SNAKE RIVER WATER DISTRICT

Water System Master Plan

March 17, 2021

AE2S Project No.: P14796-2020-001

I hereby certify that this report was prepared by me or under my direct supervision and that I am a duly Registered Professional Engineer under the laws of the State of Colorado.

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Executive Summary

Public water utilities must continuously plan to identify opportunities and to address system challenges. Water system opportunities and challenges come in many forms, such as;

- population growth,
- increasing water demands,
- aging infrastructure,
- increased regulatory standards and requirements,
- emerging technological trends and technological advancements, and
- effective capital improvements planning.

Master planning provides policymakers and the public with a detailed report on infrastructure needs and the recommended actions to accommodate those needs. Master planning helps establish priorities for the construction and implementation of necessary improvements. Lastly, a master plan can be used as a tool to pursue and support requests for capital improvement funding and adjusting user water rates. The District recognizes that prudent management of annual operation and maintenance budgets, managing investment into its existing assets, optimizing short-term capital improvement expenditures, and maximizing the benefits of long-term capital improvements require a consistent direction for the utility, which can be attained through a robust planning process.

As the District adopts and cycles through the planning process, some uncertainties and changes are expected. The impacts of these changes are best managed through a continued proactive planning approach. Responding to future challenges is most appropriately accomplished through a fluid planning process that enables the District to maintain a clear vision and consistent direction for the water system.

The 2020 Water System Master Plan provides a guide for short-term, near-term, and long-term management of capital improvements for Snake River Water District's water system. The recommended improvements included in the Capital Improvements Plan (CIP) are the basis for planning, financing, designing, constructing, and implementation of solutions to meet the District's water system needs for years to come. To this end, this master plan identifies three new infrastructure projects and prioritizes investment into existing infrastructure replacement and rehabilitation for the next 10-years.

1.0 Introduction and Background

This chapter provides an overview of the Water System Master Plan for the Snake River Water District. The overview includes background information and document the purpose and scope of the master plan.

1.1 Background

The Snake River Water District (District) is a Title 32 special district that provides high quality potable drinking water and fire protection to residents and business in Summit County in the State of Colorado. The District was created in 1982 and provides water to the Keystone Resort area along Highway 6 to the east of the Town of Dillon. The system was created as the result of rapid growth in the service area and was originally a private water system up until its creation as a public water system.

The most recent master plan adopted by the Snake River Water District was completed in 2012. Since then, the District has implemented many of the capital improvement projects identified in the 2012 master plan including a new water treatment plant, advanced metering systems, and backup power systems. The system also experienced considerable development in the service area and has monitored and responded to changing regulatory requirements.

1.2 Project Objectives and Deliverables

Ensuring the responsible management of annual operation and maintenance (O&M) budgets, optimizing short-term capital improvement expenditures, and maximizing the benefits of long-term capital improvements requires a comprehensive direction. To re-establish a vision for the water system, the District retained Advanced Engineering and Environmental Services (AE2S) to prepare this Water System Master Plan.

This Master Plan will inform policymakers and the public on the condition of existing infrastructure, requirements and alternatives of the water system infrastructure, opinions of cost, and the recommended steps to implement the preferred alternatives. Furthermore, this Master Plan will outline implementation of desired improvements within the context of a comprehensive plan to ensure compatibility and prudent management of the water utility.

1.3 Reference Documents

A summary of information provided by the Snake River Water District for this master planning effort is bulleted below:

- Water Utility Master Plan for Snake River Water District, (December 2012), Tetra Tech
- Optimal Corrosion Control Treatment Study for Snake River Water District in Summit County, CO, (March 2018), Tetra Tech
- Snake River Water District Rules and Regulation (revised February 11, 2020)
- Finished Water Storage Tank Inspection Plan (December 2016)
- Facility Drawings for the following facilities:
  - Base 1 Booster Pump Station (BPS) As-Builts (1998)
  - Base 2 Water Treatment Plant (TWP) As-Builts (1997)
  - Base 2 WTP Administrative Addition Design Drawings (1997)
  - Sunrise Tank Construction Drawings (1978)
  - Sunrise Tank Roof Reconstruction Construction Drawings (2006)
  - Schoolmarm Tank Preliminary Drawings (1982)
  - Base 3 WTP Partial Design Drawings (2019)
  - Base 3 WTP Partial As-Builts for Civil Drawings (2020)
- Various water quality sampling results from as early as 2004
- Pump information and curves for pumps throughout the system
- Utility Bills for Base 1 BPS from 2019
- Supervisory Control and Data Acquisition (SCADA) Data per AE2S request.
- Geographic Information System (GIS) Data per AE2S request
- Water Rights Information
- Insurance Services Office (ISO) Report (2015)
- Quarterly Utility Billing Records from 2012 to 2020
- Water Production Records from 2011 to 2020

2.0 Overview of Existing System

This chapter provides an overview of the existing system layout, facilities, naming conventions and operations. The Snake River Water District system is classified as a public water system by the State of Colorado. Throughout this report the Snake River Water District may be referred to as its acronym SRWD or as "District". The District is governed by a board of directors with seven members. Daily management, administration and operations of the water system is contracted to professionals in the community.

The system's service area is bound by United States Forest Service land to the north, east and south while a separate water district starts at the Snake River Water District's west boundary near the Keystone River Course. Figure 2-1 provides a map of the service area.

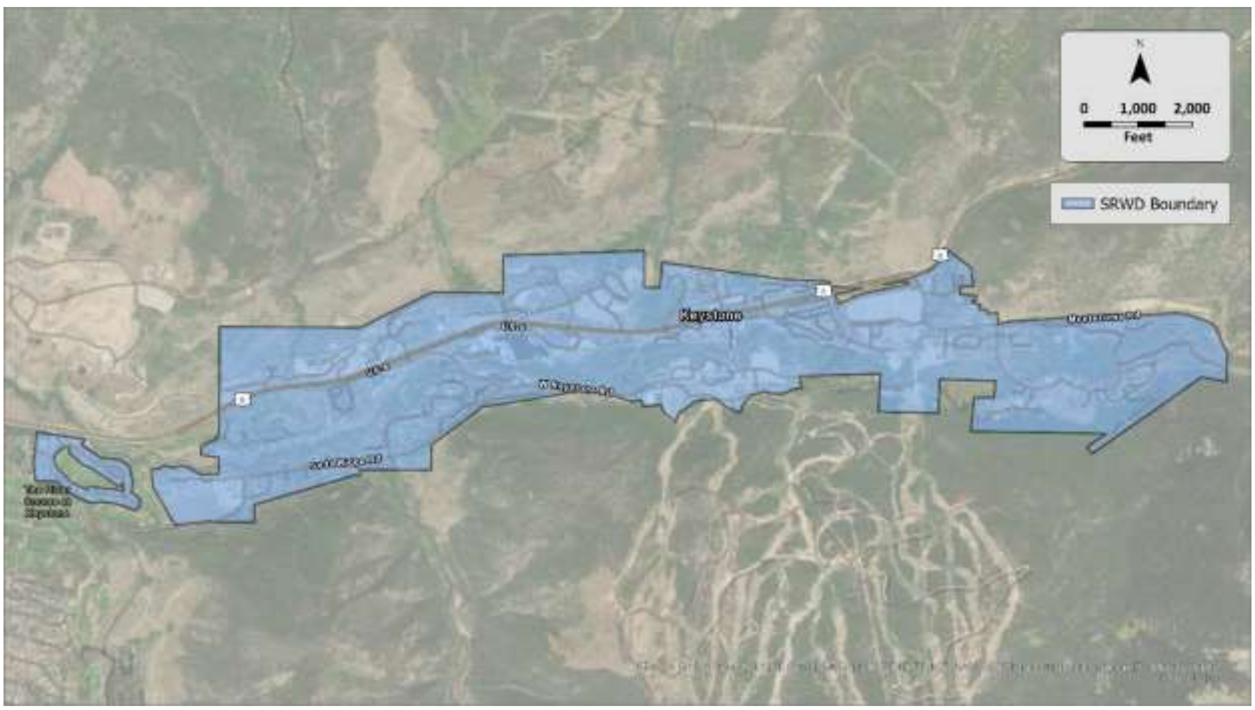


Figure 2-1 – District Water Service Area

Table 2-1 shows the system ID and service population as of October 2020 according to monitoring schedules available on the State of Colorado records. The population value includes transient population such as temporary occupation in hotel rooms and rental properties. The water sources are classified as groundwater. The distribution system is a class 2 system based on Regulation No. 100 authorized by the Colorado Revised Statutes.

Table 2-1 – Public Water System Information

System Name	Public Water System ID	Population
Snake River Water District	**CO0159105**	**9,900**

While the general scope of this master plan covers the production, storage and distribution of water, the brief overview of the system from a regulatory and high-level operational view provides a good background for the remaining report.

2.1 Water Sources and Treatment

This section describes the existing water sources and treatment techniques used at the District's facilities.

2.1.1 Water Treatment Objectives

As a public water system, the District is required to meet the regulations of the Safe Drinking Water Act (SDWA) which are enforced by the state's primacy agency, the Colorado Department of Public Health and Environment (CDPHE). The SDWA was first passed by the U.S. Congress in 1974 to establish a uniform set of regulations and standards for drinking water systems across the United States. The standards have been revised and amended many times since 1974 to add or modify items such as treatment rules, testing requirements, and maximum contaminant levels of common constituents in water among other advisory objectives and parameters. The main water quality standards the District is required to meet are the Primary Drinking Water Standards. These standards include enforceable limits on various chemicals present in water and treatment rules. All of the Safe Drinking Water Act rules are incorporated into the state level Regulation No. 11 – Colorado Primary Drinking Water Regulations.

Table 2-2 – Primary Drinking Water Rules

Rule Name	Requirements

Ground Water Rule	• Requires disinfection in groundwater systems
	• Requires sanitary surveys to be conducted by the primacy agency (CDPHE) intended to identify significant deficiencies
	• Requires compliance monitoring to ensure 4-log (99.99 percent) inactivation or removal of viruses via disinfection
	• For system that do not disinfect they're required to:
	  o conduct hydrogeologic sensitivity analysis for non-disinfected system
	  o conduct source water monitoring for microbes in systems with sensitive aquifers
	• Take corrective action with deficiencies and positive microbial samples.

Table 2-2 – Primary Drinking Water Rules Continued

Rule Name	Requirements
Lead and Copper Rule	• Established monitor and testing requirements for lead and copper at consumer taps • Established action levels for exceedance of lead and copper concentrations in drinking water. • Requires public education if the lead action level is exceeded • Requires corrosion control treatment for all large systems and study for small systems
Total Coliform Rule and Revised Total Coliform Rule	• Established microbiological standards and monitoring requirements

Stage 1 Disinfectants-Disinfection By-Products Rule	• Established MCLs for 11 disinfection by-products (DBP)
	  o Four trihalomethanes (THM), five haleoacetic acids (HAA5), chlorite and bromate
	• Established maximum residual disinfectant levels (MRDL) and goals (MRDLG) for three disinfectants
	  o Chlorine and Chloramine both at 4.0 mg/L
	  o Chlorine Dioxide at 0.8 mg/L

Stage 2 Disinfectants-Disinfection By-Products Rule	• Requires system to complete an Initial Distribution System Evaluation to identify locations of with high DBP concentrations. The system then uses these locations and sampling site for compliance monitoring. • Requires systems to determine if they've exceeded an MCL set in the Stage 1 rule.
Volatile Organic Chemicals Rule	• Established MCLs for 8 volatile organic compounds (VOC)
Phase II/IIb and Phase V Rule	• Phase II/IIb established standards for an additional 38 VOCs after the VOC rule. Two of the standards limit the use of VOCs in common drinking water treatment chemicals while MCLs were set for the other 36 VOCs. • Phase V set standards for 23 more contaminants including inorganic chemicals, VOCs, pesticides, and synthetic organic chemicals. The rule also set monitoring schedules for these contaminants.
Arsenic Rule	• Established an MCL of 10 micrograms per liter of arsenic from samples at entry points into the water distribution system. Also established a MCLG of zero.
Radionuclides Final Rule	• Established MCLs and MCLGs for 6 radionuclides

2.1.2 Water Sources

The District currently has active permits for 8 groundwater wells and water rights at 7 other well locations in the service area. As of 2020, all the sources are classified as groundwater wells by the CDPHE. A summary of the active well permits is provided in Table 2-3. The initial well pumping test information and construction of the well – casing size and screen interval depth – is provided in Table 2-4. The most significant water right not in use is the Keybase Well which allows a pumping rate of 1,045 gallons per minute.

Table 2-3 – Summary of Groundwater Sources

WTP Fed	Well Name	Installation Year	Current Pumping Capacity (gpm)	Permitted Pumping Capacity (gpm)	Permitted Annual Volume (acre ft)	Active Permit Number
Base 2 WTP	SRWD Well No. 1	1984	200	250	400	027214-F
	Owner's Well No. 1[e]	-	0	26.5	43	033365-F
	Owner's Well No. 2[c]	1995	375	675[a]	1,290	045877-F[a]
	Owner's Well No. 3	1984	875	1,000	800	027215-F
	Owner's Well No. 4[c]	1995	355	800	100	046103-F
Base 3 WTP	Site 1 Wells 1 & 2	1973	480[f]	550	400	017883-F
	Supplemental Well No. 1A[c]	1984	530	750	1,129	035027-F
	Supplemental Well No. 1B[c]	1996	415	540[b]	871	045878-F[b]
	TOTALS		3,180	3,526.5[d]	5,033	-

[a] *Correction from 800 gpm to 675 gpm issued 2/27/1997*

[b] *Correction from 750 gpm to 320 gpm 12/23/1996, Correction from 320 gpm to 540 gpm of absolute rights on 11/04/2003, 871 acre-ft listed on permit, 1,209 acre-ft listed on conditions of approval.*

[c] *This group of wells is limited to a combined flow of 1,700 gpm per 95CW99*

[d] *Reflects the combined group capacity of 1,700 gpm*

[e] *Not connected to District's water system*

[f] *Flow capacity is with both pumps running*

During the most recent sanitary survey completed in 2018, the CDPHE indicated that an evaluation of the existing wells may be required to determine if the groundwater is under the influence of nearby surface waters. If the water source is determined to be groundwater under the influence of surface water (GWUDI) then water treatment processes meeting the requirements of various rules pertaining to surface water treatment are required. The Surface Water Treatment Rules require disinfection of *Giardia lamblia* bacteria in addition to viruses. The disinfection of *Giardia* is considerably more difficult to meet than disinfection of viruses. The Enhanced Surface Water

Treatment Rules pertain to filtration requirements and Cryptosporidium removal requirements. Further examination of the GWUDI impacts will be presented in the next subsection.

Table 2-4 – Well Permit Test Results and Construction Information

	Well Test Results				Well Construction	
Well	Static Level (ft)	Test Pump Rate (gpm)	Final Pumping Level (ft)	Specific Capacity (gpm/ft)	Depth Interval (ft)	Well Casing and Screen Size
SRWD Well No. 1*	2.1	400	33.25	12.8	0-37.5	10.75-inch Casing
	2	250	22.5	12.2	37.5-53	10-inch Screen
					53-58	9.63-inch Casing
Owner's Well No. 2	7.9	600	48.8	14.7	0-63	12.75-inch Casing
					63-83	12.75-inch Screen 80 slot
					83-88	12.75-inch Casing
Owner's Well No. 3	3.2	1,200	23.63	58.7	0-66.75	12-inch Casing
					66.75-87.5	12-inch Screen
					87.5-92.5	12-inch Casing
Owner's Well No. 4	11.25	425	40	14.8	0-53	12.75-inch Casing
					53-73	12.75-inch Screen 60 slot
					73-83	12.75-inch Casing
Site Well 1 Pump 1	9.9	350	69	5.9	0-71	16-inch Casing
					71-91	12.75-inch Screen
					91-101	12-inch Casing
Site Well 1 Pump 2	9.5	310	60	6.1	0-70	16-inch Casing
					70-90	12.75-inch Screen
					90-100	12-inch Casing
Supplemental Well 1A	No Test Data Available				0-71	10.75-inch Casing
					71-102	10-inch Screen
					102-107	10.75-inch Casing
Supplemental Well 1B*	6	350	21.2	23.0	0-82	12.75-inch Casing
	10.3	580	52.9	13.6	82-112	12.75-inch Screen 80 slot
					112-117	12.75-inch Casing

Two Tests Provided

2.1.3 Water Quality

This subsection will cover disinfection and finished water quality goals in the District's system.

Disinfection

Proper disinfection is measured by meeting minimum concentration-time (CT) values set by the safe drinking water rules. CT values relate the disinfectant concentration with the contact time in water to remove or inactivate organisms before the water is delivered to the first customer. In many cases, the first customer is the water treatment plant itself, so this value is typically met when the water reaches the distribution pumps. The CT requirement value is expressed in min-mg/L.

General guidance for calculating the contact time is provided in a guidance manual[1] developed by the United States Environmental Protection Agency. The guidance manual provides baffling factors which are used to calculate the total contact time. These factors account for dead space, and the nature of mixing and short-circuiting in flow through tanks. For instances a baffled clearwell in which the water travels in a serpentine route through corridors with high length-to-width ratios has a higher baffling factor than an open clearwell. The CDPHE sites a clearwell design research study[2] as a guideline which provides more in-depth evaluation of baffling factors.

