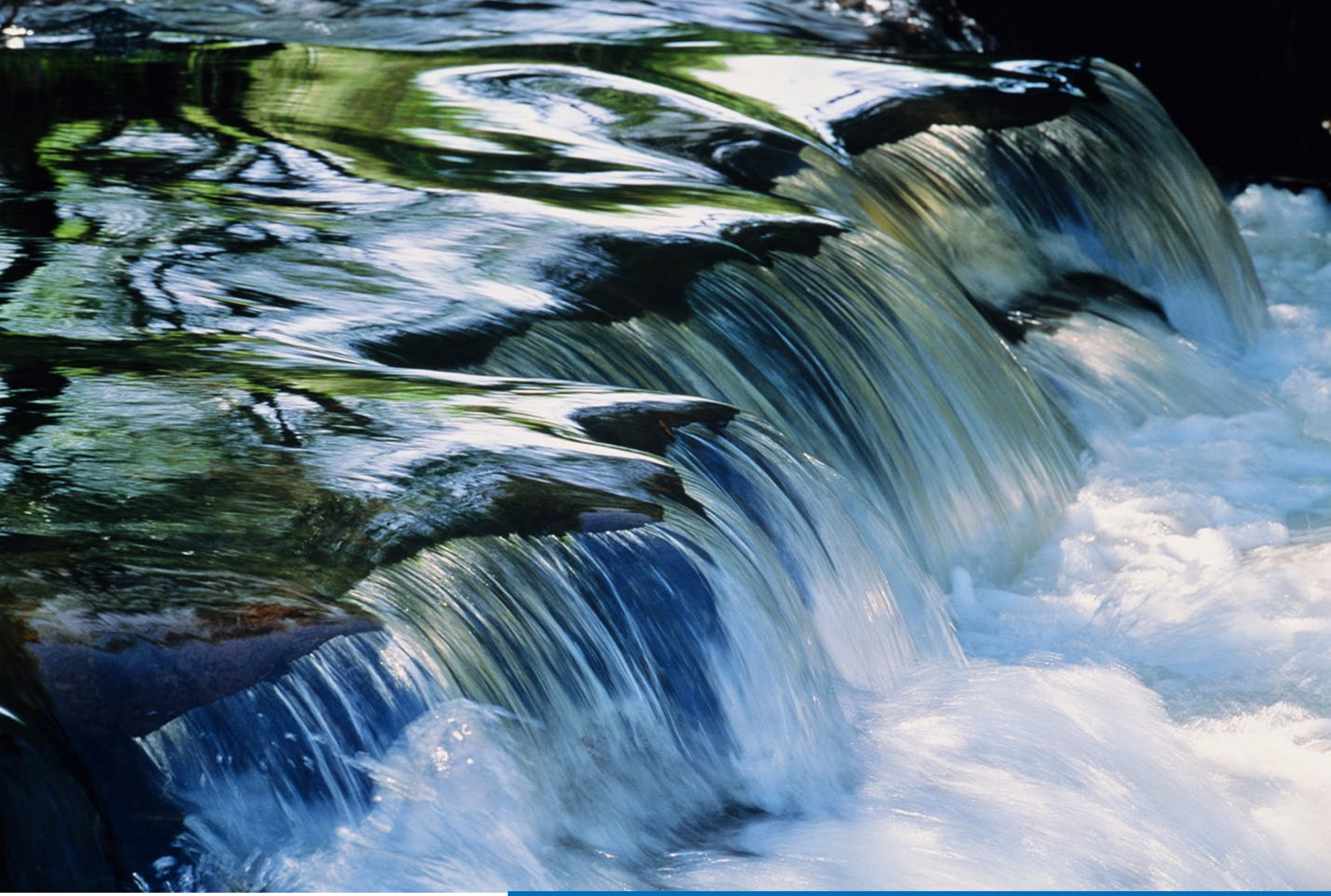

Alpine Springs County Water District

WATER AND WASTEWATER MASTER PLAN

FINAL | August 2023

ALPINE SPRINGS
COUNTY WATER DISTRICT

Alpine Springs County Water District

WATER AND WASTEWATER MASTER PLAN

FINAL | August 2023

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Date: 2023.08.11 13:05:49-07'00'

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Abbreviations

AACE	Association for the Advancement of Cost Engineering
ACP	asbestos cement pipe
ADD	average day demand
ADWF	average dry weather flow
AMEW	Alpine Meadows Estates Well
ASCWD	Alpine Springs County Water District
AWWA	American Water Works Association
BWF	base wastewater flow
CAP	condition assessment protocol
Carollo	Carollo Engineers, Inc.
CCTV	closed-circuit television
CDPH	California Department of Public Health
CFC	California Fire Code
CIP	capital improvement program
CIPP	cured-in-place pipe
CMMS	computerized maintenance management system
d/D	depth of flow to the diameter of the gravity sewer main
DIP	ductile iron pipe
DWF	dry weather flow
EPR	external point repair
EPS	extended period simulation
FCV	flow control valve
fps	feet per second
ft	feet
GIS	geographical information system
GM	gravity main
gpm	gallons per minute
GST	ground storage tank
GW	groundwater
GWI	groundwater infiltration
HGL	hydraulic grade line
HOF	high occupancy flow
hp	horsepower
ID	identification
I/I	infiltration and inflow
KPI	key performance indicator

M	miscellaneous
M-32	Manual on Distribution Network Analysis of Water Utilities
Master Plan	Alpine Springs County Water District Water and Wastewater Master Plan
MDD	maximum day demand
MG	million gallons
mgd	million gallons per day
MOL	maximum operating level
NASSCO	National Association of Sewer Service Companies
NTFPD	North Tahoe Fire Protection District
O&M	operations and maintenance
PACP	Pipeline Assessment and Certification Program
PHD	peak hour demand
POC	point of connection
PP	polypropylene
PRV	pressure reducing valve
PS	pump station
PSV	pressure sustaining valve
psi	pounds per square inch
PVC	polyvinyl chloride
PWL	potable water lateral
PWWF	peak wet weather flow
R&R	rehabilitation and replacement
RDI/I	rain-derived infiltration and inflow
ROW	right-of-way
RR	rehabilitation or replacement
S	storage
SCADA	supervisory control and data acquisition
SFR	single family residential
TPR	trenchless point repair
TRI	Truckee River Interceptor
TRPA	Tahoe Regional Planning Agency
T-TSA	Tahoe-Truckee Sanitation Agency
UFW	unaccounted-for-water
USFS	United States Forest Service
USUG	United States Geological Survey
VCP	vitrified clay pipe
VFD	variable frequency drive
WaPUG	Wastewater Planning Users Group

WM	water main
WRF	water reclamation facility
WWF	wet weather flow
WWL	wastewater lateral

EXECUTIVE SUMMARY

Alpine Springs County Water District's (ASCWD's) Water and Wastewater Master Plan (Master Plan) identifies water and wastewater system capital improvement needs through the 2045 planning horizon. The following sections summarize the Master Plan findings.

ES.1 Introduction

ASCWD provides sewer collection, water distribution, garbage collection, and parks and recreation services to the Alpine Meadows community, which is located in Placer County approximately 4 miles west of Lake Tahoe, California. The water and wastewater systems were originally developed starting in the early 1960s to provide centralized sewer and water services to Alpine Meadows residents.

This Master Plan documents ASCWD's existing water distribution and wastewater collection systems and develops a planning framework for operations and maintenance as well as development through the 2045 planning horizon. Findings from this Master Plan will help inform capital improvement program (CIP) development and other planning efforts.

ES.2 Existing and Projected Water Demands and Wastewater Flows

Water demand and wastewater flow projections were developed to evaluate the water and wastewater systems under existing and future conditions.

ES.2.1 Existing and Projected Water Demands

ASCWD's existing water demands were estimated using historical production and consumption data. The existing average day demand (ADD), which is the total annual production divided by the number of days per year, was estimated to equal 0.086 million gallons per day (mgd). The maximum day demand (MDD) is used to evaluate water system capacity and was estimated using an MDD to ADD peaking factor of 3.45. Applying this peaking factor to the existing ADD, the existing MDD equals 0.297 mgd.

Water demand projections were developed for the 2045 planning horizon using planned development data and assumed annual growth rate of 0.34 percent per year. The following two planned developments were incorporated into the projections:

- Alpenglow: This project is expected to start in year 2025 and to be completely built out by 2040. A total of 52 single family residential (SFR) units are expected, and roughly 3 units per year are assumed to be connected. Although the final development may involve fewer total units, this Master Plan is based on the units in the latest documents for this development.
- White Wolf: This project includes a total of 58 SFR units. This subdivision is assumed to begin connecting homes in 2035 and to be built out by 2040, which equates to roughly 10 added SFRs per year. Although the final development may involve fewer

total units, this Master Plan is based on the units in the latest documents for this development.

Added ADD from expected annual growth along with the planned developments was estimated to equal 0.040 mgd through the planning horizon, leading to a 2045 ADD of 0.126 mgd. The projected 2045 MDD was calculated to equal 0.434 mgd.

Table ES.1 and Figure ES.1 summarize the existing and projected water demands.

Table ES.1 Existing and Projected Water Demands

Water Demand Metric	Existing Demand (mgd)	2045 Demand (mgd)
ADD	0.086	0.126
MDD	0.297	0.434

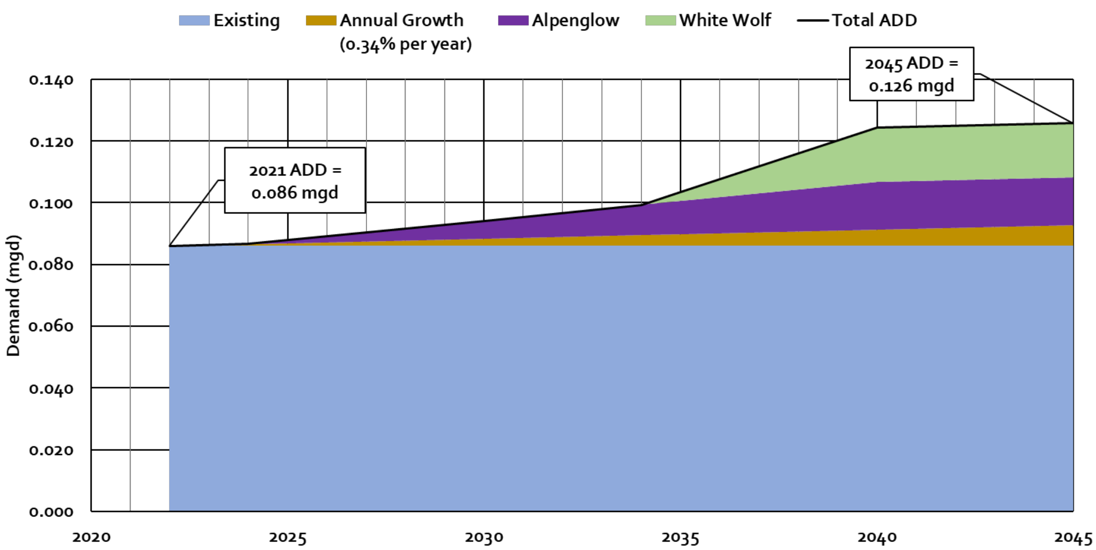


Figure ES.1 Existing and Projected Water Demands

ES.2.2 Existing and Projected Wastewater Flows

The existing wastewater flows were estimated using historical flow monitoring data from a flow meter that records ASCWD's discharges into the Tahoe-Truckee Sanitation Agency's (T-TSA) Truckee River Interceptor (TRI). Using TRI data from 2014 to 2022, ASCWD's existing average dry weather flow (ADWF), or the average daily flow between July and September, was estimated to equal 0.051 mgd. The system's existing high occupancy flow (HOF) was estimated to equal 0.123 mgd.

ASCWD's PWWF was determined by routing the 10-year, 24-hour design storm through the hydraulic model, which is discussed in Chapter 3, Existing Water and Wastewater Systems and Hydraulic Model Development, in addition to the HOF. Using this method, ASCWD's existing PWWF was estimated to be 0.541 mgd.

Wastewater flow projections were developed using a methodology consistent with the water demand projections. The added ADWF from annual growth and planned developments through the planning horizon is expected to equal 0.024 mgd, which results in a 2045 ADWF of 0.075 mgd. The 2045 HOF and PWWF were estimated to equal 0.180 mgd and 0.694 mgd, respectively.

Table ES.2 and Figure ES.2 summarize the existing and projected wastewater flows.

Table ES.2 Existing and Projected Wastewater Flows

Wastewater Flow Metric	Existing Flow (mgd)	2045 Flow (mgd)
ADWF	0.051	0.075
HOF	0.123	0.180
PWWF	0.541	0.694

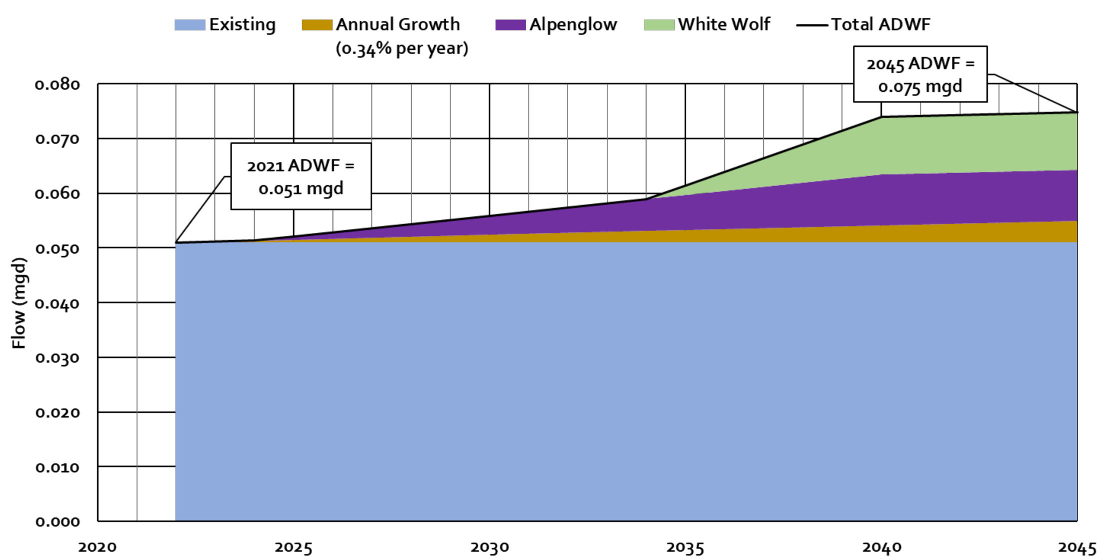


Figure ES.2 Existing and Projected Wastewater Flows

ES.3 Existing Water and Wastewater Systems

This section summarizes the existing water and wastewater systems.

ES.3.1 Existing Water System

ASCWD's existing water system ranges in elevation from approximately 6,530 feet to 6,920 feet above sea level. To maintain appropriate water pressures to customers throughout this elevation range, the system is divided into four main pressure zones, from Zone 1 at the top to Zone 4 at the bottom. Zone 3 is further divided into Zone 3 (main), Zone 3 Boosted, and Zone 3 Lower. Water is transmitted from higher to lower pressure zones via pressure regulating zones. Zone 3 Boosted, which consists of a residential neighborhood at the north end of Juniper Mountain Road, is supplied via a booster pump station.

The water system is primarily supplied by four springs that flow freely into the distribution system. A groundwater well called the Alpine Meadows Estates Well (AMEW) Number 1 was installed in 2015 but is rarely utilized due to concerns related to the well pump hydraulic parameters as well as sufficient capacity from the four springs. ASCWD also owns two additional groundwater wells at the bottom of the system that are not currently utilized to supply drinking water demands.

ASCWD has five water storage tanks that serve to equalize daily fluctuations between supply and demand, supply water for firefighting, and meet demands during emergencies such as unplanned supply source outages. Tank 4 was replaced in 2019 after the original tank failed; the other four tanks were installed in the early 1960s and have undergone only minor repairs since.

The water distribution system consists of approximately 14.5 miles of primarily asbestos cement pipeline ranging from 4 to 8 inches in diameter, with the vast majority (i.e., 84 percent) being 6 inches in diameter. Like the storage tanks, most of the distributions system pipelines were originally installed in the 1960s and have not been rehabilitated or replaced.

Figure ES.3 shows the existing water system.

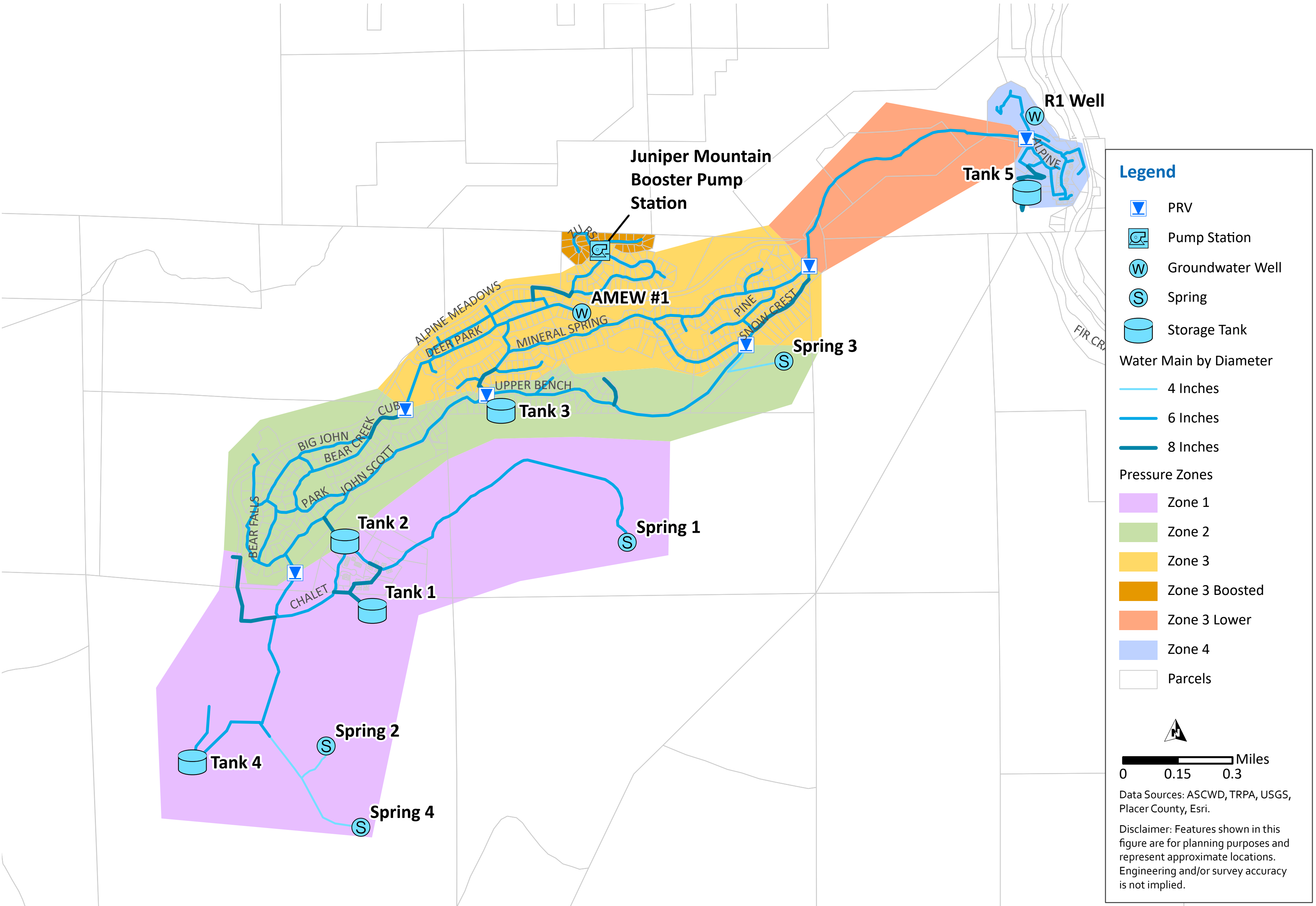


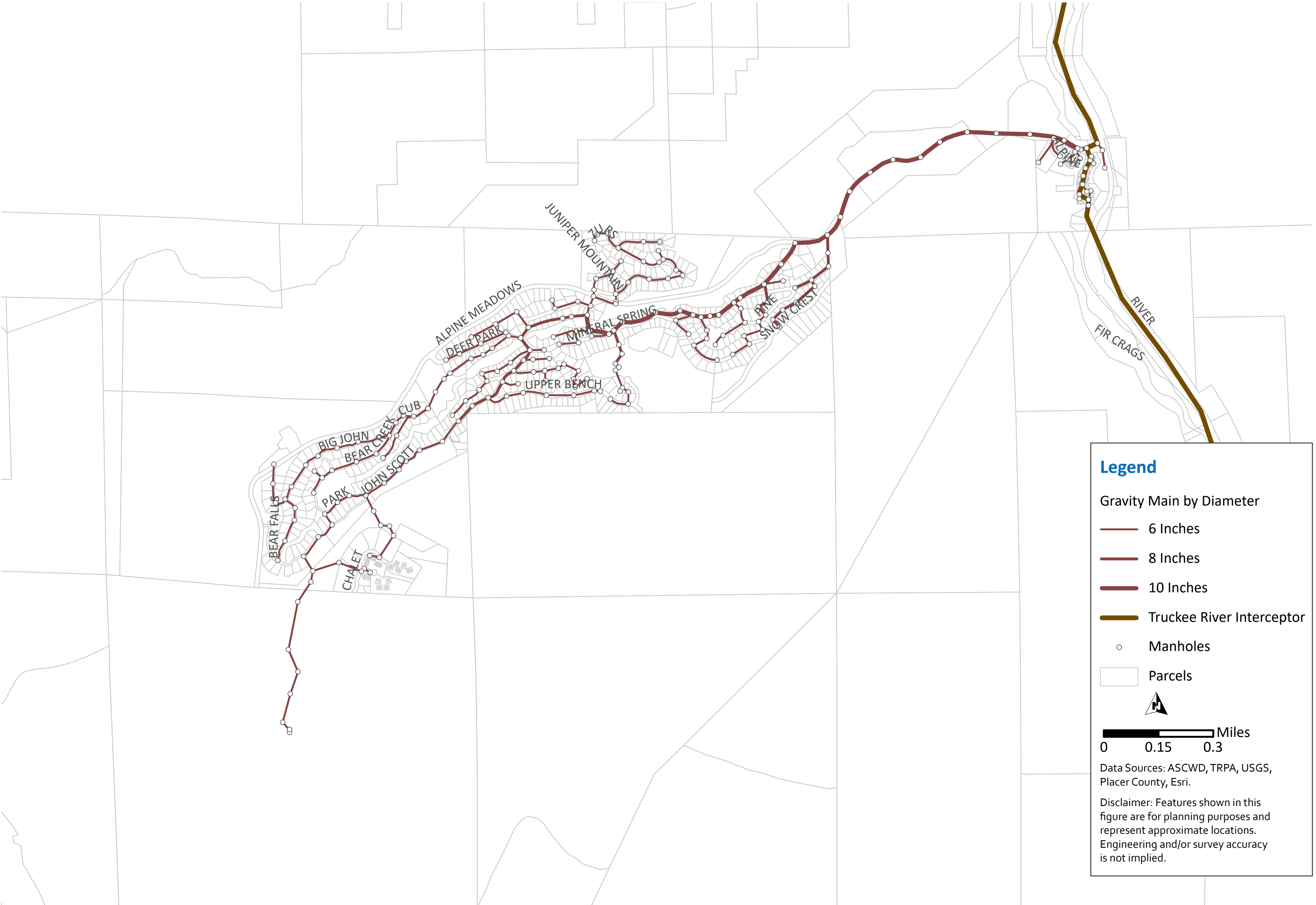
Figure ES.3 Existing Water System

ES.3.2 Existing Wastewater System

ASCWD's wastewater collection system consists of approximately 10.3 miles of gravity mains ranging in diameter from 6 to 10 inches and approximately 230 manholes. Like the water system, the wastewater collection system is constructed primarily of asbestos cement pipelines that were installed in the 1960s and have since undergone minor rehabilitation and replacement (R&R).

The collection system operates under 100 percent gravity flow and discharges into T-TSA's TRI near the intersection of Alpine Meadows Road and Highway 89. The TRI conveys ASCWD's wastewater flows, along with flows from other systems in the North Lake Tahoe area, to a regional water reclamation facility (WRF) in Martis Valley east of the Town of Truckee, California.

Figure ES.3 shows the existing wastewater system.


JUNIPER MOUNTAIN
ZURS
ALPINE MEADOWS
DEER PARK
MINERAL SPRING
PINE
SNOW CREST
UPPER BENCH
CUB
BIG JOHN
BEAR CREEK
JOHN SCOTT
PARK
BEAR FALLS
CHALET
ALPINE
RIVER
FIR CRAGS
Legend
Gravity Main by Diameter
6 Inches
8 Inches
10 Inches
Truckee River Interceptor
Manholes
Parcels
N
Miles
0
0.15
0.3
Data Sources: ASCWD, TRPA, USGS, Placer County, Esri.
Disclaimer: Features shown in this figure are for planning purposes and represent approximate locations. Engineering and/or survey accuracy is not implied.

ES.4 Water and Wastewater System Evaluations

Condition and hydraulic evaluations were conducted to identify water and wastewater system deficiencies and determine capital improvement needs. The following sections summarize the key findings from the condition and hydraulic assessments.

ES.4.1 Condition Assessment Findings

The water and wastewater system above- and below-grade assets were analyzed via field and desktop assessments to evaluate condition. The conditions assessments revealed the following main findings:

- The four original water storage tanks that were installed in the 1960s are all in poor condition and will require rehabilitation or replacement within the planning horizon.
- The water distribution and wastewater collection system pipelines are generally in good condition and have a relatively low frequency of major leaks and breaks. However, age and material data indicate that the pipelines may begin deteriorating more rapidly over the planning horizon as the assets approach their expected useful lifetimes.
- ASCWD's other assets are generally in good condition. The following minor deficiencies were identified:
  - The casing for Spring 1 leaks, which could potentially contribute to reductions in supply availability over time. Installation of groundwater monitoring wells near Spring 1 could help determine whether the leak is influencing groundwater levels.
  - The AMEW Number 1 does not have a backup generator, which could lead to supply deficits if a power outage occurs. This facility also has a pump with a suboptimal design point that could lead to advanced degradation of system appurtenances such as valves.
  - Both wells in operation (i.e., AMEW Number 1 and the R1 well) have only one duty pump and no standby pumps. This limits operational flexibility and increases the risk of supply deficits in the case that the duty pumps fail unexpectedly. Adding standby pumps at each facility will improve the system's robustness and reduce risks.
- The condition assessment revealed several data gaps that restrict ASCWD's ability to accurately evaluate the water and wastewater systems. The following activities would facilitate more accurate and complete analyses and enhance ASCWD's asset management program:
  - Update and maintain GIS asset data with specific attention to key attributes such as material, size, and installation date.
  - Develop and implement a formal Condition Assessment Protocol (CAP) to rate and record asset condition.
  - Procure a computerized maintenance management system (CMMS) or other formal tracking process to schedule and record work orders.
  - Establish key performance indicators (KPIs) and metrics to track operations and maintenance performance against District goals as well as industry standards.

ES.4.2 Water System Hydraulic Evaluation

The water system hydraulic evaluation consisted of supply and storage capacity analyses as well as distribution system hydraulic performance analyses. Table ES.3 summarizes the water system hydraulic evaluation findings.

According to the water system hydraulic evaluation, the system has supply, storage, and hydraulic performance deficiencies under existing demand conditions. The 2045 system evaluation found that existing deficits will increase marginally as demands increase over the planning horizon.

Table ES.3 Water System Hydraulic Evaluation Findings

Category	Findings
Supply capacity	• The water system has sufficient total supply capacity to meet existing and projected supply needs. However, the AMEW Number 1 lacks a standby pump to meet firm capacity requirements. Adding a standby pump at this facility would mitigate existing and projected supply deficiencies. • Historical production data indicates that ASCWD's supply sources are not being depleted over time. However, implementation of groundwater monitoring could help determine whether any actions should be taken to limit spring production and further conserve supplies.
Storage capacity	• The water system has sufficient total storage capacity to meet existing and projected 2045 operational, emergency, and fire reserve storage needs. • Although the system meets total storage capacity requirements, Zone 3 Boosted is considered storage deficient since its fire reserve storage must be delivered through the Juniper Mountain PS, which does not have adequate capacity to deliver the required fire flows of 1,500 gpm.
Distribution system performance	• The model analysis did not identify any water system hydraulic performance under typical MDD conditions. All minimum pressures at service conditions remain above the minimum pressure of 35 pounds per square inch (psi) under existing and projected 2045 MDD conditions. • The fire flow analysis found that several areas within the system are unable to achieve the minimum required fire flows under MDD conditions while maintaining the minimum 20 psi residual pressure.

ES.4.3 Wastewater System Hydraulic Evaluation

The wastewater collection system was evaluated under existing and projected PWWF conditions to identify hydraulic bottlenecks. This evaluation did not reveal any deficiencies.

Although the analysis did not identify any wastewater system hydraulic deficiencies, it was determined that some manholes at the bottom of the system near the TRI discharge have shallow manhole depths of five feet or less. It is recommended that these shallow manholes are sealed to help mitigate potential overflows if the TRI were to backup into ASCWD's collection system.

ES.5 Proposed Improvements

Improvements were developed to meet water and wastewater system needs through the 2045 planning horizon. The proposed improvements address both condition and hydraulic deficiencies.

Table ES.4 summarizes the proposed water and wastewater system improvements, and Figure ES.5 shows the proposed water system improvements.

Table ES.4 Proposed Improvements Summary

Project ID (1)	Project Name	Project Description
Water System Capacity Improvements		
PS-01	New Juniper Mountain PS	Install new booster pump station with a firm capacity of 70 gpm and a total dynamic head of 100 feet at Alpine Meadows Road and Juniper Mountain Road.(2)
S-01	New Tank 6	Install new Tank 6 above Juniper Mountain Road at an elevation of approximately 6,740 feet.(2)
GW-02	Alpine Meadows Estates Well Number 1 upgrades	Install standby pump and backup generator.
WM-01 through WM-29	Water main upsize projects	Upsize water mains to provide sufficient capacity for fire flows. See Section 8.1.3 for individual project details.
Water System Condition Improvements		
RR-S-01	Tank 2 rehabilitation or replacement	Rehabilitate or replace Tank 2.
RR-S-02	Tank 3 rehabilitation or replacement	Rehabilitate or replace Tank 3.
RR-S-03	Tank 5 rehabilitation or replacement	Rehabilitate or replace Tank 5.
RR-PWL-01	Ongoing water service lateral rehabilitation and replacement	Rehabilitate or replace about 8 water service laterals per year.
Wastewater System Condition Improvements		
RR-GM-01	Ongoing gravity main rehabilitation and replacement	Rehabilitate or replace approximately 540 linear feet of gravity main per year.
RR-WWL-01	Ongoing wastewater service lateral rehabilitation and replacement – upper laterals	Rehabilitate or replace about 8 upper wastewater service laterals per year.
RR-WWL-02	Ongoing wastewater service lateral rehabilitation and replacement – lower laterals	Rehabilitate or replace about 8 lower wastewater service laterals per year.
Miscellaneous Improvements		
M-01	Master Plan updates	Update water and wastewater master plan every 10 years.
M-02	SCADA updates	Update SCADA system to enable data extraction.

Notes:
(1) Project IDs use the following nomenclature: PS – pump station improvement; S – storage improvement; GW- groundwater well improvement; WM – water main improvement; PWL – potable water lateral improvement; GM -gravity main improvement; WWL – wastewater lateral improvement; RR – rehabilitation or replacement, M-miscellaneous.
(2) The new Juniper Mountain PS and Tank 6 specifications should be refined during more detailed planning stages for the proposed facilities. The pump station firm capacity should be sufficient to supply the adjusted Zone 3 Boosted peak hour demand.

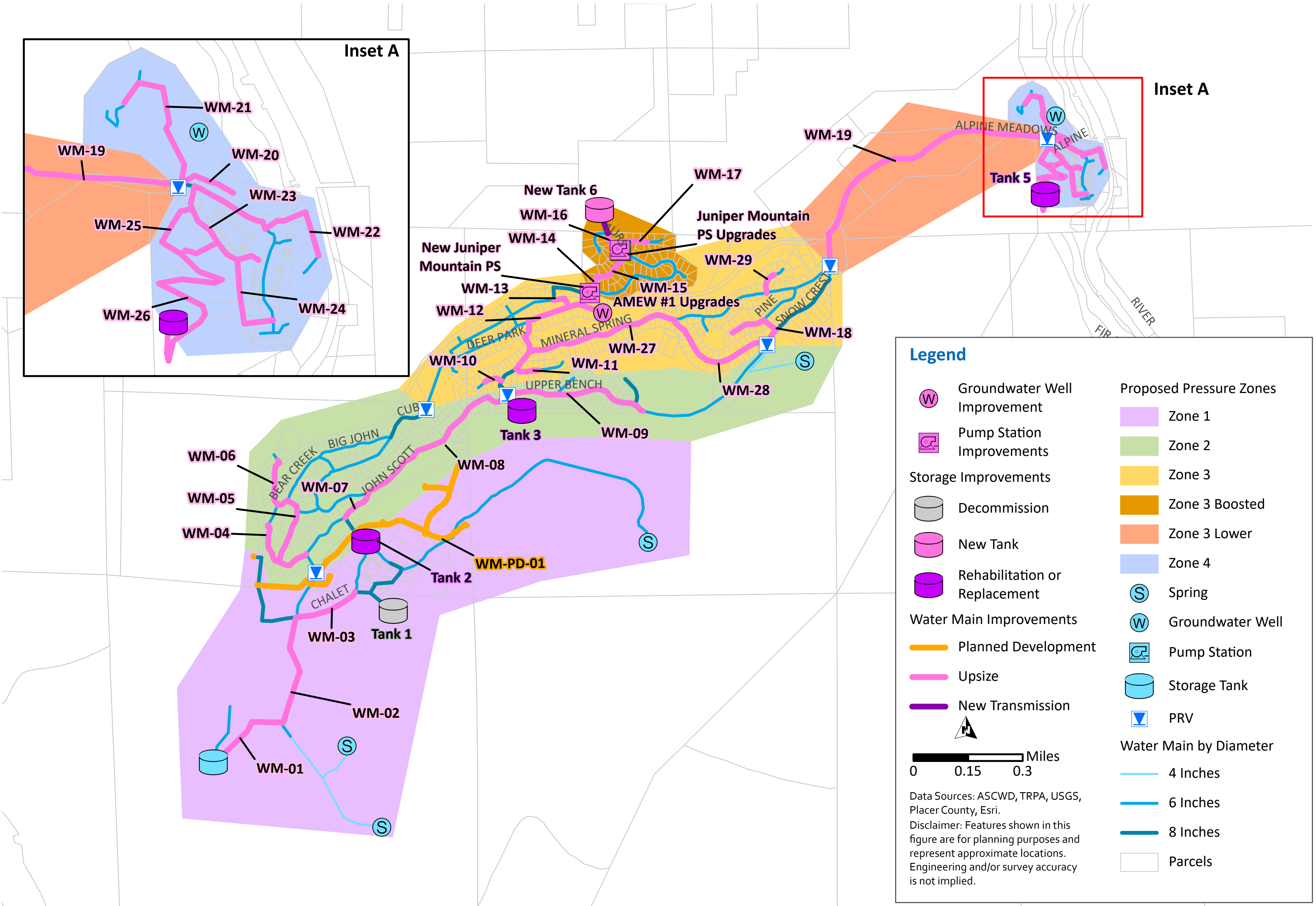


Figure ES.5 Proposed Water System Improvements

ES.6 Capital Improvement Plan

A CIP was developed to help ASCWD plan and budget for system improvements as well as for ongoing asset management over the planning horizon. Conceptual cost estimates, which are categorized as Class 5 estimates under the Association for the Advancement of Cost Engineering (AACE) guidelines and have an anticipated accuracy of +100 percent to -50 percent, were developed for each of the proposed improvements.

Table ES.5 shows the capital improvement project costs in 2022 dollars. The total CIP cost is estimated to equal $29.6 million.

A proposed capital improvement delivery plan was developed to assist ASCWD in implementing capital improvements throughout the planning horizon. The implementation plan reflects current priorities and is subject to change as a result of future assessments and available financing options. Project development, phasing, and implementation all depend on factors such as funding availability, community input, direction from the Board and Long-Range Planning Committee, and changing water and wastewater system conditions that may lead to reprioritization (e.g., system failures that require emergency repairs). Current financing mechanisms may limit ASCWD's ability to implement the improvements according to the outlined schedule; however, the plan can help the District determine how and when to budget for capital improvements.

The proposed implementation plan would enable ASCWD to execute the following high-priority projects within the first five years of the CIP at an estimated cost of $2.7 million:

- Rehabilitation of Tanks 2, 3, and 5 (i.e., RR-S-01, RR-S-02, and RR-S-03).
- Planning and design of Juniper Mountain water system improvements (i.e., WM-14, WM-15, WM-16, PS-01, and S-01).
- Ongoing water and wastewater R&R (i.e., RR-PWL-01, RR-GM-01, RR-WWL-01, and RR-WWL-02).
- AMEW Number 1 backup generator (i.e., GW-01).
- SCADA updates (i.e., M-02).

Figure ES.6 shows the annual cost distribution for the proposed CIP.

Table ES.5 Capital Improvement Project Costs

Project ID	Project Name	Proposed Amount	Unit	Unit Cost	Baseline Construction Cost (1)	Direct Construction Cost (1)(2)	Total Construction Cost (1)(3)	Total Capital Cost (1)(4)
Water System Capacity Improvements								
PS-01	New Juniper Mountain PS	2.2	hp	$7,000	$15,000	$20,000	$24,000	$27,000
S-01	New Tank 6	0.2	MG	$3.0	$600,000	$780,000	$944,000	$1,085,000
GW-02	Alpine Meadows Estates Well Number 1 upgrades	1	lump sum	$154,000	$154,000	$201,000	$229,000	$254,000
WM-01	Water main upsize from Tank 4 to Alpine Meadows Lodge	810	linear feet of 10-inch diameter water main	$410	$332,000	$432,000	$522,000	$601,000
WM-02	Water main upsize from Alpine Meadows Lodge to Chalet Road	2,070	linear feet of 10-inch diameter water main	$410	$849,000	$1,104,000	$1,335,000	$1,536,000
WM-03	Water main upsize along Chalet Road	960	linear feet of 10-inch diameter water main	$410	$394,000	$512,000	$620,000	$713,000
WM-04	Water main upsize along John Scott Trail by Bear Creek	1,090	linear feet of 8-inch diameter water main	$345	$376,000	$489,000	$591,000	$680,000
WM-05	Water main upsize along Bear Falls Lane	1,220	linear feet of 8-inch diameter water main	$345	$421,000	$547,000	$662,000	$762,000
WM-06	Water main upsize along Bear Creek Drive	680	linear feet of 10-inch diameter water main	$410	$279,000	$363,000	$439,000	$505,000
WM-07	Water main upsize along John Scott Trail west of Park Drive	480	linear feet of 10-inch diameter water main	$410	$197,000	$256,000	$310,000	$356,000
WM-08	Water main upsize along John Scott Trail east of Park Drive	2,290	linear feet of 10-inch diameter water main	$410	$939,000	$1,221,000	$1,477,000	$1,699,000
WM-09	Water main upsize along Upper Bench Road	2,470	linear feet of 10-inch diameter water main	$410	$1,013,000	$1,317,000	$1,593,000	$1,832,000
WM-10	Water main upsize along Trapper Place	260	linear feet of 8-inch diameter water main	$345	$90,000	$117,000	$142,000	$163,000
WM-11	Water main upsize along Trapper McNutt Trail	340	linear feet of 8-inch diameter water main	$345	$117,000	$152,000	$184,000	$212,000
WM-12	Water main upsize from Alpine Meadows Estates Well Number 1 to Trapper McNutt Trail	2,000	linear feet of 8-inch diameter water main	$345	$690,000	$897,000	$1,085,000	$1,248,000
WM-13	Water main upsize from Beaver Dam Trail to Deer Park Drive	290	linear feet of 8-inch diameter water main	$345	$100,000	$130,000	$157,000	$181,000
WM-14	Water main upsize from new Juniper Mountain PS to Kloster Court	240	linear feet of 8-inch diameter water main	$345	$83,000	$108,000	$131,000	$150,000
WM-15	Water main upsize along Kloster Court	570	linear feet of 8-inch diameter water main	$345	$197,000	$256,000	$310,000	$356,000
WM-16	Water main upsize along Juniper Mountain Road	410	linear feet of 8-inch diameter water main	$345	$141,000	$183,000	$222,000	$255,000

Table ES.5 Capital Improvement Project Costs (continued)

Project ID	Project Name	Proposed Amount	Unit	Unit Cost	Baseline Construction Cost (1)	Direct Construction Cost (1)(2)	Total Construction Cost (1)(3)	Total Capital Cost (1)(4)
Water System Capacity Improvements								
WM-17	Water main upsize along Cortina Court	480	linear feet of 8-inch diameter water main	$345	$166,000	$216,000	$261,000	$300,000
WM-18	Water main upsize from Snow Crest Road to Pine Trail	730	linear feet of 8-inch diameter water main	$345	$252,000	$328,000	$396,000	$456,000
WM-19	Water main upsize from R-4 to Alpine Circle Road	4,420	linear feet of 8-inch diameter water main	$345	$1,525,000	$1,983,000	$2,399,000	$2,759,000
WM-20	Water main upsize towards commercial center north of Alpine Meadows Road	350	linear feet of 8-inch diameter water main	$345	$121,000	$157,000	$190,000	$219,000
WM-21	Water main upsize towards recreational area north of Alpine Meadows Road	910	linear feet of 8-inch diameter water main	$345	$314,000	$408,000	$494,000	$568,000
WM-22	Water main upsize along Alpine Meadows Road and Highway 89 towards River Ranch	1,050	linear feet of 8-inch diameter water main	$345	$362,000	$471,000	$569,000	$655,000
WM-23	Water main upsize along Alpine Circle Road	700	linear feet of 8-inch diameter water main	$345	$242,000	$315,000	$381,000	$438,000
WM-24	Water main upsize from Alpine Circle Road towards condominium tennis court	560	linear feet of 8-inch diameter water main	$345	$193,000	$251,000	$304,000	$349,000
WM-25	Water main upsize west of Alpine Circle Road	670	linear feet of 8-inch diameter water main	$345	$231,000	$300,000	$363,000	$418,000
WM-26	Water main upsize towards Tank 5	1,640	linear feet of 10-inch diameter water main	$410	$672,000	$874,000	$1,057,000	$1,216,000
WM-27	Water main upsize along Mineral Springs Trail from John Scott Trail to west end of Snow Crest Road	2,240	linear feet of 12-inch diameter water main	$475	$1,064,000	$1,383,000	$1,674,000	$1,925,000
WM-28	Water main upsize along Snow Crest Road	1,930	linear feet of 10-inch diameter water main	$410	$791,000	$1,028,000	$1,244,000	$1,431,000
WM-29	Water main upsize along Mineral Springs Place	320	linear feet of 8-inch diameter water main	$345	$110,000	$143,000	$173,000	$199,000
			Water System Capacity Improvements Subtotal		$13,030,000	$16,942,000	$20,482,000	$23,548,000
Water System Condition Improvements								
RR-S-01	Tank 2 rehabilitation or replacement	0.1	MG	$2.5	$250,000	$325,000	$393,000	$452,000
RR-S-02	Tank 3 rehabilitation or replacement	0.1	MG	$2.5	$250,000	$325,000	$393,000	$452,000
RR-S-03	Tank 5 rehabilitation or replacement	0.1	MG	$2.5	$250,000	$325,000	$393,000	$452,000
RR-PWL-01	Ongoing water service lateral rehabilitation and replacement	150	each	$3,600	$540,000	$702,000	$849,000	$977,000
			Water System Condition Improvements Subtotal		$1,290,000	$1,677,000	$2,028,000	$2,333,000

Table ES.5 Capital Improvement Project Costs (continued)

Project ID	Project Name	Proposed Amount	Unit	Unit Cost	Baseline Construction Cost (1)	Direct Construction Cost (1)(2)	Total Construction Cost (1)(3)	Total Capital Cost (1)(4)
Wastewater System Condition Improvements								
RR-GM-01	Ongoing gravity main rehabilitation and replacement	10,800	linear feet	$130	$1,404,000	$1,825,000	$2,208,000	$2,540,000
RR-WWL-01	Ongoing wastewater service lateral rehabilitation and replacement – upper laterals	150	each	$2,400	$360,000	$468,000	$566,000	$651,000
RR-WWL-02	Ongoing wastewater service lateral rehabilitation and replacement – lower laterals	150	each	$1,200	$180,000	$234,000	$283,000	$326,000
				Wastewater System Condition Improvements Subtotal	$1,944,000	$2,527,000	$3,057,000	$3,517,000
Miscellaneous Projects								
M-01	Master Plan updates	2	Lump sum	$100,000	N/A	N/A	N/A	$200,000
M-02	SCADA updates	1	Lump sum	$5,000	N/A	N/A	N/A	$5,000
				Miscellaneous Improvements Subtotal	N/A	N/A	N/A	$205,000
				Total	**$16,264,000**	**$21,146,000**	**$25,567,000**	**$29,603,000**

Notes:
(1) All costs are in October 2022 dollars using the Engineering News Record 20-city average construction cost index of 13,175.
(2) The direct construction cost is equal to the baseline construction cost times 130 percent to account for estimating contingencies.
(3) The total construction cost is equal to the direct construction cost plus 15 percent to account for contractor general conditions and another 15 percent to account for contractor overhead and profits. These contingencies are not applied to all project costs.
(4) The total capital cost is equal to the total construction cost plus 30 percent to account for project delivery cost contingencies. These contingencies are not applied to all project costs.
(5) Abbreviations: hp = horsepower; MG = million gallons; SCADA = supervisory control and data acquisition.

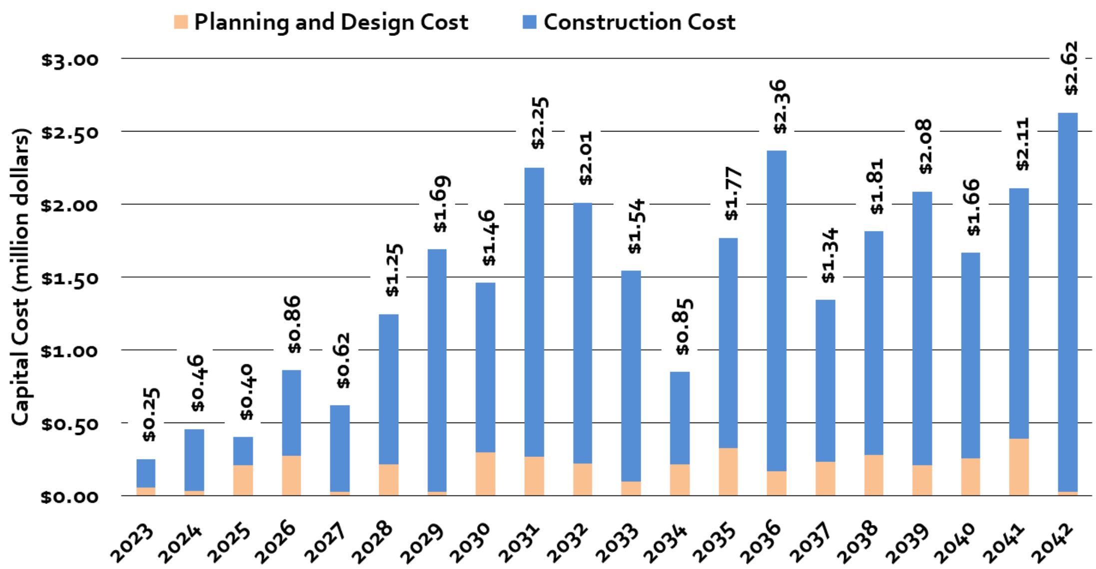


Figure ES.6 Proposed Capital Improvement Plan Annual Cost Distribution

Chapter 1

INTRODUCTION

1.1 Background

Alpine Springs County Water District (ASCWD) provides sewer collection, water distribution, garbage collection, and parks and recreation services to the Alpine Meadows community, which is located in Placer County approximately 4 miles west of Lake Tahoe, California. ASCWD was originally formed in the early 1960s to provide centralized sewer and water services to Alpine Meadows residents. This Water and Wastewater Master Plan (Master Plan) documents ASCWD's existing water distribution and wastewater collection systems and develops a planning framework for operations and maintenance as well as development through the 2045 planning horizon. Figure 1.1 shows the study area in relation to surrounding communities and landmarks.

