Water Storage Reservoir Capacity						
Reservoir	**Bottom Elevation (ft)[1]**	**Spillway Invert Elevation (ft)[1,3]**	**Top Flashboard Elevation (ft)[2]**	**Dead Storage (AF)[2]**	**Storage w/o Flashboards (AF)[1]**	**Storage w/ Flashboards (AF)[1]**
Calero	221.65	277.68	279.84	304	2,323.36	2,565.30
Chesbro	210.24	260.04	262.64	11	1,027.03	1,142.97
Clementia	162.00	182.50	184.50	50	907.10	1007

[1]Calero and Chesbro elevations and volumes verified by bathymetric survey. Clementia not verified by survey. Elevations are measured to the NGVD 1929 datum.
[2]Dead storage is unusable storage at the bottom of the reservoirs, below pumping or pipeline capabilities.
[3]Top elevation measured at top of road crest.

2-4.1.a. Calero Reservoir

Calero Reservoir stores raw water for drinking water production. This reservoir is bound by the 55-foot tall Calero Dam, constructed in 1982. Water is gravity fed or siphoned from Calero Reservoir into Chesbro Reservoir as needed for drinking water production via a 30-inch pipeline. Due to the active use of Calero Reservoir for drinking water production, no bodily contact or motorized boats are allowed. See Figure 2-6.

2-4.1.b. Chesbro Reservoir

Chesbro Reservoir stores raw water for drinking water production. The reservoir is bound by the 79 foot tall Chesbro Dam, constructed in 1972. Raw water needed to meet the community's needs is routed from Chesbro Reservoir to the WTP through a gravity-fed, 36-inch raw water supply pipeline. Aeration is used to keep the reservoir mixed and to oxidize iron and manganese. Due to the active use of Chesbro Reservoir for drinking water production, no bodily contact or motorized boats are allowed. See Figure 2-7.

2-4.1.a. Clementia Reservoir

Clementia Reservoir stores 907 AF of raw water. The reservoir is bound by the 33-foot tall Clementia Dam, constructed in 1976. A watershed of approximately 1,100 acres drains into Clementia.

In addition to raw water storage, Clementia Reservoir can be used to route water to several other areas within the community. Clementia Reservoir is also used for irrigation supply and recreational uses. Clementia Reservoir is a permitted source for domestic purposes under the District's water right, but the current drinking water permit does not allow it to be used as a source of public drinking water without first restricting body contact, as approved by the California Department of Public Health. See Figure 2-8.

Figure 2-6: Calero Reservoir

Figure 2-7: Chesbro Reservoir

Figure 2-8: Clementia Reservoir

2-4.2. Water Rights

Water right permit 16762 was issued in 1969 and has since been amended in 1980, 2000, and 2006. In 2006, the permit was renewed and extended with no new permit requirements through 2020 in consideration that the community was not at full buildout. The District requested another extension of the permit in 2020 as it had still not reached full buildout. The permit states the following:

- Between the dates of the allowable diversion period (November 1 and May 31), surface water can be diverted from the Cosumnes River at Granlees Dam into the District's water storage reservoirs.
- Diversions are limited as follows:
  1. No water may be diverted when river flows are less than 70 cfs at Michigan Bar gauging station.
  2. For river flows between 70 and 175 cfs, a maximum diversion rate of 6 cfs is allowed provided this diversion does not reduce downstream flow below 70 cfs.
  3. When river flows exceed 175 cfs, diversion of up to 46 cfs is allowed for direct use plus an additional 3,900 acre-ft for storage as follows:
     a. 1,250 acre-ft to Chesbro Reservoir.
     b. 2,610 acre-ft to Calero Reservoir.
     c. 850 acre-ft to Clementia Reservoir.
     d. 40 acre-ft to South Course Lake 10.
  4. The combined amount of items b, c, and d above cannot exceed 2,650 AFY
  5. The maximum allowable diversion rate to storage is 46 cfs.
  6. If at least 400 AF has not been diverted by February 1st, up to 46 cfs may be diverted during February if the river flow is above 70 cfs.

7. If on March 1st at least 2,000 AF has not been diverted; up to 46 cfs may be diverted during the month of March if the river flow is above 70 cfs.
8. If on April 1st at least 4,400 AF has not been diverted; up to 46 cfs may be diverted for the rest of the season if the river flow is above 70 cfs.
9. The equivalent of the continuous flow allowance by direct diversion for any 7-day period may be diverted in a shorter time if there is no interference with vested rights.
10. No water shall be diverted during the allowable period (November 1-May 31) except during such time as there is visible surface flow in the bed of the Cosumnes River from point of diversion to the McConnell gauging station at Highway 99.
11. The total amount of water taken from the river cannot exceed 6,368 AFY from October 1 to September 30.
12. Only water that originates from the river and is pumped into a reservoir can be used for municipal purposes, except for a small allowance for storm runoff into Calero and Chesbro reservoirs.

This permit authorizes the diversion to storage in all three reservoirs referenced above. The charts below show the volumetric historical diversion of water from the Cosumnes River to both the Calero/Chesbro Reservoir combination and Clementia Reservoir. A technical memorandum published in June 2023 by Wagner and Bonsignore summarizes the District's water rights and is attached as Appendix G.

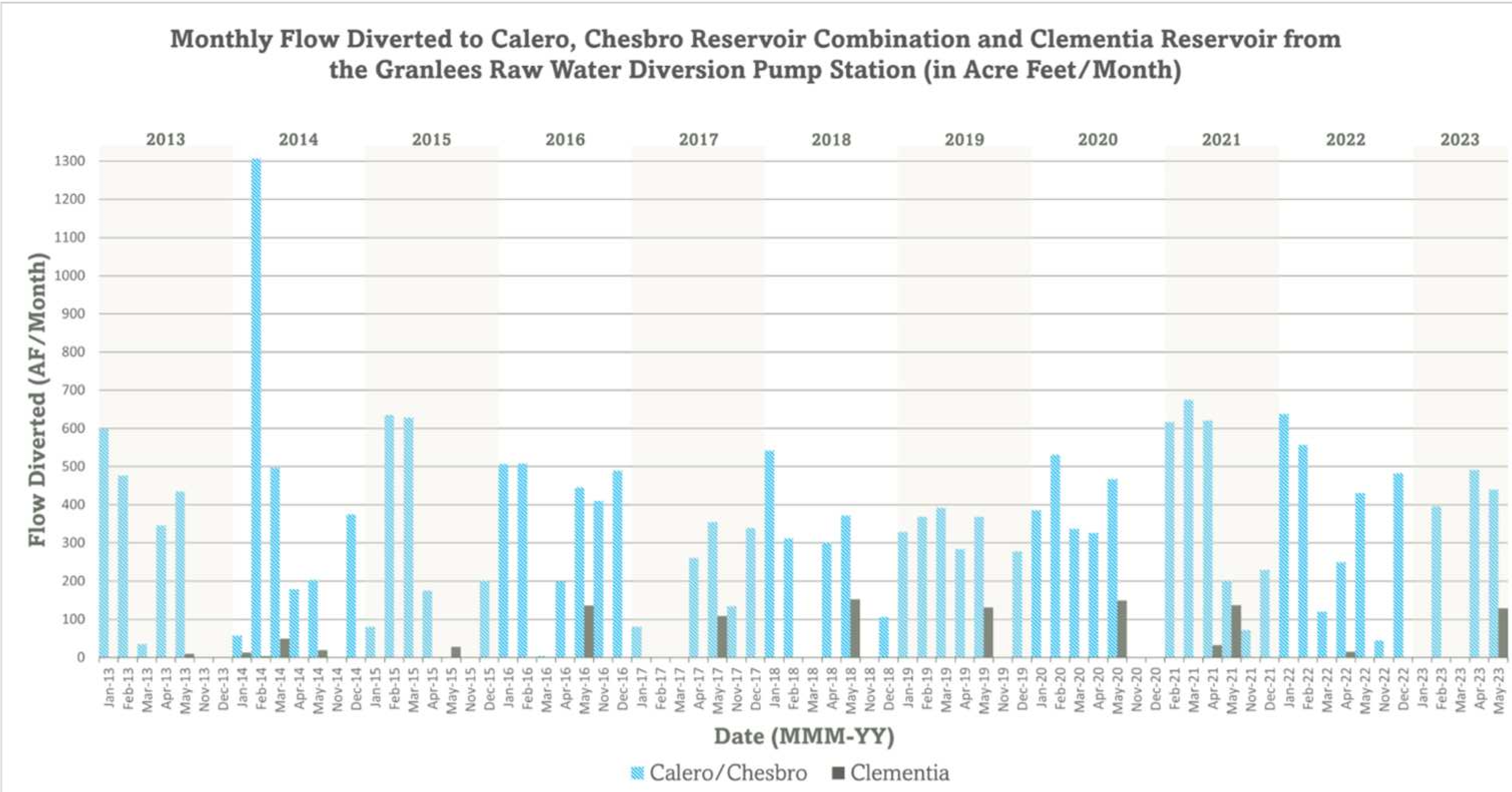


Figure 2-9: Historic Diversions in AF/month

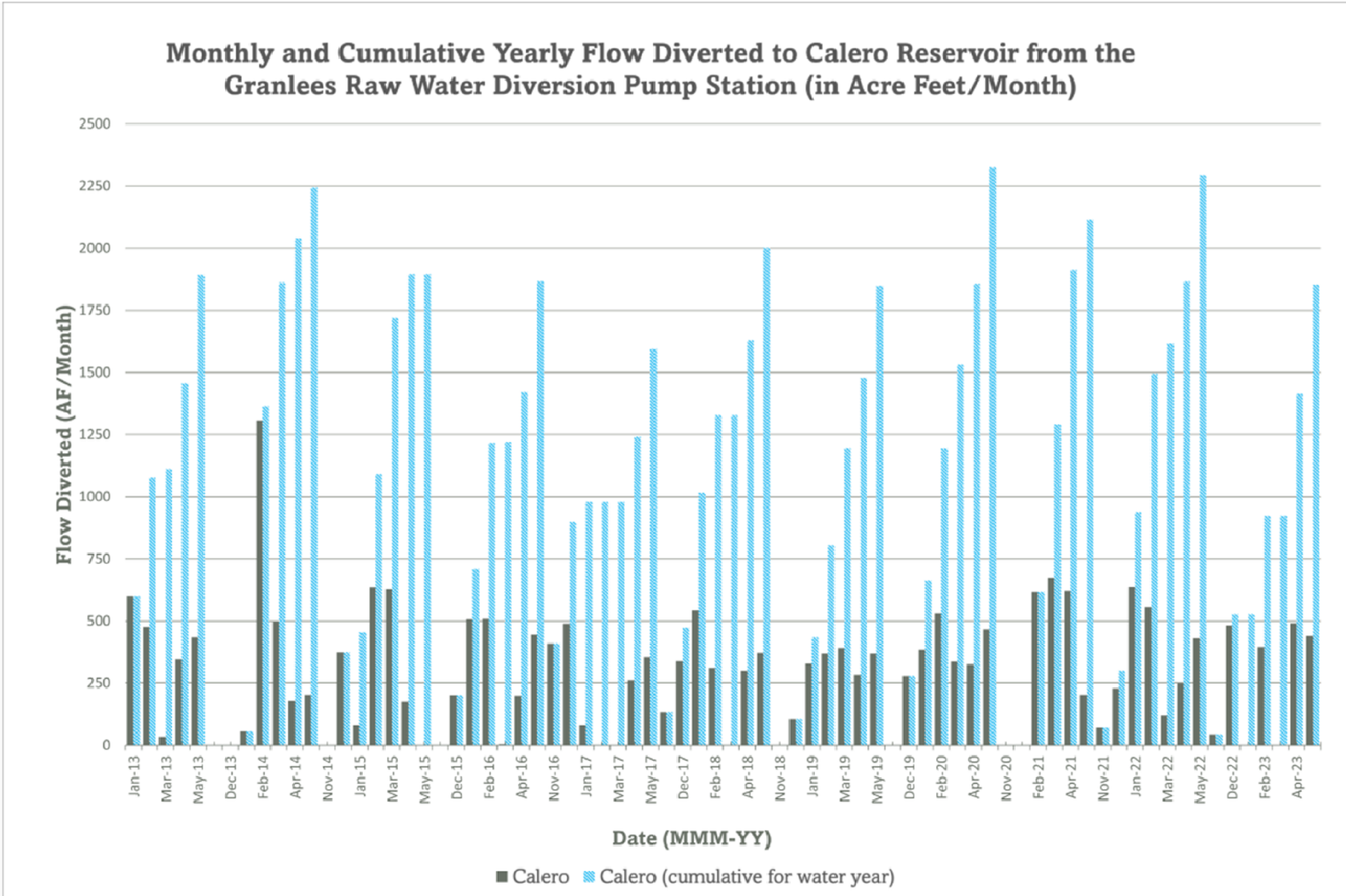


Figure 2-10: Historic Diversions to Calero

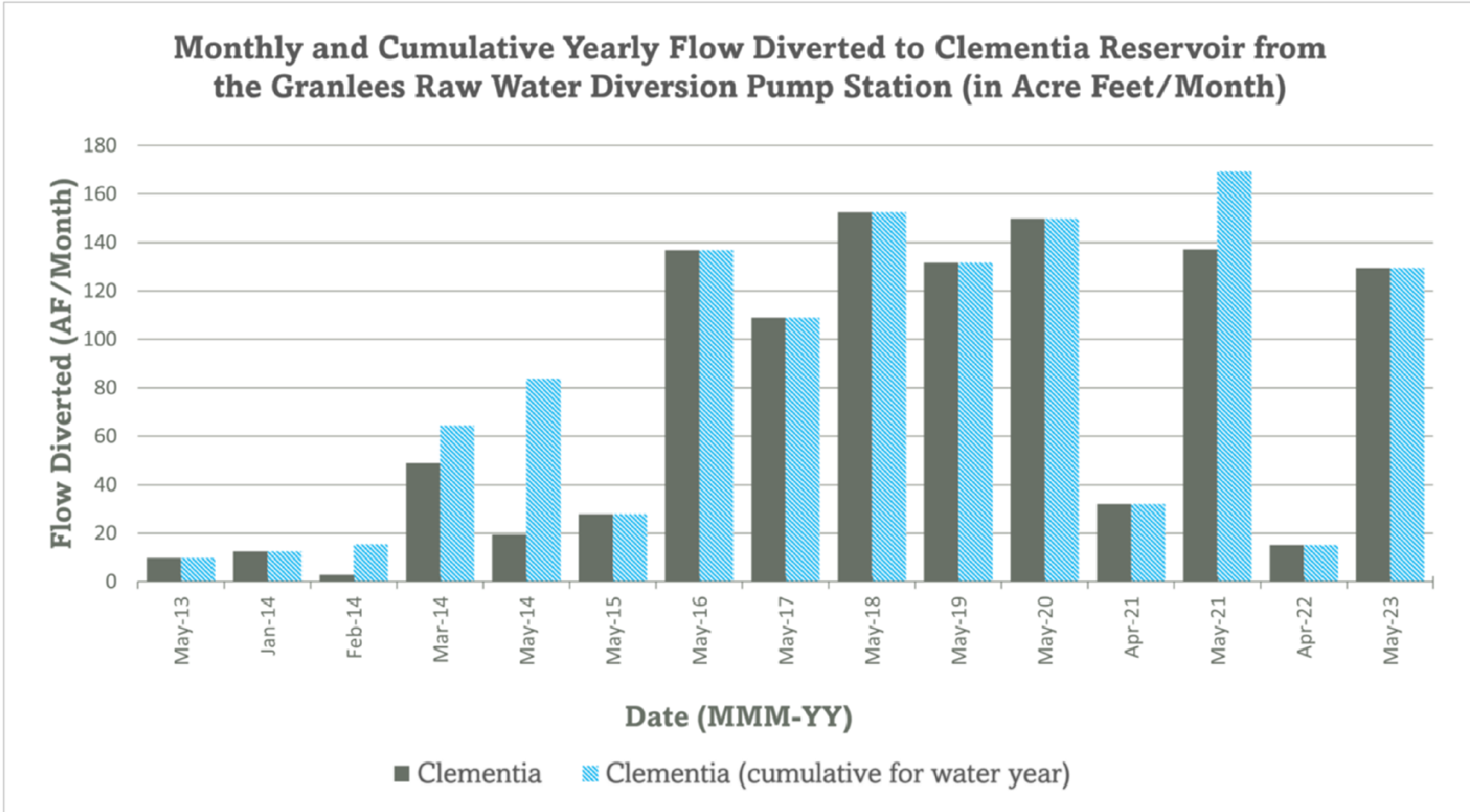


Figure 2-11: Historic Diversions to Clementia

2-4.3. Treatment

The WTP is divided into two plants based on treatment type: WTP1 is an ultra-filtration membrane treatment system with a 4.0 million gallon per day (MGD) capacity, and WTP2 is a traveling bridge filter treatment system with a 2.0 MGD capacity. Both plants disinfect via chlorine contact chambers and pump treated water to storage at the Rio Oso and Van Vleck tanks. See Figure 2-13 for a schematic of the existing water system. Figure 2-12 shows the transmission network from the raw water diversion through water treatment and to the treated water storage tanks.

In California, water is treated under the State Department of Health Services requirements as specified in Title 17 and Title 22 of the California Health and Safety Code and Chapter 7 of the California Safe Drinking Water Act. The State requires the District to periodically test the water and report the results to its customers.

WTP1 was constructed in 1975 with an original capacity of 1.5 MGD as a conventional treatment plant. In 2015, WTP1 was upgraded to its current 4.0 MGD capacity with ultra-filtration membrane treatment. It could be upgraded to 6.0 MGD capacity with the addition of more filters. WTP1 has a 10,960-gallon clearwell. Five pumps at this plant operate based on clearwell levels and pump water to the storage tanks Rio Oso and Van Vleck.

WTP2 was constructed in 1988 as a traveling bridge filter treatment plant with an original capacity of 2.0 MGD. In 1995, both plants were retrofitted to meet the new Surface Water Treatment Rules (SWTR). WTP2 has a 6,586-gallon clearwell; three pumps move water to the storage tanks using set points in the clearwell to govern operation.

RANCHO MURIETA EXISTING TRANSMISSION SYSTEM

700 0 350 700
(FEET)

SITE PLAN NOTES

1. NO TOPOGRAPHIC SURVEY WAS PERFORMED. ALL LOCATIONS OF UTILITIES SHOWN ARE APPROXIMATE. ACTUAL SITE CONDITIONS AND LOCATIONS MAY VARY.
2. THIS IS A SCHEMATIC SITE PLAN FOR THE SOLE PURPOSE OF ILLUSTRATING THE GENERAL COMPONENTS AND FACILITIES. ACTUAL SITE LAYOUT AND LOCATIONS MAY VARY AND WILL NEED TO BE DETERMINED BY TOPOGRAPHIC SURVEY AND OTHER SITE INVESTIGATIONS AS REQUIRED.

LEGEND

— — — w — — — w —	POTABLE WATER LINE
———————	RAW WATER LINE

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DISTRIBUTION AND STORAGE ALTERNATIVES
FOR
RANCHO MURIETA CSD
RAW WATER, TREATMENT, AND STORAGE

SCALE	SHOWN
DATE	08/08/2024
DRAWN BY	BAG
PROJ. NO.	3971-02

FIGURE
2-12

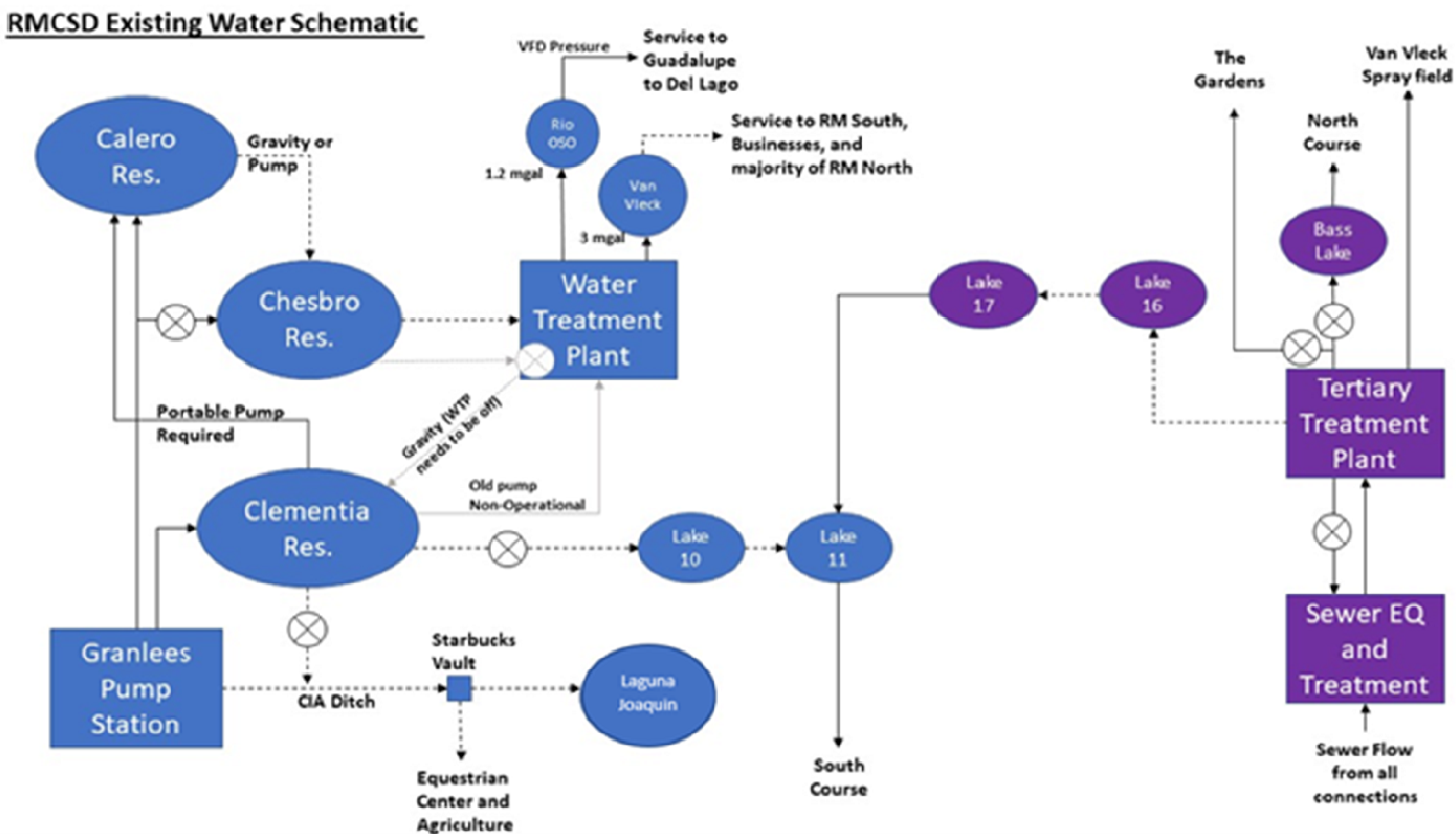

Figure 2-13: System Schematic

2-4.4. Storage

Two potable water storage tanks receive treated water from the WTP. These are Rio Oso and Van Vleck.

2-4.4.a. Rio Oso Tank

Rio Oso receives water from the WTP via a 14-inch pipeline and has a capacity of 1.2 million gallons (MG). It supplies water to the Rio Oso pressure zone, which accounts for approximately 25% of the total system demand. See Figure 2-14. The operational range for Rio Oso is currently 25 feet to 27 feet. Flows to Rio Oso are controlled by an altitude valve. When water levels fall below 25 feet in Rio Oso, this valve opens and allows water to flow through the 14” transmission pipe. When the WTP pumps are on, the water comes from the WTP. When the WTP pumps are off, water comes from Van Vleck through the same 14” transmission line. When water rises above 27 feet this valve closes. Two 125 HP pumps boost water from Rio Oso into the Rio Oso pressure zone. Pressures in the Rio Oso zone are relatively high, with hydrant tests showing upwards of 95 pounds per square inch (psi) across the pressure zone. Additionally, there is a gravity-fed pipeline that connects Rio Oso to the Van Vleck gravity zone. This pipeline is controlled by manual operation of a valve which opens and closes it. When the valve is open, Rio Oso can supplement Van Vleck’s storage capacity. The normal status of the valve was unknown at the time of this IWMP.

Figure 2-14: Rio Oso Tank

2-4.4.b. Van Vleck Tank

Van Vleck has its base at approximately 311 feet. Since this is higher than much of the district, it provides pressure to its zone via gravity. Van Vleck receives water through a 16-inch pipeline. See Figure 2-15. Water can flow to Van Vleck from the WTP through this pipeline, and water can also flow out of this pipeline to Rio Oso when the WTP pumps are off and Rio Oso's altitude valve is open. Van Vleck has a capacity of 3.0 MG and has no pumps. The operational range for Van Vleck is currently 25.5 feet to 27.5 feet, and this tank's operational range controls the operation of the WTP; when the water level falls below 25.5 feet in Van Vleck, the WTP turns "on" and when the water level rises above 27.5 feet in this tank, the WTP turns "off."

Figure 2-15: Van Vleck Tank

2-4.5. Distribution System

The existing distribution system consists of over 45 miles of treated water pipelines ranging from two inches to 20 inches in diameter. This information comes from the District's GIS system. Generally, the largest diameter pipelines in the treated water distribution system are transmission lines moving water from the WTP to the storage tanks, and the smaller diameter pipelines are for moving water from the storage tanks to water users across the District. These are summarized in Table 2-2 below and visualized in Figure 2-16.

The Rancho Murieta community was formed in 1982, and many of the community developments have occurred in phases. As such, some pipelines throughout the District are much older than others. Further, it is likely that existing pipe material varies based on when they were installed. This information was not available for review at the time of developing this report, so reasonable assumptions will be made about material, age, and design life for pipes that do not have reliable data.

Table 2-2: Distribution Pipeline Inventory

Distribution Pipeline Inventory	
Pipe Diameter (in)	**Total Length (LF)**
2	742
3	314
4	19,308
6	47,660
8	86,483
10	31,081
12	19,434
14	21,767
16	15,127
18	2,035
20	343

RANCHO MURIETA EXISTING DISTRIBUTION SYSTEM

700 0 350 700
(FEET)

SITE PLAN NOTES

1. NO TOPOGRAPHIC SURVEY WAS PERFORMED. ALL LOCATIONS OF UTILITIES SHOWN ARE APPROXIMATE. ACTUAL SITE CONDITIONS AND LOCATIONS MAY VARY.
2. THIS IS A SCHEMATIC SITE PLAN FOR THE SOLE PURPOSE OF ILLUSTRATING THE GENERAL COMPONENTS AND FACILITIES. ACTUAL SITE LAYOUT AND LOCATIONS MAY VARY AND WILL NEED TO BE DETERMINED BY TOPOGRAPHIC SURVEY AND OTHER SITE INVESTIGATIONS AS REQUIRED.

LEGEND

EXISTING WATER LINE

EXISTING FIRE HYDRANT

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DISTRIBUTION AND STORAGE ALTERNATIVES FOR RANCHO MURIETA CSD

EXISTING DISTRIBUTION SYSTEM

SCALE	SHOWN
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PROJ. NO.	3971-02

FIGURE 2-16

2-4.6. Reclaimed Water System

Reclaimed water is tertiary treated wastewater which is suitable for uses other than potable use. There are various types of reclaimed water depending on the source and level of treatment. In the District, reclaimed water is tertiary treated and used for irrigation to reduce potable water use. Tertiary treated water has been oxidized, filtered and disinfected to meet stringent criteria for reclaimed use and must satisfy CA Title 22 regulations related to reclaimed water. This water is also suitable for dual-plumbed residential irrigation use.

2-4.6.a. Raw Wastewater

The sources of raw wastewater for the WWRP are residential homes and commercial facilities (stores, restaurants, offices, etc.). There are no industrial users that discharge wastewater to the WWRP. Current influent flows are approximately 0.40 MGD, and projected flows at buildout are expected to be approximately 0.84 MGD based on the anticipated development. A detailed discussion of these projections is included in Chapter 4.

The wastewater generated at Rancho Murieta is a combination of domestic and commercial contributions. It is expected that future developments will continue to discharge domestic and commercial strength wastewater. The District's Sewer Code prohibits the discharge of toxic chemicals and other harmful compounds to the sewer. Residents and businesses routinely receive written materials describing substances that are prohibited from discharge into sewers for the protection of the wastewater treatment processes or cause the reclaimed water to be unsuitable for irrigation. See Figure 2-17 for a map of the existing reclaimed water system.

EXISTING RECLAIMED WATER INFRASTRUCTURE

SITE PLAN NOTES

1. NO TOPOGRAPHIC SURVEY WAS PERFORMED. ALL LOCATIONS OF UTILITIES SHOWN ARE APPROXIMATE. ACTUAL SITE CONDITIONS AND LOCATIONS MAY VARY.
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3. RANCHO MURIETA COUNTRY CLUB IS RESPONSIBLE FOR OPERATING AND MAINTAINING THE TRANSMISSION LINES BETWEEN THE WASTEWATER TREATMENT AND RECLAMATION PLANT AND THE NORTH AND SOUTH GOLF COURSES.
4. RANCHO MURIETA COUNTRY CLUB IS RESPONSIBLE FOR OPERATING AND MAINTAINING THE NORTH COURSE PUMP STATION, WHICH MOVES RECLAIMED WATER FROM THE WASTEWATER TREATMENT AND RECLAMATION PLANT TO THE NORTH COURSE.
5. THE TRANSMISSION LINE FROM THE WWRP TO LAKES 16/17 IS GRAVITY. THE RMCC OWNS AND OPERATES A PUMP STATION THAT LIFTS WATER FROM LAKES 16/17 TO LAKE 11.

LEGEND

EXISTING BUILDING EDGE

EXISTING RECLAIMED WATER LINE

P EXISTING RECLAIMED WATER PUMP

600 0 300 600
(FEET)

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RECLAIMED WATER ALTERNATIVES
FOR
RANCHO MURIETA CSD
EXISTING RECLAIMED DISTRIBUTION SYSTEM

SCALE	DATE	DRAWN BY	PROJ. NO.
SHOWN	08/08/2024	BAG	3971-02

FIGURE
2-17

2-4.6.b. *Wastewater Treatment*

The WWRP consists of both a secondary wastewater treatment facility and a tertiary treatment plant. The secondary treatment system is designed to treat an average annual flow of 1.55 MGD and a peak flow of 3.00 MGD in the series of five aerated facultative ponds. Seasonal storage of the secondary treated wastewater during the non-irrigation months is provided in two storage reservoirs, which have a combined storage capacity of approximately 238 MG or 728 AF with two feet of freeboard. The major components of the WWRP are as follows:

- Five aerated facultative ponds
- Two secondary storage reservoirs,
- Two dissolved air flotation (DAF) units
- Two sand filtration units
- Chlorine contact detention facilities
- Equalization (EQ) basin
- North Course Pump Station (NCPS)

Raw wastewater is pumped to the WWRP through three lift stations in the District. Raw wastewater enters the WWRP at Pond 1, which is equipped with aeration. The effluent from Pond 1 flows by gravity through the remaining ponds in sequential order. Ponds 2 and 3 each contain three aerators, Pond 4 has two aerators, and Pond 5 has one aerator. The aerators are managed by District operations staff that set the timers to maintain proper dissolved oxygen (DO) levels. There is one solar-powered mixer in each of the five treatment ponds, and the ponds are equipped with piping such that any pond can be bypassed while keeping the plant in operation. All ponds except Pond 1 can be drained completely for sludge removal and/or repairs. See Figure 2-18 for a layout map of the existing WWRP.

WASTEWATER TREATMENT AND RECLAMATION FACILITY

250 0 125 250
(FEET)

LEGEND

	EXISTING BUILDING EDGE
	EXISTING RECLAIMED WATER LINE
P	EXISTING RECLAIMED WATER PUMP

SITE PLAN NOTES

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RECLAIMED WATER ALTERNATIVES
FOR
RANCHO MURIETA CSD
EXISTING WWRP

SCALE SHOWN
DATE 08/08/2024
DRAWN BY BAG
PROJ. NO. 3971-02

FIGURE
2-18

The secondary effluent flows into two storage reservoirs, which store the secondary treated wastewater during the winter months when reclaimed water is not being produced. The storage reservoirs have a combined capacity of 728 AF, with two feet of freeboard. The reservoirs have 860 AF of capacity without freeboard.

The tertiary treatment system consists of a tertiary water pump station, coagulation, DAF units, gravity sand filters, chlorine contact tank, chlorine contact pipe, and an EQ basin. The capacity of the tertiary filtration facilities is 3.0 MGD. However, the operating capacity of the overall tertiary treatment process is 2.3 MGD due to the undersized existing chlorine contact basin. A new chlorine contact basin is currently being designed.

After tertiary treatment, the reclaimed water is stored in an EQ basin prior to conveyance to the use areas. This basin has a capacity of 1.8 MG. Approximately 6,600 LF of 20-inch plastic irrigation pipe (PIP) was installed in the EQ basin to provide additional chlorine contact time. This will be removed upon the completion of the new chlorine contact basin. Water leaving the chlorine contact pipe (CCP) is stored in the EQ basin before used for reclaimed water irrigation. From the EQ basin, reclaimed water is conveyed through both a 12-inch gravity pipeline to Lake 16 and Lake 17 to supply South Course and a pressurized 14-inch pipeline to the North Course by the NCPS.

2-4.6.c. Supplemental Supply

Currently, the WWRP does not produce enough reclaimed water to meet the full irrigation demands of the golf courses. Therefore, supplemental water must be provided to satisfy golf course irrigation demands. The reclaimed water system for the golf courses is currently supplemented with raw water from the Cosumnes River and from Clementia. RMCC’s river pumps divert water from the Cosumnes River to Bass Lake and Lake 10 where it is stored for future golf course irrigation in the spring. On average, reclaimed water production is estimated to be 468 AFY, and average golf course demands are 673 AFY. It is important to note that the District’s current obligation to provide the golf courses with reclaimed water for irrigation is 550 AFY.

No residences use reclaimed water yet, but Murieta Gardens has reclaimed water infrastructure in place. For future developments, Villages A, B, and C, the Retreats, and new commercial developments in Murieta Gardens are planned to receive reclaimed water. It is likely that the new developments that have dual plumbing installed for reclaimed water will require supplementation with potable water to meet demands. Most likely, potable water supplementation will occur at the EQ basin located at the WWRP. This is discussed in Chapter 6.

2-4.6.d. Transmission and Distribution

Based on construction drawings, it appears that a minimum 10-foot separation has been maintained between reclaimed water and potable water pipelines. For example, there are three pipelines on the Yellow Bridge: sewer, potable water, and reclaimed water. The sewer and reclaimed water pipelines are mounted on one side of the bridge, with the potable water on the other side. The District, in association with the RMCC, has developed, submitted, and gained Regional Water Quality Control Board (RWQCB) approval of an operations manual describing the delivery and use of reclaimed water at the North and South Golf Courses (May 2010).

2-4.6.d.i. Golf Courses

The reclaimed water transmission and distribution systems associated with the two golf courses were installed in 1983. The NCPS pumps reclaimed water from the EQ basin to Bass Lake. This pump station consists of two vertical turbine pumps, each of which have 100 HP motors capable of delivering 1062 gpm at 323 feet of head. Reclaimed water is conveyed through a 12-inch asbestos cement pipe (ACP) from the WWRP, under Highway 16, over the foot bridge (Yellow Bridge), to the 10th hole of the North Golf Course. From this point, the pipeline is reduced to an 8-inch ACP and runs east along the golf course fairways to Bass Lake. Reclaimed water is also conveyed from the WWRP to Lake 16 of the South Golf Course by gravity through another 12-inch ACP pipeline. The water is pumped from Lake 16 to Lake 11 by a RMCC-owned pump station to supply the South Course. The RMCC is responsible for maintaining the reclaimed water transmission and distribution systems, including pumps, pipelines, and irrigation ponds.

2-4.6.d.ii. Van Vleck Ranch

Approximately 1,800 LF of above-ground 12- and 8-inch Certa-Lok™ PVC irrigation pipe is used to convey reclaimed water to the Van Vleck Ranch boundary and about 4,050 LF of above-ground 8-, 6-, 4-, and 3-inch Certa-Lok™ PVC irrigation pipe is used to convey reclaimed water to three spray irrigation systems. The 12- and 8-inch PVC pipeline was installed in 2007 and is owned and operated by the District with the words "RECYCLED WATER/RECLAIMED WATER" stenciled on top.

The distribution system consists of approximately 29 strings of K-line irrigation systems, which are in turn composed of movable sprinklers and 40 mm HDPE piping. Each movable sprinkler is housed within a plastic pod. The connecting piping is flexible, and the entire string of sprinklers can be moved from spray field to spray field.

The District has developed, submitted, and gained RWQCB approval of an operations and management plan describing the delivery and use of reclaimed water at the Van Vleck Ranch (August 2007). The District will continue to use the existing above-ground 12- and 8-inch Certa-Lok™ PVC pipeline in the future to serve the existing and proposed spray fields as described later in this report. The Van Vleck Ranch includes approximately 96 acres of land that can receive reclaimed water, and it is permitted to receive 215 AFY.

2-4.6.d.iii. Murieta Gardens

Murieta Gardens is a mixed-use development in Rancho Murieta, just south of Highway 16, constructed between 2017 and 2020. Approximately 36.5 acres are commercial developments, including the Murieta Inn and Spa, and 16.4 acres includes 78 single-family residences. Murieta Gardens has reclaimed water pipelines in place. Each residence has dual-plumbed irrigation systems, with reclaimed water infrastructure marked by purple coloring.

This development includes 12" pipelines that tee into the existing North Course transmission line and cross beneath Jackson Road. These 12" lines travel along the north side of Legacy Lane before terminating near the Murieta Inn and Spa. Several 6" lines branch from this mainline to serve each residence. An 8" line terminates near the intersection of Murieta Drive and Cantova

Way. Currently, there are no plans to serve the existing mobile home park in this area, but this line may be considered an ideal connection point at some time in the future should the District decide to provide reclaimed water in this area. See Figure 2-17 for a map of the existing reclaimed water distribution system.

2-4.6.e. Reclaimed Water Users' Responsibility

The District and the landowners of the RMCC golf courses entered into the *Agreement for the Use of Reclaimed Wastewater* (dated May 17, 1988) and an *Amendment to Agreement for the Use of Reclaimed Wastewater* (dated May 4, 1994). These agreements, as modified by the Waste Discharge Requirements 5-01-124 issued by the Regional Board for the use of reclaimed water at Rancho Murieta, set forth the operating principles and the respective responsibilities of the District and RMCC for the use of reclaimed water on the golf courses. In general, the District is responsible for the operation and maintenance of the collection system, wastewater and tertiary treatment facilities, whereas the RMCC is responsible for the operation and maintenance of the golf course irrigation systems, including transmission pipelines from the WWRP to RMCC facilities and irrigation storage ponds (e.g., Bass Lake and Lakes 10, 11, 16, and 17).

For new commercial and residential reclaimed water connections, additional responsibilities are required and are defined in the Reclaimed Water Standards (RMCSD, October 2013). These include:

- Obtaining all permits and payment of all fees required for the establishment, operation and maintenance of the User's reclaimed water system.
- Ensuring that all materials used during the design, construction and maintenance of the system are approved or recommended for reclaimed water use.
- Routinely monitoring and inspecting the reclaimed water system for any situation that may not be in conformance with the regulatory requirements. Problems such as irrigation controller malfunctions, irrigation schedule adjustments, excessive ponding or runoff of reclaimed water, broken or out-of-adjustment sprinkler heads, etc. must be corrected as soon as they become apparent.
- Maintaining the Use Area's reclaimed water system downstream of the Point of Connection.
- Reporting all violations and emergencies to the required local governing agencies.

- Obtaining prior written authorization from the District and any required regulatory agency before making any modifications to an approved reclaimed water system, or the potable water system if it is in close proximity to the reclaimed water system.

In addition to and in accordance with their easement agreement and WDR R5-2007-0109, the District manages the treatment, distribution, and use of reclaimed water at the Van Vleck Ranch for pasture irrigation. The use of reclaimed water at the Van Vleck Ranch is coordinated by the District with the Van Vleck Ranch manager to allow for movement of the K-line irrigation lines to accommodate periodic grass cutting and cattle rotation.

CHAPTER 3. Performance Objectives

3-1. Demographics, Timeframe, and Regulations

3-1.1. Planning Period

The planning period for the development of alternatives described herein is 20 years. While the exact development schedule is unknown at this time, it is expected that the District will reach buildout conditions before 20 years have passed. Currently, the District expects 4,102 total connections at buildout, with 3,991 being residential and 111 being commercial.