The CT requirements vary based on three different water quality parameters; water temperature, pH, and disinfectant concentration. The pH of the water greatly affects the requirements as the concentration of the two free chlorine forms in water vary with the pH value; hypochlorous acid (HOCl-) and hypochlorite (OCl+). The hypochlorous acid is the more powerful disinfectant and the available percentage decreases as pH increases; therefore, disinfection is more efficient at lower pH values. Disinfection requirements are also higher when the water temperature is lower. The CT value is the product of the chlorine residual and the contact time of the residual in the water.

Table 2-5 – CT Values for Virus Inactivation by Free Chlorine

TABLE E-7

CT VALUES FOR
INACTIVATION OF VIRUSES BY FREE CHLORINE(1)

	Log Inactivation					
	2.0 pH		3.0 pH		4.0 pH	
Temperature (C)	6-9	10	6-9	10	6-9	10
0.5	6	45	9	66	12	90
5	4	30	6	44	8	60
10	3	22	4	33	6	45
15	2	15	3	22	4	30
20	1	11	2	16	3	22
25	1	7	1	11	2	15

Table E-7 taken from EPA Guidance Manual

[1] Guidance Manual for the Compliance with the Filtration Requirements for Public Water Systems using Surface Water Sources, United States Environmental Protection Agency Office of Drinking Water (March 1991 Edition).

[2] Crozes, Gil F., et al. *Improving Clearwell Design for CT Compliance.* AWWA Research Foundation (1999).

Table 2-5 provides the CT requirements for inactivation of viruses by free chlorine disinfection. The CT requirement for 4-log inactivation of viruses is 8 min-mg/L at a water temperature of 5°C and a pH value between 6 and 9 which is the closest point to the District's water quality. Using similar water quality parameters of 5°C, pH of 7.5 and a chlorine residual of 1.2 mg/L the CT requirement for 3-log *Giardia* inactivation by free chlorine is 183 min-mg/L as shown in Table 2-6. In this instance, the CT requirement for *Giardia* is over 22 times higher than the requirement for viruses. Recall that groundwater requires 4 log-inactivation of viruses while surface water requires both 4-log virus and 3-log *Giardia* inactivation.

Table 2-6 – CT Values for Inactivation of *Giardia* by Free Chlorine

TABLE E-2
CT VALUES FOR INACTIVATION
OF GIARDIA CYSTS BY FREE CHLORINE
AT 5 C (1)

CHLORINE CONCENTRATION (mg/L)	pH<=6 Log Inactivations						pH=6.5 Log Inactivations						pH=7.0 Log Inactivations						pH=7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
<=0.4	16	32	49	65	81	97	20	39	59	78	98	117	23	46	70	93	116	139	28	55	83	111	138	166
0.6	17	33	50	67	83	100	20	40	60	80	100	120	24	48	72	95	119	143	29	57	86	114	143	171
0.8	17	34	52	69	86	103	20	41	61	81	102	122	24	49	73	97	122	146	29	58	88	117	146	175
1	18	35	53	70	88	105	21	42	63	83	104	125	25	50	75	99	124	149	30	60	90	119	149	179
1.2	18	36	54	71	89	107	21	42	64	85	106	127	25	51	76	101	127	152	31	61	92	122	153	183
1.4	18	36	55	73	91	109	22	43	65	87	108	130	26	52	78	103	129	155	31	62	94	125	156	187
1.6	19	37	56	74	93	111	22	44	66	88	110	132	26	53	79	105	132	158	32	64	96	128	160	192
1.8	19	38	57	76	95	114	23	45	68	90	113	135	27	54	81	108	135	162	33	65	98	131	163	196
2	19	39	58	77	97	116	23	46	69	92	115	138	28	55	83	110	138	165	33	67	100	133	167	200
2.2	20	39	59	79	98	118	23	47	70	93	117	140	28	56	85	113	141	169	34	68	102	136	170	204
2.4	20	40	60	80	100	120	24	48	72	95	119	143	29	57	86	115	143	172	35	70	105	139	174	209
2.6	20	41	61	81	102	122	24	49	73	97	122	146	29	58	88	117	146	175	36	71	107	142	178	213
2.8	21	41	62	83	103	124	25	49	74	99	123	148	30	59	89	119	148	178	36	72	109	145	181	217
3	21	42	63	84	105	126	25	50	76	101	126	151	30	61	91	121	152	182	37	74	111	147	184	221

CHLORINE CONCENTRATION (mg/L)	pH=8.0 Log Inactivations						pH=8.5 Log Inactivations						pH<=9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
<=0.4	33	66	99	132	165	198	39	79	118	157	197	236	47	93	140	186	233	279
0.6	34	68	102	136	170	204	41	81	122	163	203	244	49	97	146	194	243	291
0.8	35	70	105	140	175	210	42	84	126	168	210	252	50	100	151	201	251	301
1	36	72	108	144	180	216	43	87	130	173	217	260	52	104	156	208	260	312
1.2	37	74	111	147	184	221	45	89	134	178	223	267	53	107	160	213	267	320
1.4	38	76	114	151	189	227	46	91	137	183	228	274	55	110	165	219	274	329
1.6	39	77	116	155	193	232	47	94	141	187	234	281	56	112	169	225	281	337
1.8	40	79	119	159	198	238	48	96	144	191	239	287	58	115	173	230	288	345
2	41	81	122	162	203	243	49	98	147	196	245	294	59	118	177	235	294	353
2.2	41	83	124	165	207	248	50	100	150	200	250	300	60	120	181	241	301	361
2.4	42	84	127	169	211	253	51	102	153	204	255	306	61	123	184	245	307	368
2.6	43	86	129	172	215	258	52	104	156	208	260	312	63	125	188	250	313	375
2.8	44	88	132	175	219	263	53	106	159	212	265	318	64	127	191	255	318	382
3	45	89	134	179	223	268	54	108	162	216	270	324	65	130	195	259	324	389

Notes:
(1) $CT_{99.9}$ = CT for 3-log inactivation

Table E-2 taken from EPA Guidance Manual

As previously discussed, water sources classified as GWUDI require treatment meeting the standards for surface water sources. The two major requirements for surface water treatment beyond ground water treatment are the water must be filtered and 3-log disinfection is required for Giardia in addition to viruses. Credit for Giardia, virus and Cryptosporidium removal is given for using filtration treatment which reduces the removal requirement via disinfection. Filtration requirements include a treatment technique established by meeting a turbidity concentration in the filtered water. Cryptosporidium removal has the most stringent filtration standard, requiring the combined filter effluent turbidity to be less than 0.3 NTU in 95% of the samples.

Source Water Quality and Treatment Objectives

The water quality of the treated water at each WTP was examined in the Optimal Corrosion Control Treatment Study completed in March of 2018. Table 2-7 provides a summary of the finished water quality parameters and the target water quality goals. Value provided are the average of two water sample test results in the optimal corrosion control study[3].

Table 2-7 - Finished Water Quality and Goals

Source Water Quality	Base 2 Wells	Base 3 Wells
pH	7.4 - 7.6	6.8 - 7.2
Temperature (°C)	7.5	7.7
Alkalinity (mg/L as CaCO3)	22	52.5
Calcium Hardness (mg/L as CaCO3)	60	52
Specific Conductance (µS/cm)	210	195
Chloride (mg/L)	11.9	23
Sulfate (mg/L)	53	10.4
Iron (mg/L)	<0.1	0.219
Manganese (mg/L)	<0.05	0.342
Finished Water Quality Goals	**Base 2 WTP**	**Base 3 WTP**
pH	8.5	8.5
Iron (mg/L)	-	0.1
Manganese (mg/L)	-	0.01
Free Chlorine at Distribution Entrance (mg/L)	1.0	1.0
Virus Inactivation	4-Log	4-Log

The pH of water is a vital value to monitor during treatment and in the distribution system as pH is a measure of acidity. Changes in pH can greatly affect the carbonate equilibrium, chlorine concentrations, and the overall corrosivity of water. As pH drops the water becomes more acidic and, therefore, more corrosive in principle. There are other factors to consider when determining corrosivity of water.

Calcium is one of two major contributors to hardness in water, the other is magnesium. The calcium hardness values near 56 mg/L as $CaCO_3$ (calcium carbonate) are indicative of a soft to medium hardness water, however, conclusions are tough to draw since magnesium hardness values were not provided.

Typically, the goal is to provide a treated water which is slightly scale forming and which forms calcium precipitates in the distribution piping. The presence of calcium allows those precipitates to form. Alternatively, the water could be slightly corrosive which causes metals in the distribution

[3] Johnson, Benjamin, (2018). *Optimal Corrosion Control Treatment Study for the Snake River Water District in Summit County, CO*, Tetra Tech

piping to leach into the water which may cause allowable levels for consumption to be exceeded. Alkalinity provides capacity to resistance to pH changes in the water, that is, more resistance is provided with higher alkalinity values.

Specific conductance closely correlates to dissolved solids concentrations in ground water as dissolved ions enable water to conduct electrical current. Dissolved solids are a secondary drinking water standard with a maximum contaminant level of 500 mg/L and can cause undesirable taste in the water.

Chloride and sulfate are two parameters to monitor as they also have a role in corrosion. The mass ratio of chloride to sulfate can be used as an indicator of corrosion potential. Lower chloride to sulfate mass ratios typically results in less leaching of lead in water.

Distribution

The District provided the annual DBP testing results from 2014 through 2019. From 2014 to 2017 the District was required to test at two sites; 23 Arabella Drive in Base 1 pressure zone near Pilot Lode tank and 155 River Course Drive at the very west end of Base 2 pressure zone. In 2019, testing at only the 23 Arabella Dr. site was required. In all instances, the TTHM results were well below the maximum contaminant level (MCL) with an average result of 2.6 µg/L compared to the MCL of 80 µg/L. In all instances, the HAA concentrations were undetectable by the testing methods except for 1 result in 2016 at 1.4 µg/L concentration. The MCL for HAA is 60 µg/L so the one detectable result is well below the water quality limit.

Nitrate, nitrite, and inorganic chemical testing results of sampled taken at distribution entry points were also provided from 2012 to 2019 for the District. The maximum nitrate-N residual concentration was 0.44 mg/L which is well below the MCL of 10 mg/L. In all instances, nitrite-N was undetectable. Inorganic chemicals were tested in 2014 and 2017 with only barium concentration results above detectable limits, the maximum barium result of 0.027 mg/L is well below the MCL of 2 mg/L.

Test results for organic chemicals were provided from 2012 to 2020. The results were all below the individual chemical MCL except for a detection of Di(2-ethylhexyl)phthalate (DEHP) in 2018. The detection result of 2.6 µg/L - which is below the MCL of 6 µg/L - triggered more routine monitoring intervals of the chemical which has not been detected since; the District currently samples and tests quarterly.

Overall, water quality in the Distribution system is very good with very little DBP formation, very low nitrate residuals, minimal detection of inorganic chemicals, and one instance of an organic chemical exceeding its MCL which hasn't been detected since.

2.1.4 Water Treatment Plants

The District currently owns and operates two water treatment plants (WTP). These plants are named the Base 2 WTP and Base 3 WTP which follows the respective pressure zone they serve.

Base 2 Water Treatment Plant

Base 2 WTP, which has a treatment schematic shown in Figure 2-2, treats groundwater from four wells and is classified as a class C water treatment facility under Colorado Regulation 100. The water plant was constructed in 1996 with administrative additions occurring afterward. The raw water is treated with soda ash for pH adjustment, and chlorine for disinfection prior to discharge into the clearwell. The water flows through a baffled clearwell and is then transferred to the distribution system via four vertical turbine pumps. The District currently uses gaseous chlorine at the Base 2 WTP. The District recently implemented pH adjustment via soda ash as a result of lead residual tests exceeding the action level which prompted the CPDHE to require a corrosion control treatment study and implementation.

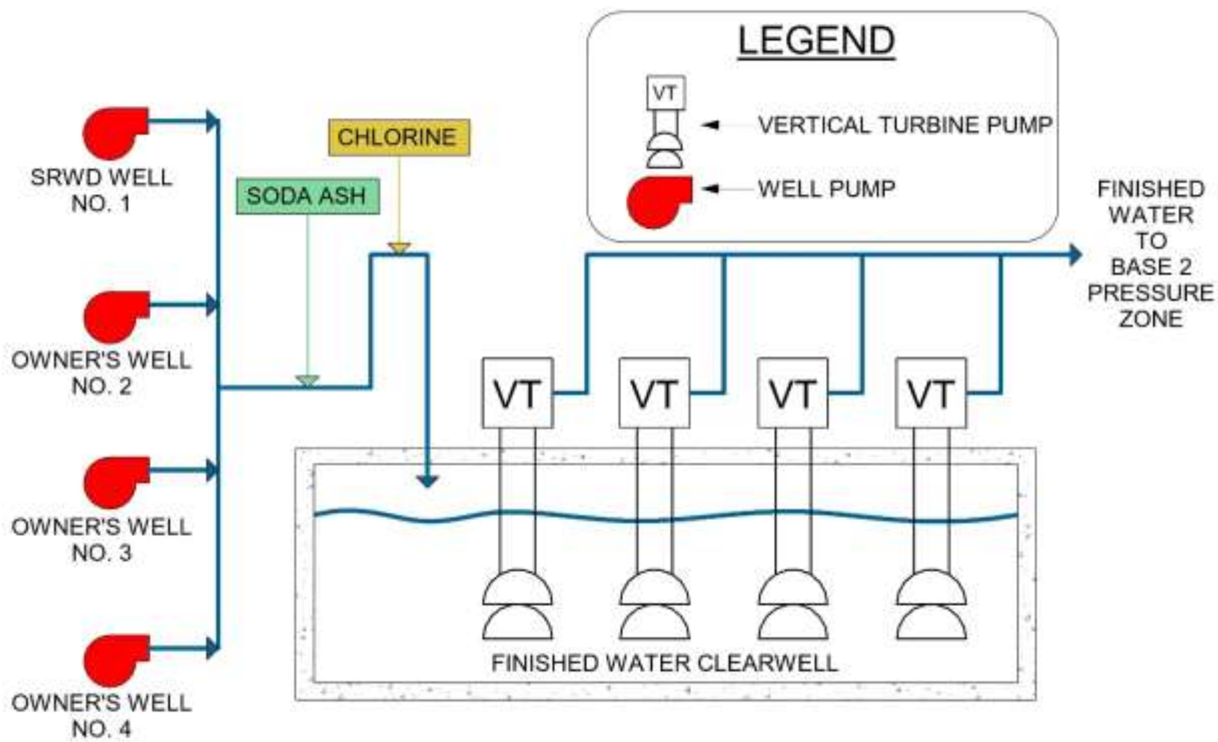


Figure 2-2 – Base 2 WTP Treatment Schematic

The clearwell for Base 2 WTP is 29 feet x 16 feet with a water depth of approximately 12.7 feet. This portion of the clearwell has baffled walls with a length-to-width of ratio near 33 which provides good baffling; the approximate path length is 116 feet and width is 3.5 feet. The water flows over a 2-foot wide weir into the end section of the clearwell where the four vertical turbine pumps are located. This end section is 29 feet by 6.67 feet with a water depth that can vary. Therefore, the baffled clearwell stores a volume of approximately 44,000 gallons and the end section stores approximately 11,500 gallons at an 8-foot water depth. At plant flow rate of 1,400 gpm, the baffled portion of the clearwell provides approximately 21.5 minutes of contact time using a baffling factor of 0.65 for baffled tank and 0.1 for the end section. Assuming a chlorine residual of 0.8 mg/L, the CT provided by free chlorine disinfection is approximately 16.4 min-mg/L which is above the assumed requirement of 8 min-mg/L at the plant's maximum flow rate.

Base 3 Water Treatment Plant

Base 3 WTP, which has a treatment schematic shown in Figure 2-3, treats groundwater from three wells and finished constructed in mid-2020. The plant is classified as a class B water treatment facility under Regulation 100.