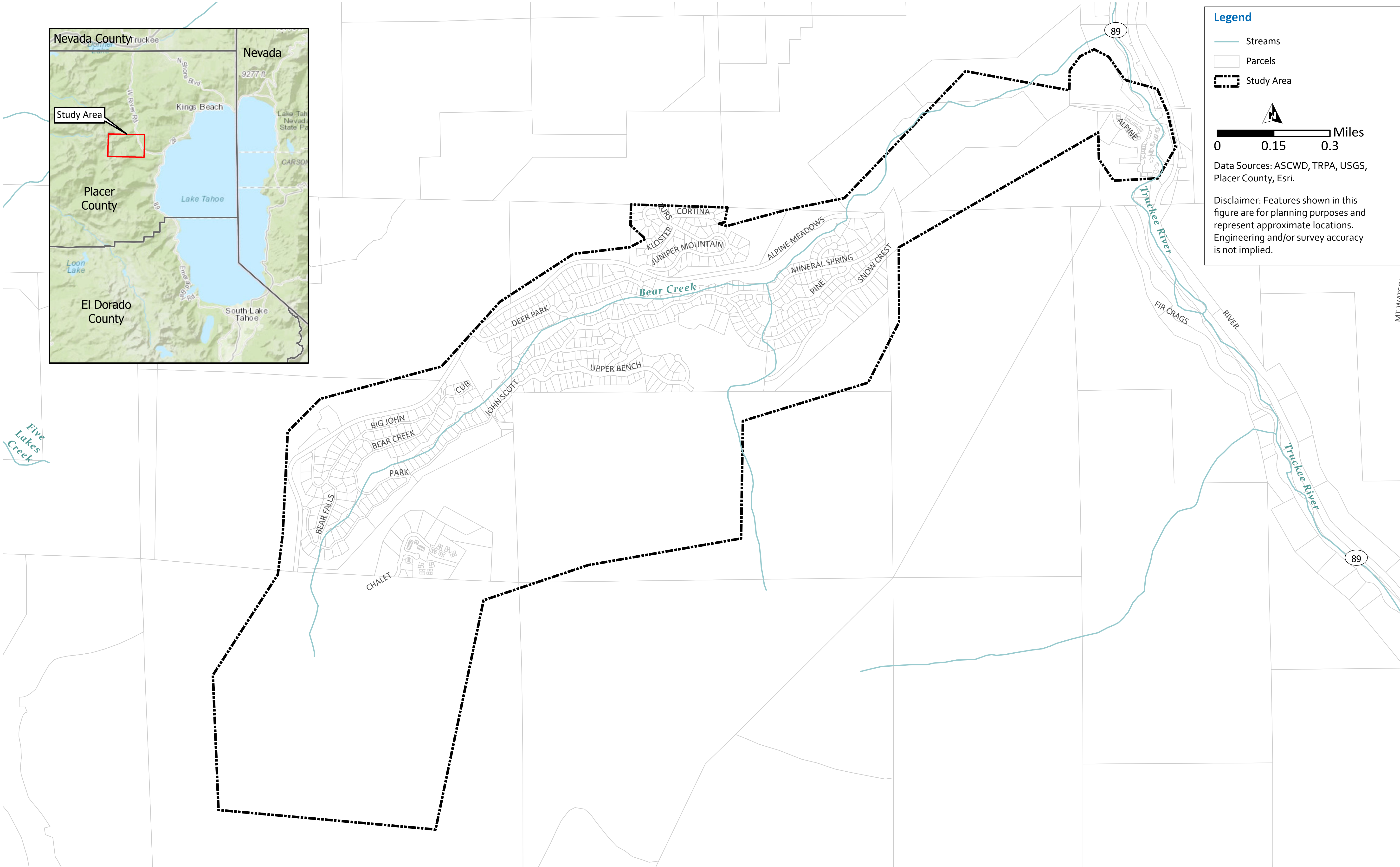


Figure 1.1 Study Area

1.2 Previous Planning Efforts

ASCWD and private developers have completed previous planning efforts to evaluate the water and wastewater systems. The major planning efforts conducted for the water and wastewater systems in the past 20 years are summarized below:

- 2006 Recommended Long Range Water and Sewer Master Plan (2006 Master Plan): The 2006 Master Plan is the most recent planning effort conducted specifically for ASCWD's water and wastewater systems in their entireties. This study evaluated the water and wastewater systems through the 2026 planning horizon and recommended improvements to mitigate system deficiencies.
- 2021 Fire Flow Alternatives Analysis: ASCWD conducted a fire flow alternatives analysis in 2021 to address deficiencies within Zone 3 of the water distribution system. The analysis evaluated three alternatives using a steady-state hydraulic water model and developed planning level cost estimates for each alternative.

1.3 Service Area

Alpine Meadows is a small community, approximately 4 miles northwest of Tahoe City, in Placer County, California. The ASCWD service area consists of approximately 37,000 acres in the valley around Bear Creek between California State Highway 89 and the Palisades Tahoe ski resort (previously known as Alpine Meadows).

1.3.1 Climate and Topography

Alpine Meadows typically experiences a Mediterranean climate characterized by dry, warm summers paired with cool, wet winters. Table 1.1 summarizes historical temperature and precipitation data from the nearest Western Regional Climate Center station in Olympic Valley, which is located directly north of Alpine Meadows. Most precipitation occurs between November and March, primarily as snow.

Table 1.1 Alpine Meadows Regional Climate Summary(1)(2)

Month	Average Maximum Temperature, degrees Fahrenheit	Average Minimum Temperature, degrees Fahrenheit	Average Total Precipitation, inches
January	39.0	13.9	10.03
February	42.0	15.7	8.01
March	44.4	20.2	8.30
April	49.1	22.1	4.40
May	63.5	30.1	1.47
June	73.7	36.8	1.16
July	80.9	41.4	1.25
August	79.6	40.5	0.84
September	72.8	34.2	1.26
October	58.7	26.9	4.61
November	43.8	19.6	9.16
December	38.6	13.3	8.54
Annual	**57.4**	**26.3**	**59.02**

Notes:
(1) Data obtained from https://wrcc.dri.edu/cgi-bin/cliMAIN.pl?ca8474. Data is from the Western Regional Climate Center station in Olympic Valley, which is directly north of Alpine Meadows.
(2) Period of record: 1971 to 2000.

Figure 1.2 shows the service area topography. The elevation within the service area ranges from 6,100 feet at the mouth of the valley to 6,800 feet at the highest inhabited area, Palisades Tahoe's ski lodge. The mountain peaks around the valley are recorded at 8,600 feet.

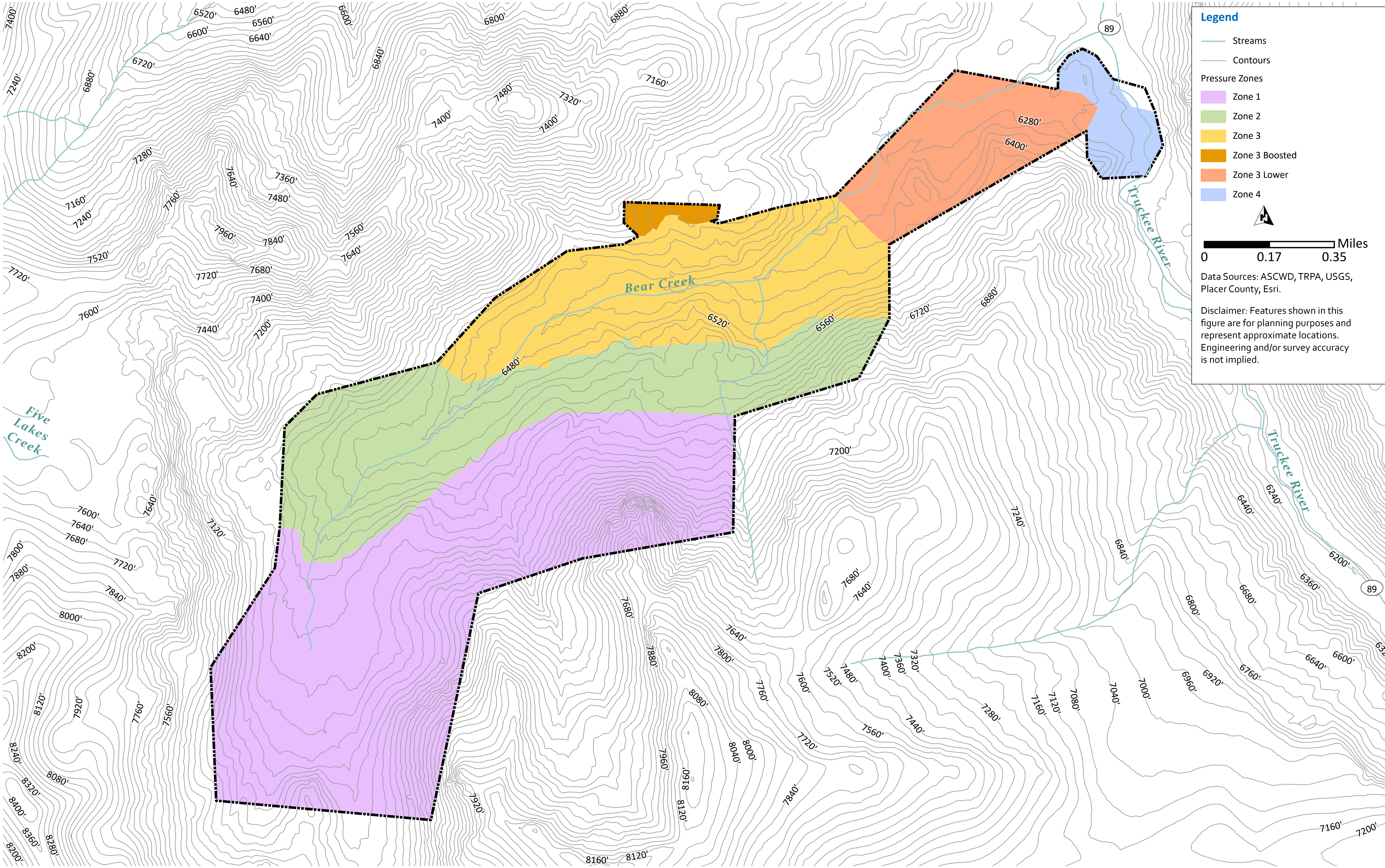


Figure 1.2 Service Area Topology

1.3.2 Land Use

ASCWD's service area is primarily residential. There is a small commercial center at the bottom of the valley, along California State Highway 89, and Palisades Tahoe ski resort is located at the top of the service area. The population varies greatly due to the prevalent tourism industry; Alpine Meadows is estimated to have a permanent population of approximately 200 people and a peak population of about 2,000.

This Master Plan considers two planned developments, Alpenglow and White Wolf, along with projected annual growth. Both planned developments are located close to the top of the system near the Palisades Tahoe ski resort. Alpenglow has planned for 52 single family residential (SFR) units to be developed and White Wolf has proposed the development of 58 SFR units. Alpine Meadows is also expected to experience a small amount of annual growth over the planning period due to the limited number of developable parcels within the service area.

Figure 1.3 shows the service area land use.

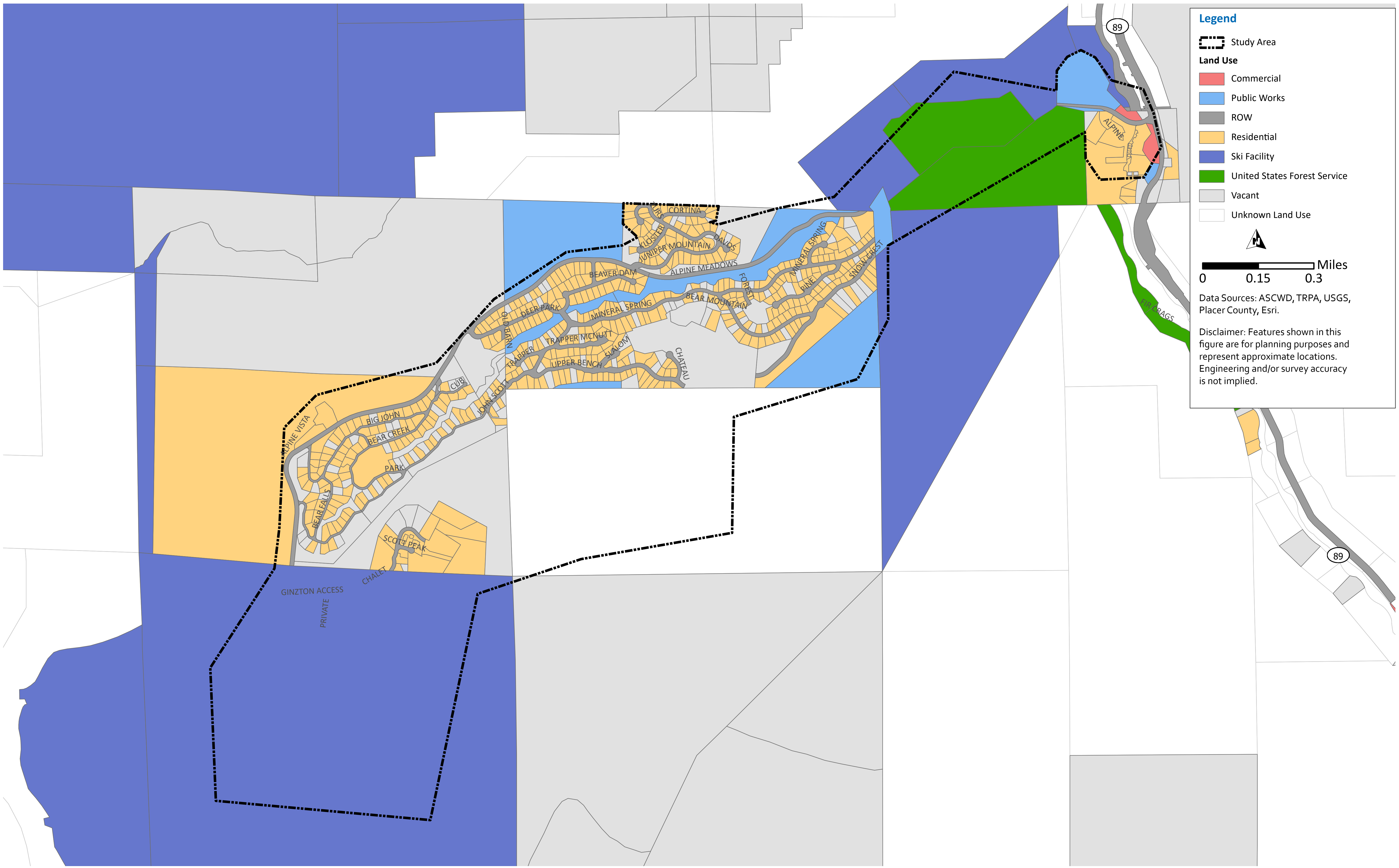


Figure 1.3 ASCWD Service Area Land Use

1.4 Project Purpose and Scope

The purpose of this Master Plan is to identify system deficiencies and recommend improvements along with planning level cost estimates. ASCWD authorized Carollo Engineers, Inc (Carollo) to evaluate ASCWD's water distribution and wastewater collection systems and develop a capital improvement program (CIP) through the 2045 planning horizon.

1.5 Report Organization

The Master Plan report contains six chapters, followed by appendices that provide supporting documentation for the information presented in the report. The chapters are briefly described below:

- Chapter 1 – Introduction. This chapter presents a brief summary of the ASCWD service area, the need for this Master Plan, and the objectives of the study.
- Water Demands and Wastewater Flows. This chapter presents the existing and future demand and flow components for both ASCWD's water distribution system and wastewater collection system.
- Chapter 3 – Existing Water and Wastewater Systems and Hydraulic Model Development. This chapter summarizes ASCWD's existing water distribution and wastewater collection system infrastructure and discusses the hydraulic models that were developed to simulate the two systems.
- Chapter 4 – Water Distribution and Wastewater Collection Systems Condition Assessment. This chapter discusses the condition of the water distribution and wastewater collection systems' above- and below-ground assets.
- Chapter 5 – Planning Criteria. This chapter discusses the planning criteria that were used to evaluate ASCWD's existing systems and to develop future water and wastewater system infrastructure.
- Chapter 6 – Water System Evaluation. This chapter discusses the water system hydraulic evaluation. The supply and storage analyses are presented along with the water system hydraulic model results.
- Chapter 7 – Wastewater System Evaluation. This chapter discusses the wastewater system hydraulic model evaluation and results.
- Chapter 8 – Water and Wastewater Systems Proposed Improvements. This chapter details the project phasing and improvements that are recommended for both the water distribution and wastewater collection systems. These improvements relate to the condition and capacity of these systems.
- Chapter 9 – Capital Improvement Plan. This chapter presents the capital improvement projects, a summary of the capital costs, and a basic assessment of the possible financial impacts to ASCWD.

Chapter 2

WATER DEMANDS AND WASTEWATER FLOWS

This chapter presents Alpine Springs County Water District's (ASCWD) existing and projected 2045 water demands and wastewater flows.

2.1 Existing Water Demands

ASCWD's existing water demands were estimated using historical production and consumption data. Historical demands and flows are discussed along with projected growth.

2.1.1 Existing Average Day Demand

A water system's average day demand (ADD) is typically defined as the system's average daily production over a typical year. ASCWD's ADD could not be calculated using raw production data due to the nature of the system's supply sources. Historically, ASCWD is supplied primarily via free-flowing springs, which flow at relatively constant rates independent of customer demands. Figure 2.1 shows ASCWD's average annual production between 2003 and 2021. Excess water that is produced by the system's sources but not consumed by customers falls under three categories:

- Overflow to snowmaking ponds: ASCWD sends water to Palisades Tahoe's snowmaking ponds when levels in its uppermost storage tank exceed a defined hydraulic grade. This operation is defined in greater detail in Chapter 3, Existing Water and Wastewater Systems and Hydraulic Model Development. Water sent to the snowmaking ponds via this method is estimated to total approximately 0.162 million gallons per day (mgd) on average.
- Excess water to ASCWD pond: Excess water that is available at the bottom of the system is discharged at ASCWD's pond located by its main office building. Discharge to the ASCWD pond is estimated to total about 0.005 mgd on average.
- Unaccounted-for-water (UFW): UFW is water that is lost through system defects such as tank leaks and pipeline cracks, unauthorized water use, inaccurate meters, or other events that cause water to be withdrawn from the system and not measured. Typical activities in which water is withdrawn from the system without being measured are system flushing, street cleaning, and firefighting. For the purposes of this study, ASCWD's UFW is assumed to account for 15 percent of the ADD.

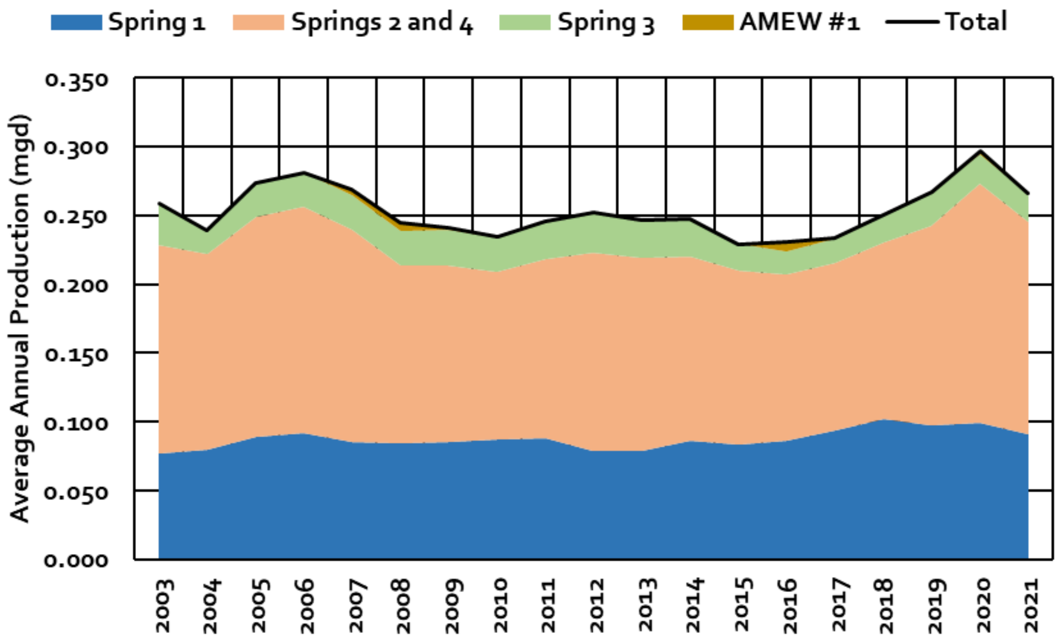


Figure 2.1 Historical Average Annual Production

Due to the lack of reliable meter data for water sent to the snowmaking ponds and ASCWD's pond, ASCWD's existing ADD was estimated using 2018 through 2020 consumption data and an assumed water loss percentage of 15 percent of the ADD, which is a typical value for similar water systems. Figure 2.2 shows ASCWD's total annual metered consumption between 2018 and 2020. The total average daily customer usage between 2018 and 2020 was calculated to be 0.073 mgd. With an additional 0.013 mgd to account for 15 percent water loss, ASCWD's existing ADD is approximately 0.086 mgd.

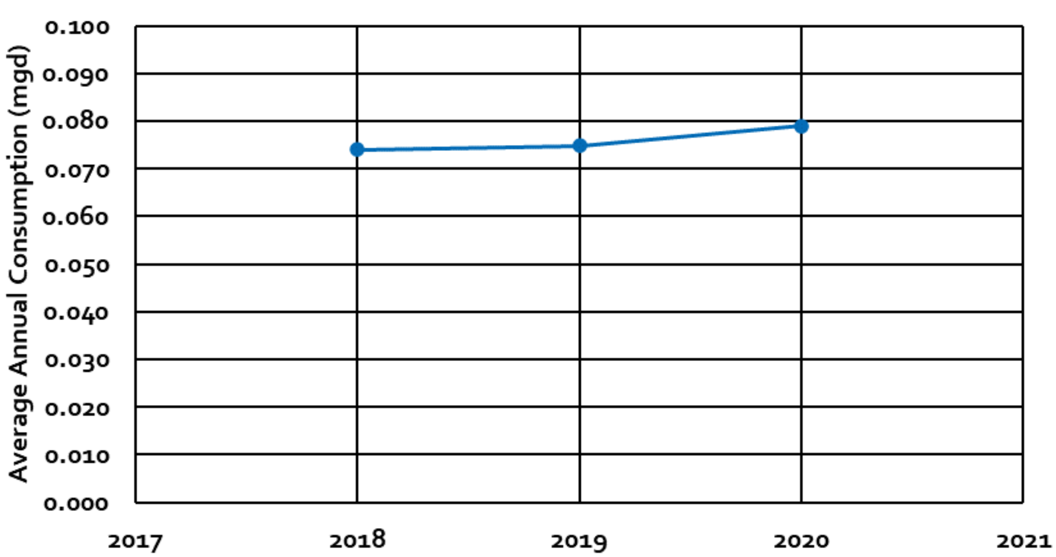


Figure 2.2 Historical Average Annual Consumption

Table 2.1 shows the metrics used to calculated ASCWD's existing ADD. Water sent to the snowmaking ponds and ASCWD's pond were excluded from the existing ADD because flows to the ponds are not controlled by the ponds' demands but rather by how much surplus water the system produces. If demands increase, excess water available for the ponds would decrease.

Table 2.1 Water System Existing Demand Categories

Demand Category	Existing Demand (mgd)	Source
Consumption	0.073	Calculated average from 2018 through 2020 water meter data.
Overflow to snowmaking ponds	0.162	Estimated using meter data from May 27, 2022, through June 7, 2022.
Excess water to ASCWD pond	0.005	Estimated according to input from ASCWD staff.
Unaccounted-for-water	0.013	Assumed to be 15 percent of average day demand.
Total production	0.253	Average total water produced by ASCWD's drinking water sources between 2003 and 2021.
Average day demand(1)	0.086	Calculated as the sum of consumption and unaccounted-for-water.

Note:
(1) Water sent to the snowmaking ponds and the ASCWD pond are excluded from the average day demand, leading to a difference between total production and average day demand of about 0.167 mgd.

2.1.2 Existing Maximum Day Demand

Water systems are generally evaluated using the highest daily demand the system is expected to experience, referred to as the maximum day demand (MDD). The difference between a water system's ADD and MDD depends on characteristics such as seasonal versus permanent population and climate. Water systems in dry regions tend to have substantially higher demands in the summer due to increased water usage for irrigation.

ASCWD's MDD was calculated using an assumed MDD to ADD peaking factor of 3.45, which is a common factor that has been calculated for other similar water systems in the Tahoe area. Using the MDD to ADD peaking factor of 3.45, ASCWD's existing MDD was calculated to be 0.297 mgd.

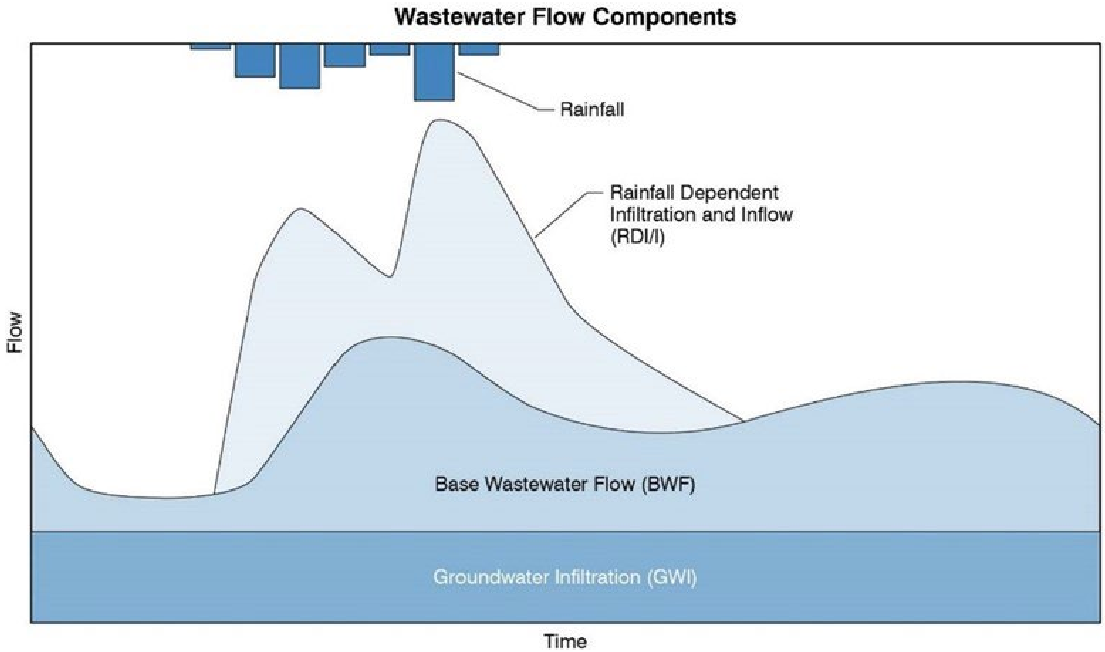
2.2 Wastewater Flow Components

Wastewater flows consist of two main components: dry weather flow (DWF) and wet weather flow (WWF). Both consist of various subcomponents, as detailed below:

- Base wastewater flow (BWF): BWF is the flow generated by ASCWD's wastewater customers. The BWF has a diurnal pattern that varies depending on land use, day of the week, and season. Residential diurnal patterns generally have peaks in the morning and evenings while commercial and industrial patterns generally have consistently high flows during the day and consistently low flows at night.

- Groundwater infiltration (GWI): GWI is groundwater that enters the gravity collection system through below-grade defects in pipes and manholes, such as cracks, misaligned joints, and breaks. Such defects may occur in both ASCWD-owned gravity mains and in privately-owned sanitary sewer laterals that discharge to the collection system.
- Inflow: Inflow is extraneous water that enters the collection system through above-grade sources. Examples of inflow are stormwater leaking through manhole covers and illicit connections.
- Infiltration and inflow (I/I): I/I describes all extraneous water that enters the collection system as either GWI or inflow.
- Dry weather groundwater infiltration: Dry weather GWI, also referred to as base infiltration, is the groundwater that enters the collection system under dry weather conditions. Dry weather GWI typically occurs when the relative depth of the groundwater table is higher than the depth of the gravity pipeline invert. Dry weather GWI often varies seasonally, coinciding with seasonal changes in groundwater ("seasonal mounding").
- Average dry weather flow (ADWF): The ADWF is the average daily flow during the dry weather season. The ADWF consists of the BWF and dry weather GWI components.
- Rain derived infiltration and inflow (RDI/I): RDI/I is infiltration and inflow that enters the collection system due to wet weather conditions. Rain derived infiltration occurs when stormwater percolates through the soil and then infiltrates the collection system through pipeline and manhole defects. Rain derived inflow is stormwater that enters the collection system via direct connections to the sanitary sewer system, such as storm drain cross connections, leaky manhole covers, and sewer cleanouts.
- Peak wet weather flow (PWWF): PWWF is the peak hourly flow that a wastewater collection system experiences during or after a storm event. PWWF consists of DWF and RDI/I and is typically driven by direct inflow.

Typical wastewater components and common sources of extraneous flow (i.e., GWI and RDI/I) that can enter a typical sanitary sewer system are shown on Figure 2.3 and Figure 2.4, respectively.



Note: Figure is not representative of flows specific to ASCWD or this study.

Figure 2.3 Typical Wastewater Flow Components

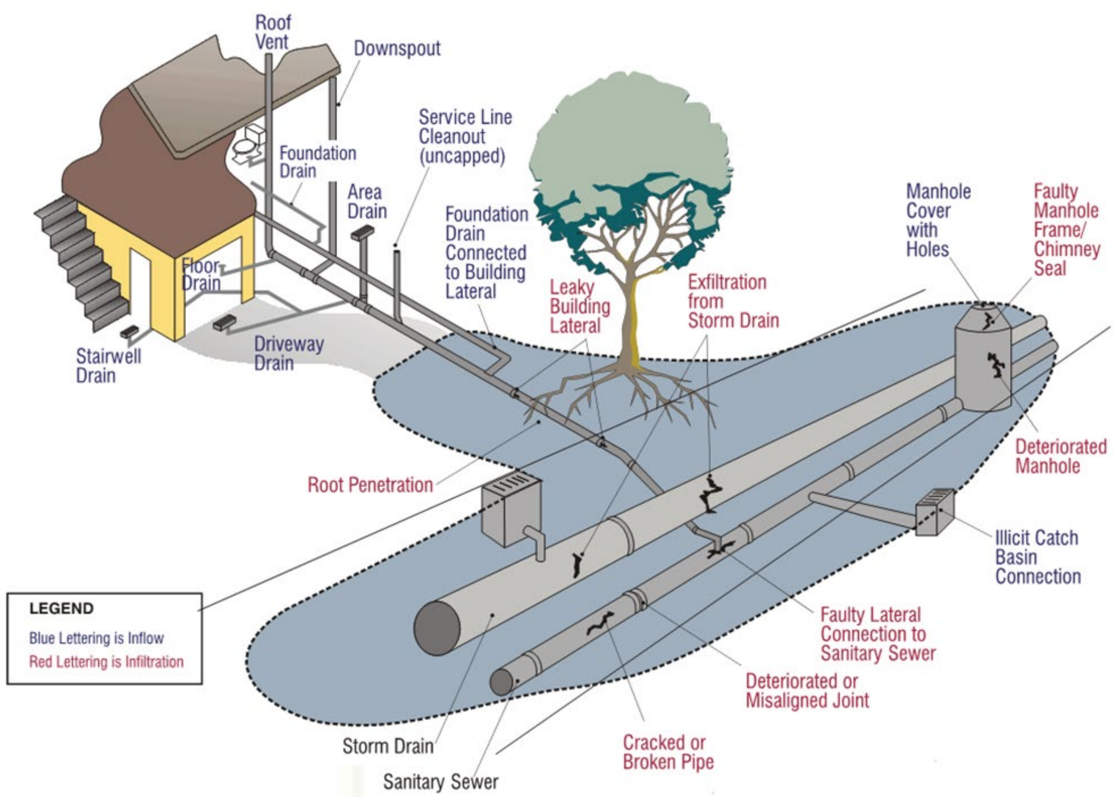


Figure 2.4 Typical Sources of Infiltration and Inflow

2.3 Existing Wastewater Flows

This section discusses ASCWD's historical wastewater flows and precipitation. The methodology used to determine existing dry and wet weather flows is also discussed.

2.3.1 Historical Wastewater Flows

ASCWD's wastewater flows discharge into the Tahoe-Truckee Sanitation Agency's (T-TSA) Truckee River Interceptor (TRI). T-TSA tracks ASCWD's wastewater flows via a permanent flow meter located just upstream of the TRI. Figure 2.5 shows the location of the permanent flow meter. As shown on Figure 2.5, the flow meter does not capture flows from ASCWD customers south of Alpine Meadows Road, most notably the River Ranch restaurant. For the purposes of this master plan, flows from those customers are assumed to be negligible.

Figure 2.6 shows average daily flows measured at the TRI flow meter between 2014 and 2022, and Figure 2.7 shows hourly flows measured between 2017 and 2022. Equivalent rainfall totals from a rain gauge located in Olympic Valley, which is just north of Alpine Meadows, are also shown on these figures. Table 2.2 summarizes ASCWD's historical wastewater flows.

Figure 2.5 ASCWD Permanent Flow Meter Upstream of the Truckee River Interceptor

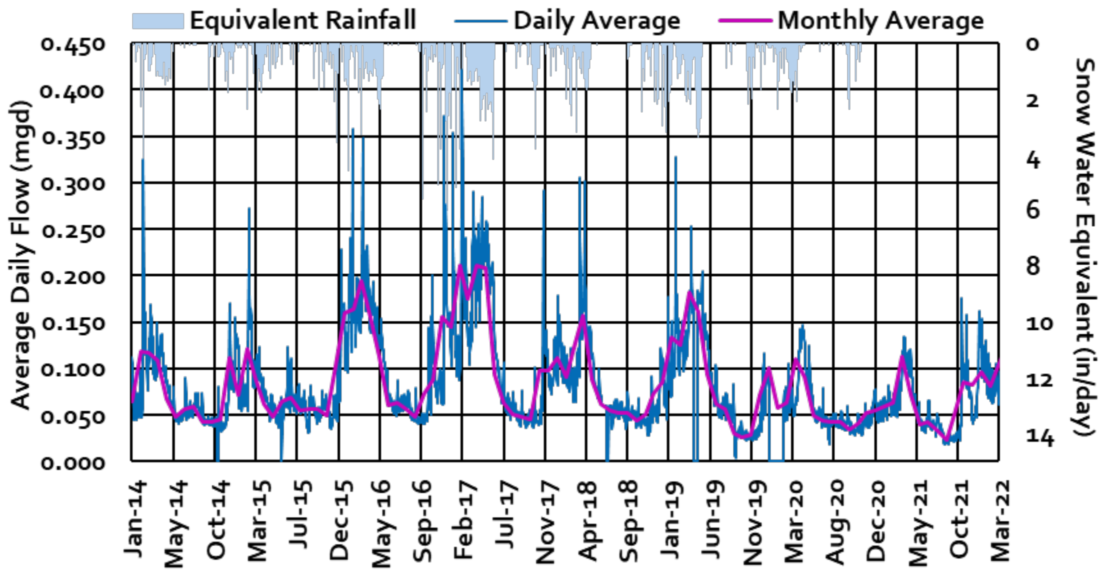


Figure 2.6 ASCWD Historical Average Daily Flow

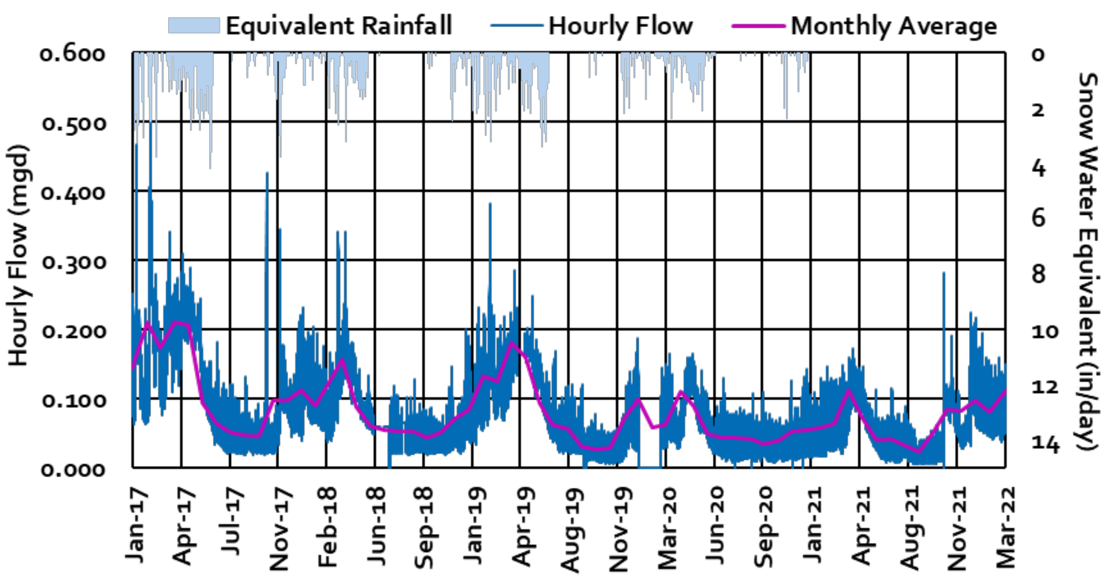


Figure 2.7 ASCWD Historical Hourly Flow

Table 2.2 ASCWD Historical Wastewater Flows and Equivalent Rainfall

Year	Total Snow Water Equivalent(1) (inches)	Average Annual Flow(2) (mgd)	Average July through September Flow(2) (mgd)	Peak Daily Flow(2) (mgd)	Peak Hourly Flow (2)(3) (mgd)
2014	81.2	0.073	0.053	0.324	N/A
2015	69.9	0.070	0.060	0.272	N/A
2016	127.4	0.111	0.056	0.372	N/A
2017	195.2	0.120	0.055	0.421	0.500
2018	87.3	0.079	0.053	0.305	0.342
2019	146.2	0.080	0.050	0.327	0.381
2020	70.8	0.050	0.043	0.146	0.187
2021	N/A	0.060	0.033	0.176	0.282
Average	**111.1**	**0.080**	**0.050**	**0.293**	**0.338**
Maximum	**195.2**	**0.120**	**0.060**	**0.421**	**0.500**

Notes:
(1) Snow water equivalent data is available only through December 2020. Values shown are from the Palisades Tahoe gauge located in Olympic Valley just north of Alpine Meadows.
(2) Flow data is from T-TSA's permanent flow meter located upstream of the TRI. Data excludes flows from customers located south of Alpine Meadows Road along Highway 89.
(3) Hourly flow data is only available for January 1, 2017, and later.

2.3.2 Existing Average Dry Weather Flow

ASCWD's existing dry weather flow is assumed to equal the average flow during the summer, specifically from July through September. The ADWF between 2014 and 2021 was approximately 0.050 mgd. For the purposes of this Master Plan, the existing ADWF is assumed to be 0.051 mgd, to be consistent other recent planning efforts.

2.3.3 Existing High Occupancy Flow

A high occupancy flow (HOF) to ADWF peaking factor was calculated using historical flow data to evaluate the wastewater collection system under peak flow conditions. ASCWD's HOF peaking factor is 2.409, leading to a HOF of 0.123 mgd.

2.3.4 Existing Peak Wet Weather Flow

ASCWD's PWWF was determined by routing the 10-year, 24-hour design storm through the hydraulic model, which is discussed in Chapter 3, Existing Water and Wastewater Systems and Hydraulic Model Development, in addition to the HOF. Using this method, ASCWD's existing PWWF was estimated to be 0.541 mgd.

2.4 Demand and Flow Projections

Demand and flow projections were developed through the 2045 planning horizon using planned development data and an assumed annual growth rate. The following sections discuss projected growth from annual growth and planned developments.

2.4.1 Projected Annual Growth

Annual growth apart from the planned developments is projected to occur at a rate of approximately 0.34 percent per year, or 2 added single-family residential units (SFR) per year. This annual growth rate was calculated from ASCWD's average yearly increase in sewer connections between 2002 and 2019.

Applying this annual growth rate to the existing ADD of 0.086 mgd equates to a total added ADD in 2045 of approximately 0.007 mgd. The total added ADWF from annual growth was calculated to be 0.004 mgd in 2045.

2.4.2 Planned Developments

The demand and flow projections incorporate two planned developments: Alpenglow and White Wolf. Both planned developments are located close to the top of the system near the Palisades Tahoe ski resort. Figure 2.8 shows the two planned developments. The two developments are discussed below:

- Alpenglow: This project is expected to start in year 2025 and to be completely built out by 2040. A total of 52 SFR units are expected, and roughly 3 units per year are assumed to be connected. Although the final development may involve fewer total units, this Master Plan is based on the units in the latest documents for this development.
- White Wolf: This project includes a total of 58 SFR units. This subdivision is assumed to begin connecting homes in 2035 and to be built out by 2040, which equates to roughly 10 added SFRs per year. Although the final development may involve fewer total units, this Master Plan is based on the units in the latest documents for this development.

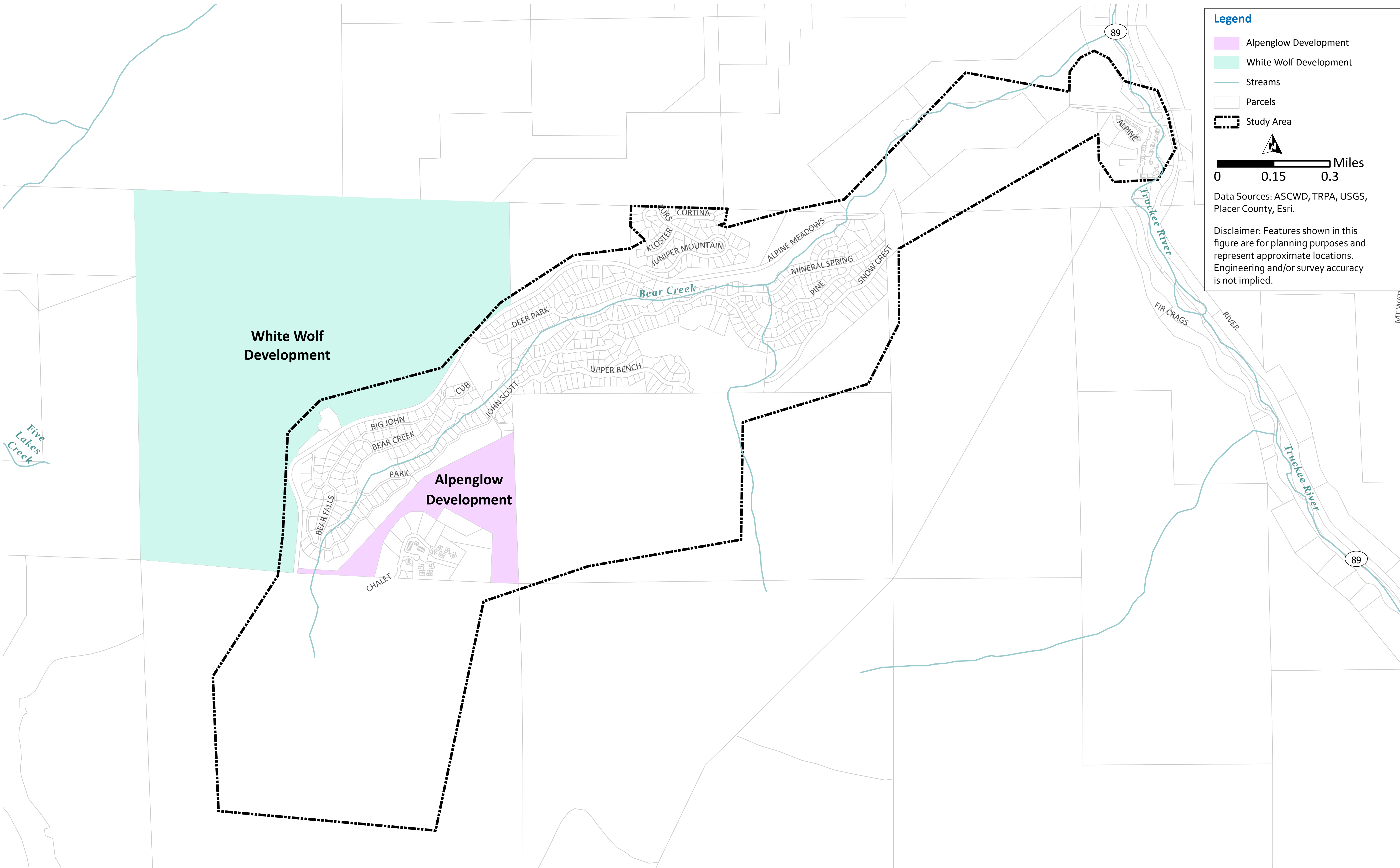


Figure 2.8 ASCWD Planned Developments

2.4.3 Demand and Flow Projections Summary

The total added ADD and ADWF from annual growth and planned developments is projected to equal 0.040 mgd and 0.024 mgd, respectively, in 2045. Table 2.3 summarizes the projected growth through 2045 within each development category. Figure 2.9 and Figure 2.10 show the projected increases in ADD and ADWF, respectively, through 2045.

Table 2.3 Projected Growth by Development Category

Development	Projected Development Schedule	Average Annual Growth Rate	Total Added SFRs in 2045	Total Added ADD in 2045 (mgd)	Total Added ADWF in 2045 (mgd)
Annual Growth	N/A	0.34% (i.e., 2 SFRs)	46	0.007	0.004
Alpenglow	2025 to 2040	3.25	52	0.016	0.009
White Wolf	2035 to 2040	9.67	58	0.017	0.010
Total	**N/A**		**156**	**0.040**	**0.024**

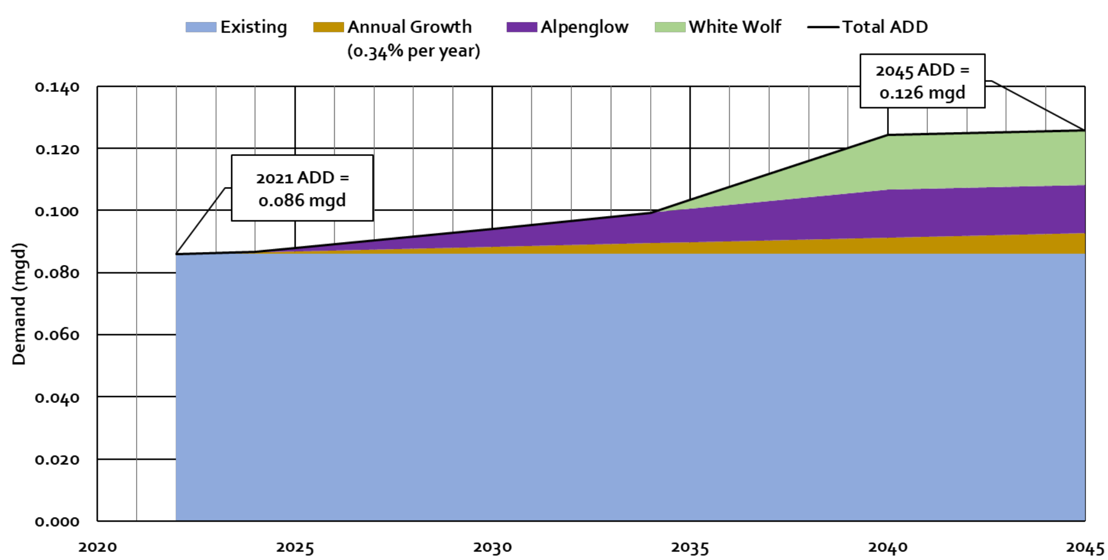


Figure 2.9 Projected ADD Growth by Development Category through 2045

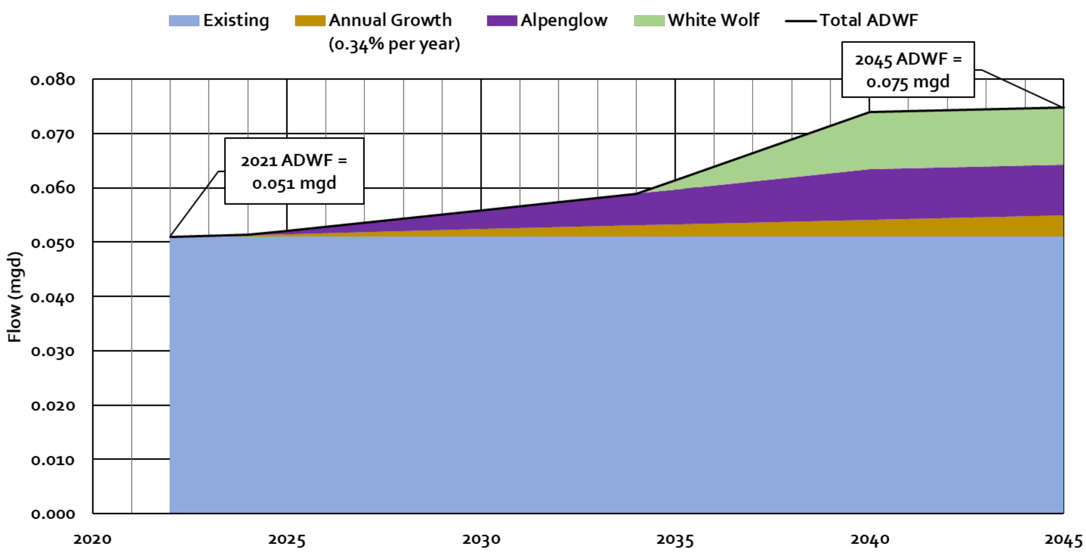


Figure 2.10 Projected ADWF Growth by Development Category through 2045

Table 2.4 summarizes ASCWD's existing and projected 2045 ADD and MDD. The systemwide MDD is projected to equal 0.434 mgd in 2045.