This plan should be revisited for an update after the following:

- The development assumptions listed in this report change such that the analysis in this report is affected.
- A weather station capable of measuring evapotranspiration and evaporation is installed near one of the raw water reservoirs and a seepage study is conducted. This will allow the District to update its water balance with better data which will affect the results of this report.
- The District collects several years of transducer water level data from the raw water reservoirs. This will also allow for a more precise water balance in conjunction with the new weather station and seepage study.

3-1.2. Regulatory Requirements

3-1.2.a. Water Planning Requirements

California does not require public water suppliers to maintain an active water master plan by law. However, California Water Code sections 10610-10656 and section 10608 require every urban water supplier that provides over 3,000 AFY or serves more than 3,000 urban connections to submit and maintain an UWMP. An UWMP involves the following:

- Assessing the reliability of water sources over a 20-year planning time frame
- Describing demand management measures and water shortage contingency plans

- Reporting progress toward meeting state-targeted 20% reduction in per-capita urban water consumption
- Discussion of the use and planned use of reclaimed water

The California Department of Water Resources (DWR) has published a guidebook with the detailed requirements of a UWMP and guidance for urban water suppliers who are developing a UWMP. This guidebook is available on the DWR website.

The District does not yet fall under the criteria that would make them an urban water supplier; they currently provide approximately 1,716 AFY to 2,729 connections, neither of which are above the 3,000 AFY or 3,000 connection thresholds. However, the District desired to complete an UWMP simultaneously with this IWMP since the efforts would use large amounts of the same data and analysis. However, the level of effort to complete the UWMP was in excess of the funds available to the District at the time of this IWMP. Further, since the District is not yet an urban water supplier, the UWMP was not essential at this time. However, with the completion of this IWMP, the District is very close to being able to complete an UWMP using the information from this IWMP. At the time when the District needs to complete a UWMP, this IWMP document, along with any new data or updated assumptions, will be critical to developing an UWMP.

3-1.2.b. Domestic Water Regulatory Requirements

Potable water quality in California is regulated by three sets of rules: The California Water Code, the California Health and Safety Code, and the California Code of Regulations (CCR). The Water Code and the Health and Safety Code are passed by the state legislature, and the CCR is established by state agencies rather than by legislation.

The regulations are extensive, so only those regulations that are discussed in this report are included in this section.

- CCR Title 22, Division 4, Chapter 15 § 64431 states the maximum contaminant levels (MCLs) for inorganic chemicals. This includes arsenic less than 0.01 mg/l.

- CCR Title 22, Division 4, Chapter 16 § 64585 (b)(4) states that distribution reservoirs must be equipped with at least one separate inlet and outlet (internal or external), and designed to minimize short-circuiting and stagnation of the water flow through the reservoir.
- CCR Title 22, Division 4, Chapter 16 § 64560 states that wells must be constructed in accordance with the community water system well requirements in California Department of Water Resources Bulletins 74-81 and 74- 90, which state that wells must be above the 100-year floodplain, and that if they are within the 100-year floodplain, they must be built up to avoid flooding.

3-1.2.c. Reclaimed Water Regulatory Requirements

Title 22 of the CCR (Water Recycled Criteria) sets the criteria for "disinfected tertiary reclaimed water." This designation allows for unrestricted use of reclaimed water for irrigation, which encompasses the current and proposed uses for reclaimed water at Rancho Murieta. The criteria are as follows:

- Contact time (CT) (the product of total chlorine residual and modal contact time measured at the same point) must be at least 450 milligram-minutes per liter at all times with a model contact time of at least 90 minutes.
- Coliform bacteria must not exceed:
  - Most probable number (MPN) of 2.2 per 100 mL (7-day median),
  - MPN of 23 per 100 mL (one sample in 30 days), and
  - Never exceed an MPN of 240 per 100 mL.
- Turbidity of filtered tertiary water must not exceed:
  - 2 Nephelometric turbidity units (NTU) (average),
  - 5 NTU (up to 5% of the time), and
  - Never exceed 10 NTU.

The District's reclaimed water meets all these criteria. Additionally, the District has a Title 22 Engineering Report published in December 2013 by AECOM. This report details how the District will meet state requirements for reclaimed water with its system.

The District adopted Reclaimed Water Standards (October 16, 2013) and the Reclaimed Water Code (January 18, 2012). District Code, Chapter 17 (Reclaimed Water Code) sets forth rules and regulations regarding the use of reclaimed water in the District. The Reclaimed Water Standards define District procedures, design, work, materials, capacities, facilities and other improvements pertaining to reclaimed water facilities or connections.

Together the Reclaimed Water Code and Reclaimed Water Standards establish and provide the means to enforce rules and regulations for reclaimed water users, design and construction of reclaimed water facilities, and the use of reclaimed water in accordance with federal and state reclamation criteria.

3-1.2.c.i. Monitoring and Reporting

The District currently monitors and reports in accordance with the requirements specified in Monitoring and Reporting Program Nos. 5-01-124 and R5-2007-0109-01, which were adopted by the Regional Board on December 1, 2006 and August 2, 2007, respectively. The water quality monitoring includes influent, secondary effluent, and tertiary effluent. In addition, the monitoring and reporting program includes monitoring of the treatment ponds, secondary storage reservoirs, golf course irrigation lakes, and reclaimed water use areas. It is anticipated that the monitoring and reporting requirements associated with the future reclaimed water uses would mirror those required for either the golf courses or the Van Vleck spray field.

The District operates a laboratory on site and performs some of the water quality analyses listed above, including chlorine residual, settleable solids, and turbidity. On-line continuous monitoring is conducted for flow, turbidity, and reclaimed water chlorine residual. The instrumentation used to perform this monitoring is calibrated regularly in accordance with manufacturer's specifications and recommendations. An Environmental Laboratory Accreditation Program (ELAP) Certified Laboratory, utilizing US EPA protocols and methods, performs all other required sample analyses.

3-1.3. *Service Population*

As of December 2022, the District served 2,629 residential connections and 100 non-residential connections, which include parks, commercial, and miscellaneous public uses. Local parks are currently being irrigated with potable water. According to Sacramento County's approved Planned Unit Development Plan at Buildout, the development of the District's service area represents roughly 5,189 residential units, though development plans for buildout estimate 3,991 residential units.

Existing population size was determined using data from the United States Census Bureau from 2020. This resulted in an existing population of 6,939 people. Buildout population size was estimated using developer estimates of total lots and lot types. Existing lot types were derived from District billing data. Lots smaller than 12,000 sf were assumed to have 2.36 persons per lot, lots larger than 12,000 sf were assumed to have 3.36 persons per lot, and ADUs were assumed to have 1.5 persons per unit. Using these lot occupancy estimates and the developer estimates of new lots resulted in a population of 10,492 people at buildout. Details of the methodology used to create person-per-lot estimates are included in Chapter 4.

3-2. Performance Objectives by Component

The design criteria shown below in 1 were developed in coordination with District staff and were used to evaluate the existing system and propose alternatives. Discussion of the system's ability to meet these criteria is included in Chapter 5.

Table 3-1: Summary of Performance Objectives

Summary of Performance Objectives	
Component	**Description**
Water Supply	- Able to provide adequate supply to meet buildout demands during historic drought - Must meet SB552 requirements for supply redundancy
Water Treatment	- Capacity must be greater than maximum day demand
Treated Water Storage	- Emergency storage = 1.75 times ADD - Fire storage = 4 hours @ 2,625 gpm = 630,000 gallons[1] - Operational storage = 2 feet in each reservoir[2] - Equalization storage = 4 times PHF – available supply - Each pressure zone able to provide its own required storage
Distribution System	- Pressure o Greater than 30 psi at peak hour on peak day o Greater than 20 psi at all times o Less than 105 psi at all times - Velocity less than 5 fps for normal conditions, less than 7 fps for fire flows - 8" minimum diameter for all pipelines that carry fire flows
Fire Protection	- Minimum 2625 gpm for 4 hours required at Murieta Inn - Minimum 1500 gpm for 2 hours required at all hydrants - All structures within 250 feet of a hydrant
Reclaimed System	- Secondary treated storage: able to store 0.84 MGD ADWF with 100-year high precipitation during non-irrigation season - Tertiary treatment & disinfection: 3.0 MGD - EQ storage: Max day irrigation demand minus tertiary production capacity - Pumping capacity greater than peak instantaneous irrigation demand - Pipe pressure/velocity: greater than 20 psi, less than 120 psi, less than 7 fps
	[1]Fire flow required for Murieta Inn, per its design planset. [2]Per District operations staff

CHAPTER 4.   Water Demands

Water use planning is an essential component of a thriving community. It is similar to creating a budget for the water that can be used now and in the future to meet a community's needs. Evaluating past water demand data for usage trends and forecasting future water demands is a necessary component of accurate planning. Without an understanding of both existing and future water needs, it is difficult to create a water budget that can withstand either predicted or unanticipated events.

Similar to how individuals establish a "rainy day" fund to prepare for unexpected expenses, robust water plans consider "dry day" funds for drought scenarios and incorporate climate change and population growth expectations (among other factors). Budgeting water supplies requires predicting, as close as possible, how much the water demands of the community will change in the future so that the agency can responsibly and effectively administer the water budget of today. It is important to carefully plan for future demands without under or over-sizing the system, as oversized water storage and distribution systems are expensive to construct and operate, and undersized systems may not reliably meet customer demands when events like drought and fire occur. There are standard engineering practices like the *AWWA Manual of Practice, M50, Water Resources Planning,* that outline the proper approaches and methods to assist with planning for future community water needs. The following descriptions outline the background of approaches and methods used to assess historic, existing, and future District demands.

4-1.   Historic Demands

Figure 4-1 below shows the annual billed consumption for the District (labeled "Water Use") in AF, active accounts, and rainfall between 1994 and 2022. Usage in this figure does not include system losses, which are discussed in a following section. The figure shows a decrease in billed consumption coinciding with the drought of 2013-2016 and a slight decrease in consumption coinciding with the COVID-19 pandemic.

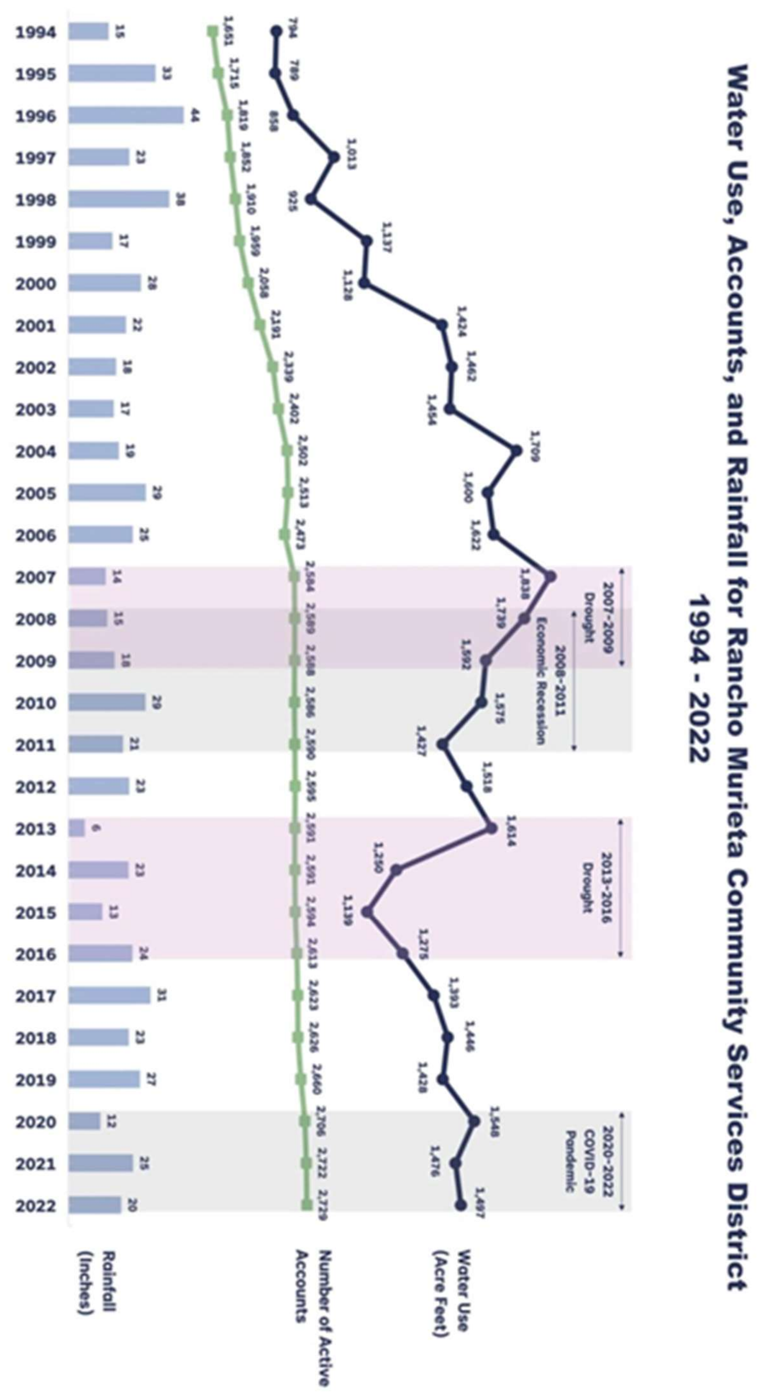


Figure 4-1: Historic Water Use (1994-2022)

4-1.1. Historic District Performance on Benchmarks

The California Water Conservation Act of 2009 (also referred to as Senate Bill X7-7) was enacted in November of 2009 and required urban retail water suppliers to develop water use targets that would achieve a 20% reduction in water use by December 31, 2020 (California Department of Water Resources - SB X7-7). Urban retail water suppliers are water suppliers that provide potable municipal water to more than 3,000 connections or provide more than 3,000 AFY.

As the District has not reached 3,000 connections, it is not yet required to comply with water use reduction targets under state law. The system had 2,729 active connections as of December 31, 2022. However, as the District expects to grow enough to be required to comply in the future, it has voluntarily implemented proactive plans in monitoring and tracking water use. The District has chosen to comply with state regulations applicable to larger systems and all permitted systems subject to the "beneficial use doctrine," to promote the efficient use of supplies to meet demands.

To monitor water use targets, the District developed the 2020 Compliance Plan, which determined that water use would have to be at or below 238.5 gallons-per-capita-per-day (gpcd) by 2020 to voluntarily comply with state regulations. The District adopted Policy 2011-06 which directed District staff to implement an efficiency program to help residents and businesses meet the state targets.

Part of the motivation behind the 2020 Compliance Plan and the District's proactive management of water use was to demonstrate good water management practices to support the District's application for a water right license. The District currently holds water right Permit #16762 which allows the District to use water as it continues development projects. Once buildout is completed, the State Water Board will determine how much water was used "beneficially" by the District and will issue a water right license. A water right license is a vested right that confirms actual water use and is awarded for the amount of water that has been reasonably and beneficially used by a community, up to the amounts listed in the permit.

Figure 4-2 below shows total water production, residential water use, and the 2020 Compliance Plan target for water use in gpcd. The figure shows that the District has successfully reached residential water use near or below the 2020 Compliance Plan target in recent years. The District was not required to comply with state mandated water use levels, but its proactive goal setting and achievement help make the case for more reliable water rights in the future. Additionally, higher water use efficiency will help to ensure more sustainable water supplies.

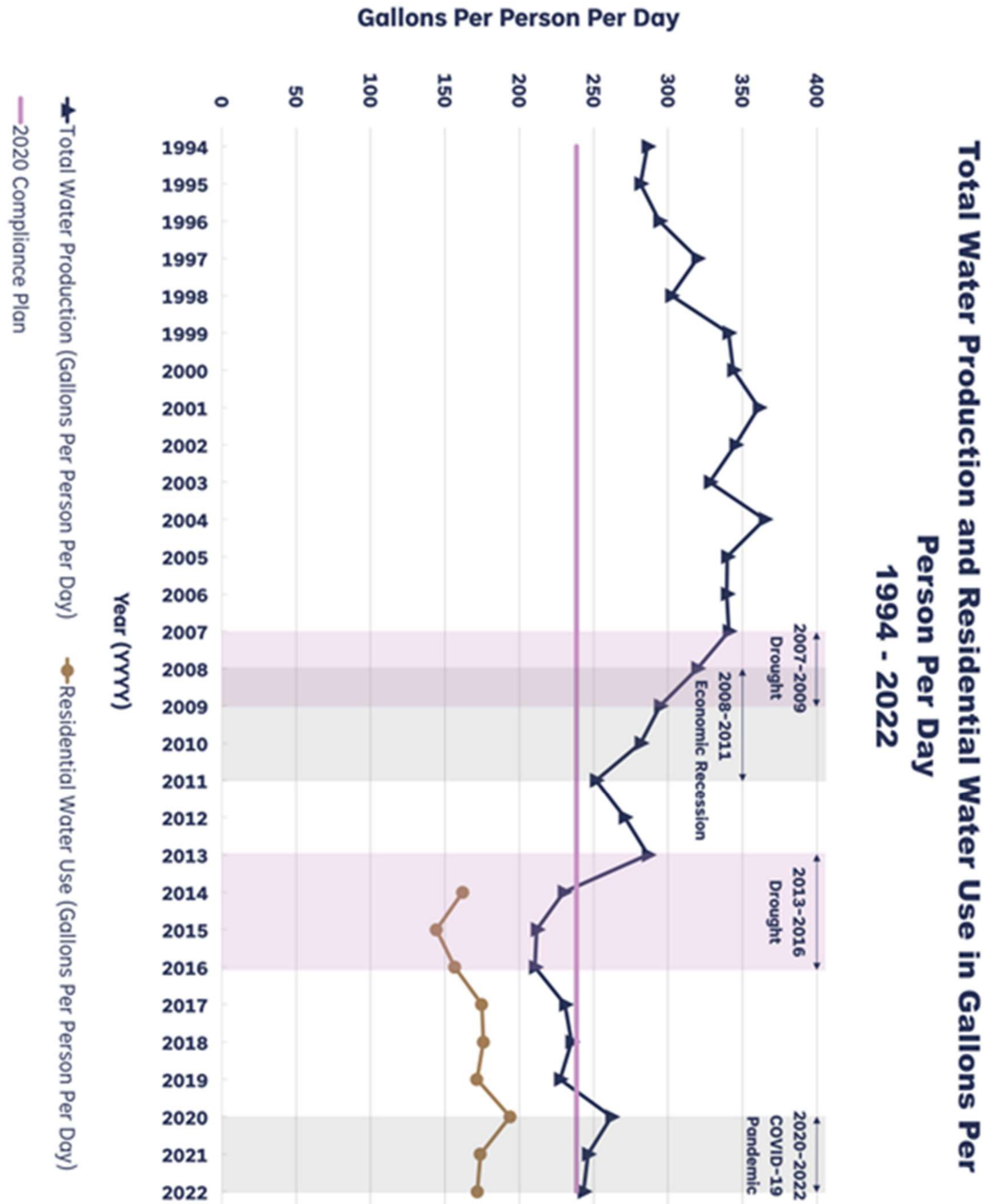


Figure 4-2: Total Production and Residential Use (1994-2022)

4-2. Existing Demands

The project team evaluated available datasets and developed an approach with District input to estimate existing water demands using an analysis of historic billing and production data between 2003 and 2022. Billed consumption data was aggregated to the address level, then grouped by lot type to generate an average gallons-per-day-per-account (gpda) by lot type by year.

Several ranges of years of data were explored alongside weather data and local knowledge to determine a year (or range of years) that would best represent average demand conditions. The 2020-2022 time period was selected for the existing demand input, as it reflects pandemic-related increases to indoor demands, is reflective of average (non-drought) weather conditions, and includes new accounts added during the last three years. The gpda by lot type was then applied to the number of lots with active billing as of 12/31/2022 to generate the existing demands. The average gpda by lot type, number of lots, and the existing demands are shown in Table 4-1 below.

Demands were divided into estimated indoor/outdoor demands based on the persons-per-household (pph) of 2.36 as reported by 2020 U.S. Census data and the observed indoor demands during winter months of 43.08 gpcd. One pph was added to estate lots larger than 12,000 sf, based on observations of higher indoor usage and the assumption that larger homes/lots are likely to contain larger households.

The technique used to estimate indoor demands is known as the minimum-month-method and was applied to all residential lot types as well as miscellaneous public uses lot types. The indoor/outdoor splits for commercial lot types were calculated based on a comparison of commercial irrigation volumes to total commercial volumes, since the commercial irrigation accounts are solely associated with outdoor use.

Additional consideration was given to system losses, which can be calculated as the difference between the production volume and consumption volume (as discussed in greater detail earlier in the following section). The observed system losses, also referred to as Non-

Revenue Water (NRW), averaged 12% from 2020-2022. 12% was added to the total estimated demand to account for system losses. NRW calculations and volumes are shown in Table 4-1 below.

Table 4-1: Existing Demands

Existing Demands by Lot Type								
Lot Type / User Class	**# of Acct**	**PPH**	**GPDA**	**In. GPDA**	**Out. GPDA**	**In. AFY**	**Out. AFY**	**Total AFY**
Residential								
Estate > 12,000 sf[1]	729	3.36	612	145	467	118	382	500
Estate < 12,000 sf[2]	577	2.36	398	102	296	66	192	257
Halfplex[2]	59	2.36	266	102	164	7	11	18
Circle[2]	454	2.36	486	102	384	52	195	8
Cottage[2]	292	2.36	369	102	267	33	87	248
Townhouses & Villas[2]	258	2.36	141	102	39	29	11	121
Murieta Village[2]	181	2.36	124	102	23	21	5	41
Murieta Gardens[2]	78	2.36	225	102	123	9	11	25
Van Vleck Ranch[2]	1	2.36	6,831	102	6,729	0.1	8	20
Residential Subtotal	**2,629**					**335**	**901**	**1,236**
Non-Residential								
Commercial (including commercial irrigation)	81	N/A	2,212	1,149	1,063	104	96	201
Park	5	N/A	7,849	-	7,849	-	44	44
Misc CSD Uses	14	N/A	1,872	712	1,160	11	18	19
Non-Residential Subtotal	**100**					**116**	**159**	**274**
Non-Revenue Water, estimated to be 12% (NRW)						61	145	206
Total Baseline Demands (with NRW)						**512**	**1,204**	**1,716**

[1]Assumed that these lots have 3.36 PPH

[2]Assumed that these lots have 2.36 PPH

[3]PPH multiplied by 43.08 indoor gpcd

Figure 4-3 shows percent of total demand by lot type. The details, demand factors, lot counts, and estimated demands for each residential lot type are shown in Figure 4-4 through Figure 4-10. In these images, parcel boundaries were obtained from the Sacramento County Assessor's office and enhanced to include additional data.

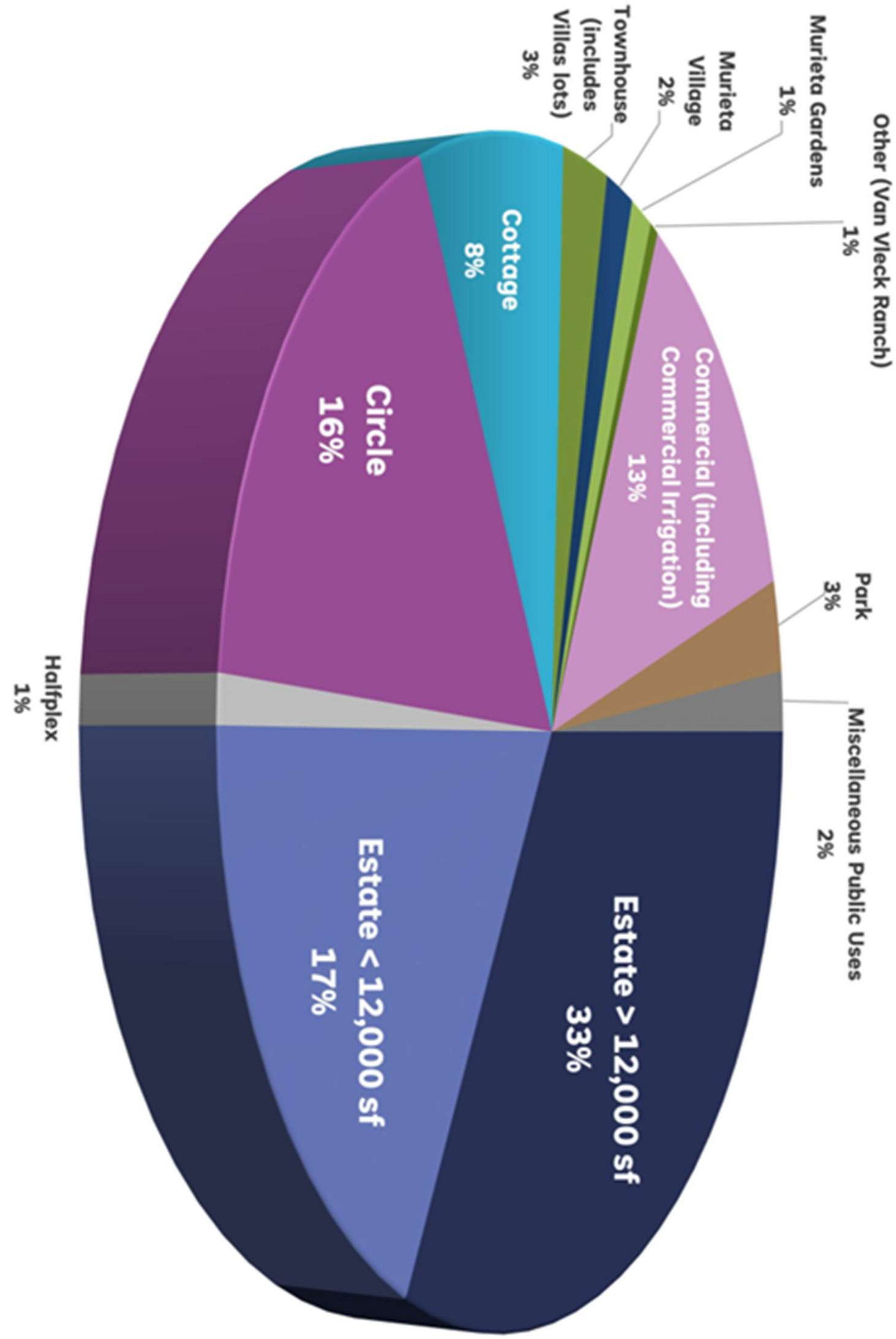


Figure 4-3: Percent of Total Demand by Lot Type

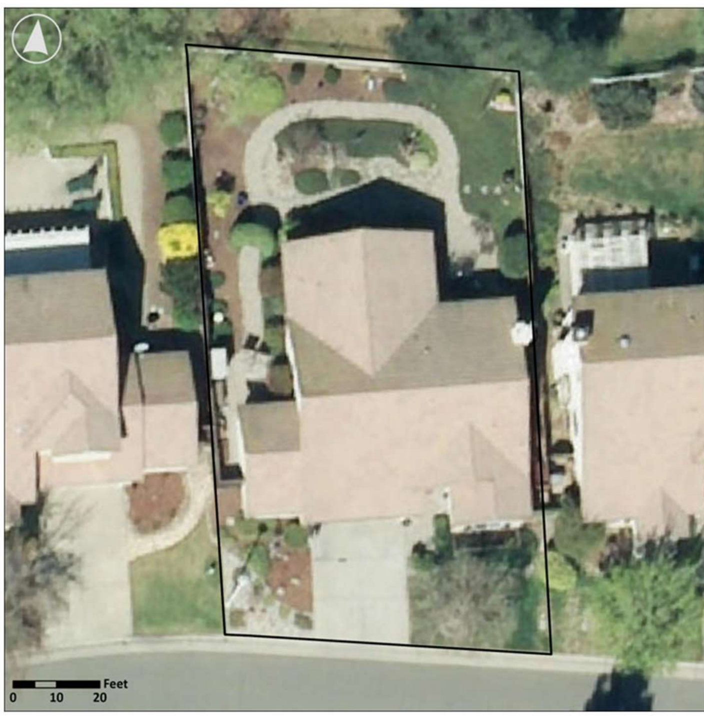
Estate Lots (over 12,000 Sq. Ft.)

- Average lot size: 14,544 square feet
- Demand Factor: 612 Gallons-per-day

Residential Lots	729*
Baseline Demand (Million Gallons per Year)	162.9
Baseline Demand (Acre Feet per Year)	500.0

*Current Active Accounts as of 12/31/2022

Figure 4-4: Estate Lots >12,000 sf Details



Estate Lots (under 12,000 Sq. Ft.)

- Average lot size: 8,161 square feet
- Demand Factor: 398 Gallons-per-day

Residential Lots	577*
Baseline Demand (Million Gallons per Year)	83.9
Baseline Demand (Acre Feet per Year)	257.4

*Current Active Accounts as of 12/31/2022

Figure 4-5: Estate Lots <12,000 sf Details

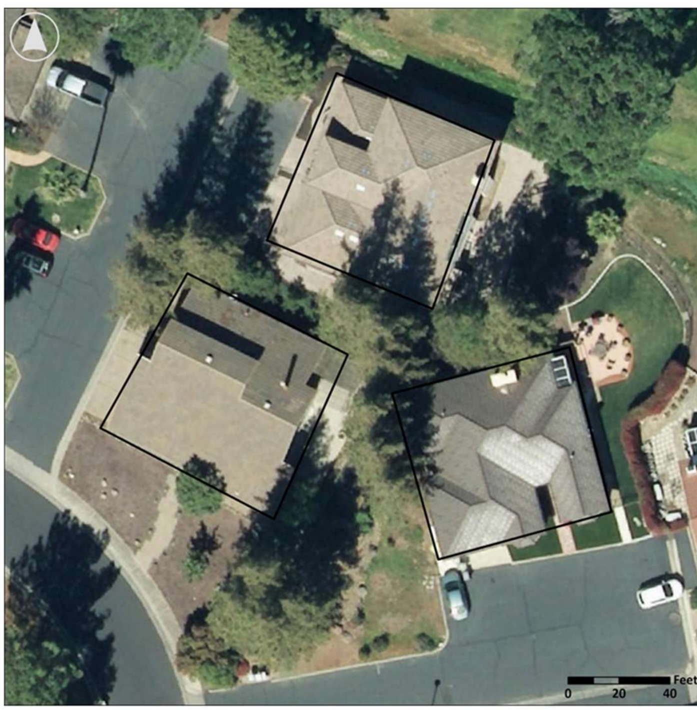
Circle Lots

- Average lot size: 6,332 square feet
- Demand Factor: 486 Gallons-per-day

Residential Lots	454*
Baseline Demand (Million Gallons per Year)	80.5
Baseline Demand (Acre Feet per Year)	247.2

*Current Active Accounts as of 12/31/2022

Figure 4-6: Circle Lots Details



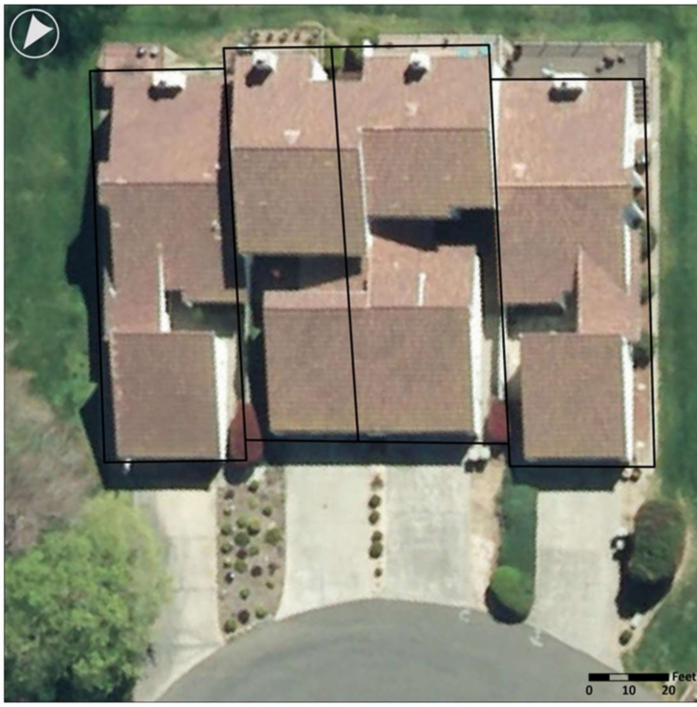
Cottage Lots

- Average lot size*: 3,412 square feet
- Demand Factor: 369 Gallons-per-day

Residential Lots	292**
Baseline Demand (Million Gallons per Year)	39.3
Baseline Demand (Acre Feet per Year)	120.7

* Lot Size, Demand Factor and Baseline Demands are per-lot (3 are shown)
**Current Active Accounts as of 12/31/2022

Figure 4-7: Cottage Lots Details



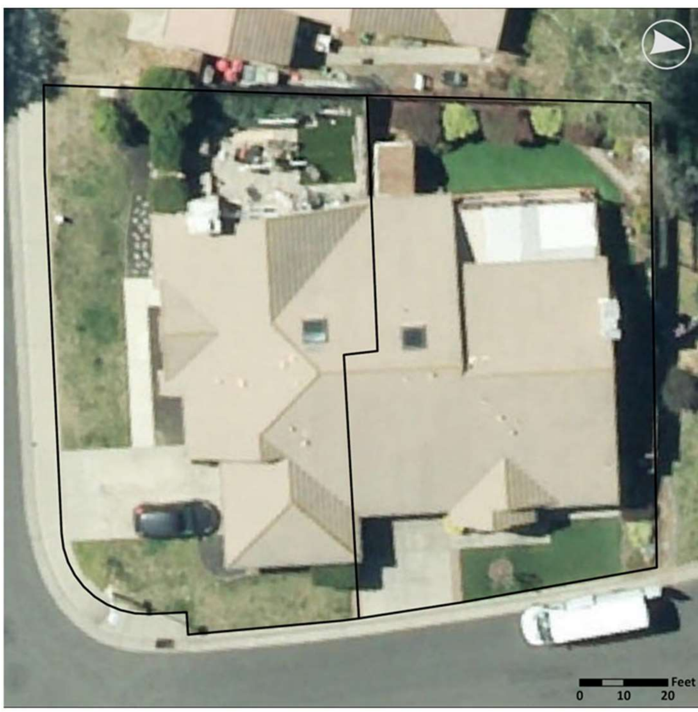
Townhouses/Villas

- Average lot size*: 2,277 square feet
- Demand Factor: 141 Gallons-per-day

Residential Lots	258**
Baseline Demand (Million Gallons per Year)	13.3
Baseline Demand (Acre Feet per Year)	40.8

* Lot Size, Demand Factor and Baseline Demands are per-lot (4 are shown)
**Current Active Accounts as of 12/31/2022

Figure 4-8: Townhomes/Villas Lots Details



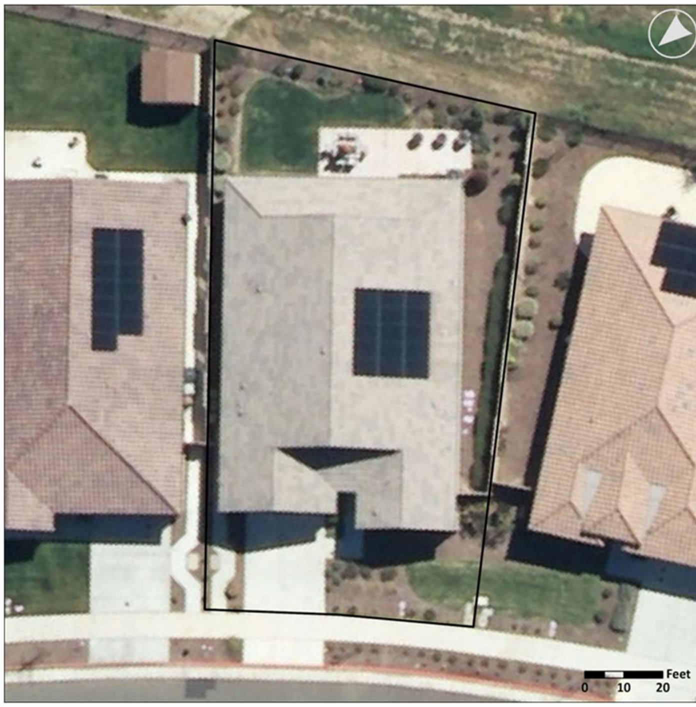
Halfplex Lots

- Average lot size*: 5,212 square feet
- Demand Factor: 266 Gallons-per-day

Residential Lots	59**
Baseline Demand (Million Gallons per Year)	5.7
Baseline Demand (Acre Feet per Year)	17.6

* Lot Size, Demand Factor and Baseline Demands are per-lot (2 are shown)
**Current Active Accounts as of 12/31/2022

Figure 4-9: Halfplex Lots Details



Murieta Gardens

- Average lot size: 6,558 square feet
- Demand Factor: 225 Gallons-per-day

Residential Lots	78*
Baseline Demand (Million Gallons per Year)	6.4
Baseline Demand (Acre Feet per Year)	19.7

*Current Active Accounts as of 12/31/2022

Figure 4-10: Gardens Lots Details

4-2.1. Incorporating Losses into Distribution System Demands

Some water is lost in the distribution system; this loss is referred to as "system losses." This is a natural occurrence in all pressurized pipe networks. Pressurization is especially important for drinking water systems, as it helps them comply with water quality regulations.

System losses are the difference between the produced volume and the consumed volume. System losses are caused by leaks in storage tanks, distribution and transmission mains, or service connections. Calculating system losses is important for water demand estimations because system losses need to be added to customer consumption to accurately represent water use. Additionally, reducing system losses increases the amount of available water without needing to increase the system's supply. An illustration of the District system, including system losses, is shown in Figure 4-11. As mentioned above, an analysis of the historic data led to an estimate of 12% for system losses or NRW. This percentage was added to the demands calculated from customer billing data to estimate the total water production required to meet demands.

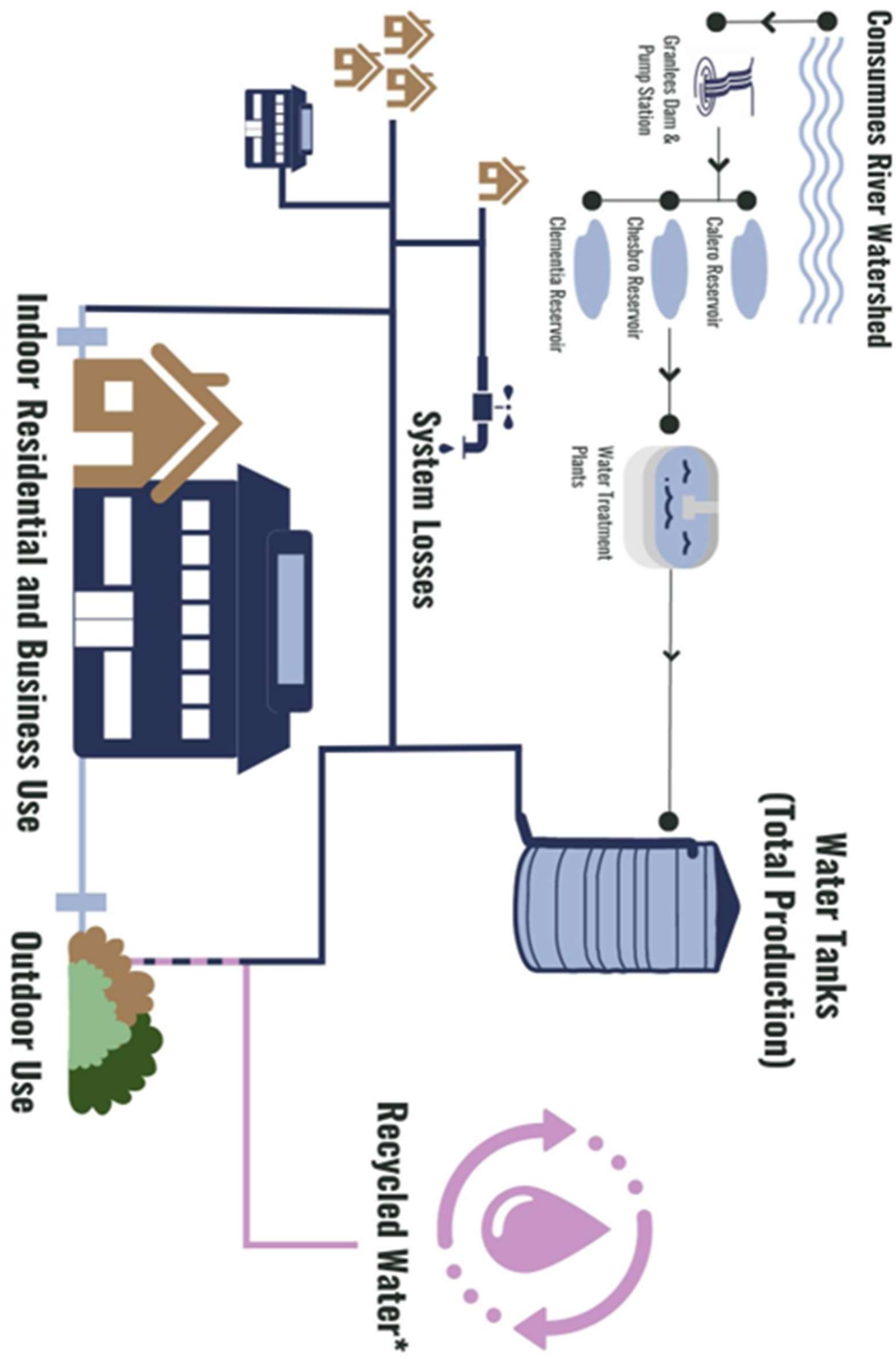


Figure 4-11: System Diagram with NRW

4-3. Future Demands

The purpose of this section is to present detailed information on the demand forecasting approach used in this IWMP and to compare this to the approach used in the 2010 IWMP.