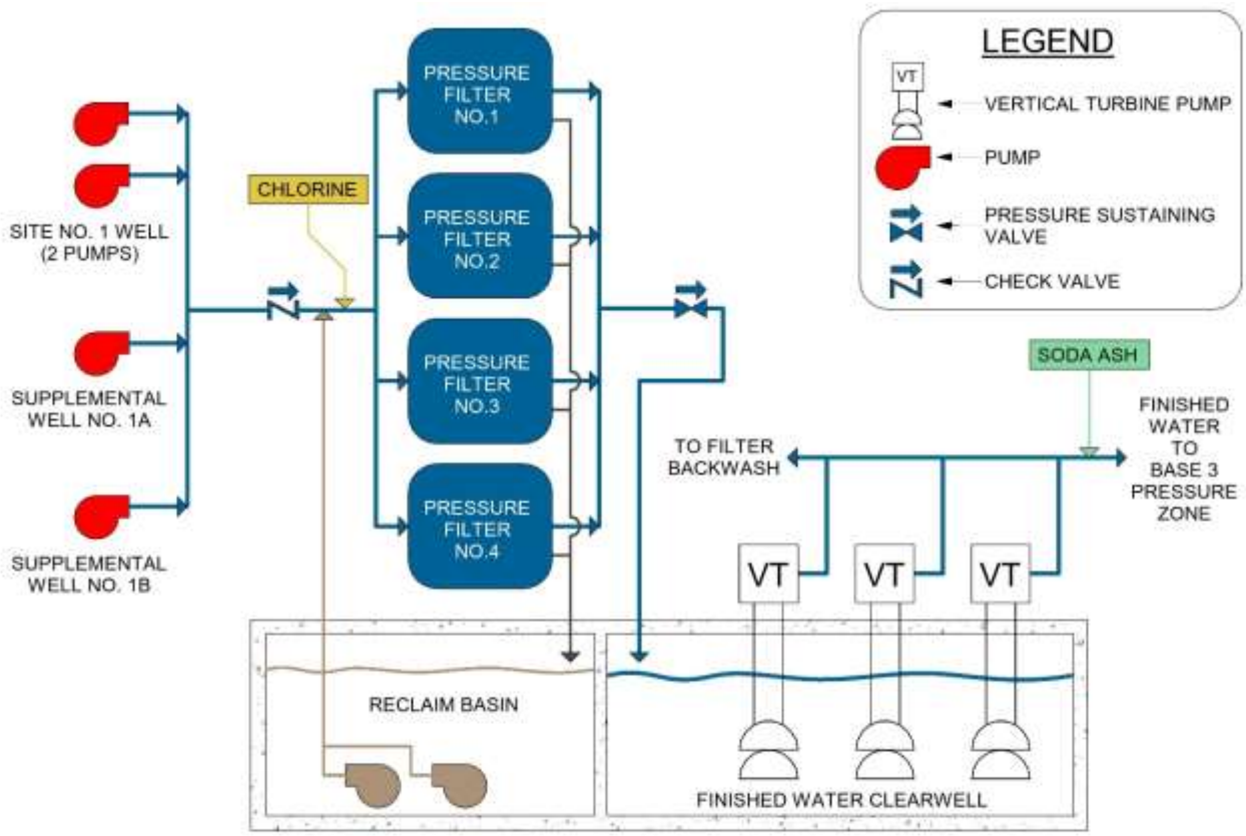


Figure 2-3 – Base 3 WTP Treatment Schematic

The Site No. 1 well has two pumps installed at the well location. The groundwater passes through a check valve before entering one of four pressure filters; the check valve prevents flow of reclaim water back to the wells. Each pressure filter is 12.5-feet in diameter (122.7 square feet of filter area per filter) with gravel, torpedo sand, silica sand and anthracite media with a design flow rate of 2.8 gpm/ft^2. The pressure filters provide iron and manganese removal along with filtration of the water. Soluble iron is oxidized to solid form using chlorine fed ahead of the filters and removed via filtration. Manganese is oxidized and removed in a similar manner except that manganese is absorbed onto the media itself. An additive is used on the media to get similar performance to greensand for manganese removal. A pressure sustaining valve on the discharge of the pressure filters maintains a constant pressure in the filters before the filtered water is discharged to the clearwell. The water flows through a baffled clearwell similar to the Base 2 WTP. Three vertical turbine pumps transfer water from the clearwell into the distribution system. Soda ash is added into the discharge pipe to adjust pH of the water for corrosion control purposes.

The WTP has a reclaim basin to reuse backwash water from pressure filter cleaning sequences. These cleaning sequences are referred to as backwashes. The pressure filters are cleaned at regular intervals or when filter performance begins to decline. The reclaimed water re-enters the system at the beginning of the treatment process per the Filter Backwash Recycle Rule. The rate of the reclaim water flow is also regulated to meet the rules which allows backwash flow to comprise up to 10-percent of influent plant flow.

The Base 3 WTP is designed to meet the treatment requirements for groundwater under the direct influence (GWUDI) of surface water. The wells are not currently classified as GWUDI; however, the design is a precaution if the system's wells were ever to be classified as a GWUDI. The treatment setup is considered a direct filtration plant as sedimentation or clarification is not provided ahead of the filters. Therefore, the WTP is likely given 2-log credit for *Cryptosporidium* removal, 2-log removal for *Giardia* removal and 1-log credit for virus removal. The 1-log CT removal requirement for *Giardia* is 61 min-mg/L when using a pH value of 7.5, water temperature of 5°C, and chlorine residual of 1.2 mg/L.

The Base 3 WTP clearwell is well baffled with long length-to-width ratios similar the Base 2 WTP. The clearwell is approximately 68 feet long by 29 feet wide with an overflow weir which maintains a water level of approximately 8.5 feet. The approximate path length in the clearwell is 288 feet with channel widths of 4.5 feet resulting in a length-to-width ratio value of 64. The water falls into the end section of the clearwell where the vertical turbine pumps are located; this section is 16.67 feet long by 8 feet wide. At a flow rate of 1,400 gpm, the baffled portion of the clearwell provides 52 minutes of contact time assuming a 0.7 baffling factor. The CT value provided in the clearwell is approximately 62.4 min-mg/L which is higher than the assumed requirement of 61 min-mg/L. Additionally, chlorine is fed ahead of the pressure filters so additional contact time is provided in the filter vessels themselves. Therefore, the Base 3 WTP appears to be appropriately configured to achieve the CT requirements for *Giardia* at the plant's maximum flow rate.

2.1.5 Summary of Sources and Treatment

A summary of the source water flow rates, considerations of source permits, treatment capacities and finished water pump capacities are shown in Figure 2-4. Key takeaway from the water source and treatment review are bulleted below:

- The source water capacity is 3,500 gpm as permitted through various well permits.
- The annual water volume permitted for withdrawal is 5,033 acre-feet as permitted through various well permits.
- The Base 2 has a firm treatment capacity of 880 gpm (or 1.27 MGD) based on capacity of the well sources with Owner's Well No. 3 out of service.
- The Base 3 WTP has a firm treatment capacity of 895 gpm (or 1.29 MGD) based on Supplemental Well No. 1A out of service.
- The CDPHE indicated the District's wells may be required to go through an evaluation to determine if the well source water is groundwater under the direct influent of surface water (GWUDI). If the water is determined to be GWUDI then additional treatment will be required at the Base 2 WTP.

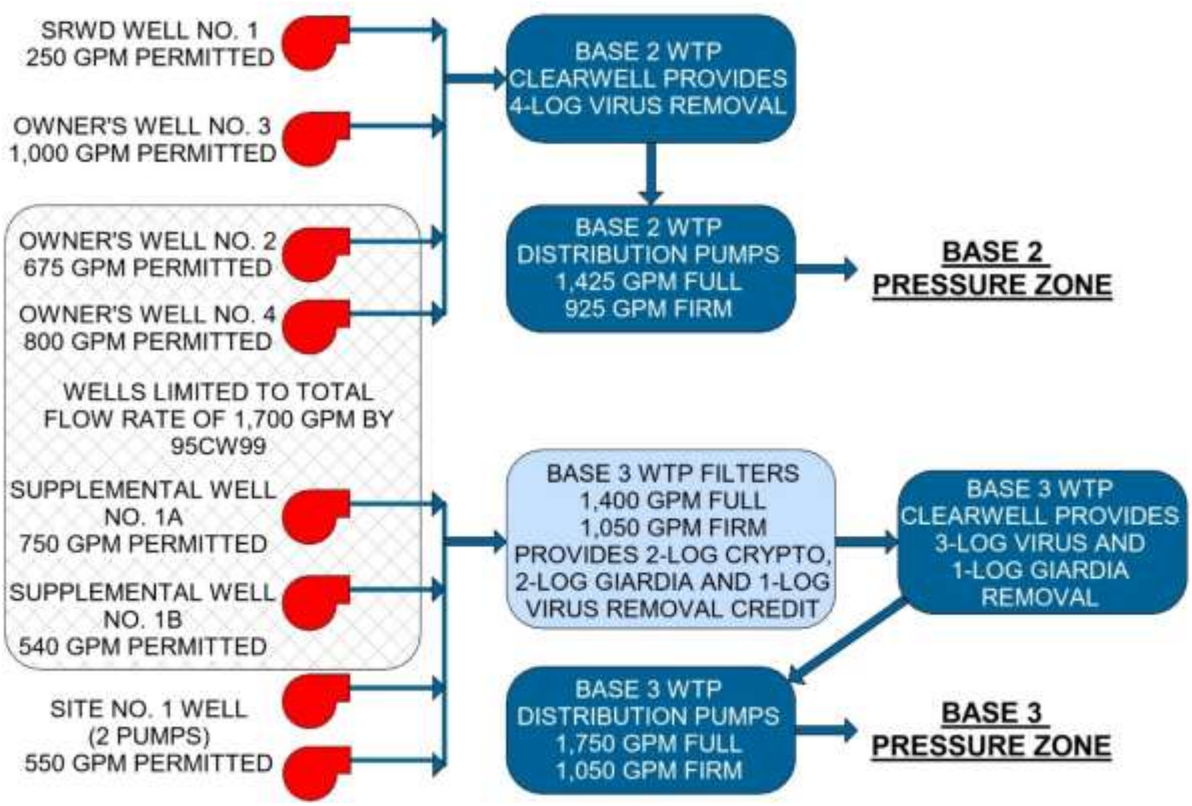


Figure 2-4 – Summary of Water Sources and Treatment

2.2 System Network and Facilities

This section will discuss the water system's infrastructure beyond the water treatment plants. The network of assets that make up the distribution system include pipe, storage tanks, pump stations, and valves. Figure 2-5 is a hydraulic profile of the system showing the elevation of tanks, pumps stations, water treatment plants, and pressure reducing valves and shows how water can move throughout the system.

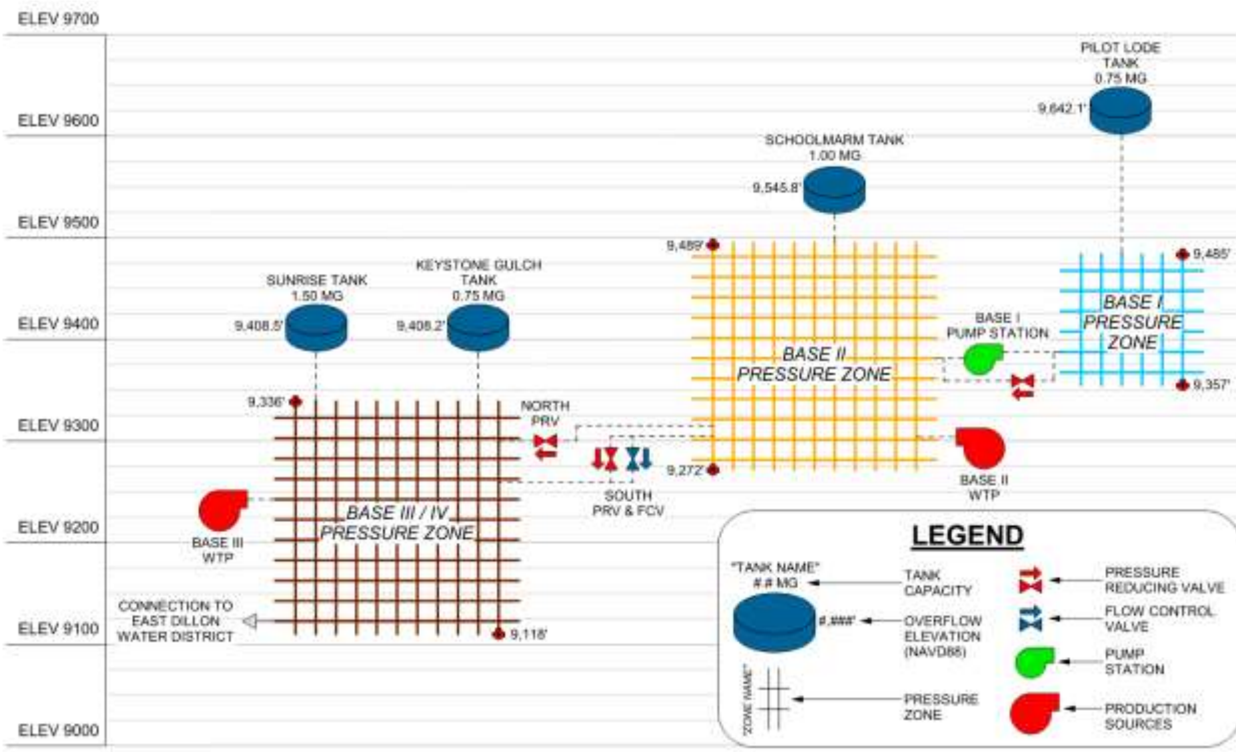


Figure 2-5 – Existing System Elevation Profile

2.2.1 Pressure Zones

There are three pressure zones in the District's system. The Base 3 and Base 4 pressure zones are combined into one pressure zone fed from two water storage tanks. Figure 2-6 shows the spatial layout of the pressure zones color coded to match the system elevation profile Figure 2-5.

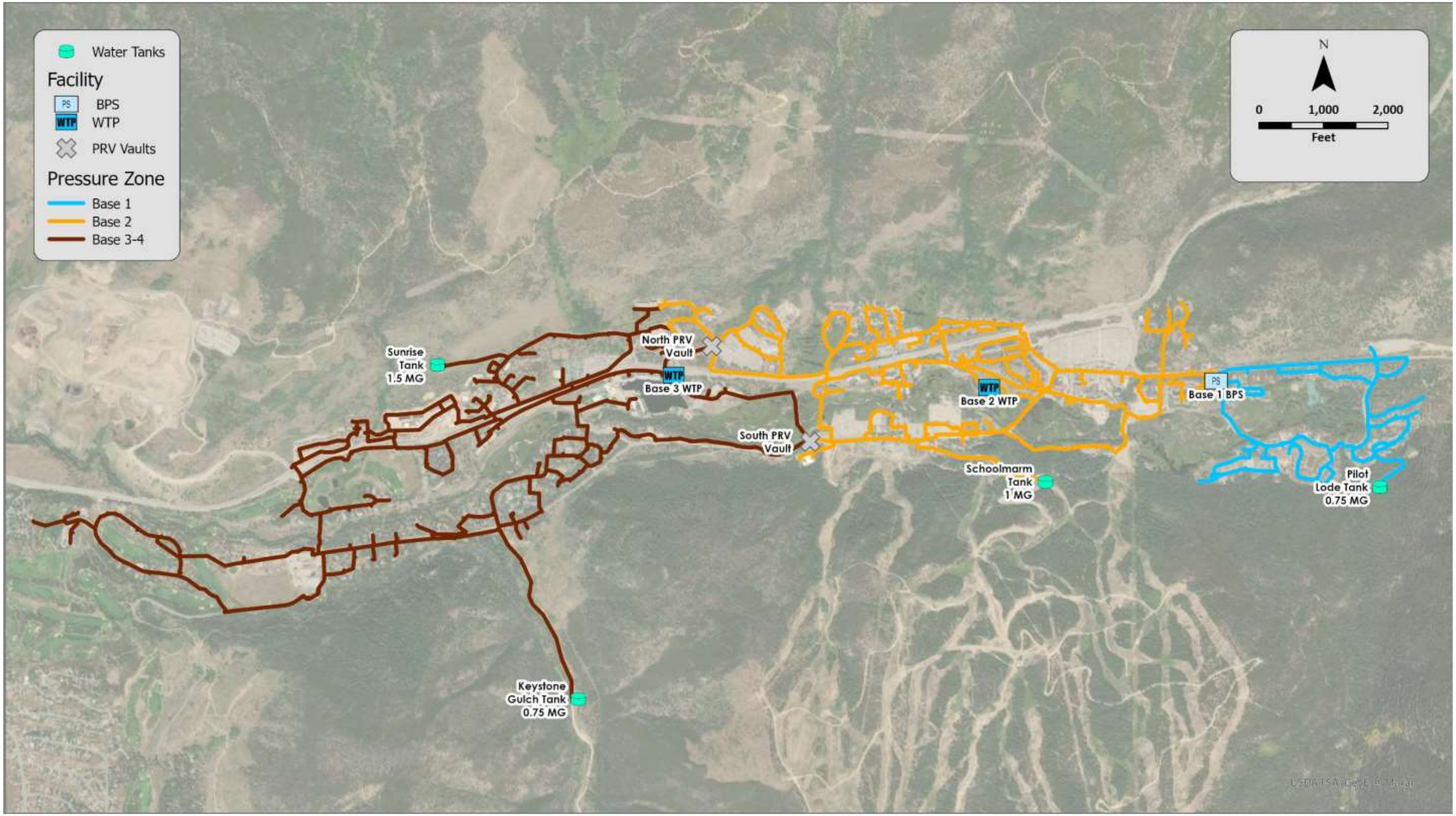


Figure 2-6 – Pressure Zone Map

2.2.2 Water Storage Tanks

There are four water storage tanks in the District. Two tanks provide storage for the Base 3 pressure zone while Base 2 and Base 1 pressure zones each have one storage tank supplying the zone. All the tanks are circular tanks with varying operating head ranges. Overall, 4.0 million gallons of tank storage capacity is available in the system. The base and overflow elevations in Table 2-8 were converted from NGVD29 elevations to NAVD88 elevations if the record drawing dates preceded 1992.

Table 2-8 – Existing Water Storage Tank Information

Water Storage Facility Name	Volume (MG)	Dimensions	Max Water Surface Depth (feet)	Base Elevation (feet) NAVD88	Overflow Elevation (feet) NAVD88	Construction Year & Type
Pilot Lode Tank	0.75	64′ diameter	31′	9,611.13′	9,642.13′	1997 – Steel – Above Grade
Schoolmarm Tank	1.0	100′ diameter	17.25′	9,528.53′	9,545.78′	1983 – Concrete – Partially Buried
Sunrise Tank	1.5	105.5′ diameter	23′	9,385.48′	9,408.48′	1978 – Concrete – Buried
Keystone Gulch Tank	0.75	76′ diameter	23′	9,385.15′	9,408.15′	2001 – Concrete – Buried

In 2007 the roof of the Sunrise Tank was demolished and replaced. The tank was then fully buried with 18 inches of earth cover over the new roof structure.