Table 2.4 ASCWD Existing and 2045 Water Demands

Water Demand Metric	Existing Demand (mgd)	2045 Demand (mgd)
ADD	0.086	0.126
MDD	0.297	0.434

Table 2.5 summarizes ASCWD's existing and projected 2045 ADWF, HOF, and PWWF. These values are reflective of the flows upstream of the TRI flow meter and exclude flows that tie into the TRI at other locations. The systemwide HOF and PWWF are projected to equal 0.180 mgd and 0.694 mgd, respectively, in 2045.

Table 2.5 ASCWD Existing and 2045 Wastewater Flows

Wastewater Flow Metric	Existing Flow (mgd)	2045 Flow (mgd)
ADWF	0.051	0.075
HOF	0.123	0.180
PWWF	0.541	0.694

Chapter 3

EXISTING WATER AND WASTEWATER SYSTEMS AND HYDRAULIC MODEL DEVELOPMENT

This chapter summarizes Alpine Springs County Water District's (ASCWD) existing water distribution system and wastewater collection system infrastructure and discusses the hydraulic models that were developed to simulate the two systems.

3.1 Water Distribution System

This section discusses ASCWD's existing water system infrastructure and operations. The system's pressure zones, supplies, storage tanks, and distribution system piping are described. Figure 3.1 shows an overview of the existing water system.

3.1.1 Pressure Zones

Water systems with varying topography are typically divided into separate hydraulic regions called pressure zones to maintain adequate pressures at each elevation range. A hydraulic grade line (HGL) is established for each pressure zone according to the elevation required to maintain a defined minimum pressure throughout the given zone. Facilities within and on the border of the zone, such as storage tanks, pressure reducing and sustaining valves, and pump stations, are operated to maintain the established HGL.

ASCWD's water system ranges in elevation from approximately 6,530 feet to 6,920 feet above sea level and consists of four main pressure zones, which are referred to as Zone 1 through Zone 4. Zone 3 is divided into three subzones: Zone 3, Zone 3 Boosted, and Zone 3 Lower. The system operates primarily via gravity conveyance, with water from the springs flowing directly into the distribution system. Water is transferred from higher to lower pressure zones through pressure regulating stations. Zone 3 Boosted, which consists of a residential neighborhood at the north end of Juniper Mountain Road, is the only pressure zone supplied via a booster pump station.

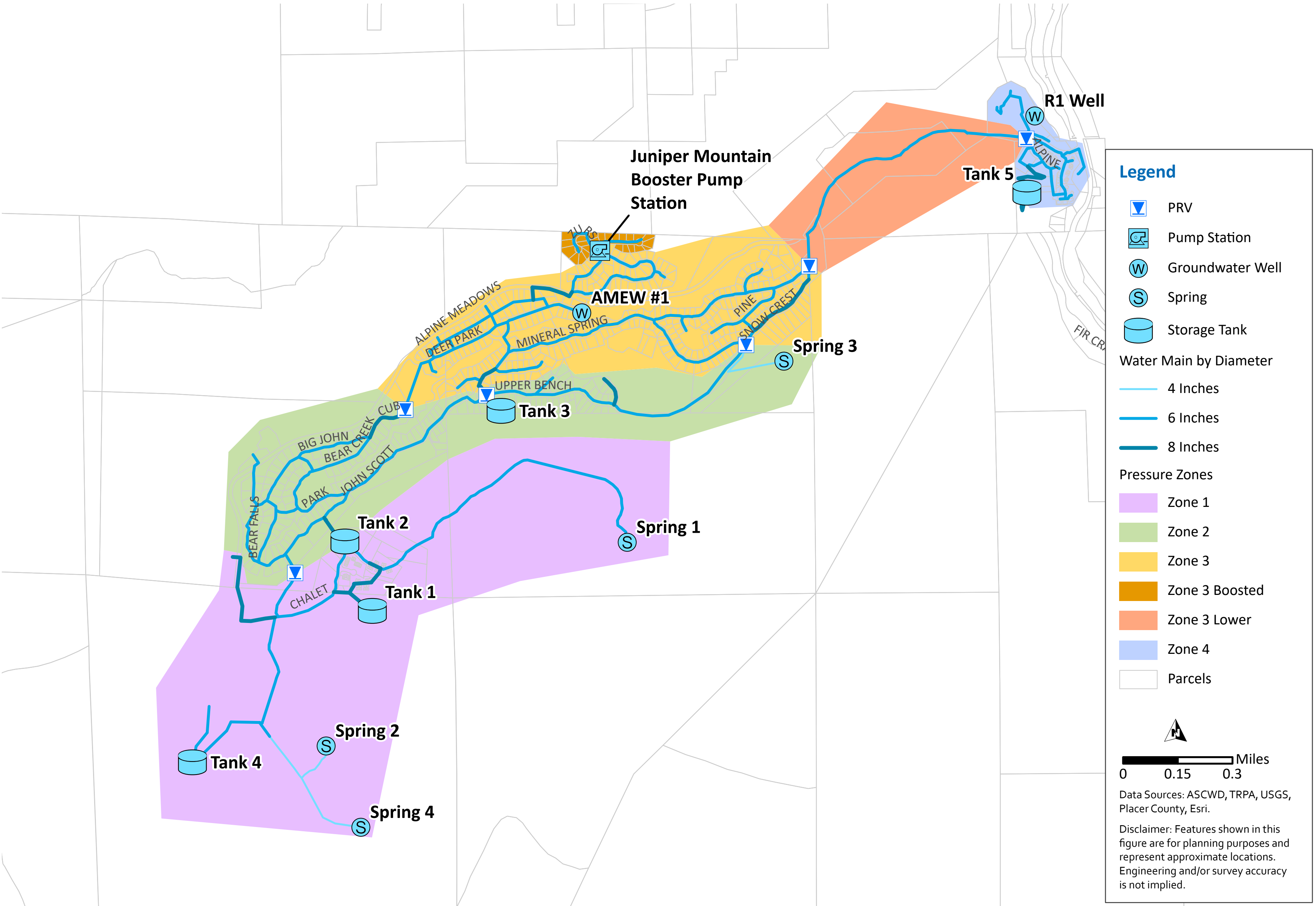


Figure 3.1 Existing Water System

Table 3.1 lists each pressure zone's nominal HGL, storage tanks, and supply sources. Table 3.2 shows each pressure zone's existing average day demand (ADD), maximum day demand (MDD), and peak hour demand (PHD). Figure 3.2 shows the system's hydraulic profile.

Table 3.1 Pressure Zones Summary

Pressure Zone	Nominal HGL[1] (ft)	Storage Tanks	Supply Sources to Pressure Zone			
			Booster Pump Stations	Pressure Regulating Stations	Springs	Groundwater Wells
Zone 1	7,063	Tank 1, Tank 4	None	None	Spring 1, Spring 2, Spring 4	None
Zone 2	6,868	Tank 2	None	A-1, A-2	None	None
Zone 3	6,672	Tank 3	None	B-1, B-2	Spring 3	AMEW #1
Zone 3 Boosted	6,754	None	Juniper Mountain PS	R-3	None	None
Zone 3 Lower	6,472	None	None	R-4	None	None
Zone 4	6,385	Tank 5	None	R-5	None	None

Note:
(1) Nominal HGL is defined by the tank high level elevation, pressure reducing valve (PRV) setpoint, or booster pump station (PS) operating point for the given pressure zone.
(2) Abbreviations: ft = feet; AMEW = Alpine Meadows Estates Well; PS = pump station.

Table 3.2 Existing Demands by Pressure Zone[1]

Pressure Zone	ADD (gpm)	MDD (gpm)	PHD (gpm)
Zone 1[1]	8.78	30.29	45.44
Zone 2	19.09	65.85	98.78
Zone 3	34.53	119.13	178.70
Zone 3 Boosted	2.99	10.30	15.46
Zone 3 Lower	0.65	2.24	3.36
Zone 4 [1]	11.47	39.57	59.35
Total	**77.50**	**267.39**	**401.08**

Note:
(1) Zone 4 totals excludes flow to the ASCWD pond. Zone 1 and Zone 4 totals exclude flows to the snowmaking system.
(2) Abbreviations: gpm = gallons per minute

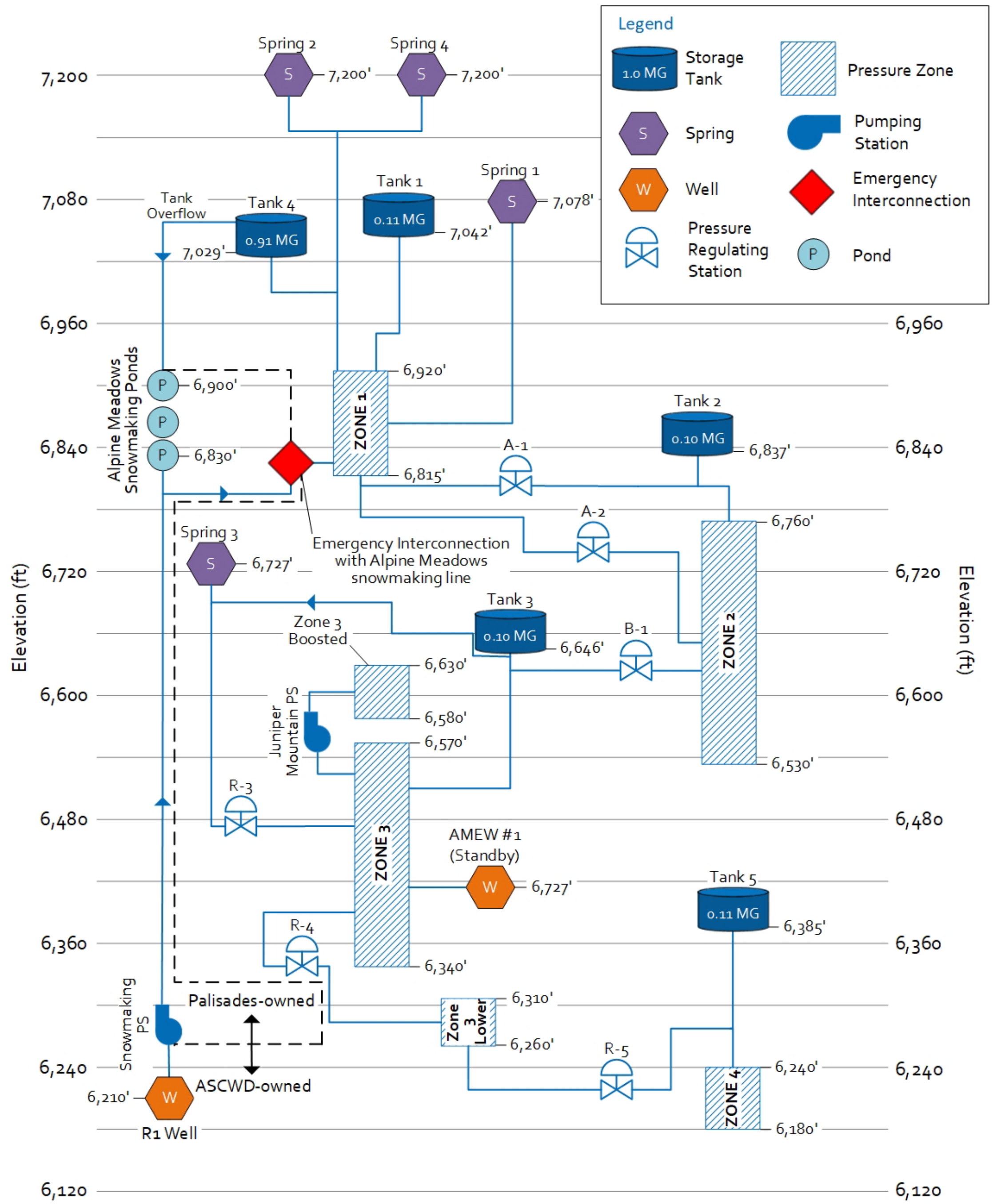


Figure 3.2 Water System Hydraulic Profile

3.1.2 Water Supply and Pumping Facilities

The water system is primarily supplied by four springs, which flow freely into the distribution system. ASCWD installed a groundwater well called the Alpine Meadows Estates Well (AMEW) Number 1 in 2015 but has only operated the well once over Summer 2020 when demands were abnormally high due to the Covid-19 pandemic. ASCWD also owns a well called R1 that is located in Zone 4. R1 supplies the snowmaking system and is typically not utilized to supply drinking water demands; however, R1 could supply Zone 4 if needed.

An additional well called R2 is located within Zone 4 but is not connected to ASCWD's system and was decommissioned in 2011 due to elevated manganese concentrations. When the well was active, it was operated and maintained by the Alpine Meadows ski resort as a snowmaking source. Future studies may evaluate the feasibility of blending groundwater from R2 with water from other supplies to produce acceptable water quality.

ASCWD's water system contains one booster PS. The Juniper Mountain PS transfers water from Zone 3 to Zone 3 Boosted at adequate pressure. Table 3.3 summarizes ASCWD's supply sources and pumping facilities.

Table 3.3 Supply Sources Summary

Supply Source	Pressure Zone	Type	Year Installed	Total Capacity (gpm)	Firm Capacity[4] (gpm)
Spring 1	Zone 1	Spring	1963	60	60
Spring 2	Zone 1	Spring	1963	60	60
Spring 3	Zone 3	Spring	1963	20	20
Spring 4	Zone 1	Spring	1963	60	60
AMEW # 1	Zone 2	Groundwater well	2015	220	0
R1[1]	Zone 4	Groundwater well	1992	350	0
R2[2] (out of service)	Zone 4	Groundwater well	1992	500	0
Juniper Mountain Pump Station	Zone 3 Boosted	Booster Pump Station	2005[3]	40	40

Notes:
(1) R1 is used for snowmaking purposes and typically does not supply the drinking water system. However, R1 could supply Zone 4 if needed.
(2) R2 was decommissioned in 2011 due to elevated manganese concentrations. Future studies may evaluate the feasibility of blending R2 groundwater with water from other supply sources to produce acceptable water quality.
(3) Original installation date is unknown. The facility has one duty pump that was installed in 2005 and a standby pump that was installed in 2022.
(4) Firm capacity is equal to the facility's total capacity with the largest pump on standby. Pump stations with only one pump have zero firm capacity.

3.1.3 Water Storage

Water distribution systems rely on stored water to help equalize daily fluctuations between supply and demand, to supply enough water for firefighting, and to meet demands during an emergency or an unplanned outage of a major source of supply. ASCWD has five storage tanks, which are shown on Figure 3.1.

Table 3.4 summarizes the five water storage tanks. Additional information related to the condition of each tank is provided in Chapter 4, Water Distribution and Wastewater Collection Systems Condition Assessment.

Table 3.4 Storage Tanks Summary

Tank Name	Pressure Zone	Year Installed	Material	Base Elevation (ft)	Nominal Diameter (ft)	Tank Height (ft)	Typical Operating Level Range (ft)	Capacity at MOL(1) (MG)	Operational Notes
Tank 1	Zone 1	1963	Concrete	7,042	30	25	15 to 20	0.106	• Tank 1 level controls Zone 1 HGL.
Tank 2	Zone 2	1963	Concrete	6,837	25	30	25 to 27	0.099	• Tank 2 is filled by an altitude valve (A-1) from Zone 1. • A bypass line was constructed in 2017 to allow Zone 2 to pull directly from Zone 1.
Tank 3	Zone 3	1963	Concrete	6,646	25	30	26 to 29	0.106	• Filled by an altitude valve (B-1) from Zone 2. • Zone 3 can also be supplied from Zone 2 via a PRV (B-2) at the end of Cub Lane; this is normally closed and acts as an emergency interconnection.
Tank 4	Zone 1	2019	Welded Steel	7,029	67	38.5	26 to 29	0.765	• Tank 4 currently acts as 100 percent fire storage. • Water from Tank 4 flows at 400 gpm to the snowmaking ponds when tank level exceeds 29 feet and stops flowing when tank level drops below 26 feet. • ASCWD occasionally increases flows from Tank 4 to snowmaking ponds during the peak snowmaking season.
Tank 5	Zone 4	1963	Concrete	6,385	30	25	17 to 20	0.106	• Tank 5 is filled by an altitude valve (R-4) from Zone 3 Lower.

Note:
(1) For the purposes of this study, each tank's MOL is considered to be four feet below the tank's height. Each tank's actual MOL can be determined as part of future structural evaluations.
(2) Abbreviations: MOL = maximum operating level; MG - million gallons.

3.1.4 Distribution System Pipelines

ASCWD's water system consists of approximately 14.5 miles of pipeline ranging from 4 to 8 inches in diameter. The vast majority of the water mains are asbestos cement pipelines that were installed during the system's original construction in the 1960s.

Table 3.5 and Figure 3.3 summarize the distribution system by diameter, and Table 3.6 and Figure 3.4 summarize the distribution system by pressure zone.

Table 3.5 Distribution System Pipelines by Diameter

Diameter (inches)	Length (feet)	Length (miles)	Length (percent of system)
4	2,700	0.5	4%
6	64,300	12.2	84%
8	9,400	1.8	12%
Total	**76,300**	**14.5**	**100%**

Table 3.6 Distribution System Pipelines by Pressure Zone

Pressure Zone	Length (feet)	Length (miles)	Length (percent of system)
Zone 1	17,200	3.2	22%
Zone 2	21,200	4.0	28%
Zone 3	24,200	4.6	32%
Zone 3 Boosted	1,700	0.3	2%
Zone 3 Lower	4,300	0.8	6%
Zone 4	7,800	1.5	10%
Total	**76,300**	**14.5**	**100%**

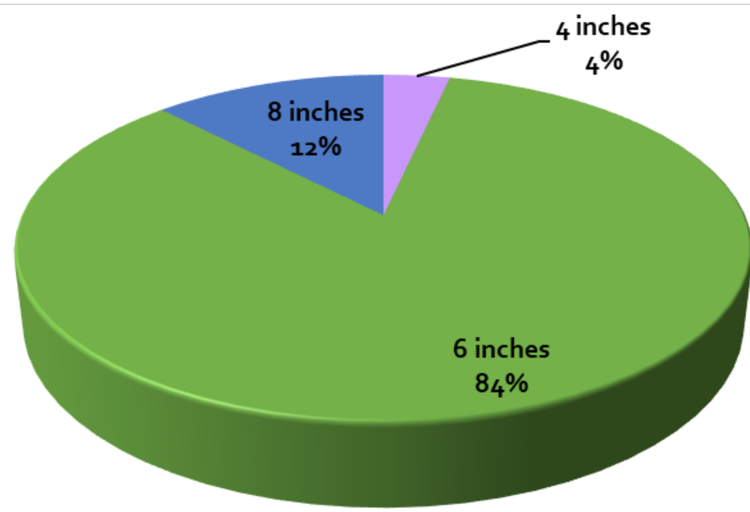


Figure 3.3 Distribution System Pipelines by Diameter

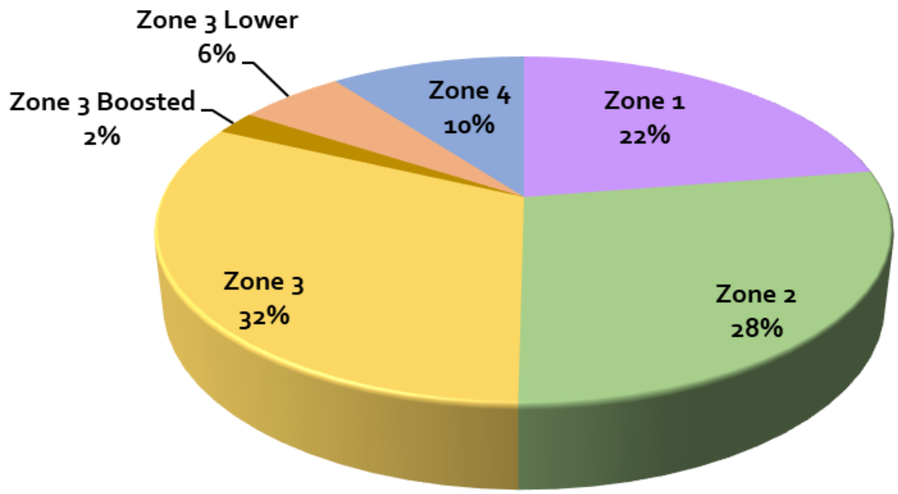


Figure 3.4 Distribution System Pipelines by Pressure Zone

3.1.5 Snowmaking System

Palisades Tahoe, a privately-owned ski resort located in Alpine Meadows, owns and maintains a snowmaking system that operates in conjunction with ASCWD's water system. The snowmaking system includes approximately 3 miles of 12-inch diameter transmission main that runs parallel to the ASCWD water system from Zone 4 to Zone 1, three snowmaking ponds, and pumping facilities that pump water from groundwater well R1 to the snowmaking ponds and from the snowmaking ponds to the ski resort.

When Tank 4 levels exceed 29 feet, water is sent from Tank 4 to the snowmaking ponds until the tank level drops below 26 feet. Deliveries from Tank 4 to the snowmaking ponds total about 0.162 million gallons per day (mgd), or 112 gpm, on average. ASCWD occasionally increases flows to the snowmaking ponds from Tank 4 during peak snowmaking season.

ASCWD owns and maintains the infrastructure from the R1 well to the booster pump station located adjacent to Alpine Meadows Road by the stables. Palisades Tahoe owns and maintains the booster pump station and piping from the pump station to the resort. Figure 3.5 shows an overview of the snowmaking system.

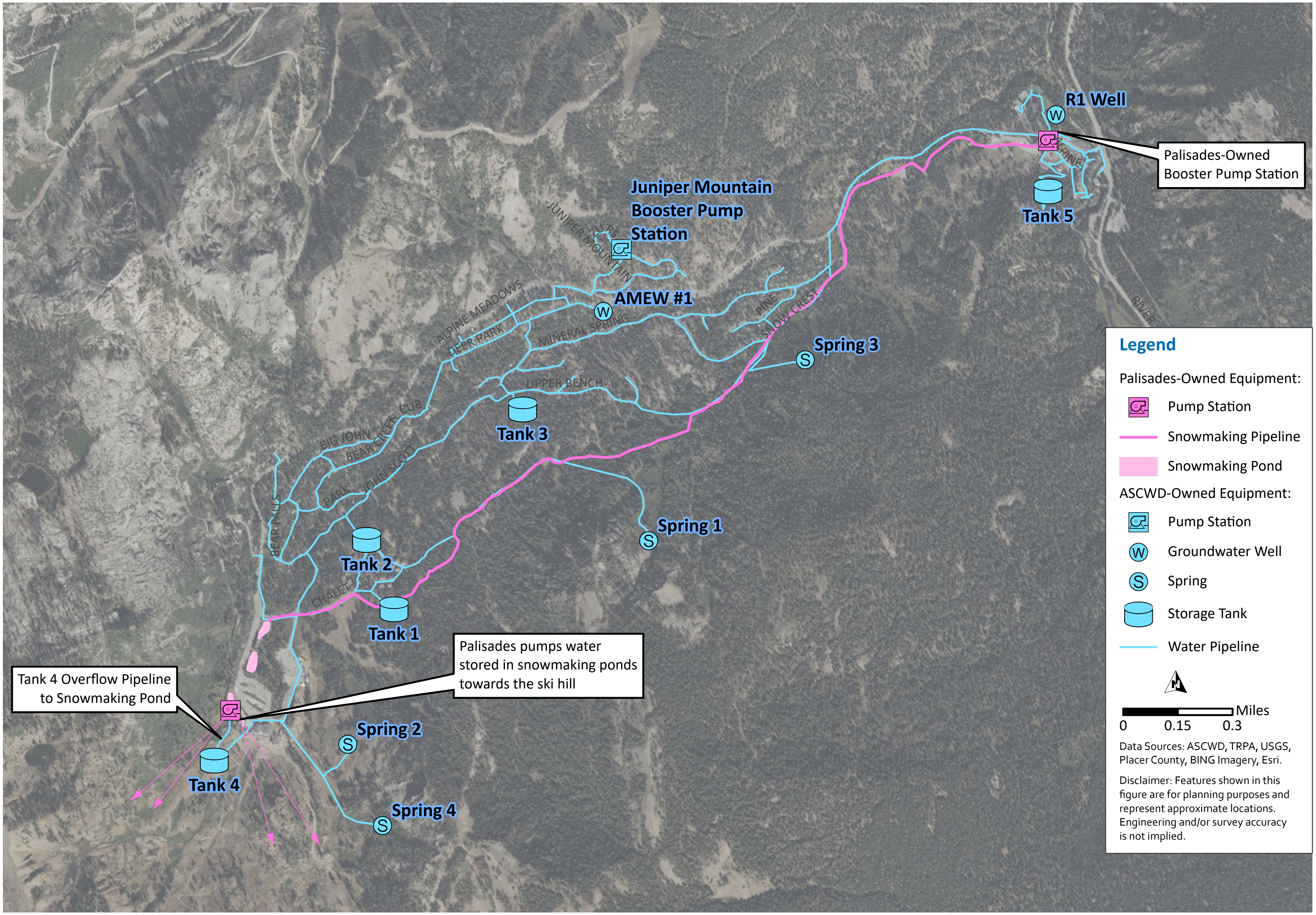


Figure 3.5 Snowmaking System Overview

3.2 Water Model Development

This section discusses the development and calibration of ASCWD's hydraulic water system model.

3.2.1 Water Model Construction

ASCWD's water model was developed in a multi-step process utilizing data from various sources. The hydraulic model was constructed using Bentley Systems, Inc.'s WaterGEMS platform based on record drawings as the primary source. Additional physical and operational data, such as the Juniper Mountain Pump Station pump curve and valve set points, were provided by ASCWD staff.

3.2.1.1 Water Model Elements

Table 3.7 describes the elements that comprise ASCWD's water model.

3.2.1.2 Water Model Demands

Demands are simulated in the hydraulic model using a base demand and diurnal pattern. The base demand is the average demand consumed at a model junction for a given scenario, and the diurnal pattern describes the temporal distribution of demand over a 24-hour period. Demands were allocated to model junctions using the following process:

1. Demand nodes, or model junctions with applied demands, were defined in the hydraulic model. Junctions outside of facilities and in close proximity to customer connections were selected as demand nodes.
2. ASCWD's customer meter data was geolocated using customer addresses. The customer meter data was scaled from the raw measured consumption such that the total demand equaled ASCWD's existing ADD. For scenarios with demands greater than or less than the ADD, each demand node's base demand is scaled up or down from the ADD.
3. Each geolocated customer meter point was allocated to the nearest demand node using geographical information system (GIS) spatial tools. The allocation results were visually inspected to verify that demands were allocated to the correct demand nodes.

The geolocated customer meter data was imported into the model and assigned the system's diurnal pattern.

Figure 3.6 shows the diurnal pattern that was applied to all residential and commercial demand junctions in ASCWD's water model. This pattern was developed using hourly production data from other similar water systems in the Tahoe area. The diurnal pattern's peak hour multiplier is 1.50, meaning that the peak demand is 1.50 times the average demand during a given day.

Table 3.7 Water Model Elements

WaterGEMS Element	ASCWD Facilities	Input Parameters	Description
Junction	• Pipeline intersections.	• Demands. • Elevation.	Junctions are used to simulate demands and to segment pipes wherever the hydraulic modeling parameters of the pipes change. Junctions are also used to collect data at specific locations, such as high or low elevation points.
Pipe	• Distribution mains. • Transmission mains. • Closed or open valves.	• From and to nodes. • Diameter. • Length. • Roughness coefficient (Hazen-Williams C Factor). • Minor loss coefficients. • Check valve (Yes/No). • Operational controls.	Pipes are used to convey water through the model. Pipes are also used to simulate 100 percent open or closed valves.
Pump	• Juniper Mountain Pump Station.	• Elevation. • Pump curve. • Initial settings (On/Off and initial relative speed). • Operational controls.	Pumps are used to transfer water to the distribution system at adequate pressure and head in the model.
Tank	• Tanks 1 through 5.	• Base elevation. • Diameter. • Minimum water level. • Maximum water level. • Initial water level.	Tanks are used to simulate water storage facilities.
Reservoir	• Springs. • Groundwater wells.	• Elevation.	Reservoirs are used to simulate supply sources. The model assumes unlimited supply from reservoirs, so additional elements are placed between the reservoir and system to control supply to the system.

WaterGEMS Element	ASCWD Facilities	Input Parameters	Description
Hydrant	• Fire Hydrants.	• Elevation. • Hydrant status (Closed/Open).	Hydrants are used to simulate fire hydrants. Fire flow analyses utilize hydrants to calculate available fire flow and residual pressures at fire hydrants.
Pressure reducing valve (PRV)	• A-2. • B-2. • R-3.	• Elevation. • Initial pressure setting. • Operational controls.	PRVs are used to simulate valves in which the downstream facilities may not exceed a specified pressure setting.
Pressure sustaining valve (PSV)	• A-1. • B-1. • R-4. • R-5.	• Elevation. • Initial pressure setting. • Operational controls.	PSVs are used to simulate valves in which the upstream facilities may not drop below a specified pressure setting.
Flow control valve (FCV)	• Flow from springs. • Flow from Tank 4 to snowmaking ponds.	• Initial flow setting. • Operational controls.	FCVs are used to limit flow to a specified setting. FCV settings in ASCWD's water model were determined from historical metering data, SCADA data, and discussions with ASCWD staff.

Notes:
(1) Abbreviations: FCV = flow control valve; PSV = pressure sustaining valve; SCADA = supervisory control and data acquisition.

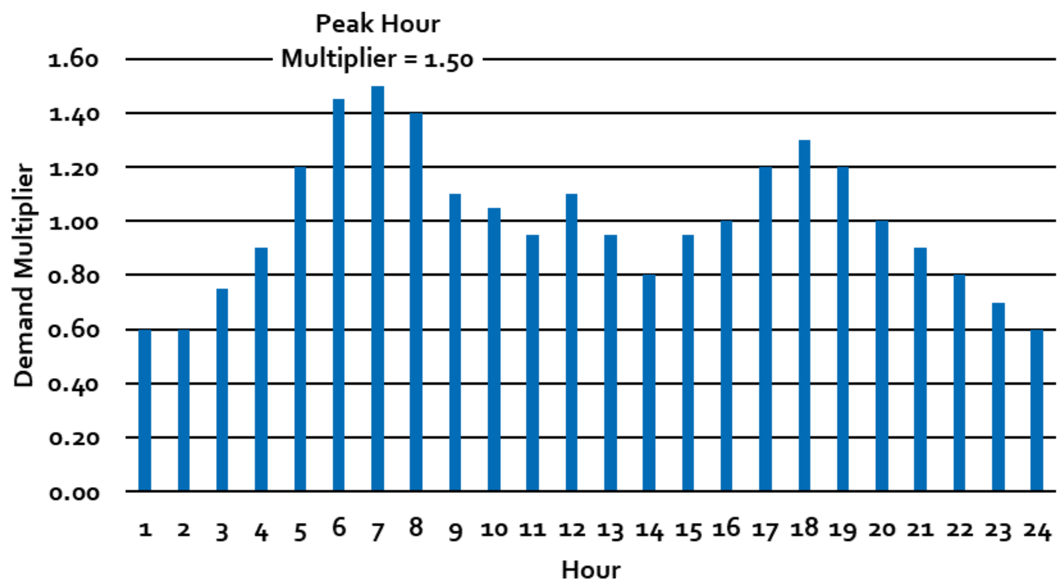


Figure 3.6 ASCWD Water System Diurnal Pattern

3.2.2 Water Model Calibration

ASCWD's water model was calibrated to verify that the model accurately simulates typical water system parameters, such as flows and pressures. ASCWD's water model was calibrated in three main steps: a macro calibration, an extended period simulation (EPS) calibration, and a fire flow calibration.

3.2.2.1 Model Calibration Data

Various data were collected to calibrate the hydraulic water model. Each major data source is described below:

- SCADA data: Data for the following facility parameters were collected from ASCWD's SCADA system and transcribed in hourly increments from May 27, 2022, through June 6, 2022:
  - Tank levels for Tanks 1 through 5.
  - Flows from Tank 4 to the snowmaking ponds.
- Temporary pressure-logging data: Temporary pressure loggers were installed on eight fire hydrants to collect continuous pressure readings from May 27, 2022, through June 6, 2022. Figure 3.7 shows the hydrant locations where temporary pressure loggers were installed.
- Fire flow test data: ASCWD provided data from nine historical fire flow tests completed in 2020 and 2021. Tank levels during the fire flow tests were assumed according to static pressure measurements taken immediately prior to the tests. Figure 3.8 shows the fire flow test locations.

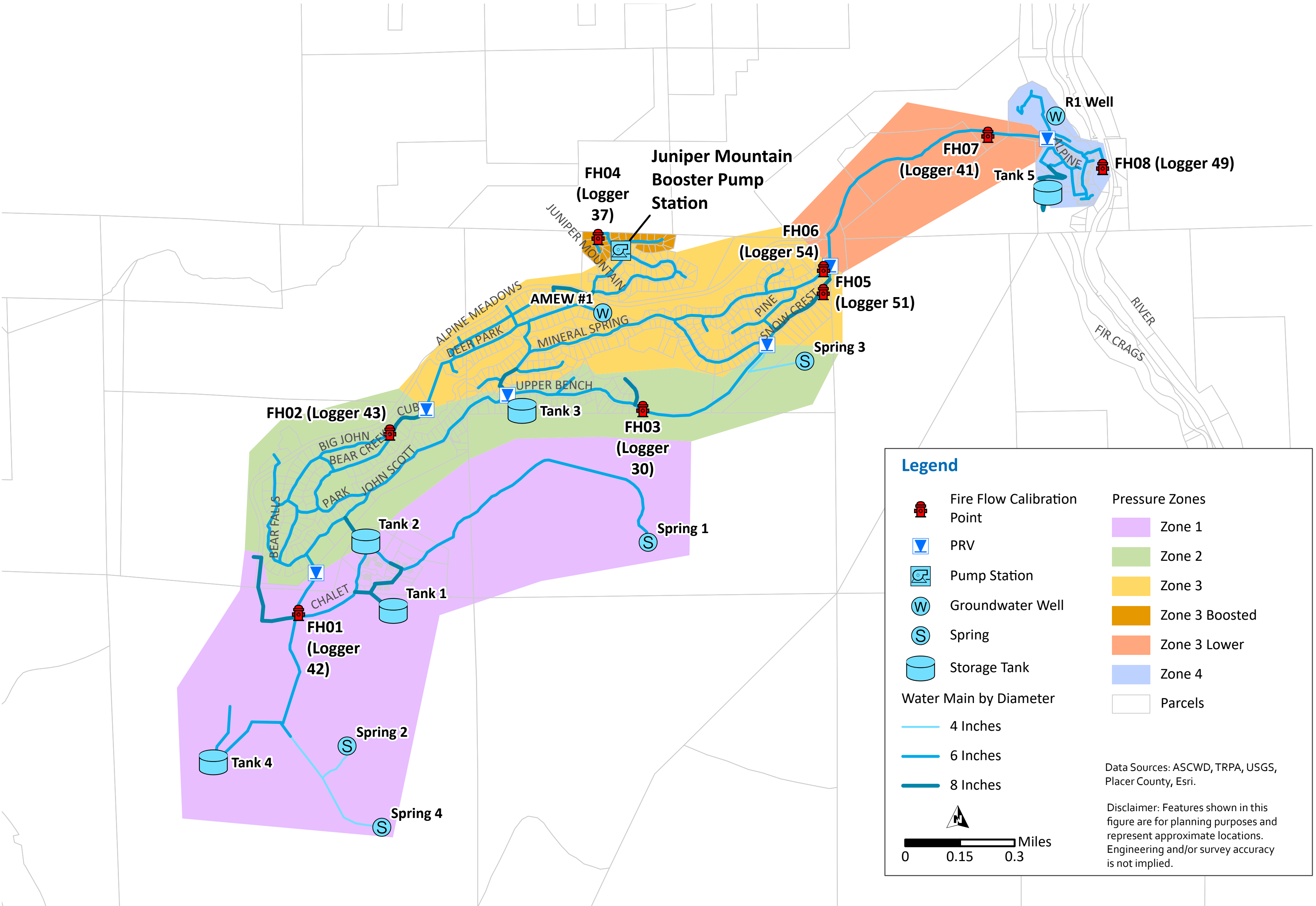


Figure 3.7 Water Model Extended Period Simulation Calibration Points

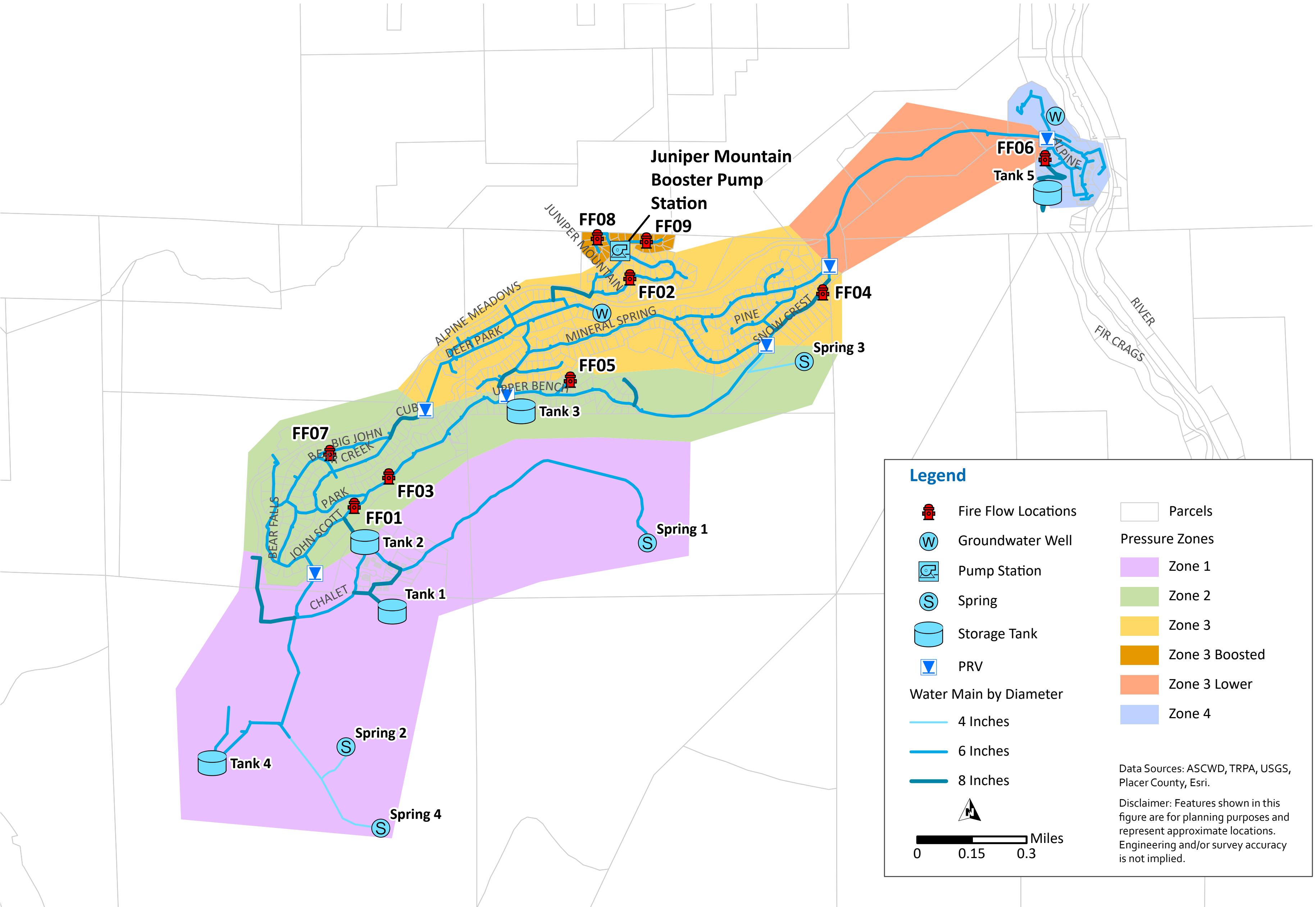


Figure 3.8 Water Model Fire Flow Calibration Points

3.2.2.2 Macro Calibration

The model was initially run under typical existing demand conditions to verify that it simulates realistic flows and pressures. As part of the macro calibration, the following model parameters were checked and adjusted as needed to produce realistic results:

- Distribution main connectivity: Distribution main connectivity was verified using WaterGEMS connectivity tools and output reports for pipeline flow and velocity. Pipelines with no flow or with unreasonably high flow or velocity were flagged and reviewed for proper connectivity.
- System pressures: Modeled pressures were compared to expected pressures within each pressure zone to identify major errors, such as inaccurate elevations. Operational controls were also added to the model to accurately reflect typical system operations.
- Facility characteristics: Model results for major facilities were compared to historical data to verify that facility attributes were accurately entered in the model and produce results comparable to what ASCWD typically experiences.

3.2.2.3 Extended Period Simulation Calibration

An EPS calibration was conducted to verify that the hydraulic model can accurately simulate real world operations of the distribution system facilities, such as pressure and flow fluctuations, tank fill and drain cycles, pressure regulating station operations, and pump station operations. Various model parameters were adjusted during the EPS calibration to enable the model to accurately replicate the measured data.

The 48-hour period from Saturday, May 28, 2022 through Sunday, May 29, 2022, was selected as the EPS calibration period. This 48-hour period was selected because it captured the Memorial Day Holiday weekend and represents a typical high-demand period. The daily demand for the calibration period was assumed to equal the system's ADD of 0.086 mgd, which assumes 0.073 mgd of raw consumption, 0.162 mgd to the snowmaking ponds from Tank 4 overflow, 0.005 mgd to the ASCWD pond, and 0.013 mgd of unaccounted for water (UFW). The UFW was calculated as 15 percent of the ADD excluding flows to the snowmaking and ASCWD ponds

Model-simulated flows, pressures, and tank levels were compared to field-measured data from the calibration period. Model parameters were adjusted until the model simulated measured flow, pressure, and level fluctuations.

Figure 3.9 shows the EPS calibration results for FH01 as an example. The remaining EPS calibration results are shown in Appendix 3A, Water Model Extended Period Simulation Calibration Results.

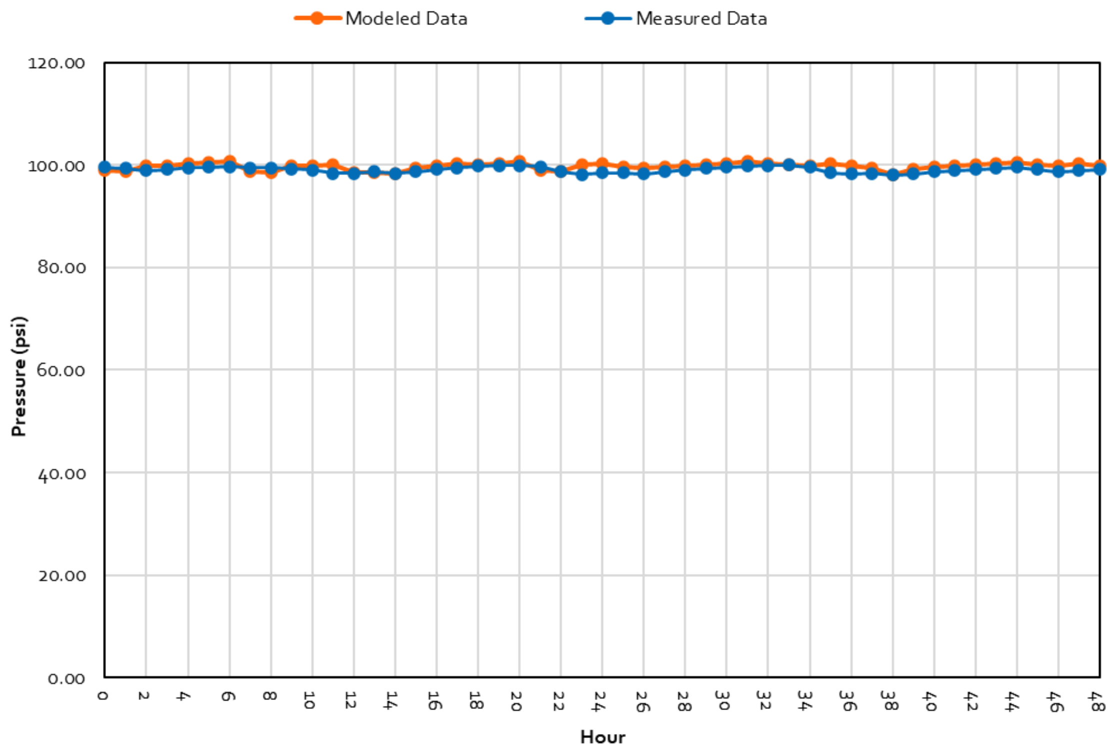


Figure 3.9 Example EPS Calibration Results: FH01

In general, the EPS results demonstrate the model's ability to predict measured distribution system pressures and tank level fluctuations to within 10 percent or 10 pounds per square inch (psi). The model could not achieve the measured pressure drop at one hydrant, FH07, with realistic water system parameters. This hydrant is located in Zone 3 Lower along Alpine Meadows Road by the Alpine Transit Center. Due to the substantial measured pressure drop at this location when flows through the 6-inch diameter water main along Alpine Meadows Road increase, it is suspected that the water main contains an obstruction somewhere between R-4 and R-5. ASCWD staff have verified the absence of closed or partially closed valves along this line. It is recommended that ASCWD further investigate potential obstructions, such as caved-in sections, by sending a camera through the water main's length.

3.2.2.4 Fire Flow Test Calibration

A fire flow test calibration was conducted to verify the model's ability to accurately simulate the water system's performance under extreme demand conditions, such as when fire hydrants are being operated. The primary varied parameter for this calibration is the pipeline roughness coefficient.

Hazen-William coefficients, or C-factors, have industry-accepted ranges according to material, diameter, and age. Characteristics specific to the ASCWD water system, such as water quality, temperature, construction methodologies, and material suppliers, may result in roughness coefficients that are at the higher or lower ends of the industry-accepted ranges.

The relative effect of roughness coefficients on water system operations is correlated with system demand. During typical demand conditions, roughness coefficients have a relatively low effect on water system operations. On higher demand days with larger flows and velocities, roughness coefficients exert a greater influence on overall system head loss. Fire flow tests artificially create high demand events to generate more head loss and enable a better estimation of pipeline roughness coefficients.

Table 3.8 summarizes the fire flow calibration results. The model accurately predicted the measured pressure drops for each historical fire flow test to within 10 psi.

Table 3.8 Fire Flow Calibration Results Summary

Fire Flow Test ID	Fire Flow Test Date	Measured Pressure Drop (psi)	Modeled Pressure Drop (psi)	Modeled versus Measured Difference (psi)
FF01	10/26/2020	6	12	6
FF02	10/26/2020	44	46	2
FF03	10/26/2020	48	45	-3
FF04	10/28/2020	58	63	5
FF05	10/28/2020	90	93	3
FF06	10/26/2020	22	21	-1
FF07	3/22/2021	32	38	6
FF08	6/2/2021	50	53	3
FF09	7/7/2021	32	30	-2

3.2.2.5 Water Model Calibration Summary

The EPS and fire flow calibration results demonstrate that the water model reliably simulates water system operations under typical and extreme demand conditions. The system accurately simulated tank fill and drain patterns during the EPS calibration period; similarly, the model predicted measured pressure drops during the nine fire flow tests used for the fire flow calibration to within less than 10 psi.

The water model should be recalibrated on a five- to ten-year basis or more frequently to verify that it continues to produce reliable results from which to evaluate the distribution system's hydraulic capacity. ASCWD should consider upgrading its SCADA software such that measured data can be easily extracted to complete future calibration efforts.