Forecasting a water system's demands is a complex process that involves analyzing the water use with the best available data at the time of analysis. The demand forecasting analysis completed for this IWMP estimates future demands based on existing customer water use and anticipated future development as of September 2023. The development plans in the District have changed over time, as has the anticipated water use associated with these development plans. Figure 4-12 shows the changes in development plans since 2021, including the fact that several previously planned developments have been cancelled at the time of this report.

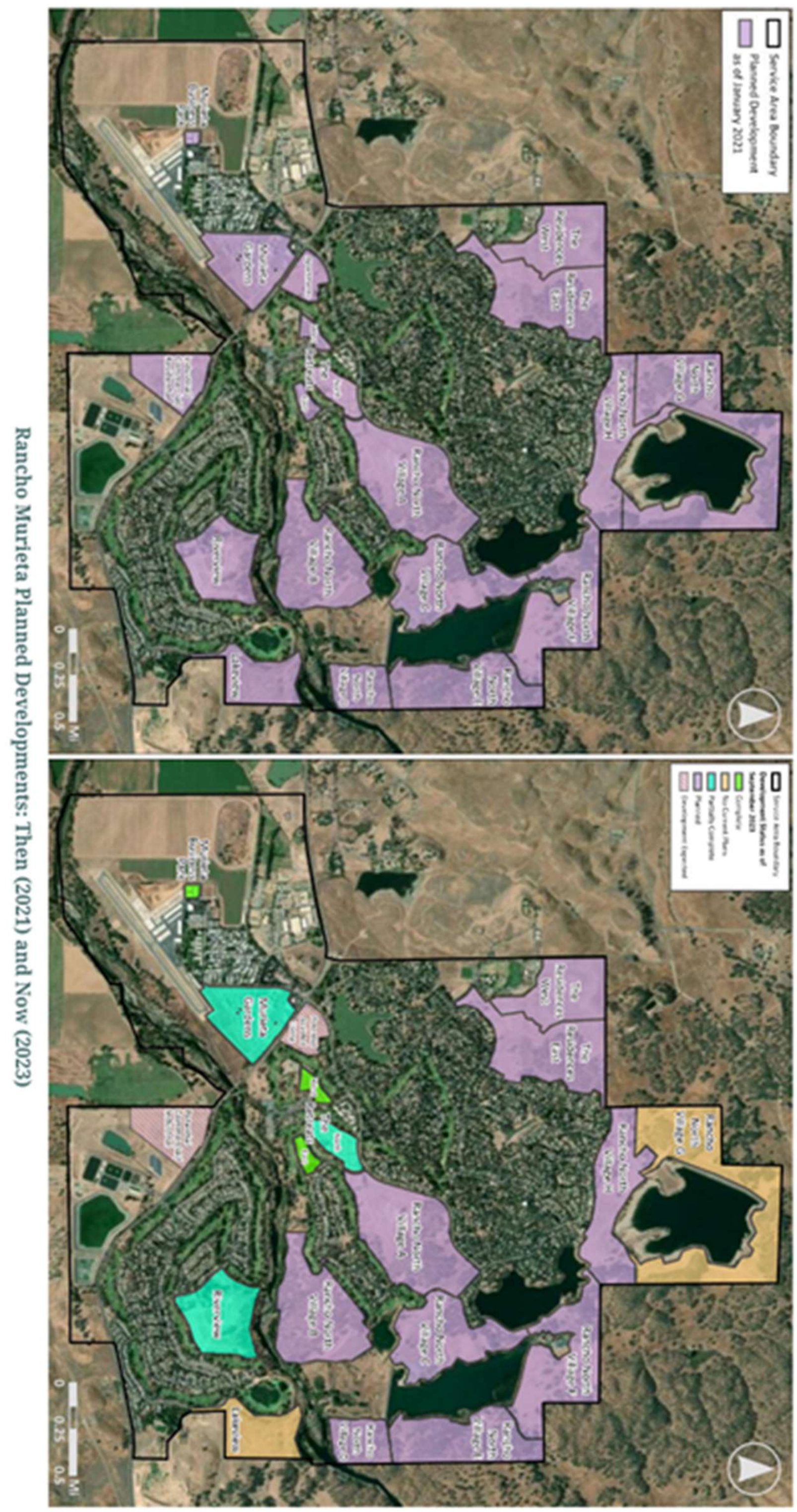


Figure 4-12: Planned Developments in 2021 vs 2023

4-3.1. Parcel-Level Demand Forecast Method

There are numerous methods to prepare demand forecasts for a community. These estimates combine existing uses and future uses of water. One of the most robust and detailed ways to estimate demand is to use the land area of planned future lots, analyzing them by type of land use and expected water use by lot type, otherwise known as demand factors. Future development data is provided to the District as detailed maps from developers (otherwise known as Tentative Maps provided to Sacramento County). Future demand estimates were generated by applying selected gpda values by lot type to the planned parcels.

Future demand estimates were generated using a modified version of the lot-specific gpda approach that was used to generate existing demands. Parcel boundaries from the Sacramento County Assessor's office were analyzed in combination with District billing data to determine the average lot size by lot type.

These average lot sizes were then used to categorize future lots, based on counts and measurements taken from drawings of future development layouts (obtained June-August 2023). For example, if a lot was between 8,500-12,000 sf, it was assigned the "Estate Lots, <12,000 sf" lot type. Demand factors by lot type (the 2020-2022 gpda previously discussed) were then applied to these counts by lot type to generate initial future demand estimates.

Additional categories were used for lots larger than 12,000 sf. Although these demand factors are substantially higher than those observed during the more recent billing data analysis, both the District and the project team believe that a conservative modeling approach is beneficial to ensure the integrity of future water supplies.

Future non-residential demands were estimated on a parcel-by-parcel basis, with research and analysis on each potential development conducted in close consultation with the District based on the latest planning documents (where available) from Sacramento County. Non-residential analysis was completed based on lot size, building square footage, percent building/parking/landscape, and landscape water budgets that model outdoor use. Demand factors representing the average water usage per square foot of building area were obtained and applied from previous studies specific to each development type. The following studies were used:

- Castaic Lake Water Agency Commercial Demand Factor Study, published in 2016 by MWM.
- Santa Clara Valley Water District Commercial, Institutional, and Industrial (CII) Water Use and Conservation Baseline Study, published in 2008 by CDM.
- Methods for Estimating Commercial, Industrial, and Institutional Water Use, published in 2009 by the University of Florida.

4-3.2. Adjustments for Accessory Dwelling Units, Climate Change, and System Water Losses

Additional consideration was given to potential future increases in demands, including the following categories:

Accessory Dwelling Units. California State Law requires local acceptance of new housing, including ADUs. As a result, small additional living areas in a converted space or studio apartments added to both existing and new parcels were assumed to be possible. ADU demands were modeled based on lot size, with 10% of all larger lots and 2.5% of all smaller lots estimated to add ADUs between 12/31/22 and buildout. ADU demands were assumed to be indoor only, and demands were estimated using an occupancy of 1.5 pph. These estimates were developed with input from the District.

Higher Outdoor Demands. Gradual shifts to higher temperatures due to the impacts of climate change, particularly nighttime temperatures that increase the dew point, are expected to increase landscape watering requirements. Consequently, increased outdoor demands were modeled using a 10% increase in total gpda for both residential and non-residential properties with outdoor use. This approach represents a conservative estimate and is a planning practice that models additional unforeseen contingency demands to help safeguard future water supplies.

Accounting for System Losses. Future system losses were also built into future demand estimates and were modeled using the same 12% NRW estimate determined during the existing demand analysis. This is slightly higher based on recent data; prior analyses estimated 10% NRW. The higher estimate was deemed appropriate given that aging system infrastructure can lead to higher losses over time, even with proactive loss control practices in place such as active pipe leak detection and repair programs.

4-3.3. Summary of Baseline Total Future Demand Forecast

The projected future demands, calculated as described above, are shown in Table 4-2 below. Figure 4-13 displays a combination of both historic, existing, and future demands, which are estimated to be 3,384 AFY.

Table 4-2: Future Demands

Future Demands by Lot Type							
Lot Type / User Class	**# of Acct**	**GPDA**	**In. GPDA[6]**	**Out. GPDA[7]**	**In. AFY**	**Out. AFY**	**Total AFY**
1: >24,500 sf[1,3]	95	2,431	145	2,286	15	243	259
2: 14,500-24,500[1,3]	248	979	145	834	40	232	272
3: 12,000-14,500[1,3]	221	910	145	765	36	189	225
4: Estate Lots <12,000 (8500-12K sf) [1,4]	235	438	102	336	27	89	115
5: Halfplex (4100 sf) [2,4]	82	293	102	191	7	13	19
6: 6500-8500 sf (Circle) [2,4]	99	534	102	433	11	48	59
7: <6500 sf (Cottage) [2,4]	140	405	102	304	16	48	64
ADU[5]	265	65	65	-	19	-	19
Residential Subtotal	**1,362**				**172**	**862**	**1,033**
Non-Residential							
New Commercial	11	35,240	26,991	8,249	333	102	435
Non-Revenue Water, estimated to be 12% (NRW)					69	131	200
Total New Demands (with NRW)					**573**	**1,095**	**1,668**

[1]Assumed that 10% of these lots will have ADUs

[2]Assumed that 2.5% of these lots will have ADUs

[3]Assumed that these lots have 3.36 PPH

[4]Assumed that these lots have 2.36 PPH

[5]Assumed that ADUs have 1.5 PPH

[6]PPH multiplied by 43.08 indoor gpcd

[7]Includes a 10% contingency above existing for climate change

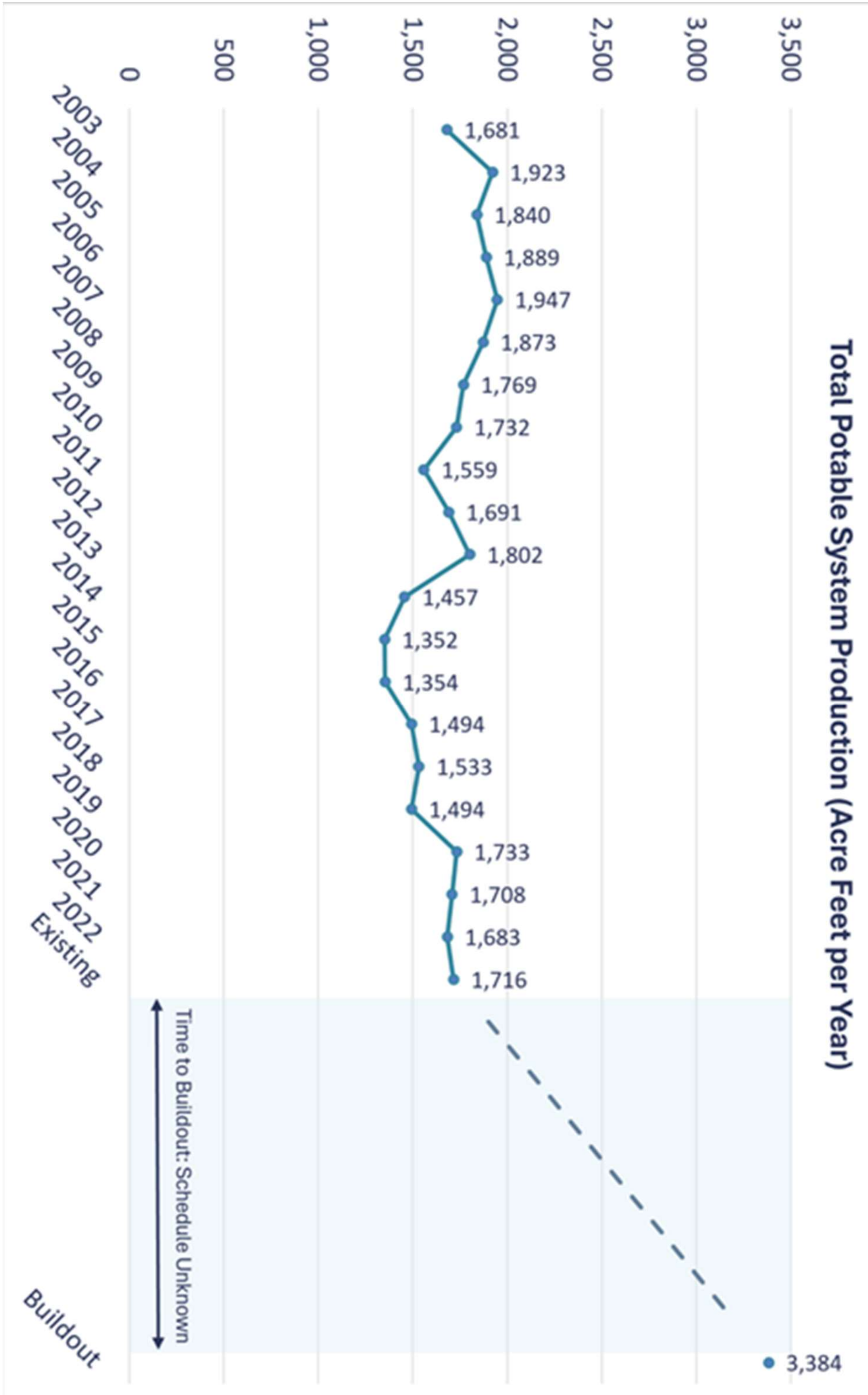


Figure 4-13: Historic, Existing, and Future Demands

4-3.4. Comparison to Legacy Forecast Methods

The following is a comparison of the forecast methods of this IWMP and the legacy forecast method completed in the 2010 IWMP, described in greater detail in Appendix A.

This method results in preliminary findings for existing demands of 1,716 AFY based on lot type demand factors derived from historic billing data and buildout estimate is 3,384 AFY based on the parcel-based lot type analysis described above. The 2010 IWMP previously found nearly identical existing demands of 1,710 AFY and higher buildout estimate of 3,659 AFY without demand curtailment and 2,927 AFY with demand curtailment. There are several factors that explain this difference, listed and described below.

More efficient customer water use habits: The 2010 IWMP was completed before the 20% by 2020 conservation targets were achieved as described above. Analysis of historic and current billing data shows a decrease in per-account water usage over time, likely driven by improvements in fixture efficiency and greater awareness and engagement with water conservation practices such as better water practices and plant selection with lower watering requirements. The effect is present for all lot types, as illustrated in the charts found in Appendix B.

More accurate and detailed inputs: The 2010 IWMP was completed using an Equivalent Dwelling Unit (EDU) basis, which applies a value of 750 gallons-per-day-per-EDU to all estimated current and future EDUs. Estimated future EDU equivalents were developed based on the best available information at the time, which did not include billing data by existing lot types or drawings of future development layouts at the parcel level. The current approach uses the parcel-level demand forecast method described above, which applies specific gpda values to known lot counts and sizes as taken from development drawings.

Future developments cancelled and/or downsized: Several future developments that were anticipated at the time of the 2010 IWMP have since been cancelled or substantially reduced. A comparison between planned developments in 2021 and the present is shown in Figure 4-12 above.

4-3.5. *District Financial Services Agreements to Provide Water*

Demand forecasts were also evaluated on a per-development basis to allow the District to evaluate how well projected demands align with the District's contractual obligations to serve certain properties that previously funded the District's WTP expansion. In 2013-2014, the District faced the need to expand its WTP and developed a plan for how to finance the design and construction costs. That effort resulted in the negotiation, preparation, and approval of two financing and services agreements (FSAs) among two different groups of District landowners: Financing and Services Agreement dated March 17, 2014 (670 FSA); and the Rancho North Properties and Murieta Gardens Financing and Services Agreement dated May 27, 2014 (Rancho North FSA). The 670 FSA covers the Residences West, Residences East, Retreats, Riverview, and Lakeview properties. The Rancho North FSA covers Rancho North Villages A-H (including the lands around the reservoirs), Murieta Gardens, and other properties. The FSAs generally obligate the District to provide water and sewer service to these properties, subject to the terms of the FSAs.

4-4. Peaking Factors and Diurnal Curve

Water systems do not have uniform demands during each hour of the day. Typically, in a system with mainly domestic users, there are peaks in demand during morning and evening hours, when residents are at home and using water, with corresponding drops in demand during other hours. The pattern of demands throughout the day is called a diurnal flow pattern. A custom diurnal flow pattern was developed by analyzing hourly production data as well as changes in tank levels. Typically, the generic diurnal flow pattern from AWWA is used for system modeling, but the custom pattern developed for the District is a more accurate representation than the generic pattern. The production flows and tank flows were combined to estimate the total water demand during each hour of the given day. The day of highest demand (peak day) was analyzed for the years 2016-2022. The demand during each individual hour was compared to the average demand for that day to calculate a multiplier for each respective hour. The average of these hourly multipliers was taken for the peak days analyzed to formulate an hourly demand pattern for the model. The highest hourly demand factor of 1.87 occurs at 6:00 AM. This is the peak hour factor (PHF). The diurnal curve is visualized in Figure 4-14.

A maximum day demand (MDD) factor was also determined by review of peak flow data from 2020 to 2022, which was the time period selected by the MWM team to best represent the use patterns in the District. Peak day factors are the ratio of the MDD and the ADD. For this time period, the MDD factor was 1.82. This factor was applied to the average demands as discussed in the previous section to represent a peak day in the model. The ratio of the flow at the peak hour of the peak day to the average flow is 3.40. See Table 4-3 below.

Table 4-3: Peaking Factors

Peaking Factors	
Criteria	**Ratio**
MDD/ADD	1.82
PHF/MDD	1.87
PHF/ADD	3.40

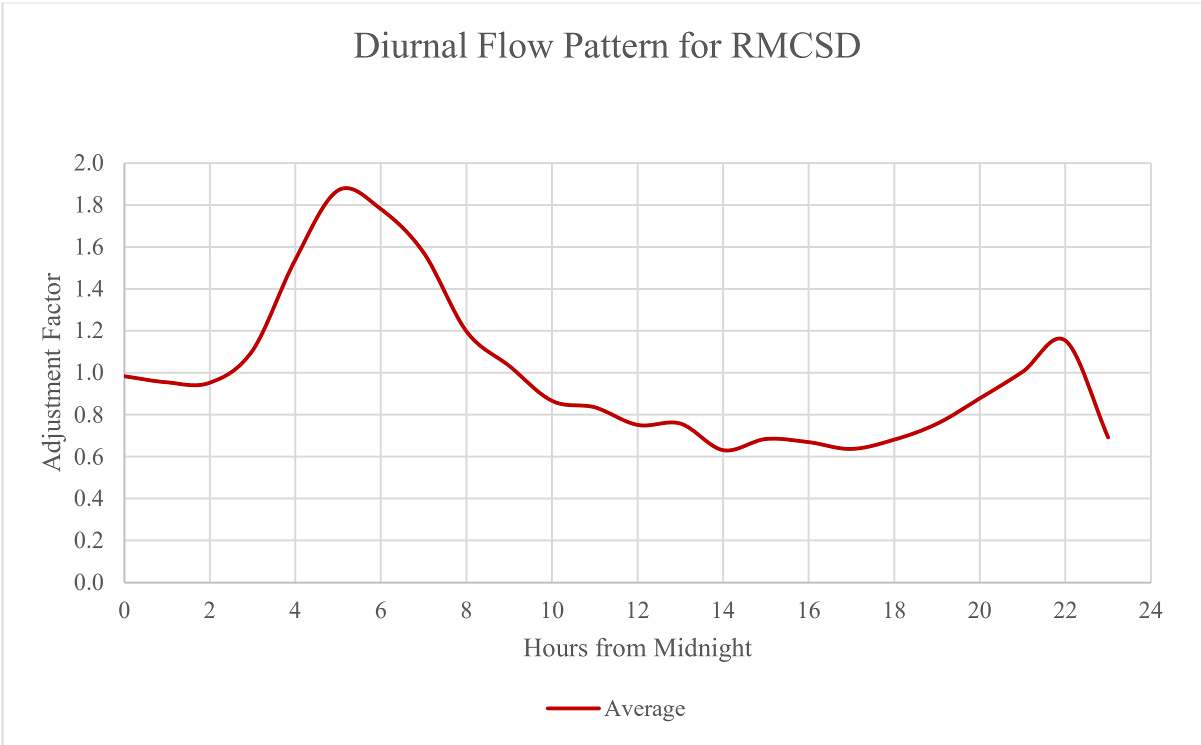


Figure 4-14: Diurnal Curve

4-5. Reclaimed Water System Demands

This section describes the existing and future demands for reclaimed water.

4-5.1. Existing Reclaimed Demands

To develop a basis of reclaimed water demands, a water balance of historic wastewater inflows, rainfall, reclaimed water production, and golf course demand was conducted. See Table 4-4 below.

Table 4-4: Historic Reclaimed Production and Golf Course Demand

Historic Wastewater Flow Data and Golf Course Demands				
Year	**Rainfall (in/year)**	**ADWF[1] (MG/month)**	**Reclaimed Water Sent to Golf Courses (AFY)**	**Total Golf Course Demand (AFY)**
2009	17.52	14.30	451.35	No Data
2010	29.32	13.66	418.18	No Data
2011	20.78	14.03	335.46	No Data
2012	23.08	12.39	416.30	681.37
2013	6.16	12.22	435.25	754.71
2014	22.86	11.01	390.22	708.85
2015	12.86	10.51	329.01	673.75
2016	24.30	10.61	368.58	629.89
2017	31.26	11.30	557.24	718.74
2018	22.92	11.36	475.43	683.68
2019	27.24	11.31	478.24	614.85
2020	12.04	12.54	413.25	673.31
2021	24.54	12.89	328.97	591.81
2022	20.02	11.30	449.96	No Data
average	21.06	12.10	417.67	673.10
minimum	6.16	10.51	328.97	591.81
maximum	31.26	14.30	557.24	754.71

[1]ADWF assumed to be June 1 through September 30.

For the analysis in this IWMP, the average golf course demand will be taken to be the average of the 10-year period analyzed, which is 673 AFY. Peak daily, weekly, and monthly demands will be discussed in Chapter 5.

4-5.2. Future Reclaimed Demands

Developments that are planned to receive reclaimed water are Murieta Gardens, the Retreats, and Villages A, B, and C. Using the demand criteria discussed in the prior sections, the outdoor uses of each of these developments were estimated using the numbers and sizes of lots currently planned for development and associated demand factors for each. These are summarized in Table 4-5 below.

Table 4-5: New Developments Reclaimed Demand

New Developments Reclaimed Demand			
Development	**Approved # Accounts[1]**	**Outdoor Demands (AFY)**	**Existing/Future Infrastructure**
Murieta Gardens - Residential	78	11	Existing
Murieta Gardens – Commercial[2]	62	203	Existing
The Retreats	82	19	Existing
Village A	215	110	Future
Village B	136	120	Future
Village C	94	83	Future
Total:		546	

[1]Combined existing and proposed accounts per development.

[2]An analysis was performed of existing commercial accounts and irrigation-only commercial accounts to determine which ones can be served in Murieta Gardens. This was combined with the projected new outdoor commercial demands to find this value.

The total estimated demand for reclaimed water from the proposed areas to be served is 546 AFY. With 673 AFY of golf course demand, this totals 1219 AFY of demand. Van Vleck does not have demand but can allow disposal of up to 215 AFY. This results in a total reclaimed water disposal capacity of 1434 AFY.

CHAPTER 5. System Analysis

5-1. Model Development and Calibration

5-1.1. Model Description

EPANet2.2 was the hydraulic modeling software used for this IWMP. This software performs extended period simulation of hydraulic and water quality behavior in pressurized pipe networks. It tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, and the concentration of a chemical species throughout the network during a simulation period comprised of multiple time steps. In addition to chemical species, water age and source tracing can also be simulated. EPANet2.2 provides an integrated environment for editing network input data, running hydraulic and water quality simulations, and viewing the results in a variety of formats. These include color-coded network maps, data tables, time series graphs, and contour plots. EPANet 2.2 is a free software developed by the EPA. Therefore, the District can use the models developed in this study to evaluate future developments without needing to maintain costly software licenses.

5-1.2. Physical Data

System geometry was imported from District GIS files into Civil3D and assigned elevations using publicly available LiDAR data. This was exported to the model in the form of links and nodes representing pipes and junctions, respectively. The Adkins team coordinated with District staff for relevant physical data including pump curves, stage-storage curves, pumping plant configuration and valve orientation, set points, and other system control rules.

For the potable water system, existing and proposed pipe networks were assumed to be two feet below the ground surface. For the reclaimed water system, existing and proposed pipe networks were assumed to be three feet below the ground surface.

System controls were determined through correspondence with District staff. These included the WTP pumps controlled by the clearwells at each of the WTPs, the Van Vleck tank levels turning the WTPs on and off, and a pressure node controlling the Rio Oso pumps. The flow limits for WTP 1 and 2 were determined through review of historic production data. The maximum flow day observed over the past ten years was July 20, 2022. Using Supervisory Control and Data Acquisition (SCADA) data, which reports every minute throughout each day, the average inflow at WTP1 for this peak day was 939.9 GPM and the average inflow at WTP2 was 1047.6 GPM. These are summarized in the tables below. Table 5-1 shows the control set points for the WTP pumps, which are controlled by each plant's clearwell levels, and Table 5-2 shows the set points for Van Vleck and Rio Oso tanks.

Table 5-1: Set points for the WTPs 1 (left) and 2 (right) pumps.

WTP 1 Clearwell		
Clearwell Level (ft)	**Start**	**Stop**
Pump 1	5.5	4.5
Pump 2	6.0	4.6
Pump 3	6.2	4.7
Pump 4	6.4	4.8
Pump 5	6.5	4.9

WTP 2 Clearwell		
Clearwell Level (ft)	**Start**	**Stop**
Pump 1	6.5	3.0
Pump 2	7.5	3.5
Pump 3	8.2	4.0

Table 5-2: Set points for Van Vleck (left) and Rio Oso (right) tank controls.

Van Vleck Controls		
	Start	**Stop**
Van Vleck Tank Level (ft)	25.5	27.5
WTP1 Flow Limit (GPM)	939.9	0
WTP2 Flow Limit (GPM)	1,047.6	0

Rio Oso Pressure Control		
n313[1] pressure (psi)	**Start**	**Stop**
Rio Oso Pump 1	65	81
Rio Oso Pump 2	35	81
[1]Highest node in Rio Oso zone		
Rio Oso Tank Level (ft)	**Open**	**Closed**
Rio Oso Inlet	25	27
[1]n313 is the highest node in the Rio Oso zone		

5-1.3. Demand Nodes

Development of demands is discussed at length in Chapter 4. Average Day Demand (ADD) was the key criteria for model development. ADD by account type for existing and buildout conditions is summarized in Table 5-3.

Table 5-3: Existing and Future Demands by Lot Type

Existing and Future Demands by Lot Type					
Lot Type/User Class	**Current GPDA (Total)[1]**	**Future GPDA (Total)[2]**	**Existing Accounts**	**Planned Accounts**	**Total Accounts @ Buildout**
Residential					
Estate: >24,500 sf	n/a	2,431	n/a	95	95
Estate: 14,500-24,500	n/a	979	n/a	248	248
Estate: 12,000-14,500	n/a	910	n/a	221	221
Estate: > 12,000 sf	612	673	729	n/a	729
Estate: < 12,000 sf	398	438	577	235	812
Halfplex	266	293	59	59	118
Circle	486	534	454	99	553
Cottage	369	405	292	140	432
Townhouse (Villas)	141	155	258	0	258
Murieta Village	124	137	181	0	181
Murieta Gardens	225	248	78	0	78
ADU	0	65	0	265	265
Other	6,831	6,831	1	0	1
		Subtotal	2,629	1,362	3,991
Non-Residential					
Commercial	2,212	2,433	81	0	81
New Commercial	n/a	35,240	0	11	11
Parks	7,849	8,634	5	0	5
Misc. Public Uses	1,872	2,059	14	0	14
		Subtotal	100	11	111

[1]Based on analysis of water usage records. See Chapter 4.

[2]Adds 10% to existing demands for climate change contingency. See Chapter 4.

ArcMap 10.3 was used to process data and group adjacent lots by lot type and water use. This resulted in the existing accounts being condensed into 183 "demand nodes." This was done to reduce the size and complexity of the model to increase ease of use. For example, if 15 Circle lots were in close proximity to a node in the model, that node would be assigned a demand corresponding to 15 Circle lots. The buildout model has 140 additional demand nodes to represent the future connections.

5-1.4. Sources of Information

Sources of information used to develop the hydraulic model are summarized in Table 5-4.

Table 5-4: Sources of Model Inputs

Sources of Information Used for Model Development		
Data	**Source**	**Purpose**
System GIS maps	District database	Pipe layout, size, and minor losses
Billing and use data	District staff	Demands, peaking factors, and demand node grouping
LiDAR	USGS online database	Elevations
Reservoir bathymetry	Adkins survey	Stage-storage curves for Calero and Chesbro
Reservoir loss equations	MWM	Losses due to seepage and evaporation
Pump curves	District staff	Pump flows and capacity
System set points	District staff	System controls for pumps, tanks, and the WTP
Hydrant testing records	District staff	Model calibration

5-1.5. Model Calibration

Hydrant testing data is a standard method used to calibrate and verify hydraulic software models. A hydrant test involves measuring the static pressure at two adjacent hydrants. One hydrant is then opened and allowed to flow fully. The static pressure at the non-flowing hydrant is measured, and the flow from the open hydrant is estimated with a pitot gauge. In the hydraulic model, a demand equal to the flow measured is applied at the flowing hydrant, and the pressure drop in the model at the non-flowing hydrant is compared to the value measured in the field.

The District provided hydrant testing data from 2015 to 2023. The Adkins team used this data to calibrate the model. Calibration used both static pressures and the residual pressure at the non-flowing hydrant, iterating the model's hydraulic input parameters (pipe roughness and loss coefficients) until the modeled pressures deviated less than 15% from the in-field static and residual pressures. These are summarized in Figure 5-1.

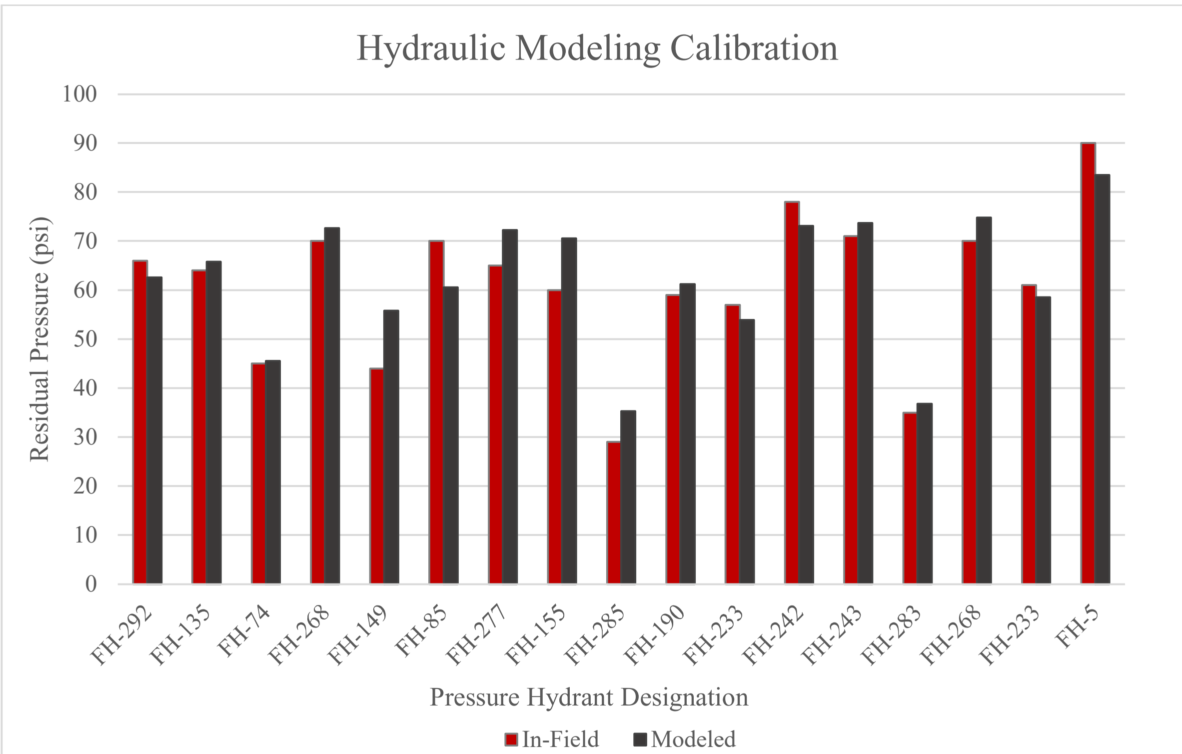


Figure 5-1: Hydrant Data for Calibration

5-1.6. Key Assumptions

- Pipes were assigned Hazen-Williams Coefficients (CHW) of 130-135 with loss coefficients ranging from 0.5 to 1.5 based on number of reducers, elbows, connections, valves, and other fittings. These were iterated during calibration, as discussed above, such that the loss coefficients ranged from 0.7 to 1.2. Some pipes were assigned a CHW of 125 to calibrate the model to the hydrant test data.

- Flow from Chesbro Reservoir into the WTPs was assumed to be limited by the observed peak day flows from 2015-2022. These were relatively consistent across the period, with maximum observed instantaneous flow rates of 1,664 GPM and 1,555 GPM for WTP 1 and 2, respectively. The plants have a larger design capacity than these flow rates, but neither reached that design capacity flow in the peak days reviewed.

5-2. Water Supply Evaluation

5-2.1. Water Supply Model Description

The water supply evaluation was performed using the Shared Vision Planning Model (SVM). The SVM is implemented as an Excel-based tool that allows testing of various supply, demand, and infrastructure scenarios. The tool includes all source data used in the simulated water balance exercise. The simulated water balance is computed in the Excel spreadsheet and is presented in over 60 columns that perform sequential calculations considering District pumping rights based on Cosumnes River flows and many other elements.

The ultimate goal of the modeling effort is to test the District's water supply system for resiliency under a variety of conditions as part of a thorough engineering exercise. These include normal baseline conditions, which reflect the supply and demands in a normal weather year with both current and future demands applied. The model also allows the District to simulate "worst-case" scenarios, based on conditions observed during historic droughts, with the impacts of climate change applied and with elements of the supply system offline. Additionally, the District can simulate potential supply augmentation options such as future expansion of the reclaimed water system and/or new supply wells. Note that the outputs of the model do not represent a predictive forecast that prescribes exactly what the District will do in the future, but rather provide a scenario testing tool to explore potential future conditions and potential options to meet water demands.

Detailed modeling steps were taken to accurately quantify all elements impacting District water supply availability under simulated current and future conditions, including demand scenarios, hydrology/climate conditions, and reclaimed water availability. Individual elements in the model can be switched on and off, with simulated impacts to water supply availability shown in real time based on selections.

5-2.2. Modeling Components

A comprehensive list of potential scenario inputs were considered during model development as representative of current and future District demands and infrastructure. Model inputs were narrowed down based on data available from the District and other trusted sources. The final selected elements are listed and described in the next sections.

5-2.2.a. Demand Scenario

The Demand Scenario drop-down allows the District to select between current water demands (as of 12/31/2022) and buildout demands as modeled using billed consumption data grouped by lot type to generate an average GPDA by lot type by year. Development of future demand estimates is described in Chapter 4.

5-2.2.b. Hydrology/Climate Scenario

Historic flows were evaluated to select three time periods valuable to investigate for future scenario planning. Three hydrology scenarios were developed: Historic Drought (Nov ’75 to Dec ’78), Recent Drought (Nov ‘13 to Dec ‘16), and Average Recent Year (Nov ‘21 to Oct ’22). See Figure 5-2 for historic Cosumnes River flows.

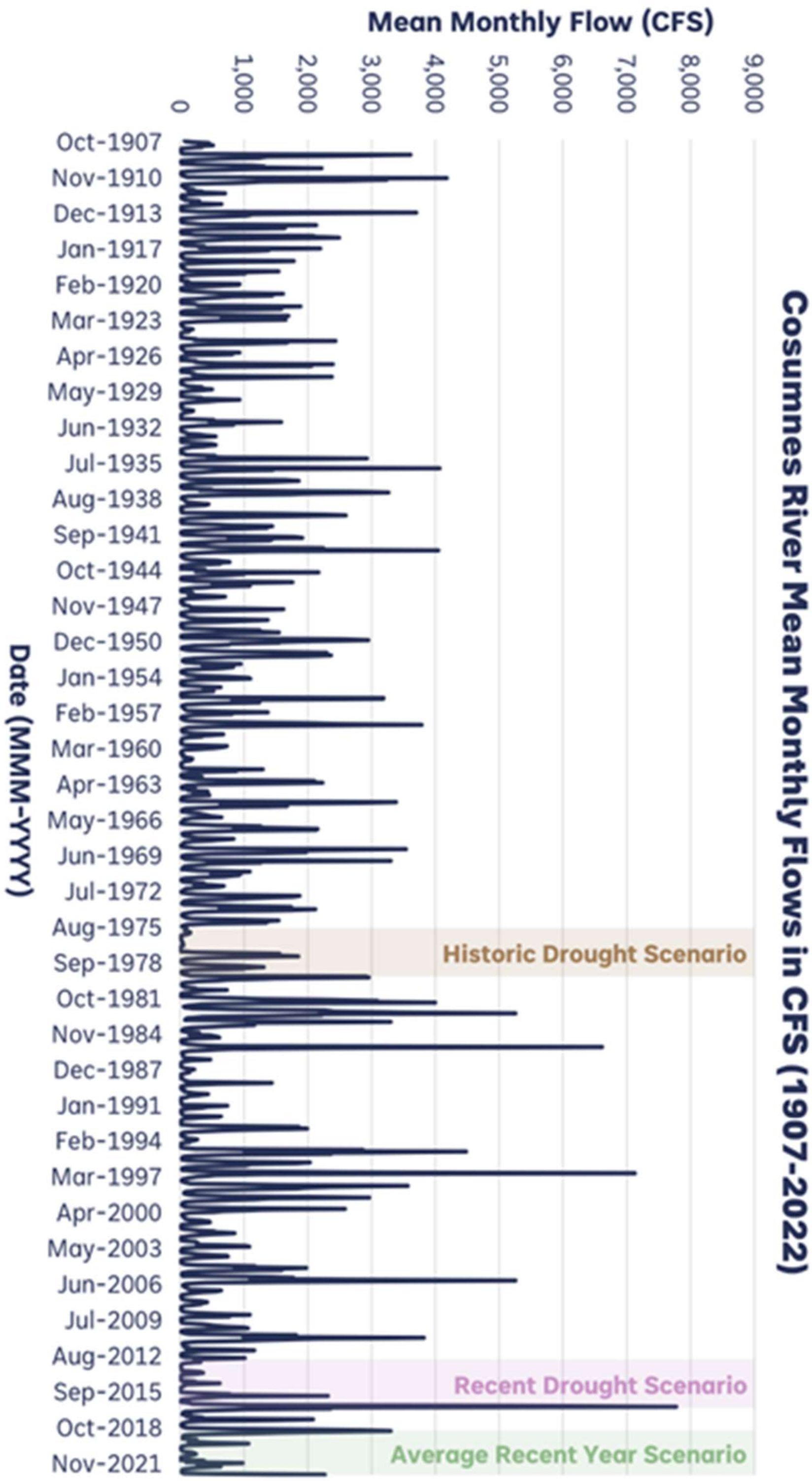


Figure 5-2: Cosumnes River Historic Flows

Figure 5-2 above shows the mean monthly flows for the entire hydrologic record for the Cosumnes River (as of 2023), as well as overlays to show roughly where the hydrology scenarios developed for the model fit into the hydrologic record. Monthly river flow data was obtained from the USGS website. See Figure 5-3 and Figure 5-4 below for smaller scale drafts of the drought scenarios used in the model.