2.2.3 Pump Stations

The Base 1 BPS moves water from the Base 2 pressure zone to the Base 1 pressure zone. The pump station contains three horizontal end suction pumps and a pressure reducing valve that allows water to flow from Base 1 to Base 2 in the event of a high demand event in Base 2. This pressure reducing valve is set to open at 40 psi downstream pressure according to District staff. The capacities of the pumps in the Base 1 BPS are provided in Table 2-9. The capacities of the well pumps and finished water pumps at both the water treatment plants are also provided in Table 2-9.

Table 2-9 – Existing Pump Information

Pump Station	Station Full / Firm Capacity (gpm)	Pumps To	Pump 1	Pump 2	Pump 3	Pump 4
Base 1 BPS	525 / 225	Schoolmarm Tank to Pilot Lode Tank	7.5 HP 75 gpm 168' TDH	15 HP 150 gpm 170' TDH	25 HP 300 gpm 175' TDH	-
Base 2 WTP Finished Water Pumps	1,425 / 925	Base 2 WTP Clearwell to Schoolmarm Tank	50 HP 500 gpm 255' TDH	30 HP 350 gpm 262' TDH	40 HP 375 gpm 255' TDH	20 HP 200 gpm 262' TDH
Base 2 WTP Well Pumps	2,450 / 1,450	Groundwater to Base 2 WTP	*SRWD Well No. 1* 5 HP 150 gpm 140' TDH	*Owner's Well No. 2* 15 HP 800 gpm 59' TDH	*Owner's Well No. 3* 25 HP 1000 gpm 68' TDH	*Owner's Well No. 4* 10 HP 500 gpm 48' TDH
Base 3 WTP Finished Water Pumps	1,750 / 1,050	Base 3 WTP Clearwell to Keystone Gulch and Sunrise Tanks	25 HP 350 gpm 180' TDH	50 HP 700 gpm 180' TDH	25 HP 700 gpm 180' TDH	-
Base 3 WTP Well Pumps	1,589 / 989	Groundwater to Base 3 WTP	*Site 1 Well – Pump 1* 10 HP 232 gpm 80' TDH	*Site 1 Well – Pump 2* 10 HP 232 gpm 80' TDH	*Supplemental Well No. 1A* 25 HP 600 gpm 100' TDH	*Supplemental Well No. 1B* 20 HP 525 gpm 86' TDH

Note that the full and firm capacities are based on nameplate ratings and may differ from capacities listed elsewhere such as the water source summary table.

2.2.4 Pressure Reducing Valves

Two pressure reducing vaults allow water to flow from Base 2 to Base 3 pressure zone. These vaults are named the North and South PRV Vaults. The South PRV also has a flow control valve that allows operators to remotely control the flow rate from Base 2 to Base 3 via SCADA. The North PRV vault has a 6-inch valve while the South PRV has an 8-inch valve. Both of these valves are set to open at a downstream pressure of 35 psi according to District operations staff. The PRV in Base 1 BPS is discussed in subsection 2.2.3.

2.2.5 Water Transmission Main

The SRWD system consists of ductile iron pipe and cast-iron pipe material totaling 156,191 linear feet (29.6 miles) of pipe, ranging from 4-inch to 16-inch in diameter. Table 2-10 provides the total lengths by size and material in the system. There is very little cast iron pipe remaining the system and the material makes up less than 1 percent of total system pipe length.

Table 2-10 – Water Main Information by Size and Material

Pipe Diameter	Length of Pipe by Material (feet)		Total Length (feet)	Total Length (miles)	% by Size
	Ductile Iron	Cast Iron			
4-Inch	1,229	1,446	2,675	0.51	1.7%
6-Inch	24,102	-	24,102	4.56	15.4%
8-Inch	77,240	-	77,240	14.63	49.5%
10-Inch	18,853	-	18,853	3.57	12.1%
12-Inch	32,301	-	32,301	6.12	20.7%
16-Inch	1,020	-	1,020	0.19	0.7%
Total Length (feet)	154,745	1,446	156,191		
Total Length (miles)	29.31	0.27		29.58	
% by Material	99.1%	0.9%			

Table 2-11 – Water Main Length by Installation Year

	Installation Year									
Pipe Diameter	1970 1974	1975 1979	1980 1984	1985 1989	1990 1994	1995 1999	2000 2004	2005 2009	2010 2014	2015 2019
4-Inch	1,696	-	123	33	23	465	65	26	244	-
6-Inch	4,787	1,957	3,642	2,250	1,475	6,919	496	312	200	2,064
8-Inch	15,850	1,631	14,547	1,961	4,335	21,354	8,325	1,704	2,504	5,029
10-Inch	6,777	103	2,520	4,812	-	3,227	4	-	-	1,410
12-Inch	5,802	2,127	12,469	-	3,113	6,779	2,011	-	-	-
16-Inch	-	-	-	1,020	-	-	-	-	-	-
Period Length (feet)	34,912	5,818	33,301	10,076	8,946	38,774	10,901	2,042	2,948	8,503
Cumulative Length (miles)	6.61	7.71	14.02	15.93	17.62	24.96	27.03	27.41	27.97	29.58
% of Total Length	22.4%	3.7%	21.3%	6.5%	5.7%	24.8%	7.0%	1.3%	1.9%	5.4%

The growth of the water main system is broken into 5-year increments in Table 2-11 The District experienced large growth in the 1980 - '84 and 1995 - '99 periods which correlate with tank, pump station and water treatment plant ages. The 21 percent growth in '80 - '84 can be attributed to the 1982 Schoolmarm tank installation while the 25 percent growth in '95 - '99 can be attributed to the 1997 Pilot Lode tank installation and growth in Base 1 and the River Run area of Base 2 pressure zone. Maps of the pipe sizes and pipe ages are presented in Figure 2-7 and Figure 2-8, respectively.

Table 2-12 provides the length of pipe in each pressure zone along with the approximate volume of water within the distribution piping. The volume of water within the piping can be useful in determining water tank operation setpoints in attempting to pump water with lower age into the tank. Tank operations will be reviewed in chapter 7.0.

Table 2-12 – Water Main Length by Pressure Zone

Pipe Diameter	Length of Pipe by Pressure Zone (feet)		
	Base 1	Base 2	Base 3 4
4-Inch	375	1,955	345
6-Inch	1,910	11,609	10,584
8-Inch	11,549	22,676	43,014
10-Inch	1,415	4,990	12,448
12-Inch	7,872	16,703	7,726
16-Inch	-	1,020	-
Total Length (feet)	23,121	58,953	74,117
% by Pressure Zone	14.8%	37.7%	47.5%
Volume in Pipe (gallons)	85,200	206,700	224,250

2.2.6 Hydrants

The District currently has 295 fire hydrants throughout its system according to the GIS data. There are three brands of hydrants throughout the system with hydrant installation dates ranging from 1972 to 2018. Table 2-13 provide hydrant ages grouped by decade along with the total hydrants by brand. Over 75% of the hydrants were installed before the year 2000 while a majority of the hydrants installed since 1990 are Mueller brand. The higher percentage of hydrants installed in the 1980s and 90s corresponds to the higher length of pipe installed during the same time periods.

Table 2-13 – Hydrant Age and Brand Information

	Hydrant Brand			Total by Year	% By Year
	Mueller	Pacific States	Waterous		
1972-1979		2	47	49	16.6%
1980-1989	18		67	85	28.8%
1990-1999	88		8	96	32.5%
2000-2009	38		3	41	13.9%
2010-2020	18		6	24	8.1%
Total by Brand	162	2	131	295	
% By Brand	54.9%	0.7%	44.4%		

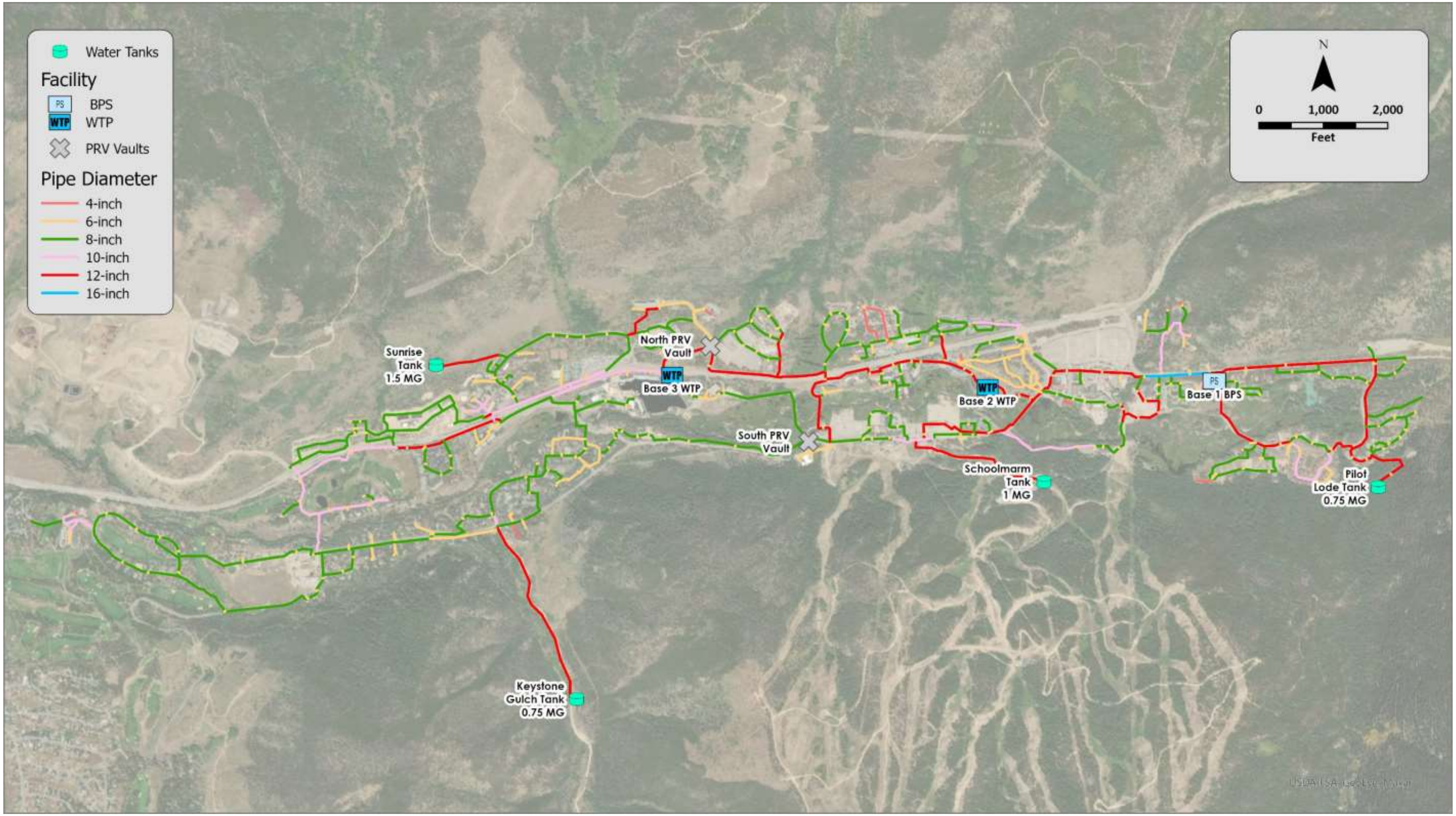


Figure 2-7 – Existing Pipe Size Map

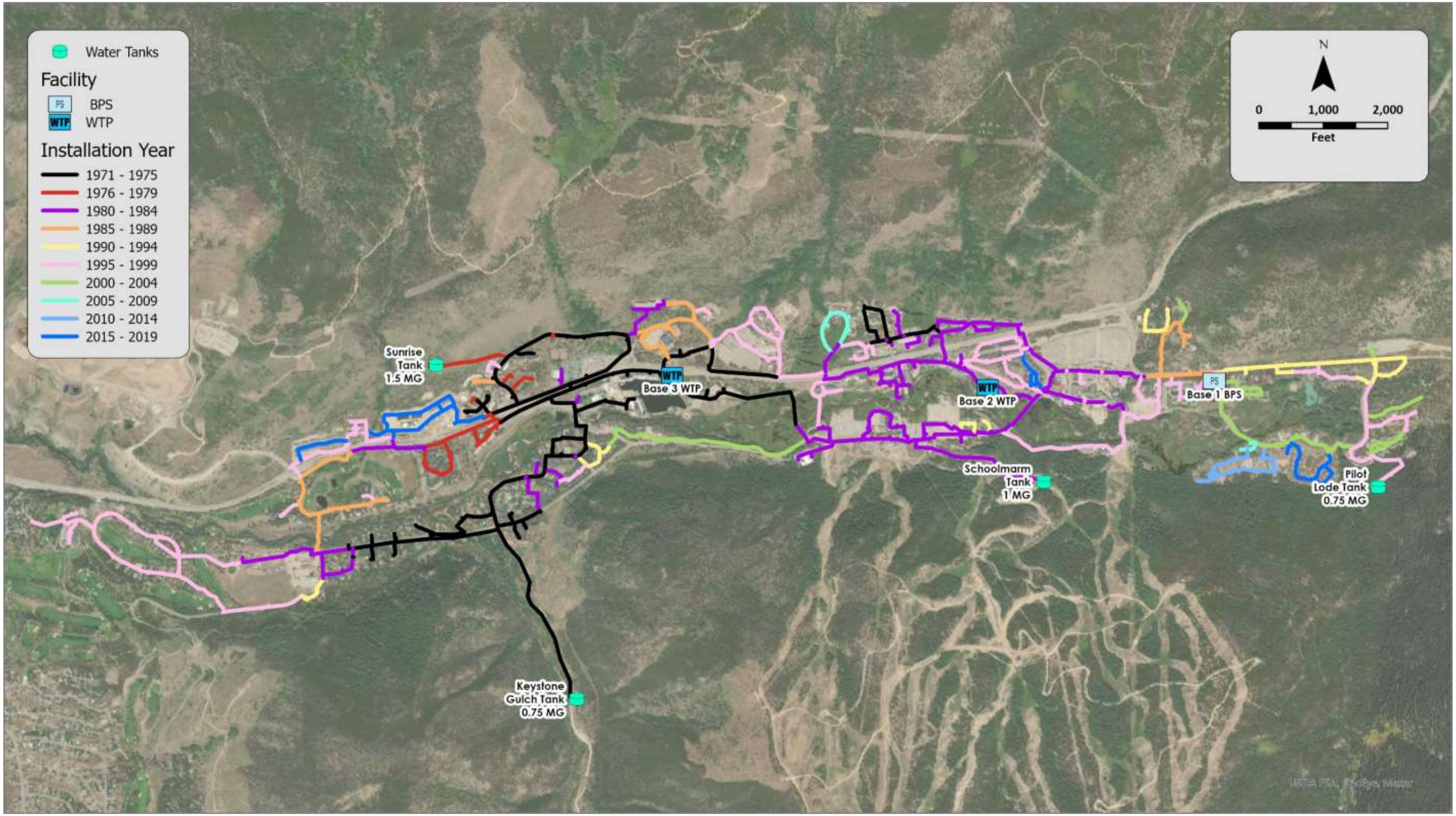


Figure 2-8 – Existing Pipe Age Map

3.0 Land Development

This chapter will cover previous land planning efforts and EQR projections, cover the definition of EQR, and provide future estimated EQRs spatially allocated throughout the District's service area.

3.1 Previous Planning Efforts and Growth

The District's service area lies within unincorporated areas of Summit County so county regulations are used for land development. Much of the land within the District's service area is governed by the Keystone Resort Planned Unit Development (PUD). A PUD can be described as a "do-it-yourself" version of zoning, where the PUD document serves a legal document guiding density, lot size, building heights, roads, and land use, amongst other guidelines. There are other PUDs in the District's service area, much smaller in size, with their own legal documents.

The 2012 Master Plan listed an existing EQR total of 4,066.85 and projected a buildout of 6,694 EQRs which represented the potential for 65% growth in the service area. Similarly, the Snake River Planning Commission (SRPC), which is a branch of the Summit County planning commission, completed a master plan in 2010 which projected 38% growth in residential buildout in unincorporated areas of planning boundary. Additionally, the commercial space in the Keystone Resort PUD was 50% built out as of 2010. The SRPC boundaries are larger than the District's service area which encompasses the incorporated Towns of Montezuma and Dillon along with unincorporated areas of Summit Cove.

3.1.1 Single-Family Equivalent Rating (EQR) Evaluation

Define EQRs

The District uses a single-family equivalent value (EQR) for water rights, tap fee and water billing purposes. Each account is assigned an EQR value which represents the water use equivalent to a single-family dwelling unit of 3 bedrooms or less and is then used to estimate the impact upon the District's water supply. For instance, a large home may have an EQR value of "2" indicating that water consumption is estimated to be twice the amount of a single family 3-bedroom home. The EQR values may be a fraction or multiple of a single EQR and are determined by the District's Rate Schedule adopted by the Board of Directors and further explained in the District's Rules and Regulations. There are multiple types of account types which will be further evaluated in Chapter 4.0; this chapter will focus on overall growth.

3.1.2 EQR Growth

Table 3-1 provides the EQRs in the system as of the 2nd quarter of 2020 billing records. The Base 2 pressure zone serves well over half of the EQRs in the system while Base 1 pressure zone serves less than 8-percent of the EQRs.