3.3 Wastewater Collection System

ASCWD's wastewater collection system consists of approximately 10.3 miles of gravity mains ranging in diameter from 6 to 10 inches and approximately 230 manholes. Wastewater flows from ASCWD and discharges into the Tahoe-Truckee Sanitation Agency's (T-TSA) Truckee River Interceptor (TRI), which conveys wastewater from several North Lake Tahoe communities to a regional Water Reclamation Facility (WRF) in Martis Valley east of the Town of Truckee, California. Figure 3.10 shows an overview of the existing wastewater system.

Figure 3.11 and Table 3.9 summarize the collection system gravity mains by diameter. Table 3.10 summarizes the system's manholes by depth, and Figure 3.12 shows the wastewater system's manhole depths.

The vast majority of ASCWD's collection system was constructed in the 1960s using asbestos cement pipeline. Few rehabilitation projects have been completed since the system was originally built. The wastewater collection system's condition is further discussed in Chapter 4, Water Distribution and Wastewater Collection Systems Condition Assessment.

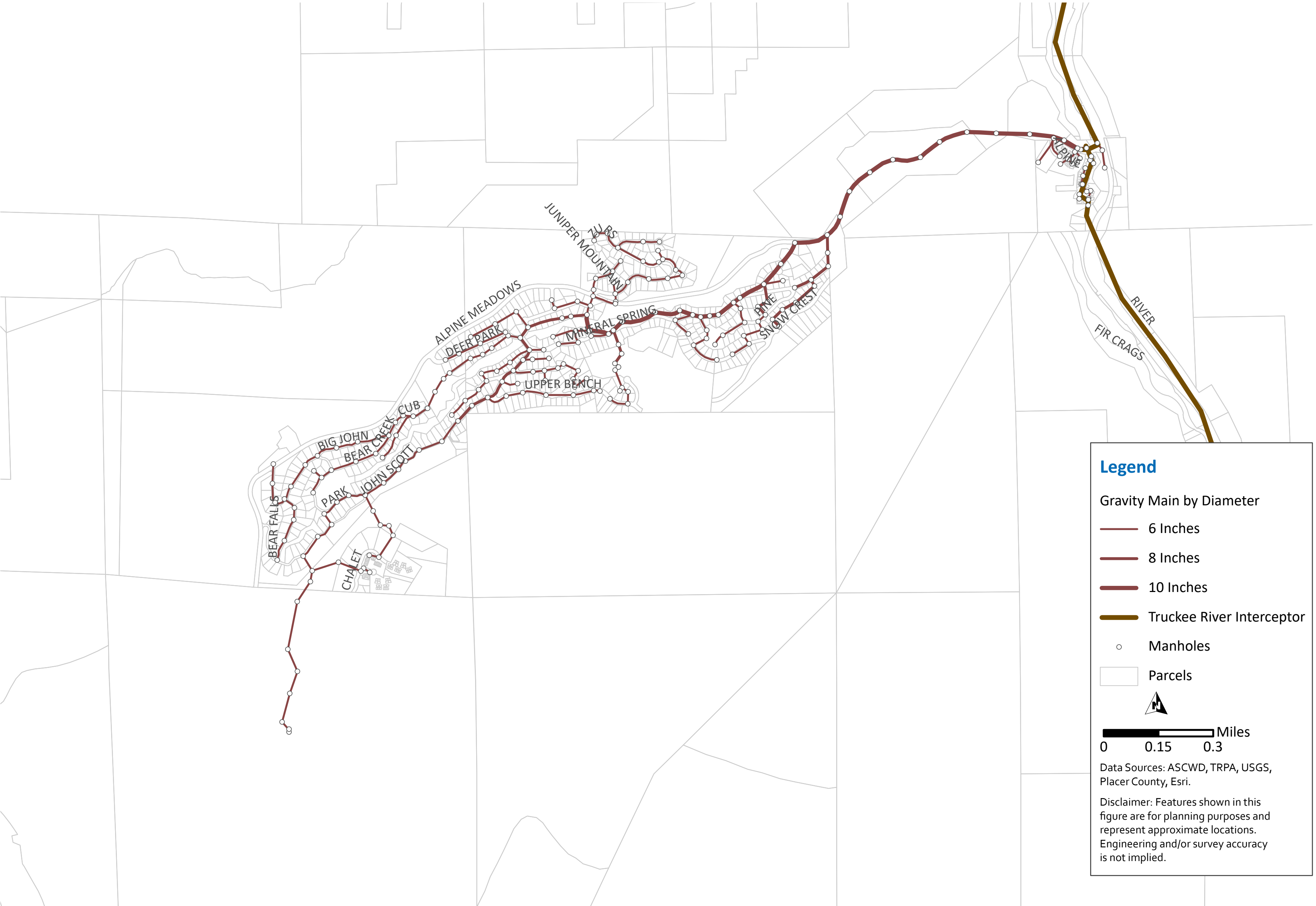


Figure 3.10 Existing Wastewater System

Table 3.9 Gravity Mains by Diameter

Diameter (inches)	Length (feet)	Length (miles)	Length (percent of system)
6	39,300	7.4	73%
8	6,100	1.2	11%
10	8,800	1.7	16%
Total	**54,200**	**10.3**	**100%**

Table 3.10 Existing Manhole Depths

Manhole Depth (feet)	Number of Manholes	Percent of Existing Manholes
< 3	0	0.0%
3 to 5	6	2.6%
5 to 10	216	93.5%
> 10	9	3.9%
Total	231	100%

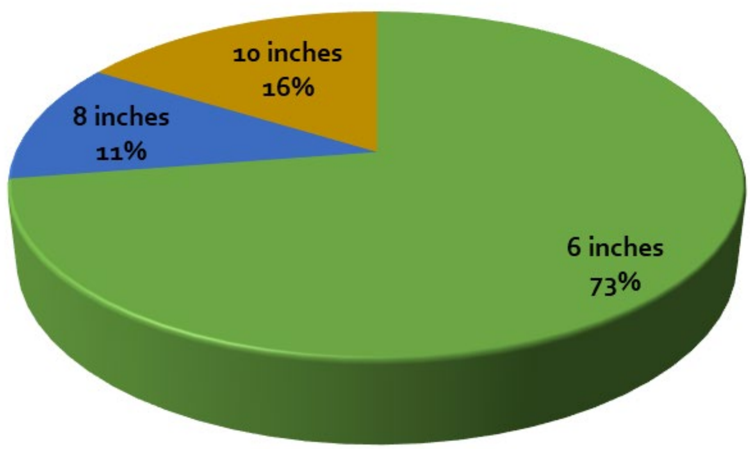
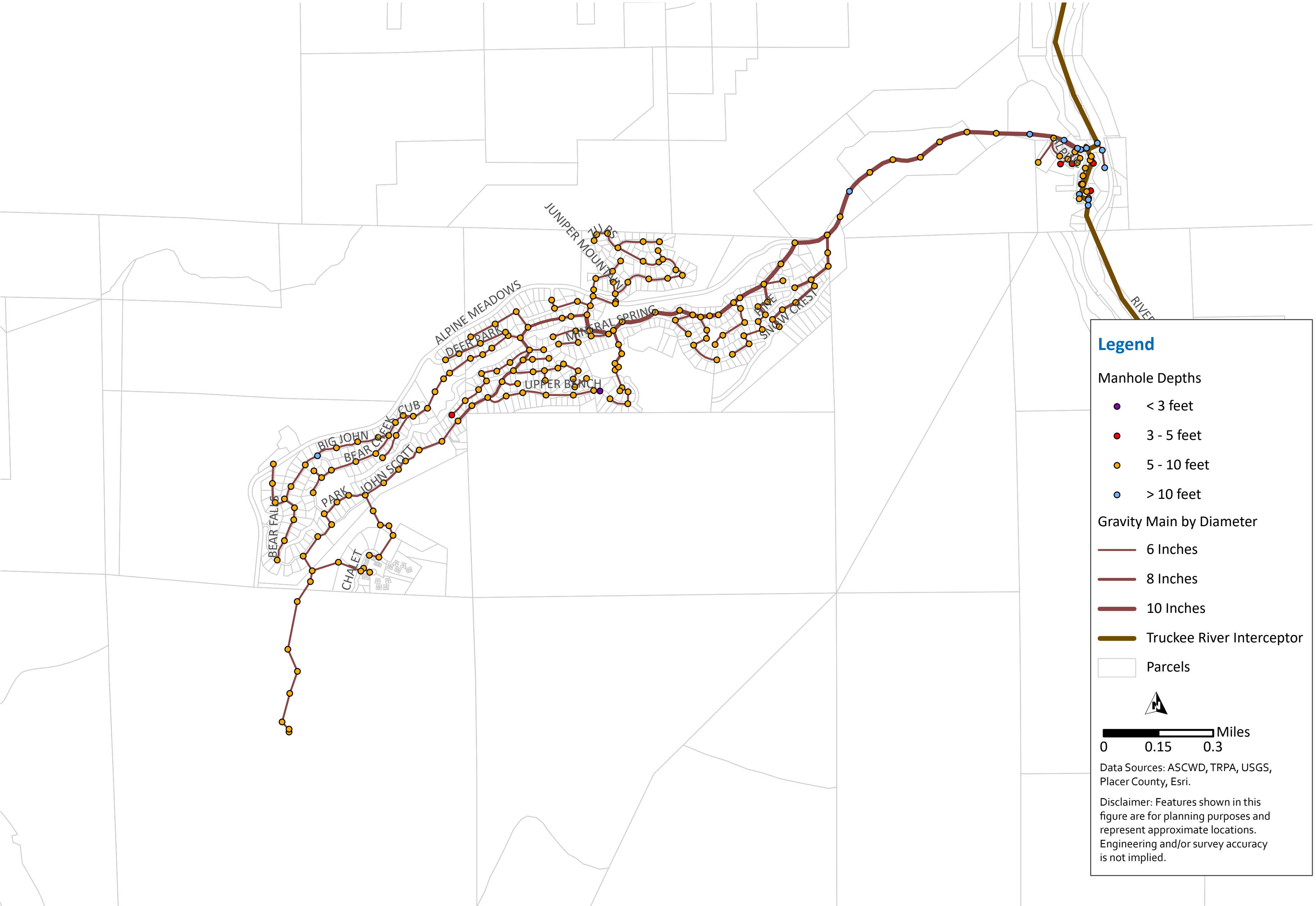


Figure 3.11 Existing Gravity Mains by Diameter


JUNIPER MOUNTAIN
ALPINE MEADOWS
DEER PARK
MINERAL SPRING
UPPER BENCH
CUB
BIG JOHN
BEAR CREEK
JOHN SCOTT
PARK
BEAR FALLS
CHALET
SNOW CREST
RIVER
Legend
Manhole Depths
< 3 feet
3 - 5 feet
5 - 10 feet
> 10 feet
Gravity Main by Diameter
6 Inches
8 Inches
10 Inches
Truckee River Interceptor
Parcels
N
Miles
0 0.15 0.3
Data Sources: ASCWD, TRPA, USGS, Placer County, Esri.
Disclaimer: Features shown in this figure are for planning purposes and represent approximate locations. Engineering and/or survey accuracy is not implied.

3.4 Wastewater Model Development

This section discusses the construction and calibration of ASCWD's wastewater collection system hydraulic model.

3.4.1 Wastewater Model Construction

ASCWD's wastewater collection system model was constructed in Bentley Systems, Inc.'s SewerGEMS platform and used record drawings as the primary source. Additional physical and operational information was determined from discussions with ASCWD staff.

3.4.1.1 Wastewater Model Elements

ASCWD's wastewater collection system model consists of ASCWD-owned manholes and gravity mains as well as T-TSA-owned collection system infrastructure. Table 3.11 describes the model elements that comprise the wastewater collection system model.

3.4.1.2 Wastewater Flows Allocation

Two types of wastewater flows are simulated in the ASCWD's SewerGEMS model:

1. Base dry weather flow (DWF). Base DWF consists of the sanitary flows and groundwater infiltration that enters the wastewater collection system under average dry weather conditions.
2. Rain-derived infiltration and inflow (RDI/I). RDI/I consists of additional I/I that enters the system during wet weather conditions.

Base DWF and RDI/I were allocated to model nodes in the ASCWD's SewerGEMS model as outlined below:

- **Step 1: Geolocation of water billing data.** ASCWD provided customer water billing data from 2018 through 2020.This data includes the total monthly water usage for each service connection and the address of the service connection. The average daily usage for each customer was calculated from the 2018 through 2020 data. The calculated base usage for each connection was then georeferenced using GIS techniques. Billing data described as "Irrigation" or "Pool" were not allocated in the SewerGEMS model.
- **Step 2: Catchments development.** RDI/I catchments were developed by drawing 25-foot buffers around each gravity main. A given catchment was assigned the nearest upstream manhole as its outfall node.
- **Step 3: Base flows allocation.** The total water usage for each model manhole was summed and entered into the "Sanitary Load Control Center" in the model. The base flows were adjusted during the DWF calibration. Figure 3.13 shows the diurnal pattern assigned to the base flows. This pattern was calculated using historical TRI flow data.
- **Step 4: RDI/I allocation.** The sewersheds were imported to the model as catchments. The SewerGEMS model calculates the RDI/I flows into each node using the catchment's area and RTK set, as well as the storm event pattern associated with the given scenario.

Table 3.11 Wastewater Collection System Model Elements

SewerGEMS Element	ASCWD Facilities	Input Parameters	Description

Manhole	• Sewer manholes.
	• Sewer cleanouts.
		• Diameter.
		• Invert elevation.
		• Rim elevation.
		• Ground elevation.
		• Inflows:
		  – Base flows.
		  – I/I.
		• Initial depth.
			Manholes are used to access the wastewater collection system for inspections, cleanings, and repairs. In general, all modeled manholes in the ASCWD's SewerGEMS model are assumed to be 3 feet in diameter and to not have a bolted cover.

Conduit	• Gravity mains.	• Diameter. • Length. • Material. • Friction factor (Manning's n). • Upstream invert elevation. • Downstream invert elevation. • Fill depth (sediment depth).	Conduits convey wastewater flows from a higher hydraulic grade line to a lower hydraulic grade line. Conduits can operate both under open-channel flow (non-pressurized) conditions and surcharged (pressurized) conditions. In ASCWD's SewerGEMS model, all conduits operate under non-pressurized conditions.
Catchment	• Sewersheds for wet weather I/I.	• Area. • Outflow element (typically a manhole). • Storm event pattern. • Runoff method (unit hydrograph, modified-rational, time-area, etc.). • Unit hydrograph method (RTK is used in this study). • RTK set.	Catchments are used in wet weather scenarios to simulate I/I into the collection system. Catchments were developed for ASCWD's model by creating 25-foot buffers around each gravity main.
Outfall	• Downstream TRI infrastructure.	• Boundary conditions. • Invert elevation. • Ground elevation.	Outfalls represent areas where flow leaves the system. ASCWD's sewer model uses an outfall to simulate downstream TRI infrastructure.

Notes:
(1) Abbreviations: I/I = infiltration and inflow.

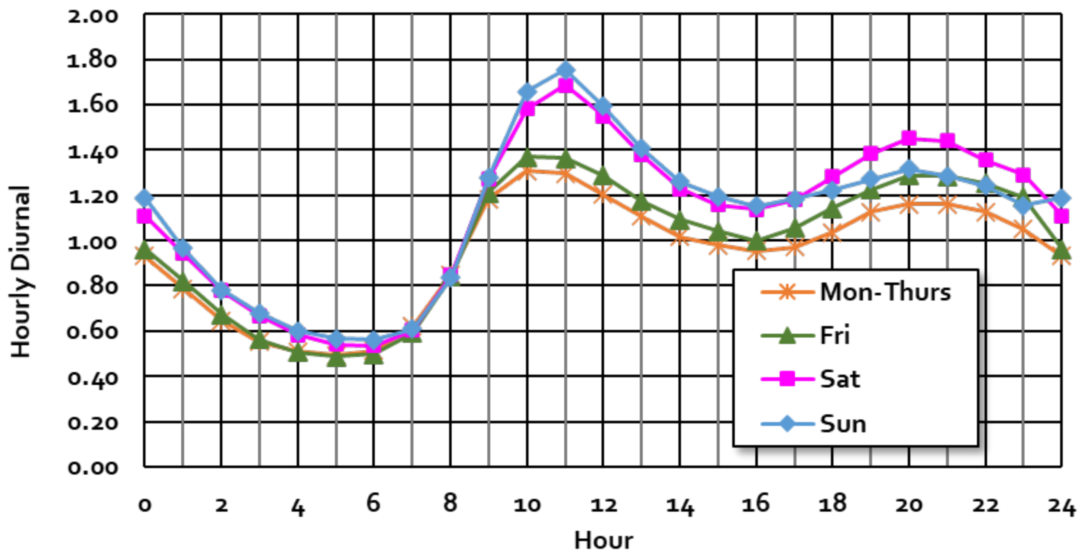


Figure 3.13 Wastewater Collection System Diurnal Pattern

3.4.2 Wastewater Model Calibration

ASCWD's SewerGEMS model was calibrated under dry and wet weather conditions using historical TRI data.

3.4.2.1 Calibration Standards

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

- **Dry weather calibration standards:** Dry weather calibration should be carried out for two dry weather days and the modeled flows and depths should be compared to the field measured flows and depths. Both the modeled and field measured flow hydrographs should closely follow each other in both shape and magnitude. In addition to the shape, the flow hydrographs should also meet the following criteria as a general guide:
  - The timing of flow peaks and troughs should be within 1 hour.
  - The peak flow rate should be within the range of ±10 percent.
  - The volume of flow (or the average rate of flow) should be within the range of ±10 percent. If applicable, care should be taken to exclude periods of missing or inaccurate data.
- **Wet weather calibration standards:** The model simulated flows should be compared to the field measured flows. The flow hydrographs for both events should closely follow each other in both shape and magnitude, until the flow has substantially returned to

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

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

3.4.2.2 Dry Weather Flow Calibration

The DWF calibration consisted of scaling the base wastewater flows from the raw water consumption data to equal the system's total average dry weather flow (ADWF) of 0.050 mgd. The raw water consumption data was scaled down to this value from the raw total water consumption of 0.072. The existing return-to-sewer ratio, or the ratio of ADWF to ADD, was calculated to be approximately 0.60.

3.4.2.3 Wet Weather Flow Calibration

The purpose of the wet weather flow (WWF) calibration is to verify that the hydraulic model can accurately simulate RDI/I into the collection system during a significant storm event. The WWF calibration process consisted of several steps, as outlined below:

- **Step 1: Identify calibration period.** A period from January 6, 2017, through March 3, 2017, was selected as the WWF calibration period since the highest recorded flow from the TRI flow meter occurred during this period. During this period, Alpine Meadows experienced two major rain-on-snow events that produced substantial wet weather flow responses within the collection system. Figure 3.14 shows ASCWD's wastewater flows measured at the TRI flow meter during the WWF calibration period. According to equivalent rainfall data from the Olympic Valley rain gauge, three events within the calibration period were greater than a one-year event, as shown on Figure 3.15. The February 7, 2017, event measured as approximately a 100-year, 6-hour event.

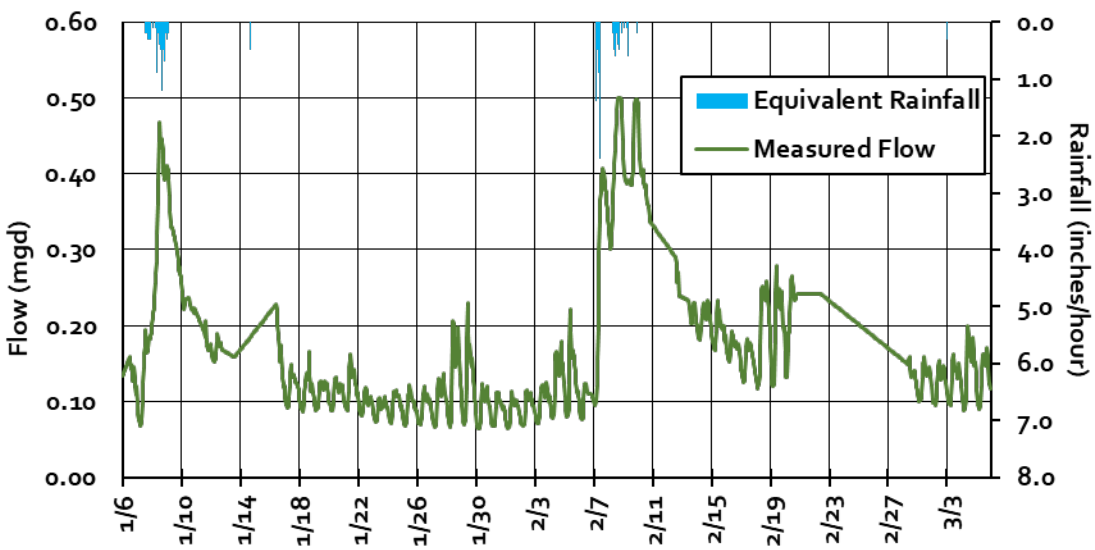


Figure 3.14 January through March 2017 Wet Weather Flow Calibration Period

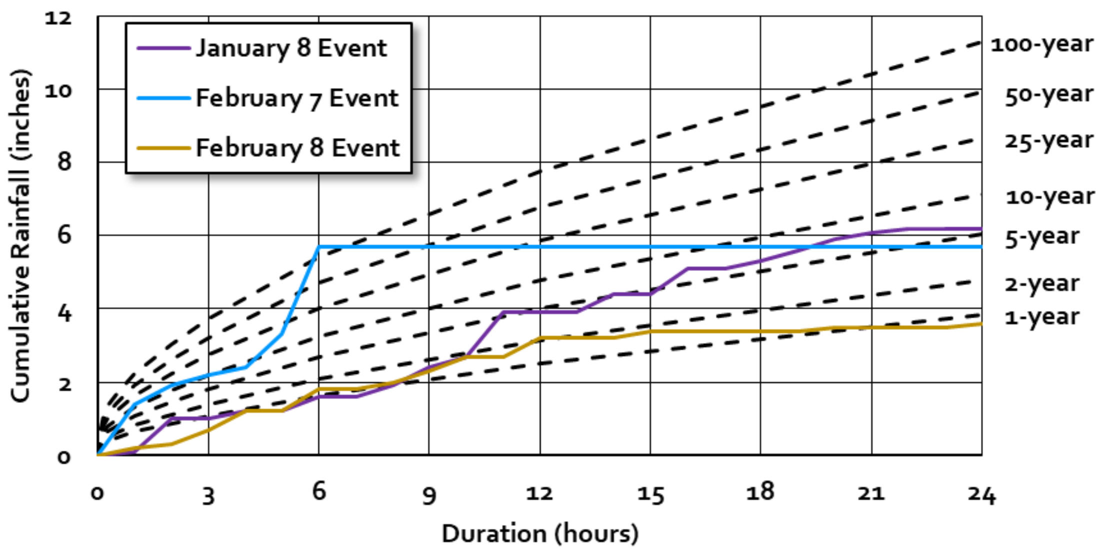


Figure 3.15 24-Hour Cumulative Rainfall for January and February 2017 Events

- **Step 2: Define RDI/I tributary areas.** For the WWF calibration, RDI/I flows are superimposed on top of the DWF. The model calculates RDI/I by assigning "RDI/I Inflows" to each node in the model. RDI/I inflows consist of both a unit hydrograph and the total area that is tributary to the model node. The RDI/I tributary areas were calculated in GIS using the sewersheds, or catchments. The tributary area provides a means to transform hourly rainfall depth from the rainfall hyetographs into a rainfall volume. The rainfall volume is transformed into actual RDI/I flows using the unit hydrograph, as described in the next step.
- **Step 3: Develop unit hydrograph to match field measured RDI/I responses.** A custom unit hydrograph was developed to simulate RDI/I responses from the collection system's tributary area using the RTK Method, which is widely used in collection system master planning. Using the RTK Method, the RDI/I unit hydrograph is the summation of three separate triangular hydrographs (short term, medium term, and long term), which are each defined by three parameters: R, T, and K. R represents the fraction of rainfall over the sewer basin that enters the collection system; T represents the time to peak of the of the hydrograph; and K represents the ratio of time to recession to the time to peak. In total, an RTK unit hydrograph consists of nine separate variables. An example RDI/I unit hydrograph using the RTK Method is shown on Figure 3.16.

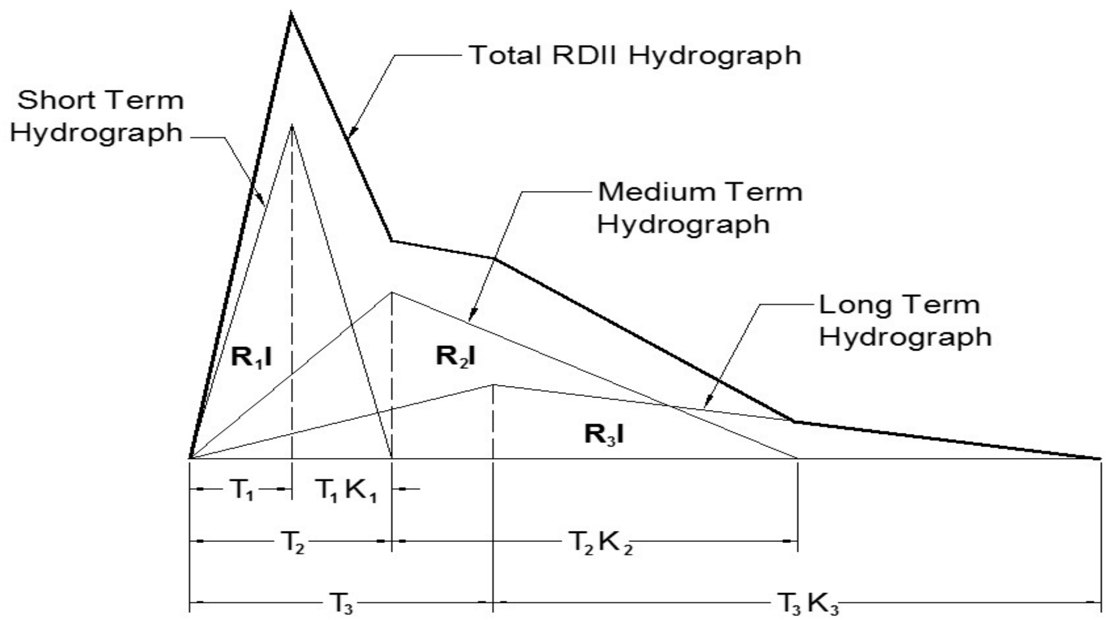


Figure 3.16 Example Rainfall Derived Infiltration and Inflow Unit Hydrograph Using RTK Method

The hydrograph utilizes the R-values (percent of rainfall that enters the collection system) calculated for each basin to simulate RDI/I. The nine variables in each unit hydrograph were initially set based on engineering judgment and then adjusted until the model simulated flows (both peak flows and average flows) matched closely with the field measured flows. Comparisons were made for average and peak flows as well as the temporal distribution of flow until flows returned to their baseline levels.

Figure 3.17 shows the measured and modeled wet weather flows at the TRI flow meter site. As shown, the modeled peak flows during the two main rain events generally match the measured peak flows. The model simulates a peak of about 0.70 mgd at the beginning of the February 7, 2017, storm that is not reflected in the measured data. This difference is likely due to differences between the actual rainfall in Alpine Meadows at this time compared to the measured rainfall at the rain gauge, which is located in Olympic Valley. The rain gauge measured a spike in rainfall intensity of almost 1.5 inches per hour; however, the TRI flow meter did not record a correlational increase in flow, suggesting that this extreme rainfall intensity spike did not occur within the service area. For the purposes of this study, the model accurately simulates the amount of I/I that enters the system during major storm events.

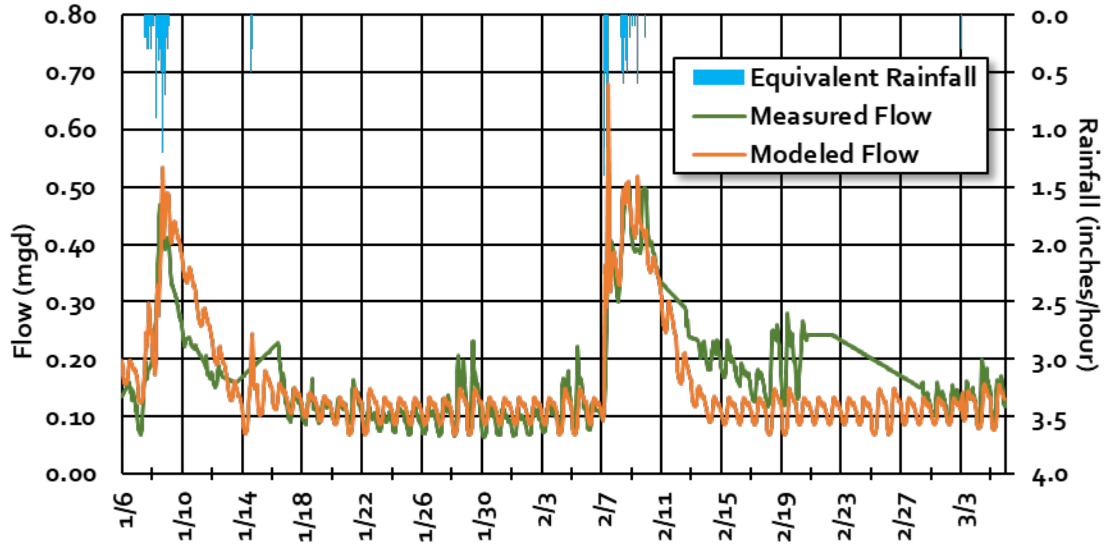


Figure 3.17 WWF Calibration Results

3.4.2.4 Wastewater Model Calibration Summary

The wastewater model calibration results demonstrate the model's ability to accurately predict the system's response to wet weather events. The wastewater model should be recalibrated to TRI flow meter data on a five- to ten-year basis or more frequently to verify that the model continues to provide reliable results from which to evaluate the wastewater collection system's hydraulic capacity.

Chapter 4

WATER DISTRIBUTION AND WASTEWATER COLLECTION SYSTEMS CONDITION ASSESSMENT

This chapter describes the condition of Alpine Springs County Water District's (ASCWD) water distribution and wastewater collection assets (facilities and pipelines) and the process used to perform the assessments. The intent of this chapter is to explain the current state of these assets and is separated into two condition assessment processes. The first half of the chapter covers the visual condition assessment of the facilities and equipment assets, while the second half covers the geographical information system (GIS) evaluation of the water and wastewater pipelines.

4.1 Above-Ground Assets Condition Assessments

A visual condition assessment was performed on the majority of ASCWD's above-ground assets. This section summarizes the methodology and results of the visual condition assessment that was conducted on July 26, 2022. This section also contains descriptions, observations, and recommendations for each of the assessed sites.

4.1.1 Above-Ground Condition Assessment Process

The site visit consisted of a visual condition assessment conducted by an engineering team accompanied by ASCWD operations staff. Throughout the assessment, the Carollo Engineers (Carollo) team asked questions of the ASCWD staff to capture anecdotal maintenance and performance history since maintenance records for individual assets were not available. This information was especially useful for assets that were not visible or readily accessible, such as storage tanks, valve boxes, and the underground portion of ASCWD's wells. The condition assessment also considered other sources of available information, such as hydraulic flow diagrams and as-built drawings, where available.

4.1.2 Condition Assessment Observations and Findings

The condition assessment team visited the vast majority of ASCWD's water system sites including a booster pump station, elevated storage tanks, a well, valve boxes, and a snowmaking facility. Source water springs were not visually assessed due to the constraints of time and difficulty of accessibility. The results of the condition assessment were used along with other sources of information including anecdotal information from ASCWD staff and planned capital improvement program (CIP) to determine the necessary rehabilitation and replacement timing and needs. No operation and historical maintenance information was available for individual assets; therefore, the input from ASCWD staff was relied upon to provide additional insight into condition.

The following sections describe the main findings from each site visited during the condition assessment. Additional information captured during the field assessment can be found within Appendix 4A, Site Visit Notes.

4.1.2.1 Pump Stations

The condition assessment team visited Juniper Mountain Pump Station (PS). The Juniper Mountain PS includes a single pump, associated valves, and electrical equipment contained within a concrete vault. The vault is accessible from the ground surface via a metal roof hatch, designed to protect the mechanical and electrical components from the elements.

In general, the assets at this site were found to be in fair to good condition. The pump showed typical wear for an asset of its type and age (installed 2005), but ASCWD staff indicated that the pump is continually under operation. There is no standby pump at Juniper Mountain PS. During fire hydrant tests, the check valve on the bypass pump locked open, and staff had to manually close it; slowly opening hydrant valves has helped prevent this valve from malfunctioning.

Recommendations in the following sections assume each tank will remain in service and continue to provide storage for operational, emergency, and fire reserve purposes. Chapter 8 further discusses alternative system improvements that may allow for the decommissioning of storage tanks.

4.1.2.2 Ground Storage Tanks

ASCWD's system includes five ground storage tanks (GSTs), which serve all pressure zones across the distribution system. The condition assessment focused on the exterior components of tanks, visible from the ground; the assessment team did not enter any of GST. Dive reports were not available. Videography was provided to the Carollo structural discipline lead, post field visit, to verify initial findings and confirm the assigned condition scores to each of the GSTs. Appendix 4B, Concrete Tank Condition Assessment, further discusses the condition assessment and recommendations for the four concrete tanks.

Tank 1

Tank 1 has a capacity of 110 thousand gallons and was constructed in 1963. Tank 1, along with Tank 4, serves Pressure Zone 1. While the internal condition of the GST could not be evaluated, review of the exterior concrete surface indicates overall very poor condition. Large cracks and active leaks were observed on the surface of tank. The retaining wall surrounding the site is severely eroded and in need of replacement. It is recommended that Tank 1 be fully evaluated for conformance to current applicable codes and then either rehabilitated or replaced, depending on the results of the evaluation.

Tank 2

Tank 2 has a capacity of 100 thousand gallons and was constructed in 1963. Tank 2 serves Pressure Zone 2. While the internal condition of the GST could not be evaluated, review of the exterior concrete surface indicates overall very poor condition. Medium to large cracks were observed on the surface of tank, along with active leaks. The altitude valve A-1, which feeds the tank, was recently replaced and is in very good condition. It is recommended that Tank 2 be fully evaluated for conformance to current applicable codes and then either rehabilitated or replaced, depending on the results of the evaluation.

Tank 3

Tank 3 has a capacity of 100 thousand gallons and was constructed in 1963. Tank 3 serves Pressure Zone 3. While the internal condition of the GST could not be evaluated, review of the exterior concrete surface indicates overall very poor condition. Medium to large cracks were observed on the surface of tank, along with active leaks. The transducer and pipes below the tank were indicated to be significantly deteriorated and failing. It is recommended that Tank 3 be fully evaluated for conformance to current applicable codes and then either rehabilitated or replaced, depending on the results of the evaluation.

Tank 4

Tank 4 has a capacity of 910 thousand gallons and was constructed in 2019. Tank 4, along with Tank 1, serves Pressure Zone 1. While the internal condition of the GST could not be evaluated, review of the welded steel surface indicates overall good condition. Despite the relatively young age of the GST, moderate rusting is already apparent on the welds. ASCWD staff suspects that tank sealing/coating was not performed to specifications.

Tank 5

Tank 5 has a capacity of 110 thousand gallons and was constructed in 1963. Tank 5 serves Pressure Zone 4. While the internal condition of the GST could not be evaluated, review of the exterior concrete surface indicates overall very poor condition. Medium to large cracks were observed on the surface of the tank, along with active leaks. It is recommended that Tank 5 be fully evaluated for conformance to current applicable codes and then either rehabilitated or replaced, depending on the results of the evaluation.

4.1.2.3 Wells

As discussed in Chapter 3, ASWCD owns three groundwater wells: Alpine Meadows Estates Well Number 1 (AMEW #1), R1, and R2. The following sections discuss each well.

AMEW #1

The AMEW #1 is a single electric-powered groundwater production well for domestic use. The site assessment focused only on the above-ground assets at the well site. The condition scores assigned to the well were formulated from information provided by staff, the time since the well's last major rehabilitation, the design and current production capacity of the well, and other operational information.

The AMEW #1 was constructed in 2015. AMEW # 1 is located in Pressure Zone 3 but has enough supply to support the entire ASCWD water distribution system. If AMEW # 1 was utilized to supply the entire system, booster pumps would be required to pump water to Zones 1 and 2. The well is approximately 525 feet deep and remains full of water, with little to no drawdown exhibited during pumping.

The well pump delivers excessive head beyond its original intended design, which has created excess strain and premature degradation of assets at the site. High pressures are likely contributing to the cracking in the pavement around the well along with equipment failure. The variable frequency drives (VFDs) on the well pumps have been inoperable since they were installed. When the well is running, the pumps operate at 220 gallons per minute (gpm). Operations staff noted that this site is only used during abnormally high demand or operational flushing twice monthly to remove rust buildup. Operations staff noted that this well overflows in the winter, which causes the area around the well building to become icy and causes safety issues for both staff and members of the public.

Assets at this site generally range from fair to very poor condition.

R1 Well

The R1 well is used for snowmaking purposes and typically does not supply the distribution system but could potentially augment supply to Zone 4 if needed. This asset was not examined as part of this condition assessment. According to input from District staff, the well and well pump are in good condition.

R2 Well

The R2 well was decommissioned in 2011 due to elevated manganese concentrations and was not assessed as part of this study. Future studies may evaluate the feasibility of blending R2 groundwater with water from other supply sources to produce acceptable water quality. The well's assets should be assessed at that time.

Bypass

A bypass was constructed between Zones 1 and 2 in 2017 and consists of 6-inch diameter piping and two control valves. One valve allows a small amount of flow from Zone 1 to Zone 2 under typical daily operations, and the other larger valve acts as an emergency interconnection and is typically closed. The bypass runs from Chalet Road to John Scott Trail along a gravel maintenance road.

4.1.2.4 Springs

ASCWD's system includes four springs which provide the bulk of supply. Springs 1, 2, and 4 are all located in Zone 1, and Spring 3 is located in Zone 3. The springs were not visually inspected as part of this assessment due to accessibility issues. However, ASCWD staff provided recent photos of the spring casings, along with anecdotal information regarding the assets' condition, for Carollo to review.

According to ASCWD staff, the springs generally operate without any issues and deliver consistent flows. Staff noted that Spring 1's casing leaks, and ASCWD has not been able to address the leak due to accessibility issues; the spring is located on United States Forest Service (USFS) land, and the District would need to construct a road to enable access for repairs. According to District staff, the leak has not substantially reduced supply from the spring.

4.2 Pipeline Condition and Remaining Life Assessment

A condition assessment was performed on the pipeline assets using ASCWD's GIS records. No site visits or visual condition assessment were performed. This section summarizes the methodology and results of the pipeline condition assessment.

4.2.1 Condition Assessment Process and Scoring

ASCWD's GIS data served as the basis for the condition assessment of the pipeline assets. The GIS data contained information about each pipe segment, including their location, which was used to estimate the condition and remaining life of each segment.

The GIS data was imported into a GIS-based modeling program, Innovyze® InfoAsset™ Planner, for evaluation. Additional information was loaded into the model to assist in the evaluation of the pipelines. This information included closed-circuit television (CCTV) inspection data and water pressure at various points in the system. Separate models were set up for water and wastewater pipe evaluations. The models evaluated each segment of pipe against the criteria shown in Table 4.1.

Table 4.1 Pipeline Condition Criteria

Criteria	Water Pipeline Criteria	Sewer Pipeline Criteria
Operational Data	N/A	**CCTV Inspection Data** - The quick score from CCTV data were used to reduce the remaining asset life as follows: • Condition 5 = minimum of 1 grade-5 defect • Condition 4 = minimum of 1 grade-4 defect. • Condition 3 = minimum of 1 grade-3 defect. • Condition 2 = minimum of 1 grade-2 defect. • Condition 1 = minimum of 1 grade-1 defect or the presence of no defects.
Age	**Age** - the age of the pipelines was compared to the useful life estimate shown in Table 4.3 and remaining life ranges in Table 4.4.	**Age** - the age of the pipelines was compared to the useful life estimate shown in Table 4.3 and remaining life ranges in Table 4.4.

The models assigned a condition score to each of the pipe segments based on the worst result from any of these criteria. The condition score was used to determine how much remaining life the pipeline had left using the ranges shown in Table 4.2.

Table 4.2 Pipeline Condition Scoring Descriptions

Condition Score	Remaining Life Range
1 (Best)	More than 50 years
2	31 - 50 years
3	16 - 30 years
4	6 - 15 years
5 (Worst)	5 years or less

Notes:
(1) Remaining life range is an estimate based on typical service life and criteria shown in Table 4.3.

4.2.1.1 CCTV Data Summary

ASCWD performs regular National Association of Sewer Service Companies (NASSCO) standardized Pipeline Assessment and Certification Program (PACP) CCTV inspections of their wastewater collection pipelines. Observations captured by the inspection crews are for each inspection and stored within a database format, which were linked back to their respective host pipelines where possible.

Due to recent efforts undertaken, within the scope of the broader master planning effort to hydraulically model the sewer system, buried sewer infrastructure has been digitized into GIS compatible format. Standardized pipe naming convention has supplanted older legacy identifiers and historical information. Consequently, some of the PACP inspections could not be matched back to their respective pipes and therefore lack CCTV-derived condition information despite an inspection previously being conducted.

ASCWD provided CCTV inspection data for 2018 to 2020. This data was loaded into InfoAsset™ Planner software and linked to the appropriate pipe segments.

Of the 10.5 miles of wastewater pipelines, just over half (5.4 miles, 52 percent) were linked to a CCTV inspection record. ASCWD has inspected the entire system using CCTV; however, errors in the pipeline naming captured within inspections can cause compatibility issues when trying to link the data together. These compatibility issues were exacerbated by the digitization effort described above. ASCWD is continuously collecting more CCTV data, which can be used in future evaluations of the system. The map in Figure 4.1 shows the pipelines with CCTV inspection data.

The data from each inspection was analyzed for specific defects and summarized into a single digit "peak score." The peak quick score represents the highest severity defect found on the pipe. CCTV defects are graded on a one to five scale with one being the best and five being the worst. Examples of the worst rated defects found in the CCTV data include:

- Hole/ Hole Soil Visible.
- Infiltration Runner.
- Water Level Sags.
- Roots Ball/ Medium.

The peak score for each pipe was used to estimate the remaining life for the pipeline.

4.2.1.2 Useful Life Assumptions

The original useful life is the estimated amount of time from when the pipeline was installed to when it needs to be replaced. The remaining useful life of each asset was evaluated based on the original useful life for each type of asset and the asset's age. The criteria in Table 4.1 further adjust the useful life and remaining useful life for each pipe segment. The original useful lives shown in Table 4.3 were developed during a workshop with ASCWD staff. The lives are estimated based on ASCWD staff experience and knowledge of ASCWD's pipeline assets.

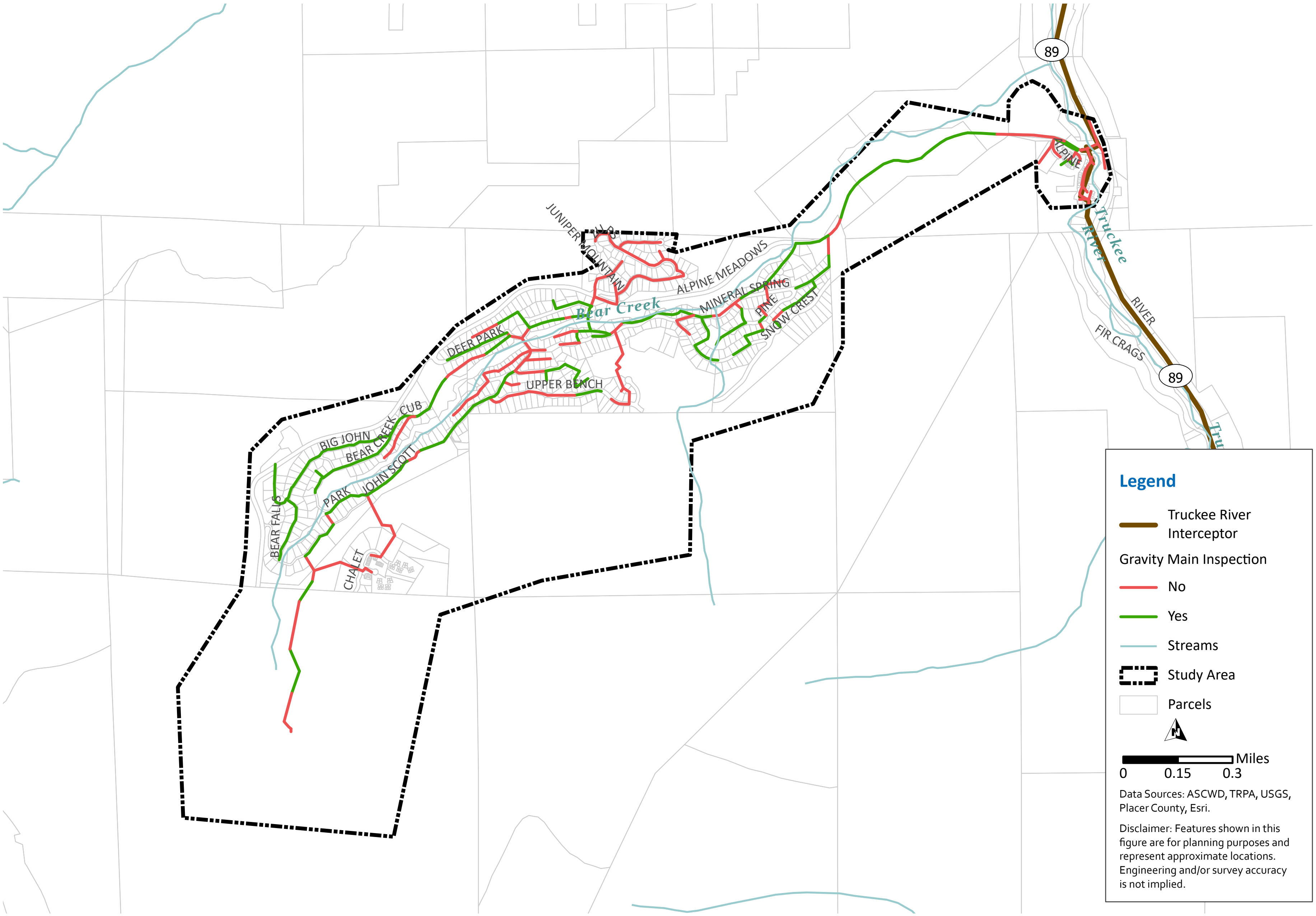


Figure 4.1 CCTV Inspection Data Map

Table 4.3 Pipeline Remaining Useful Life Assumptions

Asset Type	Original Useful Life (Years)(1)	Length of Pipe (miles)	Percentage of Length
Water Distribution Pipes		**16.8**	
Asbestos Cement (ACP)	85	16.8	100
Wastewater Collection Pipes		**10.5**	
Asbestos Cement (ACP)	85	10.2	97
Ductile Iron (DIP)	85	< 1	<1
Polypropylene (PP)	75	< 1	<1
Polyvinyl Chloride (PVC)	70	< 1	<1
Vitrified Clay (VCP)	75	< 1	1
Other Pipeline Asset Types			
Manholes	75	238 assets	N/A

Notes:
(1) Useful life estimates based on industry benchmarks, regional data, and on ASCWD input.
(2) Asset register information based off digitized data sets developed during the project.

The condition of the manholes was based solely on the age of the assets and the remaining life estimate ranges in Table 4.2.

4.2.2 Condition Assessment and Remaining Useful Life Evaluation

The results of the pipeline condition assessment are shown in Table 4.4, Figure 4.2, and Figure 4.3. The table shows the percentage of the assets that fall into each condition score and remaining life range.

Table 4.4 Pipeline System Condition and Remaining Life Results

Condition Score(1)	Condition 1 (> 50 years)	Condition 2 (31-50 years)	Condition 3 (16-30 years)	Condition 4 (6-15 years)	Condition 5 (≤ 5 years)
Water Pipelines	0% (0 miles)	0% (0 miles)	100% (16.8 miles)	0% (0 miles)	0% (0 miles)
Wastewater Pipelines	0% (0 miles)	0% (0 miles)	95% (10.1 miles)	4% (0.4 miles)	<1% (<0.1 miles)
Manholes	0%	0%	100%	0%	0%

Notes:
(1) Remaining life ranges per Table 4.3.
(2) All assets modeled (water and wastewater) did not have available installation year data. The assumption was made that an installation year of 1965 would be used.
(3) All assets modeled (water and wastewater) did not have available material data. For the wastewater system, pipe inspection data was used to infer material type where possible. When pipes had unknown pipe material, it was assumed to be asbestos cement.