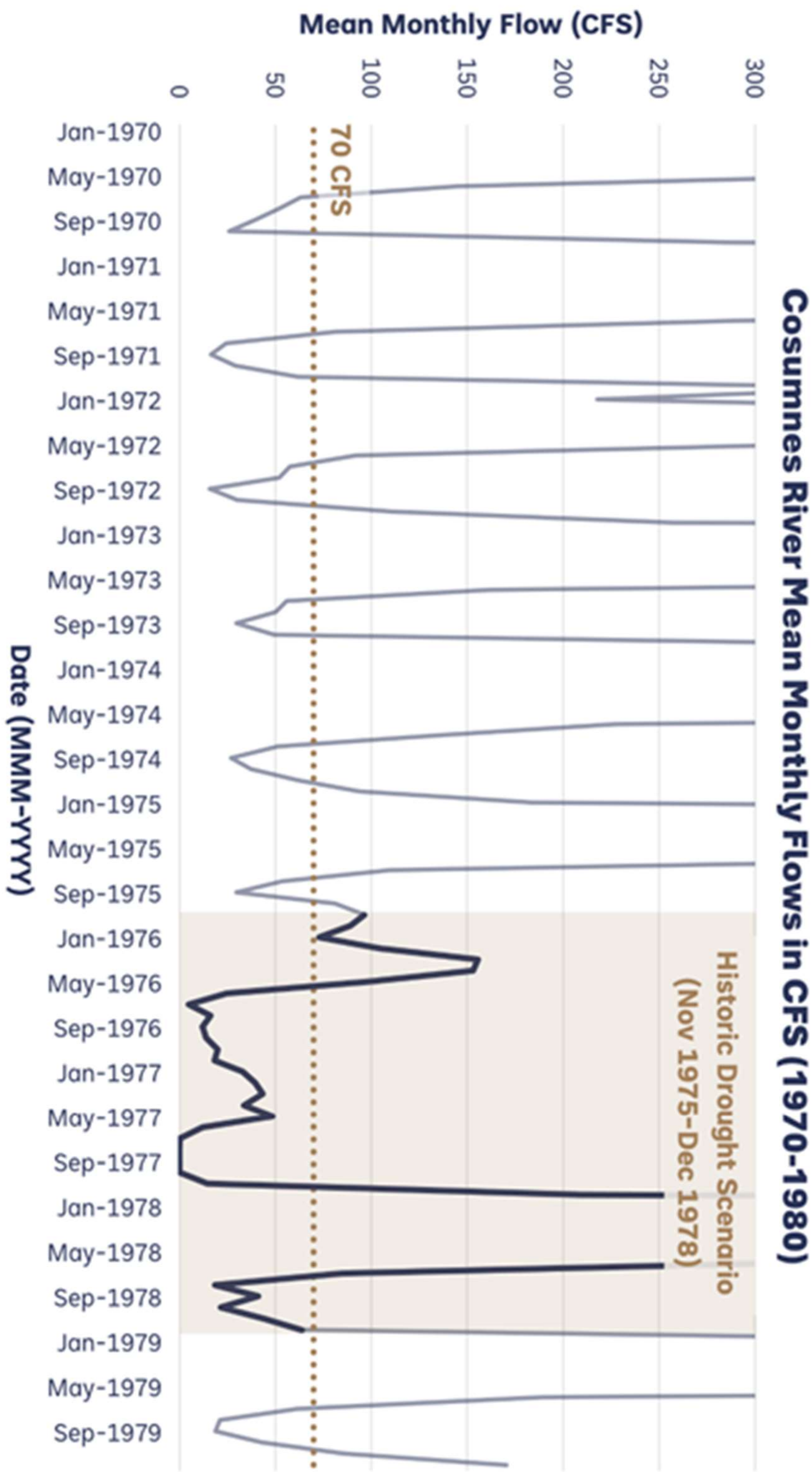


Figure 5-3: Historic Drought River Flows

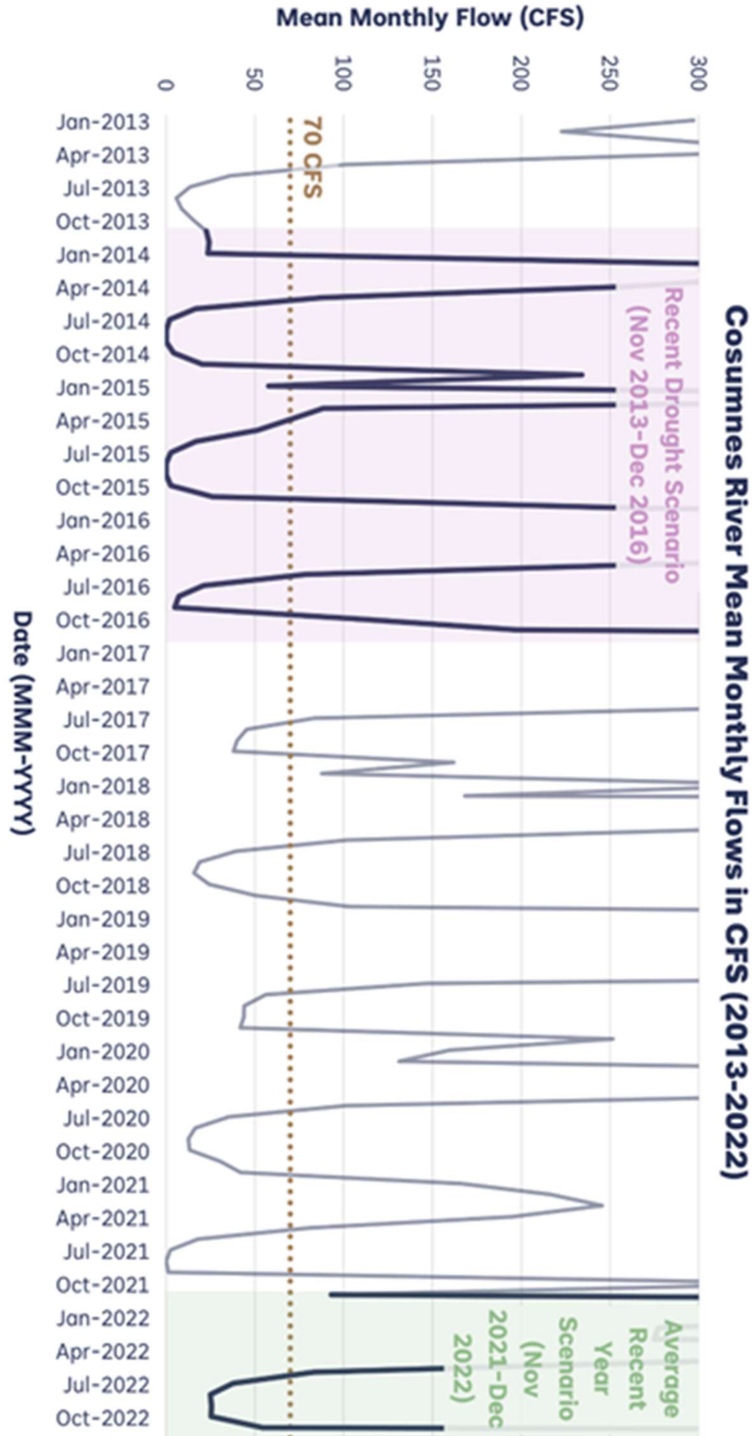


Figure 5-4: Recent Drought River Flows

Figure 5-3 shows the "Historic Drought" hydrology scenario and has a horizontal line at a 70 cfs flow. A 70 cfs flow is the minimum flow required for the District to pump water from the river. From May 1976 to November 1977, river flows did not reach the 70 cfs threshold and no pumping was allowed. This meant that the District went one entire water year without pumping, which stressed the water supply significantly. This is the period of the lowest flows recorded for the Cosumnes River and is used in the model as the "worst-case" scenario for this reason.

Figure 5-4 shows the "Recent Drought" scenario and the "Average Recent Year" scenario used in the model. The "Recent Drought" hydrology scenario coincides with the drought declared by the State of California during 2013-2016. During this period the District was able to pump sufficiently in the November 1 to May 31st pumping window to fill reservoirs enough to meet demands. There were months where flows were below the 70 cfs threshold in non-eligible months. It is important to note that although rainfall runoff has supported river flows in other dry years, the District's water rights have been stressed with curtailment at the direction of the Water Resources Control Board during dry years (such as individual days with curtailed pumping to better support downstream river flows into the Sacramento-San Joaquin Bay Delta). Reservoir supplies from Calero and Chesbro were still adequate to meet current demands in the most recent dry years of 2020-2022. These types of curtailments were considered in this assessment.

The "Average Recent Year" hydrology scenario was chosen after comparing mean and median river monthly flows in the last 10 years to the mean and median monthly flows for the entire Cosumnes River hydrologic record (115 years of data). In the past 10 years, the period of November 2021 to December 2022 most closely resembled the monthly flows of an average year for the Cosumnes River.

Additional climate change impacted versions of each hydrology scenario are also included, simulating future Cosumnes River flows under the influence of climate change. Per discussions with the consulting team (Woodard and Curran) on the American River Basin Study, the Variable Infiltration Capacity modeling results from the 2013 *Analysis of Climate Change Impact on Water Resources in the American River Basin (ARB) Region* study represent the most current modeling efforts specific to the Cosumnes River and are recommended for use in the this IWMP.

5-2.2.c. Drought Plan and Settings

This modeling component simulates demand reductions based on the District's drought plan, including user-selected percent cutbacks for various drought stages. Drought triggers are also displayed within the model's Usable Supply chart, and are based on percent of monthly available storage based on an average recent supply year (November 2021 to October 2022). The following percentages are used for drought stages within the model:

- Stage 1: Normal. Full storage in reservoirs (>95%);
- Stage 2: Water Alert. 90-95% storage in reservoirs;
- Stage 3: Water Warning. 75-89% storage in reservoirs;
- Stage 4: Water Crisis. 50-74% storage in reservoirs;
- Stage 5: Water Emergency. Less than 50% storage in reservoirs.

The Drought Plan settings are informed by the District's 2012 Water Shortage Contingency Plan (WSCP). Within the engineering assessment performed using the model, demand curtailments have been capped at 30% cutbacks based on cutback percentages deemed feasible by the District and the consulting team. Note that the WSCP outlines options for up to 50% reductions. This is to provide for further emergency demand mitigation measures that may be required from a variety of emergency conditions (e.g., supply interruption due to a main break), and this level of planning to 50% reduction is needed to meet the requirements of California Water Code, Section 10632.

5-2.2.d. Early Pumping

This component simulates early use of the District's 500 HP pump. Due to operational costs, this pump is typically not engaged until later in the pumping season (February) but the District may want to engage as early as November in times of prolonged multi-year drought. Note that in all simulated scenarios the early use of the large pump did not result in substantial improvements to water supply as filling the reservoirs earlier in the season results in higher seepage and evaporative losses.

5-2.2.e. Supply Reductions

Simulates reduced supply due to the following circumstances:

- Clementia Reservoir is not able to get licensed as a potable water reservoir.
- Calero Reservoir is offline due to any future extraordinary event,
- Raised stop logs (flashboards) are not able to be utilized. Raised stop logs are installed at the top of each reservoir's spillway or crest to increase the effective storage depth of each reservoir by two feet.

Simulating stop log removal, as well as the offline reservoir scenarios, is helpful for the District to simulate supply conditions under various potential operational situations.

5-2.2.f. Supply Augmentation

This component simulates increased water supply under conditions that reduce potable water demands by increasing the availability of reclaimed water for outdoor usage in current and future developments, or by increasing available potable supply by adding a new water supply well or series of wells. The model has four supply augmentation options that can be selected by the user.

1. Serve golf courses using Cosumnes River rather than Reclaimed Water: this option creates additional reclaimed water volume based on an existing water rights permit that would allow the golf courses to shift outdoor irrigation demands from reclaimed to direct Cosumnes River water use from May through October. This permit allows for direct diversion up to about 74 AF per month if river flows are sufficiently high.
2. Use Reclaimed Water for New Connections: simulates a reduction to potable demands if reclaimed water is used for outdoor irrigation in new (planned) developments with planned infrastructure suitable for reclaimed water, as identified by the District and modeled for this IWMP. Planned reclaimed water infrastructure improvements are included in Chapter 6. Developments planned to receive reclaimed

water include Murieta Gardens (which is already dual-plumbed), Retreats, Village A, Village B, and Village C.

3. Use Reclaimed Water for New Connections and Existing Dual-Plumbed Connections: simulates reduction to potable demands if reclaimed water is used for outdoor irrigation in both new (planned) and established dual-plumbed developments with infrastructure suitable for reclaimed water, as identified by the District and modeled for this IWMP. The current reclaimed water system produces about 437 AFY of reclaimed water. At buildout, reclaimed water availability is estimated to be about 987 AFY during an average year, 910 AFY during a recent drought year, and 858 AFY during a historic drought year.
4. Add a New Supply Well: adds additional volume to supply based on selected pumping capacity for new supply well(s). The District pursued potential groundwater wells in the years immediately following the 2010 IWMP. A test well was drilled with the results shown in the 2013 DE Memo previously referenced , which identified the potential for a 370 gpm well within the western portion of the confined alluvial basin within District boundaries. The required well flow rates to meet ADD at the 3,000-connection level and the buildout level are 1,169 gpm and 2,097 gpm, respectively. The model allows the user to select between well flow rates as part of this supply augmentation option.

5-2.3. Resilience Testing

The SVM was applied to test different scenarios and the options available to meet demands under different circumstances. This testing process involved running through different simulations by changing the components selected in the model (such as demand scenario, hydrology scenario, drought settings, potential supply reductions, and supply augmentation options described above) to identify circumstances that resulted in water stress and the options that could alleviate that stress. The results of the model will help the District in planning for system resiliency. Results from the scenarios that are most impactful to planning efforts are presented below. It is important to note that the outputs of the model provide a scenario testing tool to explore potential future conditions and potential options to meet water demands, and are not a forecast to prescribe an exact course of action for the District. Note that the years presented in the charts are for simulation purposes and do not reflect actual hydrology and demands during past years (for charts labeled 2021-2024, or the "current demand" case) or any estimates on when buildout may occur.

5-2.3.a. Scenario 1: An Average Recent Year

In this scenario, hydrology is for the average recent year, reclaimed water serves planned connections and existing double-plumbed connections, the golf courses are served by raw river water, Clementia reservoir is not used for storage, and no drought plan is followed. See Figure 5-5 for model results under current demands and Figure 5-6 for results under buildout demands.

Figure 5-5: Scenario 1, Existing Demands

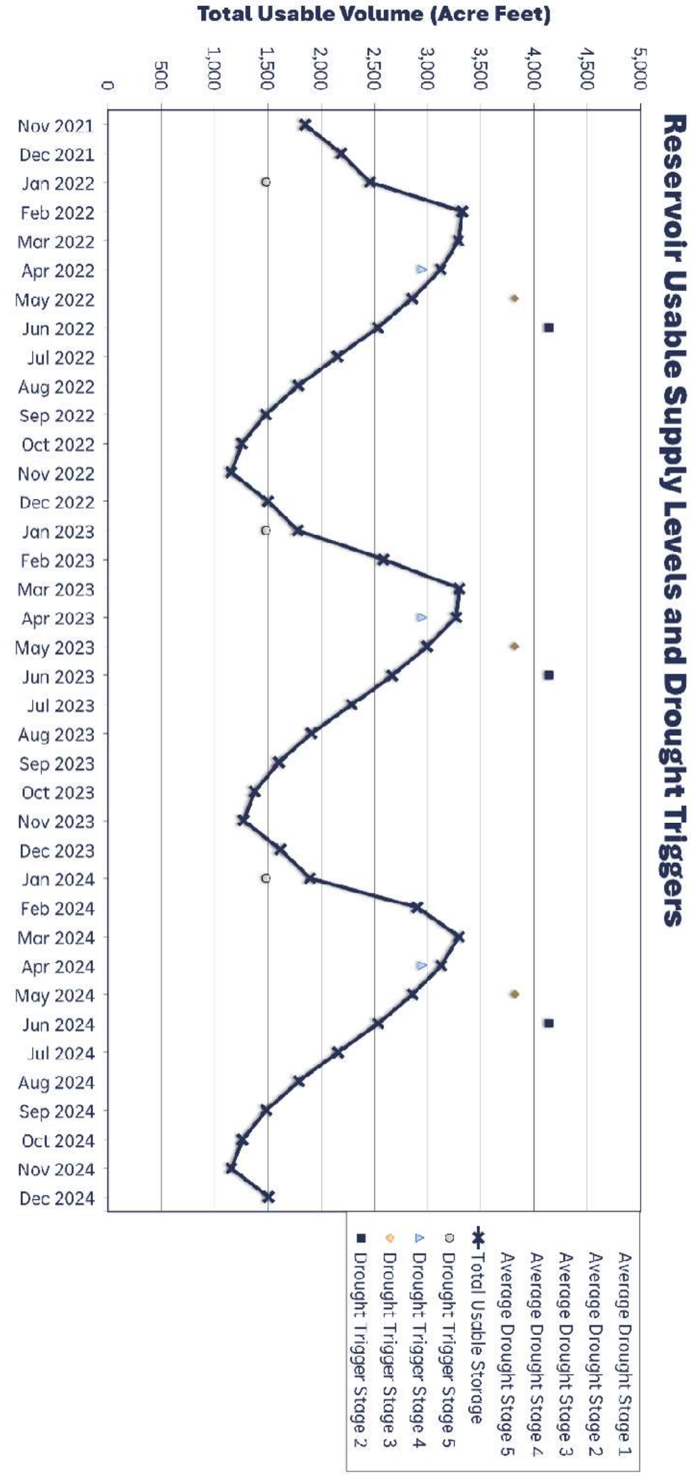


Figure 5-6: Scenario 1, Buildout Demands

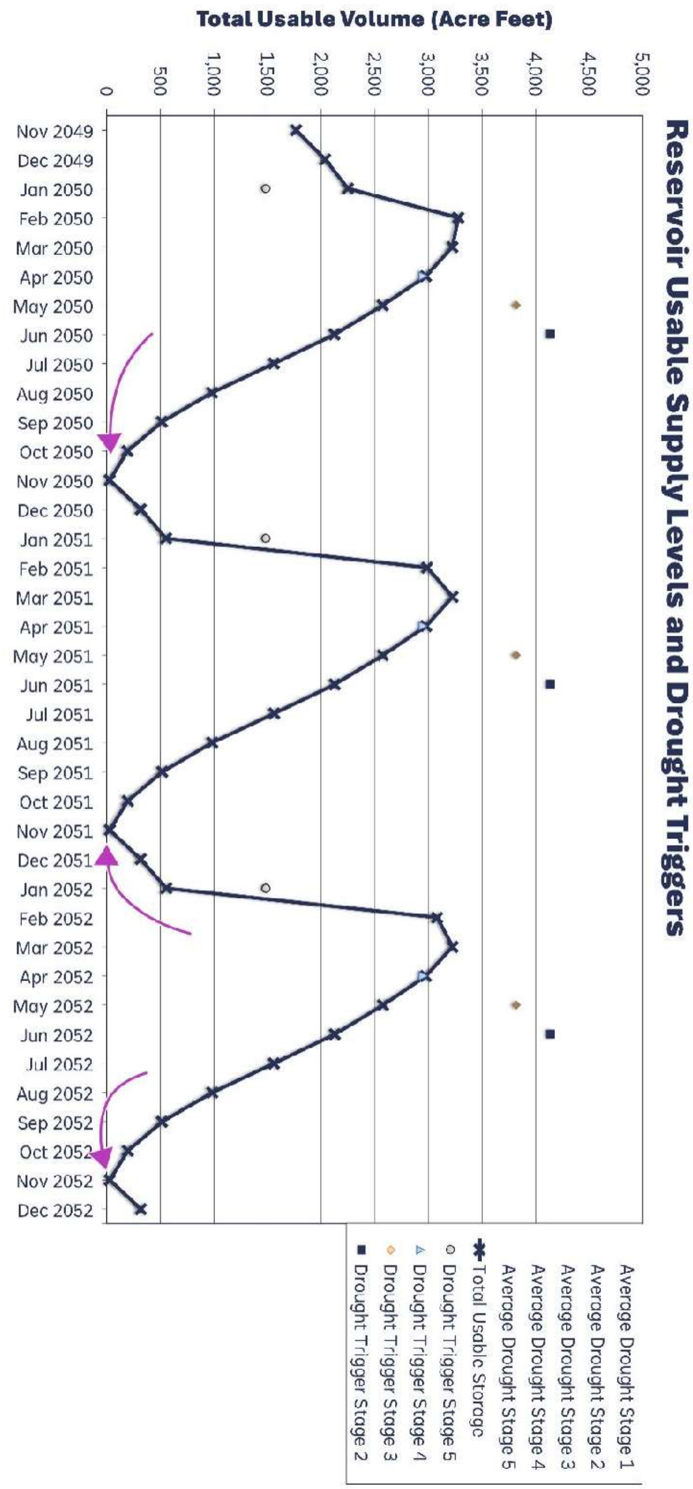


Figure 5-5 shows that there is adequate water to meet current demands under Scenario 1. Figure 5-6 shows that at buildout demand, there is less than 100 AF of remaining supply left under Scenario 1.

5-2.3.b. Scenario 2: Worst-Case Drought Year

In this scenario, hydrology is for the worst-case drought year, reclaimed water serves planned connections and existing double-plumbed connections, the golf courses are served by raw river water, Clementia reservoir is not used for storage, and a drought plan is implemented with 30% cutbacks at Stage 4 and Stage 5. See Figure 5-7 for model results under current demands and Figure 5-8 for results under buildout demands.

Figure 5-7: Scenario 2, Existing Demands

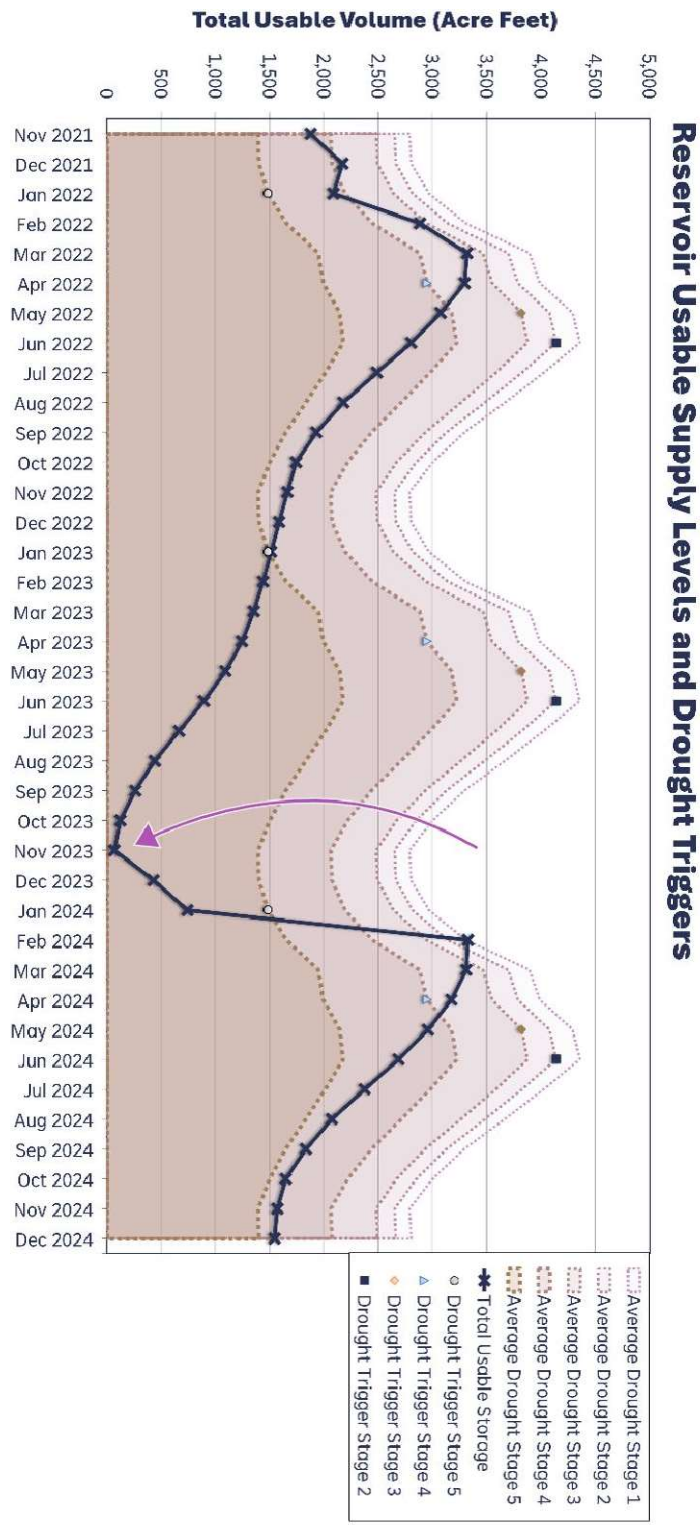


Figure 5-8: Scenario 2, Buildout Demands

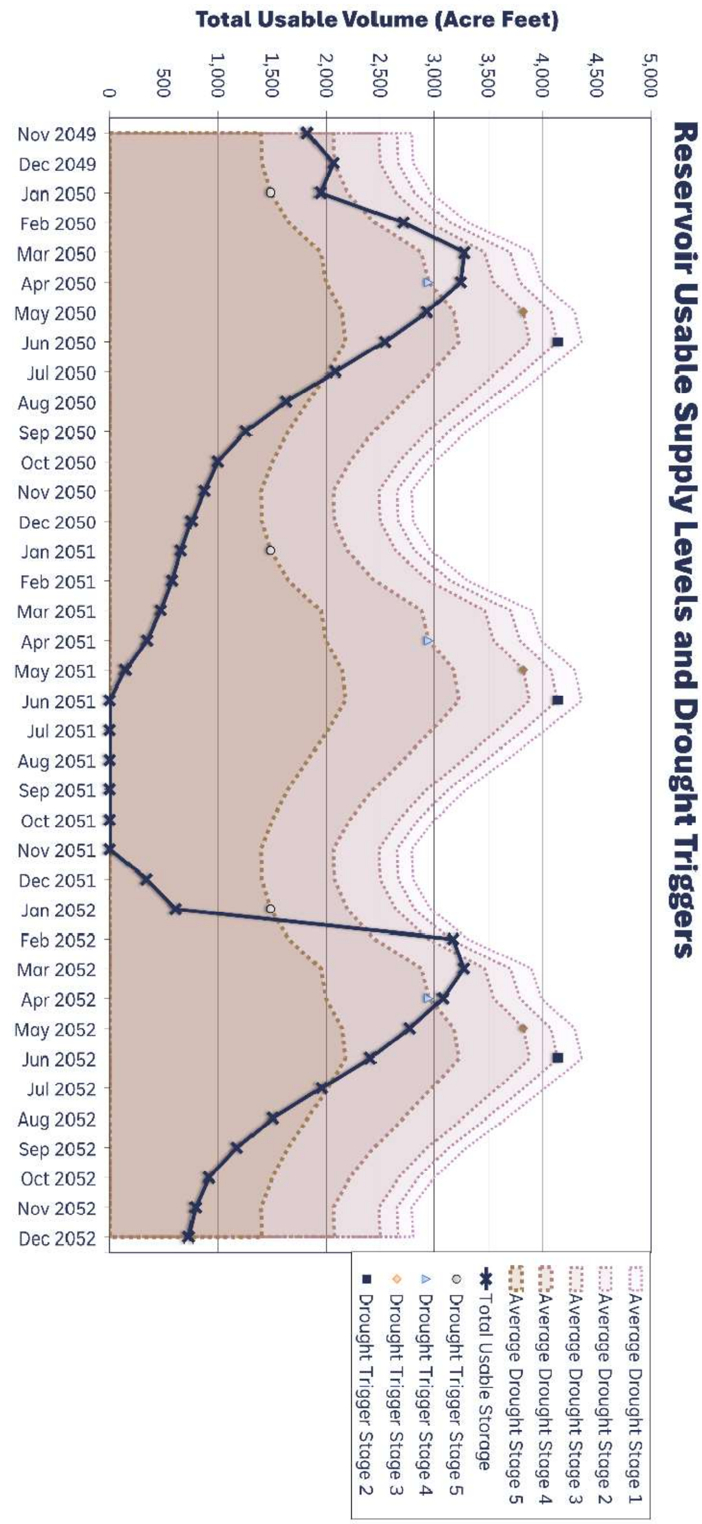


Figure 5-7 shows that at current demand under Scenario 2, the system reaches close to zero supply levels. Figure 5-8 shows that at buildout demand under Scenario 2, the system runs out of water with significant deficits. See Figure 5-9 for the system supply shortfall under this scenario.

Figure 5-9: Scenario 2, Buildout Demands Shortfall

5-2.3.c. *Scenario 3: Worst-Case Drought with Supply Augmentation*

The previous sections and figures identify water stress in an average year scenario and significant stress in a worst-case drought scenario. This section presents multiple scenarios with different supply augmentation options to address the water stress identified. These scenarios assume that the entire reclaimed water demand discussed in Chapter 4 is being met with reclaimed water, with no domestic water supplementation. Under this scenario, the golf courses would be pumping directly from the river to supplement their allotment of reclaimed water.

Figure 5-10: Scenario 3c, 2000 GPM Well & Clementia Offline

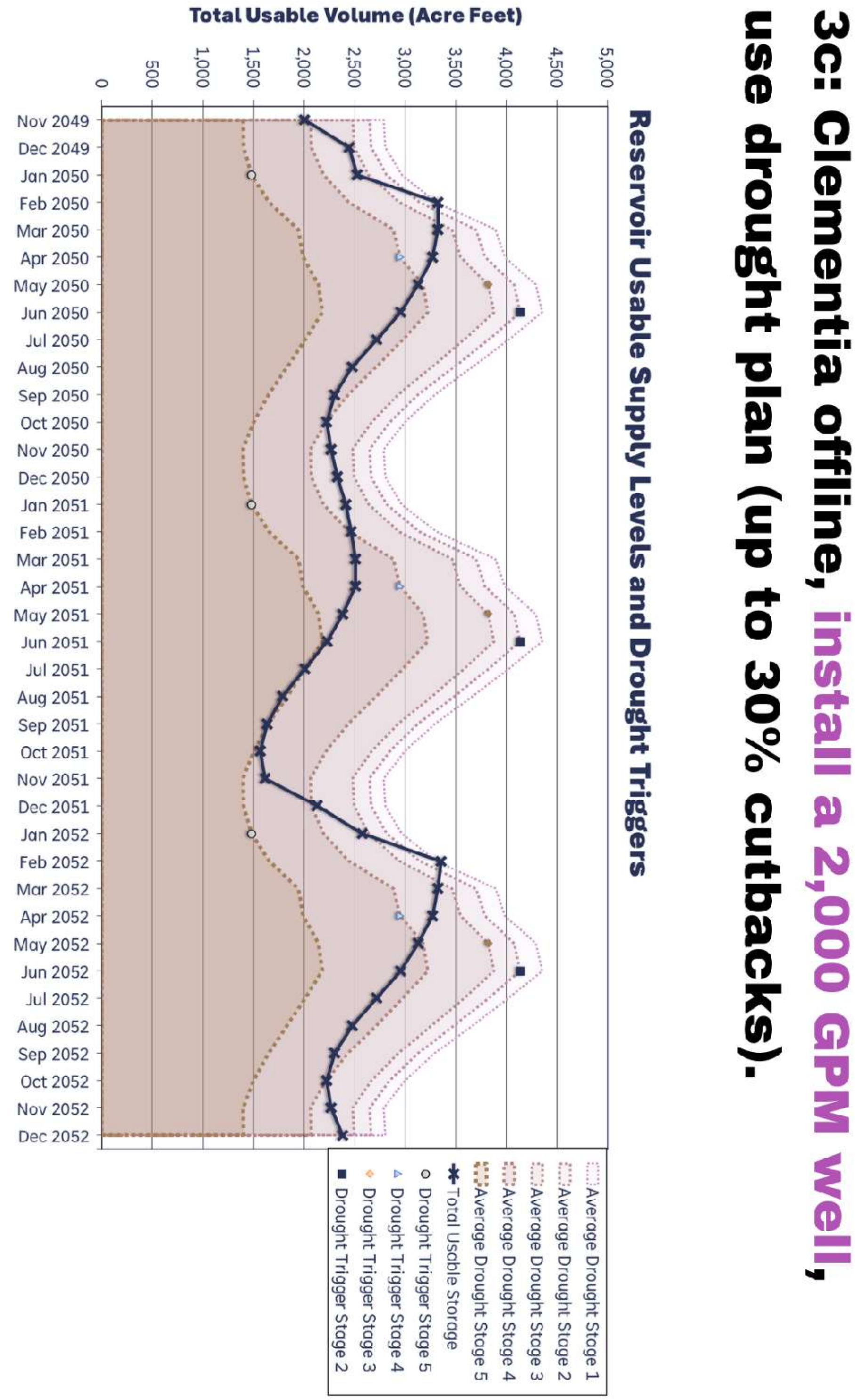


Figure 5-11: Scenario 3d, 1200 GPM Well & Clementia Online

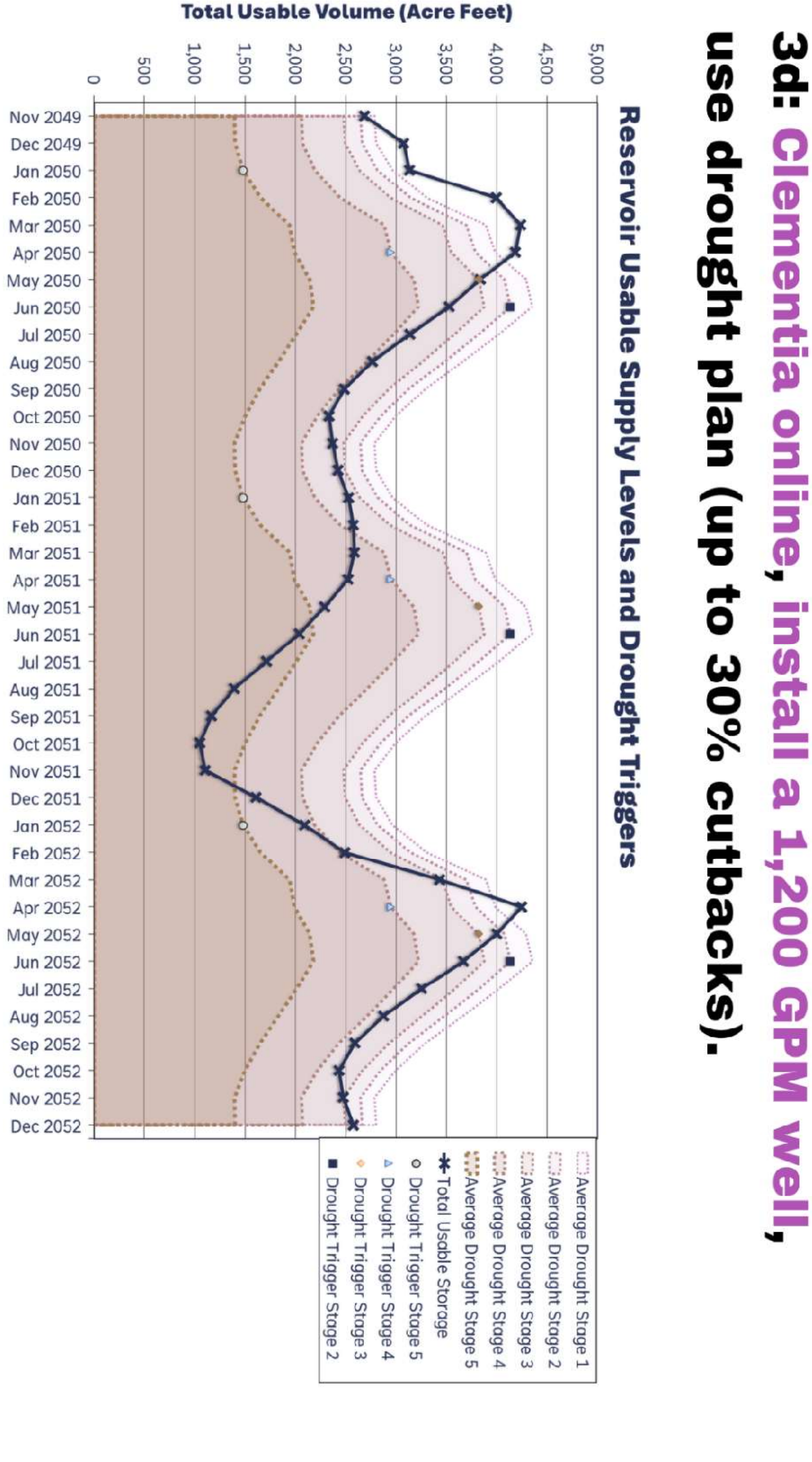


Figure 5-10 shows that installing wells capable of 2,000 gpm along with up to 30% usage cutbacks would meet the system's needs, without the need of Clementia's storage. Figure 5-11 shows that 1,200 gpm of well supply would be sufficient to meet supply needs if Clementia was used for storage.

5-2.3.d. Summary of Augmentation Options

The sections above presented the circumstances under which the water system would undergo stress and the potential solutions identified as part of the engineering assessment. What follows is a more detailed discussion of those solutions (i.e. installing wells, using Clementia for supply, and cutbacks).

- Clementia Option: If Clementia Reservoir is not able to be licensed as a potable water reservoir, there are two alternatives available to use water stored in Clementia Reservoir as part of the system supply. One of these is for the District to apply for a statutory exemption from the California Health and Safety Code. Other reservoirs that have obtained this exemption include Sly Parks Reservoir in El Dorado County, all the reservoirs in San Diego County, the Nacimiento Reservoir in San Luis Obispo County, and Canyon Lake Reservoir in Riverside County to name a few. The other alternative is to apply recreational use restrictions similar to Calero and Chesbro reservoirs.
- Groundwater Option: The option to increase supplies through the installation of wells provides flexibility in supply resources and helps meet the requirements of Senate Bill 552 Back-up supply law. In the scenarios depicted through the figures, the options mention "install a 1,200 GPM well" or install a "2,000 GPM well". This was done to keep the presentation of these options more streamlined, but the term "well" in the options slides refers to what may be a series of wells that need to be installed. See Chapter 6 for a further discussion of groundwater alternatives. Long term groundwater supply augmentation may be explored, including Aquifer Storage and Recovery (ASR) well construction.

- More Curtailment Option: In the extreme "worst-case" drought there is potential to conserve more water to increase supply availability. A 50% reduction in water use would save a total of about 850 AF throughout a drought year (roughly the usable storage volume of Clementia Reservoir which is ~900 AF) . This conservation effort would comprise achieving an indoor use of about 50 GPCD and an outdoor use of about 75 GPCD.

In summary, the scenario testing helped identify circumstances that stress the system at current demand and buildout demand. A summary of options to address the stress to the system is presented in Table 5-5 below.

Table 5-5: Summary of Water Supply Modeling Scenarios

Water Supply Modeling Scenarios								
Scenario	**1a**	**1b**	**1c**	**2a**	**2b**	**3a**	**3c**	**3d**
Hydrology	Avg Year	Avg Year	Avg Year	Hist. Drought	Hist. Drought	Hist. Drought	Hist. Drought	Hist. Drought
Demand	Current	Buildout	Buildout	Current	Buildout	Current	Buildout	Buildout
Use Clementia	No	No	Yes	No	No	No	No	Yes
Additional Source	No	Backup Needed	No	Backup Needed	Necessary	1,200 gpm	2,000 gpm	1,200 gpm
Outcome	Meets demand and SB552	Does not meet SB552	Meets demand and SB552	Does not meet SB552	Significant shortfall	Meets demand and SB552	Meets demand and SB552	Meets demand and SB552

This section demonstrates that for buildout demands and drought scenarios, the District needs to pursue additional supply sources, add Clementia as a storage facility, and/or consider extreme drought curtailment measures in order to ensure adequate water supplies for its customers. Project alternatives are discussed in Chapter 6.