Table 3-1 – Existing EQRs by Pressure Zone

Pressure Zone	EQRs as of Q2 2020	% of Total EQRs
Base 1	**349.39**	**7.8%**
Base 2	**2,456.23**	**54.5%**
Base 3-4	**1,699.45**	**37.7%**
Total	**4,505.07**	

The quarterly billing data provided by the District from the second quarter of 2012 to the second quarter of 2020 was used to evaluate recent growth of EQRs in the system as shown in Figure 3-1. The district had steady EQR growth with an average growth rate of 15.6 EQRs between 2013 and 2016. Growth in base 1 accounted from much of the growth in the service area from 2014 to 2016. After 2016, there was a sharp increase with an average growth rate of 87.5 EQRs between 2017 and 2020 with base 3 experiencing the largest EQR growth in 2019 and 2020. Overall, the District's growth in EQRs from 2013 to mid-2020 was 10% or an average of 1.2% annually.

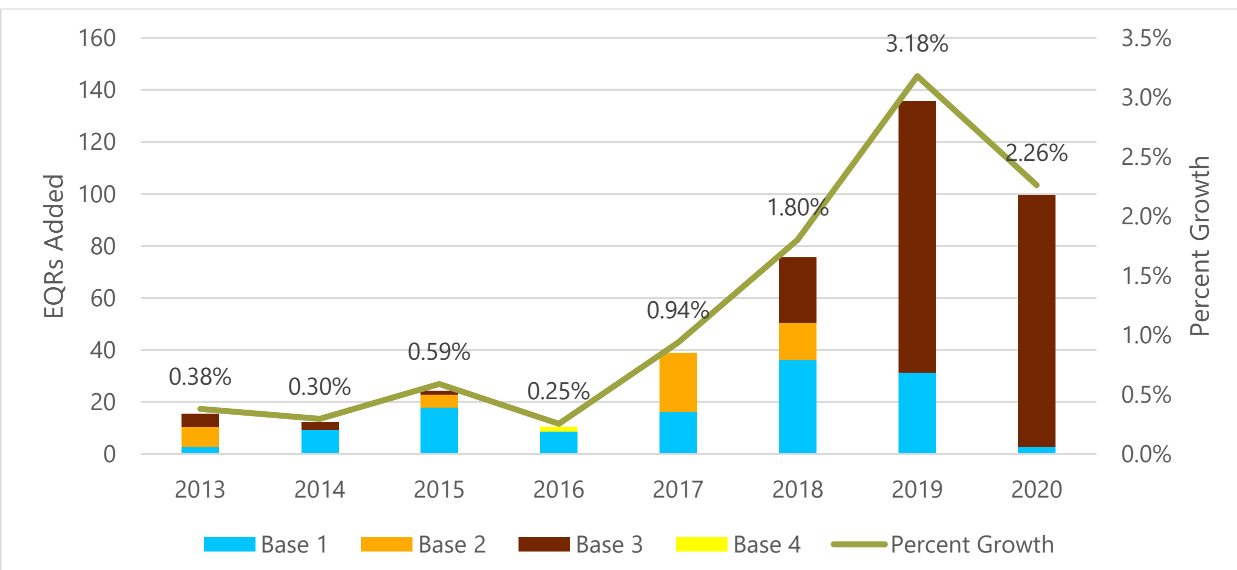


Figure 3-1 – Yearly EQR Growth

3.2 Anticipated Developments

The Summit County planning department provided data throughout the District's service area which showed the units built to date, units allowed and remaining units to be built. Many of the PUDs within the service area are broken out by parcels or areas which will help spatially allocate the remaining units to be built. For instance, the Keystone Resort PUD is broken out into neighborhoods and further subdivided into parcels or areas from there; the parcels in this definition differ from platted property parcels. The neighborhoods in the Keystone Resort PUD are Ski Tip, River Run, Mountain House, Lakeside, Old Keystone and Wintergreen.

The remaining units in each neighborhood and sub-area from the County's data are summarized in Table 3-2 through Table 3-4 below. These three tables are divided by pressure zone and show each sub-area or parcel served by the zone. The existing EQRs of surrounding properties in each sub-area where used to project the future EQRs in that area using the remaining units to be built. For instance, the single-family home units in Dercum's Dash exhibited larger EQR values (approximately 1.7 EQR per unit) than single family home units in Loveland Pass Village (approximately 1.1 EQR per unit). This methodology was applied throughout the service area for the single-family home, condo, and employee unit types. The commercial unit type assumed 1 EQR per 1,250 SF which was the result of analyzing existing commercial EQR and square footage totals. The 2012 Master Plan assumed 1 EQR per 1,000 SF of commercial space.

Table 3-2 – Base 1 Future EQR Growth Estimates

Area	Single Family Home	Condos	Employee	Commercial Square Footage	Future SFH	Future Condo	Future Employee	Future Commercial
	Remaining Units to be Built				**Estimated Future EQRs**			
Dercum's Dash	17		1		29.1		1.7	
Alders Lots 1-14	3				4.5			
Alders Lots 16-23 and Estates	2	10		7,800	3.0	12.0		6.5
Settlers Creek	11				16.1			
Trappers Crossing								
Ski Tip Ranch								
Ski Tip West		16		1,000		18.2		0.8
Base 1 Total	**33**	**26**	**1**	**8,800**	**52.6**	**30.2**	**1.7**	**7.3**

There are 91.9 additional EQRs projected to be added to the Base 1 pressure zone with a majority of the EQRs being single family homes or condos. Additionally, most of these EQRs appear to be infill where existing pipe infrastructure is already in place.

There are 1,502.3 additional EQRs projected to be added to the Base 2 pressure zone. There are three parcels in the Keystone Resort PUD that contribute 1,049.7 of the additional EQRs; these are River Run Neighborhood Parcel A and Mountain House Neighborhood Parcels B and C. These additional EQRs are anticipated to be added via large multi-story condo building with commercial space on ground level similar to the River Run Village. Approximately 397.4 of these EQRs are designated in Lakeside Neighborhood Parcel A which holds Saints John Condos and Tenderfoot Employee housing complexes along with the Keystone Conference Center. The remaining EQRs are single-family homes and condos in areas outside the Keystone Resort PUD.

There are 377.4 additional EQRs projected to be added to the Base 3 pressure zone. Four of the Wintergreen buildings were not included in the meter data during gathering of those records, therefore, the additional 80 EQRs are shown in Table 3-4. The four remaining Wintergreen building were approved for occupancy by January 2021.

Table 3-3 – Base 2 Future EQR Growth Estimates

Area	Single Family Home	Condos	Employee	Commercial Square Footage	Future SFH	Future Condo	Future Employee	Future Commercial
	Remaining Units to be Built				**Estimated Future EQRs**			
North Fork River Estates	1				1.4			
North Fork	3				4.5			
Government Lot 59	1				1.4			
Miller PUD		3				3.5		
Sonne PUD	2			2,236	2.3			1.9
Loveland Pass Village	1				1.1			
Sanctuary	1	10	2		1.4	13.8	2.8	
Cinnamon Ridge		2				2.0		
Snowdance		2				1.3		
Liftside		9				8.3		
Tenderfoot Lodge		3	1			2.9		
Slopeside				8,000				6.7
Mountain House Neighborhood Parcel B		281.5		14,000		292.2		11.7
Mountain House Neighborhood Parcel C		307		22,493		318.7		18.7
River Run Neighborhood		363		37,966		376.8		31.6
Tenderfoot Sub (Lakeside Parcel J)								
St Johns (Lakeside Parcel A)		318	94	15,000		303.5	81.4	12.5
Base 2 Total	**9**	**1298.5**	**97**	**99,695**	**12.0**	**1323.1**	**84.1**	**83.1**

Table 3-4 – Base 3 Future EQR Growth Estimates

Area	Single Family Home	Condos	Employee	Commercial Square Footage	Future SFH	Future Condo	Future Employee	Future Commercial
	Remaining Units to be Built				Estimated Future EQRs			
Clearwater (Lakeside Parcel B)		65				70.5		
Keystone Lake (Lakeside Parcel C)		60		1,423		60.6		1.2
Seasons at Keystone (Lakeside Parcel D)		41				52.3		
Soda Ridge Road East (Lakeside Parcel E)	8	25	16		9.9	22.8	13.8	
Homestead and Pines (Lakeside Parcel F)								
Keystone Village and Quicksilver (Lakeside Parcel G)				1,000				0.8
Sunrise Employee Housing (Lakeside Parcel H)				4,903				4.1
Wintergreen & Antlers Gulch						80		
Soda Ridge Road West (Old Keystone Parcel A)	4	15		480	5.2	16.5		0.4
Elk Crossing Ln - West Pines (Old Keystone Parcel B)	4				5.2			
Elk Circle - Golf Course (Old Keystone Parcel C)	21			6,020	29.1			5.0
Base 3-4 Total	**37**	**206**	**16**	**13,826**	**49.4**	**302.7**	**13.8**	**11.5**

Figure 3-2 provides the location of the future EQRS throughout the system. The EQRs in Saints John (Lakeside Parcel A) exist near the boundary of Base 2 and Base 3 pressure zones, they were assumed to be connected for Base 2 for this master plan. During this master plan project, several developments were in progress which are not included in the current EQR totals and counted in the remaining units to be built; these include:

- Village at Wintergreen Complex
    - The Village at Wintergreen Complex consists of 10 building with employee housing, deed-restricted housing, and long-term rental spaces. Some of these buildings received certificate of occupancy during the project while others continued through construction during master plan writing. Six of the ten buildings had water accounts and EQRs provided in the meter data, so the remaining four buildings were assumed 20 EQRs each as they're similar sized building. The county's data showed all the units as built so that is why there is zero remaining units in Table 3-4 for Wintergreen and Antler's Gulch.
- Seasons at Keystone Condo
    - The Seasons at Keystone Condos consists of 5 existing townhomes while additional townhomes where in construction during this master plan project. There appears to be space south of West Keystone Road for development of the remaining units.
- Clearwater Condos
    - The Clearwater Condos project is planned to consist of 3 multi-unit condo buildings just east of Keystone Lake. One building was nearing completion at the time of master planning writing and construction of foundations for the second unit was underway. All of these condos are expected to be completed by Fall of 2021. The first building has an EQR value of 23.5, the two remaining building are expected to be similar.
- One River Run
    - The One River Run development has been in the planning process for many years and plans to add a major building housing condos, lodges, and commercial space in the River Run Neighborhood. The building will be located between the River Run Gondola and Springs at River Run in what is currently the Hunki Dori parking lot. The project design currently has 107 Lodge Units, 95 multi-family condo units, 24,141 square feet of commercial space and 22,913 square feet of resort support space. This new development is anticipated to have an EQR value of 230 based on similar buildings nearby.

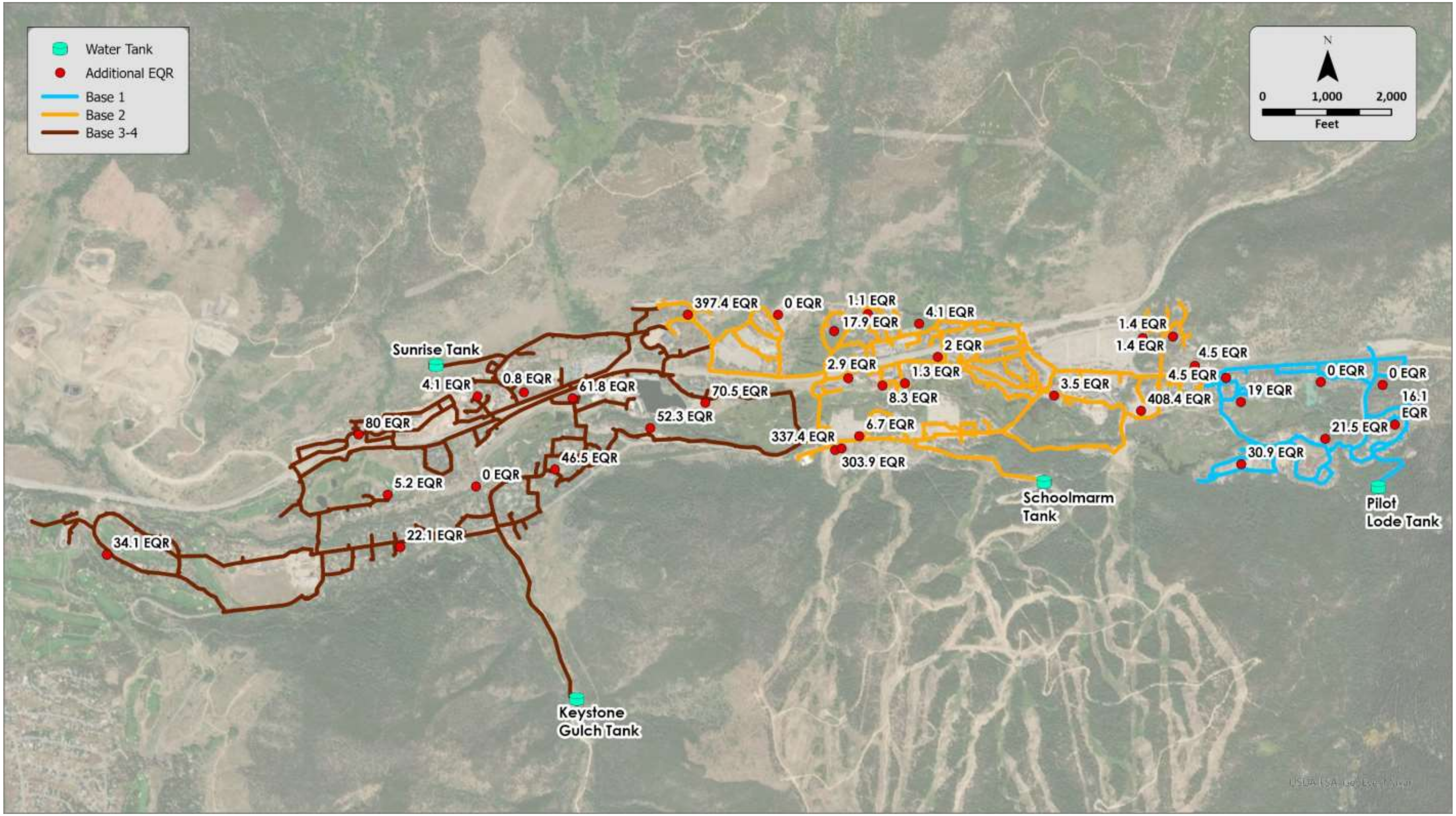


Figure 3-2 – Location of Future EQRs

Table 3-5 – Summary of EQR Growth

Pressure Zone	Single Family Homes	Condos	Employee Housing	Commercial	Total	Percent of Growth
	Future EQRs					
Base 1	52.6	30.2	1.7	7.3	**91.9**	4.7%
Base 2	12.0	1,323.1	84.1	83.1	**1,502.3**	76.2%
Base 3-4	49.4	302.7	13.8	11.5	**377.4**	19.1%
Total	**114.0**	**1,655.9**	**99.7**	**101.9**	**1,971.6**	-

Table 3-5 provides a summary of growth by type of EQR in each pressure zone. Over 76 percent of the growth is anticipated on the Base 2 pressure zone while about 19 percent is anticipated in the Base 3-4 pressure zone. Overall, the additional 1,971.6 EQRs estimated represents a growth of 43% of existing EQRs as of the second quarter of 2020. The total buildout EQRs are estimated to be 6,476.7 in the District's service area which is about 217 EQRs less than the 2012 master plan projections.

Using the historical growth rate of 1.2% observed from 2012 to 2020 the District is estimated to serve 5,156.5 EQRs by 2030 which includes an additional 80 EQR at Wintergreen anticipated to be in service in the 2nd half of 2020. However, the growth can change rapidly when large complexes are built such as One River Run. Recall, there is potential for other large complexes in the Mountain House and Saints John areas.

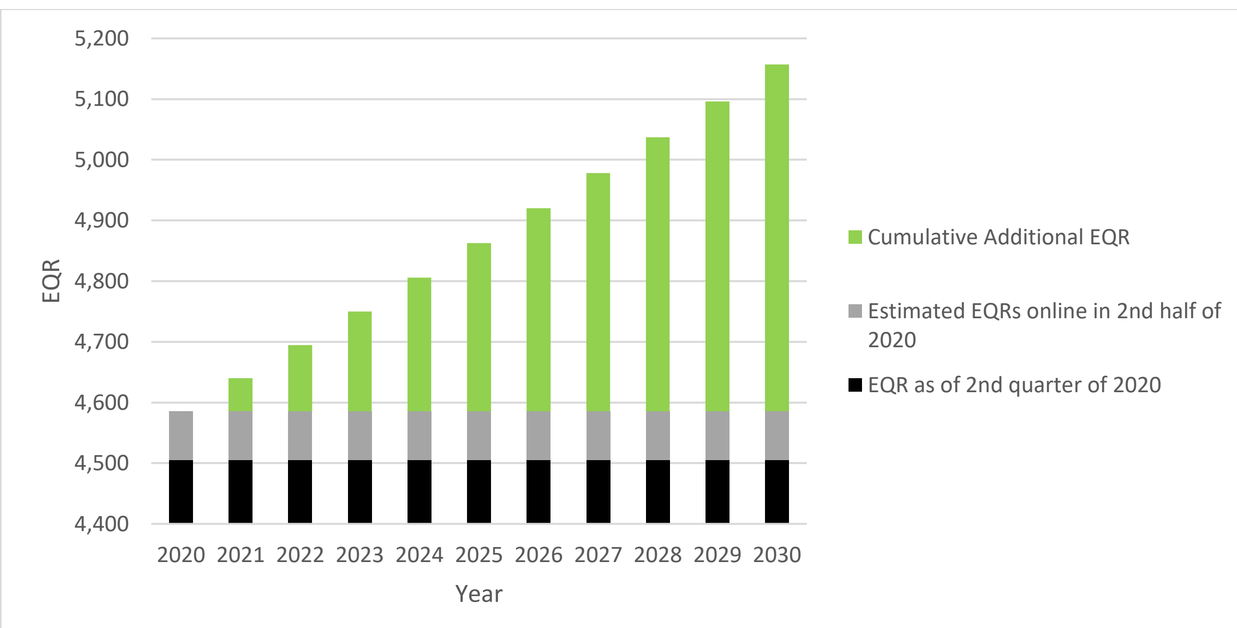


Figure 3-3 – Yearly EQR Growth

4.0 Water Use Characterization and Planning

Water use characterization of the SRWD system involves analysis of the existing water production and demand data to better understand the system's water use. Water use characterization is necessary to assess the capabilities of the existing facilities to adequately serve current water demands and to ensure the design and operation of proposed water system components can sufficiently accommodate future water demands in the District's service area.