Overall, the pipeline systems are in good condition, with the exception of a few wastewater pipelines that have significant defects which will need to be addressed in the near- to mid-term.

4.2.3 Proposed Improvements

Pipeline deficiencies were identified for ASCWD's gravity collection system exclusively. Repair and replacement (R&R) strategies used to address these gravity pipe failures are broadly classified into those requiring excavation (external) or trenchless (internal). Figure 4.4 depicts the R&R strategies recommended to treat defects and failures that are structural or operations and maintenance (O&M) in nature.

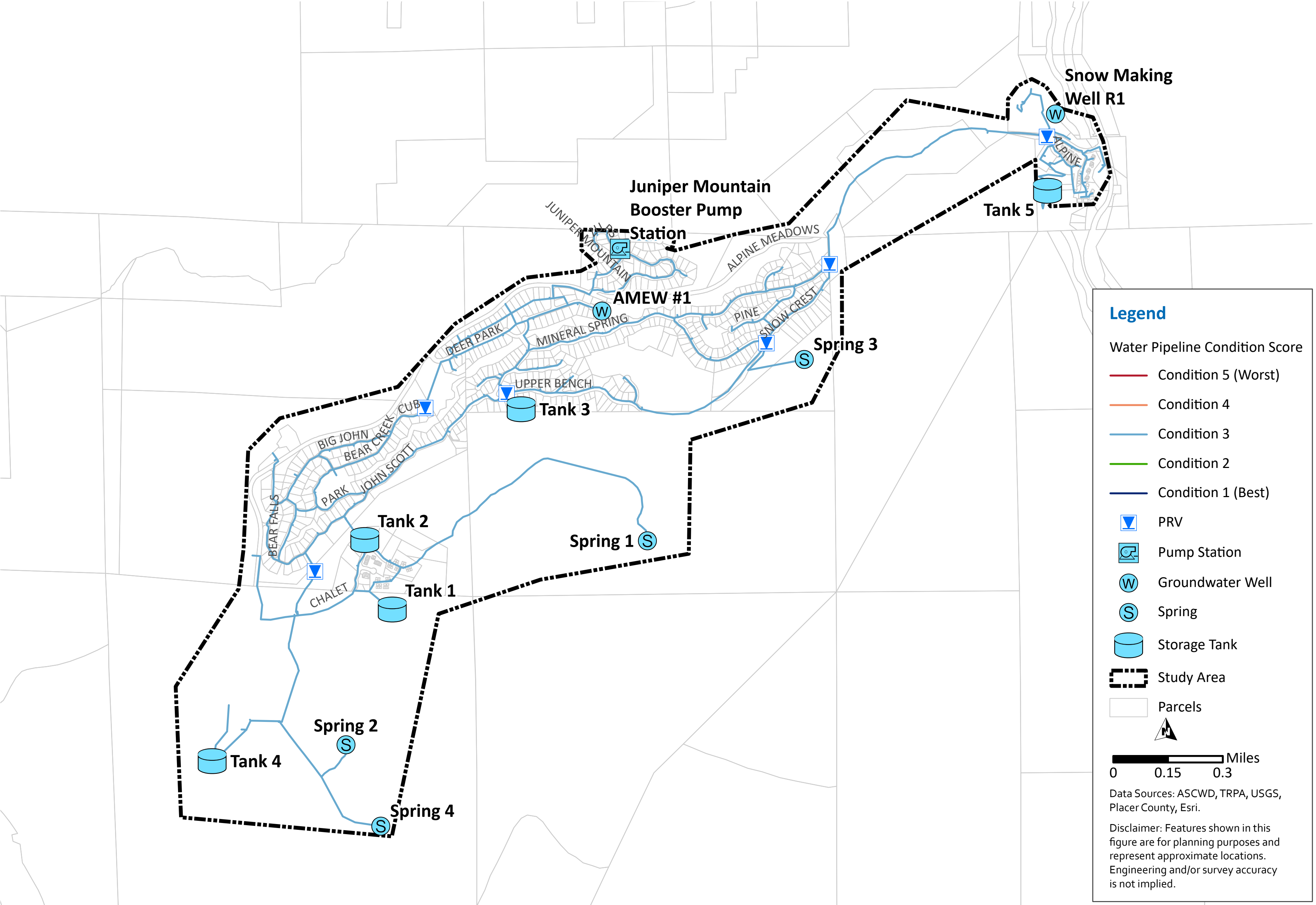


Figure 4.2 Water Pipeline Condition Map

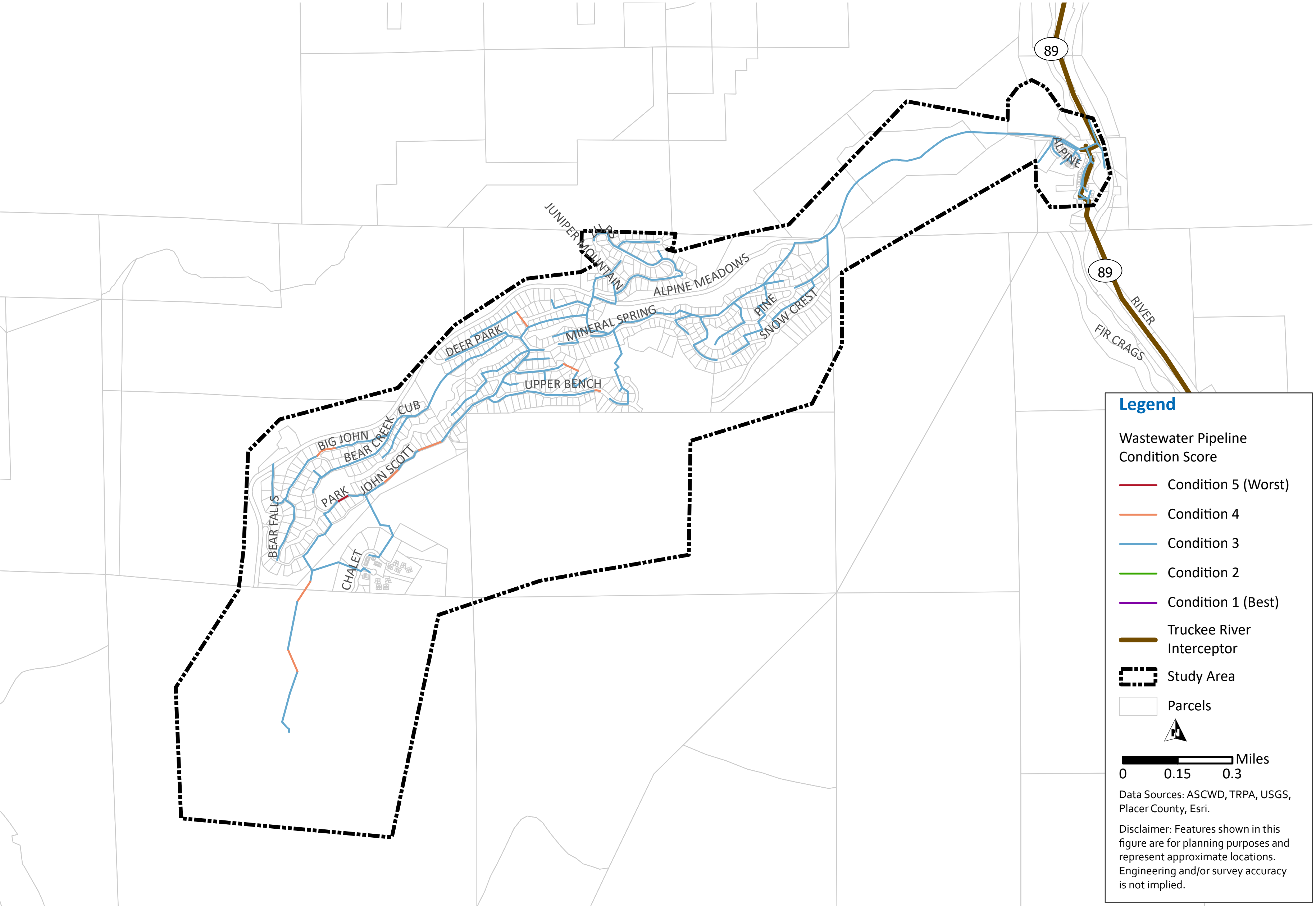
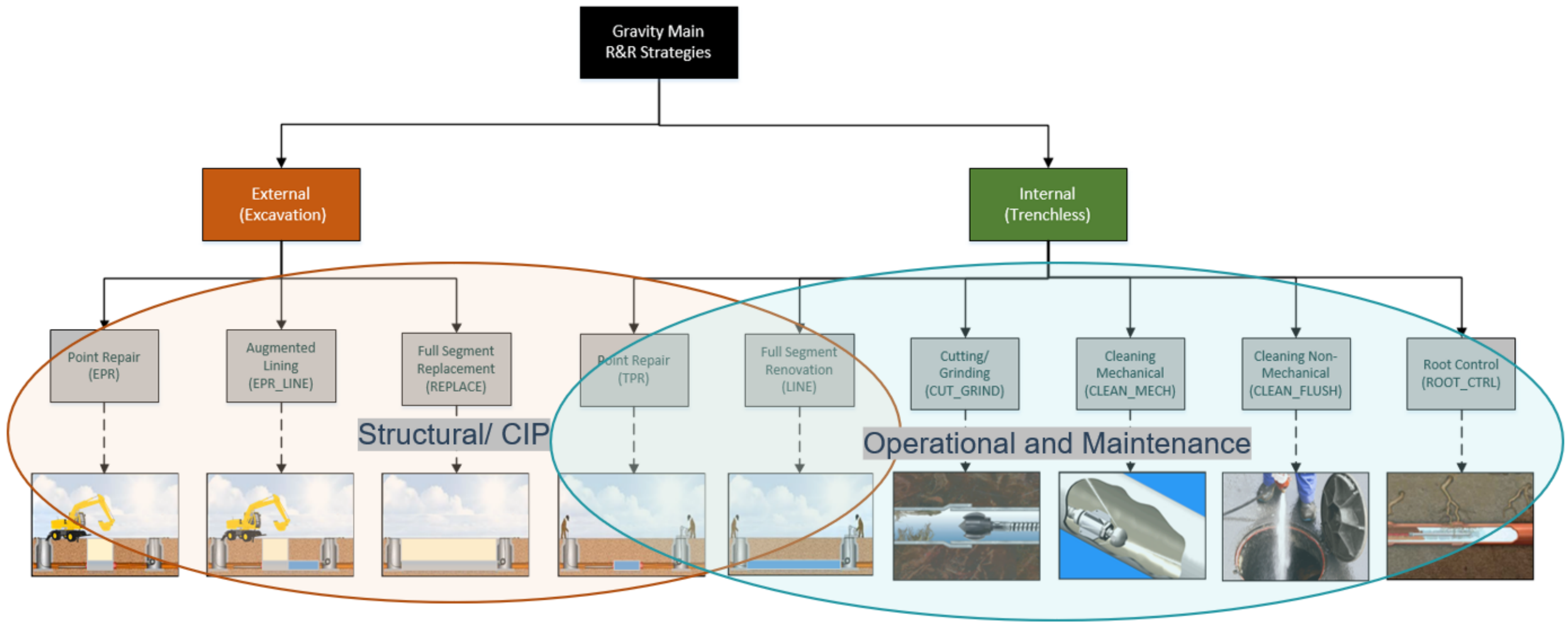


Figure 4.3 Wastewater Pipeline Condition Map



Note: *Cleaning Non-Mechanical (CLEAN_FLUSH) is performed in conjunction w/ trenchless point repairs (TPR), full segment renovation (LINE), and augmented lining (EPR_LINE).*

Figure 4.4 Gravity Main R&R Strategies

Gravity main R&R strategies are defined below with the extent of the host pipe addressed, the type of improvement, and technologies available. Buffer widths describe the distance upstream and downstream from a given defect which the repair strategy extends, whereas merge widths describes the maximum allowable distance between the same type of repair for which they will not be consolidated and treated as single repair.

- External (Excavation):
  - ***Point Repair (EPR).*** Describes replacement performed through excavation activities when a section of the host pipe is removed and substituted with a compatible section of new pipe. This strategy is often used when a trenchless technique would otherwise not suffice.
    - *Extent*: partial pipe; 5-foot (ft) buffer width, 10-ft merge width.
    - *Type**:*** capital improvement.
    - *Technologies:* N/A.
  - ***Full Segment Replacement (REPLACE).*** Describes replacement of the entire pipe segment from structure to structure (i.e., manhole to manhole) performed through excavation. The host pipe is removed and substituted with a compatible new pipe. This strategy is often used where a trenchless technique would otherwise not suffice. This strategy is also frequently employed when hydraulic deficiencies are identified, and the replacement pipe is increased in diameter for additional flow volume.
    - *Pipe Extent*: whole pipe segment.
    - *Type**:*** capital improvement.
    - *Technologies:* N/A.
  - ***Augment Lining (EPR_LINE.)*** Describes a strategy which employs at least one or more external point repairs followed by a lining activity.
    - *Pipe Extent*: whole pipe segment.
    - *Type**:*** capital improvement.
    - *Technologies*: see both EPR and LINE.
- Internal (Trenchless):
  - ***Point Repair (TPR).*** Describes a rehabilitation performed through trenchless activities in which a discrete section of the host pipe is rehabilitated. This strategy is often used when an external point repair is not necessary, and TPR is economically more feasible and easier to perform.
    - *Pipe Extent*: partial pipe; 5-ft buffer width, 10-ft merge width.
    - *Improvement Type**:*** capital improvement or O&M.
    - *Technologies:* reinforced shotcrete, injection grouting, corrosion projection grouting, centrifugally cast concrete, cast-in-place concrete, etc.
  - ***Full Segment Renovation (LINE).*** Describes a strategy which is installed continuously from one from structure to structure within a host pipe. Linings provide structural renewal of the pipe barrel, improve the performance of the existing sewer, and are appropriate for various pipe sizes and shapes.
    - *Pipe Extent*: whole pipe segment.
    - *Type**:*** capital improvement.
    - *Technologies*: cured-in-place pipe (CIPP), sliplining, spiral wound pipe, fold and form pipe, etc.

- ***Cutting and Grinding (CUT_GRIND).*** Describes a strategy used for intrusions that stick out into the cross-section profile of the host pipe. A common example of this type of defect is intruding taps.
  - *Pipe Extent*: partial pipe; 5-ft buffer width, 10-ft merge width.
  - *Type:* O&M or construction.
  - *Technologies*: rodding w/ specialized adaptor.
- ***Cleaning Mechanical (CLEAN_MECH).*** Describes a strategy used to removed obstructions from the pipe which cannot be removed only by hydraulic methods.
  - *Pipe Extent*: whole pipe segment.
  - *Type:* O&M.
  - *Technologies*: rodding, snaking, bucket machine, cable machine, etc.
- ***Cleaning Non-Mechanical (CLEAN_FLUSH).*** Describes a strategy used to remove debris or generally clean a host pipe.
  - *Pipe Extent*: whole pipe segment.
  - *Type:* O&M.
  - *Technologies*: flushing, jetting, etc.
- ***Root Control (ROOT_CTRL).*** Describes the initial strategy used to address root defects within a host pipe through chemical means. This strategy alone may not be sufficient to address more sever root defects.
  - *Pipe Extent*: whole pipe segment.
  - *Type:* O&M.
  - *Technologies*: chemical treatment, etc.

R&R strategies employed can be either standalone or combined depending on the specific condition recorded for the pipe. It is common practice to perform non-mechanical cleaning prior to capital rehabilitation methods to prepare the host pipe for rehabilitation or repair.

Table 4.5 summarizes high-level cost estimates required to rehabilitate the wastewater collection system pipes according to the condition assessment findings. These costs assume typical conditions and do not account for specific site and market factors associated with ASCWD's assets. Additional rehabilitation recommendations and corresponding cost estimates are discussed in Chapters 8 and 9. Figure 4.5 provides a summary of the rehabilitation methods recommended for ASCWD's wastewater collection system.

Table 4.5 Recommended Rehabilitation Methods

Improvement Type	Rehab Method	Total Pipe Length (ft)	Plan Cost(1)
Structural	LINE	283	$29,723
	TPR	544	$7,380
Operational	CLEAN_FLUSH	27,809	$250,284
	ROOT_CTRL	637	$210,764
Grand Total		29,273	$498,151

Notes:
(1) Plan cost represents 2022 dollars and assumes material cost and labor only.

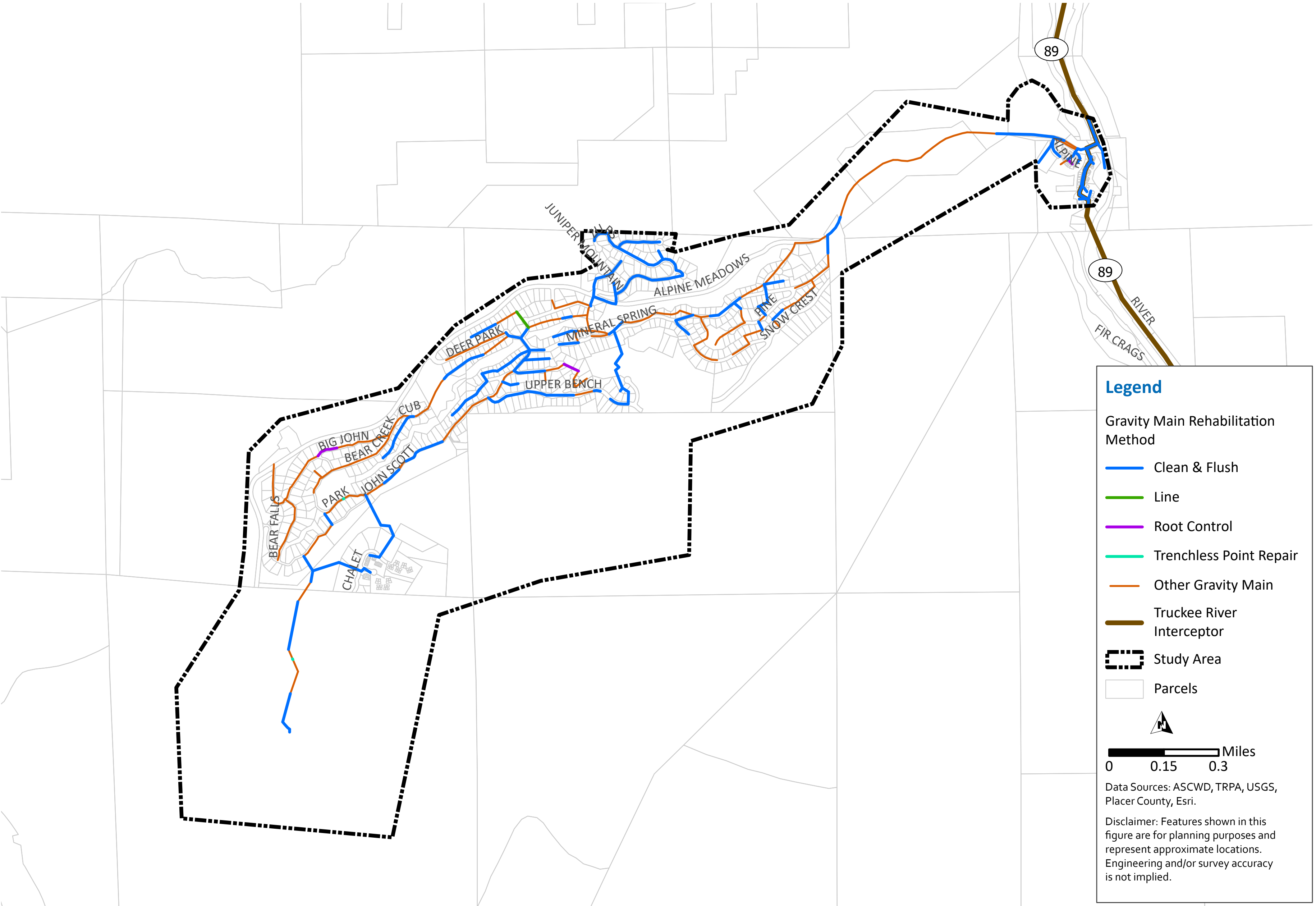


Figure 4.5 Wastewater Repair and Rehabilitation Map

4.3 Asset Management Program Recommendations

This condition assessment represents the current condition of ASCWD's assets. To better understand changes in condition over time, ASCWD should continue monitoring and assessing the water and wastewater infrastructure. The following actions are proposed to further improve ASCWD's Asset Management Program.

- Update and maintain GIS asset data, including key attribute information (material, diameter, and installation date). The condition assessment analysis performed in this chapter is only as good as the data which feed it and there were numerous assumptions used to fill data gaps, which limits the overall utility and reliability of the results.
- Develop and implement a formal Condition Assessment Protocol (CAP). Moving forward, ASCWD staff should implement a protocol to rate and record the condition of the assets on a regular basis. As ASCWD staff visit each site for operations and maintenance activities, they should be able to collect information and store it in a computerized maintenance management system (CMMS) for use in future planning efforts.
- Consider procuring a CMMS or developing a formal tracking process as part of a Work Order and Maintenance Program. The CMMS can be used to schedule and record work orders for the assets. A formal program would allow ASCWD to track what work is being done and store that information in the CMMS. Historical work order information can be used for various analyses, including asset lifecycles and rehabilitation and replacement cost estimating.
- Establish key performance indicators (KPIs) and performance metrics. ASCWD may already have some metrics related to overall financial performance; however, establishing asset-level KPIs and metrics can improve overall operations and maintenance performance. Asset performance metrics can be used in addition to physical condition to evaluate asset condition and likelihood of failure.

Chapter 5

PLANNING CRITERIA

This chapter presents the planning criteria that were used to evaluate Alpine Springs County Water District's (ASCWD) existing water and wastewater systems and to develop future water and wastewater systems infrastructure.

5.1 Water System Planning Criteria

This section presents the planning criteria and methodologies for evaluating ASCWD's existing water distribution system and for sizing future improvements. The planning criteria address the water supply capacity, fire flow criteria, storage capacity, acceptable service pressures, distribution main performance, and water efficiency. The criteria are separated into the following categories:

- Water supply capacity.
- Fire flow criteria.
- Water storage requirements.
- Service pressures.
- Distribution mains.
- Water efficiency criteria.

The planning criteria for each of these categories are discussed in the following sections.

5.1.1 Water Supply Capacity

In accordance with industry standard practices, as well as the California Department of Public Health's (CDPH) 2008 Water Works Standards criteria for "New and Existing Source Capacity," the water system's water source shall have the capacity to meet the system's maximum day demand (MDD). Demands in excess of the MDD required for peak hour demand (PHD) or for fire flows are planned to come from storage.

5.1.2 Fire Flow Criteria

Fire flows stress a water system in the area of the fire and often identify existing deficiencies. The deficiencies are generally associated with pipe size (i.e., diameter) or age (i.e., roughness) that results in high head loss and lower pressures. The fire flow criteria measure a system's ability to deliver high flows while maintaining a minimum pressure.

To evaluate the effect of fire flows throughout the distribution system, large point demands are applied at fire hydrants. The fire flow demands are run concurrent with the MDD. Simulating MDD plus fire flows also demonstrates the performance of supply sources, booster pumps, and storage tanks operating under the upper limit high demand conditions.

The fire flow criteria for this study were developed in coordination with ASCWD staff and the North Tahoe Fire Protection District (NTFPD) and are meant to provide sufficient levels of conservatism to meet California Fire Code (CFC) requirements. The following summarizes the fire flow criteria by land use:

- Residential: 1,500 gallons per minute (gpm) for a duration of 2 hours.
- Commercial: 1,750 gpm for a duration of 2 hours.
- Structures with sprinklers: 500 gpm.

The fire flow criteria used for this study are typical flows per land use type and are thus appropriate for this planning level criteria. Specific fire flow requirements for individual building sites may vary depending on specific occupancy use, square footage, building height, and construction type. This Master Plan assumes that all required fire flows in excess of 1,750 gpm would be met through private onsite water supplies or supplemental storage. This approach is consistent with industry standard practice.

5.1.3 Water Storage Requirements

The principal function of storage is to provide a reserve supply of water for: 1) operational equalization, 2) fire reserve, and 3) emergency needs. Operational storage is directly related to the amount of water necessary to meet peak demands. The intent of operational storage is to provide the difference in quantity between the customer's peak demands and the system's reliable available supply. Fire storage reserves are the amount of stored water required to meet the necessary fire flow demands. The volume of water allocated for emergency uses is determined from the historical record of emergencies, the amount of time expected to lapse before a hypothetical emergency can be corrected, and the system's level of supply and storage redundancy.

5.1.3.1 Operational Storage

Operational storage is the desirable amount of stored water in a system to regulate fluctuations in demand so that extreme variations will not be imposed on the source of supply. Operational storage typically serves the peak demands exerted within the MDD. With operational storage, system pressures are improved and stabilized to better serve customers throughout the service area.

Operational storage is commonly estimated to be between 10 and 50 percent of the MDD. The American Water Works Association (AWWA) Manual on Distribution Network Analysis of Water Utilities (M-32) states that operational storage is typically between 10 to 15 percent of the MDD for large systems but could exceed 30 percent for small systems or arid climates. An operational equalization storage equal to 25 percent of ASCWD's MDD is recommended for this planning effort.

5.1.3.2 Fire Storage

Fire storage is the amount of stored water required to meet the necessary fire flow demands. The fire storage volume is determined by multiplying the highest required fire flow by its corresponding duration. For systems with multiple pressure zones, fire storage is determined using the largest required fire flow volume per pressure zone.

ASCWD's recommended fire flows and durations were developed from Carollo Engineers (Carollo) experience on similar projects. Table 5.1 lists the recommended fire flows and durations by pressure zone. This volume will be reviewed in greater detail during design of future storage infrastructure and in consideration of CFC requirements.

Table 5.1 Fire Flow Requirements by Pressure Zone

Pressure Zone	Required Fire Flow (gpm)	Required Fire Flow Duration (hours)	Required Fire Flow Volume (MG)
Zone 1	1,750	2	0.21
Zone 2	1,500	2	0.18
Zone 3	1,500	2	0.18
Zone 3 Boosted	1,500	2	0.18
Zone 3 Lower	1,500	2	0.18
Zone 4	1,750	2	0.21

Notes:
(1) Abbreviations: gpm = gallons per minute; MG = million gallons.

5.1.3.3 Emergency Storage

Emergency storage is the volume recommended to meet demands during emergency situations such as pipeline failures, major distribution main failures, pump failures, electrical power outages, and natural disasters. The amount of emergency storage included within a water distribution system is an owner option, based on an assessment of risk, the desired degree of system dependability, economic considerations, and water quality concerns. Emergency storage criteria are typically expressed as a multiplier of the MDD and can range from 0 to 100 percent or more of the MDD.

An emergency storage volume equal to 100 percent of ASCWD's MDD was selected for this Master Plan. When designing individual storage improvements, it is at the District's discretion to increase or decrease the emergency storage volume according to specific project factors, such as water age concerns.

5.1.3.4 Total Storage

The total storage requirements are the sum of the operational, fire reserve, and emergency storage volumes. At this planning level stage, these amounts provide for the general scale of storage infrastructure warranted for the system. During detailed design of future facilities, these figures may need to be adjusted depending on variables such as:

- Firm source capacity.
- Existing and future built environment.
- CFC requirements.

Table 5.2 summarizes the water system's storage requirements by pressure zone. Chapter 6, Water System Evaluation, evaluates these requirements against the storage available to each pressure zone.

Table 5.2 Storage Requirements by Pressure Zone

Pressure Zone	Existing and 2045 Required Operational Storage(1) (MG)	Required Fire Storage(1) (MG)	Existing and 2045 Required Emergency Storage (MG)	Existing and 2045 Required Total Storage(1) (MG)
Zone 1	0.01, 0.04	0.21	0.03, 0.15	0.25, 0.40
Zone 2	0.02, 0.02	0.18	0.07, 0.09	0.27, 0.29
Zone 3	0.03, 0.04	0.18	0.13, 0.14	0.35, 0.36
Zone 3 Boosted	<0.01, <0.01	0.18	0.01, 0.01	0.19, 0.19
Zone 3 Lower	<0.01, <0.01	0.18	<0.01, <0.01	0.18, 0.18
Zone 4	0.01, 0.01	0.21	0.04, 0.05	0.27, 0.27

Notes:
(1) Values shown correspond to existing and 2045 storage requirements, respectively.

5.1.4 Service Pressures

Pressures maintained within the distribution system vary depending on distribution system operations and pressure zone topography. It is essential that the water pressure in a consumer's residence or place of business be neither too high nor too low. Low pressures cause flow reductions when multiple water-using appliances are being operated simultaneously. High pressures may cause faucets to leak and valve seats to wear out quickly. Additionally, high service pressures often result in wasted water and elevated water utility bills. In areas where water pressures exceed 80 pounds per square inch (psi), service connections can be provided with pressure-reducing valves (PRVs).

The AWWA M-32 indicates that pressures between 30 psi and 90 psi are generally expected during the range of system water demands. For the purposes of this Master Plan, service pressure criteria were developed for various demand conditions, as summarized below:

- Average Day Demand (ADD): Maximum service pressures of 80 psi are recommended during ADD conditions. It is recommended that ASCWD install a PRV on laterals with pressures that exceed 80 psi during typical ADD conditions.
- PHD: To provide adequate service pressures, it is recommended that ASCWD maintains a minimum service pressure of 35 psi during typical PHD conditions.
- MDD + Fire Flow: This pressure criterion is related to fire flows and was devised to ensure adequate positive pressures during a fire. It is recommended that ASCWD fire pressure criterion requires a minimum acceptable residual pressure of 20 psi at the connecting hydrant.

5.1.5 Distribution Mains

Distribution mains are generally sized to carry the greater of the PHD or the MDD plus fire flow. Other criteria related to distribution piping include maximum and minimum velocities and maximum allowable friction losses.

High velocities may cause damage to the pipes and to their appurtenances. Normally, velocities of 10 feet per second (fps) (AWWA M-32), or higher, do not cause ill effects if they occur for a limited duration. It is normally good practice to limit pipe velocities to no more than 8 fps on a continuous basis. For ASCWD, a maximum pipe velocity of 6 fps is recommended for existing distribution mains.

New distribution mains less than 16 inches in diameter should be sized for a maximum pipeline velocity of 5 fps, while new distribution/transmission water mains 16 inches in diameter or more should be sized for a maximum pipeline velocity of 4 fps.

Provided that the maximum velocity criteria and the pressure criteria are not exceeded, high pipeline head loss by itself is not a controlling factor. However, it may be an indication that the pipe is nearing the limit of its carrying capacity and may not have sufficient capacity to perform under stringent conditions. Good practice dictates monitoring pipes that have a head loss in excess of 10 feet (ft) per 1,000 ft (AWWA M-32). A maximum head loss of 10 ft per 1,000 ft is recommended for ASCWD.

5.1.6 Water Efficiency Criteria

Water loss in distribution systems leads to increased energy needs, wasted resources, and ultimately lost revenues. ASCWD aims to maintain an average water loss of less than 10 percent of the water system's total ADD.

5.2 Wastewater System Planning Criteria

This section presents the planning criteria and methodologies for evaluating ASCWD's existing wastewater collection system and for sizing future improvements. The planning criteria address the collection system capacity, acceptable gravity sewer pipeline slopes, maximum allowable depth of flow, and changes in pipe size. The criteria are separated into the following categories:

- Gravity sewer mains criteria.
- Design storm for sewer system planning.

The planning criteria for each of these categories are discussed in the following sections.

5.2.1 Gravity Sewer Mains Criteria

Gravity sewer mains criteria were developed to evaluate the capacity of existing gravity mains and to design new gravity mains. Gravity sewer main capacities are dependent on pipe roughness, the maximum allowable depth of flow downstream, and limiting velocity and slope. Criteria for minimum gravity sewer main slopes are designed to maintain mean pipeline velocities of no less than 2 fps when flowing at 50 percent full. Criteria were developed for sewer gravity mains between 8 and 42 inches in diameter assuming a Manning's roughness of 0.013. Table 5.3 lists the minimum gravity sewer main slopes by diameter.

Table 5.3 Minimum Gravity Sewer Main Slopes

Diameter (inches)	Minimum Slope (ft/100 ft)
8	0.400
10	0.280
12	0.240
15	0.120
18	0.108
21	0.088
24	0.068
27	0.060
30	0.052
33	0.044
36	0.040
42	0.032

The peak flow depth criterion is the primary criterion used to identify gravity sewer main capacity deficiencies and to size new sewer improvements. This criterion is defined as the maximum ratio between the normal depth of flow to the diameter of the gravity sewer main (d/D). Design d/D ratios typically range from 0.5 to 0.92[1]. More conservative values are typically used for smaller pipes, which may experience flow peaks greater than design flow and generally have higher rates of blockages from debris, paper, or rags.

Table 5.4 summarizes the planning and evaluation criteria for the City's gravity sewer mains. A less conservative maximum d/D ratio was selected to evaluate existing gravity mains compared to new gravity mains because it is more cost-effective to design new, larger pipes than it is to replace existing pipes with minor deficiencies.

Table 5.4 Planning and Evaluation Criteria for Gravity Sewer Mains

Parameter	Criteria
Minimum Pipe Size	8 inches
Minimum Pipe Slopes	See Table 5.3
Maximum d/D Ratio for Existing Gravity Mains	0.92 (Full Flow)
Maximum d/D Ratio for New Gravity Mains	0.50

[1] A d/D ratio of 0.92 is considered "full flow" for gravity mains.

5.2.2 Design Storm for Sewer System Planning

Design storms are storm events used to analyze the performance of a collection system under extreme wet weather events and consist of a distribution, recurrence interval, and duration. ASCWD's wastewater system largest historical wet weather responses have resulted from rain-on-snow events, which are events during which rainfall occurs when the ground is covered by a large snowpack. Therefore, this study assumes the design storm will occur when a large snowpack is present.

California industry standard is to use a design storm with a 10-year recurrence interval and a 24-hour duration for analyzing wastewater collection system performance under peak wet weather flow (PWWF) conditions. The National Oceanic and Atmospheric Administration Atlas 14 defines Alpine Meadow's 10-year, 24-hour design storm volume to be 7.03 inches[2]. ASCWD's design storm distribution was developed using rainfall data from early January 2017, which represents the most significant rain-on-snow event in the Alpine Meadows area in recent years.

Figure 5.1 shows the 10-year, 24-hour design storm. This design storm was routed through the collection system model to evaluate the collection system under PWWF conditions, as discussed in Chapter 7, Wastewater System Evaluation.

[2] https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html

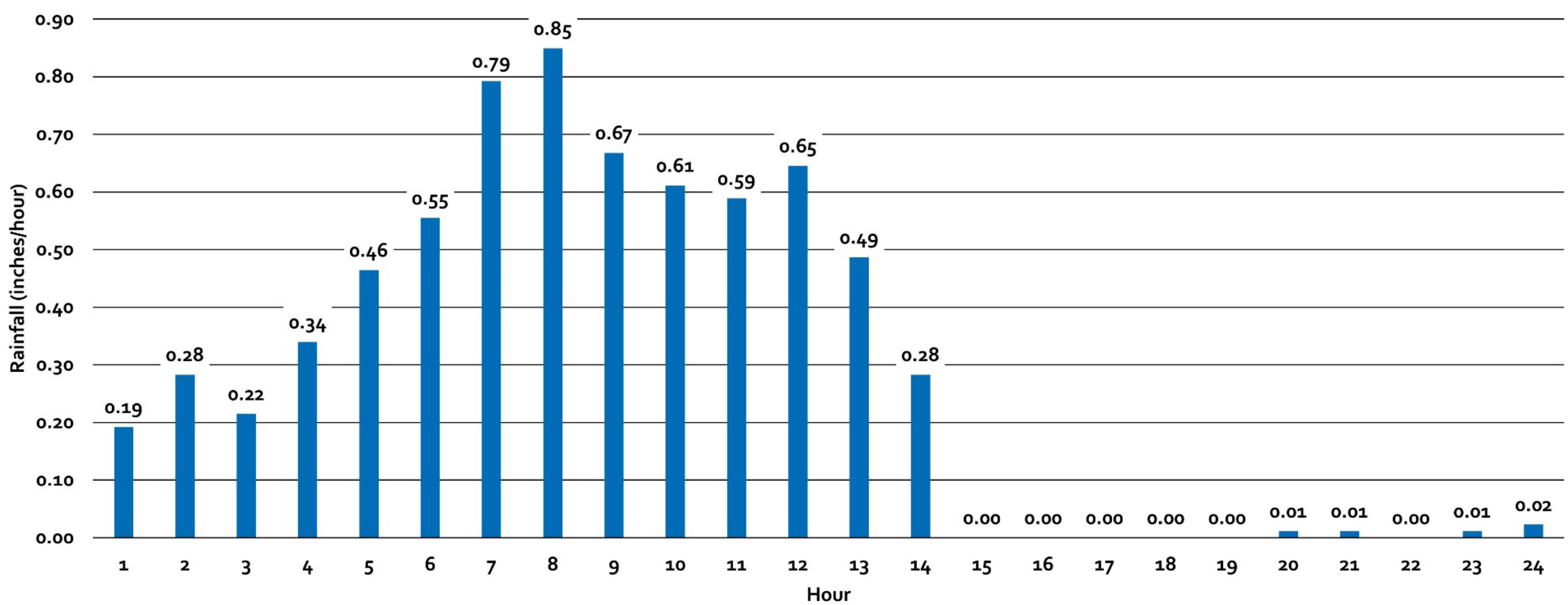


Figure 5.1 10-Year, 24-Hour Design Storm

Chapter 6

WATER SYSTEM EVALUATION

This chapter discusses the water system hydraulic evaluation performed for Alpine Springs County Water District (ASCWD). The supply and storage analyses are presented along with the water system hydraulic model results. The hydraulic results discussed in this chapter were used to prioritize system replacement, as discussed in Chapter 8.

6.1 Supply Evaluation

ASCWD's supply sources were evaluated against the supply criteria defined in Chapter 5. As discussed in Chapter 5, ASCWD supply sources should be sufficient to supply the maximum day demand (MDD) within each pressure zone. Booster pump stations should have sufficient firm capacity to supply the peak hour demand (PHD) in the boosted pressure zone.

Table 6.1 shows the supply evaluation results by pressure zone under existing and 2045 demand conditions. The analysis found that the system has supply deficiencies under existing and 2045 demand conditions due to lack of standby pumping capacity at the Alpine Meadows Estates Well (AMEW) Number 1. Adding a standby pump at this facility would mitigate existing and projected supply deficiencies.

Table 6.1 Supply Evaluation

Pressure Zone	Required Supply(1) (gpm)		Supply Capacity(2) (gpm)		Supply Surplus/ (Deficit) (gpm)	
	Existing	2045(3)	Existing	2045	Existing	2045
Zone 1	23.5	103.4	178.0	178.0	154.5	74.6
Zone 2	51.1	60.1	154.5	74.6	103.4	14.5
Zone 3	92.5	97.5	117.4	28.5	24.9	(69.0)
Zone 3 Boosted	12.0	12.2	24.9(4)	(55.0)(4)	12.9	(67.2)
Zone 3 Lower	1.7	1.7	16.9	(77.1)	15.1	(78.9)
Zone 4	30.7	32.6	15.1	(78.9)	(15.6)	(111.5)

Notes:
(1) Required supply is equal to the maximum day demand within the given pressure zone. For pressure zones supplied by booster pump stations (e.g., Zone 3 Boosted), required supply is equal to the peak hour demand.
(2) Available supply capacity is equal to the cumulative upstream firm supply capacity minus the cumulative upstream maximum day demand. Negative numbers indicate that available supply is fully consumed by upstream demands prior to reaching the given pressure zone.
(3) Added supply requirements per pressure zone to account for added demands between existing and 2045 were allocated to each pressure zone according to the planned developments and developable parcel acreage within the given pressure zone.
(4) Zone 3 Boosted supply capacity is equal to the smaller of the Juniper Mountain booster pump station firm capacity and the available supply capacity from Zone 3 minus Zone 3's maximum day demand.
(5) Abbreviations: gpm = gallons per minute.

This supply analysis assumes stable groundwater supplies that are not being depleted over time. According to historical flow records, ASCWD's springs have maintained consistent flows since ASCWD began keeping records in the early 2000s, indicating that groundwater supplies are stable. A more detailed supply source analysis can help verify that groundwater levels within the service area are not decreasing. Chapter 8 discusses potential monitoring activities ASCWD could implement to further evaluate groundwater supplies.

6.2 Storage Capacity Evaluation

The storage capacity evaluation evaluated the existing storage capacity within each pressure zone against the storage criteria defined in Chapter 5. The existing storage capacity within each pressure zone was determined using the following process:

1. The available storage within each existing tank was determined by calculating the volume with the tank filled to the maximum operating level (MOL). The MOL was assumed to be the height of the tank minus a 4-foot freeboard.
2. The storage required within each zone for operational purposes was calculated by multiplying the given zone's MDD by 25 percent.
3. The storage required within each zone for firefighting purposes was calculated by multiplying the highest required fire flow in the given zone by its corresponding duration.
4. The storage required within each zone for emergency purposes was calculated by multiplying the given zone's MDD by 100 percent.
5. The storage available to each zone was calculated by subtracting the total upstream required operational storage from the total upstream storage volume. For example, Zone 3 Lower's available storage capacity was calculated by subtracting the required operational storage in Zones 1, 2, 3, and 3 Boosted from the available storage from Tanks 1, 4, 2, and 3.

Table 6.2 shows the storage evaluation results under existing and 2045 demand conditions. The evaluation found that the existing system has sufficient storage capacity.

Table 6.2 Storage Evaluation

Pressure Zone	Required Storage(1) (MG)		Storage Capacity(2) (MG)		Storage Surplus/ (Deficit) (MG)	
	Existing	2045 (3)	Existing	2045	Existing	2045
Zone 1	0.25	0.40	1.02	1.02	0.77	0.62
Zone 2	0.27	0.29	1.11	1.08	0.84	0.79
Zone 3	0.35	0.36	1.19	1.15	0.84	0.80
Zone 3 Boosted	0.19	0.19	1.25	1.21	1.05	1.02
Zone 3 Lower	0.18	0.18	1.15	1.11	0.97	0.93
Zone 4	0.27	0.27	1.26	1.23	0.99	0.96

Notes:
(1) Required storage is equal to the total operational, fire reserve, and emergency storage requirements within the given pressure zone.
(2) Available storage capacity is equal to the cumulative upstream storage capacity minus the cumulative upstream operational storage requirement.
(3) Added storage requirements per pressure zone to account for added demands between existing and 2045 were allocated to each pressure zone according to the planned developments and developable parcel acreage within the given pressure zone.
(4) Abbreviations: MG = million gallons.

6.3 Distribution System Hydraulic Performance Evaluation

The calibrated water model was used to evaluate water distribution system hydraulic performance under existing and projected 2045 conditions. The following sections discuss hydraulic model results for existing and future scenarios in relation to the hydraulic performance criteria defined in Chapter 5.

6.3.1 Existing System Hydraulic Performance

The existing system hydraulic performance was evaluated against the pressure, velocity, and fire flow criteria defined in Chapter 5. The model pressure results discussed in the following sections refer to modeled junctions that represent potential service connections. Model junctions within facilities, such as pump stations, were excluded from this analysis since water system facilities are typically designed to withstand higher pressures and velocities than domestic appurtenances.

6.3.1.1 Existing Average Day Demand Results

The existing water system was modeled under average day demand (ADD) conditions to identify areas with high pressures. As noted in Chapter 5, high pressures can lead to greater water loss and can cause system appurtenances to degrade more quickly. Maintaining a maximum pressure of no more than 80 pounds per square inch (psi) can help mitigate these issues.

Figure 6.1 shows the existing ADD maximum pressure results. According to the model results, multiple areas within ASCWD's water system experience pressures greater than 80 psi under existing ADD conditions. These areas consist of the lower elevation regions within each pressure zone.

6.3.1.2 Existing Maximum Day Demand Results

ASCWD's water system was modeled under existing MDD conditions to identify areas with pressure below the target minimum pressure of 35 psi. Figure 6.2 shows the modeled minimum pressures and maximum pipeline velocities under existing MDD conditions.

According to the model results, all model junctions have minimum pressures greater than 35 psi under this scenario. Similarly, the model did not identify any water mains with maximum velocities greater than 6 feet per second (fps); maximum pipeline velocities are generally less than 2 fps, with some pipes experiencing velocities between 2 and 4 fps.

Although the model did not identify any existing MDD deficiencies, a few high-elevation areas have modeled minimum pressures below 50 psi. Due to potential discrepancies between modeled and actual elevations, minimum pressures in these regions may drop close to 35 psi under high demand conditions. The following high-elevation areas have modeled minimum pressures between 35 and 50 psi under existing MDD conditions:

- The western portion of Zone 2 along John Scott Trail and at Alpine Vista Road.
- The Juniper Mountain area, both in Zone 3 and Zone 3 Boosted.