5-3. Water Treatment Evaluation

5-3.1. Treatment Plant Capacity

The two WTPs have a total of 8 pumps (5 at WTP1 and 3 at WTP2) that move water from the clearwells to the two storage tanks. The current production capacity of WTP1 is 4.0 MGD or 2,778 gpm. The current production capacity of WTP2 is 2.0 MGD, or 1,389 gpm. The total WTP capacity is the combination of these two, or 6.0 MGD/4,167 gpm. Water supply pumps are generally designed to meet peak daily demands without having to provide 24-hour service. It is preferrable that pumps operate a maximum of 18 hours per day to allow for maintenance when necessary. The existing pumps at the two WTPs can meet the District's current MDD of 1,936 gpm by operating for just over 11 hours per day. The MDD at buildout is estimated to be 5.5 MGD/3,817 gpm. The combined pumps would need to operate for approximately 22 hours per day during the estimated peak demand at buildout. This suggests that the existing WTP operation and capacity are adequate but may not operate under ideal conditions during peak demand periods at buildout.

5-3.2. Groundwater Supply

California Senate Bill 552 (SB 552) requires that small water suppliers, defined as 3,000 connections or fewer, must have a backup supply source, either a groundwater well or intertie to a neighboring system. Adkins evaluated the availability of groundwater as a part of the IWMP process and published a Technical Memorandum in May 2024 that summarizes the available literature on the topic. The memo concludes that groundwater is likely available, but any wells constructed are only feasible as backup or emergency sources, not for long-term supply for the District.

At 3,000 connections, the District's ADD was calculated to be 1,169 GPM. This is considered the existing conditions. At buildout conditions, the ADD is 2,097 GPM. Based on prior work by Dunn Environmental (DE) in 2013, test hole locations on the southwest side of the District could produce potential well yields ranging from 150 to 500 GPM. It is assumed that three wells are required to produce 1,169 GPM, and five wells to produce 2,097 GPM. Each of these wells would need to be drilled to a total well depth of 500 feet to meet the appropriate depth within the water bearing zones. These are discussed as alternatives in Chapter 6.

5-4. Treated Domestic Water Storage Evaluation

Calculating required storage involves estimating the volume of several required storage components. These include operational, equalization, emergency, and fire reserve storage components. Required storage was calculated based on the District's design criteria, discussed in Chapter 3.

5-4.1. Fire Reserve Storage

Reserve storage for fire suppression is usually determined from either the recommendation of the Insurance Services Office Commercial Risk Services, Inc., the recommendation of a city's fire chief, or calculations from the building code. In the District, the largest required fire flow was used to determine the maximum required fire reserve storage. This was determined to be the Murieta Inn and Resorts, and its required fire flow is stated on its plans and designs. This flow is 2,625 GPM for 4 hours, resulting in a maximum theoretical fire reserve storage volume of 630,000 gallons.

5-4.2. Emergency Storage Reserve

Emergency storage is provided to supply water in the event of a power outage, mechanical problem, or other system failure that would interrupt the supply of water. This is intended to cover the amount of time required to repair the faulty component. While emergency storage reserves are not a regulated requirement for municipalities, it is generally reasonable to maintain between one- and three-days' supply of emergency reserves. This amount is decided by the water supplier. These reserves assume that a water supply source will be available to fill the tank within the decided timeframe after a water supply source failure. Maintaining emergency reserves could be critical due to the District's total reliance on WTP pumps to consistently meet water demands. The District has chosen 1.75 days of ADD as the emergency storage criteria. To provide an emergency reserve of 1.75 days of ADD, a total emergency storage volume of 2.68 MG would be required for the existing conditions and 5.28 MG for buildout conditions.

5-4.3. *Operational Storage*

Operational storage is generally provided to facilitate operation of pumps in a water system. For example, when water system demands result in the water level lowering in a tank, the water level will reach a certain point that triggers activation of pumps to refill the tank. The storage needed to activate water supply sources is typically referred to as operational storage. This zone of operation can be set as desired but is often set to facilitate tank mixing during each pump run cycle. This allows water to cycle through the tank to help maintain water quality by preventing stagnation, while keeping the tank as full as possible. The current zone of operation for each tank is 2.0 feet. Thus, the calculated operational storage volume for existing conditions is 255,858 gallons. The operational storage at buildout is dependent on the size and number of tanks at buildout. For the alternatives suggested in Chapter 6, the operational storage is 450,314 gallons.

5-4.4. *Equalization Storage*

Equalization storage must be provided to supply the difference between peak hour demand and water supply capacity during high flow periods. The method for estimating the required equalization storage uses the difference between the peak hour flow and the peak water supply availability for a specific number of peak hours per day. The District's current available supply flow of 4,167 GPM from the WTP exceeds the existing peak hourly flow, so the equalization storage for existing conditions is zero. Based on 2.5 peak hours for the estimated buildout peak flow of 7,370 GPM, the required buildout equalization storage would be 532,271 gallons.

5-4.5. *Storage Mixing*

An important part of storage performance is the ability for water to mix within a storage tank. This prevents water from becoming stagnant in the tank and prevents chlorine residuals from dropping below allowable levels. CCR Title 22, § 64585 (b)(4) states that storage tanks shall be "equipped with at least one separate inlet and outlet…designed to minimize short-circuiting and stagnation of the water flow through the [tank]." Van Vleck currently has a connection that acts as both an inlet and an outlet, receiving water from the WTP and also discharging water to Rio Oso during different demand scenarios. This is not in conformance to the CCR requirement for separate inlet and outlet ports.

5-4.6. Global Storage Evaluation

The design criteria developed in Chapter 3 indicate that the District has storage approximately equal to its requirement. To satisfy the storage design criteria under buildout conditions, the District would need approximately 3.1 MG of additional storage. These values represent the District's overall storage needs, or global storage. A summary of the global storage evaluation is shown below in Figure 5-12. See the following section for a discussion of the District's local storage requirements.

RANCHO MURIETA COMMUNITY SERVICES DISTRICT

WATER SYSTEM GLOBAL STORAGE EVALUATION

	Year 2023	Year 2043
Residential Service Connections	2,629	4,189
Commercial Service Connections	100	119
Design Population [1]	6,939	10,492
Supply		
Average Daily Volume (gpd)[2]	1,531,172	3,019,094
Average Daily Demand (gpcd)	221	288
Average Daily Flow Rate (gpm)	1,063	2,097
Max Daily Volume (gpd)[3]	2,788,264	5,497,769
Max Daily Demand (gpcd)	402	524
Max Daily Flow Rate (gpm)	1,936	3,818
Peak Hourly Flow (PHF)[4] (gpm)	3,913	7,715
Supply Flow Required[6] (gpm)	1,936	3,818
Estimated Available Supply Flow[7] (gpm)	4,167	4,167
Fire Flow[8]		
Residential (gpm)	1,500	1,500
Duration (hrs)	2	2
Murieta Inn (gpm)	2,625	2,625
Duration (hrs)	4	4
Storage		
Equalization Storage (gal)[9]	0	532,271
Operating Storage[5]	255,858	451,853
Fire Reserve (gal)	630,000	630,000
Emergency Reserve (gal)[10]	2,679,551	5,283,414
Total of Storage Components	3,565,409	6,897,537
Existing Storage Capacity	3,837,875	3,837,875
Potential Additional Storage Needed	**-272,466**	**3,059,662**

Notes:

[1]Existing design population by US Decennial Census (2020) with values interpolated using number of households, number of active accounts, and persons per household. Projected population based on number of approved accounts in development phases and persons per household by account/lot type.

[2]Average daily volume determined by billed water use for years 2020-2022, with 12% added for non-revenue water. Year 2043 adds estimated buildout demand, 10% to account for climate change based demand increases, and 12% for non-revenue water.

[3]Peak day factor of 1.82 determined by Max Day Demand and Average Day Demands for 2020-2022.

[4]Max hour, 7/20/2022, from SCADA report. 2043 max hour is the ratio of buildout ADD to existing ADD, multiplied by existing max hour.

[5]Equal to the volume of two feet of storage in the existing tanks and proposed tanks at buildout.

[6]Max daily volume conversion to gallons per minute.

[7]Max capacity of WTP1 and WTP2.

[8]RMCSD follows the California Fire Code on fire flows.

[9]Difference between peak hourly flow and available supply flow for a 2.5-hour period. If the available supply is higher than the peak hourly flow, 0 is used.

[10]42 hours (1.75 days) supply at average daily demand, per Director of Operations.

Abbreviations:

gal = gallons
gpcd = gallons per capita day
gpd = gallons per day
gpm = gallons per minute
hrs = hours

GLOBAL STORAGE EVALUATION for RMCSD

FIGURE 5-12

5-4.7. Local Storage Evaluation

The District's operating rules for the tanks allow Van Vleck to provide operational volumes to Rio Oso, and this happens regularly. However, the District does not want to rely on the tanks' ability to supplement each other, since this would not be possible if the transmission line that connects the tanks and the WTP were to fail. For this reason, storage requirements and capacity were evaluated for both the existing and proposed pressure zones individually. Each zone was evaluated for fire, emergency, operational, and equalization storage requirements. See Figure 5-13 and Figure 5-14 for maps of the existing and buildout pressure zones, respectively.

RANCHO MURIETA EXISTING DISTRIBUTION SYSTEM

700 0 350 700
(FEET)

SITE PLAN NOTES

1. NO TOPOGRAPHIC SURVEY WAS PERFORMED. ALL LOCATIONS OF UTILITIES SHOWN ARE APPROXIMATE. ACTUAL SITE CONDITIONS AND LOCATIONS MAY VARY.
2. THIS IS A SCHEMATIC SITE PLAN FOR THE SOLE PURPOSE OF ILLUSTRATING THE GENERAL COMPONENTS AND FACILITIES. ACTUAL SITE LAYOUT AND LOCATIONS MAY VARY AND WILL NEED TO BE DETERMINED BY TOPOGRAPHIC SURVEY AND OTHER SITE INVESTIGATIONS AS REQUIRED.

LEGEND

EXISTING WATER LINE

EXISTING FIRE HYDRANT

EXISTING RIO OSO PRESSURE ZONE

ALL AREAS OUTSIDE OF THE RIO OSO PRESSURE ZONE ARE PART OF THE VAN VLECK GRAVITY ZONE.

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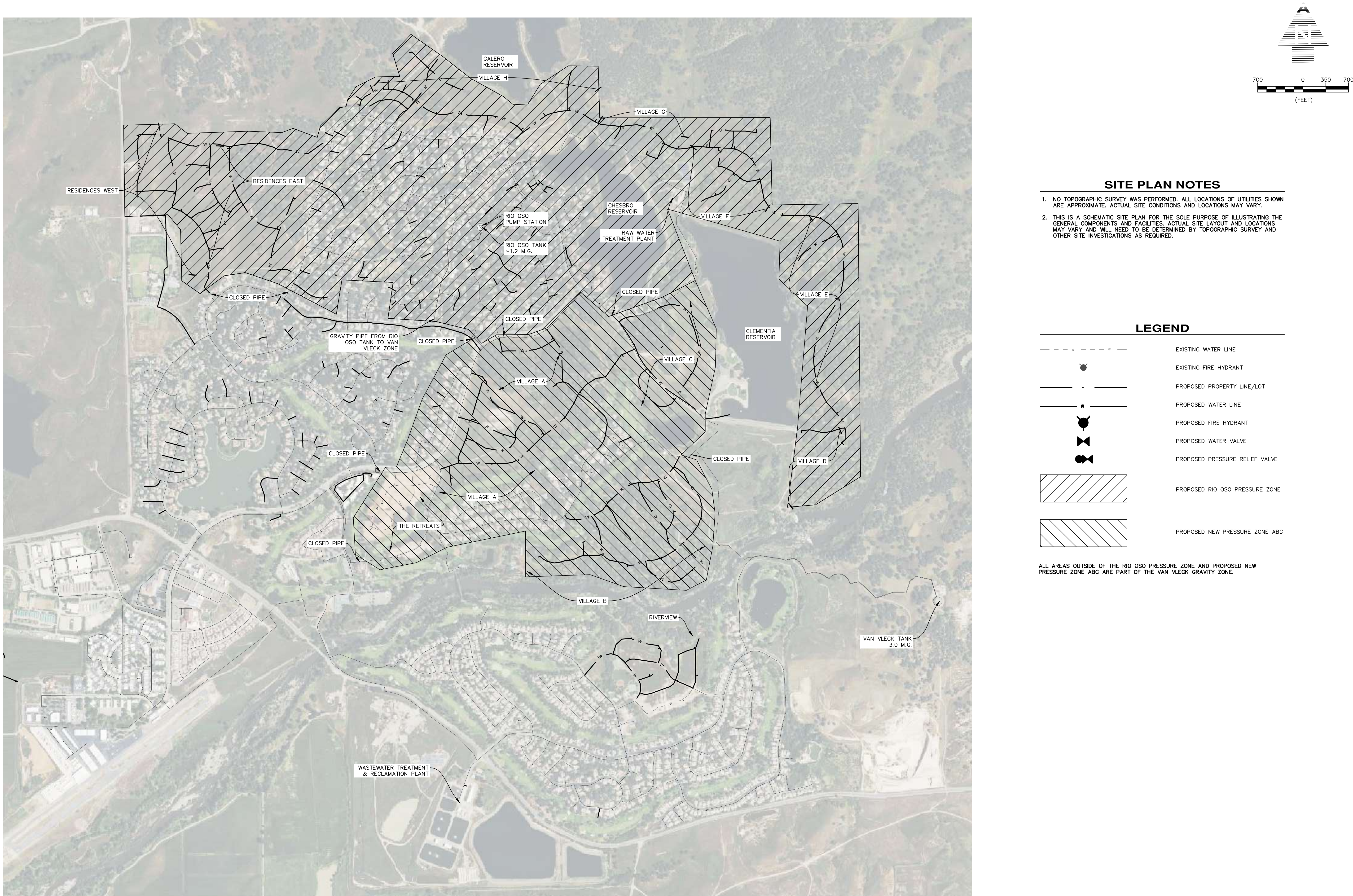
DISTRIBUTION AND STORAGE ALTERNATIVES
FOR
RANCHO MURIETA CSD
EXISTING PRESSURE ZONES

SCALE	SHOWN
DATE	08/08/2024
DRAWN BY	BAG
PROJ. NO.	3971-02

FIGURE
5-13



RANCHO MURIETA PROPOSED WATER DISTRIBUTION SYSTEM

SITE PLAN NOTES

1. NO TOPOGRAPHIC SURVEY WAS PERFORMED. ALL LOCATIONS OF UTILITIES SHOWN ARE APPROXIMATE. ACTUAL SITE CONDITIONS AND LOCATIONS MAY VARY.
2. THIS IS A SCHEMATIC SITE PLAN FOR THE SOLE PURPOSE OF ILLUSTRATING THE GENERAL COMPONENTS AND FACILITIES. ACTUAL SITE LAYOUT AND LOCATIONS MAY VARY AND WILL NEED TO BE DETERMINED BY TOPOGRAPHIC SURVEY AND OTHER SITE INVESTIGATIONS AS REQUIRED.

LEGEND

EXISTING WATER LINE
EXISTING FIRE HYDRANT
PROPOSED PROPERTY LINE/LOT
PROPOSED WATER LINE
PROPOSED FIRE HYDRANT
PROPOSED WATER VALVE
PROPOSED PRESSURE RELIEF VALVE
PROPOSED RIO OSO PRESSURE ZONE
PROPOSED NEW PRESSURE ZONE ABC

ALL AREAS OUTSIDE OF THE RIO OSO PRESSURE ZONE AND PROPOSED NEW PRESSURE ZONE ABC ARE PART OF THE VAN VLECK GRAVITY ZONE.

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DISTRIBUTION AND STORAGE ALTERNATIVES FOR RANCHO MURIETA CSD
PROPOSED PRESSURE ZONES

SCALE	SHOWN
DATE	08/08/2024
DRAWN BY	BAG
PROJ. NO.	3971-02

FIGURE 5-14

5-4.7.a. Local Storage Existing Conditions

Demands for the existing pressure zones were estimated by adding all the demand nodes in the model that are within the respective zones. Of the existing demand, 26% is estimated to be in the Rio Oso pressure zone, and the remaining 74% is estimated to be in the Van Vleck gravity zone. As described above, Van Vleck can supplement water to Rio Oso through the transmission line that connects them both to the WTP, and this happens regularly. Similarly, Rio Oso can provide water to the Van Vleck zone through a gravity discharge line that connects to the Van Vleck zone, if Van Vleck's level drops to Rio Oso's level. The operation of this gravity line from Rio Oso to Van Vleck is managed manually by District staff.

While it can be operationally advantageous to have the tanks supplement each other, the District has indicated that they want to move away from having the storage tanks be dependent on each other for adequate capacity. The current arrangement makes the system's storage capacity vulnerable to catastrophic failure; if the transmission line between the tanks and the WTP were to become compromised, the tanks would not be able to supplement each other. For this reason, the District wants each pressure zone to have all its storage requirements satisfied by a tank that is dedicated to that zone, without reliance on tanks in other zones. While the tanks will still be able to supplement each other unless something fails, this will limit the system's exposure to a catastrophic failure. See Table 5-6 below for an evaluation of the existing pressure zones' separate storage capacity.

Table 5-6: Storage Evaluation by Zone - Existing

Existing Zone Storage Evaluation (gallons)			
Storage Type	**Rio Oso**	**Van Vleck**	**Combined**
[1]Emergency	692,450	1,987,101	2,679,551
[2]Operational	75,197	180,661	255,858
[3]Fire	180,000	630,000	810,000
[4]Equalization	0	0	-
Total Required	947,647	2,797,762	3,745,409
Existing Capacity	1,127,957	2,709,918	3,837,875
Excess/(Deficiency)	180,310	(87,845)	92,466

[1]Emergency storage required was calculated based on 74% of existing demand in Van Vleck zone and 26% in Rio Oso zone. Existing customer demands were increased by 12% for NRW and another 10% for the climate change contingency, per Chapter 4. This value represents 1.75 days of ADD.

[2]Operational storage is the volume of 2 feet in the existing tanks.

[3]Rio Oso only serves residential customers, so 1500 GPM for 2 hours was used for the required fire storage.

[4]Since the available supply flow from the WTP exceeds the existing peak hour flow, no equalization storage is required.

The table shows that under existing conditions and the stated storage criteria, the Van Vleck zone needs 87,845 gallons of additional storage to be self-sufficient.

5-4.7.b. Buildout Conditions

As shown in Figure 5-14 above, Villages A, B, and C, along with the Retreats and some existing residences along De La Cruz Drive are proposed to comprise a new pressure zone, called Zone ABC in this IWMP. Villages D, E, F, G, and H, and the Residences East and West are proposed to be added to the existing Rio Oso pressure zone. Riverview and the new commercial developments anticipated in Murieta Gardens are proposed to be added to the Van Vleck gravity zone. The estimated storage needs for each of the three proposed buildout zones are shown in Table 5-7 below.

Table 5-7: Storage Evaluation by Zone - Buildout

Buildout Zone Storage Evaluation (gallons)				
Storage Type	**Rio Oso**	**Van Vleck**	**New Zone ABC**	**Combined**
[1]Emergency	1,632,352	2,920,429	707,227	5,260,008
[2]Operational	124,839	275,833	51,181	450,314
[3]Fire	180,000	630,000	180,000	990,000
[4]Equalization	165,181	295,524	71,566	532,271
Total Required	2,102,372	4,121,786	1,009,974	7,232,592
Existing Capacity	1,127,957	2,709,918	0	3,837,875
Existing Excess/(Deficiency)	(974,414)	(1,411,868)	(1,009,974)	(3,394,717)
[5]Proposed New Capacity	992,838	1,427,571	1,023,621	3,413,246
Proposed Total Capacity	2,120,795	4,137,489	1,023,621	7,251,121
Excess/(Deficiency)	18,423	15,703	13,648	18,529

[1]Emergency storage required was calculated based on demand estimates for each zone. These account for NRW and the 10% climate change contingency, per Chapter 4. These values represent 1.75 days of ADD.

[2]Operational storage is the volume of 2 feet in the proposed/existing tanks.

[3]Rio Oso and Zone ABC only serve residential customers, so 1,500 GPM for 2 hours was used for the required fire storage.

[4]The total system peak hour flow at buildout is estimated to be 7,715 GPM. The maximum supply flow from the WTP is 4,167 GPM. This results in a global equalization storage requirement of 532,271 gallons. This was prorated to each pressure zone by proportion of total demand.

[5]See Chapter 6 for proposed storage improvement alternatives.

The table above shows that under buildout conditions, the Rio Oso pressure zone requires approximately 1.0 MG of additional storage, the Van Vleck gravity zone requires approximately 1.4 MG of additional storage, and the new Zone ABC requires approximately 1.0 MG of storage. These storage volumes would provide the District with much greater storage resiliency, with each zone able to provide adequate storage for itself, independent of the rest of the system. Figure 5-12 shows the total required storage for the system as 6,895,998 gallons, while the table above shows the total required storage total as 7,251,121 gallons. The reason for this discrepancy is that the global evaluation only considers the maximum fire event for the entire system, which requires 630,000 gallons of storage, while the local evaluation considers the maximum fire in each zone. This results in 360,000 additional gallons being added to the local evaluation, with

Rio Oso and Zone ABC each having their own independent fire storage. The rest of the discrepancy is due to rounding in the demand calculations.

5-5. Water Distribution System Evaluation

5-5.1. Fire Flows

Generally, required fire flows follow the IFC based on building size, intended number of persons occupying the space, construction materials, availability of installed fire suppression technologies such as automatic sprinklers or foams, and more. For the sake of this IWMP, and without conducting a detailed fire engineering analysis which is outside of the scope of this analysis, 1,500 GPM for 2 hours was selected as the criteria for evaluating the distribution system's adequacy for a fire. 1,500 GPM for 2 hours is the fire flow requirement for residential areas. 2,625 GPM for 4 hours, which is the fire flow requirement for the hotel, was selected as the criteria for storage, since this is the largest fire flow required in the District.

Fire nodes were selected throughout the model based on global trends. For example, the residential area along De La Cruz Drive had consistently low pressures (below 30 psi) during normal modeling due to its relatively high elevation in the Van Vleck gravity zone. Four zones were identified as global concerns during normal modeling, and fire nodes were selected in these zones to observe the effects of fire flows. These are summarized in Table 5-8 below and the deficiencies identified were used to develop alternatives for improving the distribution system, discussed in Chapter 6.

Table 5-8: Zones Tested for Fire Flow

Global Areas of Concern During Fire Flows				
General Location	**Nodes Tested**	**Nearby Nodes**	**Pressure (psi)**	**Notes**
Top of De La Cruz Drive	n157 FH-283	FH-284 n157	8.5 6.4	Entire neighborhood drops below 20psi during fire flows.
Guadalupe Drive between Rio Oso and Murieta Parkway	n348	n375	16.6	4 nodes in this area drop below 20 psi during fire flows.
Top of hill near Equestrian Center	n192	n612	9.9	Only this node drops below 20 psi.
Stonehouse Park, Escuela Drive	n421	PARK_02	0.1	Hydrant node and park node drop below 20 psi.

5-5.2. Pressure and Service to Customers

The District currently has two pressure zones that serve the population: the Rio Oso pressure zone and the Van Vleck zone, which is controlled by gravity. Water levels in Van Vleck control the pressures in its zone, and the Rio Oso tank and booster station control the pressures in the Rio Oso zone.

A minimum pressure of 20 psi under all conditions is required by the California Water Resources Control Board (WRCB) Drinking Water Program (DWP) and the 2022 California Plumbing Code recommends a maximum pressure at point of service of no more than 80 psi. Typically, pressures in the distribution system should be higher than the minimum pressure suggested by the DWP and can be slightly higher than the maximum residential pressure suggested by the Plumbing Code. Minimum distribution system pressures are generally considered to be 20 psi at the customer's property line, as suggested by the DWP.

Hydrant testing shows that pressures are regularly above the 80 psi threshold in the Rio Oso pressure zone and in Murieta Gardens, the mixed-use commercial development in the southwestern part of District, which is part of the Van Vleck gravity zone. Pressures that exceed 80 psi can damage water infrastructure and often require pressure regulators installed at the home.

5-5.3. Fire Hydrant Coverage

Hydrant coverage rules come from the 2015 IFC, Appendix C, Sections 101 through 105. The minimum number of adjacent hydrants and maximum spacing for hydrants are dependent on fire flow requirements for individual buildings or areas containing many buildings. Because a detailed fire engineering analysis is outside the scope of this IWMP, it was assumed that all residential buildings require no more than 1,500 GPM fire flows and all commercial and industrial buildings are properly equipped to meet IFC standards regarding additional fire suppression technologies such as automatic sprinklers, foams, and more. Thus, the maximum distance between hydrants is 500 feet, or a 250-foot radius around each hydrant.

The existing District system has some gaps in fire coverage, especially along dead-end lines in the Rio Oso pressure zone. A map of existing fire hydrant coverage is provided in Figure 5-15. Additional hydrants are included in the alternatives developed to improve the existing distribution system, discussed in Chapter 6.

RANCHO MURIETA EXISTING FIRE HYDRANT COVERAGE AND DISTRIBUTION SYSTEM

600 0 300 600
(FEET)

FIRE COVERAGE NOTES

1. FIRE COVERAGE REQUIREMENTS FROM 2015 INTERNATIONAL FIRE CODE (IFC), APPENDIX C, SECTIONS 101 – 105.
   1.1. SECTION 102.1 STATES THAT MINIMUM NUMBER OF ADJACENT HYDRANTS AND MAXIMUM SPACING FOR HYDRANTS ARE DEPENDENT ON FIRE–FLOW REQUIREMENTS FOR AN INDIVIDUAL BUILDING OR AREA CONTAINING MANY BUILDINGS.
   1.2. THE SPACING DEPICTED HEREIN OPERATES UNDER THE ASSUMPTION THAT RESIDENTIAL BUILDINGS REQUIRE NO MORE THAN 1,750 GPM FIRE–FLOW, AND THAT INDUSTRIAL AREAS ARE PROPERLY EQUIPPED TO MEET IFC STANDARDS REGARDING INSTALLED FIRE SUPPRESSION TECHNOLOGIES.
   1.3. SECTION 103.1 STATES THAT FIRE APPARATUS ACCESS ROADS AND PUBLIC STREETS PROVIDING REQUIRED ACCESS TO BUILDINGS IN ACCORDANCE WITH SECTION 503 OF THE INTERNATIONAL FIRE CODE SHALL BE PROVIDED WITH ONE OR MORE FIRE HYDRANTS, AS DETERMINED BY SECTION C102.1.
   1.4. SECTION 104.1 STATES THAT EXISTING FIRE HYDRANTS ON PUBLIC STREETS ARE ASSUMED TO BE CONSIDERED AS AVAILABLE TO MEET THE REQUIREMENTS OF SECTIONS C102 AND C103.
2. THUS, THIS MAP DEPICTS HYDRANTS WITH A SPACING REQUIREMENT OF 500–FEET, WITH A MAXIMUM DISTANCE FROM ANY POINT ON STREET OR ROAD FRONTAGE TO A HYDRANT OF 250–FEET.
3. THE SPACING DEPICTED SHOULD ONLY BE USED FOR CONCEPTUAL AND PRELIMINARY DESIGNS. ACTUAL SITE CONDITIONS AND FIRE CODE REQUIREMENTS MAY VARY FROM THOSE USED IN THE DEVELOPMENT OF THESE FIGURES.

LEGEND

EXISTING WATER LINE

EXISTING FIRE HYDRANT

EXISTING FIRE HYDRANT COVERAGE RADIUS – 250 FEET

SITE PLAN NOTES

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DISTRIBUTION AND STORAGE ALTERNATIVES
FOR
RANCHO MURIETA CSD
EXISTING FIRE HYDRANT COVERAGE

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FIGURE
5-15

5-6. Reclaimed System Evaluation

5-6.1. Supply

The population of Rancho Murieta is expected to increase from 6,939 to 10,492 by the end of the planning horizon in 2044. Reclaimed water supply evaluations developed in this section utilize the District's current and projected wastewater production.

As developed in Chapter 2, the supply for reclaimed water at the District is wastewater returned to the WWRP. The amount of wastewater estimated is based on the indoor residential and commercial uses of the community and historic influent measurements, which include infiltration and inflow (I/I) contributions. Based on analysis of historic inflow data and projections of future production, the existing ADWF is about 0.39 MGD with a buildout estimate of 0.84 MGD.

When determining the amount of potential reclaimed water that can be produced by a WWRP, both user-generated wastewater flows and I/I flows should be considered. Infiltration refers to water other than sanitary wastewater that enters a system through pipes, joints, connections, and manholes that may be defective; inflow refers to water other than sanitary wastewater that enters the system from point sources such as roof, cellar, and foundation drains, manhole covers, connections to storm sewers, and catch basins that are connected to the sewer system. For many systems, I/I coincide with high rainfall events, indicating rainfall induced infiltration (RII) which results from rainfall saturated soils causing infiltration into the collection system through defective joints or pipes.

To estimate the amount of I/I that the WWRP will receive along with normal wastewater inflows, the Average Dry Weather Flow (ADWF) was determined. This flow represents the dry season; therefore, the measured inflows at the WWRP are assumed to include little to no I/I. For the purposes of this IWMP, the months of June through September were considered the dry weather months. Measured inflows during these months were averaged over time and determined to be 11.6 MG/month between 2012 and 2022.

I/I was estimated by comparing the wet weather months to the dry weather months and calculating the difference in flows as a percentage. From 2012-2022, average I/I was estimated to be 9.3% with a median value of 6.5%. 2017, an exceptionally wet year, had an estimated I/I of 29.5%, while the exceptionally dry years of 2013-2014 had a combined estimated I/I of 2.7%. The correlation between I/I and rainfall is an expected phenomenon. For the purposes of this IWMP and estimating future wastewater flows, I/I was estimated to be 9.06%. This will vary on a yearly basis depending on rainfall and other conditions.

Using a water balance approach that considered average rainfall, I/I estimates, pan evaporation, and drought modifiers to adjust these inflows and outflows in the water balance, a series of scenarios were evaluated for the global availability of reclaimed water in terms of annual supplies. These are summarized in Table 5-9 below.

Table 5-9: Reclaimed Water Balance Results

Reclaimed Water Balance Under Planning Scenarios				
Scenario	**Variable(s)[1,2]**	**ADWF (MGD)**	**Total Reclaimed Water Produced (AFY)**	**Max Secondary Storage Volume (AF)**
Base (Existing)	GPCD	0.402	437	277
Buildout, 1[3]	GPCD, Precipitation	0.840	1124	670
Buildout, 2[4]	GPCD, Precipitation	0.840	987	580
Buildout, 3[5]	GPCD, Precipitation	0.840	910	530
Buildout, 4[6]	GPCD, Precipitation	0.840	858	495

[1]Existing GPCD=43.08, Buildout target GPCD=42.0
[2]Average year precipitation, recent drought modifier=62.7%, historic drought modifier=37.3%, 100-year highest precipitation modifier=167%
[3]Buildout conditions under 100-year highest precipitation, assumes I/I=15.13%
[4]Buildout conditions under average precipitation years, assumes I/I=9.06%
[5]Buildout conditions under recent drought years, assumes I/I=5.68%

[6]Buildout conditions under historic drought years, assumes I/I=3.38%

A key takeaway from this table is that buildout conditions during an average year provide just over double the existing wastewater inflows, which in turn results in a greater availability of reclaimed water. Under average precipitation conditions for buildout, the potential amount of reclaimed water produced is 412 AFY greater than the District's obligations to provide 550 AFY to the golf courses. This indicates that the current supply of reclaimed water is adequate to meet the District's current obligations.

5-6.2. Storage

5-6.2.a. Secondary Treated Storage

As mentioned in Chapter 2, the WWRP includes two reservoirs to store secondary treated wastewater during the non-irrigation months. These reservoirs have a combined capacity of 728 AF with two feet of freeboard. Since the inflows to the WWRP are projected to be more than double the existing conditions at buildout, the adequacy of the existing storage capacity for secondary treated wastewater was evaluated. To perform the evaluation, a water balance spreadsheet was created for the WWRP. A water balance considers all inflows and outflows from a closed system, with the difference representing the change in storage. This water balance was based on the water balance included in the report titled *Recycled Water Program Preliminary Design Report*, published by Kennedy/Jenks Consultants in June 2017 (2017 PDR). The water balance in that report was updated to reflect current data and projections and is included in Appendix C. See Table 5-10 for a summary of the inputs for the water balance.

Table 5-10: Reclaimed Water Balance Inputs

Reclaimed Water Balance Under Planning Scenarios	
Inflows	
Wastewater ADWF	0.840 MGD per previous sections
Infiltration and Inflow	Average year = 9.06% of ADWF, multiplied by precipitation modifiers for each scenario (15.13% for 100-year high precip, 3.38% for worst drought)
Direct Pond Precipitation	Product of total pond surface area and precipitation (secondary treatment lagoons, secondary storage ponds, and RMCC irrigation lakes)
Site Runoff	Product of tributary area, runoff coefficient, and precipitation. WWRP = 7.5 acres & 0.9 coefficient, secondary storage reservoirs = 40 acres – current water surface area & 0.9 coefficient, irrigation lakes = 15 acres & 0.2 coefficient
Outflows	
Direct Pond Evaporation	Product of total pond surface area, pan evaporation, and pan evaporation coefficient (secondary treatment lagoons, secondary storage ponds, and RMCC irrigation lakes)
Seepage	Assumed to be negligible due to ponds being lined
Irrigation	Sum of golf courses, proposed new residential/commercial, and Van Vleck. For storage “worst-case” (100-year high precip), assumed 550 AF to GCs, 215 AF to Van Vleck, and remainder to residential/commercial. Monthly percentages of total annual developed from historic GC demands.
Other Data	
Average Precipitation	Historic data provided by the District.
100-Year Precip	Log-Pearson Type III analysis of 112 years of data at station Sacramento 5 ESE and 28 years of data at WWRP site. Both resulted in ~35 inches.
Evaporation	Historic data provided by the District
Areas	As-built data and Google Earth

Using the inputs summarized above, the water balance analysis suggests that the existing secondary treated effluent storage capacity is sufficient for the 100-year highest precipitation. The highest effluent storage anticipated in the 100-year scenario is 670 AF, as shown in Table 5-9 above. The existing storage capacity with two feet of freeboard is 728 AF.

The primary reason that this water balance resulted in different maximum storage requirements from the 2017 PDR is the different estimate for 100-year precipitation. The 2017 PDR estimated 45.3 inches of precipitation in the 100-year scenario. There is no documentation in that report explaining the method used to estimate this value. For this IWMP, the 100-year precipitation amount was estimated using the Log-Pearson Type III method and two separate data sets. The first data set used was the historic precipitation at NOAA Station Sacramento 5 ESE for the past 112 years. The Log-Pearson method estimated 34 inches to be the amount with a 1% exceedance probability. The second data set used was the rainfall data measured at the WWRP site during the past 28 years. The Log-Pearson method estimated 35 inches to be the amount with a 1% exceedance probability. 35 inches of precipitation was used for the water balance scenario, which resulted in much less water being stored during non-irrigation months than the 2017 PDR estimate with 45.3 inches of precipitation. See Appendix D for Log-Pearson calculation results.

5-6.2.b. Tertiary Treated Equalization Storage

In addition to storage of secondary treated effluent, the reclaimed system also has storage of tertiary treated effluent. This allows the system to balance the periods of high irrigation demand and the tertiary treatment plant's production capacity. Currently, this equalization storage is comprised of a 1.8 MG EQ basin, which the tertiary treatment plant discharges into. The NCPS draws from this basin, and the gravity line to Pond 16/17 and the South Course drains from it as well. The adequacy of the existing equalization storage under buildout conditions was analyzed for this IWMP.

The golf course demands were analyzed on several different time steps. See Table 5-11 for these values. The equalization storage required for each time step is shown in Table 5-12, along with the adequacy of the available golf course storage. These available storage values are from the capacities of the golf course irrigation lakes. Bass Lake provides storage for the North Course, and Lakes 10, 11, 16, and 17 provide storage for the South Course.

Table 5-11: Golf Course Max Demands

Golf Course Max Demand Periods						
# of Days	**North Course (GPD)**	**Start Date**	**South Course (GPD)**	**Start Date**	**Both Courses (GPD)**	**Start Date**
1	2,394,749	5/6/2018	1,741,175	7/1/2019	3,042,090	5/6/2018
2	1,409,736	6/30/2015	1,741,175	7/1/2019	2,401,855	7/6/2017
3	1,371,821	7/4/2017	1,416,240	7/1/2019	2,315,777	7/4/2017
7	1,263,712	7/3/2017	1,058,035	7/1/2019	2,208,987	7/3/2017
14	1,138,331	7/3/2017	960,643	6/23/2017	2,014,265	7/3/2017
30	**1,064,625**	7/3/2017	**904,570**	6/23/2017	**1,925,036**	6/29/2017
60	911,688	7/3/2017	799,123	6/14/2017	1,694,514	6/14/2017

Table 5-12: Golf Course Required Equalization

Golf Course Equalization Required[1]			
# of Days	**North Course (gal)**	**South Course (gal)**	**Both Courses (gal)**
1	1,330,124	836,605	1,117,054
2	690,221	1,673,209	953,637
3	921,589	1,535,008	1,172,222
7	1,393,613	1,074,252	1,987,658
14	1,031,888	785,020	1,249,201
Available Storage:	12,121,657	15,559,385	27,681,042
Adequate?	Yes	Yes	Yes

[1]Required equalization is calculated by subtracting the 30-day max GPD from the GPD at each time interval and multiplying that difference by the number of days. This assumes that the 30-day max GPD is available from the supply.

For the analysis below, it is assumed that the WWRP must be able provide the maximum month GPD value to the golf courses. This equates to 1,064,625 GPD to the North Course via the NCPS and 904,570 GPD to the South Course. The irrigation lakes are able to provide equalization storage to balance between these values and the peak single-day demands for the courses, as shown in Table 5-12.

To estimate the reclaimed MDD for the new residential and commercial developments to be served by the WWRP, the ADD was calculated and then multiplied by an outdoor-specific peaking factor. This peaking factor was calculated by removing the estimated indoor demands from the max day and average day, respectively, and re-calculating the ratio of one to the other. This resulted in an outdoor-specific peaking factor of 2.66, as shown in Table 5-13 below. It is reasonable for the outdoor-specific peaking factor to be higher than the general peaking factor of 1.82 because the difference between average and peak outdoor demands is higher due to its seasonal nature, whereas indoor demand typically remains more consistent throughout the year and results in a lower peaking factor.

Table 5-13: Outdoor-Specific Peaking Factor

Outdoor-Specific Peaking Factor		
Usage	**7/20/2022**	**Entire Year Average**
Total Use (gal)	2,882,497	1,335,161
Estimated Indoor Use (gal)[1]	402,009	402,009
Calculated Outdoor Use (gal)	2,480,488	933,152
Peaking Factor	2.66	

[1]Required equalization is calculated Indoor use was estimated using the existing accounts and GPCD estimates developed in Chapter 4.

This allowed for the calculation of a total MDD for the reclaimed system. This is summarized below in Table 5-14.

Table 5-14: ADD and MDD/MMD for Reclaimed Users

ADD and MDD/MMD for Reclaimed Users		
Development	**ADD**	**MDD/MMD**
Village A	98,080	260,713[2]
Village B	106,816	283,936[2]
Village C	73,855	196,319[2]
Retreats	16,521	43,915[2]
Mur. Gar Res	9,633	25,606[2]
Mur. Gar Comm	180,912	480,896[2]
North GC	327,625	1,064,625[1]
South GC	273,236	904,570[1]
Residential/Commercial Subtotal	485,816	1,291,386
Golf Course Subtotal	600,861	1,969,195
Total	1,086,677	3,260,582

[1]This value is the maximum month demand (MMD) from the real demand data analyzed in Table 5-11.

[2]This is calculated by multiplying the ADD by the peaking factor calculated in Table 5-13.

The tables above show the total MDD/MMD for the reclaimed system to be 3.26 MGD. As previously discussed, the current capacity of the WWRP is 2.3 MGD, which is limited by the capacity of the disinfection system. The design capacity of the WWRP is 3.0 MGD. The capacity of the disinfection system is currently in the process of being expanded to match the overall WWRP capacity. After this upgrade is completed, the WWRP will be nearly able to meet the MDD/MMD for the proposed developments to be served in addition to the golf courses.

The last step in determining the storage adequacy is evaluating the daily equalization required. As shown above in Table 5-14, the total estimated MDD for all the reclaimed water users is 3.26 MGD. With the required disinfection upgrades, the production capacity of the WWRP is 3.0 MGD. The required additional supply for the MDD is estimated to be 0.26 MG.

For the golf courses, it is assumed that the WWRP will supply the MMD/MDD over a 16-hour period to refill the storage lakes. This results in a total of 2051 gpm leaving the EQ basin to the golf courses on the maximum day, 1,109 gpm of which will be supplied by the NCPS to the North Course, and the rest by gravity to Lake 16 and the South Course.