This chapter provides an overview of the District's historical water use trends and presents recent water production and demand trends. The historical trends were used to define water demand factors which will be used with the anticipated land development data to estimate future demands. The results of this water use analysis were incorporated into the distribution system hydraulic model to evaluate both existing and future system performance. Results from the modeling analysis will guide future recommended water system capital improvements.

4.1 Definition of Terms

Water demand is described in the following terms:

- Average Daily Demand (ADD) – The total volume of water delivered to the system over a year divided by 365 days. The average use in a single day expressed in gallons per day.
- Maximum Month Demand (MMD) – The gallons per day average during the month with the highest demand. The highest monthly usage typically occurs during a summer month.
- Maximum Day Demand (MDD) – The largest volume of water delivered to the system in a single day expressed in gallons per day.
- Peak Hour Demand (PHD) – The maximum volume of water delivered to the system in a single hour.
- Peaking Factor (PF) – The ratio of the MDD over ADD.
- Single-Family Equivalent Rating (EQR) – EQRs are used to equate the water usage of different properties or uses.
- Water Demand Factor (WDF) – WDFs are assigned to the meter types and are expressed as gallons per day (gpd) per EQR (gpd/EQR). The WDFs are used to estimate future water demands.

4.2 Historical Water Use

Existing water production and customer meter data was provided by SRWD for analysis and use in this water system master plan. Table 4-1 below outlines the applicable data set and the associated time period.

Table 4-1 – Production and Customer Meter Data Sets

Data Set	Time Period	Description
Water Production Data	2013 through June 2020	Data included daily volume of production at each facility along with daily volume of the system.
Customer Meter Data	2013 through 2019	Data included quarterly water use volume, EQR values and meter type for each meter account in the District service area.

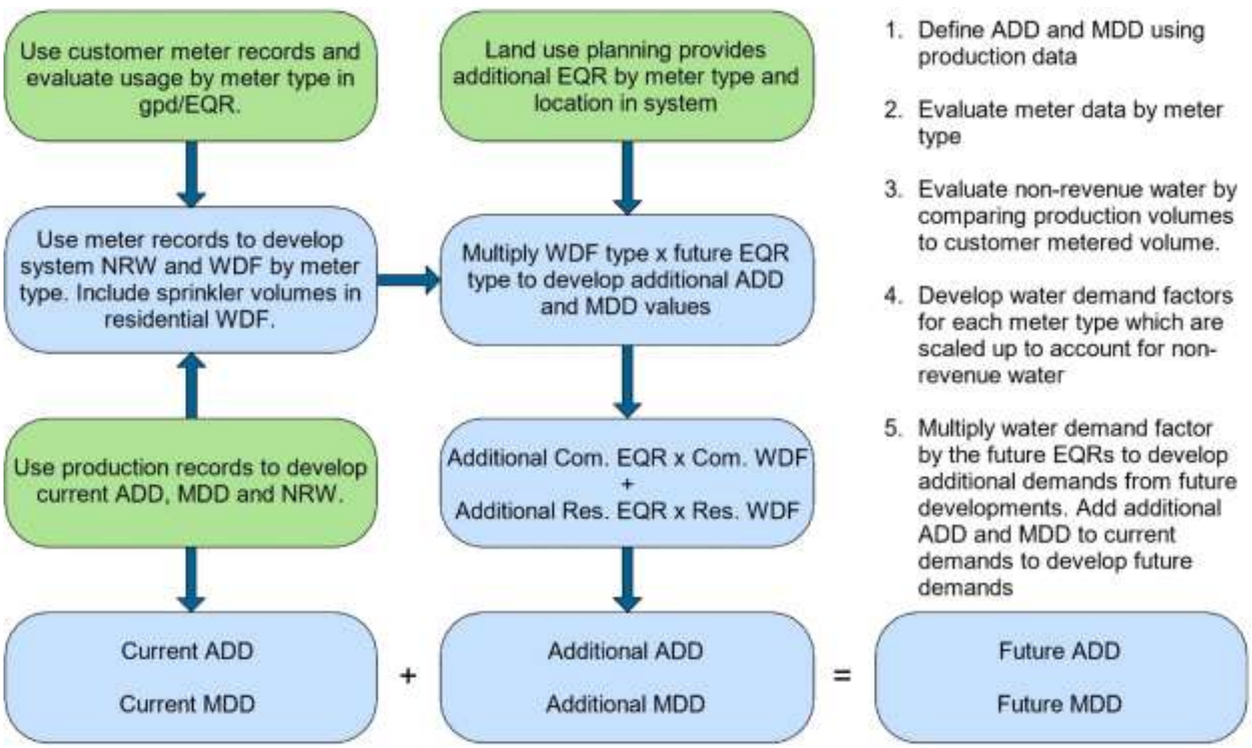


Figure 4-1 – Water Use Characterization Flow Chart

Figure 4-1 shows a flow chart of the water use characterization and future demand development process. The production data was analyzed to develop current ADD and MDD for the system. The analysis will also define the seasonal operations of the system which will drive the scenarios setup in the hydraulic model. The customer meter data will be used to analyze water use by the type of meter and ultimately develop water demand factors (WDF) for each meter type. The water demand factors will be used in conjunction if the land use planning effort to develop the additional ADD and MDD throughout the system.

4.2.1 Water Production

Water sources for the District include various groundwater sources throughout the service area. A graph representing monthly water production over the last 7 years is provided in Figure 4-2. The figure generally shows steady water production between 2013 and 2017 with an increase in production starting in 2018. Season production variances will be reviewed in a later subsection. In 2018, the District transferred water to neighboring East Dillon Water District causing the large increase during the summer of 2018.

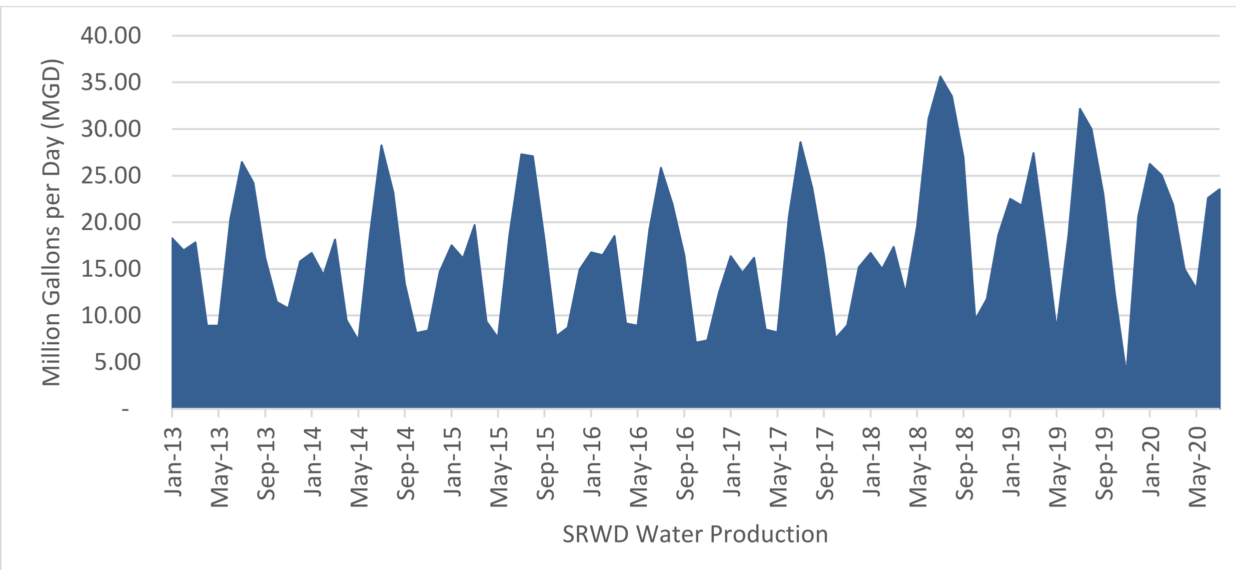


Figure 4-2 – Water Production by SRWD

Production by Facility

The annual production volume of the District's two water sources greatly varies as shown in Figure 4-3 with the Base 2 WTP contributing the most production in the system. Base 3 WTP was shut down for construction of the new water treatment plant in mid-2019. A part of the difference in water production is the Base 3 WTP can only feed demands in the Base 3 pressure zone while water from the Base 2 WTP can be distributed throughout the entire system. Prior to construction of the new Base 3 WTP, the District would annually transfer an average of 10 million gallons from Base 2 to Base 3.

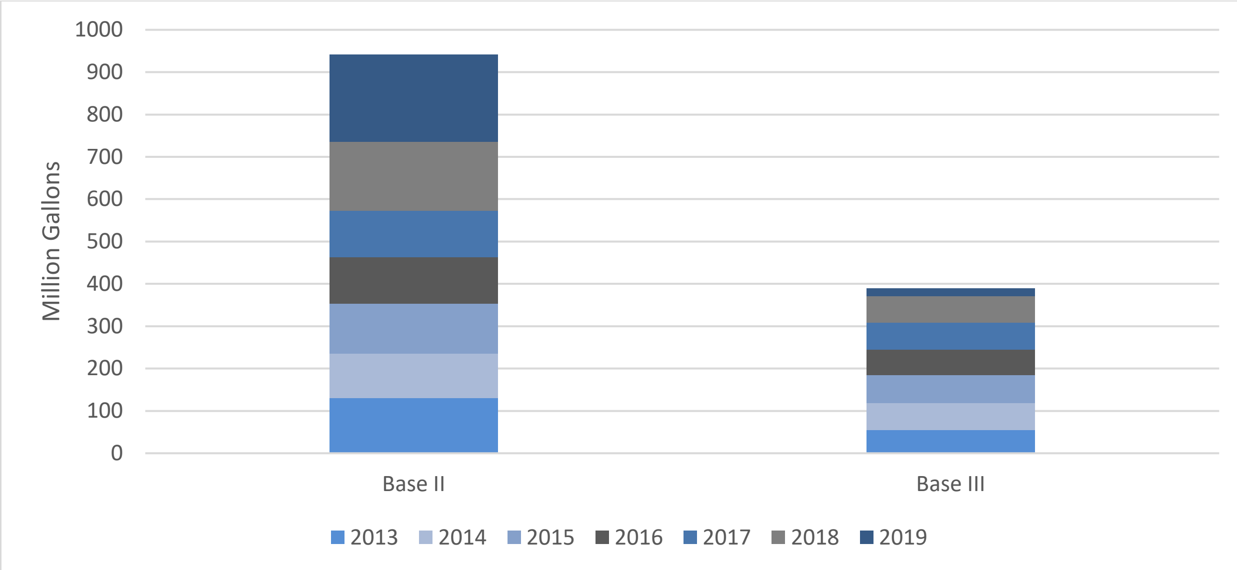


Figure 4-3 – Historical Production Totals by Facility

Seasonal Variations

Water production and water usage varies greatly depending on the season. The average daily water usage per month was evaluated to determine which months had the highest water demand. Figure 4-4 shows the monthly water production variations from 2013 to 2019 for the District.

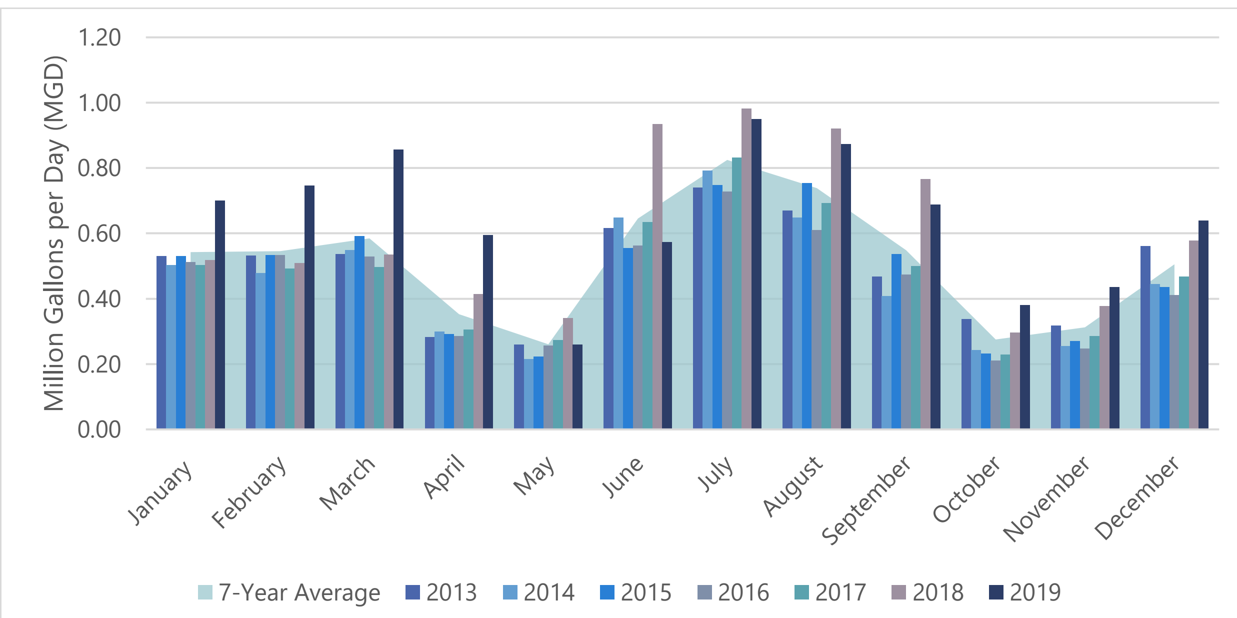


Figure 4-4 – Seasonal Variation in Water Production

As shown in the previous figure, water production fluctuates depending on the month and season. Water use is generally higher in the summer compared to other seasonal periods likely due to sprinkler and irrigation use, as well as other summer recreational activities such as swimming pools and outdoor water features. The lowest water production occurs in the fall and spring months which is referred to as shoulder season throughout this master plan. The occurrence of low demands during the shoulder seasons highlights the impact Keystone Mountain resort has on water demands in the District as the resort is typically shut down during these periods. Water production in the winter months of December through March are noticeably higher than the shoulder season.

Table 4-2 below presents the three seasonal periods used in this Master Plan and the respective average water demand during those seasonal periods; these seasonal periods are used in model simulations. The months of July and August do appear to be slightly higher than June and September so a summer peak demand scenario with seasonal average day demands of 0.781 MGD was calculated.

Table 4-2 – Seasonal Average Water Demands and Model Scenarios

Seasonal Period	Included Months	Seasonal Average Day Demand (MGD)*
Summer Demands	**June, July, August, September**	**0.691**
Summer Peak Demands	**July, August**	**0.781**
Spring and Fall Demands (Shoulder Period)	**April, May, October, November**	**0.300**
Winter Demands	**December, January, February, March**	**0.544**

* ***Average Demand in this table is calculated by dividing the total water produced during the seasonal periods by the number of days within the respective seasonal period.***

Average Day Demand (ADD)

Average Day Demand (ADD) is defined as the total volume of water delivered to the system over a year divided by the number of days in that year. ADD is an important metric to understand because it is utilized when analyzing existing water demands as well as estimating future water demands. Likewise, future estimated ADD should be utilized when planning for future source water availability and appropriations securement.

Figure 4-5 presents the average day demand, maximum month demand (MMD), maximum day demand (MDD) and highest annual peaking factors for the District. The MMD, MDD and peaking factors will be further evaluated in this chapter. The ADD for the District remained steady from 2013 to 2017 with the years of 2018 and 2019 showing slight increases up to near 600,000 gallons per day.