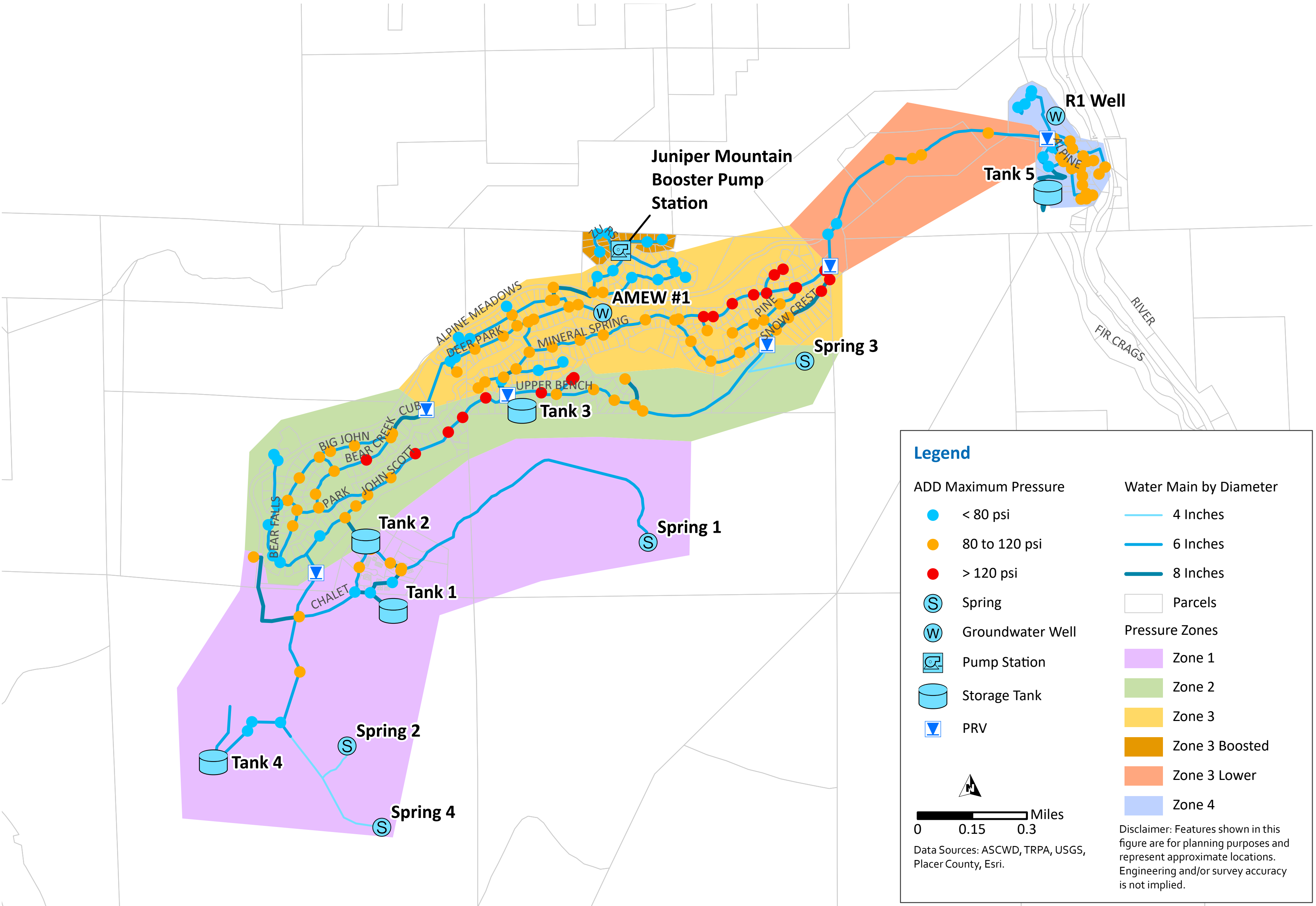


Figure 6.1 Existing Average Day Demand Maximum Pressures

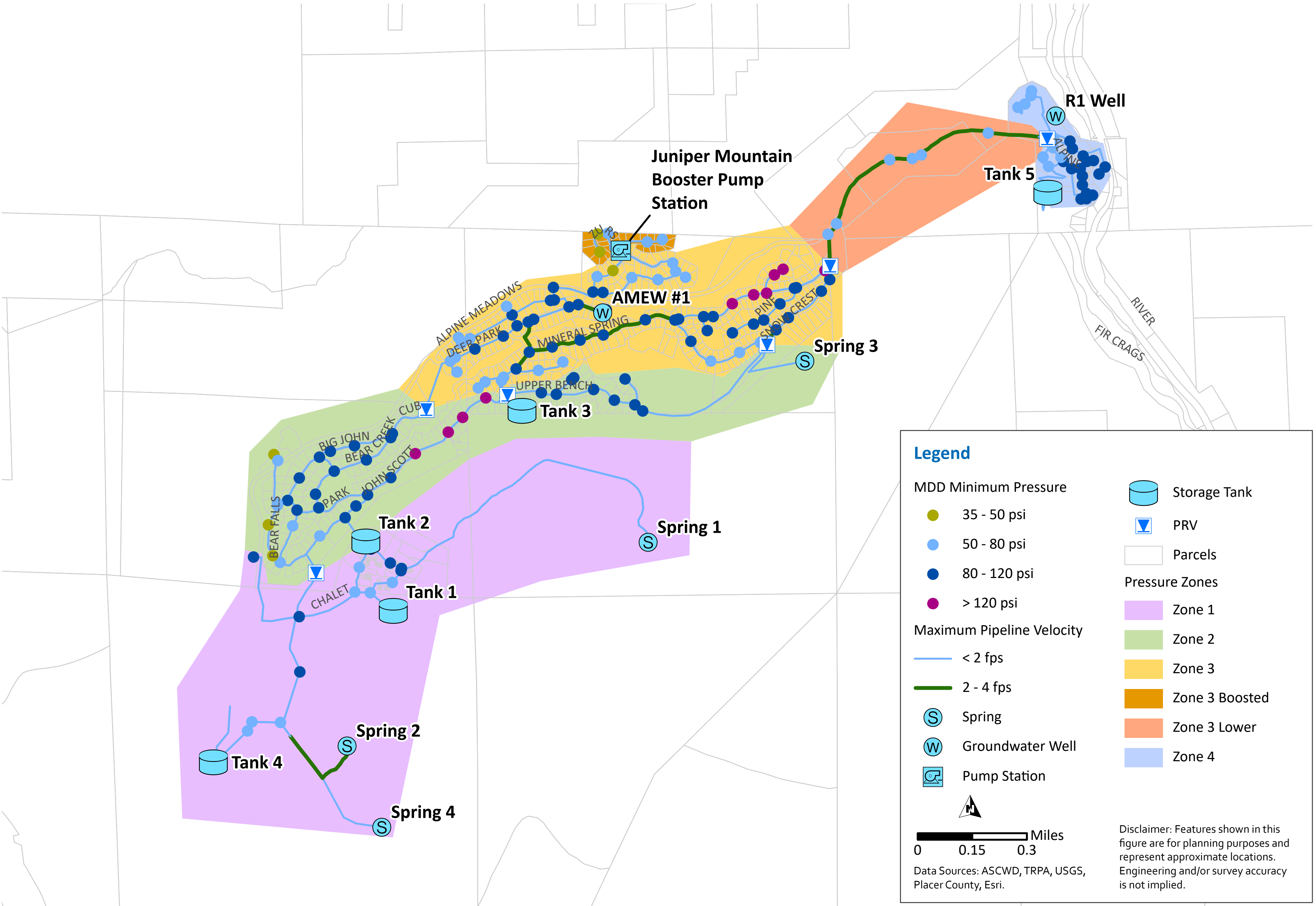


Figure 6.2 Existing Maximum Day Demand Minimum Pressures and Maximum Pipeline Velocities

6.3.1.3 Existing Fire Flow Results

As discussed in Chapter 5, one of the water system's functions is to provide sufficient capacity for fighting fires. A fire flow analysis was conducted under existing MDD conditions to determine the fire flow each hydrant can provide after 2 hours while maintaining a residual pressure of 20 psi.

Figure 6.4.pdf shows available fire flows at each hydrant with 20 psi residual pressure, and Figure 6.4 summarizes the existing fire flow results. Hydrants in Zones 1 and 4, which contain commercial areas, were considered deficient if unable to deliver 1,750 gpm for a duration of 2 hours with maintaining a residual pressure of 20 psi. Zones 2, 3, 3 Boosted, and 3 Lower consist of residential buildings; hydrants in these zones were considered sufficient if they could maintain a 20-psi residual after providing 1,500 gpm for 2 hours.

As shown, a substantial number of fire hydrants are unable to achieve desired fire flows under existing conditions. These results indicate that the water system requires capacity upgrades to enhance hydraulic performance and improve reliability for fire-fighting purposes.

It should be noted that this analysis assumes each hydrant must be capable of delivering the total fire flow requirement for the given pressure zone. Where hydrants are located in close proximity to one another, firefighters may utilize more than one hydrant. In such areas, fire flows could be split between the hydrants to achieve a combined flow equal to the total fire flow requirement. ASCWD can coordinate with the North Tahoe Fire Protection District (NTFPD) to determine the feasibility of this approach when evaluating individual fire flow improvement projects.

6.3.2 Future System Hydraulic Performance

The water distribution system hydraulic performance was modeled under projected 2045 conditions to evaluate the system's ability to accommodate projected demands. Future demands were added to the hydraulic model as follows:

- Planned developments: Infrastructure and demands associated with the planned Alpenglow and White Wolf developments were added to the model according to plans provided by ASCWD staff.
- Other developments: Demands associated with projected growth were allocated to model nodes in proportion to the developable parcel acreage within each pressure zone.

The following sections discuss hydraulic model results under 2045 conditions.

6.3.2.1 2045 Average Day Demand Results

Similar to the existing system analysis, the water system was evaluated under projected 2045 ADD conditions with the planned infrastructure to identify areas with high pressures. Figure 6.5 shows the modeled maximum pressures under 2045 ADD conditions.

As shown in Figure 6.5, the model results are similar to the results under existing ADD conditions. As discussed in Section 6.3.1.1, services with modeled pressures greater than 80 psi can be equipped with pressure reducing valves (PRVs) to mitigate damages from high pressures.

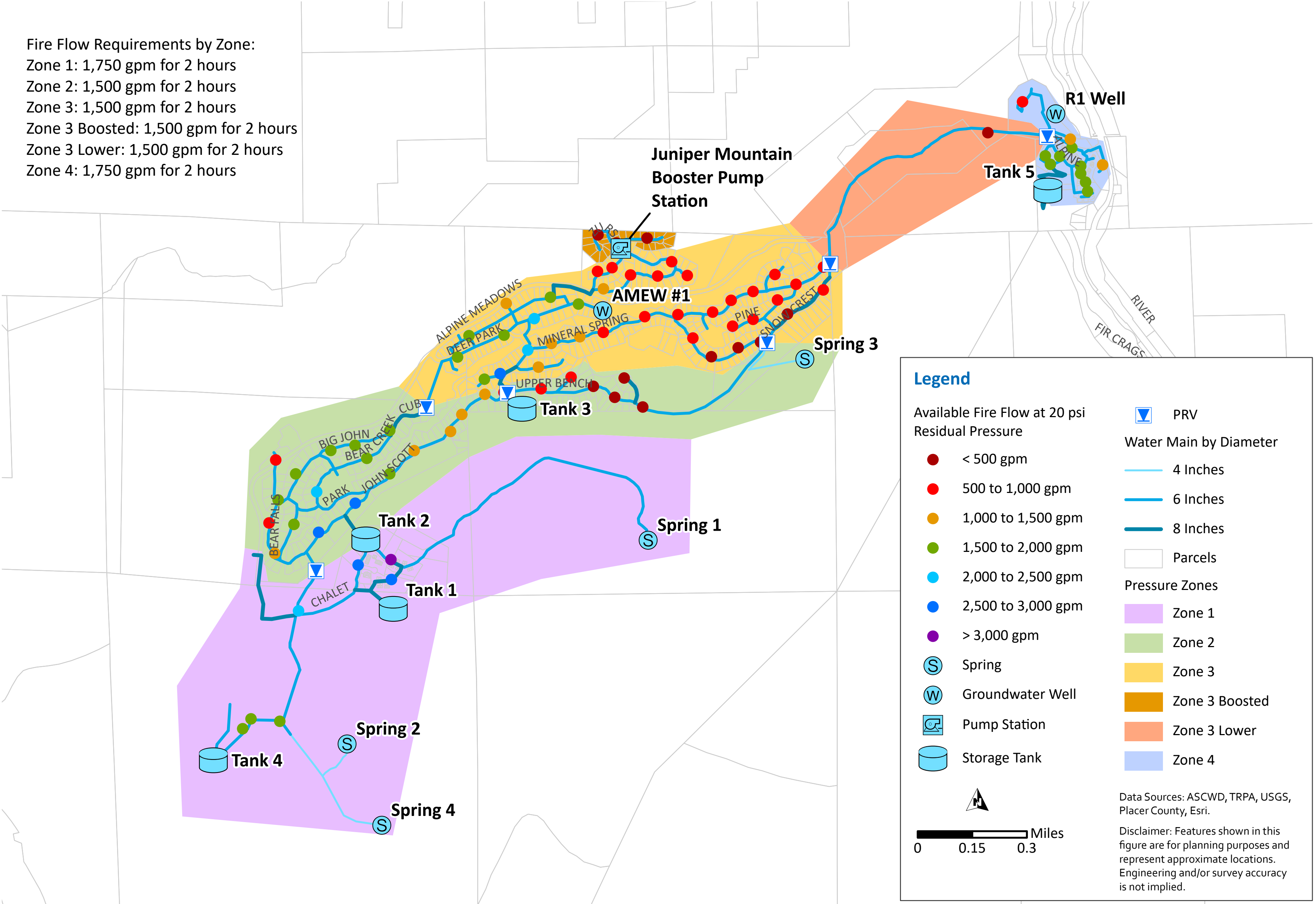


Figure 6.3 Available Fire Flows under Existing Maximum Day Demand Conditions

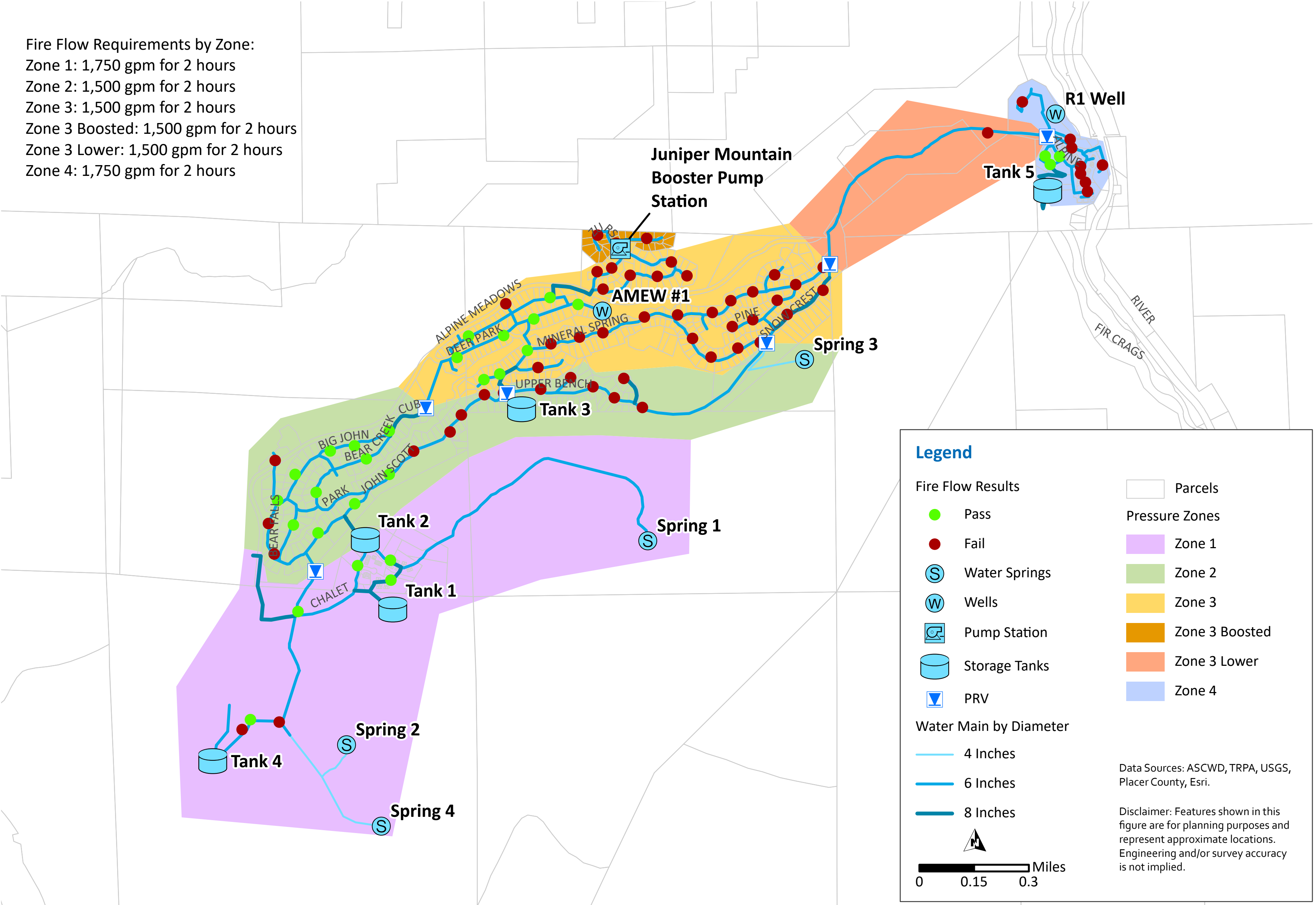


Figure 6.4 Existing Fire Flow Results

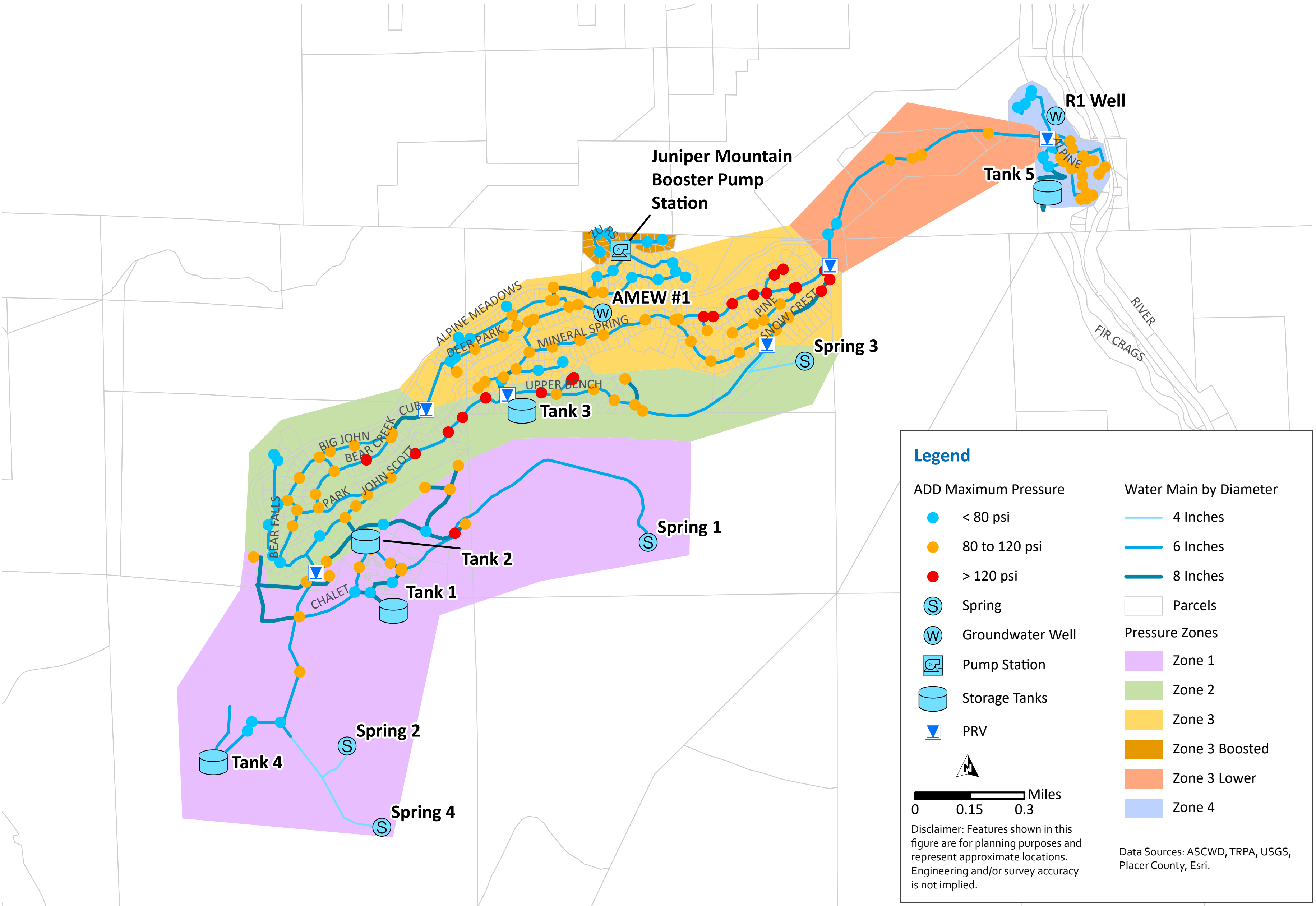


Figure 6.5 2045 Average Day Demand Maximum Pressures

6.3.2.2 2045 Maximum Day Demand Results

The water system was modeled under 2045 MDD conditions to identify areas that may experience low-pressure deficiencies as demands increase over the planning horizon.

Figure 6.6 shows modeled minimum pressures and maximum velocities for the 2045 MDD scenario.

The MDD results under 2045 conditions are similar to the existing system results, suggesting that increased demands from planned developments and other projected growth will not cause low-pressure deficiencies under typical high-demand conditions. Similar to the existing MDD results, high elevation areas in Zone 2, Zone 3, and Zone 3 Boosted have modeled pressures below 50 psi in the 2045 scenario. As discussed in Section 6.3.1.2, these areas are not considered deficient under MDD conditions but may experience pressures close to 35 psi, which could lead to low-pressure complaints.

6.3.2.3 2045 Maximum Day Demand Plus Fire Flow Results

A fire flow analysis was conducted under 2045 MDD conditions to determine potential implications of planned developments and projected growth on available fire flow capacity. Figure 6.7 shows hydrant available fire flows under 2045 conditions after 2 hours while maintaining 20 psi residual pressures, and Figure 6.8 shows hydrants that do not meet the fire flow criteria for their respective pressure zones.

The 2045 fire flow analysis indicates that most hydrants can deliver similar fire flows under 2045 conditions relative to existing conditions. In some areas, notably Zone 1, available fire flows increase between the existing and 2045 scenarios due to planned infrastructure that increases system looping. Other areas in Zones 2 through 4 generally experience marginal decreases in available fire flows between the existing and 2045 scenarios.

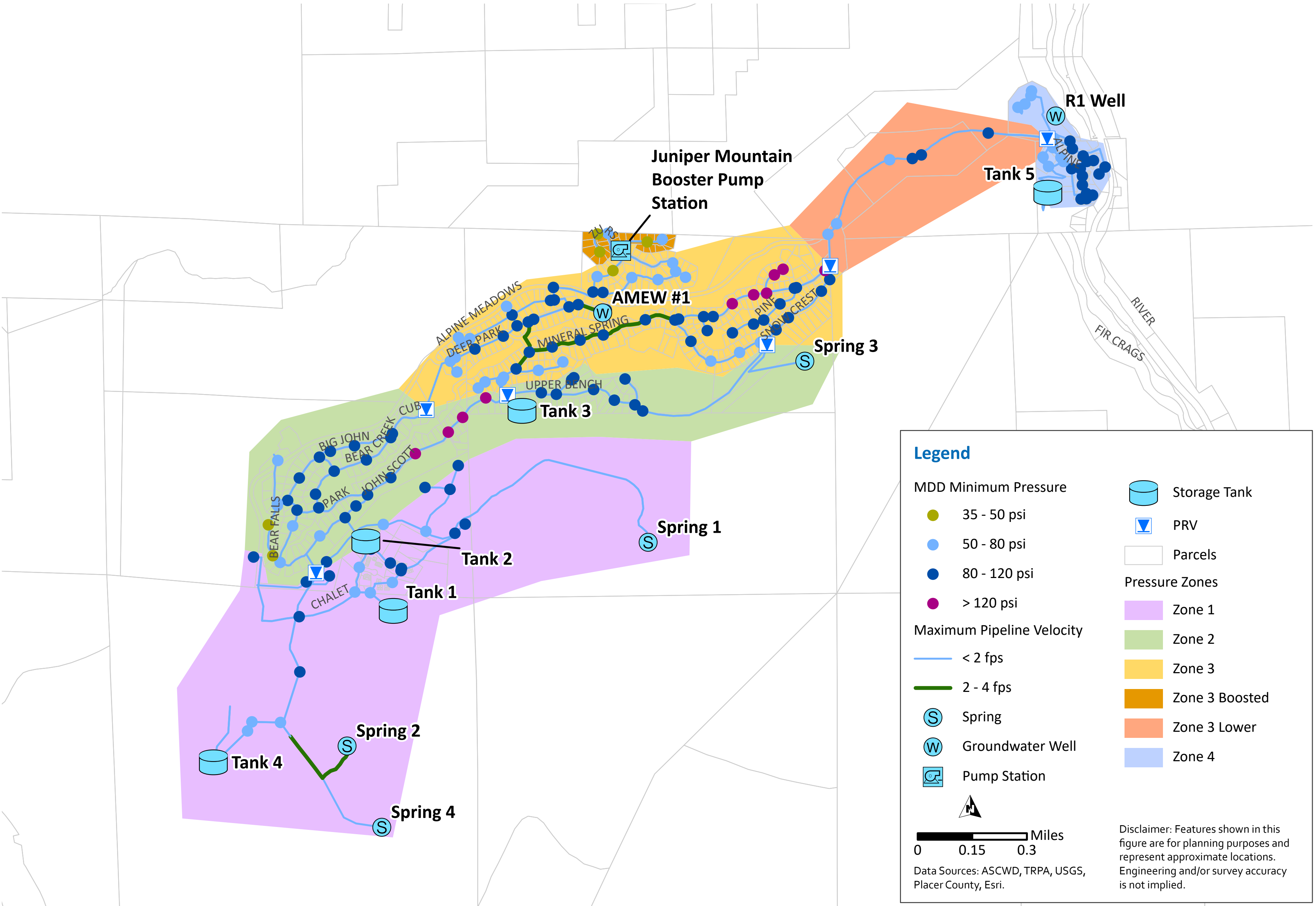


Figure 6.6 2045 Maximum Day Demand Minimum Pressures and Maximum Pipeline Velocities

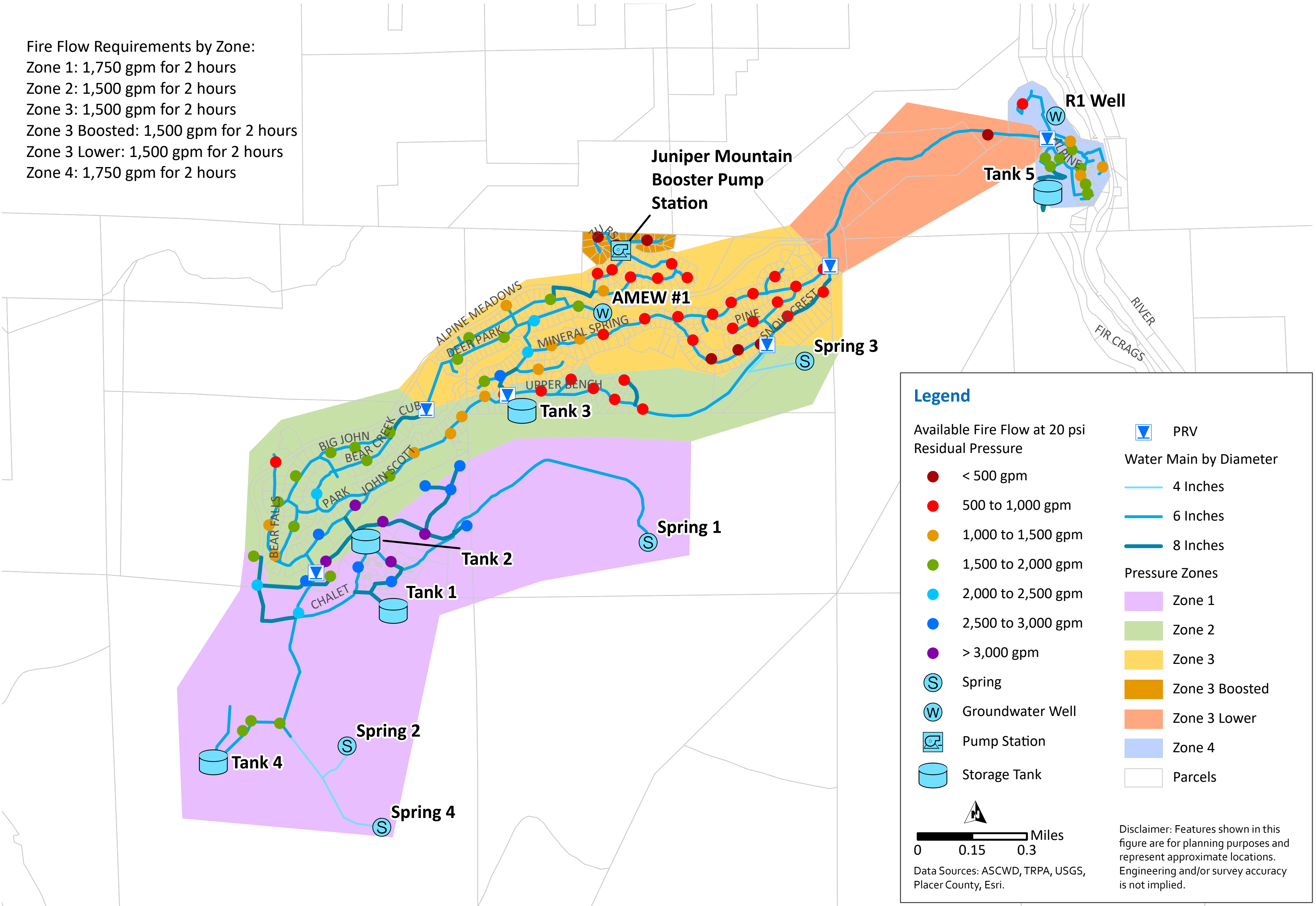


Figure 6.7 2045 Available Fire Flows under Existing Maximum Day Demand Conditions

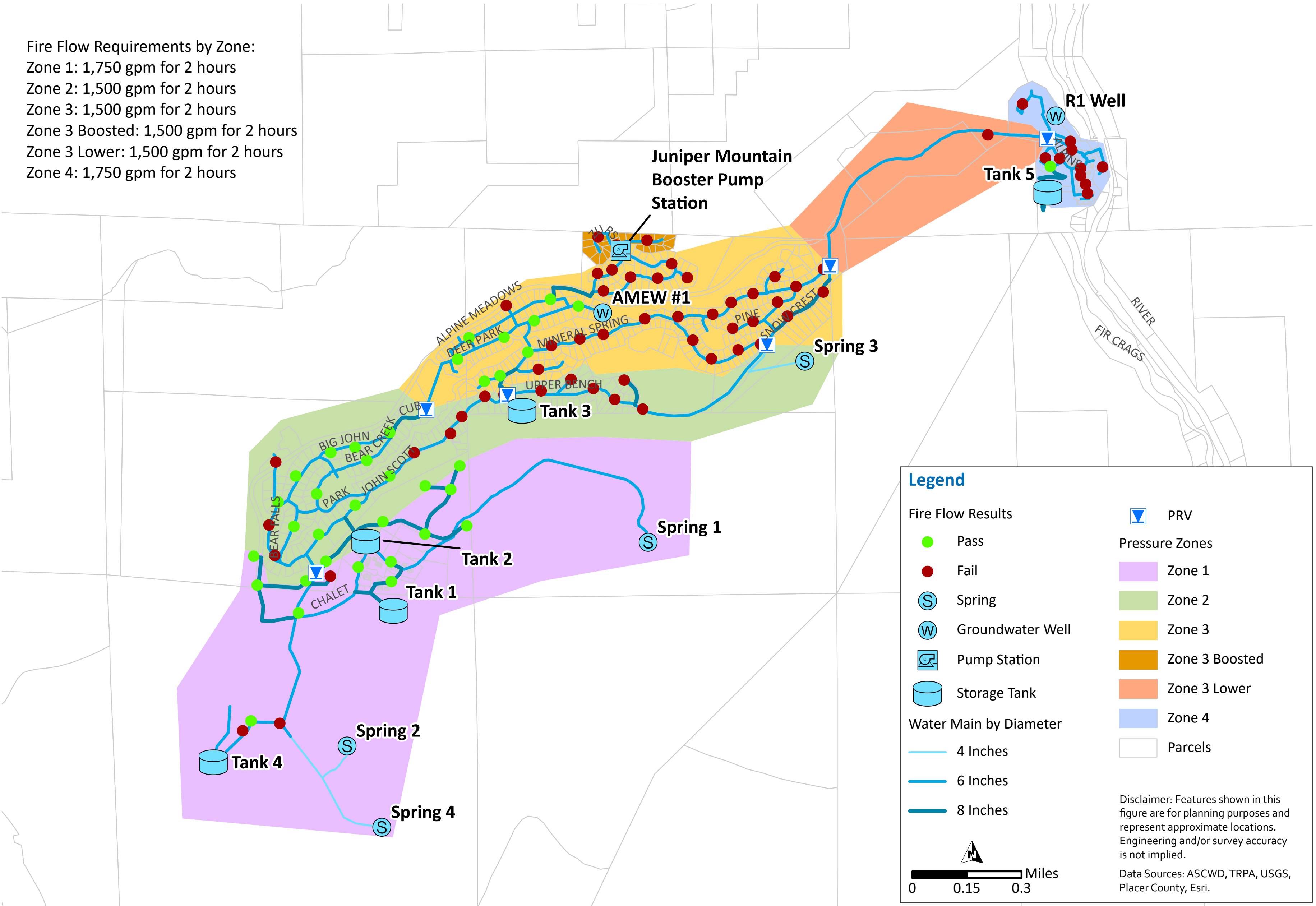


Figure 6.8 2045 Fire Flow Results

6.4 Water System Hydraulic Evaluation Summary

The water system hydraulic evaluation results indicate that ASCWD's water system has relatively reliable supply and storage and generally meets hydraulic performance targets. The supply analysis found the system does not have sufficient supply capacity to meet existing and projected 2045 requirements; upgrading the existing supply facilities' firm capacities can mitigate these deficiencies. According to the storage evaluation, each pressure zone has surplus storage capacity to meet operational, fire reserve, and emergency storage requirements through 2045. However, fire flow deficiencies in high-elevation regions, notably the Juniper Mountain area, suggest that storage could be moved to more optimal locations to improve hydraulic performance.

Hydraulic model results under existing and 2045 conditions indicate that the water system generally performs well under typical demand conditions. High pressures over 80 psi in the lower elevation regions within each pressure zone could be mitigated by installing PRVs. The model did not identify any areas with pressures below 35 psi under existing or 2045 MDD conditions, but high elevation areas in Zones 2, 3, and 3 Boosted have modeled pressures below 50 psi and may experience lower pressures during high-demand periods as the system ages.

The fire flow analysis found that hydrants throughout the distribution system are unable to achieve desired fire flows under existing and projected 2045 conditions. These results indicate areas in which capacity improvements can be implemented to improve hydraulic performance. As ASCWD replaces its system, the fire flow results should be used as one component to determine which portions of the system should be prioritized to achieve the greatest hydraulic benefits.

Chapter 8 presents proposed improvements that address water system hydraulic deficiencies, and Chapter 9 further discusses prioritization of the improvements to maximize system benefits. The hydraulic results were considered along with other factors, such as infrastructure condition and redundancy concerns, to develop an implementation plan.

Chapter 7

WASTEWATER SYSTEM EVALUATION

This chapter presents the wastewater collection system capacity evaluation performed for Alpine Springs County Water District (ASCWD). The capacity evaluation methodology, analysis, and results are discussed.

7.1 Collection System Capacity Evaluation Methodology

System bottlenecks raise the hydraulic grade line of upstream sewers, leading to backwater conditions. The greater the capacity deficiency, the higher water levels will surcharge upstream of the bottleneck. ASCWD's calibrated hydraulic model was used to identify bottlenecks in the collection system and to size improvements to eliminate capacity deficiencies.

Capacity analyses were performed on ASCWD's wastewater collection system to identify capacity deficiencies under existing and 2045 flow conditions. The system's gravity mains were evaluated against the criteria defined in Chapter 5.

7.1.1 Model Scenarios for Wastewater System Capacity Analyses

The calibrated SewerGEMS model was used to assess the collection system's ability to convey existing and future flows. The wastewater flow projections described in Chapter 2 were incorporated into the hydraulic model to simulate 2045 conditions. The model scenarios used to evaluate ASCWD's collection system are summarized in Table 7.1.

Table 7.1 Collection System Capacity Analysis Model Scenarios

Planning Horizon	Infrastructure	Modeled Dry Weather Flows	Modeled Wet Weather Flows
Existing	Existing	• 2021 HOF(1)	• 2021 RDI/I(1)
2045	Existing	• 2021 HOF(1) • Added HOF(1) from 2021 through 2045 growth • Added HOF(1) from Alpenglow and White Wolf developments	• 2021 RDI/I(1) • Added RDI/I(1) from Alpenglow and White Wolf developments

Notes:
(1) Abbreviations: HOF = high occupancy flow; RDI/I = rain-derived inflow and infiltration.

7.1.1.1 Modeled Infrastructure for Capacity Analyses

The collection system's existing infrastructure, as described in Chapter 3, was modeled for the capacity analyses. Additional sewer mains that will serve the planned Alpenglow and White Wolf developments were added to the model according to the most recent available plans for each development. It was assumed that additional projected developments, aside from Alpenglow and White Wolf, will connect to existing gravity mains and will not require new infrastructure, apart from private sewer laterals.

7.1.1.2 Modeled Future Flows

Future flows were allocated in the hydraulic model according to the available information for each development.

- **Planned developments:** The point of connection (POC) associated with each planned development was determined using the most recent plans for the respective developments. The 2045 average dry weather flow (ADWF) for the two planned developments were allocated in the hydraulic model at the POCs, as shown on Figure 7.1. Catchments were drawn upstream of the POCs to simulate added RDI/I from the new gravity mains. The catchment areas were calculated using the same method that was used for the existing system catchments; the total length of planned gravity main was calculated by an assumed 50-foot buffer.
- **Other projected developments:** Added ADWF associated with other projected growth was spread evenly throughout the system due to the uncertainty regarding where this growth will occur. Each manhole was assigned an additional 0.0012 gallons per minute (gpm) to represent projected future growth. As previously discussed, it is assumed that future developments within this category will not require new gravity mains, so no additional catchments were added to the model to simulate added RDI/I.

7.2 Collection System Capacity Analysis Results

The hydraulic model did not identify any surcharged gravity mains under existing or 2045 PWWF conditions. One 10-inch diameter gravity main located along Alpine Meadows Road has a maximum modeled depth to diameter (d/D) between 0.50 and 0.66 under existing PWWF conditions. The model results under 2045 PWWF conditions indicate that several additional gravity mains would have d/D ratios greater than 0.50 with the planned and projected developments, but most still have maximum d/D ratios less than 0.66. More detailed evaluations should be performed to determine potential implications from the planned developments.

Figure 7.2 and Figure 7.3 show the existing and 2045 collection system capacity analysis results, respectively.

7.3 Collection System Hydraulic Evaluation Summary

The collection system capacity analysis indicates that ASCWD's wastewater collection system performs well under existing and projected 2045 conditions. No capacity deficiencies were identified.

Although the model results indicate that added flows from the planned developments would not lead to downstream capacity deficiencies, each development should be evaluated in further detail to verify potential implications on the wastewater system infrastructure.

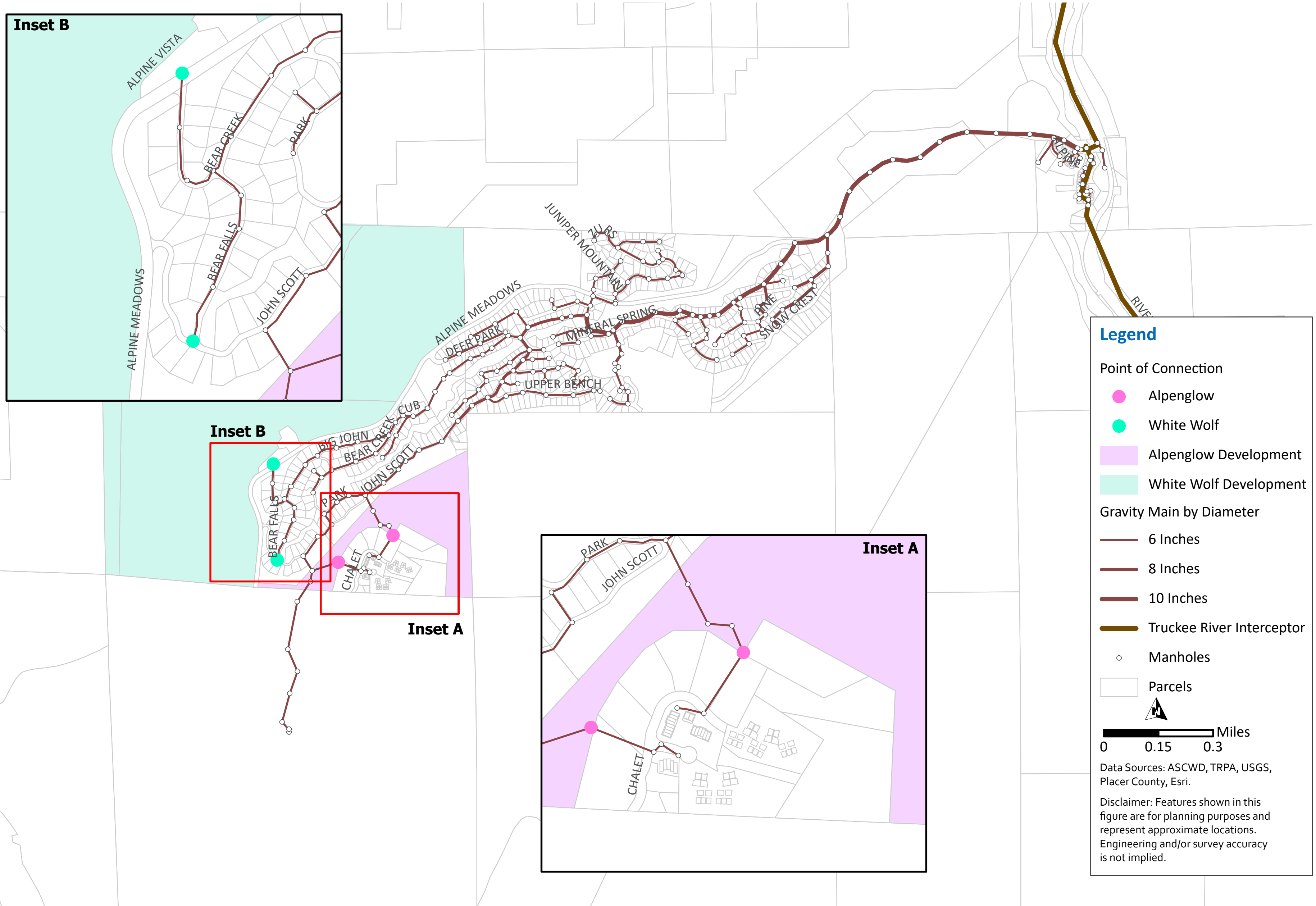


Figure 7.1 Wastewater System Points of Connection for Planned Developments

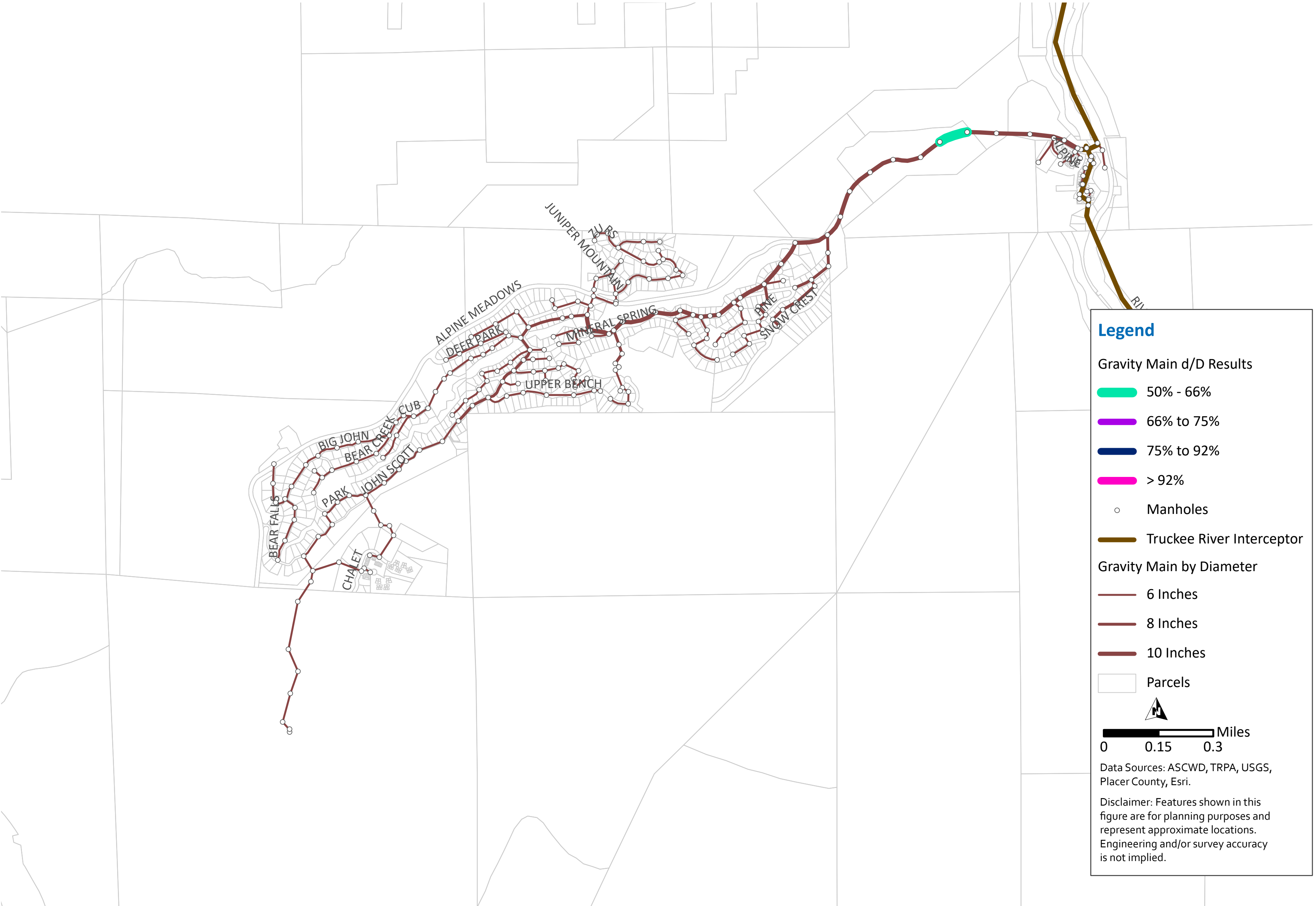


Figure 7.2 Existing Wastewater System Capacity Analysis Results

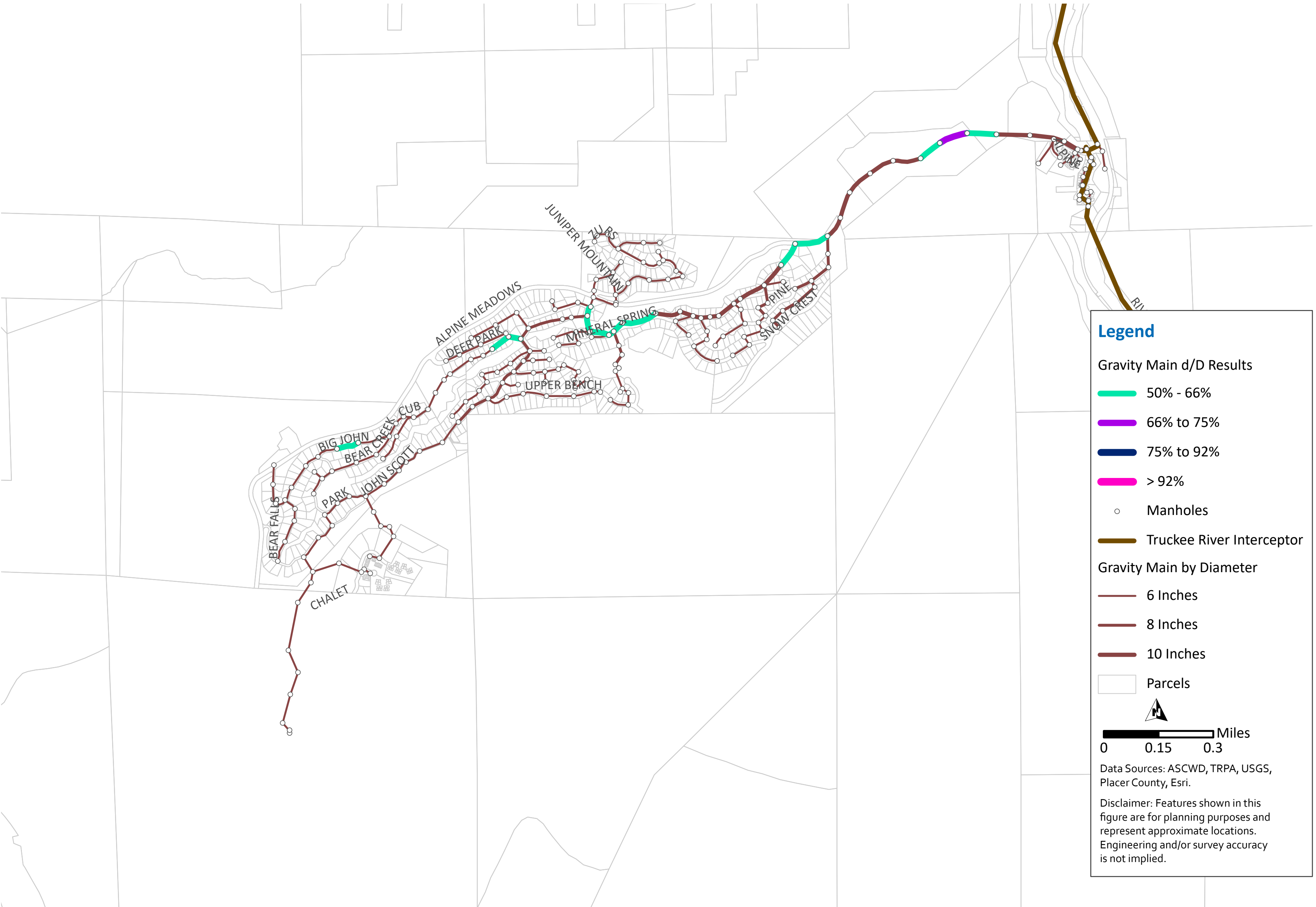


Figure 7.3 2045 Wastewater System Capacity Analysis Results

Chapter 8

WATER AND WASTEWATER SYSTEMS PROPOSED IMPROVEMENTS

This chapter details proposed improvements to address hydraulic and condition-related deficiencies within Alpine Springs County Water District's (ASCWD) water distribution and wastewater collection systems.

8.1 Water System Improvements

The following sections discuss improvements identified to mitigate water system condition and capacity deficiencies.

8.1.1 Supply Improvements

The water system has sufficient total supply capacity to meet projected demands through the 2045 planning horizon. However, the Alpine Meadows Estates Well (AMEW) Number 1 does not have backup power to provide sufficient firm capacity with the largest pump on standby. To enhance supply reliability, ASCWD should install backup generators and standby pumps at AMEW No. 1.