For the residential and commercial users, two different demand scenarios were evaluated. Scenario 1 assumes that the demand will occur during eight hours, presumably during the night when most users irrigate, and that the golf courses do not receive water during those eight hours. The residential/commercial MDD over an eight-hour period results in a flow rate of 2690 gpm. Scenario 2 assumes that demand will occur over 24 hours, and that 16 of those hours will coincide with the filling of the golf course lakes. This scenario respects the fact that the District's Reclaimed Water Standards requires that reclaimed water always be available to its users. The MDD over 24 hours results in a flow rate of 896 gpm. Adding this to the golf course flow results in 2947 gpm leaving the EQ basin, 2005 gpm of which will be pumped by the NCPS (942 gpm goes to the south course by gravity). At 3.0 MGD, the supply flow available from the WWRP is 2083 gpm, resulting in a flow deficit of 864 gpm over the 16-hour period, or 0.83 MG. Therefore, Scenario 1 controls the sizing of the NCPS, with a maximum required flow of 2690 gpm, while Scenario 2 controls the required equalization storage, with a required equalization flow of 864 gpm for 16 hours, or 0.83 MG. The EQ basin has 1.8 MG of storage, so it has sufficient storage to equalize the maximum flows at buildout. Further, the existing 8" potable water line at the WWRP can provide approximately 0.8 MGD during max day while maintaining adequate residual pressures throughout the system. This additional flow can help equalize peak days as well. See Table 5-15 below for a summary of the equalization scenarios.

Table 5-15: Reclaimed High Flow Scenarios

Reclaimed High Flow Scenarios			
Scenario	**GPD**	**GPM**	**NCPS GPM**
Scenario 1: Residential/Commercial Demand spread over 8 hrs	1,291,386	2,690	2,690
Scenario 2: Residential/Commercial Demand spread over 24 hrs plus Golf Course Demand spread over 16 hours	3,260,582	2,947	2,005

5-6.3. Distribution

The NCPS has two vertical turbine pumps, each capable of delivering 1062 gpm at 323 feet of head. At buildout, the highest anticipated flow through the pump station is 2690 gpm, as shown in Table 5-15. Therefore, the current pump station is not sufficient to deliver the required flows at buildout.

5-6.3.a. North Course Transmission Pipeline

As described in Chapter 2, the transmission line from the NCPS to Bass Lake consists of some 12" ACP and some 8" ACP. See Figure 2-17 for a map of the existing system. The existing pipeline reduces to 8" after the branch to Murieta Gardens. Therefore, the pipeline beyond that point is responsible for carrying flows to Villages A, B, C, the Retreats, and the North Course. The estimated MDD for Villages A, B, C, and the Retreats is 784,884 gallons, which equates to 1635 gpm over an 8-hour irrigation period. In the existing 8" pipeline, this would result in a velocity of 10.4 fps and excessive head loss. Further, the existing ACP pipe is not able to handle the high operating pressures anticipated in the system. Therefore, the pipe needs to be replaced with a larger diameter PVC pipe.

CHAPTER 6. Improvement Alternatives

This chapter presents water system improvement alternatives, capital cost estimates, project phasing, and implementation considerations. As a part of implementation, the District should include capital improvements from this IWMP in its overall capital planning efforts. The results of the overall capital improvement planning will allow the District to appropriately update its user rates and developer charges. Detailed cost estimates for the alternatives are included in Appendix E.

6-1. Overview

Figure ES-1-1 shows water system improvement alternatives. Alternatives include new wells, pipelines, tanks, pump stations, reclaimed water treatment improvements, and new operational practices. The alternatives are based on water system analyses described in Chapter 5 and the performance objectives in Chapter 3.

Alternatives are summarized in a Capital Improvement Program (CIP). The CIP includes the costs of improvements required for all major facilities, including improvements to existing pipelines. The CIP does not include the cost of new pipeline extensions to areas that are currently undeveloped and not served by an existing pipeline. It is assumed that these facilities would be constructed by developers as a part of the new developments. However, major pipeline extensions are described in this section for planning purposes. Developers may also be required to contribute to the cost for new water production, storage, and pumping facilities as required by District standards.

Projects included in the CIP are:

- New groundwater supply wells
- Expanded surface water storage (use of Clementia)
- New treated water storage tanks
- A new booster pump station

- Improvements to existing pipelines
- Improvements to fire suppression infrastructure
- WWRP improvements
- A new reclaimed water pump station
- Reclaimed water distribution improvements

CIP projects are staged by timeframe needed:

- Existing – to correct existing deficiencies and provide some capacity for future growth
- Buildout of the remaining lots to be developed within the District boundary which will occur on an unknown timeline.

Project staging information is intended as a guideline for District staff. Specific implementation priorities and timing for projects will be determined by District staff considering the timing of development and overall District needs, such as coordination with other projects.

6-1.1. Basis of Cost Estimates

Planning-level capital cost estimates were developed for improvements. Estimated capital costs include construction costs, construction contingencies, and project implementation costs. The accuracy of the estimates is consistent with AACE class 5 standards, which allow for -50% to 100% variability to actual construction costs.

Construction contingencies were estimated as 20% of construction costs to account for additional work identified during design, uncertainties in the bidding climate, and change orders during construction. Project implementation costs were estimated as 25% of construction costs, and include project management, design, construction management, environmental work, and inspection.

Construction costs are based on cost data from other Adkins projects, publicly available bid results, estimates used in past District planning publications, and direct input from District staff. The unit costs assume a normal (average) construction environment and do not include circumstances such as significant rock excavation or dewatering, unusual working hours, or exotic construction methods. Pipeline unit costs include valves and appurtenances, as well as pavement removal and replacement and a general allowance for correction of utility interferences where applicable. Pump station costs are based on an expandable above ground enclosed building and standby pump, backup power, and telemetry. Tank costs include average site work, valve vault, telemetry, piping, and appurtenances. Well costs include standby power and disinfection. See Appendix E for detailed cost estimates for each of the alternatives below.

6-2. Supply Improvements

6-2.1. Groundwater Supply

The alternatives developed for the purpose of this IWMP differ based on water treatment needs of the well water. As summarized in the previously referenced Adkins' groundwater literature review tech memo (see Appendix F), groundwater from test wells evaluated in 2013 had elevated arsenic levels. However, it is typical for water quality to improve after well development is completed. Therefore, it is possible that no water treatment would be needed after new wells are fully developed, but this section explores alternatives for a range of different required treatment levels. These required levels cannot be known for certain until the new wells are developed. The following five alternatives for treatment were developed:

1) No water treatment required
2) Treating a portion of the water from the wells at a new WTP and blending with the remaining water
3) Leased portable water treatment units as needed
4) Treating all water from the wells at a new WTP
5) New pipeline to send water from all wells to the existing WTP

Each of these alternatives are explored for existing conditions (the 3,000 connections threshold for SB 552) and buildout conditions.

For each alternative, pump motors and pipelines were sized using EPANet2.2. Pump power was balanced with motor size to maintain best efficiency points, resulting in 75 HP pumps and motors for all existing and buildout conditions. Pipelines were sized using a maximum allowable velocity of 5 fps to optimize function and cost. This resulted in mostly 8” diameter transmission lines, with some 10” lines for the combined flows of multiple wells returning to the distribution system.

For Alternative 2, for both existing and buildout conditions, the new WTP performs side-stream treatment on the well water to achieve quality standards. To estimate the amount of mixing required for Alternative 2, the three proposed wells for existing conditions were assumed to have the largest observed arsenic concentration from test hole A from the 2013 DE investigation, 0.018 mg/L. For buildout conditions, all five wells were assumed to have this higher arsenic concentration. Well development may determine that different arsenic concentrations are present; this alternative is therefore conservative. A mass balance approach was used to calculate the portion of the stream that should be treated to dilute the arsenic to below the EPA MCL, 0.01 mg/L with a 20% margin of safety, bringing the maximum expected concentration of the blended water down to 0.008 mg/L. The reduction of arsenic from 0.018 mg/L to 0.008 mg/L represents a 56% reduction in concentration. See the general form equation below:

$$Untreated\ Stream = \frac{Total\ Stream\ \times\ C_{MCL} \times (1-\sigma)}{C_{AS}}$$

$$Treated\ Stream =\ Total\ Stream - Untreated\ Stream$$

Where:

C_{AS} = measured concentration of Arsenic (mg/L)

σ = margin of safety, 20%

C_{MCL} = EPA Maximum Contaminant Level for Arsenic (0.01 mg/L)

Flows are given in GPM

Stated simply, 56% of the well water stream must be treated to reduce the arsenic concentration by 56%. For existing conditions, the treated stream was calculated to be 655 gpm, which allows for an untreated stream of 514 gpm. For buildout conditions, the treated stream was calculated to be 1,174 gpm, which allows for an untreated stream of 923 gpm.

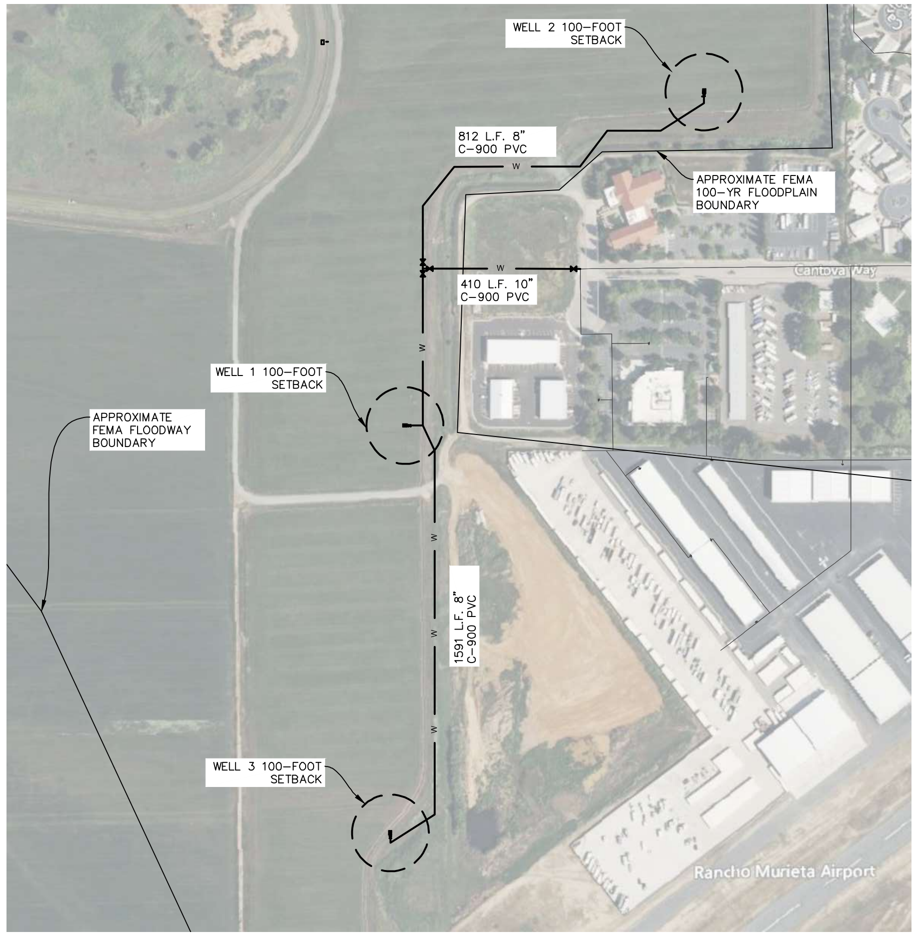
The five existing conditions alternatives consider meeting the 3,000-connection ADD of 1,169 gpm via three wells. These wells are proposed to be drilled approximately 500 feet deep with 12” diameter casings. Installation of the wells includes full well development and test pumping, installation of 75 HP pumps and motors, shafts, columns, pump house including necessary piping, valves, flowmeters, chlorination equipment, Variable Frequency Drives (VFDs), panels, SCADA controls, power distribution, and access roads. To connect the wells to the distribution system, approximately 410 LF of 10-inch C-900 PVC and 2,680 LF of 8-inch C-900 PVC are proposed. Three gate valves with thrust blocks are proposed to allow the District to isolate one or more of the transmission lines from the distribution system.

The five buildout conditions alternatives consider meeting the buildout ADD of 2,097 GPM via five wells. These wells are proposed to be drilled approximately 500 feet deep with 12-inch diameter casings. Installation of the wells includes full well development and test pumping, and installation of 75 HP pumps and motors, shafts, columns, pump house including necessary piping, valves, flowmeters, chlorination equipment, VFDs, panels, SCADA controls, power distribution, and access roads. To connect the wells to the distribution system, approximately 638 LF of 10-inch C-900 PVC and 4,382 LF of 8-inch C-900 PVC are proposed. Three gate valves with thrust blocks are proposed to allow the District to isolate one or more of the transmission lines from the distribution system.

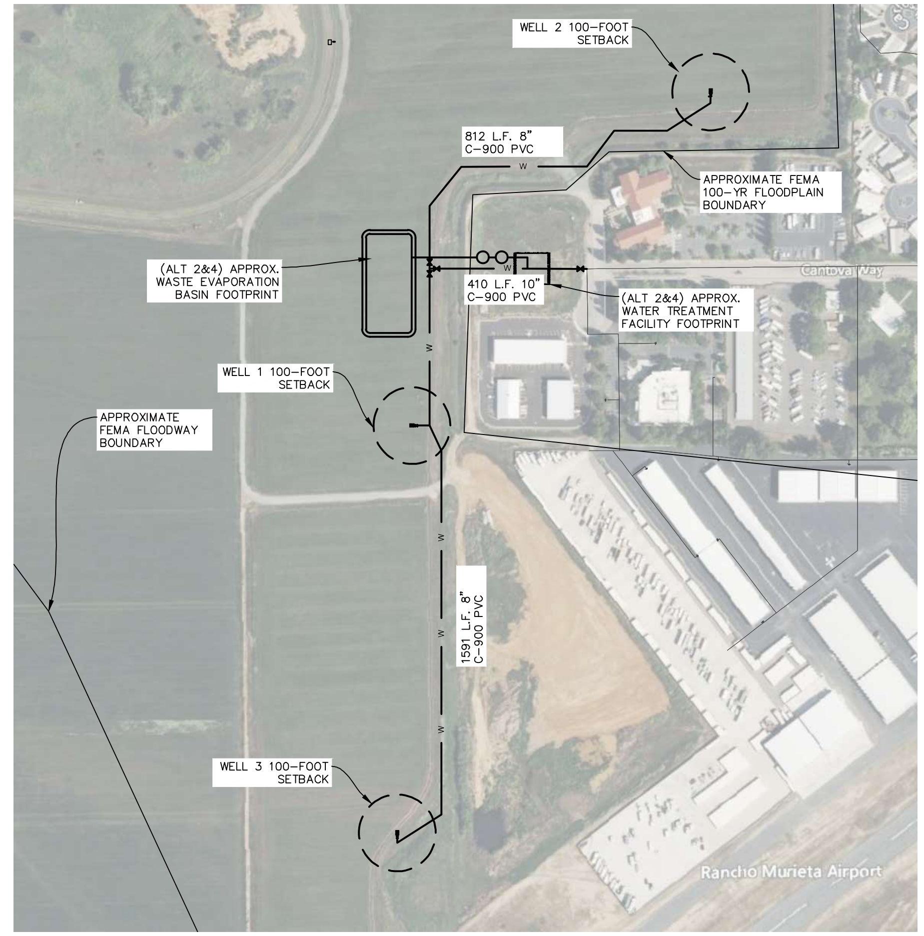
For both existing and buildout conditions, isolated aquifer testing should be conducted during the well construction process. This will allow the District to determine if the arsenic in the groundwater is coming from an isolated depth range. If this is the case, then this contaminant source could be avoided altogether by strategically placing the casing screen at a different depth than the contaminating section of the well.

Estimated costs for well development and necessary components are the same for each alternative. The alternatives vary based on potential water quality, which will be determined during well development. The test holes investigated by DE in 2013 indicated that arsenic was present in test hole A, while iron and manganese were present in test hole B. As it is difficult to determine water quality of a specific well site without drilling and developing the well, the following alternatives are analyzed to determine a range of costs based on water quality and treatment needs for the District.

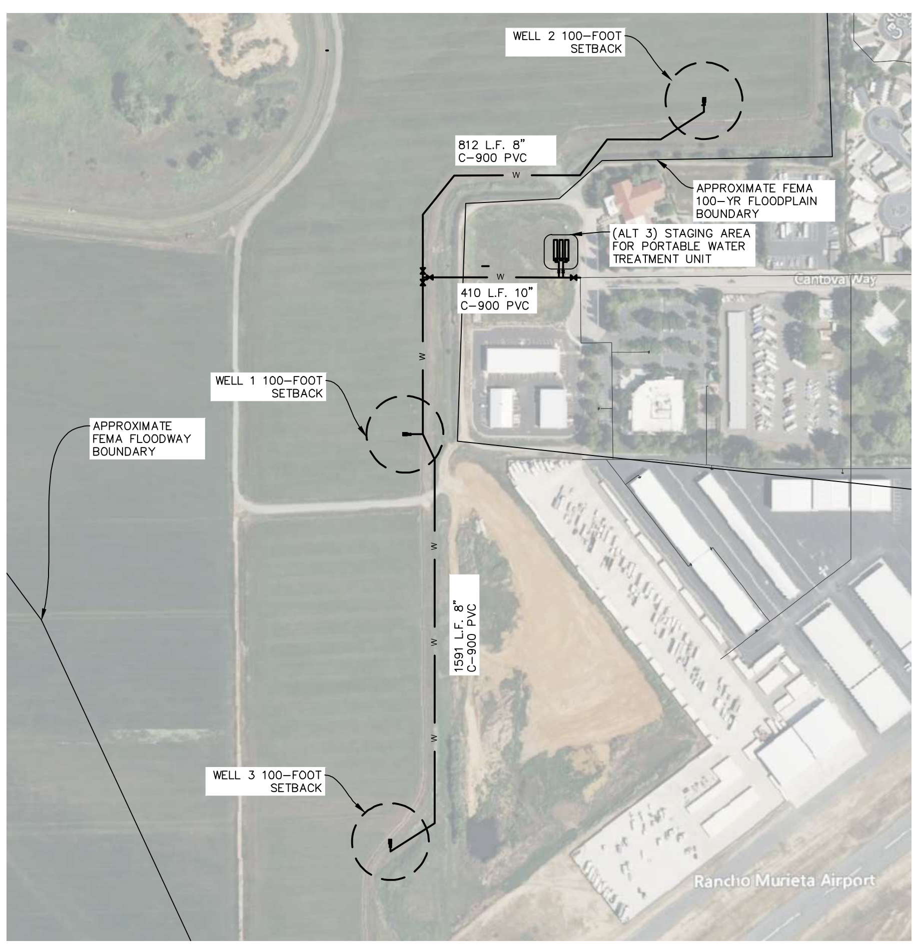
Based on State Water Board standards, the wells need to be located outside of the 100-year flood plain, or elevated above the floodplain using acceptable structural fill. However, the proposed well locations are all within the FEMA 100-year floodplain. While this is not an ideal scenario, this wellfield location is the only one that has been studied previously and that has somewhat predictable outcomes. To comply with state regulations for wells within the 100-year floodplain, each well site should have structural fill added to the site to raise the wellhouse and well casing above the 100-year flood elevation, along with any other protection measures that the state may require for the specific sites. It is estimated that 1-3 feet of structural fill would be needed to elevate the well sites above the 100-year floodplain. Base flood elevation surveys would be required to establish these elevations precisely prior to design. These alternatives consider the use of the wells for backup or emergency use only. See Figure 6-1 for a concept map of well placement for existing conditions alternatives and Figure 6-2 for buildout demands alternatives.



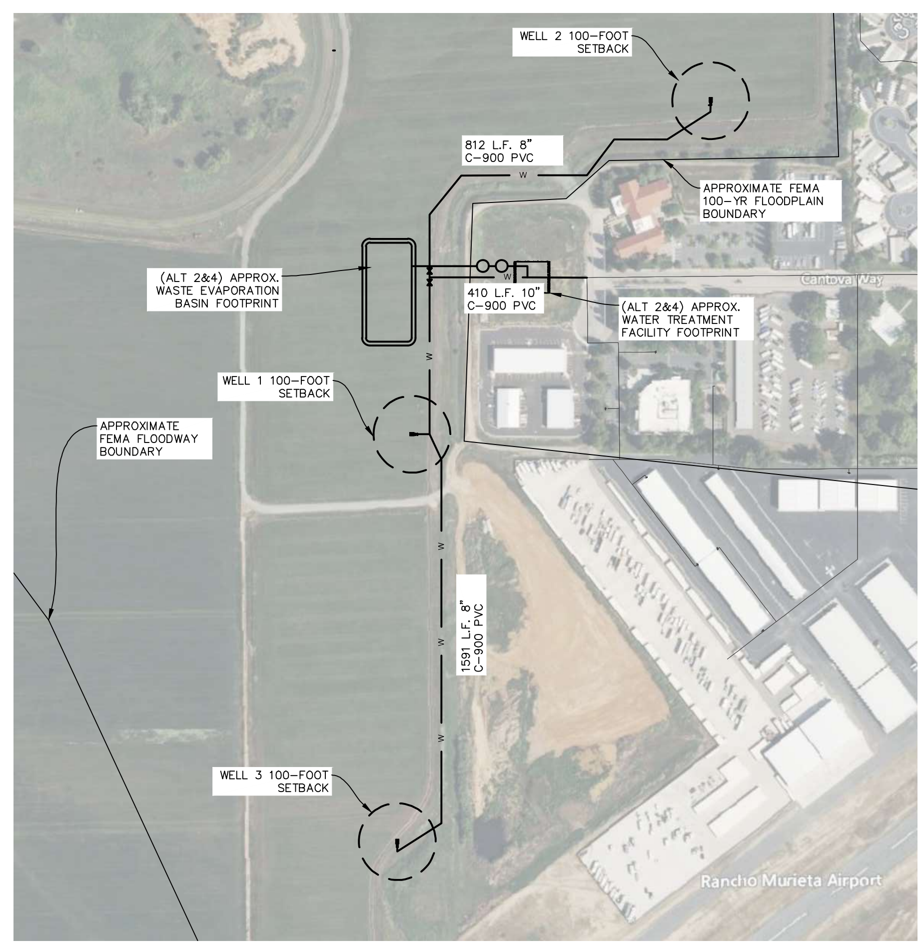
SITE PLAN FOR ALTERNATIVE 1: 3,000 CONNECTIONS



SITE PLAN FOR ALTERNATIVE 2: 3,000 CONNECTIONS



SITE PLAN FOR ALTERNATIVE 3: 3,000 CONNECTIONS



SITE PLAN FOR ALTERNATIVE 4: 3,000 CONNECTIONS

250 0 125 250
(FEET)

LEGEND

EXISTING WATER MAIN
PROPOSED WATER TRANSMISSION LINE
PROPOSED WATER VALVE
PROPOSED WELL
PROPOSED WELL HOUSE

SITE PLAN NOTES

1. NO TOPOGRAPHIC SURVEY WAS PERFORMED. ALL LOCATIONS OF UTILITIES SHOWN ARE APPROXIMATE. ACTUAL SITE CONDITIONS AND LOCATIONS MAY VARY.
2. THIS IS A SCHEMATIC SITE PLAN FOR THE SOLE PURPOSE OF ILLUSTRATING THE GENERAL COMPONENTS AND FACILITIES THAT WILL BE NEEDED FOR EACH ALTERNATIVE. ACTUAL SITE LAYOUT WILL BE DETERMINED IN THE 30% AND FINAL DESIGN PHASES OF THE PROJECT FOR THE SELECTED ALTERNATIVE.
3. 3,000 CONNECTION ALTERNATIVES ASSUME 3 WELLS, WITH 125 HORSEPOWER PUMPS AND MOTORS, EACH TO DELIVER 390 GPM AT 503 T.D.H.
   - 3.1. ALTERNATIVE 1 CONSIDERS NO WATER TREATMENT.
   - 3.2. ALTERNATIVE 2 CONSIDERS TREATING PART OF THE WELL WATER AT A PERMANENT WATER TREATMENT FACILITY.
   - 3.3. ALTERNATIVE 3 CONSIDERS LEASED PORTABLE WATER TREATMENT UNITS AS NEEDED.
   - 3.4. ALTERNATIVE 4 CONSIDERS THE INSTALLATION OF A PERMANENT WATER TREATMENT FACILITY.

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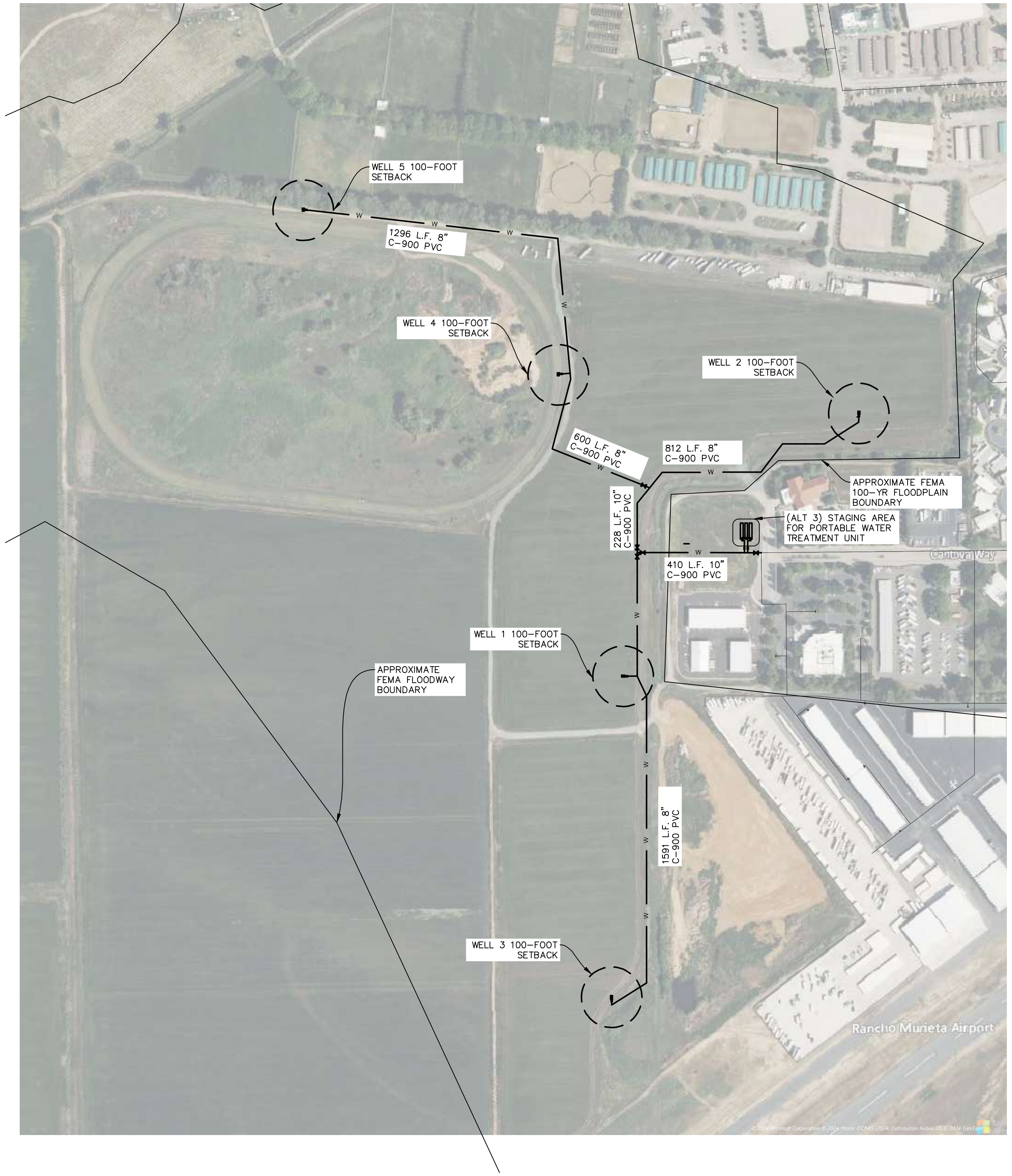
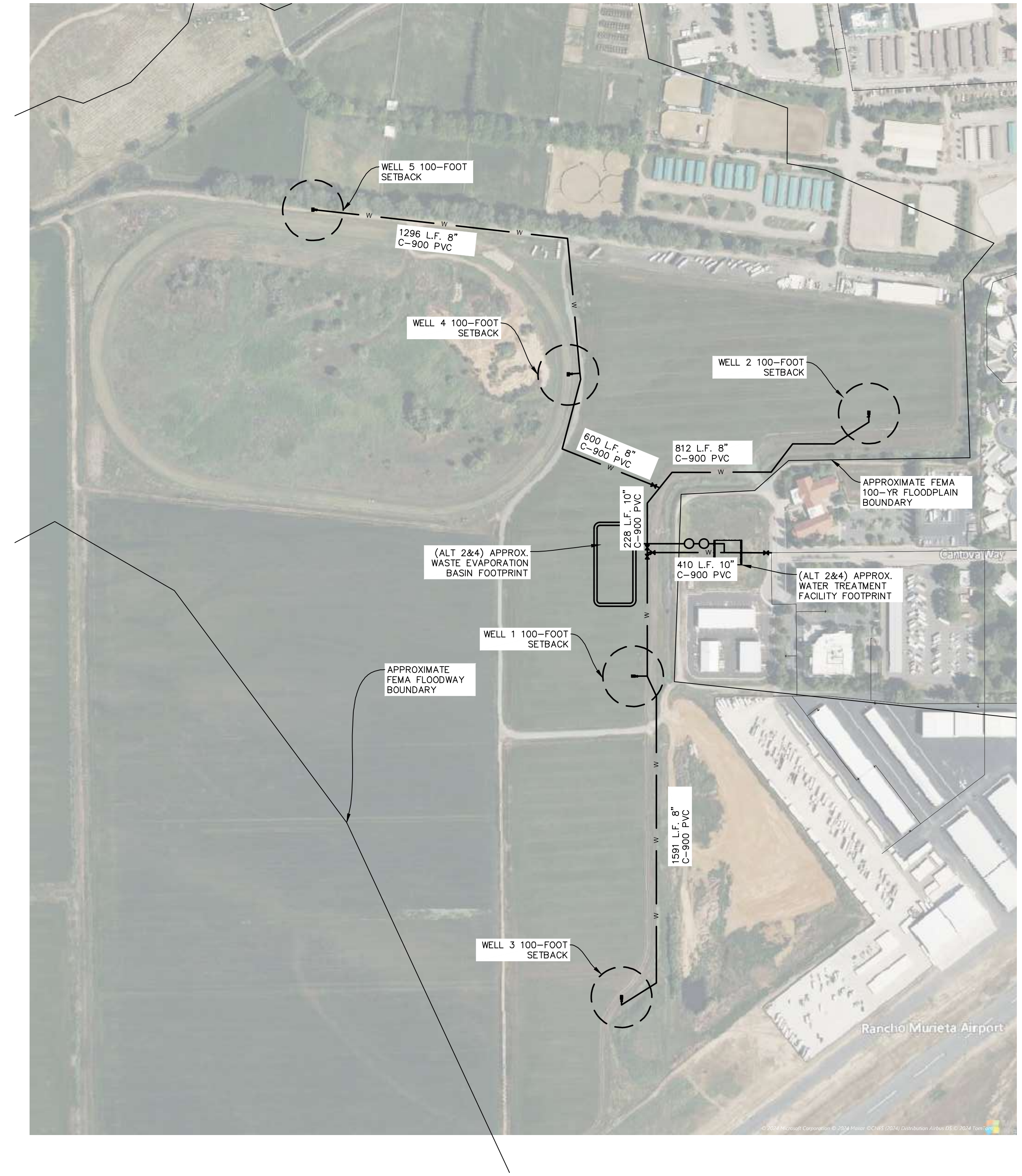
GROUNDWATER ALTERNATIVES
FOR
RANCHO MURIETA CSD
3000 CONNECTION GROUNDWATER ALTERNATIVES

SCALE	SHOWN
DATE	08/08/2024
DRAWN BY	BAG
PROJ. NO.	3971-02

FIGURE 6-1



SITE PLAN FOR ALTERNATIVES 1 & 3: BUILD OUT CONDITIONS



SITE PLAN FOR ALTERNATIVES 2 & 4: BUILD OUT CONDITIONS

250 0 125 250
(FEET)

LEGEND

EXISTING WATER MAIN

PROPOSED WATER TRANSMISSION LINE

PROPOSED WATER VALVE

PROPOSED WELL

PROPOSED WELL HOUSE

SITE PLAN NOTES

1. NO TOPOGRAPHIC SURVEY WAS PERFORMED. ALL LOCATIONS OF UTILITIES SHOWN ARE APPROXIMATE. ACTUAL SITE CONDITIONS AND LOCATIONS MAY VARY.
2. THIS IS A A SCHEMATIC SITE PLAN FOR THE SOLE PURPOSE OF ILLUSTRATING THE GENERAL COMPONENTS AND FACILITIES THAT WILL BE NEEDED FOR EACH ALTERNATIVE. ACTUAL SITE LAYOUT WILL BE DETERMINED IN THE 30% AND FINAL DESIGN PHASES OF THE PROJECT FOR THE SELECTED ALTERNATIVE.
3. BUILD OUT CONDITIONS ALTERNATIVES ASSUME 5 WELLS, WITH 125 HORSEPOWER PUMPS AND MOTORS, EACH TO DELIVER 420 GPM AT 503' T.D.H.

3.1. ALTERNATIVE 1 CONSIDERS NO WATER TREATMENT.

3.2. ALTERNATIVE 2 CONSIDERS TREATING PART OF THE WELL WATER AT A PERMANENT WATER TREATMENT FACILITY.

3.3. ALTERNATIVE 3 CONSIDERS LEASED PORTABLE WATER TREATMENT UNITS AS NEEDED.

3.4. ALTERNATIVE 4 CONSIDERS THE INSTALLATION OF A PERMANENT WATER TREATMENT FACILITY.

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GROUNDWATER ALTERNATIVES
FOR
RANCHO MURIETA CSD
BUILDOUT GROUNDWATER ALTERNATIVES

SCALE	DATE	DRAWN BY	PROJ. NO.
SHOWN	08/08/2024	BAG	3971-02

FIGURE
6-2

6-2.1.a. Alternative 1: No Water Treatment Required

This alternative assumes that after well development, no water treatment is required for arsenic, iron, and/or manganese. Disinfection of well water is achieved by chlorine dosing at each well house, and the disinfected water is sent directly to the distribution system. A cost summary for the 3000-connection alternative is provided in Table 6-1, and a cost summary for the buildout alternative is provided in Table 6-2.

Table 6-1: Alternative 1A Cost Summary

Alternative 1A – Existing Conditions, No Water Treatment	
Project Component	**Estimated Cost**
Mobilization/Demobilization, Construction Surveying:	$246,100
3 New Wells, 75 HP Pumps & Motors, Well Development:	$1,826,000
Well Houses, Controls, Power, Access Roads:	$1,639,300
Install Pipelines and Connect to Existing:	$601,600
Subtotal:	$4,313,000
Construction Contingencies:	$863,000
Design, Engineering, Construction Admin:	$1,079,000
Environmental, Permitting, Legal, Land Acquisition:	$94,000
Total Estimated Project Cost:	$6,349,000

Table 6-2: Alternative 1B Cost Summary

Alternative 1B – Buildout Conditions, No Water Treatment	
Project Component	**Estimated Cost**
Mobilization/Demobilization, Construction Surveying:	$407,600
5 New Wells, 75 HP Pumps & Motors, Well Development:	$3,043,200
Well Houses, Controls, Power, Access Roads:	$2,728,800
Install Pipelines and Connect to Existing:	$963,200
Subtotal:	$7,142,800
Construction Contingencies:	$1,429,000
Design, Engineering, Construction Admin:	$1,786,000
Environmental, Permitting, Legal, Land Acquisition:	$97,000
Total Estimated Project Cost:	$10,455,000

6-2.1.b. Alternative 2: Permanent Water Treatment Plant for Partial Flow

This alternative assumes that after well development, there are arsenic concentrations in the water that can be addressed by treating a portion of the water and blending it with the remainder of the water to achieve dilution requirements.

Backwash water is a common byproduct to oxidation filtration methods of arsenic treatment. Backwash cycles continuously regenerate and clean filter media and must either be disposed of or reclaimed through a settling tank and pump-assisted return line. Another consideration of treatment is that the pH must be adjusted to less than 8.0 (ideally 7.5) to facilitate the coprecipitation of iron and arsenic. The test holes from DE (2013) showed a pH of between 6.5 and 8.2 between the two test holes. Thus, the pH of the well water likely needs to be pH adjusted. However, a lower pH significantly affects the oxidation rates of iron and manganese. These are important operational considerations to be weighed if well development indicates the need for treatment.

Treating a portion of the water includes the construction and implementation of a permanent WTP. The proposed WTP footprint is approximately 1 acre and utilizes oxidation and filtration methods. However, other treatment methods could also be used. Relevant components of a WTP to dual-treat arsenic, iron, and manganese by oxidation and filtration include: a water treatment building large enough to house treatment equipment, chemical feed stations, chemical storage, instrumentation and controls, booster pumps, office spaces for operators, backwash recovery facilities that include backwash settling tanks, booster pumps, evaporation lagoons, sludge removal, disinfection treatment, back-up generator, and an automatic transfer switch. A cost summary for the WTP to treat 56% of the well water (for both existing and buildout conditions) is included in Appendix E.

It is proposed that the water from all the wells would enter the new WTP. From there, the appropriate portion of the flow would be redirected to be treated, while the remaining portion would bypass treatment. The treated stream would be blended with the untreated stream after treatment and the resulting stream would be within the MCL requirements.

The WTP could be located in the undeveloped parcel to the west of the existing Catholic Church. This is the only undeveloped parcel outside of the 100-year floodplain that is near the proposed wellfield. However, the WTP could possibly be located on the same parcel as the wellfield if acceptable structural fill was provided to elevate the WTP above the floodplain with State Water Board approval. For the conceptual site maps shown in Figure 6-1 and Figure 6-2, the WTP is shown on the undeveloped parcel west of the Catholic Church, outside of the floodplain.

A baseline cost estimate was developed for a WTP capable of treating the entire buildout flow of 2,097 gpm. This estimate was scaled for each respective flow requirement in each alternative. A cost summary for the 3000-connection alternative is provided in Table 6-3. A cost summary for the buildout alternative is provided in Table 6-4.

Table 6-3: Alternative 2A Cost Summary

Alternative 2A – Existing Conditions, Partial Treatment	
Project Component	**Estimated Cost**
Mobilization/Demobilization, Construction Surveying:	$481,400
3 New Wells, 75 HP Pumps & Motors, Well Development:	$1,826,000
Well Houses, Controls, Power, Access Roads:	$1,639,300
Construct Permanent WTP for 655 gpm:	$3,888,000
Install Pipelines and Connect to Existing:	$601,600
Subtotal:	$8,436,300
Construction Contingencies:	$1,688,000
Design, Engineering, Construction Admin:	$2,109,000
Environmental, Permitting, Legal, Land Acquisition:	$300,000
Total Estimated Project Cost:	$12,533,000

Table 6-4: Alternative 2B Cost Summary

Alternative 2B – Buildout Conditions, Partial Treatment	
Project Component	**Estimated Cost**
Mobilization/Demobilization, Construction Surveying:	$825,600
5 New Wells, 75 HP Pumps & Motors, Well Development:	$3,043,200
Well Houses, Controls, Power, Access Roads:	$2,728,800
Construct Permanent WTP for 1174 gpm:	$6,910,000
Install Pipelines and Connect to Existing:	$963,200
Subtotal:	$14,471,000
Construction Contingencies:	$2,895,000
Design, Engineering, Construction Admin:	$3,618,000
Environmental, Permitting, Legal, Land Acquisition:	$300,000
Total Estimated Project Cost:	$21,284,000

6-2.1.c. Alternative 3: Leased Treatment Unit

This alternative assumes that after well development, full water treatment for arsenic, iron, and/or manganese is required, but this is achieved through leased portable water treatment units as needed. The basis of design used for this IWMP is the portable Rapisand treatment unit leased through WesTech Engineering, Inc. This portable unit is approximately 53 feet long, 8.5 feet wide, and 13.5 feet tall, and can be delivered to the site by truck. Each treatment unit can treat 700 gpm. The units use a combined flocculation and sedimentation process. They start by adding a coagulant to the raw water stream to destabilize suspended particles, followed by mixing with a polymer and recycled microsand. This allows rapid sedimentation and clarification of the water. The solids are then directed to waste while the separated sand is reintroduced into the initial flocculation tank. Each unit produces a constant waste stream of approximately 45 gpm. This waste could be piped to the District's sewer system via a connection to the gravity collection line at the end of Cantova Way. Two treatment units would be required for the 3000-unit alternative and three treatment units would be required for the buildout alternative. Each unit costs approximately $38,000/month to rent and $15,000 to ship to and from the site. Training and inspection cost approximately $20,000. This alternative considers a staging area for the portable water treatment unit west of the Catholic Church, in the same location that the new WTP proposed in Alternative 2 would be located. Since this alternative is identical to Alternative 1 with the exception of the leased treatment units, which are not capital expenditures, refer to Table 6-1 and Table 6-2 for cost summaries for this alternative. The methodology for comparing the net present value of this alternative with the other groundwater alternatives is discussed later in this section.