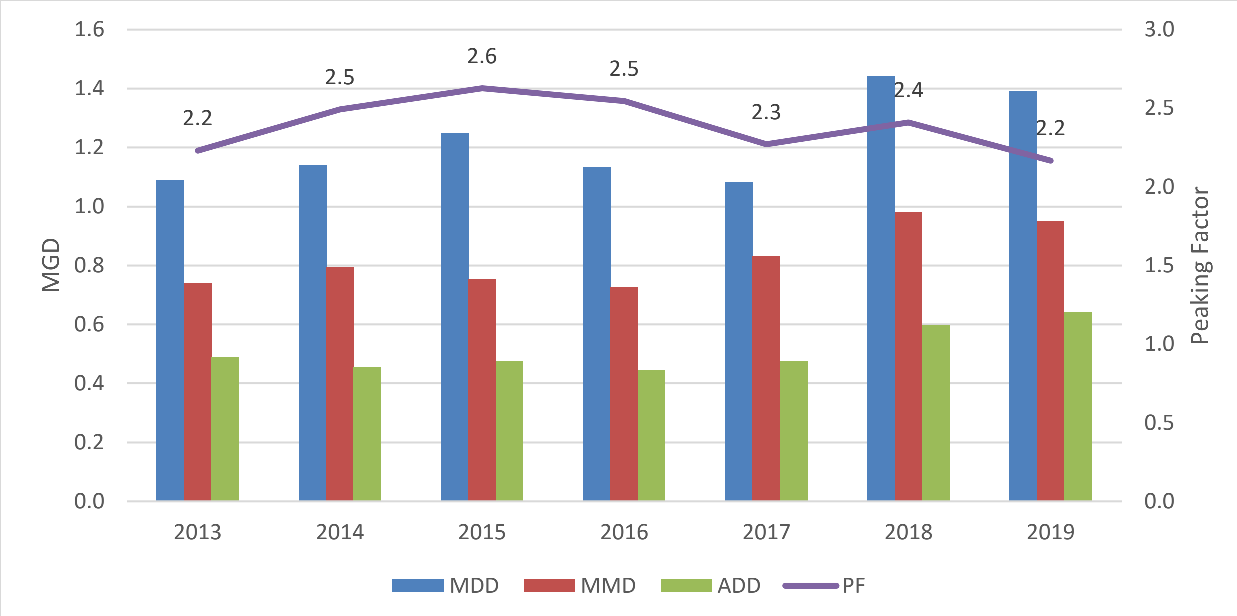


Figure 4-5 – Average Day Demands

Maximum Month Demand (MMD)

Maximum Month Demand (MMD) reviews average day demands by each month and identifies the month of the year with the highest demand. MMD is commonly used to better understand seasonal variations in water production. The MMD typically occurs during a summer month. Table 4-3 displays monthly demands and identifies the MMD for the District.

Table 4-3 – Maximum Month Demand

Month	2013	2014	2015	2016	2017	2018	2019
MMD in Million Gallons per Day (MGD)							
January	0.53	0.50	0.53	0.51	0.50	0.52	0.70
February	0.48	0.44	0.49	0.49	0.45	0.46	0.68
March	0.54	0.55	0.59	0.53	0.50	0.53	0.86
April	0.27	0.29	0.28	0.28	0.29	0.40	0.58
May	0.26	0.21	0.22	0.26	0.27	0.34	0.26
June	0.60	0.63	0.54	0.54	0.61	0.90	0.55
July	0.74	0.79	0.75	0.73	0.83	0.98	0.95
August	0.67	0.65	0.75	0.61	0.69	0.92	0.87
September	0.45	0.39	0.52	0.46	0.48	0.74	0.67
October	0.34	0.24	0.23	0.21	0.23	0.30	0.38
November	0.31	0.25	0.26	0.24	0.28	0.37	0.42
December	0.56	0.45	0.43	0.41	0.47	0.58	0.64
MMD	0.74	0.79	0.75	0.73	0.83	0.98	0.95
Month of Record	July	July	July	July	July	July	July

Maximum Day Demand (MDD)

Maximum Day Demand (MDD) is the largest volume of water delivered to the system in a single day expressed in million gallons per day. MDD is also commonly referred to as peak daily or peak water demand. Figure 4-5 presents the MDD for the District which shows similar trends to the ADD; the MDD remains steady from 2013 to 2017 then increases in 2018 and 2019 to near 1.4 million gallons per day.

Table 4-4 – Maximum Day Demands and Date of Record

Year	Base 2 MDD	Base 3 MDD	System MDD[a]
	MDD expressed in MGD		
2013	0.94	0.48	1.09
Date of record.	*8/5/2020*	*6/26/2013*	*7/7/2013*
2014	0.87	0.68	1.14
Date of record.	*7/12/2014*	*7/29/2014*	*7/12/2014*
2015	0.91	0.46	1.25
Date of record.	*7/5/2015*	*6/22/2015*	*8/1/2015*
2016	0.8	0.57	1.13
Date of record.	*7/9/2016*	*6/18/2016*	*7/9/2016*
2017	0.79	0.48	1.08
Date of record.	*7/2/2017*	*6/21/2017*	*7/21/2017*
2018	1.08	0.49	1.44
Date of record.	*7/9/2018*	*8/2/2018*	*8/2/2018*
2019	1.39	0	1.39
Date of record.	*8/14/2019*	*offline*	*8/14/2019*
2020	1.04	0	1.04
Date of record	*7/6/2020*	*offline*	*7/6/2020*

[a] *System MDD = Base 2 Production + Base 3 Production*

Table 4-4 further illustrates the maximum days of production both individually for Base 2 and Base 3 water treatment plants along with the date of maximum combined production of the two WTP. The maximum day of production for the Base 2 WTP occurred on August 14, 2019 when 1.39 million gallons of water were produced. At this time, the Base 2 WTP provided water for the entire system as the new Base 3 WTP was under construction. Prior to construction of Base 3 WTP, the Base 2 WTP maximum production was 1.08 MGD. The MDD combined WTP production occurred in 2018 when 1.44 million gallons of water produced. Similar to the ADD in Figure 4-5, the MDD displayed a noticeable increase in 2018 and 2019.

Water Production and Appropriation Summary

The district is permitted 5,328 acre-feet of water between the 7 wells in use. In 2018 and 2019 nearly 700 acre-feet of water was produced which is roughly 13 percent of the permitted water available to the district. Therefore, it is apparent the District has ample water right capacity to accommodate future growth in the service area. Furthermore, the appropriated volume of water for each of the Base 2 and Base 3 wells is greater than the overall system consumption.

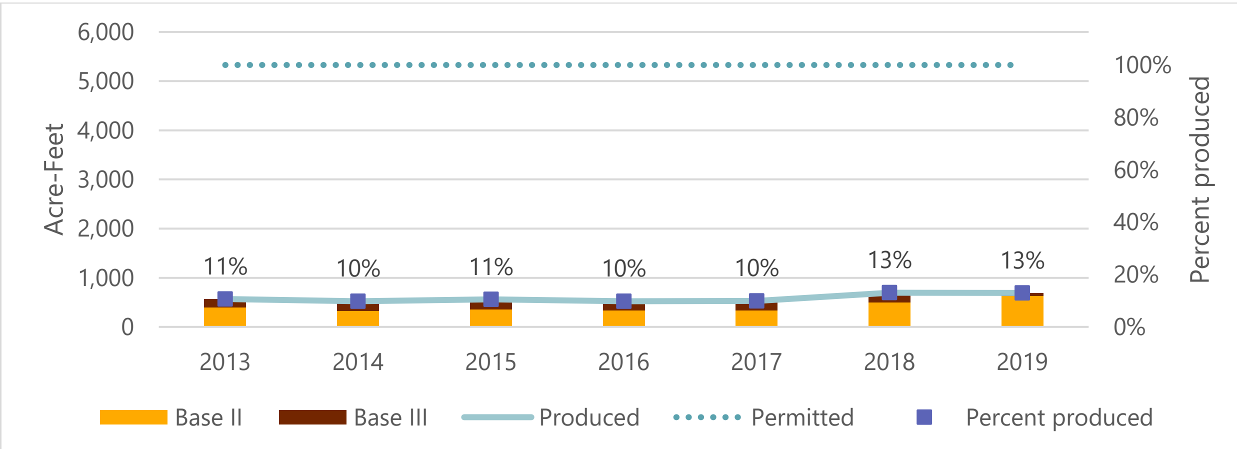


Figure 4-6 – SRWD Production and Appropriation Comparison

4.2.2 Metered Water

This subsection will switch focus to analysis of the metered water data provided in the customer billing data set.

User Types

The water billing data included the user type and EQR for each individual account. There are currently ten types of user accounts in the billing data which are bulleted below:

- Single Family Home
- Condos
- Employee Housing
- Mixed Commercial
- Other Commercial
- Restaurants
- Pools
- Recreation Center
- Irrigation
- Vacant Lots

The meter user type nomenclature from the billing data provided was compared to the 2012 Master Plan to develop a baseline for comparison of trends since 2012. These user type accounts may be further refined into traditional user types of "Residential" and "Commercial". Table 4-5 shows a comparison of the 2012 Master Plan user types, user types from in the 4th quarter of the billing data, total EQRs and the grouping of residential and commercial types. It appears the 2012 master plan added the restaurant EQRs into the multi-family user types.

Table 4-5 – EQR User Types

<table>
<tr><th>Group</th><th>User Types from Q4 2012 Billing Data</th><th>2012 Master Plan User Types</th><th>2012 Master Plan EQRs</th><th>Billing Data EQRs as of Q4 2012</th></tr>
<tr><td rowspan="4">Residential</td><td>Single Family Home</td><td rowspan="2">Single Family Homes</td><td rowspan="2">485.92</td><td>525.66</td></tr>
<tr><td>Vacant Lot</td><td>1</td></tr>
<tr><td>Condos</td><td>Multi-Family Homes</td><td>2,417.73</td><td>2,273.1</td></tr>
<tr><td>Employee Housing</td><td>Employee Housing</td><td>240.59</td><td>255.6</td></tr>
<tr><td rowspan="6">Commercial</td><td>Mixed Commercial</td><td rowspan="2">Commercial</td><td rowspan="2">860.28</td><td>819.81</td></tr>
<tr><td>Other Commercial</td><td>16.8</td></tr>
<tr><td>Restaurants</td><td>Appears to be included in Multi-Family Homes of 2012 MP</td><td>-</td><td>138.02</td></tr>
<tr><td>Pools</td><td>Resort Support</td><td>62.33</td><td>61.55</td></tr>
<tr><td>Recreation Center</td><td></td><td></td><td>1.15</td></tr>
<tr><td>Irrigation</td><td>Irrigation</td><td>-</td><td>-</td></tr>
</table>

Analysis of the customer meter demand information was used to determine overall customer water consumption, develop water use trends by meter type, aid in determining future water use values and estimate non-revenue water present in the system by comparing metered water with production water values. The customer meter data was provided in quarterly occurrences.

Overall, residential accounts are contributing the most water demand to the system with commercial and mixed-use accounts exhibiting a relatively steady water demand trend. Sprinkler and irrigation accounts continue to exhibit typical seasonal trends with most use generally occurring during the third quarter of the year. While some residential parcels have separate sprinkler meters, the large spikes in residential use during the summer months is likely attributed to increased outdoor use such as lawn watering and water used for other recreational activities. A chart showing total volume used by account type for the District service area are shown in Figure 4-7. The total volume used coincides with the number of EQRs in the system with condos covering both the most EQRs and water volume used.

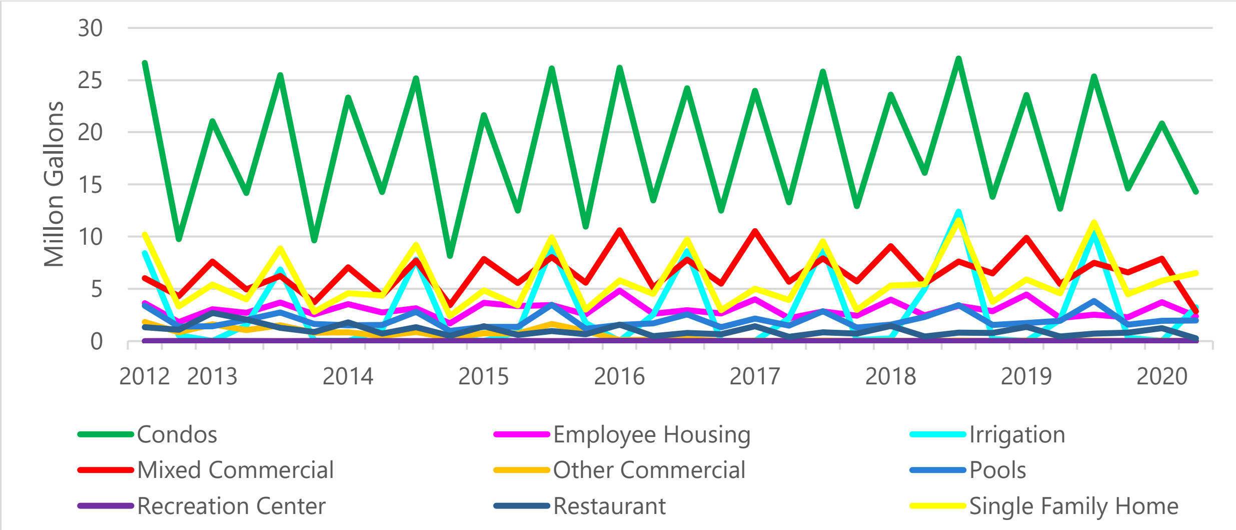


Figure 4-7 – Water Use by Account Type

Figure 4-8 provides the water use in gallons per day per EQR separated by residential and commercial account type which are grouped in Table 4-5, as well as the totalized gallon per day per EQR. The overall gpd/EQR line shows a general trend downward from 2013 to 2019. The larger instances of commercial gpd/EQR in 2013-2015 are likely attributed to a laundromat account that used significant amounts of water but appeared to cease operations in 2016.

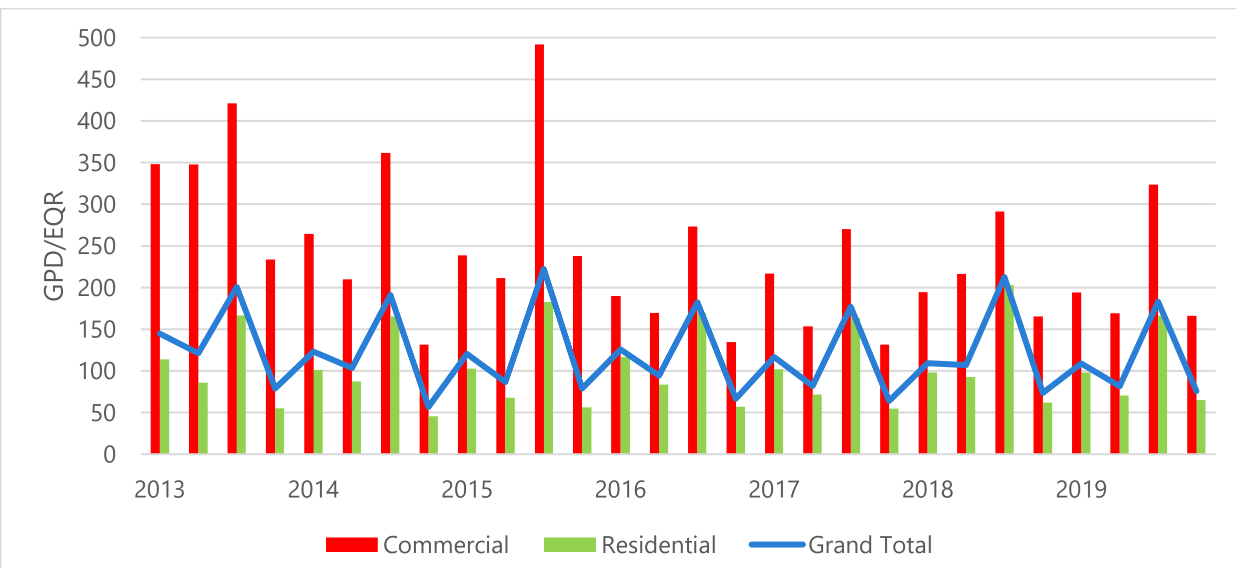


Figure 4-8 – System Water Use

20 Largest Users

Table 4-6 shows the twenty largest volume users for the period of meter data provided. While these largest users aren't a vital part of the analysis the table provides a background of where the largest volumes of water are consumed.

Table 4-6 – 20 Largest Users from Metered Data

Account Number	Pressure Zone	Type	Service Address	Account Name	Avg. Gallons Used per Year
335800-01	Base 3	Mixed Commercial	Keystone Lodge #2	Keystone Lodge Main	7,084,286
229520-01	Base 2	Condos	Buffalo & Dakota	Buffalo & Dakota	4,157,757
229400-01	Base 2	Condos	23110 US Highway 6	Gateway	2,823,679
229540-01	Base 2	Mixed Commercial	Silvermill	Silvermill	2,814,388
220900-01	Base 2	Condos	52 Hunkidori Ct	The Springs	2,822,028
229530-01	Base 2	Condos	Expedition Station	Expedition Station	2,671,027
239150-01	Base 2	Mixed Commercial	23044 US Highway 6	The Inn @ Keystone	2,731,200
367900-01	Base 3	Employee Housing	245 Tennis Club Rd	Sunrise	2,479,234
229500-01	Base 2	Mixed Commercial	Black Bear & Jack Pine	Jackpine, Blackbear	2,552,593
234900-01	Base 2	Mixed Commercial	633 Conference Center Dr	Keystone Conference Center	2,493,900
221000-01	Base 2	Condos	280 Trailhead Dr	Lone Eagle	2,292,517
221010-01	Base 2	Condos	20 Hawk Cir	Red Hawk Lodge	2,265,801
265201-01	Base 2	Employee Housing	1515 Lone Pine Cir	Tenderfoot Employee Housing	2,135,042
227054-01	Base 2	Condos	22784 US Highway 6	Tenderfoot Lodge	2,029,972
333800-01	Base 3	Other Commercial	185 Tennis Club Rd	Laundry	1,459,507
367800-01	Base 3	Employee Housing	95 & 155 Tennis Club Rd	Sunrise I A & B	1,789,406
221700-01	Base 2	Condos	22097 Sts. John Rd	Saint John Condos	1,575,703
322400-01	Base 3	Condos	21700 US Highway 6	Lodgepole	1,662,471
220010-01	Base 2	Condos	22864 US Highway 6	Liftside Lodge	1,604,743
265202-01	Base 2	Employee Housing	1530 Lone Pine Cir	Tenderfoot Housing LLC	1,620,958

4.2.3 Non-Revenue Water (NRW)

Water utilities routinely produce more water than the volume of the customer's metered water. This difference in water produced versus water billed is termed non-revenue water (NRW) which is broken into specific types of water loss including unbilled authorized consumption, apparent losses, and real losses. This definition of non-revenue water, real losses, apparent losses and unbilled authorized consumption is provided in the IWA/AWWA Water Balance shown in Table 4-7.