As discussed in Chapter 4, the Spring 1 casing experiences a persistent leak. Given the topographical challenges that limit access to Spring 1 for repairs, along with Spring 1 production data that indicates continued stable flows despite the leak, repair of the leak is considered a lower priority than other supply improvements. ASCWD should continue to monitor flows from the spring and other supplies to verify that the leak does not contribute to decreased supply availability. Similarly, ASCWD may also consider drilling a groundwater testing well near Spring 1 to enable periodic groundwater level testing. Data from this well could help determine whether the leak is substantially affecting groundwater levels and available supply.

8.1.2 Storage Rehabilitation and Replacement Improvements

The water system has sufficient storage capacity to meet each pressure zone's operational, fire reserve, and emergency storage requirements. As discussed in Chapter 4, four out of the five existing storage tanks (i.e., Tanks 1, 2, 3, and 5) have major condition deficiencies that require attention to mitigate reliability concerns. These tanks should be rehabilitated or replaced to reduce the risks of catastrophic failure and the consequent need for emergency replacements.

The hydraulic water model was used to determine the feasibility of decommissioning Tank 1 as an alternative to rehabilitation or replacement. The water system would have sufficient storage capacity without Tank 1, but available fire flows would decease due to hydraulic limitations of the piping within Zone 1. In particular, 6-inch diameter water mains between Tank 4 and Tank 2 limit flows from Tank 4 to hydrants along Chalet Road. If Tank 1 was decommissioned, these mains would need to be upsized to mitigate fire flow deficiencies. These alternative improvements are discussed in Section 8.1.3.

8.1.3 Water Distribution System Improvements

Water distribution system improvements were developed to mitigate existing and projected capacity and condition deficiencies. As discussed in Chapter 4, most of the water distribution system pipelines were installed in the 1960s and '70s and are approaching the end of their useful lifetimes. To mitigate risk of widespread failures, it is recommended that ASCWD implement an accelerated pipeline replacement program that will enable full system replacement within the next 50 years. This corresponds with a replacement rate of about 1,600 linear feet per year.

In addition to mitigating risk of failure, distribution system improvements can also help enhance hydraulic capacity and performance. The model results discussed in Chapter 6 indicate that hydraulic bottlenecks throughout the water system limit fire flow availability, contributing to lower reliability for firefighting purposes. Upsizing water mains, as well as adding new water mains to increase system looping, can alleviate bottlenecks and mitigate fire flow deficiencies. Moving storage and transmission to more optimal locations can also increase available fire flows.

The following sections discuss specific distribution system improvements that were identified to mitigate hydraulic deficiencies. Chapter 9 discusses prioritization of the proposed distribution system improvements.

8.1.3.1 Water Main Upsize Improvements

A total of 28 water main upsize improvements were identified to mitigate fire flow deficiencies under existing and projected 2045 conditions. These improvements total approximately 32,000 linear feet, which encompasses about 42 percent of the distribution system. Table 8.1 summarizes the proposed water main upsize improvements.

Table 8.1 Proposed Improvements Summary

Project ID	Project Name	Pressure Zone	Existing Diameter (inches)	Proposed Diameter (inches)	Length (feet)
WM-01	Water main upsize from Tank 4 to Alpine Meadows Lodge	Zone 1	6	10	810
WM-02	Water main upsize from Alpine Meadows Lodge to Chalet Road	Zone 1	6	10	2,070
WM-03	Water main upsize along Chalet Road	Zone 1	6	10	960
WM-04	Water main upsize along John Scott Trail by Bear Creek	Zone 2	6	8	1,090
WM-05	Water main upsize along Bear Falls Lane	Zone 2	6	8	1,220
WM-06	Water main upsize along Bear Creek Drive	Zone 2	6	10	680
WM-07	Water main upsize along John Scott Trail west of Park Drive	Zone 2	6	10	480
WM-08	Water main upsize along John Scott Trail east of Park Drive	Zone 2	6	10	2,290

Project ID	Project Name	Pressure Zone	Existing Diameter (inches)	Proposed Diameter (inches)	Length (feet)
WM-09	Water main upsize along Upper Bench Road	Zone 2	6	10	2,470
WM-10	Water main upsize along Trapper Place	Zone 3	6	8	260
WM-11	Water main upsize along Trapper McNutt Trail	Zone 3	6	8	340
WM-12	Water main upsize from AMEW #1 to Trapper McNutt Trail	Zone 3	6	8	2,000
WM-13	Water main upsize from Beaver Dam Trail to Deer Park Drive	Zone 3	6	8	290
WM-14	Water main upsize from new Juniper Mountain PS to Kloster Court	Zone 3	6	8	240
WM-15	Water main upsize along Kloster Court	Zone 3	6	8	570
WM-16	Water main upsize along Juniper Mountain Road	Zone 3	6	8	410
WM-17	Water main upsize along Cortina Court	Zone 3	6	8	480
WM-18	Water main upsize from Snow Crest Road to Pine Trail	Zone 3	6	8	730
WM-19	Water main upsize from R-4 to Alpine Circle Road	Zone 3 Lower	6	8	4,420
WM-20	Water main upsize towards commercial center north of Alpine Meadows Road	Zone 4	6	8	350
WM-21	Water main upsize towards recreational area north of Alpine Meadows Road	Zone 4	6	8	910
WM-22	Water main upsize along Alpine Meadows Road and Highway 89 towards River Ranch	Zone 4	6	8	1,050
WM-23	Water main upsize along Alpine Circle Road	Zone 4	6	8	700
WM-24	Water main upsize from Alpine Circle Road towards condominium tennis court	Zone 4	6	8	560
WM-25	Water main upsize west of Alpine Circle Road	Zone 4	6	8	670

Project ID	Project Name	Pressure Zone	Existing Diameter (inches)	Proposed Diameter (inches)	Length (feet)
WM-26	Water main upsize towards Tank 5	Zone 4	8	10	1,640
WM-27	Water main upsize along Mineral Springs Trail from John Scott Trail to west end of Snow Crest Road	Zone 3	6	12	2,240
WM-28	Water main upsize along Snow Crest Road	Zone 3	6	10	1,930
WM-29	Water main upsize along Mineral Springs Place	Zone 3	6	8	320

8.1.3.2 Juniper Mountain Fire Flow Improvements

An additional transmission and storage project in the Juniper Mountain area (i.e., Zone 3 Boosted) was developed to enable hydrants in this high-elevation region to achieve desired fire flows. The following improvements are proposed to mitigate Juniper Mountain fire flow deficiencies:

- Decommission existing Juniper Mountain PS.
- Install new Juniper Mountain PS at the base of Juniper Mountain Road by Alpine Meadows Road.
- Install new Tank 6 with a capacity of 0.2 million gallons (MG) above Zone 3 Boosted at an elevation of approximately 6,740 feet.
- Install 10-inch diameter transmission main from new Tank 6 to Zone 3 Boosted.

Implementing the proposed improvements described above would expand Zone 3 Boosted to the entirety of the Juniper Mountain neighborhood and would increase the zone's hydraulic grade line to maintain static service pressures greater than 50 pounds per square inch (psi). The new Tank 6 was sized to provide 1,500 gallons per minute (gpm) of fire flow to Zone 3 Boosted for a duration of 2 hours starting at 90 percent full. In addition to these improvements, the water main upsize improvements identified in Section 8.1.3.1 within the Juniper Mountain area are also required to mitigate fire flow deficiencies (i.e., WM-14, WM-15, WM-16, and WM-17).

8.1.4 Water System Service Improvements

To maintain system stewardship goals, it is recommended that ASCWD plans to rehabilitate or replace about one percent of water system laterals per year, which equates to about 150 laterals over 20 years. It is recommended that lateral replacements are coordinated with water distribution main improvement where possible.

8.1.5 Water System Improvements Summary

Figure 8.1 shows the proposed water system improvements.

8.2 Wastewater System Improvements

The following discuss proposed improvements to mitigate existing and projected deficiencies within the wastewater collection system. The proposed improvements address gravity main rehabilitation and replacement (R&R) and laterals.

8.2.1 Wastewater System Rehabilitation and Replacement

As discussed in Chapter 4, the wastewater system is generally in good condition, with a relatively low frequency of defects requiring R&R. However, the system is continually aging, and gravity mains and laterals may begin deteriorating more quickly as the assets approach the end of their useful lifetimes.

To maintain system stewardship goals, ASCWD should plan to rehabilitate or replace approximately one percent of the collection system per year. This rate will enable the District to rehabilitate or replace the entire wastewater system on a one-hundred-year basis, which is approximately equal to the expected useful lifetime of the system's gravity mains and laterals. The District should reexamine this rate at least every ten years to determine if a higher R&R rate is warranted.

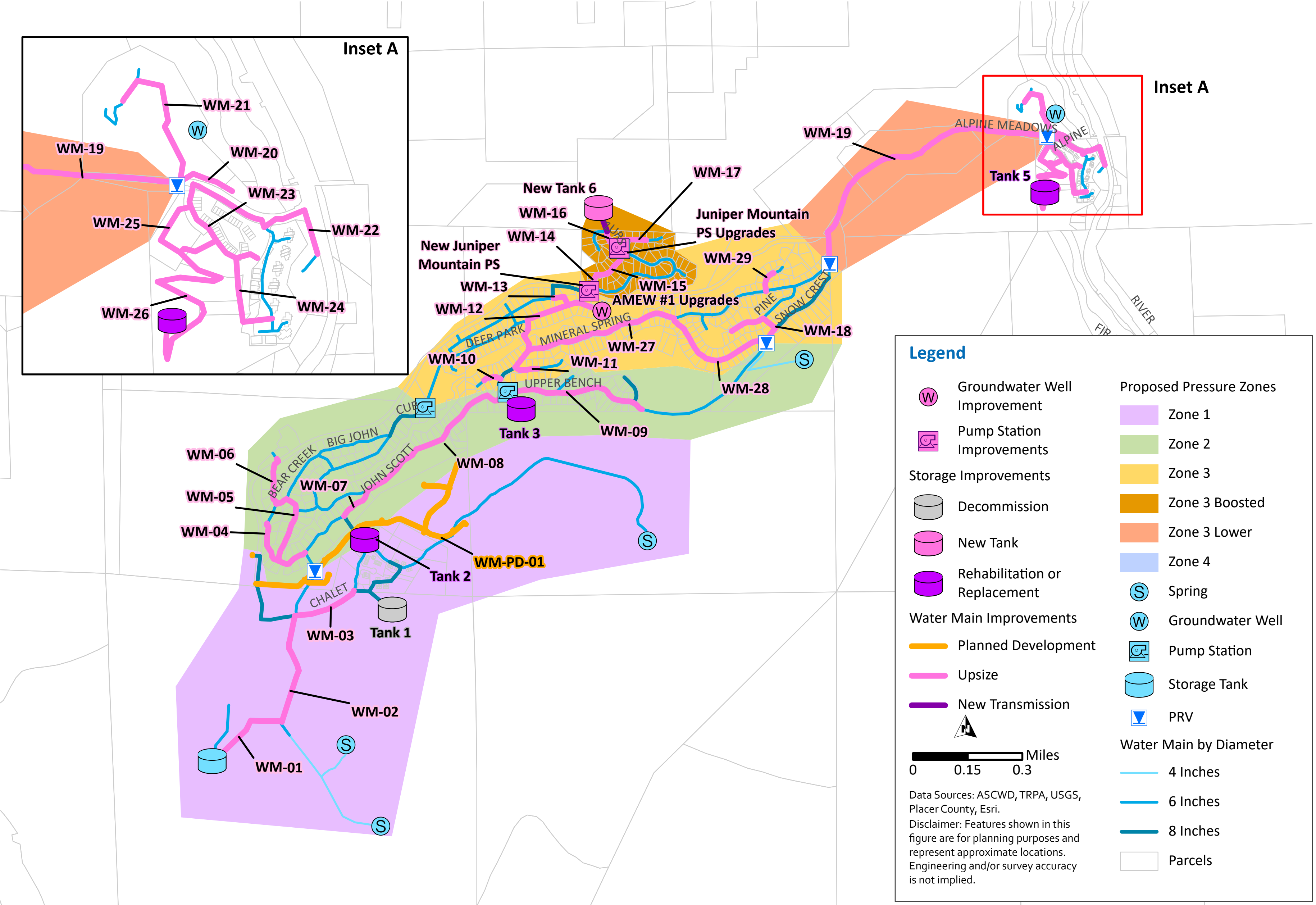


Figure 8.1 Proposed Water System Improvements

8.2.2 Wastewater System Service Improvements

ASCWD should coordinate with private property owners to inspect and rehabilitate or replace the upper and lower portions of sewer laterals. Service laterals can be a primary source of infiltration and inflow (I/I) and should therefore be inspected regularly to identify repair needs. It is recommended that lateral R&R is coordinated with gravity main R&R to minimize construction disturbance and optimize resources.

Consistent with gravity main R&R, it is recommended that ASCWD plans to rehabilitate or replace approximately one percent of sewer laterals per year, or about 7 to 8 laterals per year. This equates to about 150 laterals over the 20-year planning horizon.

8.3 Water and Wastewater Systems Monitoring and Analysis

In addition to the infrastructure improvements outlined in the preceding sections, monitoring and analysis activities are also proposed to maintain accurate data, evaluate effectiveness of system improvements, and optimize system operations. These activities will help ASCWD verify that resources are allocated efficiently and will enable more robust tracking of system condition and performance.

The following monitoring and analysis activities are proposed to facilitate ongoing data collection and evaluation:

- Model recalibrations: It is proposed that ASCWD recalibrate the water and wastewater models developed as part of this master plan approximately every ten years. The water and wastewater models should be recalibrated as follows:
  - Water model: To enable more accurate recalibration of the water model, ASCWD should conduct the following activities:
    - ASCWD should coordinate with fire district personnel who conduct fire flow tests to verify that the following information is recorded during each fire flow test:
      - Test hydrant static pressure reading.
      - Test hydrant residual pressure reading.
      - Static reading time.
      - Residual reading time.
      - Tank levels during test.
      - Test hydrant location.
      - Pump flowrates during test.
      - Flowing hydrant location(s).
      - Flowing hydrant flowrate(s).
      - Test duration.
    - To help facilitate more efficient and accurate water model recalibrations, ASCWD should consider purchasing more robust supervisory control and data acquisition (SCADA) data record-keeping software. This will enable ASCWD to more easily mine historical data, thus enhancing both the quantity and quality of data for which to recalibrate the water model.
  - Wastewater model: The wastewater model should be recalibrated using the permanent Truckee River Interceptor (TRI) flow meter and nearby rain gauge data.
- Condition assessment data aggregation: It is proposed that ASCWD aggregate condition assessment data, such as closed-circuit television (CCTV) data, using methods consistent with the water and wastewater models. The identification (IDs) assigned to

assets in the models should also be used in the condition assessment database to reduce confusion and facilitate more efficient analysis.

- Master Plan updates: It is proposed that ASCWD update its Water and Wastewater Master Plan every ten years to reevaluate the water and wastewater systems with more recent data and to determine appropriate financing adjustments.

8.4 Improvements Summary

Table 8.2 summarizes the proposed water and wastewater system improvements, and Chapter 9 discusses conceptual costs and considerations for implementing the proposed improvements.

Table 8.2 Proposed Improvements Summary

Project ID (1)	Project Name	Project Description
Water System Capacity Improvements		
PS-01	New Juniper Mountain PS	Install new booster pump station with a firm capacity of 70 gpm and a total dynamic head of 100 feet at Alpine Meadows Road and Juniper Mountain Road.(2)
S-01	New Tank 6	Install new Tank 6 above Juniper Mountain Road at an elevation of approximately 6,740 feet.(2)
GW-02	Alpine Meadows Estates Well Number 1 upgrades	Install standby pump and backup generator.
WM-01 through WM-29	Water main upsize projects	Upsize water mains to provide sufficient capacity for fire flows. See Section 8.1.3 for individual project details.
Water System Condition Improvements		
RR-S-01	Tank 2 rehabilitation or replacement	Rehabilitate or replace Tank 2.
RR-S-02	Tank 3 rehabilitation or replacement	Rehabilitate or replace Tank 3.
RR-S-03	Tank 5 rehabilitation or replacement	Rehabilitate or replace Tank 5.
RR-PWL-01	Ongoing water service lateral rehabilitation and replacement	Rehabilitate or replace about 8 water service laterals per year.
Wastewater System Condition Improvements		
RR-GM-01	Ongoing gravity main rehabilitation and replacement	Rehabilitate or replace approximately 540 linear feet of gravity main per year.
RR-WWL-01	Ongoing wastewater service lateral rehabilitation and replacement – upper laterals	Rehabilitate or replace about 8 upper wastewater service laterals per year.
RR-WWL-02	Ongoing wastewater service lateral rehabilitation and replacement – lower laterals	Rehabilitate or replace about 8 lower wastewater service laterals per year.

Project ID [1]	Project Name	Project Description
Miscellaneous Improvements		
M-01	Master Plan updates	Update water and wastewater master plan every 10 years.
M-02	SCADA updates	Update SCADA system to enable data extraction.

Notes:

(1) Project IDs use the following nomenclature: PS – pump station improvement; S – storage improvement; GW -- groundwater well improvement; WM – water main improvement; PWL – potable water lateral improvement; GM -- gravity main improvement; WWL – wastewater lateral improvement; RR – rehabilitation or replacement, M -- miscellaneous.

(2) The new Juniper Mountain PS and Tank 6 specifications should be refined during more detailed planning stages for the proposed facilities. The pump station firm capacity should be sufficient to supply the adjusted Zone 3 Boosted peak hour demand.

Chapter 9

CAPITAL IMPROVEMENT PLAN

This chapter presents the proposed capital improvement plan (CIP) for the Alpine Springs County Water District (ASCWD). Capital cost estimates are presented along with a discussion of potential financial implications to ASCWD.

9.1 Capital Improvement Projects

The proposed water and wastewater system improvements discussed in Chapter 8 set the foundation for ASCWD's CIP. The cost estimates presented in the following sections are opinions developed from recent bid tabulations, vendor quotes for equipment, information obtained from previous studies, and experience on similar projects. The costs are provided in 2022[1] dollars and must be escalated to account for factors such as inflation in future years.

9.2 Cost Estimating Accuracy

The cost estimates in this CIP were prepared for general master planning purposes and for guidance in project evaluation and implementation. Each project's final cost will depend on actual labor and material costs, competitive market conditions, final project scope, and implementation schedule. Final project scopes will require preliminary alignment generation, investigation of alternative routings, and detailed utility and topography surveys.

The Association for the Advancement of Cost Engineering (AACE) defines an "Order of Magnitude Estimate", deemed appropriate for master plan studies, as an approximate estimate made without detailed engineering data. The following sections present the assumptions used in developing order of magnitude cost estimates for this study. Unless otherwise noted, the cost estimates are considered Class 5 accuracy level. Table 9.1 lists AACE guidelines for anticipated cost estimate accuracy according to the type of estimate.

Table 9.1 Anticipated Cost Estimate Accuracy

Estimate Class	Purpose	Anticipated Accuracy
Class 5	Conceptual	+100% to -50%
Class 4	Planning Level	+50% to -30%
Class 3	Preliminary Design	+30% to -15%
Class 2	50% to 70% Design Completion	+20% to -10%
Class 1	Prebid	+15% to -5%

The cost estimates for this CIP were developed during a period of increased economic instability. Capital improvement costs changed rapidly during this period due to abnormal spikes in demand and disruptions to supply chains. This variability is not accounted for in the cost estimates.

[1] All costs are in October 2022 dollars using the Engineering News Record 20-city average construction cost index of 13,175.

9.2.1 Cost Estimating Methodology

Capital project costs consist of baseline construction costs, estimating contingencies, and other contingencies consistent with a Class 5 estimate. The following sections outline the assumptions used to estimate baseline construction, total construction, and total project costs.

9.2.1.1 Baseline Construction Cost

The baseline construction cost is the total estimated construction cost, in dollars, of the proposed improvements for pipelines and appurtenances, storage facilities, pump stations (PS), and other facilities. Construction costs used for this study are representative of water and wastewater system facilities under normal construction conditions and schedules.

For project components in which the unit costs are known, the baseline construction costs were developed by multiplying the number of units to be replaced, rehabilitated, or newly installed by the unit cost. For other project components, such as pumps, tanks, and piping within facilities, costs were developed on a case-by-case basis depending on relevant upgrades. Vendor quotes were obtained for pumps and tanks.

9.2.1.2 Contingency, Engineering, and Easements

Contingency costs must be reviewed on a case-by-case basis because they will vary considerably with each project due to uncertainties associated with the preliminary layout of a project. Unexpected construction conditions, the need for unforeseen mechanical items, and variations in final quantities are examples of items that can increase project costs. To assist ASCWD in making financial decisions for these future construction projects, contingency costs will be added to the planning budget as percentages of the baseline construction cost.

Project construction contingency costs include, but are not limited to, costs associated with project engineering, construction phase professional services, and project administration. Engineering services associated with new facilities, such as preliminary investigations and reports, right-of-way (ROW) acquisition, surveying and staking, sampling of testing material, and start-up services, vary depending on specific project requirements. Construction phase professional services cover items such as legal fees, environmental compliance requirements, permitting compliance, financing expenses, administrative costs, and interest during construction. Allowances for yard piping, paving and grading, coatings, electrical systems, and instrumentation systems were also included as a percent of construction.

Table 9.2 lists the assumed contingencies used to calculate total project costs.

Table 9.2 Contingency Assumptions

Contingency	Assumption(1)
Estimating contingency	30 percent of baseline construction cost
Direct construction cost as percentage of baseline cost	**130 percent**
Contractor general conditions	10 percent of direct construction cost
Contractor overhead and profit	10 percent of direct construction cost
Total construction cost as percentage of baseline cost	**157 percent**
Project delivery cost(2)	15 percent of total construction cost
Total project cost as percent of baseline construction cost	**181 percent**

Notes:
(1) The listed contingencies were assumed for most project costs. Certain projects, such as the Juniper Mountain PS upgrades, do not require all contingencies.
(2) Project delivery costs consist of project and construction management, permitting, engineering, services during construction, commissioning, close-out, and legal and administrative fees.

9.2.2 Total Capital Improvement Project Cost

The total capital improvement cost is the sum of the direct construction cost and the contingency costs. A summary of the capital project costs is presented in Table 9.3. This table identifies the projects, provides a brief description of the project, identifies facility size (e.g., pipe diameter and length), and provides the capital improvement cost.

Table 9.3 Capital Improvement Project Costs

Project ID	Project Name	Proposed Amount	Unit	Unit Cost	Baseline Construction Cost (1)	Direct Construction Cost (1)(2)	Total Construction Cost (1)(3)	Total Capital Cost (1)(4)
Water System Capacity Improvements								
PS-01	New Juniper Mountain PS	2.2	hp	$7,000	$15,000	$20,000	$24,000	$27,000
S-01	New Tank 6	0.2	MG	$3.0	$600,000	$780,000	$944,000	$1,085,000
GW-02	Alpine Meadows Estates Well Number 1 upgrades	1	lump sum	$154,000	$154,000	$201,000	$229,000	$254,000
WM-01	Water main upsize from Tank 4 to Alpine Meadows Lodge	810	linear feet of 10-inch diameter water main	$410	$332,000	$432,000	$522,000	$601,000
WM-02	Water main upsize from Alpine Meadows Lodge to Chalet Road	2,070	linear feet of 10-inch diameter water main	$410	$849,000	$1,104,000	$1,335,000	$1,536,000
WM-03	Water main upsize along Chalet Road	960	linear feet of 10-inch diameter water main	$410	$394,000	$512,000	$620,000	$713,000
WM-04	Water main upsize along John Scott Trail by Bear Creek	1,090	linear feet of 8-inch diameter water main	$345	$376,000	$489,000	$591,000	$680,000
WM-05	Water main upsize along Bear Falls Lane	1,220	linear feet of 8-inch diameter water main	$345	$421,000	$547,000	$662,000	$762,000
WM-06	Water main upsize along Bear Creek Drive	680	linear feet of 10-inch diameter water main	$410	$279,000	$363,000	$439,000	$505,000
WM-07	Water main upsize along John Scott Trail west of Park Drive	480	linear feet of 10-inch diameter water main	$410	$197,000	$256,000	$310,000	$356,000
WM-08	Water main upsize along John Scott Trail east of Park Drive	2,290	linear feet of 10-inch diameter water main	$410	$939,000	$1,221,000	$1,477,000	$1,699,000
WM-09	Water main upsize along Upper Bench Road	2,470	linear feet of 10-inch diameter water main	$410	$1,013,000	$1,317,000	$1,593,000	$1,832,000
WM-10	Water main upsize along Trapper Place	260	linear feet of 8-inch diameter water main	$345	$90,000	$117,000	$142,000	$163,000
WM-11	Water main upsize along Trapper McNutt Trail	340	linear feet of 8-inch diameter water main	$345	$117,000	$152,000	$184,000	$212,000
WM-12	Water main upsize from Alpine Meadows Estates Well Number 1 to Trapper McNutt Trail	2,000	linear feet of 8-inch diameter water main	$345	$690,000	$897,000	$1,085,000	$1,248,000
WM-13	Water main upsize from Beaver Dam Trail to Deer Park Drive	290	linear feet of 8-inch diameter water main	$345	$100,000	$130,000	$157,000	$181,000
WM-14	Water main upsize from new Juniper Mountain PS to Kloster Court	240	linear feet of 8-inch diameter water main	$345	$83,000	$108,000	$131,000	$150,000
WM-15	Water main upsize along Kloster Court	570	linear feet of 8-inch diameter water main	$345	$197,000	$256,000	$310,000	$356,000
WM-16	Water main upsize along Juniper Mountain Road	410	linear feet of 8-inch diameter water main	$345	$141,000	$183,000	$222,000	$255,000

Table 9.3 Capital Improvement Project Costs (continued)

Project ID	Project Name	Proposed Amount	Unit	Unit Cost	Baseline Construction Cost (1)	Direct Construction Cost (1)(2)	Total Construction Cost (1)(3)	Total Capital Cost (1)(4)
Water System Capacity Improvements								
WM-17	Water main upsize along Cortina Court	480	linear feet of 8-inch diameter water main	$345	$166,000	$216,000	$261,000	$300,000
WM-18	Water main upsize from Snow Crest Road to Pine Trail	730	linear feet of 8-inch diameter water main	$345	$252,000	$328,000	$396,000	$456,000
WM-19	Water main upsize from R-4 to Alpine Circle Road	4,420	linear feet of 8-inch diameter water main	$345	$1,525,000	$1,983,000	$2,399,000	$2,759,000
WM-20	Water main upsize towards commercial center north of Alpine Meadows Road	350	linear feet of 8-inch diameter water main	$345	$121,000	$157,000	$190,000	$219,000
WM-21	Water main upsize towards recreational area north of Alpine Meadows Road	910	linear feet of 8-inch diameter water main	$345	$314,000	$408,000	$494,000	$568,000
WM-22	Water main upsize along Alpine Meadows Road and Highway 89 towards River Ranch	1,050	linear feet of 8-inch diameter water main	$345	$362,000	$471,000	$569,000	$655,000
WM-23	Water main upsize along Alpine Circle Road	700	linear feet of 8-inch diameter water main	$345	$242,000	$315,000	$381,000	$438,000
WM-24	Water main upsize from Alpine Circle Road towards condominium tennis court	560	linear feet of 8-inch diameter water main	$345	$193,000	$251,000	$304,000	$349,000
WM-25	Water main upsize west of Alpine Circle Road	670	linear feet of 8-inch diameter water main	$345	$231,000	$300,000	$363,000	$418,000
WM-26	Water main upsize towards Tank 5	1,640	linear feet of 10-inch diameter water main	$410	$672,000	$874,000	$1,057,000	$1,216,000
WM-27	Water main upsize along Mineral Springs Trail from John Scott Trail to west end of Snow Crest Road	2,240	linear feet of 12-inch diameter water main	$475	$1,064,000	$1,383,000	$1,674,000	$1,925,000
WM-28	Water main upsize along Snow Crest Road	1,930	linear feet of 10-inch diameter water main	$410	$791,000	$1,028,000	$1,244,000	$1,431,000
WM-29	Water main upsize along Mineral Springs Place	320	linear feet of 8-inch diameter water main	$345	$110,000	$143,000	$173,000	$199,000
			Water System Capacity Improvements Subtotal		$13,030,000	$16,942,000	$20,482,000	$23,548,000
Water System Condition Improvements								
RR-S-01	Tank 2 rehabilitation or replacement	0.1	MG	$2.5	$250,000	$325,000	$393,000	$452,000
RR-S-02	Tank 3 rehabilitation or replacement	0.1	MG	$2.5	$250,000	$325,000	$393,000	$452,000
RR-S-03	Tank 5 rehabilitation or replacement	0.1	MG	$2.5	$250,000	$325,000	$393,000	$452,000
RR-PWL-01	Ongoing water service lateral rehabilitation and replacement	150	each	$3,600	$540,000	$702,000	$849,000	$977,000
			Water System Condition Improvements Subtotal		$1,290,000	$1,677,000	$2,028,000	$2,333,000

Table 9.3 Capital Improvement Project Costs (continued)

Project ID	Project Name	Proposed Amount	Unit	Unit Cost	Baseline Construction Cost (1)	Direct Construction Cost (1)(2)	Total Construction Cost (1)(3)	Total Capital Cost (1)(4)
Wastewater System Condition Improvements								
RR-GM-01	Ongoing gravity main rehabilitation and replacement	10,800	linear feet	$130	$1,404,000	$1,825,000	$2,208,000	$2,540,000
RR-WWL-01	Ongoing wastewater service lateral rehabilitation and replacement – upper laterals	150	each	$2,400	$360,000	$468,000	$566,000	$651,000
RR-WWL-02	Ongoing wastewater service lateral rehabilitation and replacement – lower laterals	150	each	$1,200	$180,000	$234,000	$283,000	$326,000
				Wastewater System Condition Improvements Subtotal	$1,944,000	$2,527,000	$3,057,000	$3,517,000
Miscellaneous Projects								
M-01	Master Plan updates	2	Lump sum	$100,000	N/A	N/A	N/A	$200,000
M-02	SCADA updates	1	Lump sum	$5,000	N/A	N/A	N/A	$5,000
				Miscellaneous Improvements Subtotal	N/A	N/A	N/A	$205,000
				Total	**$16,264,000**	**$21,146,000**	**$25,567,000**	**$29,603,000**

Notes:
(1) All costs are in October 2022 dollars using the Engineering News Record 20-city average construction cost index of 13,175.
(2) The direct construction cost is equal to the baseline construction cost times 130 percent to account for estimating contingencies.
(3) The total construction cost is equal to the direct construction cost plus 15 percent to account for contractor general conditions and another 15 percent to account for contractor overhead and profits. These contingencies are not applied to all project costs.
(4) The total capital cost is equal to the total construction cost plus 30 percent to account for project delivery cost contingencies. These contingencies are not applied to all project costs.
(5) Abbreviations: hp = horsepower; MG = million gallons; SCADA = supervisory control and data acquisition.

9.3 Proposed Implementation Plan

A proposed capital improvement delivery plan was developed to assist ASCWD in implementing capital improvements throughout the planning horizon. The implementation plan reflects current priorities and is subject to change as a result of future assessments and available financing options. Project development, phasing, and implementation all depend on factors such as funding availability, community input, direction from the Board and Long-Range Planning Committee, and changing water and wastewater system conditions that may lead to reprioritization (e.g., system failures that require emergency repairs). Current financing mechanisms may limit ASCWD's ability to implement the improvements according to the outlined schedule; however, the plan can help the District determine how and when to budget for capital improvements.

The CIP was divided into the following implementation phases:

- Phase 1: 2023 – 2027.
- Phase 2: 2028 – 2032.
- Phase 3: 2033 – 2037.
- Phase 4: 2038 – 2042.

Each proposed improvement was allocated to a phase according to its relative importance and urgency. Table 9.4 lists each project's proposed implementation phase, and Table 9.5 summarizes the costs per phase. Phase 1 consists of the following high priority projects and is estimated to cost approximately $2.9 million:

- Rehabilitation of Tanks 2, 3, and 5 (i.e., RR-S-01, RR-S-02, and RR-S-03).
- Planning and design of Juniper Mountain water system improvements (i.e., WM-14, WM-15, WM-16, PS-01, and S-01).
- Ongoing water and wastewater R&R (i.e., RR-PWL-01, RR-GM-01, RR-WWL-01, and RR-WWL-02).
- AMEW Number 1 backup generator (i.e., GW-01).
- SCADA updates (i.e., M-02).

Figure 9.1 shows the annual distribution of CIP costs through the planning period, and Figure 9.2 shows the proposed water system improvements by implementation phase.

Table 9.4 Proposed Capital Improvement Implementation Plan

Project ID	Capital Cost (1)	Proposed Phase (2)	Projected Implementation Timeline		
			Planning and Design Start Year	Construction Start Year	Completion Year
Water System Capacity Improvements					
PS-01	$27,000	2	2026	2030	2030
S-01	$1,085,000	2	2026	2030	2030
GW-02	$254,000	1	2023	2024	2024
WM-01	$601,000	3	2036	2037	2037
WM-02	$1,536,000	4	2037	2038	2038
WM-03	$713,000	4	2038	2039	2039

Project ID	Capital Cost (1)	Proposed Phase (2)	Projected Implementation Timeline		
			Planning and Design Start Year	Construction Start Year	Completion Year
WM-04	$680,000	3	2032	2033	2033
WM-05	$762,000	3	2032	2033	2033
WM-06	$505,000	3	2033	2034	2034
WM-07	$356,000	2	2030	2031	2031
WM-08	$1,699,000	2	2030	2031	2031
WM-09	$1,832,000	2	2031	2032	2032
WM-10	$163,000	3	2035	2036	2036
WM-11	$212,000	3	2035	2036	2036
WM-12	$1,248,000	2	2028	2029	2029
WM-13	$181,000	2	2028	2029	2029
WM-14	$150,000	2	2026	2028	2028
WM-15	$356,000	2	2026	2028	2028
WM-16	$255,000	2	2026	2029	2029
WM-17	$300,000	4	2039	2040	2040
WM-18	$456,000	3	2036	2037	2037
WM-19	$2,759,000	4	2041	2042	2042
WM-20	$219,000	4	2040	2041	2041
WM-21	$568,000	4	2040	2041	2041
WM-22	$655,000	4	2039	2040	2040
WM-23	$438,000	4	2039	2040	2040
WM-24	$349,000	4	2040	2041	2041
WM-25	$418,000	4	2040	2041	2041
WM-26	$1,216,000	4	2038	2039	2039
WM-27	$1,925,000	3	2035	2036	2036
WM-28	$1,431,000	3	2034	2035	2035
WM-29	$199,000	4	2040	2041	2041
Water System Condition Improvements					
RR-S-01	$452,000	1	2025	2027	2027
RR-S-02	$452,000	2	2025	2028	2028
RR-S-03	$452,000	1	2025	2026	2026
RR-PWL-01	$977,000	All	Ongoing	Ongoing	Ongoing
Wastewater System Condition Improvements					
RR-GM-01	$2,540,000	All	Ongoing	Ongoing	Ongoing
RR-WWL-01	$651,000	All	Ongoing	Ongoing	Ongoing
RR-WWL-02	$326,000	All	Ongoing	Ongoing	Ongoing

Project ID	Capital Cost (1)	Proposed Phase (2)	Projected Implementation Timeline		
			Planning and Design Start Year	Construction Start Year	Completion Year
Miscellaneous Improvements					
M-01	$200,000	2 and 4	2032 and 2042	N/A	N/A
M-02	$5,000	1	2024	N/A	N/A

Notes:
(1) All costs are in October 2022 dollars using the Engineering News Record 20-city average construction cost index of 13,175.
(2) The proposed phase corresponds with the year in which construction begins.

Table 9.5 Capital Project Costs by Implementation Phase

Phase	Implementation Timeframe	Total Capital Cost (1) (million dollars)
Phase 1	2023 – 2027	$2.9
Phase 2	2028 – 2032	$8.9
Phase 3	2033 – 2037	$7.9
Phase 4	2038 - 2042	$10.6

Notes:
(1) All costs are in October 2022 dollars using the Engineering News Record 20-city average construction cost index of 13,175.

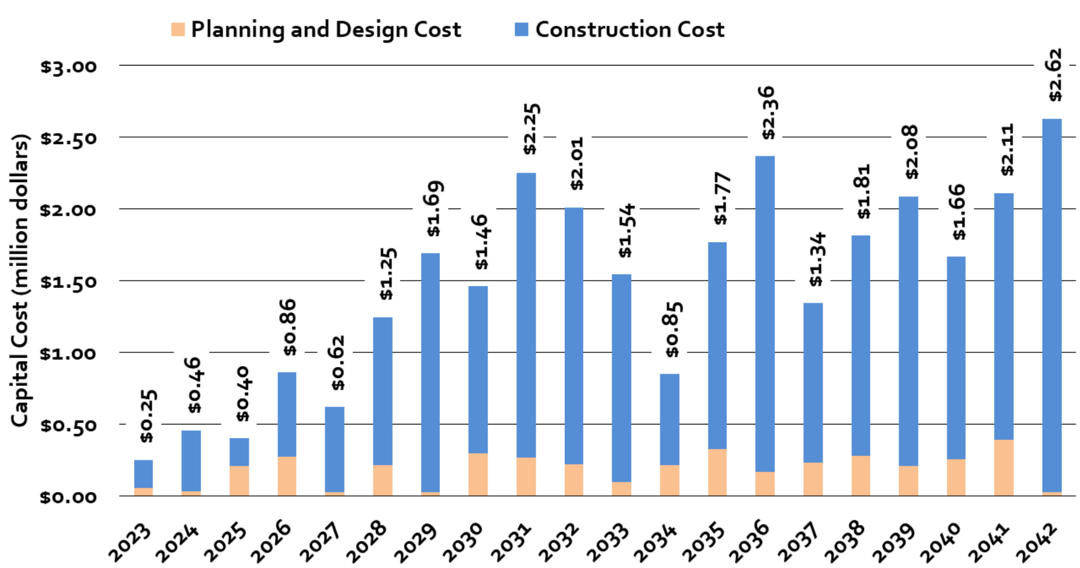


Figure 9.1 Annual Distribution of Capital Improvement Plan Costs

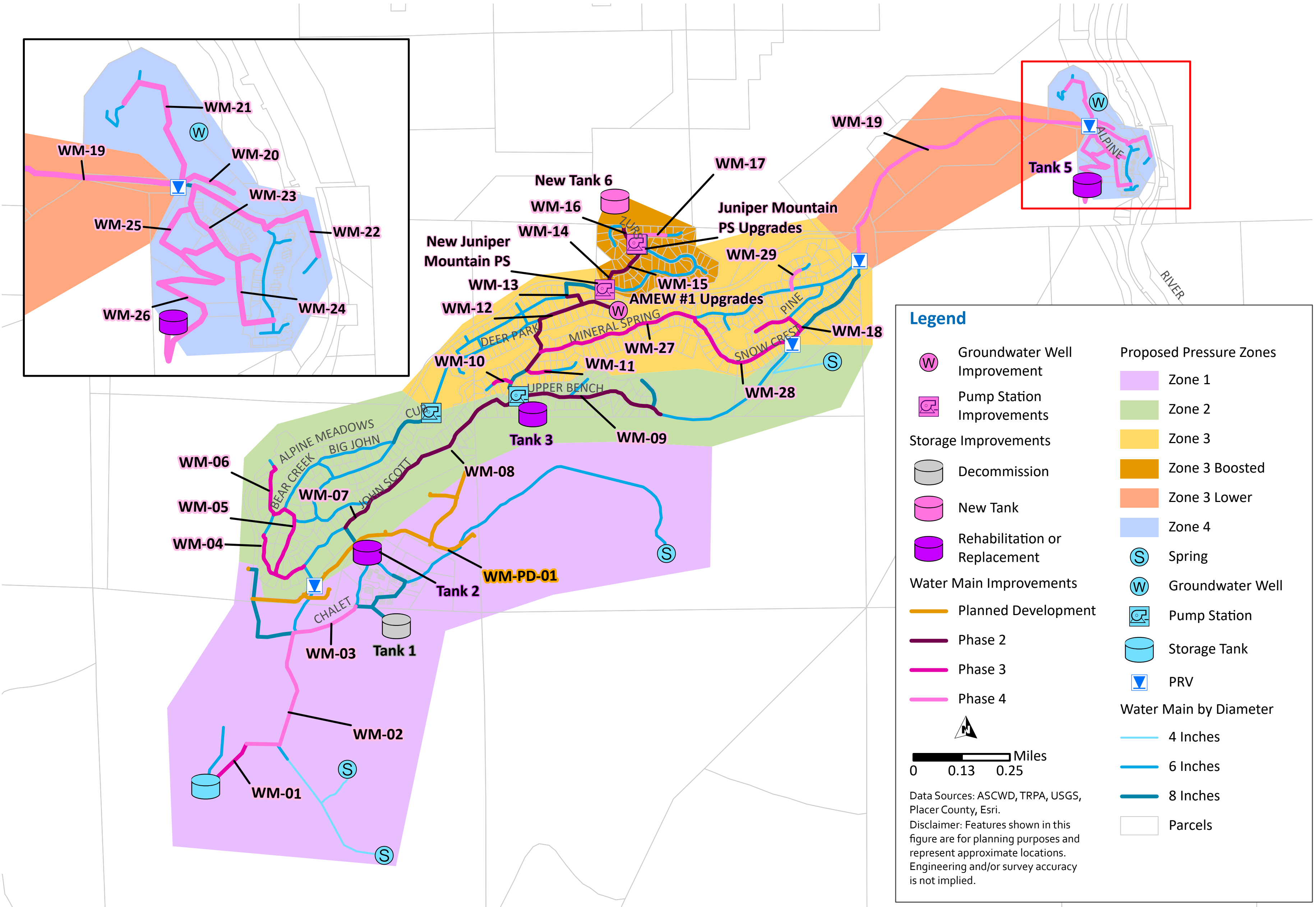


Figure 9.2 Proposed Water System Improvements by Implementation Phase

9.4 Capital Improvement Plan Summary

ASCWD's CIP is designed to address water and wastewater system needs through the2045 planning horizon. The CIP's total capital cost was estimated to be approximately $29.6 million in 2022 dollars, or about $1.5 million per year over 20 years.

The CIP was divided into four phases to help the District implement the proposed projects. Projects were allocated into phases according to relative importance and urgency. High priority projects in the first phase are estimated to cost approximately $2.9 million.

Appendix 3A

WATER MODEL EXTENDED PERIOD SIMULATION CALIBRATION RESULTS

Figure 3A.1 through Figure 3A.13 show the extended period simulation (EPS) calibration results for each water distribution system facility.