6-2.1.d. Alternative 4: Permanent Water Treatment Plant for Full Flow

This alternative assumes that after well development, full water treatment for arsenic, iron, and/or manganese is required for all wells, and this is achieved through the construction of a permanent WTP capable of treating the entire stream. The siting and treatment considerations for the WTP are the same as described above for Alternative 2. A cost summary for the 3,000-connection alternative is provided in Table 6-5 and a cost summary for the buildout alternative is provided in Table 6-6.

Table 6-5: Alternative 4A Cost Summary

Alternative 4A – Existing Conditions, Permanent Water Treatment Plant	
Project Component	**Estimated Cost**
Mobilization/Demobilization, Construction Surveying:	$662,400
3 New Wells, 75 HP Pumps & Motors, Well Development:	$1,826,000
Well House, Controls, Power, Access Roads:	$1,639,300
Install Pipelines and Connect to Existing:	$601,600
Installation of Permanent WTP for 1169 gpm:	$6,880,000
Subtotal:	$11,609,000
Construction Contingencies:	$2,322,000
Design, Engineering, Construction Admin:	$2,903,000
Environmental, Permitting, Legal, Land Acquisition:	$350,000
Total Estimated Project Cost:	$17,184,000

Table 6-6: Alternative 4B Cost Summary

Alternative 4B – Buildout Conditions, Permanent Water Treatment Plant	
Project Component	**Estimated Cost**
Mobilization/Demobilization, Construction Surveying:	$1,150,000
5 New Wells, 75 HP Pumps & Motors, Well Development:	$3,043,200
Well House, Controls, Power, Access Roads:	$2,728,800
Install Pipelines and Connect to Existing:	$958,200
Installation of Permanent WTP for 2097 gpm:	$12,277,000
Subtotal:	$20,157,000
Construction Contingencies:	$4,032,000
Design, Engineering, Construction Admin:	$5,040,000
Environmental, Permitting, Legal, Land Acquisition:	$350,000
Total Estimated Project Cost:	$29,579,000

6-2.1.e. Alternative 5: Send Well Water to Existing WTP

This alternative assumes that after well development, full water treatment for arsenic, iron, and/or manganese is required for all wells, and this is achieved through piping the well water to the existing WTP at Chesbro Reservoir. As the use of groundwater is considered only during circumstances when the surface water supply is compromised or unavailable, the existing WTP capacity is considered adequate to treat the required flows from the proposed wells. Thus, this alternative is identical to Alternative 1 in terms of well installation but adds a new 17,200 LF pipeline to deliver the well water across the District to the existing WTP. A 10-inch pipe would be required for the 3,000-connection flow of 1,169 gpm, and a 14-inch pipe would be required for the buildout flow of 2,097 gpm. A cost summary for the 3,000-connection alternative is presented in Table 6-7 and a cost summary for the buildout alternative is presented in Table 6-8.

Table 6-7: Alternative 5A Cost Summary

Alternative 5A – Existing Conditions, Treat at Existing WTP	
Project Component	**Estimated Cost**
Mobilization/Demobilization, Construction Surveying:	$466,000
3 New Wells, 75 HP Pumps & Motors, Well Development:	$1,826,000
Well Houses, Controls, Power, Access Roads:	$1,639,300
Install Pipelines to Connect Wells to Existing WTP:	$4,235,300
Subtotal:	$8,166,600
Construction Contingencies:	$1,634,000
Design, Engineering, Construction Admin:	$2,042,000
Environmental, Permitting, Legal, Land Acquisition:	$144,000
Total Estimated Project Cost:	$11,987,000

Table 6-8: Alternative 5B Cost Summary

Alternative 5B – Buildout Conditions, Treat at Existing WTP	
Project Component	**Estimated Cost**
Mobilization/Demobilization, Construction Surveying:	$657,400
5 New Wells, 75 HP Pumps & Motors, Well Development:	$3,043,200
Well Houses, Controls, Power, Access Roads:	$2,728,800
Install Pipelines to Connect Wells to Existing WTP:	$5,092,800
Subtotal:	$11,522,200
Construction Contingencies:	$2,305,000
Design, Engineering, Construction Admin:	$2,881,000
Environmental, Permitting, Legal, Land Acquisition:	$147,000
Total Estimated Project Cost:	$16,855,000

6-2.2. Groundwater Regulatory Requirements

6-2.2.a. Arsenic

Arsenic is a contaminant listed by the National Primary Drinking Water Regulations (NPDWRs), which are outlined by the EPA as legally enforceable standards that apply to public water systems. The maximum contaminant level (MCL) for arsenic is 0.01 mg/L. Based on test well data, the assumed arsenic concentration in the groundwater is 0.018 mg/L.

6-2.2.b. Iron

Iron is part of the EPA's National Secondary Drinking Water Regulation (NSDWR) that are non-mandatory water quality standards for various contaminants in drinking water. Increased concentrations of iron can cause water to have a rusty color with visible sedimentation, have a metallic taste, and leave red or orange staining. The secondary MCL for iron is 0.3 mg/L. Based on test well data, the assumed iron concentration in the groundwater is 0.5 mg/L.

6-2.2.c. Manganese

The EPA established a NSDWR that set non-mandatory water quality standards for manganese. When manganese is present in drinking water at levels above the secondary MCL

(0.05 mg/L), it may cause a black or brown appearance, black staining, or a bitter metallic taste. Based on test well data, the assumed manganese concentration in the groundwater is 0.37 mg/L.

6-2.2.d. Well Siting

Title 22 of the California Code of Regulations (CCR), Section 64417 states that wells must be sited above the 100-year floodplain or elevated above the floodplain using acceptable structural fill. Section 16.06.040(a) of Sacramento County Design Code states that any groundwater wells must have a 50-foot setback from any sewer lines and surface waters, a 100-foot setback from septic tanks, leach lines, or animal enclosures, and a 150-foot setback from leaching pits or hazardous materials tanks.

The technical memorandum published by Dunn Environmental, Inc (DE) in 2013 recommended that wells be drilled within 50-feet of test holes A and B. However, the *Methods for Determining the Proper Spacing of Wells in Artesian Aquifers* (Lang, USGS 1962) recommends 500- to 1,500-feet of space between multiple wells in the same aquifer. For the purposes of this report, proposed wells are spaced 1,000 feet apart along the southwestern side of the District, spaced between test holes A and B as determined by DE in 2013.

6-2.3. Water Treatment Technologies

Arsenic removal can be achieved with technologies including ion exchange, adsorption, coagulation and filtration, oxidation and filtration, or reverse osmosis. To remove iron and manganese, oxidization of soluble forms of iron and manganese to insoluble forms followed by filtration is commonly used. Filtration of the oxidized precipitates can be achieved using either a synthetic membrane or filter media. The oxidation and filtration process to remove iron and manganese can also be used to remove arsenic when adequate iron is present to facilitate the co-precipitation of the two. A brief description of oxidation and filtration is described below.

Oxidation is commonly used to convert soluble forms of iron and manganese to insoluble forms prior to filtration. Either chlorine or potassium permanganate is injected and mixed into the stream to oxidize iron, manganese, hydrogen sulfide, and arsenic. When a sufficient iron to arsenic ratio is present (usually 20:1), the co-precipitation of iron and arsenic occurs, and filtration effectively removes both constituents from the stream. Filtration can be achieved with pressure media filters or membranes. In both cases, the filters will become clogged as insoluble compounds are filtered, and periodic backwash cycles are needed to facilitate regeneration of the media or cleaning of the membrane. The backwash water is either disposed of or sent to a settling tank. After particulates settle, the clarified water (called supernatant) is recovered by returning to the beginning of the treatment facility while the concentrated sludge is disposed of.

Media filters utilize several different media types including silica sand, Greensand Plus, and pyrolusite. In addition to oxidation by means of a chemical feed upstream of the filters, these media also oxidize iron and manganese in place on the media surface. Because of this ability, a lesser amount of oxidation by chemical injection can be achieved.

6-2.4. Summary of Groundwater Improvements

Previous sections developed the need for a backup or emergency groundwater source for the District, discussed general considerations from prior studies, and outlined five alternatives for both existing and buildout conditions.

At 3,000 connections, the District's average day demand (ADD) was estimated to be 1,169 gpm. This is considered the existing conditions. At buildout conditions, the ADD estimated to be 2,097 gpm. Based on the study by DE in 2013, test hole locations on the southwest side of the District could produce potential well yields ranging from 150 to 500 gpm. To achieve 1,169 gpm, three wells are proposed. To achieve 2,097 gpm, five wells are proposed. Each of these wells would need to be drilled to a total well depth of 500 feet to meet the appropriate depth within the water bearing zones.

A life cycle cost analysis was performed to evaluate both the present and future costs for a 20-year timeframe to directly compare each of the technically feasible alternatives. The life cycle costs, or net present value (NPV), is a way to present the value of a project by summing the capital costs and operations and maintenance (O&M) minus the present worth of the salvage value. This analysis utilized a 20-year planning period with a 2.0% discount interest rate to determine straight-line depreciation of components.

The NPV equation and variables are defined as (Agriculture, 2013):

NPV = C + USPW (O&M) – SPPW (S)

C = Capital Cost

USPW (O&M) = Uniform Series Present Worth of Annual O&M

SPPW (S) = Single Payment Present Worth of Salvage Value

Of the components of each alternative in this project, any new transmission pipelines, well columns, shafts, pumps, and motors have a lifespan at or beyond the planning period used, meaning that they are components with a salvage value.

Other components of the alternatives, such as transmission line gate valves, wellhouse piping and valves, panels and controls are considered short-lived assets and thus will be included in the O&M. While it is difficult to accurately predict when various components will need servicing or replacing, general practice assumes that smaller components will have a relatively shorter life than larger components. The two time periods used to develop the short-lived asset reserve were a 5-year and a 15-year period, with the assumption that wellhouse piping and valves may need replacing in 5-year intervals and the panels, controls, and gate valves may need replacing in 15-year intervals.

O&M costs were estimated for each alternative by combining estimates of labor, utilities, supplies, parts, repairs, chemicals, and various equipment replacement costs. Labor costs were estimated based on median salary in the District and the number of hours an operator might work under each alternative. Since the wells would only be used under emergency or backup conditions, it was assumed that the number of hours dedicated to operations and maintenance of Alternatives 1 and 3 were relatively low compared to the permanent WTP in Alternatives 2 and 4. Utilities were estimated by calculating the amount of energy that running the pumps for two weeks at the current cost per kilowatt hour in California, $0.33. Costs for chemical supplies, miscellaneous repairs, and equipment replacement were estimated using a proportion of the capital costs for each item. For Alternatives 2 and 4, the WTP operational costs were assumed to include chemical feed pumps and equipment, controls and instrumentation, standby power systems, tank cathodic protection systems, heating, electrical, air conditioning, ventilation, potassium permanganate for oxidation of the raw water stream, and filter media replacements.

O&M costs for Alternative 3 assume that the portable treatment units will be rented for 1 year out of every 10 years. The water supply assessment in Chapter 5 shows that the groundwater source will only be required during drought years if Clementia is used for domestic water storage. If Clementia is not used for storage, then groundwater would be required for the average year at buildout. This alternative assumes that Clementia will be used for domestic water storage and that groundwater supplementation will only be required in drought conditions.

Further, the NPV of Alternative 3 was evaluated in comparison to each of the other alternatives. An analysis was performed to determine what percentage of years the mobile treatment units would have to be rented in order to have an NPV equal to each of the other alternatives. For example, for the NPV of Alternative 3B (portable treatment, buildout conditions) to be as high as Alternative 2B (side-stream treatment, buildout conditions), the portable treatment units would have to be rented for 52% of the entire planning period. Since Alternative 3 can never have an NPV higher than Alternative 4 or lower than Alternative 1, percentages were not calculated with respect to these alternatives. These percentages are included in Table 6-9. For the NPV value shown for Alternative 3, it is assumed that the treatment units are rented for 10% of the planning period.

A summary of the present worth of the capital cost, annual and present worth O&M, and current and present worth salvage value is provided in Table 6-9. Detailed NPV analysis is included in Appendix E. A summary of these alternatives and their costs is presented in Table 6-9 below.

Table 6-9: Summary of Groundwater Alternative Costs

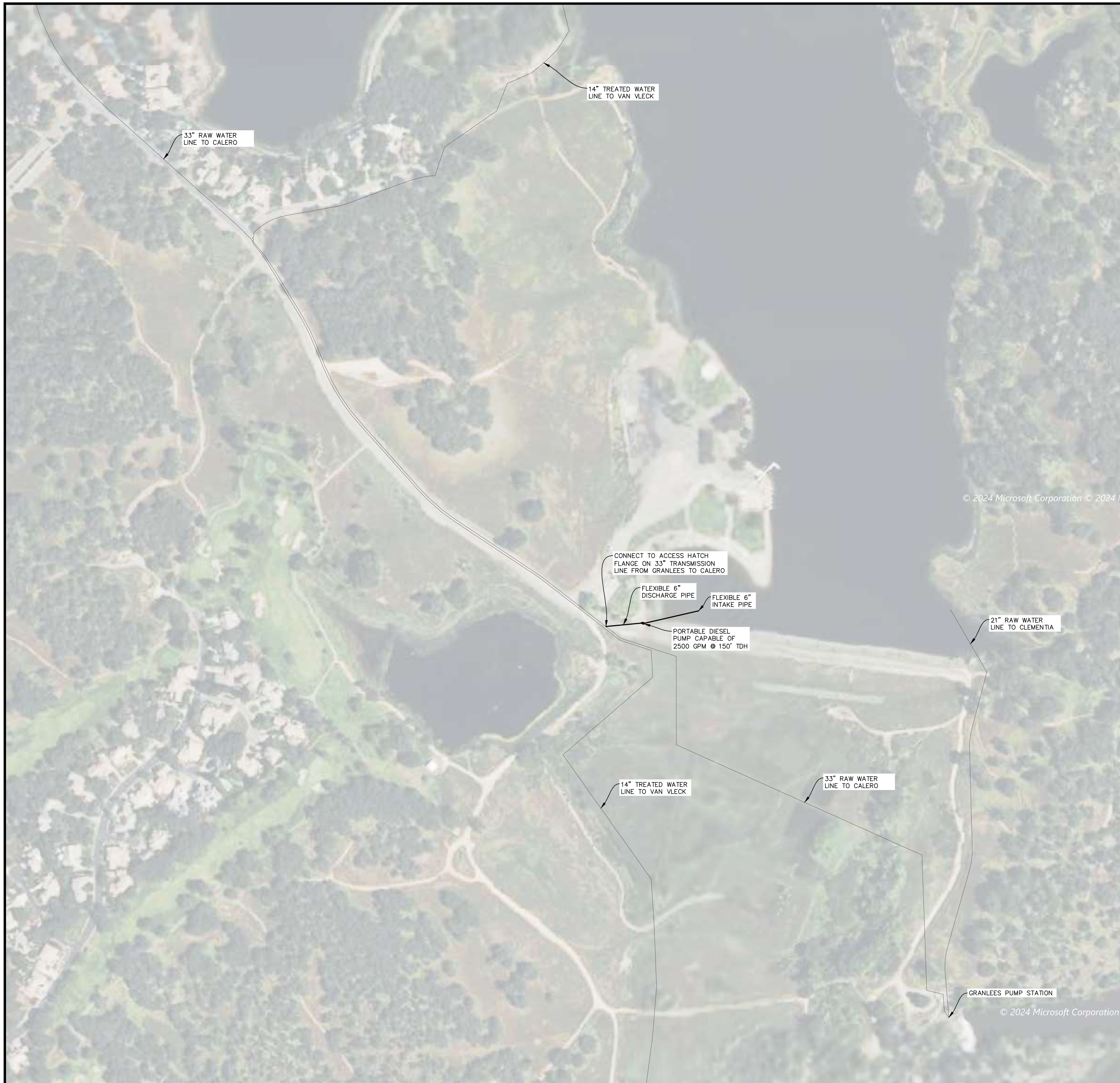
Summary of Groundwater Alternative Costs						
Alt #	**Description**	**NPV, Existing**	**NPV, Buildout**	**Capital Cost, Existing**	**Capital Cost, Buildout**	**Alt 3 Usage % for equal NPV**
1	No treatment	$7,212,200	$11,778,000	$6,349,000	$10,455,000	n/a
2	Side-stream treatment	$16,087,800	$24,177,000	$12,533,000	$21,284,000	54%[1] 52%[2]
3	Leased treatment (10% usage)	$9,480,200	$14,803,000	$6,349,000	$10,455,000	n/a
4	Full treatment	$24,204,000	$35,857,500	$17,184,000	$29,579,000	n/a
5	Use existing WTP	$11,376,700	$16,502,200	$11,987,000	$16,855,000	23%[1] 18%[2]
[1]Existing conditions alternative [2]Buildout conditions alternative						

6-2.5. Use Clementia for Storage

This alternative considers making the improvements necessary to begin using the storage capacity of Clementia for domestic water storage. This would include both infrastructure improvements and legal changes.

6-2.5.a. Infrastructure Improvement

The necessary infrastructure improvements to allow raw water storage for the potable system in Clementia include a portable pump station to lift water from Clementia to Calero. This would be achieved by connecting the portable pump station's discharge to the existing 33-inch transmission line from Granlees to Calero. The existing 33-inch transmission line has an access hatch located close to the southwest corner of Clementia. District staff have indicated that this hatch could be retrofitted to allow a pressurized connection to a pump station drawing water from Clementia. This would allow the use of the existing transmission line to transport water from Clementia to Calero, and then from Calero into the rest of the potable system. The pump should be sized for approximately the average day demand at buildout, which is nearly 2,100 gpm. It would need to be able to deliver between 100 and 150 feet of TDH depending on operating conditions and reservoir levels. The usable storage of Clementia is approximately 957 AF; it would take approximately 103 days for the pump to completely empty the reservoir. Therefore, the cost of renting the pump was estimated for approximately 100 days. See Figure 6-3 for a conceptual plan of this alternative.



LEGEND

— — — — —	EXISTING WATER LINE
— — — RW —	EXISTING RAW WATER LINE
—— · ——	PROPOSED PROPERTY LINE/LOT
—— ▾ ——	PROPOSED WATER LINE
P	PROPOSED PUMP

GENERAL NOTES

1. PORTABLE PUMP TO BE RENTED AND OPERATED PER MANUFACTURER'S INSTRUCTIONS DURING SEASON WHEN PUMPING FROM CLEMENTIA IS REQUIRED. APPROXIMATE PUMP DUTY POINT WILL BE 2100 GPM AT 150 TDH.

SITE PLAN NOTES

1. NO TOPOGRAPHIC SURVEY WAS PERFORMED. ALL LOCATIONS OF UTILITIES SHOWN ARE APPROXIMATE. ACTUAL SITE CONDITIONS AND LOCATIONS MAY VARY.
2. THIS IS A SCHEMATIC SITE PLAN FOR THE SOLE PURPOSE OF ILLUSTRATING THE GENERAL COMPONENTS AND FACILITIES. ACTUAL SITE LAYOUT AND LOCATIONS MAY VARY AND WILL NEED TO BE DETERMINED BY TOPOGRAPHIC SURVEY AND OTHER SITE INVESTIGATIONS AS REQUIRED.

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FOR
RANCHO MURIETA CSD
CONCEPTUAL SITE PLAN

SCALE	DATE	DRAWN BY	PROJ. NO.
SHOWN	08/08/2024	BAG	3971-02

FIGURE
6-3

Figure 6-4: Access Hatch to 33" Raw Water Transmission Main

6-2.5.b. *Legal Changes*

Currently, Clementia is not allowed to be used to store water that will ultimately be used in the potable water system. This is due to the California Health and Safety Code (HSC) section 115825, subdivision b, which states that reservoirs that permit body contact recreation cannot use their water for domestic use, unless the reservoir is specifically exempted under one of the statutory exemptions spelled out in HSC sections 115840 through 115843.6. Clementia is currently used by District residents for a variety of recreational activities, including body contact activities like swimming and boating. A technical memorandum published by West Yost on March 15, 2024 lists two possible options that would allow for the use of Clementia as a drinking water supply:

1. Bring the recreational use restrictions of Clementia in line with Chesbro and Calero by prohibiting body contact and gas motors. This would allow the District to apply for a permit for domestic use of Clementia.
2. Pursue State legislation to obtain a statutory exemption for the reservoir to allow continued use of body contact simultaneous with domestic use.

To pursue option 1, the District would need to complete the necessary internal process to ban body contact recreation in Clementia. It could then begin the application process with the State Division of Drinking Water (DDW). The permitting process could include further lake and watershed studies to evaluate potential contaminants in Clementia that may not be present in Calero and Chesbro. DDW has indicated that they will detail the required studies at the time of the permit request.

To pursue option 2, the District would need to have further discussions with the California Office of Chief Counsel (OCC). The DDW Sacramento District Engineer has offered to facilitate these discussions as there is not a set process for evaluating and establishing an exemption. Once the process is better defined, DDW would work together with the OCC to make the determination and set the conditions for use. Finally, if the exemption is granted, the District would still need to complete the permitting process and the DDW permit requirements discussed previously would still apply. See Table 6-10 for a cost summary.

Table 6-10: Alternative 6 Cost Summary

Alternative 6 – Use Clementia for Domestic Storage	
Project Component	**Estimated Cost**
Retrofit access hatch to allow 6-inch connection	$10,000
100-day rental of portable pump and suction/discharge pipes	$27,700
Diesel fuel for 100-day run time	$44,400
Legal fees to attain statutory exemption	$100,000

6-3. Treated Domestic Storage Improvements

6-3.1. Overview

As developed in Chapter 5, the District has a deficit in treated water storage in both existing and buildout conditions. Alternatives presented in this section aim to address the District's deficit in storage by installing three new storage tanks totaling 3.4 MG of storage. These tanks will operate in tandem with Van Vleck; the operating water levels in the proposed tanks will be at the same height as those in Van Vleck.

In each of these alternatives, it is assumed that the Rancho North developments (Villages D through H) and the Residences East and West will be annexed into the Rio Oso pressure zone and that a new booster station (described later) will provide pressure to a new pressure zone that will include Villages A, B, and C, along with the Retreats and parts of the existing system along De La Cruz Drive. The additional 3.4 MG of storage allows Rio Oso to be filled sufficiently by the WTP and the other storage tanks to meet the needs of its pressure zone without exceeding Rio Oso's pump capacities.

See Figure 6-5 for a concept map showing the proposed booster station (described later) and storage tank in Village C and Figure 6-6 for a concept map showing the proposed new tank in Village H and the proposed new tank at the existing Van Vleck tank site.

NEW TANK AND BOOSTER STATION AT VILLAGE C

100 0 50 100
(FEET)

PROPOSED IMPROVEMENTS

1. INSTALL NEW GLASS-LINED BOLTED STEEL 1.0 MG TANK.

   DIAMETER: 65 FEET
   HEIGHT: 40 FEET
   BASE ELEVATION: 301 FEET

   OPERATIONAL RANGE: 35.5 TO 37.5 FEET

   TANK OPERATIONAL RANGE IN TANDEM WITH VAN VLECK TANK.

2. TIE-IN TO EXISTING 16" ACP TRANSMISSION PIPELINE WITH 12" C900 PVC PIPE.
3. INSTALL APPROXIMATELY 850 LF OF 12" C900 PVC PIPE. VALVING SHALL BE SUCH THAT WATER CAN ONLY FLOW INTO THE TANK ALONG THIS PIPELINE.
4. INSTALL BOOSTER STATION AT TOP OF VILLAGE C.

   INSTALL TWO PUMPS: ONE LEAD AND ONE LAG.
   PUMPS TO DELIVER 1390 GPM AT 75 FEET TDH.
   PUMP MOTORS 40 HP.

5. INSTALL APPROXIMATELY 910 LF OF 12" C900 PVC PIPE.
6. TIE INTO NEW 12" PVC PIPE (FORMERLY 8" ACP).

LEGEND

	EXISTING WATER LINE
	EXISTING FIRE HYDRANT
	PROPOSED PROPERTY LINE/LOT
	PROPOSED WATER LINE
	PROPOSED FIRE HYDRANT
	PROPOSED WATER VALVE
P	PROPOSED PUMP
	PROPOSED BUILDING EDGE

GENERAL NOTES

1. NEW TANK TO OPERATE IN TANDEM WITH VAN VLECK. IT SHARES OPERATIONAL LEVELS AND OPERATIONAL RULES WITH VAN VLECK.

   1.1. UNDER NORMAL OPERATION, WATER IS ONLY ALLOWED TO FLOW INTO THE NEW TANK FROM THE EXISTING VAN VLECK TRANSMISSION LINE. IN EMERGENCY CASES WHERE OTHER SYSTEM TANK LEVELS DROP SIGNIFICANTLY, THE TANK COULD BE CONFIGURED TO ALLOW REVERSE FLOW BACK INTO THE TRANSMISSION LINE TO THE WTP.

   1.2. IN THIS ALTERNATIVE, THE RANCHO NORTH DEVELOPMENTS (VILLAGES D, E, F, G, AND H) ARE SUPPLIED AS PART OF THE RIO OSO PRESSURE ZONE.

   1.3. THIS ALTERNATIVE PROVIDES LOCAL STORAGE FOR VILLAGES A, B, C, THE RETREATS, AND EXISTING CONNECTIONS ON DE LA CRUZ DRIVE.

2. BOOSTER STATION OPERATES USING PRESSURE AT THE HIGHEST ELEVATION NODE IN THE PRESSURE ZONE.

SITE PLAN NOTES

1. NO TOPOGRAPHIC SURVEY WAS PERFORMED. ALL LOCATIONS OF UTILITIES SHOWN ARE APPROXIMATE. ACTUAL SITE CONDITIONS AND LOCATIONS MAY VARY.
2. THIS IS A SCHEMATIC SITE PLAN FOR THE SOLE PURPOSE OF ILLUSTRATING THE GENERAL COMPONENTS AND FACILITIES. ACTUAL SITE LAYOUT AND LOCATIONS MAY VARY AND WILL NEED TO BE DETERMINED BY TOPOGRAPHIC SURVEY AND OTHER SITE INVESTIGATIONS AS REQUIRED.

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DISTRIBUTION AND STORAGE ALTERNATIVES
FOR
RANCHO MURIETA CSD
TANK AND BOOSTER IN VILLAGE C

SCALE	DATE	DRAWN BY	PROJ. NO.
SHOWN	08/08/2024	BAG	3971-02

FIGURE
6-5

NEW TANK AT VILLAGE H

NEW TANK AT VAN VLECK

100 0 50 100
(FEET)

LEGEND

— — w — — w —	EXISTING WATER LINE
	EXISTING FIRE HYDRANT
	PROPOSED PROPERTY LINE/LOT
—— w ——	PROPOSED WATER LINE
	PROPOSED FIRE HYDRANT
	PROPOSED WATER VALVE
	PROPOSED BUILDING EDGE

SITE PLAN NOTES

1. NO TOPOGRAPHIC SURVEY WAS PERFORMED. ALL LOCATIONS OF UTILITIES SHOWN ARE APPROXIMATE. ACTUAL SITE CONDITIONS AND LOCATIONS MAY VARY.
2. THIS IS A SCHEMATIC SITE PLAN FOR THE SOLE PURPOSE OF ILLUSTRATING THE GENERAL COMPONENTS AND FACILITIES. ACTUAL SITE LAYOUT AND LOCATIONS MAY VARY AND WILL NEED TO BE DETERMINED BY TOPOGRAPHIC SURVEY AND OTHER SITE INVESTIGATIONS AS REQUIRED.

GENERAL NOTES

1. NEW TANKS TO OPERATE IN TANDEM WITH VAN VLECK. THEY SHARE OPERATIONAL LEVELS AND OPERATIONAL RULES WITH VAN VLECK.

1.1. THE TANK IN VILLAGE H HAS BOTH AN INLET AND DISCHARGE PIPELINE TO IT. THESE CONNECT TO THE 14" TRANSMISSION LINE FROM THE WTP TO RIO OSO. WATER IS ONLY ALLOWED TO FLOW INTO THE TANK VIA THE INLET LINE AND OUT OF THE TANK VIA THE DISCHARGE LINE. THE LINK OF TRANSMISSION PIPELINE BETWEEN THE INLET AND DISCHARGE CONNECTIONS SHALL BE CLOSED, SO THAT WATER FROM THE NEW TANK CANNOT FLOW BACK TOWARD THE WTP AND INTO VAN VLECK. UNDER NORMAL OPERATING CONDITIONS, ALL WATER DISCHARGED FROM THE TANK SHALL FLOW TO RIO OSO. IN EMERGENCY CONDITIONS WHERE OTHER TANKS IN THE SYSTEM DROP BELOW A CERTAIN POINT, THE VILLAGE H TANK SHALL BE ALLOWED TO PROVIDE WATER THROUGH THE TRANSMISSION LINE FROM THE WTP.

1.2. THE NEW TANK NEXT TO VAN VLECK SHALL OPERATE IN TANDEM WITH VAN VLECK. THE GENERAL OPERATING CONDITIONS SHALL NOT ALLOW WATER TO FLOW BOTH DIRECTIONS, BUT SHALL HAVE ALL WATER DISCHARGED INTO THE VAN VLECK GRAVITY ZONE. IN EMERGENCY CONDITIONS WHERE OTHER TANKS IN THE SYSTEM DROP BELOW A CERTAIN POINT, THE VAN VLECK TANKS SHALL BE ALLOWED TO PROVIDE WATER THROUGH THE TRANSMISSION LINE FROM THE WTP.

1.3. IN THIS ALTERNATIVE, THE RANCHO NORTH DEVELOPMENTS (VILLAGES D, E, F, G, AND H) ARE SUPPLIED AS PART OF THE RIO OSO PRESSURE ZONE.

1.4. THIS ALTERNATIVE PROVIDES ADDITIONAL LOCAL STORAGE TO THE VAN VLECK GRAVITY ZONE AND THE RIO OSO PRESSURE ZONE.

PROPOSED IMPROVEMENTS

1. INSTALL NEW GLASS–LINED BOLTED STEEL 1.0 MG TANK.

   DIAMETER: 65 FEET
   HEIGHT: 40 FEET
   BASE ELEVATION: 301 FEET

   OPERATIONAL RANGE: 35.5 TO 37.5 FEET

   TANK OPERATIONAL RANGE IN TANDEM WITH VAN VLECK TANK.

2. INSTALL NEW GLASS–LINED BOLTED STEEL 1.4 MG TANK.

   DIAMETER: 90 FEET
   HEIGHT: 30 FEET
   BASE ELEVATION: 311 FEET

   OPERATIONAL RANGE: 25.5 TO 27.5 FEET

   TANK OPERATIONAL RANGE IN TANDEM WITH VAN VLECK TANK.

3. TIE–IN TO EXISTING 14" ACP TRANSMISSION PIPELINE WITH 12" C900 PVC PIPE.
4. PIPE LINK TO BE CLOSED.
5. INSTALL APPROXIMATELY 660 LF OF 12" C900 PVC INLET AND DISCHARGE PIPES. VALVING SHALL BE SUCH THAT WATER CAN FLOW ONE DIRECTION ALONG THESE PIPES.
6. INSTALL APPROXIMATELY 200 LF OF 12" C900 PVC INLET AND DISCHARGE PIPES. VALVING SHALL BE SUCH THAT WATER CAN FLOW ONE DIRECTION ALONG THESE PIPES IN NORMAL OPERATING CONDITIONS. VALVING SHALL ALLOW WATER TO FLOW BOTH DIRECTIONS IN EMERGENCY CONDITIONS.
7. TIE–IN TO EXISTING 16" ACP TRANSMISSION PIPE WITH 12" C900 PVC PIPE.
8. DO NOT CONNECT TO DISTRIBUTION SYSTEM.

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DISTRIBUTION AND STORAGE ALTERNATIVES FOR RANCHO MURIETA CSD

NEW TANKS AT VILLAGE H AND VAN VLECK

SCALE SHOWN
DATE 08/08/2024
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FIGURE 6-6

6-3.2. Tank in Village C

This alternative considers the installation of a new 1.0 MG tank at the highest elevation in Village C, at the end of the proposed cul-de-sac in Village C just east of Camino Del Lago Drive. The tank would be located on proposed Lot A and Lot G, which share the top of the hill. Precise siting will be determined during the design phase. This tank's base should be near 300 feet, with a height of 40 feet and a diameter of 66 feet. In order to match its operational range with Van Vleck water levels, the new tank's operational range will be roughly between 35.5 feet and 37.5 feet. To receive flows, a 850 LF 12" C900 PVC pipeline is proposed to tie into the existing 16" transmission line between Van Vleck and the WTP. Approximately 15 LF of 12" C900 PVC pipeline is proposed to deliver water from the tank to the booster station, which will pump water into the new distribution system at the end of the proposed cul-de-sac. The new tank will operate in tandem with the existing Van Vleck tank and provide 1.0 MG of storage for the new ABC pressure zone, which is one of the requirements discussed in Chapter 5.

Installation of this tank includes the tank itself, site work and excavation, a concrete slab tank foundation, overflow piping, SCADA, telemetry, controls, and connecting to the existing distribution system. If selected, this tank should be installed at or near the time that the booster station at Village C is being constructed to optimize working schedules, road closures and traffic controls, and excavation work. A summary of estimated costs for this tank is shown in Table 6-11.

Table 6-11: Alternative 7 Cost Summary

Alternative 7 – Village C Tank	
Project Component	**Estimated Cost**
Mobilization/Demobilization, Traffic Controls:	$127,800
Site Work, Excavation, Tank Foundation:	$617,800
1.0 MG Tank, Piping, Valves, SCADA, Controls:	$1,277,600
Install Pipelines and Connect to Existing:	$215,400
Subtotal:	$2,239,000
Construction Contingencies:	$448,000
Design, Engineering, Construction Admin:	$560,000
Environmental, Permitting, Legal, Land Acquisition:	$25,000
Total Estimated Project Cost:	$3,272,000

6-3.3. Tank in Village H

This alternative considers the installation of a new 1.0 MG tank to the east of the proposed cul-de-sac in Village H . This tank's base should be near 300 feet, with a height of 40 feet and a diameter of 65 feet. To match its operational range with Van Vleck water levels, the new tank's operational range would be roughly 35.5 feet to 37.5 feet. To receive and discharge flows, two 650 LF 12" C900 PVC pipelines, one for incoming water and one for outgoing water, are proposed to tie into the existing 14" transmission line between Rio Oso and the WTP. The small section of transmission pipe between the connections to incoming line and outgoing line should be closed, and a check valve installed on the transmission line into the tank. This will cause water coming from the WTP to Rio Oso to pass through the new tank in only one direction. Effectively, this tank increases the storage in the Rio Oso pressure zone by 1.0 MG, which is one of the current deficiencies discussed in Chapter 5.

Installation of this tank includes the tank itself, site work and excavation, a concrete slab tank foundation, overflow piping, SCADA, telemetry and other controls, and connecting to the existing distribution system. If selected, this tank should be installed at or near the time that the new waterlines for Village H are being constructed to optimize working schedules, road closures and traffic controls, and excavation work. A summary of estimated costs for this tank is shown in Table 6-12.

Table 6-12: Alternative 8 Cost Summary

Alternative 8 – Village H Tank	
Project Component	**Estimated Cost**
Mobilization/Demobilization, Traffic Controls:	$134,300
Site Work, Excavation, Tank Foundation:	$617,800
1.0 MG Tank, Piping, Valves, SCADA, Controls:	$1,277,600
Install Pipelines and Connect to Existing:	$323,400
Subtotal:	$2,353,000
Construction Contingencies:	$471,000
Design, Engineering, Construction Admin:	$589,000
Environmental, Permitting, Legal, Land Acquisition:	$25,000
Total Estimated Project Cost:	$3,438,000

6-3.4. Tank at Van Vleck

This alternative considers the installation of a new 1.4 MG tank to the east of the existing Van Vleck Tank. This tank's base should be at the same elevation as the existing Van Vleck tank (approximately 311 feet), with a height of 30 feet and a diameter of 90 feet. To match its operational range with Van Vleck water levels, the new tank's operational range would be roughly 25.5 feet to 27.5 feet. To receive and discharge flows, two 200 LF 12" C900 PVC pipelines, one for incoming water and one for outgoing water, are proposed to tie into the existing 16" transmission lines from the existing Van Vleck tank. The tank supply line should tap into the 16" pipe from the WTP, and the tank discharge line should tap into the 16" pipe to Murieta South. The new tank will operate in tandem with the existing Van Vleck tank and increase the storage for the Van Vleck gravity pressure zone by 1.4 MG, which is one of the current deficiencies discussed in Chapter 5.

Installation of this tank includes the tank itself, site work and excavation, a concrete slab tank foundation, overflow piping, SCADA, telemetry and other controls, and connecting to the existing distribution system. A summary of estimated costs for this tank is shown in Table 6-13.

Table 6-13: Alternative 9 Cost Summary

Alternative 9 – New Van Vleck Tank	
Project Component	**Estimated Cost**
Mobilization/Demobilization, Traffic Controls:	$166,400
Site Work, Excavation, Tank Foundation:	$884,100
1.4 MG Tank, Piping, Valves, SCADA, Controls:	$1,748,700
Install Pipelines and Connect to Existing:	$116,600
Subtotal:	$2,916,000
Construction Contingencies:	$584,000
Design, Engineering, Construction Admin:	$729,000
Environmental, Permitting, Legal, Land Acquisition:	$25,000
Total Estimated Project Cost:	$4,254,000

6-3.5. Alternatives Not Considered

To address the global storage deficiency under the buildout conditions, providing a single storage tank to provide the required additional storage was considered. However, after further analysis, this option was rejected. As developed in Chapter 5, the District wants to improve the resiliency of its storage by providing sufficient storage within each zone. A single new tank would not provide the same level of resiliency as three tanks that each provide storage to their respective zone. Further, as mentioned in Chapter 5, the CCR requires tanks to have separate inlet and outlet connections. In order for a single storage tank to provide required storage, it would be required to have a single inlet/outlet connection to the system and would "float" its operating level based on Van Vleck, with water flowing in both directions through its supply pipe. This is not in accordance with the CCR and is not recommended. The proposed alternatives for storage allow the new tanks to operate in tandem with Van Vleck, but also maintain separation and one-directional flow through the system.

6-3.6. Operational Recommendations

The above discussion demonstrates the benefits of providing separate storage capacity for each pressure zone, as well as for preventing the flow of water in both directions from a tank. For those reasons, it is recommended that the bi-directional flow from Van Vleck be discontinued after the new storage tanks are constructed. The new tank in Village H will provide adequate storage to the Rio Oso zone, so additional flows from Van Vleck should not be necessary. All flows that enter the Van Vleck tanks should be discharged to the Van Vleck pressure zone via the pipeline to Murieta South. Similarly, all flows to Rio Oso and the new Village H tank should be discharged to the Rio Oso pressure zone; the gravity connection from Rio Oso to the Van Vleck pressure zone should be closed during normal operating conditions. These operating conditions will allow the District to comply with the CCR by only allowing flow to enter its tanks via inlet pipelines and exit its tanks via discharge pipelines. This also will improve the accuracy of metering at the tanks.

However, the operating rules can still allow for the tanks to help each other in emergency scenarios. For example, if the level in Rio Oso drops below 15 feet due to a fire, the valves that would normally be closed to prevent bi-directional flow along the transmission line from the WTP to Van Vleck could open, allowing Van Vleck's capacity to assist the Rio Oso zone. In the opposite scenario, if Van Vleck's level dropped below a certain mark, the gravity pipe that connects Rio Oso to the Van Vleck gravity zone (normally closed) could open if a control valve was installed, allowing Rio Oso and the new Village H tank to assist the Van Vleck zone. The gravity line between Rio Oso and the Van Vleck pressure zone is currently operated manually. This operating strategy would allow the District to maintain adequate storage separately in each zone while also taking advantage of the global storage in the system in case of emergency.

6-4. Distribution System Improvements

6-4.1. Overview

As developed in Chapter 5, review of existing infrastructure and fire coverage rules were utilized to determine alternatives for distribution improvements. Only the distribution improvements that upgrade existing infrastructure or benefit the entire system are included in the CIP – distribution improvements that only serve new developments are assumed to be funded by the developers.