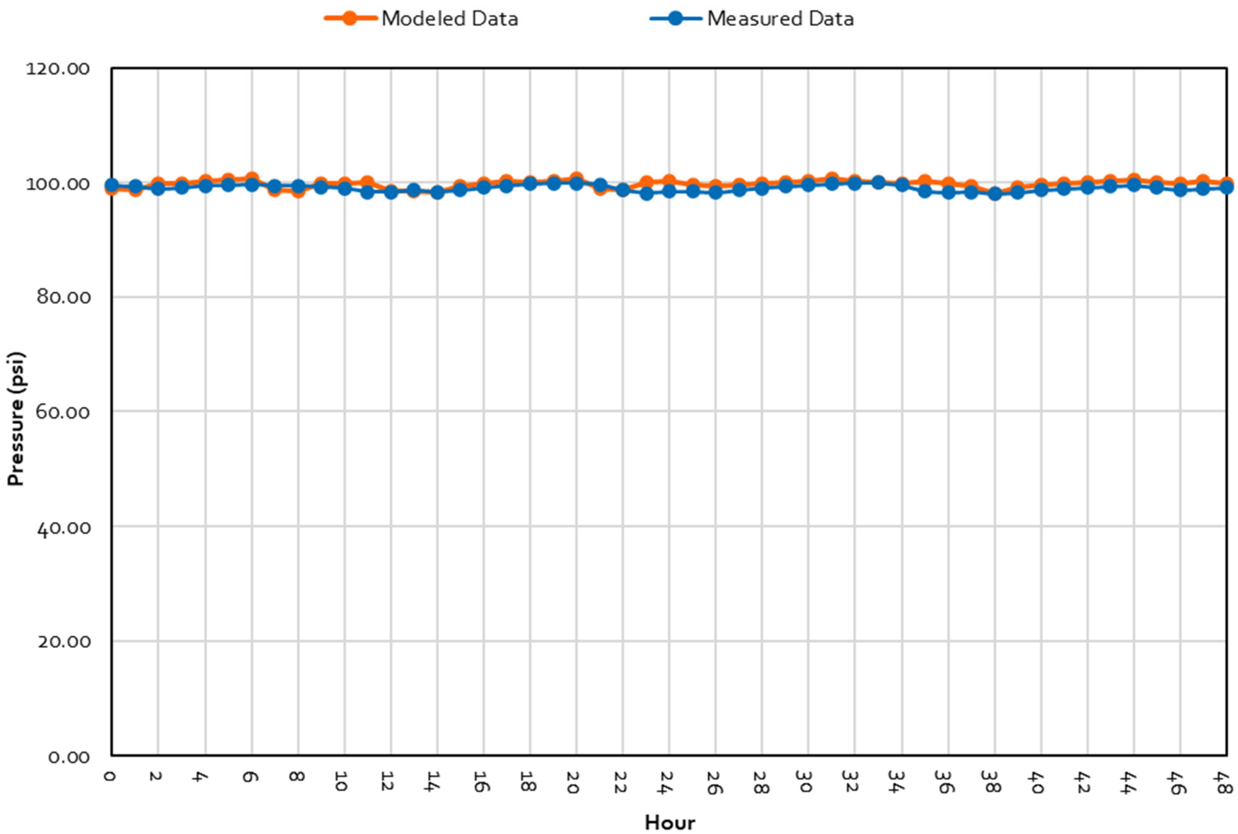


Figure 3A.1 FH01 EPS Calibration Results

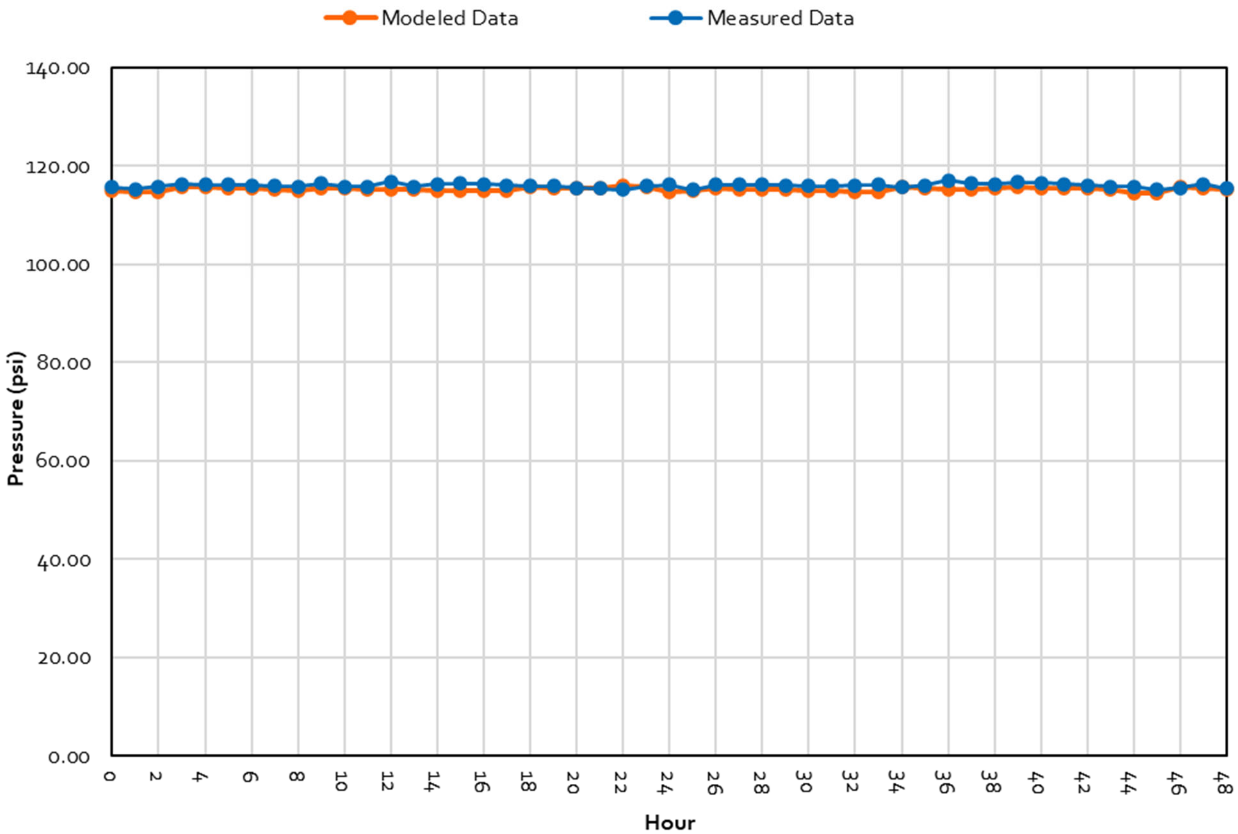


Figure 3A.2 FH02 Pressure EPS Calibration Results

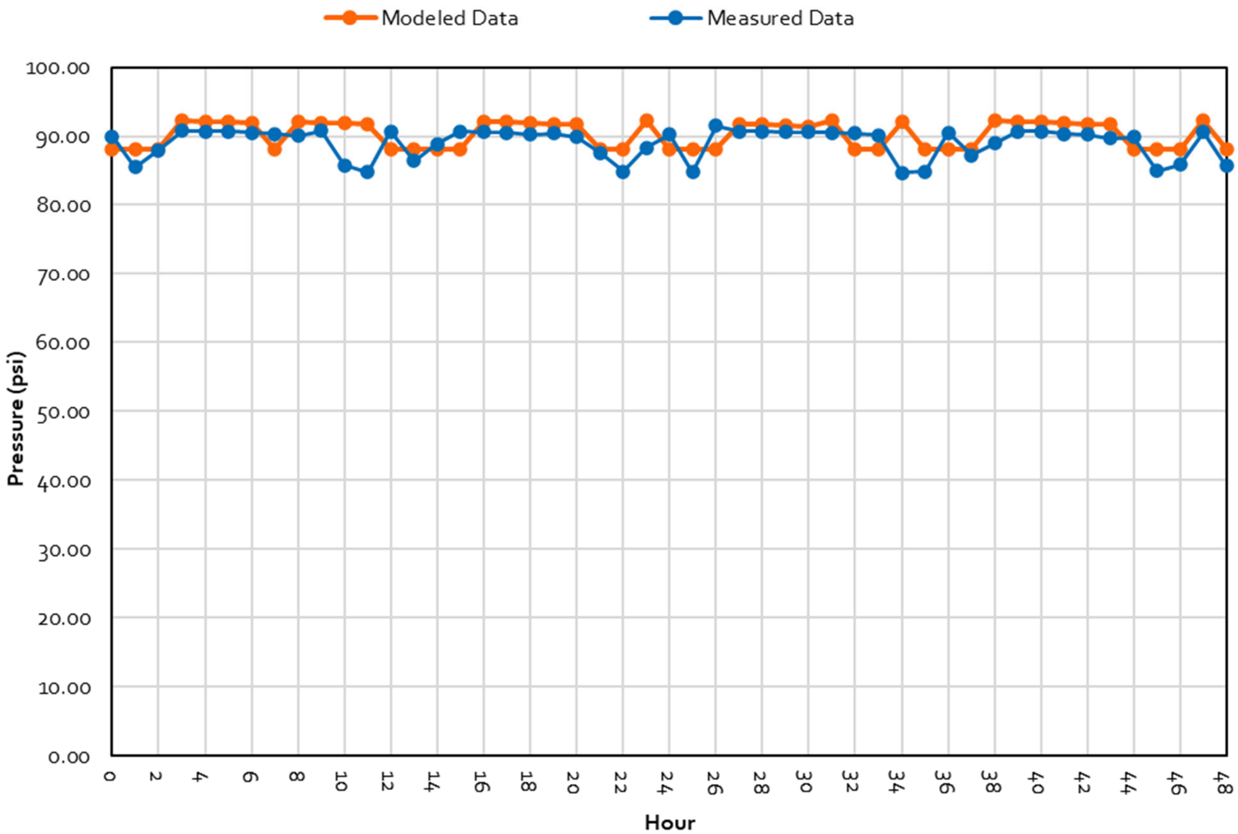


Figure 3A.3 FH03 Pressure EPS Calibration Results

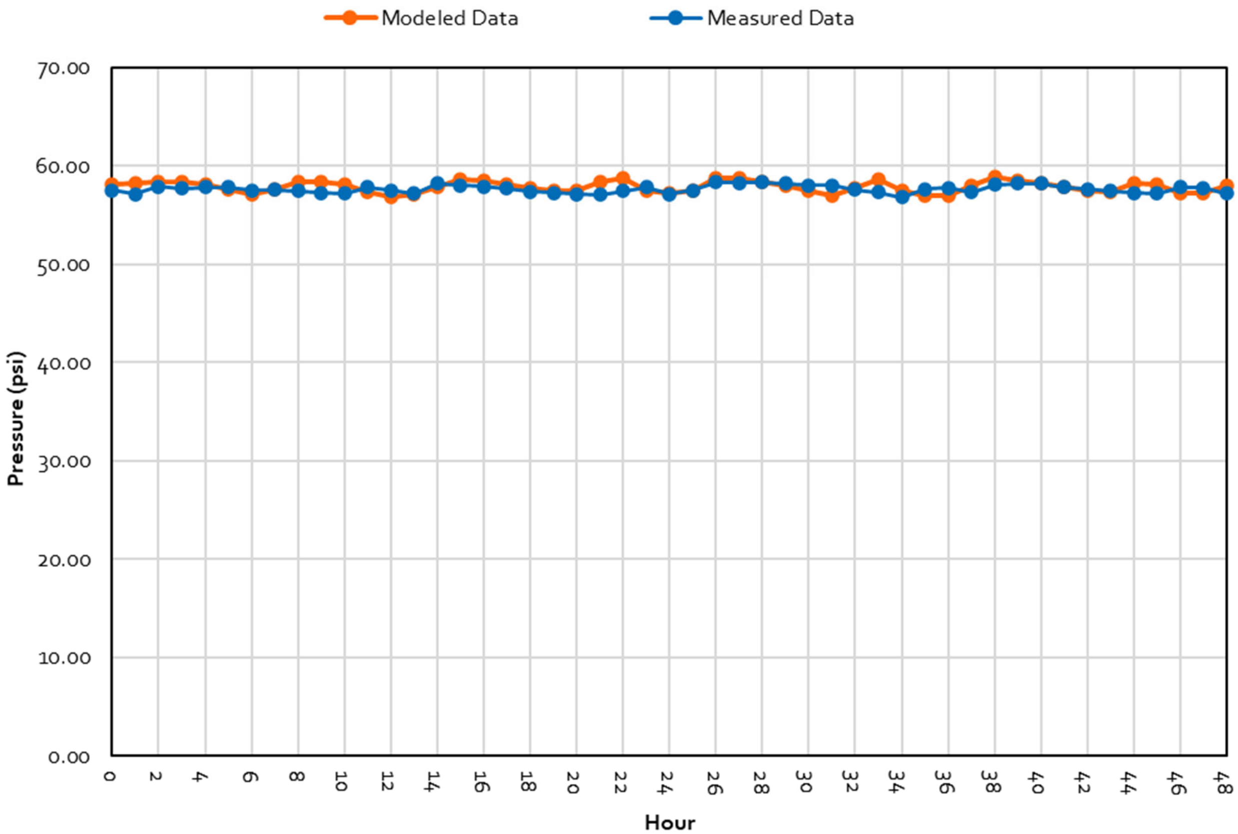


Figure 3A.4 FH04 Pressure EPS Calibration Results

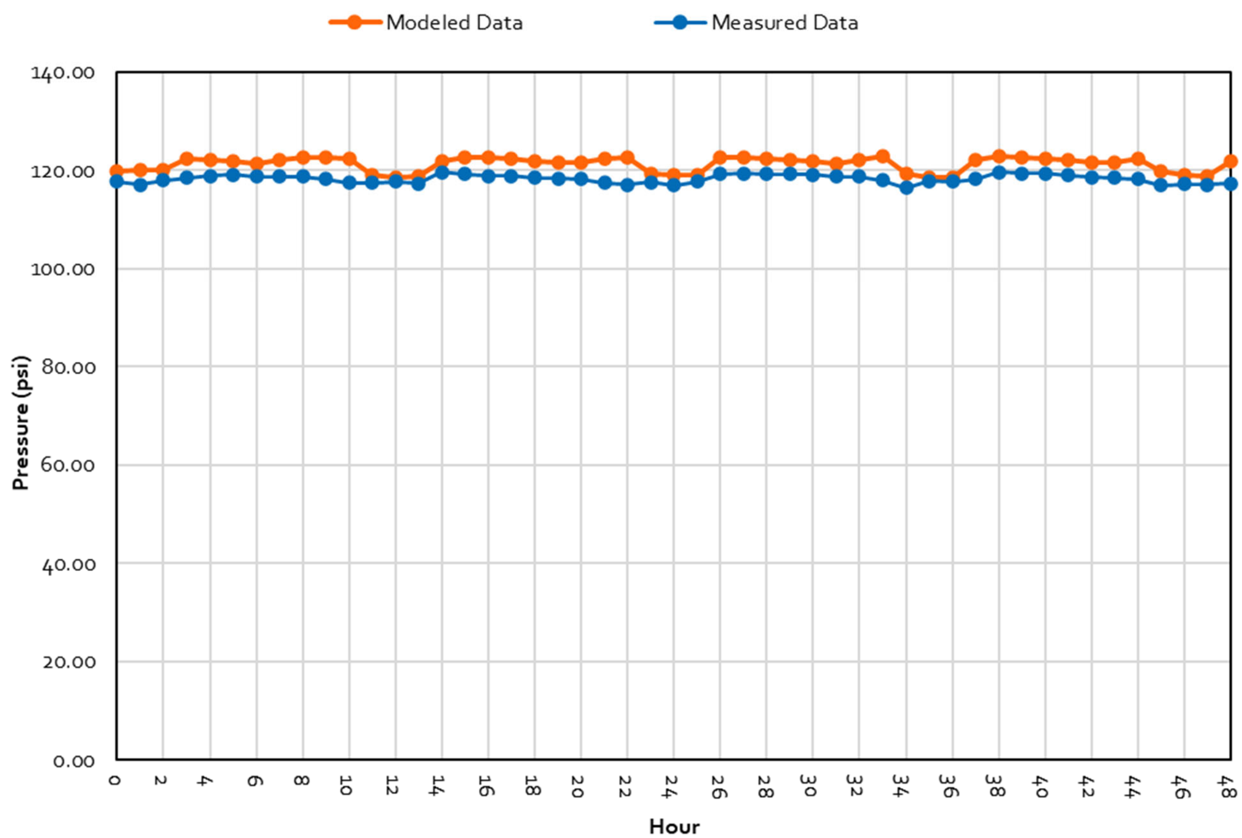


Figure 3A.5 FH05 Pressure EPS Calibration Results

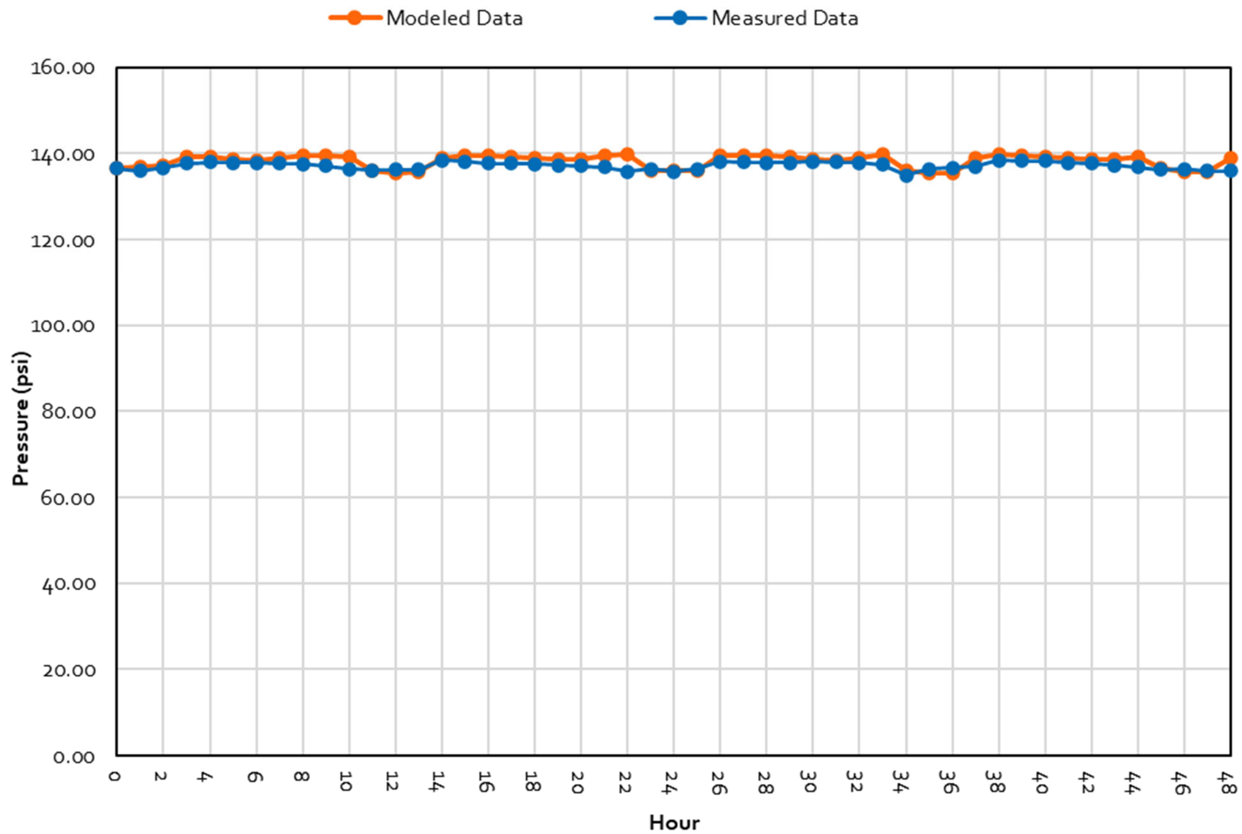


Figure 3A.6 FH06 Pressure EPS Calibration Results

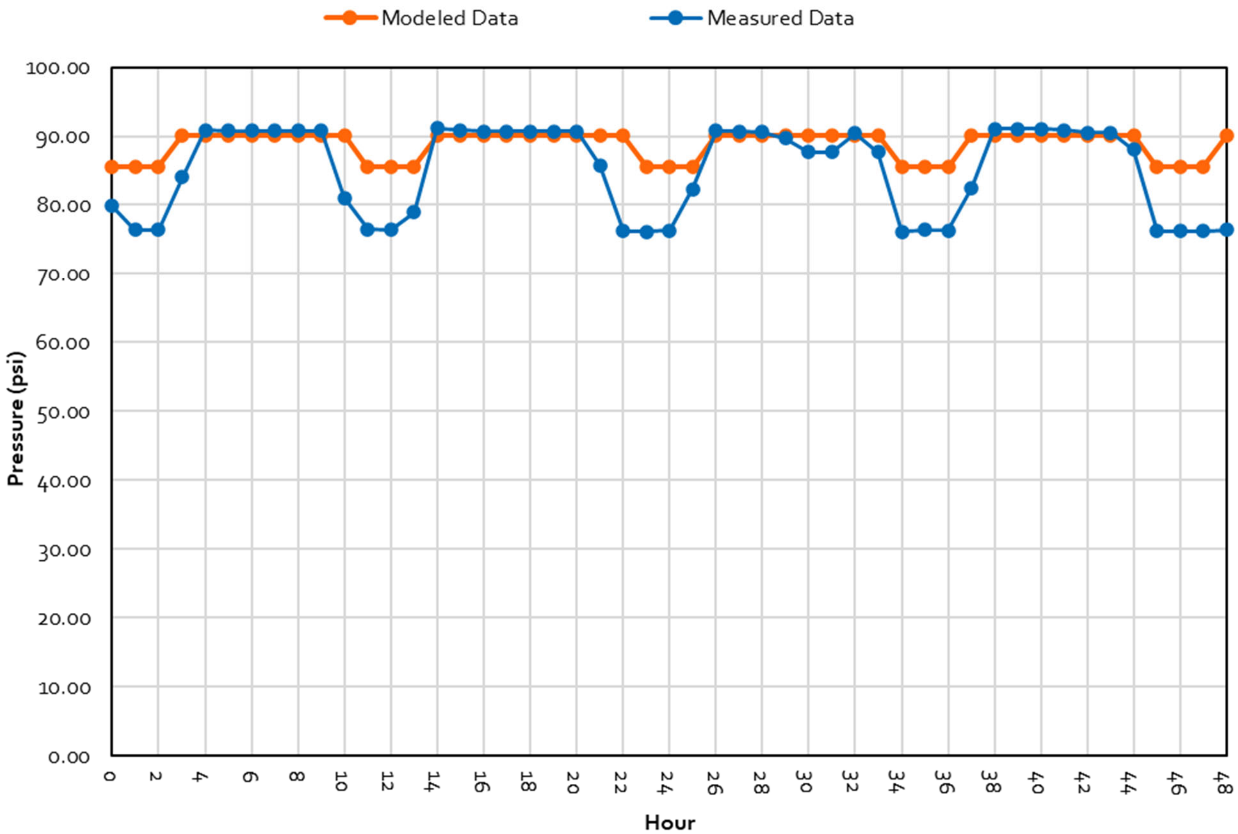


Figure 3A.7 FH07 Pressure EPS Calibration Results

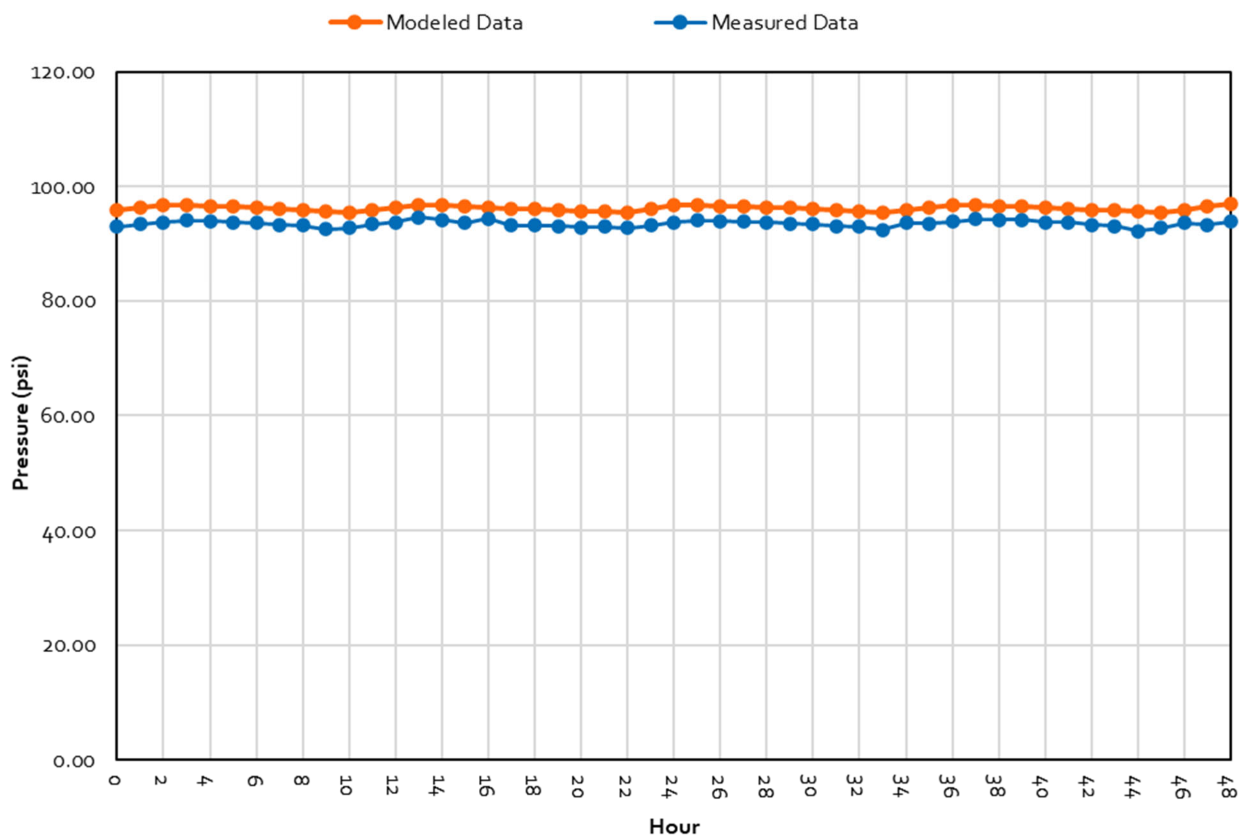


Figure 3A.8 FH08 Pressure EPS Calibration Results

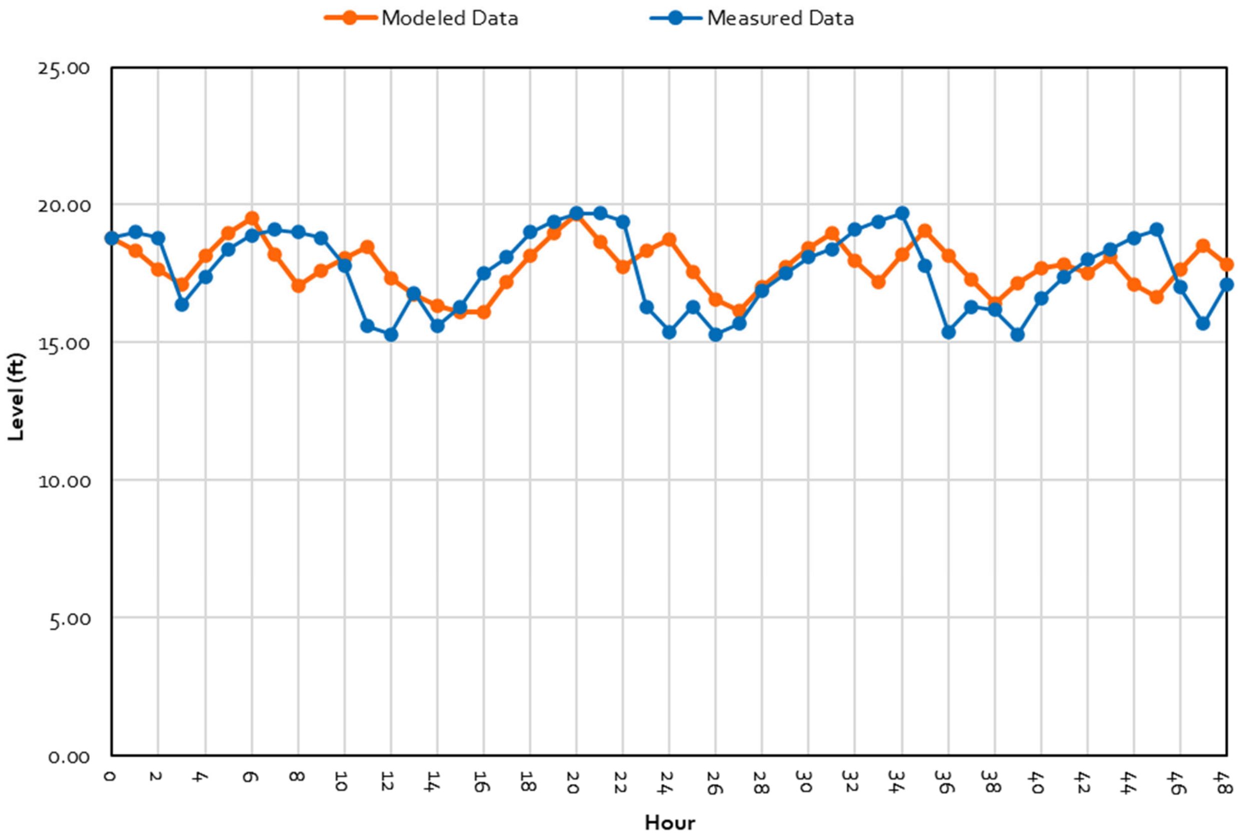


Figure 3A.9 Tank 1 Level EPS Calibration Results

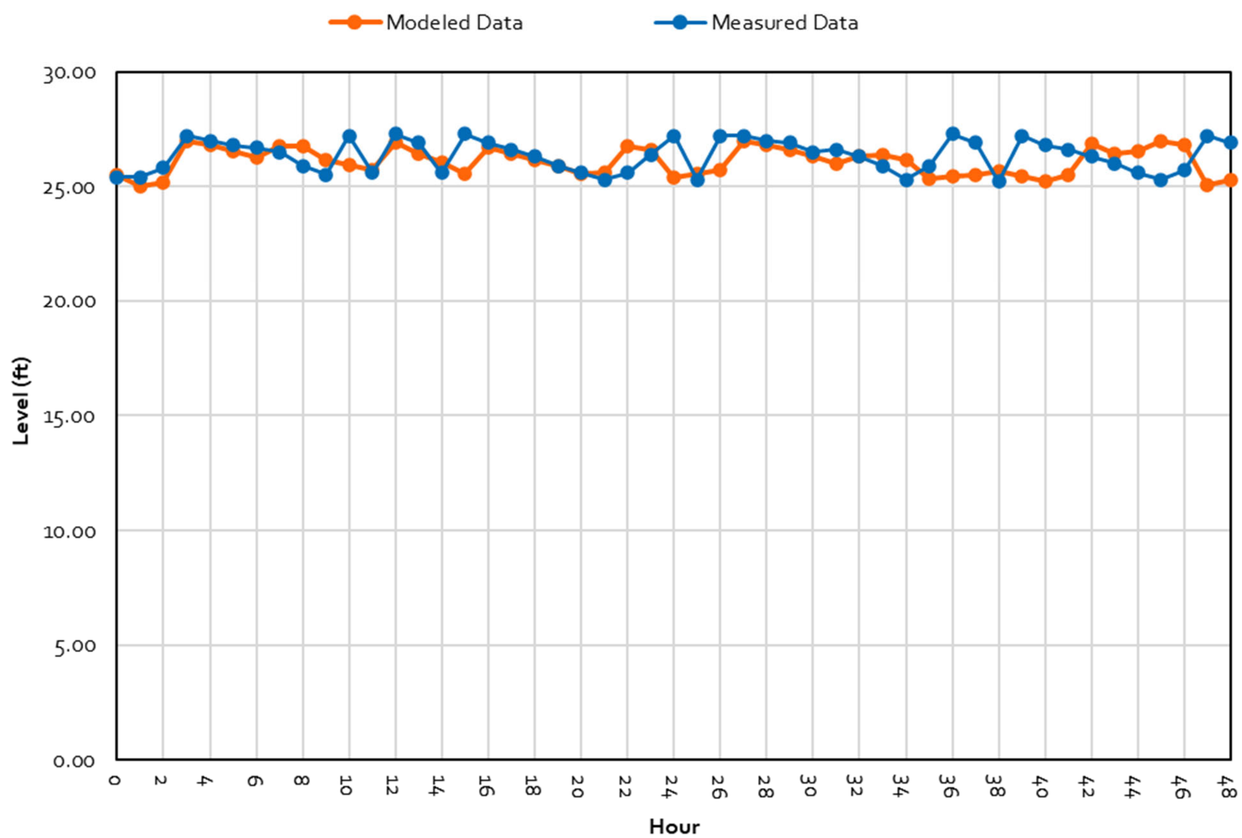


Figure 3A.10 Tank 2 Level EPS Calibration Results

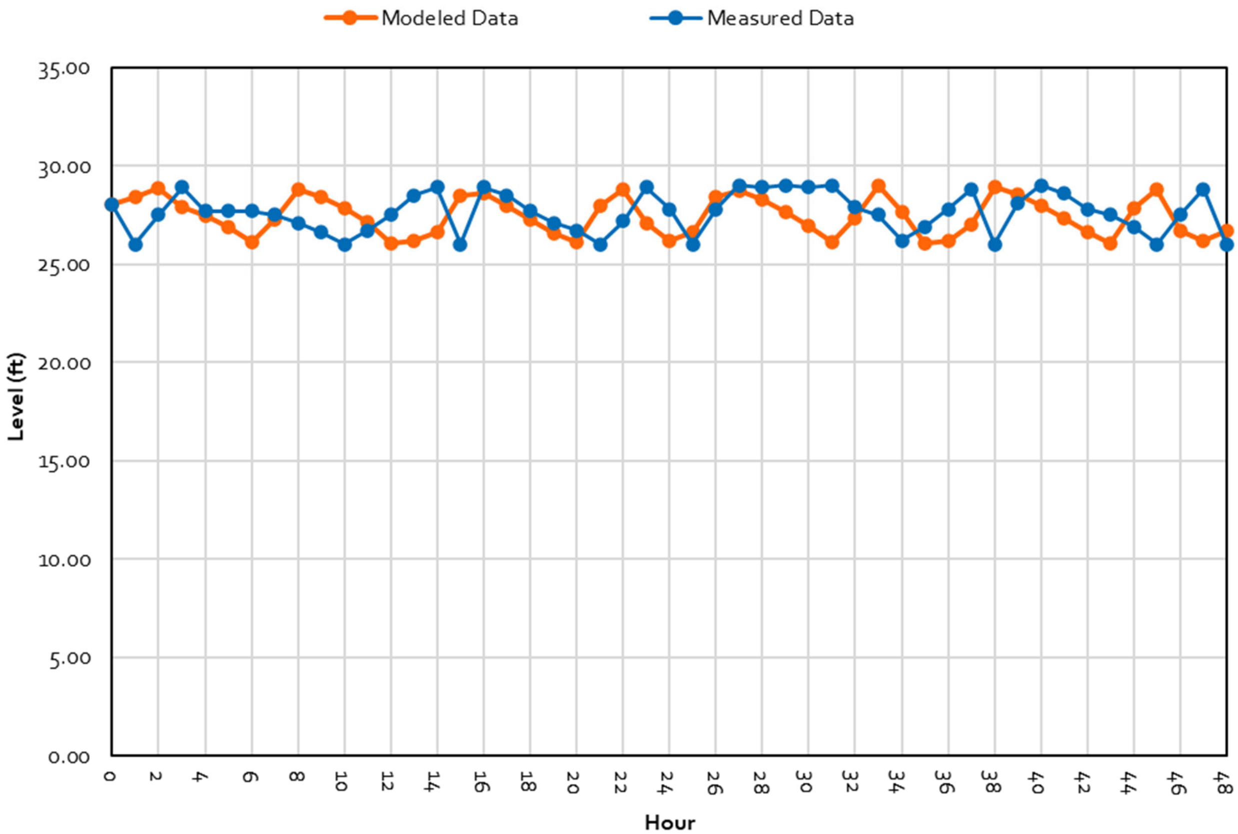


Figure 3A.11 Tank 3 Level EPS Calibration Results

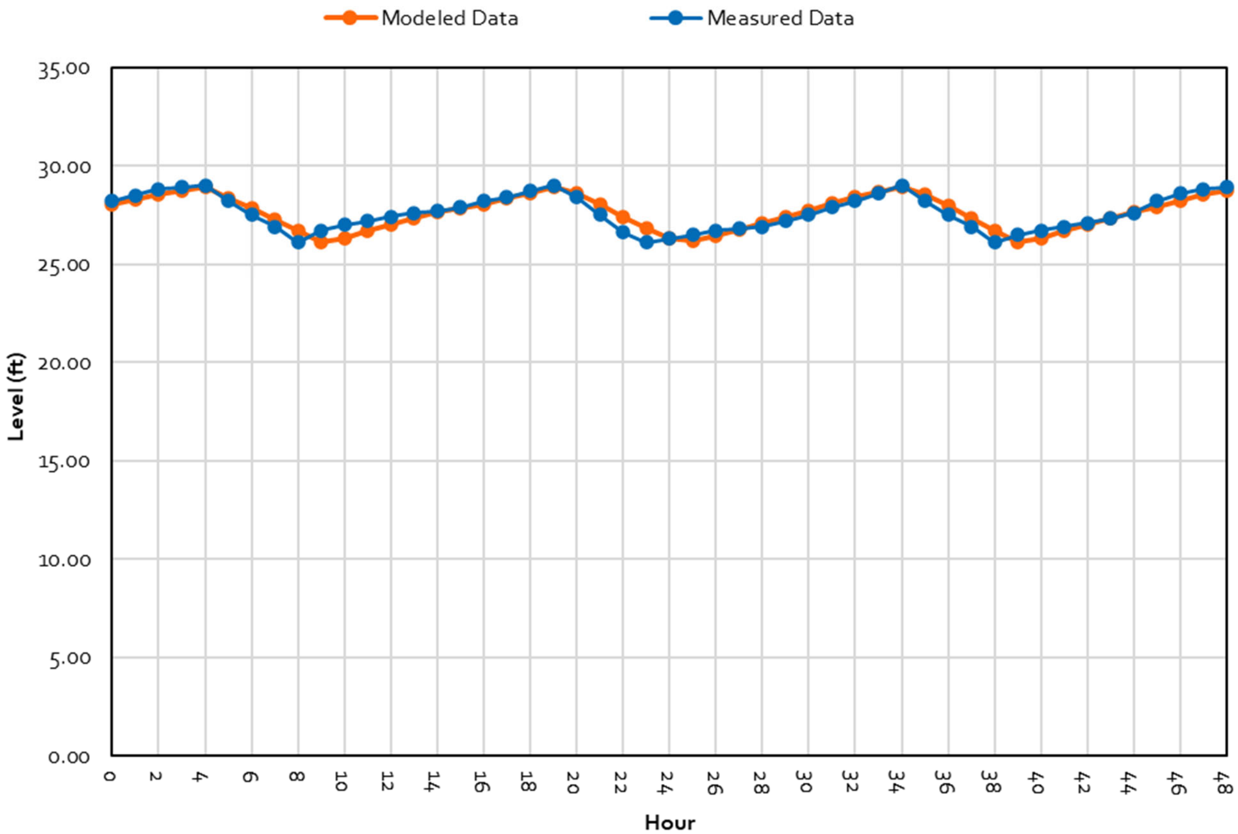


Figure 3A.12 Tank 4 Level EPS Calibration Results

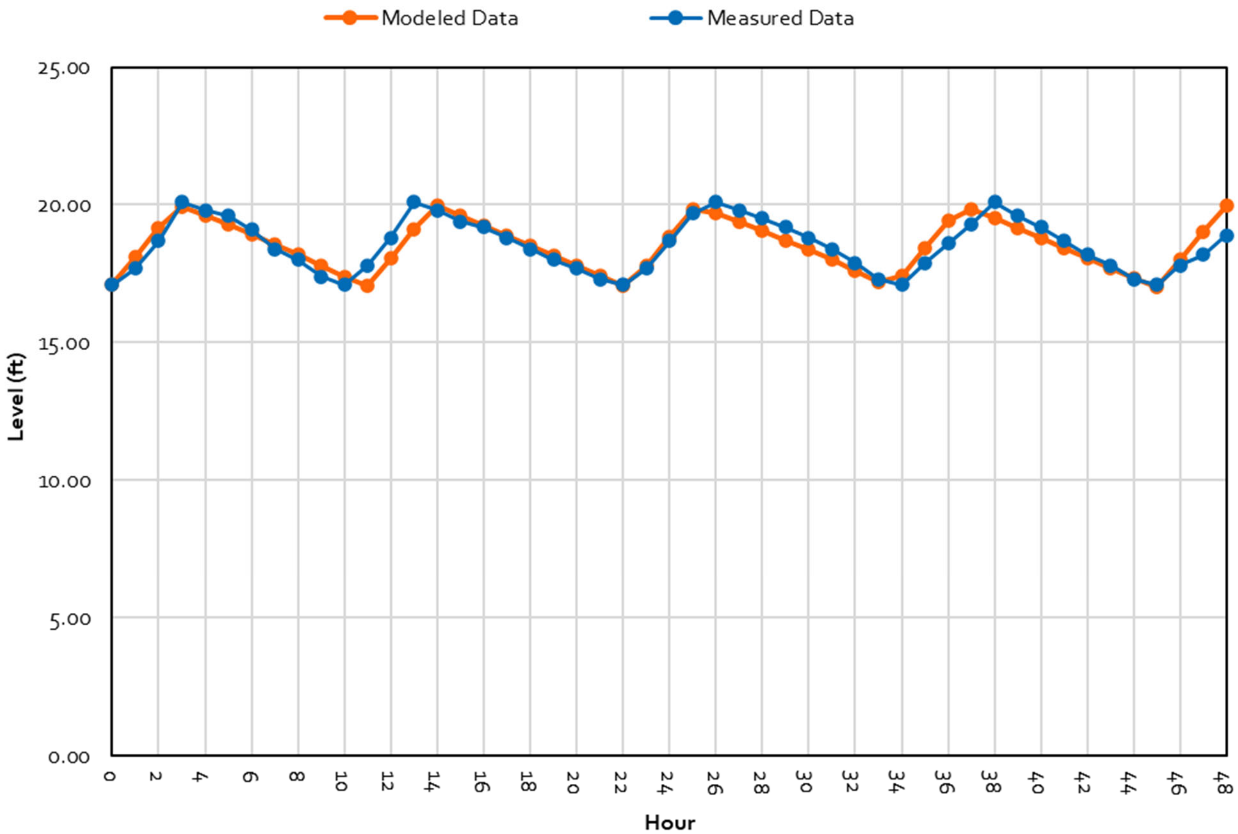


Figure 3A.13 Tank 5 Level EPS Calibration Results

Appendix 4A

SITE VISIT NOTES

MEETING MINUTES

WATER AND WASTEWATER MASTER PLAN

Alpine Springs County Water District

Issue Date: August 5, 2022

Project No.: 200859

Purpose:	Master Plan Site Visit	
Meeting Date:	July 26, 2022	
Meeting Location:	Alpine Springs County Water District Office and Field	
Prepared By:	Carollo Team	
Attendees:	**ASCWD:**	**Carollo:**
	Miguel Ramirez	Julia Semmens
	Rob (unknown last name)	Ryan Orgill
Distribution:	Attendees, Joe Mueller, Tim Loper, Coral Taylor, Andy Burton	

Discussion:

The following is our understanding of the subject matter covered in this conference. If this differs from your understanding, please notify us.

Meeting Purpose

The purpose of this meeting was to visually inspect ASCWD's water system facilities and to transfer operational knowledge on the water and wastewater systems to the Carollo team.

Site Visit

The project team toured most of the water system's major facilities, including each storage tank, most of the valve boxes, the Juniper Mountain booster pump station (BPS), the Alpine Meadows Estates Well (AMEW) #1, and a snowmaking facility that is maintained and operated by Palisades Tahoe. Hydraulic and condition information related to each tank are shown in Table 1 and Table 2, respectively. Additional discussion is summarized below:

- The team toured the snowmaking facilities and discussed past and current operations.
  - The snowmaking facilities consist of the following, each of which is owned and maintained by Palisades Tahoe:
    - Three ponds located adjacent to the Palisades Tahoe parking lot in Zone 1.
    - A snowmaking well called R1 that is located at the bottom of the system in Zone 4.
    - A 12-inch diameter transmission main that transmits water from the R1 well to the snowmaking ponds via a well pump.
    - A pump station building adjacent to Pond 1 that pumps water from the ponds to snow guns within the ski resort.
      - This building houses ASCWD's valve that controls how much water flows into the ponds from Tank 4.
    - Intake pipes in Pond 1 and pipes between each pond that run under the parking lot.

  - The ponds are filled by Tank 4 overflow and snowmaking wells at the bottom of the system in Zone 4.
  - The ponds often are completely drained during peak snowmaking season, which is approximately late October through January. It was noted that the ponds previously provided habitat for various fish species, but frequent draining of the ponds decimated the ponds' fish populations and reduced or eliminated populations in the downstream Bear Creek.
- A bypass was constructed between Zones 1 and 2 in 2017.
  - The bypass consists of 6-inch diameter piping and two control valves. One valve allows a small amount of flow from Zone 1 to Zone 2 under typical daily operations, and the other larger valve acts as an emergency interconnection and is typically closed.
  - The bypass runs from Chalet Road to John Scott Trail along a gravel maintenance road. A gravity sewer main also runs under the maintenance road.
- The AMEW #1 is located in Zone 3 at the east end of Beaver Dam Trail.
  - The well was constructed in 2015. It was designed by Stantec and constructed by Tim Longo. Eric Zindel completed the electrical work.
  - The well was designed at too high a head for the system, which has contributed to cracking in the pavement around the well. The excessive pressure is likely also causing the well's components to degrade at a faster rate.
  - The variable frequency drives (VFDs) have never worked.
  - The well is typically not utilized. ASCWD pumped from the well only for about a month in 2020 when demands were abnormally high due to the Covid-19 pandemic. Staff flushes the well about twice a month. Staff runs the well for about an hour during flushing exercises to clear out rust buildup.
  - Due to rust buildup, water is directed towards a riprap-lined settling pond adjacent to the well building during flushing. The corrosive water settles through the pond to remove rust and other contaminants before flowing into Bear Creek.
  - The well is about 525 feet deep and is entirely full. There is virtually no drawdown when it is pumped.
  - Pumps at 220 gpm when running.
  - Staff noted that the well has enough supply to feed the entire system if necessary, but ASCWD would need to install booster pumps to pump up to Zones 2 and 1.
  - Staff noted that the well overflows in the winter, which causes the area around the well building to become icy. This causes extreme safety issues both for staff and for members of the public who pass the building to use the trails system.
  - It was discussed that several trees had to be removed when the well was constructed.
- The Juniper Mountain BPS, which boosts pressures in Zurs Court within Zone 3, is located in a vault under the road southeast of Juniper Mountain Road and Cortina Court.
  - Fire hydrant tests on Zurs Court have caused issues with the BPS. The check valve on the bypass pump locks open during tests, and staff must manually close it. ASCWD has discussed with the fire department that hydrant valves need to be opened slowly to prevent the check valve from malfunctioning.
  - The pump was last replaced in 2005.
  - There is no standby pump.
  - The pump operates continually to boost pressures in Zurs Court.

- The system is supplied primarily by four springs. Springs 1, 2, and 4 are all located in Zone 1, and Spring 3 is located in Zone 3. None of the springs were toured during the site visit due to their relatively large distance from the system. ASCWD is sending photos of the spring casings for Carollo to review.
  - Spring 3 operations were discussed while visiting the R3 valve box, which feed Zone 3 from Zone 2 and Spring 3. Spring 3 flows at about 19 gpm in the summer and 12 gpm in the winter. The upstream pressure at R3 was about 120 psi during the visit, and the downstream pressure was about 85 psi.
- The pressure zones were discussed during the visit. The zone between Zones 3 and 4 is currently unnamed. For the purposes of the Master Plan, this zone will be referred to as "Zone 3 Lower".
- It was discussed during the site visit that the storage tanks are the water system's most condition-deficient assets. The Master Plan will develop a prioritization plan for replacing the tanks.
  - It was discussed that some tanks may not be worth replacing and could potentially be decommissioned. The hydraulic model will help determine whether decommissioning any tanks is hydraulically acceptable.

Table 1 Storage Tanks Hydraulic Summary

Tank Name	Pressure Zone	Base Elevation (ft)	Diameter (ft)	Height (ft)	Typical Operating Level Range (ft)	Capacity at Maximum Operating Level (MG)	Operational Notes
Tank 1	Zone 1	7,038	30	25	15 to 20	0.106	• Level controls Zone 1 HGL.
Tank 2	Zone 2	6,837	25	30	25 to 27	0.099	• Filled by an altitude valve from Zone 1 called A1. • A bypass line was constructed in 2017 to allow Zone 2 to pull directly from Zone 1.
Tank 3	Zone 3	6,646	25	30	26 to 29	0.106	• Filled by an altitude valve from Zone 2 called B-1. • Zone 3 can also be fed from Zone 2 via a PRV called B-2 at the end of Cub lane, but this is normally closed and acts as an emergency interconnection.
Tank 4	Zone 1	7,042	67	38.5	26 to 29	0.765	• Low level is controlled by Tank 1 HGL. • Flows at 400 gpm to snowmaking ponds when level exceeds 29 feet and turns off when level drops below 26 feet. • ASCWD will sometimes send more water to the snowmaking ponds from Tank 4 during peak snowmaking season.
Tank 5	Zone 4	6,385	30	25	17 to 20	0.106	• Filled by an altitude valve from the transmission zone to Zone 4 called R4.

Table 2 Storage Tanks Condition Summary

Tank Name	Year Built	Material	Condition Notes
Tank 1	1963	Concrete	• Appears to be in worst condition relative to other tanks. • Major cracking and leaks. • Cracks are regularly patched but still leak. • Retaining wall above tank is eroded, no riprap. • Tank's ladder was removed several years ago to prevent people from climbing the tank.
Tank 2	1963	Concrete	• Major cracking but not as bad as Tanks 1, 3, and 5. • Altitude valve A1 that feeds tank is new. • Altitude valve box is covered in graffiti.
Tank 3	1963	Concrete	• Major cracking and leaks. • Similar condition as Tank 2. • Transducer under tank is failing and will be replaced with lines on the top of the tank. • Lines under tank are corroding.
Tank 4	2019	Welded steel	• Required emergency replacement after original tank from 1960s failed. • Original paint job was poor and ASCWD had contractor redo outer painting. • Welds are already rusting; ASCWD suspects poor coating.
Tank 5	1963	Concrete	• Major cracking and leaks.

Attachments

None.

Appendix 4B

CONCRETE TANK CONDITION ASSESSMENT

Background

The Alpine Springs County Water District (ASCWD) has five water storage tanks with capacities ranging from 0.100 million gallons (MG) to 0.910 MG. Tank 4 is a welded steel tank which was constructed in 2019 and has a capacity of 0.910 MG. All other tanks are either conventionally reinforced concrete or prestressed concrete tanks built in the early 1960s. Carollo performed a site visit ASCWD's facilities on July 26, 2022. All the water storage tanks were also visited, and visual observation of the tanks was performed from the exterior. It was discussed during the site visit that the storage tanks are the water system's most condition-deficient assets.

Concrete Tanks Condition Assessment

The concrete tanks are 25 feet (ft) to 30 ft in diameter and height as shown in Table 1 below. The construction materials, methods, and condition of the foundation and the base for these tanks is currently unknown. Some of these tanks seem to be prestressed concrete tanks, based on the exterior tank labels. Based on the manufacturer's name, i.e., Crom Corporation, the tanks are wire and strand-wound prestressed concrete meeting American Water Works Association (AWWA) D110 standards. There is visual evidence of multiple closely spaced and long cracks in the concrete walls.

The concrete tanks were designed in the early 1960s and most likely they do not meet current seismic codes. If the hydrostatic and current code seismic demands are higher than the tanks' capacity to withstand seismic demands, then the tanks will need to be retrofitted. To determine the seismic capacity of the existing tanks, the material properties need to be investigated. The material properties include concrete compressive strength, reinforcing steel size, spacing, and yield strength. For reinforced concrete tanks, destructive and non-destructive methods can be utilized to obtain material properties. For prestressed concrete tanks, forces in the prestressing steel are very difficult to determine and destructive methods cannot be done without releasing the prestressing forces. Therefore, retrofitting a prestressed concrete tank without knowledge of its seismic capacity can be expensive since very conservative assumptions have to be made.

The following steps are recommended for fully evaluating the conformance of the existing water storage concrete tanks to current applicable codes and then making recommendations to address any deficiencies identified in the evaluation process. Given the age and construction type of these tanks, it is likely that all four concrete tanks would require either extensive retrofitting to come up to current standards or should be replaced.

- Collection and review of data (drawings, geotechnical report(s), specifications, shop drawings [if welded steel, bolted steel, or prestressed], dive reports, previous inspection reports, evaluations, and assessments). This would include contacting tank manufacturers and incorporating any additional information ASCWD has.
- Perform tank diving inspections and additional site visits to perform visual observations from both the exterior and interior of the tank. The site visits shall include determining tank foundation plan dimensions and thickness, concrete strength, base of the tank and scanning for rebar spacings in walls, base, footings, roof of the tank, and all piping connections. Perform destructive testing to determine reinforcing steel bar spacing and concrete compressive strength. These site visits may require temporary shutting down of the tank.
- Perform condition assessment report based on the review of existing data, new tank inspections (and reports), and additional site visits.
- Perform seismic evaluation of the tanks and all connections based on American Concrete Institute (ACI) 350, American Society of Civil Engineers (ASCE) 7, AWWA D110, and other relevant standards.
- Identify deficiencies.

- Develop conceptual mitigation alternatives. These usually include:
  - Lowering the maximum operating level, i.e., tank water level.
  - Strengthening measures.
  - Replacement.
- Prepare planning level cost estimate for all alternatives.

Table 1 Existing Concrete Storage Tanks

Tank Name	Construction Year	Pressure Zone	Base Elevation (ft)	Diameter (ft)	Height (ft)	Typical Operating Level Range (ft)	Capacity at Maximum Operating Level (MG)
Tank 1	1963	Zone 1	7,038	30	25	15 to 20	0.110
Tank 2	1963	Zone 2	6,837	25	30	25 to 27	0.100
Tank 3	1963	Zone 3	6,646	25	30	26 to 29	0.100
Tank 5	1963	Zone 4	6,385	30	25	17 to 20	0.110