6-4.2. New Booster Station in Village C

To provide pressure and flow from the proposed new tank in Village C (see Alternative 7), a new pump station located adjacent to the new tank is proposed. The proposed pump motors for the booster station and pipelines were sized iteratively using EPANet2.2. Pump power was balanced with motor size to maintain appropriate price-points. For the regular duty pumps, two 25-HP pumps were determined to be sufficient to provide max day flow, with one pump operating as the lead pump and the second coming on to provide additional pressure during high points during the day. For fire flows, an additional 40-HP pump was determined to be sufficient to provide fire flows to the new pressure zone. For each of the three pumps, an efficiency of 60% was assumed. The pumps should also be equipped with VFDs to allow for a range of operating points. Pipelines were sized using target velocity of 5 fps during normal operation and a maximum velocity of 7 fps during a fire to optimize function and cost. This resulted in a 14" discharge pipeline from the pump station to the distribution system. See Figure 6-5 for a concept map of this alternative together with the adjacent new tank alternative.

Installation of the booster station includes two regular duty 25-HP pumps and one 40-HP fire pump station (which includes a backup 40-HP pump), motors, a pump house with necessary piping, valves, flowmeters, VFDs, panels, SCADA controls, and power distribution. The pump station location is proposed to be next to the new tank in Village C at the end of the proposed cul-de-sac, and the pipeline to serve this pump station is proposed to come out of the new tank. 850 LF of 12" C900 PVC pipe is required to connect the new tank to the transmission line from the WTP to Van Vleck. The cost of this pipeline is included in the estimate for the new tank. 54 LF of 14" C900 PVC pipe is proposed to deliver water from the booster station to the distribution system by tying into the new proposed distribution line along the proposed cul-de-sac in Village C. The cost for this pipeline is included in the estimate for this booster station. The distribution piping for the new villages will allow the booster to serve all the required areas.

This booster station should be installed near the time that the distribution system for Villages A, B, and C is being constructed to optimize working schedules, road closures and traffic controls, and excavation work. A summary of estimated costs for this booster station is shown in Table 6-14.

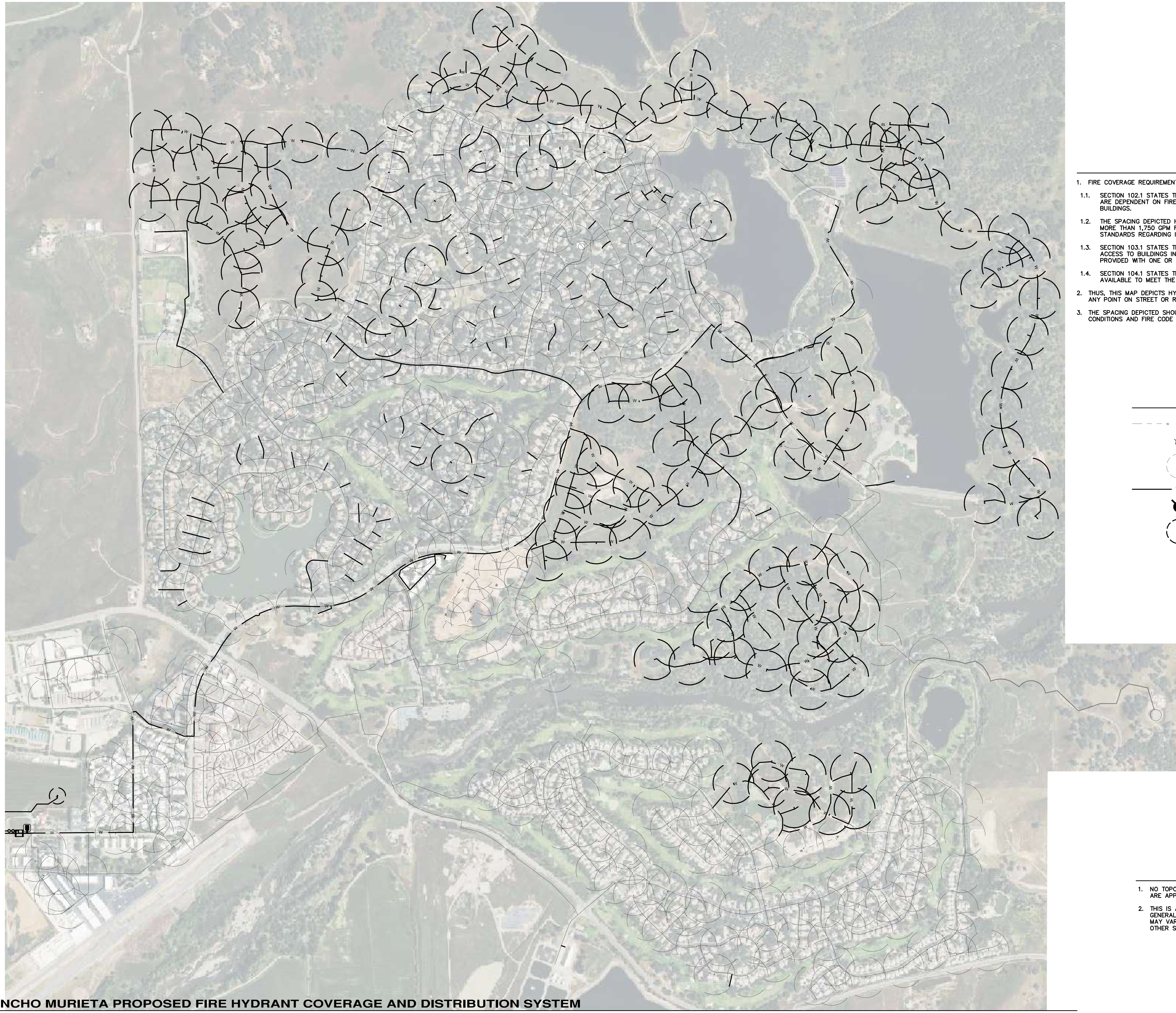
Table 6-14: Alternative 10 Cost Summary

Alternative 10 – Village C Booster Station	
Project Component	**Estimated Cost**
Mobilization/Demobilization, Traffic Controls:	$65,300
Install two 25-HP Pumps, two 40-HP Pumps, Motors, & Generators:	$304,800
Pump House, Controls, Power:	$529,300
Install Pipelines and Connect to Existing:	$32,500
Subtotal:	$1,143,000
Construction Contingencies:	$229,000
Design, Engineering, Construction Admin:	$286,000
Environmental, Permitting, Legal, Land Acquisition:	$20,000
Total Estimated Project Cost:	$1,678,000

6-4.3. Fire Suppression Improvements

As described above, the system's ability to provide fire protection during buildout conditions was evaluated. Four primary criteria were evaluated: fire hydrant coverage, pipeline velocities, available flow, and residual pressure during a fire event. Adequate fire hydrant coverage was determined by drawing 250-foot radius circles around each existing hydrant and determining which areas need new hydrants to achieve coverage.

Figure 6-7 shows the proposed fire hydrant coverage map, with proposed new hydrants in the new developments, as well as some new hydrants in the existing developments where insufficient coverage was discovered. In total, it was determined that 13 additional hydrants are required to provide sufficient coverage within the existing system, in addition to the 117 new proposed hydrants in the new developments, for a total of 130 new hydrants. See Figure 6-7 for a concept map of the proposed new hydrant locations, each with 250-foot radius circles around them.



RANCHO MURIETA PROPOSED FIRE HYDRANT COVERAGE AND DISTRIBUTION SYSTEM

600 0 300 600
(FEET)

FIRE COVERAGE NOTES

1. FIRE COVERAGE REQUIREMENTS FROM 2015 INTERNATIONAL FIRE CODE (IFC), APPENDIX C, SECTIONS 101 – 105.
   1.1. SECTION 102.1 STATES THAT MINIMUM NUMBER OF ADJACENT HYDRANTS AND MAXIMUM SPACING FOR HYDRANTS ARE DEPENDENT ON FIRE–FLOW REQUIREMENTS FOR AN INDIVIDUAL BUILDING OR AREA CONTAINING MANY BUILDINGS.
   1.2. THE SPACING DEPICTED HEREIN OPERATES UNDER THE ASSUMPTION THAT RESIDENTIAL BUILDINGS REQUIRE NO MORE THAN 1,750 GPM FIRE–FLOW, AND THAT INDUSTRIAL AREAS ARE PROPERLY EQUIPPED TO MEET IFC STANDARDS REGARDING INSTALLED FIRE SUPPRESSION TECHNOLOGIES.
   1.3. SECTION 103.1 STATES THAT FIRE APPARATUS ACCESS ROADS AND PUBLIC STREETS PROVIDING REQUIRED ACCESS TO BUILDINGS IN ACCORDANCE WITH SECTION 503 OF THE INTERNATIONAL FIRE CODE SHALL BE PROVIDED WITH ONE OR MORE FIRE HYDRANTS, AS DETERMINED BY SECTION C102.1.
   1.4. SECTION 104.1 STATES THAT EXISTING FIRE HYDRANTS ON PUBLIC STREETS ARE ASSUMED TO BE CONSIDERED AS AVAILABLE TO MEET THE REQUIREMENTS OF SECTIONS C102 AND C103.
2. THUS, THIS MAP DEPICTS HYDRANTS WITH A SPACING REQUIREMENT OF 500–FEET, WITH A MAXIMUM DISTANCE FROM ANY POINT ON STREET OR ROAD FRONTAGE TO A HYDRANT OF 250–FEET.
3. THE SPACING DEPICTED SHOULD ONLY BE USED FOR CONCEPTUAL AND PRELIMINARY DESIGNS. ACTUAL SITE CONDITIONS AND FIRE CODE REQUIREMENTS MAY VARY FROM THOSE USED IN THE DEVELOPMENT OF THESE FIGURES.

LEGEND

EXISTING WATER LINE
EXISTING FIRE HYDRANT
EXISTING FIRE HYDRANT COVERAGE RADIUS – 250 FEET
PROPOSED WATER LINE
PROPOSED FIRE HYDRANT
PROPOSED FIRE HYDRANT COVERAGE RADIUS – 250 FEET

SITE PLAN NOTES

1. NO TOPOGRAPHIC SURVEY WAS PERFORMED. ALL LOCATIONS OF UTILITIES SHOWN ARE APPROXIMATE. ACTUAL SITE CONDITIONS AND LOCATIONS MAY VARY.
2. THIS IS A SCHEMATIC SITE PLAN FOR THE SOLE PURPOSE OF ILLUSTRATING THE GENERAL COMPONENTS AND FACILITIES. ACTUAL SITE LAYOUT AND LOCATIONS MAY VARY AND WILL NEED TO BE DETERMINED BY TOPOGRAPHIC SURVEY AND OTHER SITE INVESTIGATIONS AS REQUIRED.

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DISTRIBUTION AND STORAGE ALTERNATIVES
FOR
RANCHO MURIETA CSD
PROPOSED FIRE HYDRANT COVERAGE MAP

SCALE	SHOWN
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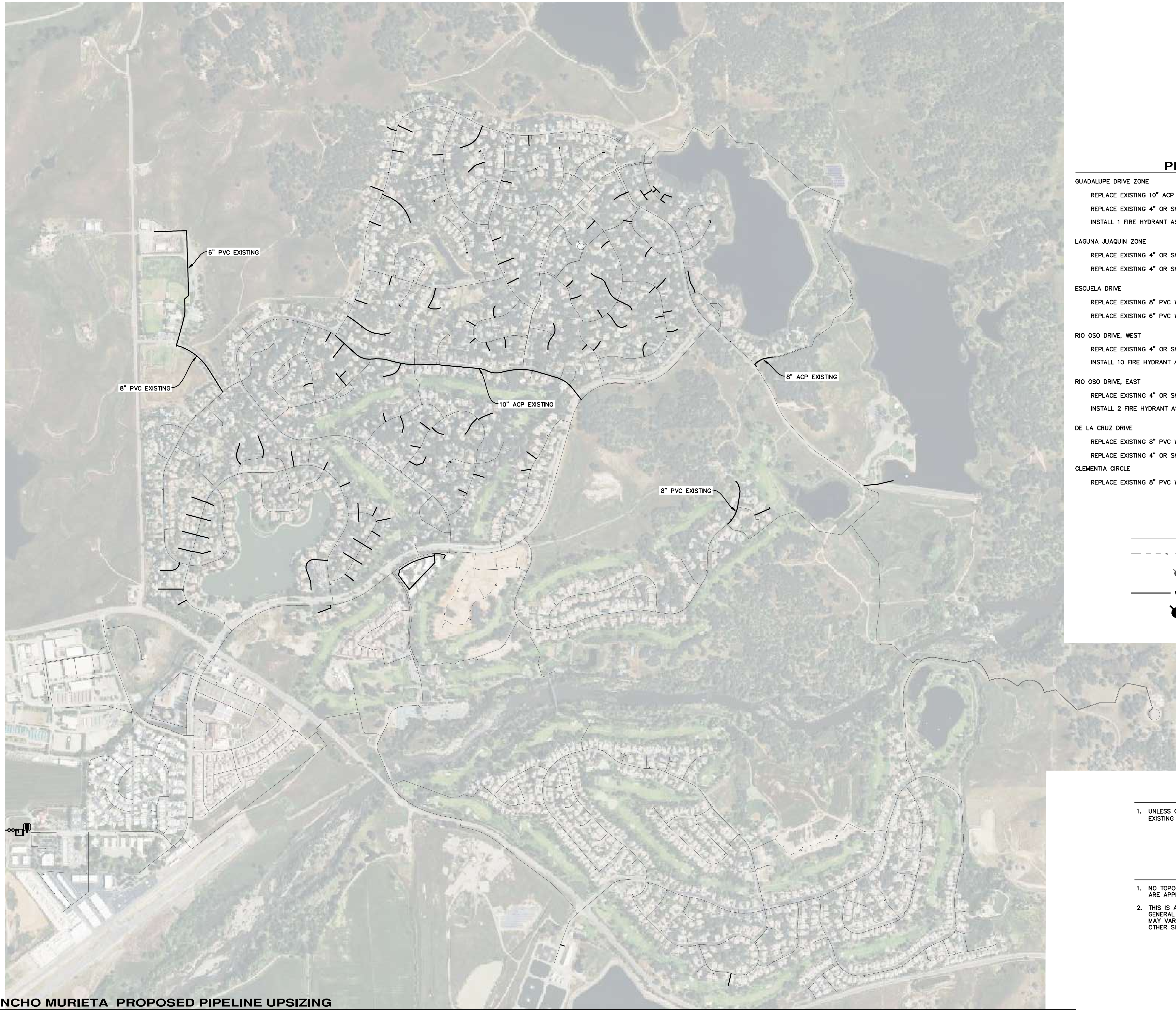
FIGURE 6-7

In addition to new fire hydrants, the system's ability to convey fire flows within the design criteria was evaluated. As discussed in the design criteria section, a value of 7 fps was determined to be the maximum allowable velocity within the system during a fire event. Excessively high velocities have several negative effects on a system, including excessive head loss, higher pumping costs, and decreased water quality from scale being dislodged from pipe walls. The simplest way to decrease the velocity is to replace the existing pipe with a larger one.

Pipe velocities above 7 fps were deemed unacceptable for fire performance. This includes all 4" and smaller diameter pipes. A 4" pipe is not capable of carrying fire flow volumes efficiently—1500 GPM through a 4" pipe results in a velocity of 38 fps. Many of the cul-de-sacs and other dead-end pipes in the system are 4". Therefore, it is recommended that all pipes with a 4" or smaller diameter be replaced with 8" diameter pipe. Additionally, there are several other pipes in the system that are undersized for fire flows. These include the 10" ACP pipe along Guadalupe Drive and the 8" and 6" pipe extending to Escuela Park. There are also existing pipes that should be upsized in anticipation of the new developments and their demands. These include the 8" pipe at the northeast end of De La Cruz Drive, which will serve as a key connection between Villages A and B, and the 8" pipe along Clementia Circle, which will be the primary discharge pipe from the new booster station in Village C. See Figure 6-8 below for a concept map of the pipes to be upsized and see Table 6-15 below for a summary of the estimated costs.

Table 6-15: Alternative 11 Cost Summary

Alternative 11 – Fire Suppression Improvements	
Project Component	**Estimated Cost**
Mobilization/Demobilization, Traffic Controls:	$528,300
4” and Smaller Pipeline Upsizing:	$3,578,400
13 New Fire Hydrant Assemblies:	$114,400
Upsizing for Current Deficiencies:	$1,332,900
Upsizing for Buildout Deficiencies:	$209,000
Subtotal:	$5,763,000
Construction Contingencies:	$1,153,000
Design, Engineering, Construction Admin:	$1,441,000
Environmental, Permitting, Legal, Land Acquisition:	$40,000
Total Estimated Project Cost:	$8,397,000



600 0 300 600
(FEET)

PROPOSED IMPROVEMENTS

GUADALUPE DRIVE ZONE

REPLACE EXISTING 10" ACP WITH APPROXIMATELY 3,282 LF OF 12" C900 PVC.

REPLACE EXISTING 4" OR SMALLER PIPE OF UNKNOWN MATERIAL WITH APPROXIMATELY 2,910 LF OF 8" C900 PVC.

INSTALL 1 FIRE HYDRANT ASSEMBLY.

LAGUNA JUAQUIN ZONE

REPLACE EXISTING 4" OR SMALLER PIPE OF UNKNOWN MATERIAL WITH APPROXIMATELY 5,131 LF OF 8" C900 PVC.

REPLACE EXISTING 4" OR SMALLER PIPELINES AT THE VILLAS WITH APPROXIMATELY 1,507 LF OF 8" PVC.

ESCUELA DRIVE

REPLACE EXISTING 8" PVC WITH APPROXIMATELY 837 LF OF 10" C900 PVC.

REPLACE EXISTING 6" PVC WITH APPROXIMATELY 1,842 LF 10" C900 PVC.

RIO OSO DRIVE, WEST

REPLACE EXISTING 4" OR SMALLER PIPE OF UNKNOWN MATERIAL WITH APPROXIMATELY 4,651 LF OF 8" C900 PVC.

INSTALL 10 FIRE HYDRANT ASSEMBLIES.

RIO OSO DRIVE, EAST

REPLACE EXISTING 4" OR SMALLER PIPE OF UNKNOWN MATERIAL WITH APPROXIMATELY 3,990 LF OF 8" C900 PVC.

INSTALL 2 FIRE HYDRANT ASSEMBLIES.

DE LA CRUZ DRIVE

REPLACE EXISTING 8" PVC WITH APPROXIMATELY 585 LF OF 12" C900 PVC.

REPLACE EXISTING 4" OR SMALLER PIPE OF UNKNOWN MATERIAL WITH APPROXIMATELY 220 LF OF 8" C900 PVC.

CLEMENTIA CIRCLE

REPLACE EXISTING 8" PVC WITH APPROXIMATELY 280 LF OF 12" C900 PVC.

LEGEND

EXISTING WATER LINE

EXISTING FIRE HYDRANT

PROPOSED WATER LINE

PROPOSED FIRE HYDRANT

GENERAL NOTES

1. UNLESS OTHERWISE NOTED, THE PROPOSED PIPELINES SHOWN ARE TO REPLACE EXISTING DISTRIBUTION LINES 4" OR SMALLER

SITE PLAN NOTES

1. NO TOPOGRAPHIC SURVEY WAS PERFORMED. ALL LOCATIONS OF UTILITIES SHOWN ARE APPROXIMATE. ACTUAL SITE CONDITIONS AND LOCATIONS MAY VARY.
2. THIS IS A SCHEMATIC SITE PLAN FOR THE SOLE PURPOSE OF ILLUSTRATING THE GENERAL COMPONENTS AND FACILITIES. ACTUAL SITE LAYOUT AND LOCATIONS MAY VARY AND WILL NEED TO BE DETERMINED BY TOPOGRAPHIC SURVEY AND OTHER SITE INVESTIGATIONS AS REQUIRED.

RANCHO MURIETA PROPOSED PIPELINE UPSIZING

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DISTRIBUTION AND STORAGE ALTERNATIVES FOR RANCHO MURIETA CSD

PROPOSED PIPELINE UPSIZING

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FIGURE 6-8

6-4.4. Developer-Funded Distribution Improvements

As a part of the modeling effort for this IWMP, it was necessary to model the projected buildout distribution system with the projected future demands to ensure that improvement alternatives were appropriate for the buildout conditions. This resulted in the development of a model of the distribution system at buildout. Pipelines were assigned a minimum size of 8” while larger transmission lines were sized in EPANet2.2. These are often 12” in size, though some smaller 10” lines were determined to be adequate. See Figure 6-9 and Figure 6-10 for concept maps of the probable layout of developer-funded distribution networks in the new developments. See Table 6-16 for a summary of estimated footages and sizes of new distribution networks in the new developments.

Table 6-16: Summary of Estimated Developer Distribution Improvements

Summary of Estimated Developer-Funded Distribution Improvements						
Development	**8” (LF)**	**10” (LF)**	**12” (LF)**	**14” (LF)**	**Hydrants**	**Pressure Zone**
Village A	7,750	0	4,300	0	19	ABC (new)
Village B	9,000	1,700	450	0	20	ABC (new)
Village C	4,375	0	450	910	7	ABC (new)
Residences	10,800	2,950	0	0	22	Rio Oso
Riverview	6,100	0	0	0	8	Van Vleck
Rancho North	9,450	3,900	8,050	0	41	Rio Oso
Total	**39,725**	**8,550**	**8,950**	**910**	**117**	**n/a**

VILLAGES A, B, AND C

THE RESIDENCES EAST AND WEST AT RANCHO MURIETA

RIVERVIEW

PROPOSED IMPROVEMENTS

VILLAGE A:

INSTALL APPROXIMATELY 7,750 LF OF 8" C900 PVC AND APPROXIMATELY 4,300 LF OF 12" C900 PVC.

INSTALL 19 FIRE HYDRANT ASSEMBLIES.

VILLAGE B:

INSTALL APPROXIMATELY 9,000 LF OF 8" C900 PVC, 1,700 LF OF 10" C900 PVC AND 450 LF OF 12" C900 PVC.

INSTALL 20 FIRE HYDRANT ASSEMBLIES.

VILLAGE C:

INSTALL APPROXIMATELY 4,375 LF OF 8" C900 PVC, 450 LF OF 12" C900 PVC, AND 910 LF OF 14" C900 PVC.

INSTALL 7 FIRE HYDRANT ASSEMBLIES.

THE RESIDENCES (EAST & WEST):

INSTALL APPROXIMATELY 10,800 LF OF 8" C900 PVC AND APPROXIMATELY 2,950 LF OF 10" C900 PVC.

INSTALL 22 FIRE HYDRANT ASSEMBLIES.

INSTALL 1 PRESSURE RELIEF VALVE ON NORTHEAST MAINLINE FROM RIO OSO PRESSURE ZONE.

RIVERVIEW:

INSTALL APPROXIMATELY 6,100 LF 8" C900 PVC.

INSTALL 8 FIRE HYDRANT ASSEMBLIES.

SITE PLAN NOTES

1. NO TOPOGRAPHIC SURVEY WAS PERFORMED. ALL LOCATIONS OF UTILITIES SHOWN ARE APPROXIMATE. ACTUAL SITE CONDITIONS AND LOCATIONS MAY VARY.
2. THIS IS A SCHEMATIC SITE PLAN FOR THE SOLE PURPOSE OF ILLUSTRATING THE GENERAL COMPONENTS AND FACILITIES. ACTUAL SITE LAYOUT AND LOCATIONS MAY VARY AND WILL NEED TO BE DETERMINED BY TOPOGRAPHIC SURVEY AND OTHER SITE INVESTIGATIONS AS REQUIRED.

LEGEND

EXISTING WATER LINE

EXISTING FIRE HYDRANT

PROPOSED PROPERTY LINE/LOT

PROPOSED WATER LINE

PROPOSED FIRE HYDRANT

PROPOSED WATER VALVE

PROPOSED PRESSURE RELIEF VALVE

300 0 150 300
(FEET)

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DISTRIBUTION AND STORAGE ALTERNATIVES
FOR
RANCHO MURIETA CSD
VILLAGES A, B, & C, RESIDENCES, RIVERVIEW

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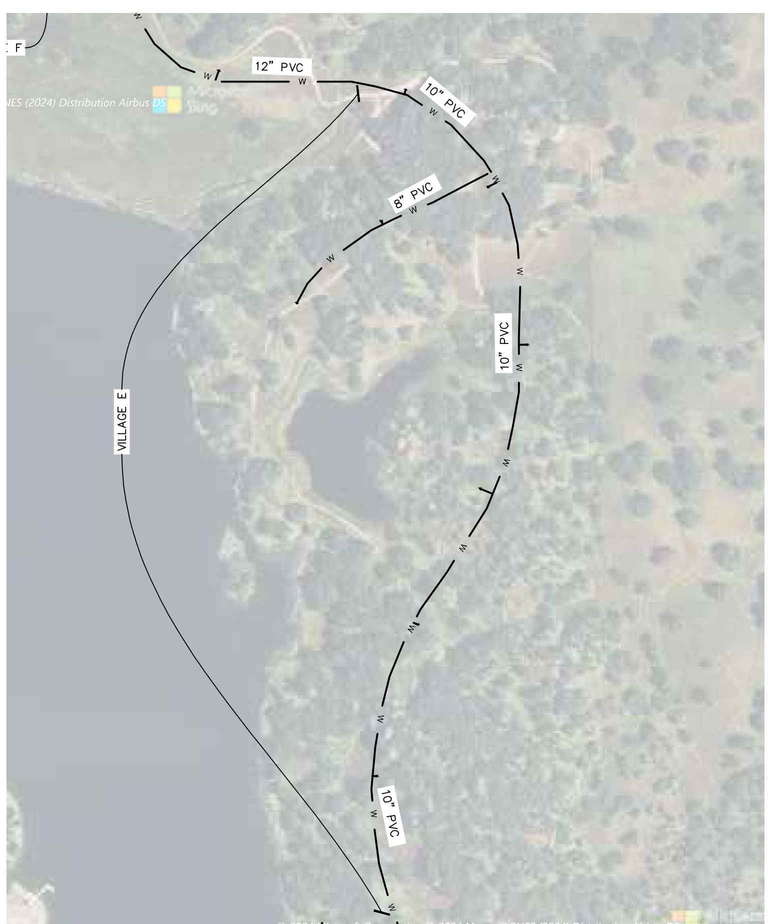
FIGURE
6-9

VILLAGES F, G, AND H



VILLAGE E

VILLAGE D

300 0 150 300
(FEET)

SITE PLAN NOTES

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LEGEND

EXISTING WATER LINE
EXISTING FIRE HYDRANT
PROPOSED PROPERTY LINE/LOT
PROPOSED WATER LINE
PROPOSED FIRE HYDRANT
PROPOSED WATER VALVE
PROPOSED PRESSURE RELIEF VALVE

PROPOSED IMPROVEMENTS

VILLAGE D:

INSTALL APPROXIMATELY 969 LF OF 8" C900 PVC AND APPROXIMATELY 812 LF OF 10" C900 PVC.

INSTALL 4 FIRE HYDRANT ASSEMBLIES.

VILLAGE E:

INSTALL APPROXIMATELY 748 LF OF 8" C900 PVC AND APPROXIMATELY 2,945 LF OF 10" C900 PVC.

INSTALL 8 FIRE HYDRANT ASSEMBLIES.

VILLAGE F:

INSTALL APPROXIMATELY 2,929 LF OF 8" C900 PVC AND APPROXIMATELY 2,505 LF OF 12" C900 PVC.

INSTALL 10 FIRE HYDRANT ASSEMBLIES.

VILLAGE G:

INSTALL APPROXIMATELY 652 LF OF 8" C900 PVC, AND APPROXIMATELY 1,529 LF OF 12" C900 PVC.

INSTALL 5 FIRE HYDRANT ASSEMBLIES.

INSTALL 1 PRESSURE RELIEF VALVE ON MAINLINE BETWEEN VILLAGE G AND VILLAGE H.

VILLAGE H:

INSTALL APPROXIMATELY 4,137 LF OF 8" C900 PVC, APPROXIMATELY 154 LF OF 10" C900 PVC, AND APPROXIMATELY 4,129 LF 12" C900 PVC.

INSTALL 14 FIRE HYDRANT ASSEMBLIES.

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DISTRIBUTION AND STORAGE ALTERNATIVES
FOR
RANCHO MURIETA CSD
RANCHO NORTH

SCALE	DATE	DRAWN BY	PROJ. NO.
SHOWN	08/08/2024	BAG	3971-02

FIGURE
6-10

6-5.

6-5. Reclaimed Water System Improvements

6-5.1. WWRP Improvements

6-5.1.a. EQ Basin Potable Water Air Gap Connection

The dual plumbing installed for residential and commercial use of reclaimed water will likely mean that the reclaimed water users only have irrigation systems connected to the reclaimed system, so it is important that the NCPS can meet irrigation demands even when WWRP production is not sufficient. The connection to the potable water system at the EQ basin will make this possible. This improvement is required to supplement reclaimed water with potable water and meet peak reclaimed water demands while maximizing the use of reclaimed water. This improvement involves connecting to the existing 8" potable water pipeline located immediately north of the EQ basin at the WWRP, installing an 8" extension to the EQ basin, and installing an 8" air gap connection to deliver potable water to the EQ basin. The connection between the existing potable water pipeline and the air gap will require approximately 20 feet of 8" C900 PVC pipe, a flow meter, isolation and control valves, and elbows. Based on the buildout domestic model, the existing 8" potable water pipe can deliver 0.8 MGD to the EQ basin while maintaining 40 psi of residual pressure in the rest of the pressure zone during peak day demand. This flow will allow the EQ basin and the NCPS to provide sufficient flows to the residential and commercial reclaimed water users throughout the irrigation season.

6-5.1.b. Disinfection Facilities Upgrade

As mentioned in a previous chapter, the WWRP is currently limited in its capacity by the disinfection system, which has a capacity of 2.3 MGD. It is proposed that the existing CCP be removed, and an additional chlorine contact chamber be added to increase the disinfection facility's capacity to 3.0 MGD.

As described in *WWRP Modified Chlorine Contact Disinfection System Compliance Report* (HSe, July 2006), the chlorine contact basin (CCB) was tested in 2003 for actual modal contact time at flows of 1 and 3 MGD. The estimated modal contact time through the CCB at 3 MGD is 27 minutes. In accordance with Title 22, disinfected tertiary reclaimed water requires a minimum

90-minute modal contact time. Therefore, the proposed chlorine contact chamber is to have minimum modal contact time of 63 minutes.

A new concrete chlorine contact chamber next to the existing EQ basin at the WWRP is currently in the design phase. This will increase disinfection capacity. The water surface elevation of the new chlorine contact chamber will approximately match the elevation of the existing chlorine contact basin. The water surface elevation immediately downstream of the new chlorine contact chamber will approximately match the elevation of the existing EQ basin.

This improvement also includes the removal and disposal of the existing 20” CCP located inside the EQ basin.

6-5.1.c. Dechlorination System

The WDR for the WWRP requires at least 4.5 mg/L of chlorine residual at the discharge point of the reclaimed system. However, due to seasonal challenges with high temperatures and other variables, District staff often maintain chlorine residuals of 6-10 mg/L. These levels of chlorine are toxic to landscaping, which require water with less than 2 mg/L of chlorine. Currently, water from the WWRP is pumped to the golf course irrigation lakes before it is applied to the golf courses. The time that the water spends in the lakes allows the chlorine residual to dissipate and avoid damaging the landscaping. However, for the residential and commercial users, it is proposed that the reclaimed water be pumped directly from the EQ basin to the users. For this reason, a dechlorination stage is proposed to reduce the chlorine residuals to a safe level for irrigation. This improvement will involve a building, approximately 8 feet by 8 feet, adjacent to the NCPS, which will store the sodium bisulfate used for dechlorination and the feed pump. The pump will feed sodium bisulfate into the stream exiting the rehabilitated NCPS.

6-5.1.d. DAF Pump Improvements

The 2017 PDR mentions the need for improvements to the third DAF feed pump. This improvement should be completed along with the other recommendations in this chapter. See Table 6-17 below for a cost summary of the WWRP improvement alternatives.

Table 6-17: Alternative 15 Cost Summary

Alternative 15 – WWRP Improvement Cost Summary	
Project Component	**Estimated Cost**
Mobilization/Demobilization, Traffic Controls:	$14,100
EQ Basin Air Gap	$57,500
New Chlorine Contact Basin (project in progress)	n/a
Dechlorination Building	$45,300
DAF Pump Improvements	$128,000
Subtotal:	$245,000
Construction Contingencies:	$49,000
Design, Engineering, Construction Admin:	$62,000
Environmental, Permitting, Legal, Land Acquisition:	$20,000
Total Estimated Project Cost:	$376,000

6-5.2. Reclaimed Transmission Improvements

6-5.2.a. North Course Pump Station Upgrades

For buildout demands, the NCPS will need to be able to deliver 2,690 gpm of reclaimed water during peak demand. To achieve this need, it is proposed that three vertical turbine pumps be installed to replace the existing pumps (two duty, one standby). Each of these pumps will provide 1,500 gpm of flow at 300 feet TDH. This provides a firm capacity of 3,000 gpm, which is greater than the flow estimated during peak day demand. Each of these pumps will also have VFDs installed to allow them to operate efficiently through a wide range of demands. As with the existing NCPS, the rehabilitated NCPS will be able to deliver water to either the North Course and Residential/Commercial users, or to Van Vleck ranch, depending on the needs at the time.

6-5.2.b. North Course Transmission Pipeline

As discussed in Chapter 5, portions of the existing pipeline from the NCPS to Bass Lake are undersized for buildout demands. Further, the entire pipeline is aging ACP, which has a low maximum operating pressure. For these reasons, it is proposed that the entire pipeline from the

NCPS to Bass Lake be replaced with 12” C900 PVC. The existing pipeline can be abandoned in place, with the new pipeline alongside it, if that is the more affordable option. See Table 6-18 below for a cost summary of the reclaimed transmission alternatives.

Table 6-18: Alternative 16 Cost Summary

Alternative 16 – Reclaimed Transmission Improvement Cost Summary	
Project Component	**Estimated Cost**
Mobilization/Demobilization, Traffic Controls:	$276,800
North Course Transmission Replacement	$2,668,000
North Course Pump Station Rehabilitation	$862,700
Subtotal:	$3,808,000
Construction Contingencies:	$762,000
Design, Engineering, Construction Admin:	$952,000
Environmental, Permitting, Legal, Land Acquisition:	$25,000
Total Estimated Project Cost:	$5,547,000

6-5.3. Reclaimed Distribution Systems for New Developments

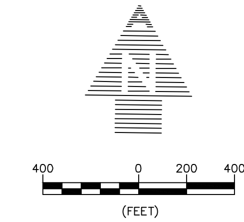
As with the domestic distribution systems for the new developments, it was required that the networks be modeled to ensure that other improvements would be sufficient to provide adequate service at buildout. These reclaimed distribution improvements will be funded by the developers. See Table 6-19 for a summary of the estimated footage of different reclaimed distribution pipelines that will be installed by developers.

Table 6-19: Summary of Estimated Developer Reclaimed Distribution Improvements

Summary of Estimated Developer-Funded Reclaimed Distribution Improvements		
Development	**6” (LF)**	**8” (LF)**
Village A	2,800	9,150
Village B	5,800	4,700
Village C	5,300	1,600
Retreats	5,200	0
Total	**19,100**	**15,450**

See Figure 6-11 for a map of all proposed reclaimed system improvements.

MAP OF PROPOSED IMPROVEMENTS



PROPOSED IMPROVEMENTS

1. INSTALL 65 LF OF 8" PVC FROM POTABLE WATER SYSTEM TO EQ BASIN WITH AIR GAP CONNECTION.
2. INSTALL NEW CHLORINE CONTACT BASIN.
3. INSTALL DECHLORINATION BUILDING AND FEED.
4. REPLACE DAF FEED PUMP
5. REPLACE (2) EXISTING PUMPS WITH (3) PUMPS WITH VFDS PUMPS TO PROVIDE 1,500 GPM AT 300 FEET TDH
6. REPLACE EXISTING 12" ACP WITH APPROXIMATELY 4,824 LF OF 12" PVC.
7. REPLACE EXISTING 8" ACP WITH APPROXIMATELY 7,288 LF OF 12" PVC.
8. CONNECT RETREATS DISTRIBUTION SYSTEM TO NORTH COURSE MAIN WITH 353 LF OF 8" C900 PVC.
9. CONNECT VILLAGE B DISTRIBUTION SYSTEM TO NORTH COURSE MAIN WITH 10 LF OF 8" C900 PVC.
10. CONNECT VILLAGE C DISTRIBUTION SYSTEM TO NORTH COURSE MAIN WITH 613 LF OF 8" C900 PVC.
11. REMOVE EXISTING CHLORINE CONTACT PIPE.

LEGEND

	EXISTING BUILDING EDGE
	EXISTING RECLAIMED WATER LINE
	PROPOSED BUILDING EDGE
	PROPOSED RECLAIMED WATER LINE
P	PROPOSED RECLAIMED WATER PUMP
	PROPOSED RECLAIMED WATER VALVE

SITE PLAN NOTES

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RECLAIMED WATER ALTERNATIVES
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ALL PROPOSED RECLAIMED IMPROVEMENTS

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FIGURE
6-11

6-6. Capital Improvement Plan

See Table 6-20 below for a recommended CIP for the District. This table summarizes all the alternatives outlined in this chapter. It includes the total estimated cost for each alternative and indicates if the project corrects a deficiency in existing or buildout conditions. The actual selection of projects and their timelines and funding are up to the discretion of the District. This CIP only represents the alternatives that resulted from the analysis performed to support this IWMP.

Table 6-20 Capital Improvement Plan

Capital Improvement Plan			
#	Description	**Existing/Buildout**	**Estimated Cost**
1A	3 New Wells, No Treatment	Existing	$6,349,000
1B	5 New Wells , No Treatment	Buildout	$10,455,000
2A	3 New Wells, Partial Treatment	Existing	$12,533,000
2B	5 New Wells, Partial Treatment	Buildout	$21,284,000
3A	3 New Wells, Portable Treatment	Existing	$6,349,000
3B	5 New Wells, Portable Treatment	Buildout	$10,455,000
4A	3 New Wells, Full Treatment	Existing	$17,184,000
4B	5 New Wells, Full Treatment	Buildout	$29,579,000
5A	3 New Wells, Treat at 3 New Wells WTP	Existing	$11,987,000
5B	5 New Wells, Treat at Existing WTP	Buildout	$16,855,000
6	Use Clementia for Domestic Storage	Buildout	n/a[1]
7	New Tank in Village C	Buildout	$3,272,000
8	New Tank in Village H	Buildout	$3,438,000
9	New Tank at Van Vleck	Buildout	$4,254,000
10	Village C Booster Station	Buildout	$1,678,000
11	New Hydrants and Pipeline Upsizing	Existing	$8,397,000
12	WWRP Improvements	Existing	$376,000
13	Reclaimed Transmission Improvements	Buildout	$5,547,000
[1]Since the cost for this alternative is primarily for pump rental, the capital cost is not comparable and is not included in this table.			

CHAPTER 7. Conclusion and Recommendations

This report summarizes the analysis of the District's potable and reclaimed water systems. It evaluated the existing facilities, performance objectives, existing and future demands, and system adequacy. Finally, alternatives were recommended and cost estimates presented for improvements that will help the District select appropriate projects. The alternatives presented in this IWMP were developed to meet the system's performance objectives based on the guidance that was provided by the District. At this point, the District can review the alternatives and decide how to proceed by selecting one or more of the alternatives presented in the report. This final section summarizes the recommendations for the District going forward from this IWMP.

1. Conduct a seepage study. One of the limitations of the domestic water balance conducted as a part of this IWMP was the lack of real seepage data. Historic empirical equations were used to estimate seepage for this water balance, which allows for uncertainty that could be corrected with data from a real seepage study. The District should retain a licensed geotechnical engineer to perform a seepage study for the three raw water storage reservoirs.

2. Install new weather station near the raw water storage reservoirs. Collecting accurate precipitation, evaporation, and temperature data is essential for the District to continue planning its water resources properly.

3. Update water balance. After the seepage study is complete, the data should be used to update the domestic water balance. The evaporation data gathered from the new weather station can be used to conduct an accurate seepage study and ultimately for the District to update its water balance.

4. IWMP Update. At such a time as the assumptions used in this IWMP are out of date, i.e. the planned developments change, data from the new weather station is available, water usage trends change significantly, the District should update the document to ensure it continues to be a useful planning tool for its water infrastructure. The hydraulic models should also be updated in accordance with these changes.