Table 4-7 – IWA/AWWA Water Balance[1]

<table>
<tr><th colspan="7">The IWA/AWWA Water Balance</th></tr>
<tr><td rowspan="9">Volume From Own Sources (corrected for known errors)</td><td rowspan="11">System Input Volume</td><td>Water Exported (corrected for known errors)</td><td colspan="3">Billed Water Exported</td><td>Revenue Water</td></tr>
<tr><td rowspan="10">Water Supplied</td><td rowspan="4">Authorized Consumption</td><td rowspan="2">Billed Authorized Consumption</td><td>Billed Metered Consumption</td><td rowspan="2">Revenue Water</td></tr>
<tr><td>Billed Unmetered Consumption</td></tr>
<tr><td rowspan="2">Unbilled Authorized Consumption</td><td>Unbilled Metered Consumption</td><td rowspan="8">Non-revenue Water</td></tr>
<tr><td>Unbilled Unmetered Consumption</td></tr>
<tr><td rowspan="6">Water Losses</td><td rowspan="3">Apparent Losses</td><td>Customer Metering Inaccuracies</td></tr>
<tr><td>Unauthorized Consumption</td></tr>
<tr><td>Systematic Data Handling Errors</td></tr>
<tr><td rowspan="3">Real Losses</td><td>Leakage on Transmission and Distribution Mains</td></tr>
<tr><td rowspan="2">Water Imported (corrected for known errors)</td><td>Leakage and Overflows at Utility's Storage Tanks</td></tr>
<tr><td>Leakage on Service Connections up to the Point of Customer Metering</td></tr>
<tr><td colspan="7">NOTE: All data in volume for the period of reference, typically one year.</td></tr>
</table>

In the past, the non-revenue water performance was measured using percentage performance indicators including volumetric type and financial type percentages. In recent years, the AWWA has recommended to discontinue the percentage-based indicators and use new indicators. The percentages can be skewed since even if the water loss volume remains the same year over year, increased production and demand values will lower the percentages. The new indicators include the Loss Cost Rate and Normalized Water Losses indicator. These new indicators still use financial information and volumetric information in the calculations; however, they use number of service connection in the metric. Since the scope of this master plan does not evaluate cost of service and the loss cost rate indicator requires production cost information, only the normalized water losses indicator will be evaluated.

Figure 4-9 shows the annual water production volume, metered volume including temporary water volumes (water through hydrants for construction or sold to adjacent water districts) and normalized water losses for the District. Normalized water losses are measured in gallons per connection per day. All types water losses were combined for the analysis of this indicator.

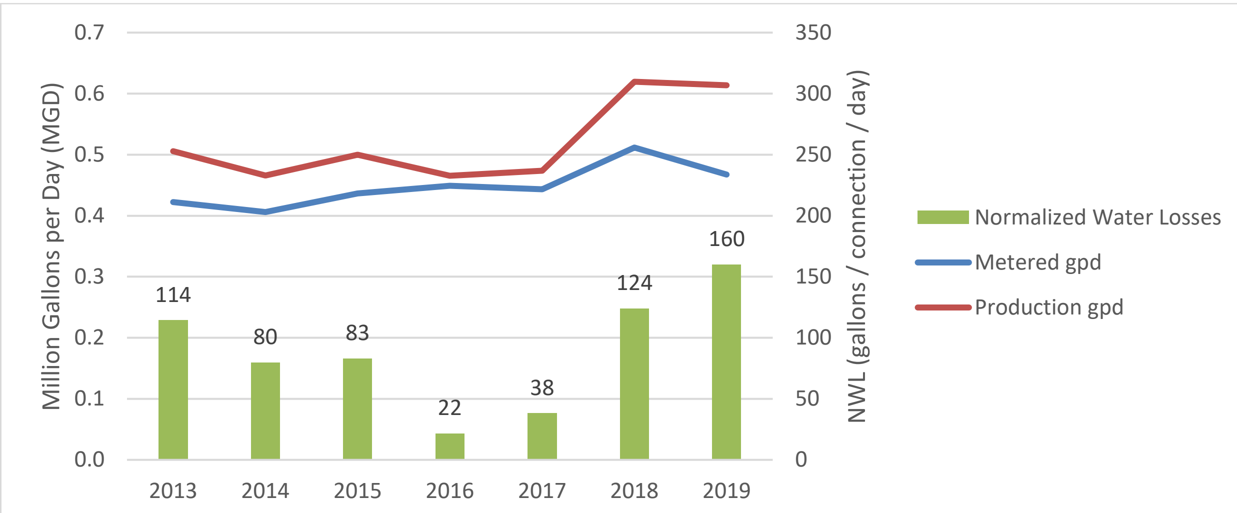


Figure 4-9 – Historical Annual Normalized Water Losses

The normalized water loss indicators are relatively new so very little benchmarking exists at state or nationwide level. The AWWA recommends utilities evaluate water loss using these indicators and then set goals to reduce water losses in the systems. An alternate method of evaluation using miles of transmission main pipe to normalized non-revenue water can be used when the connections are less than 32 per mile of pipe.

Since very little benchmark data exists for normalized water losses, the District's water loss as a percentage was evaluated to be 13.3% on average which is similar to other water Districts in the region.

4.2.4 Water Demand Factors

The final goal of reviewing the meter data is to develop water demand factors. These factors provide a water use in gallons per day per EQR for the various account types. These factors can vary by type to match observations in the meter data and are also scaled to account for water loss and appropriate demands correctly in the model. Additionally, the irrigation demand is distributed into the residential account types as irrigation meters do not have EQRs themselves. Therefore, the maximum day demand water factors are significantly higher than the average day demand water factors. Table 4-8 provide the water demand factors for the various account types and demand scenarios.

Table 4-8 – Water Demand Factors

Demand Scenarios	MDD	Summer ADD	Shoulder ADD	Winter ADD
Types	gpd/EQR			
Condos	270	180	60	110
Employee Housing	170	100	85	165
Irrigation	N/A	N/A	N/A	N/A
Mixed Commercial	215	140	85	135
Other Commercial	110	75	25	55
Pools	900	525	350	250
Recreation Center	255	150	35	200
Restaurant	160	85	55	135
Single Family Home	455	235	60	100
Vacant Lot	N/A	N/A	N/A	N/A

Water Demand Factor Calibration

The water demand factors presented in Table 4-8 where checked with their respective EQR types to ensure that the total volume summations closely matched the model scenarios presented in Table 4-2. This shows that the water demand factors are calibrated to the existing demand and will be used with the future EQRs to project future water demands.

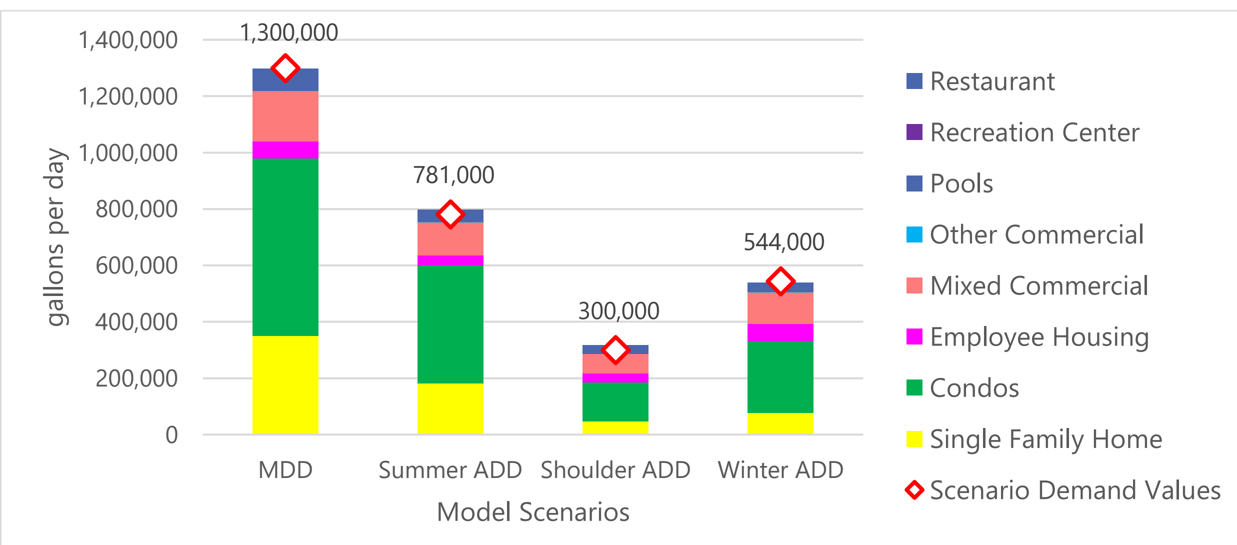


Figure 4-10 – WDF Calibration Check

4.3 Water Demand Projections

The water demand projections will combine the EQR growth estimates from Chapter 3 with the water demand factors identified in Table 4-8. The data will be combined to provide maximum day and average demands at buildout in the service area. The maximum day demands will be used in the hydraulic model to evaluate distribution system performance with these added demands while average day demand can be used to very water rights held by the District are sufficient for the anticipated growth.

4.3.1 Future Water Demand Projections

The EQR types used to project future water demands were single family homes, condos, employee housing and commercial types. The additional EQR types were multiplied by their respective water demand factor to provide additional demands for the four demand scenarios. Additional demands by pressure zone along with total additional demands at buildout are provided in Figure 4-11 and Figure 4-12, respectively.

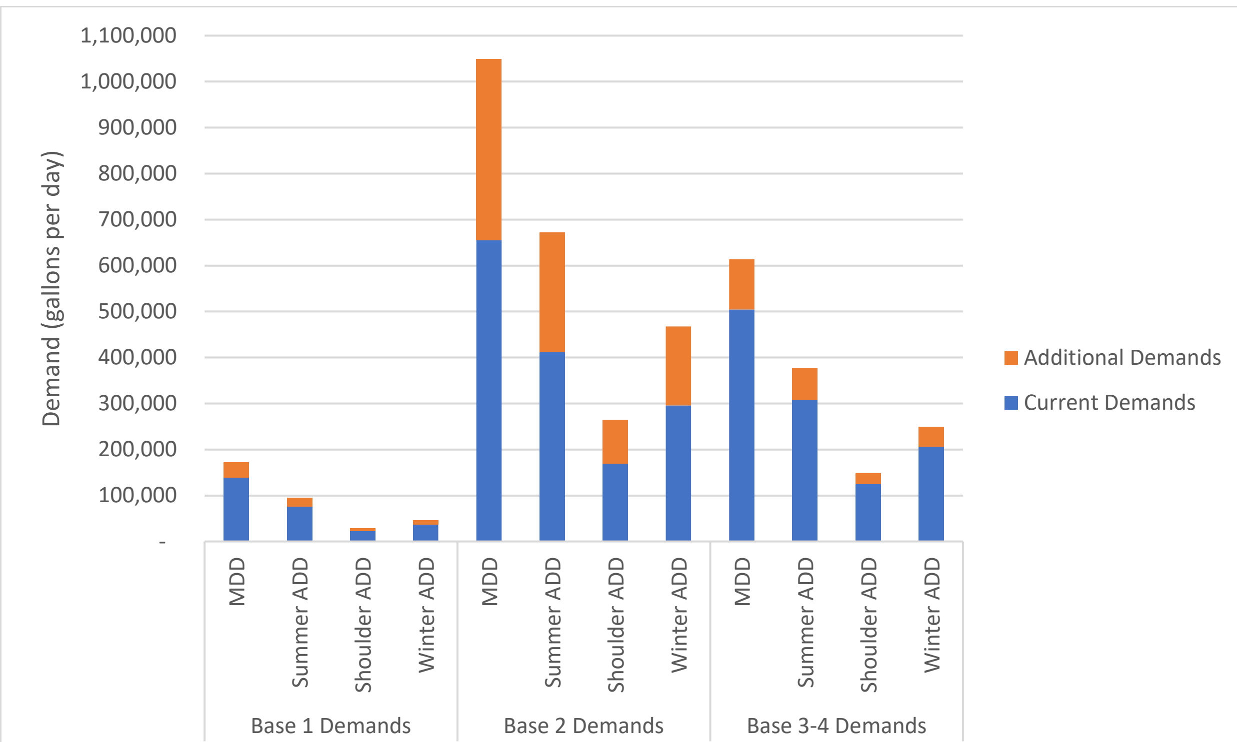


Figure 4-11 – Current and Additional Demands by Zone and Scenario

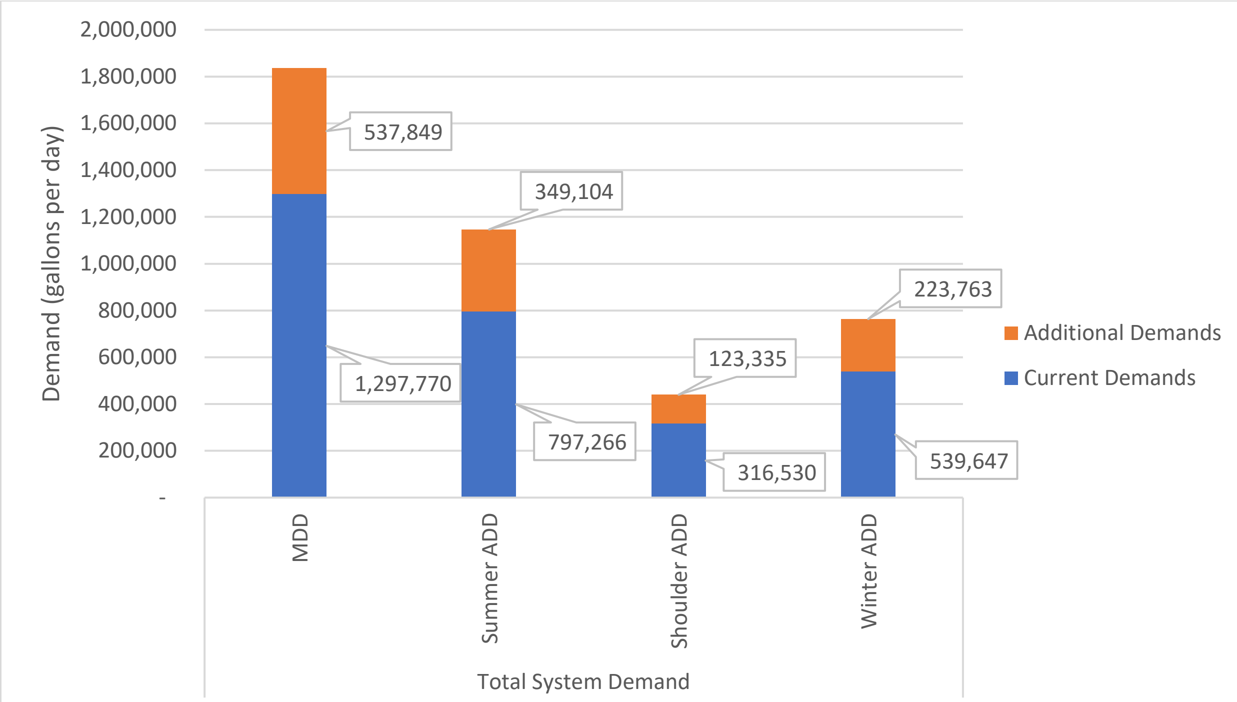


Figure 4-12 – Total System Current and Additional Demands

4.4 Summary and Takeaways from Water Use Characterization

- System wide water use appears to be trending downward on a gallon per day per EQR basis. This downward trend is typical of public water systems due to increased public education on water conservation, installation of more efficient water fixtures and water system repairs.
- The MDD at buildout is estimated to be 1.84 MGD, with pressure zone estimates shown below. These projects assume the occupancy rate of residences in the District remains steady. If more residences are occupied full time, then demands may increase.
  - Base 1 = 172,000 gallons
  - Base 2 = 1,050,000 gallons
  - Base 3-4 = 613,000 gallons
- These MDD values are used to size pump stations, tanks, and water treatment plants to ensure adequate water supply is available.
- The District appears to have adequate volume and instantaneous flow water rights secured to meet future demands. Buildout ADD is projected to be 922,000 gallons per day which is approximately 1,033 acre-feet annually.

5.0 Water Distribution System Model Update

This chapter provides an overview of the data sources used to create the hydraulic model for the District's system.

InfoWater Pro® (Version 3.0) hydraulic modeling software was used for the development and calibration of the model. InfoWater Pro® is a fully GIS integrated water distribution modeling and management software application. InfoWater®, which runs on the EPANET hydraulic engine, integrates water network modeling with ArcGIS Pro.

5.1 Existing Model Conversion and Development

The following information was provided by the District for use in the development of the hydraulic model:

- A GIS geodatabase of the water system components was used to develop the model network. Components in these data included water main, valves, hydrants, tanks, pump stations, water treatment plants, and pressure reducing valves.
- As-builts of the water treatment plants and booster station were used to develop facility elements within the model
- Water main sizes are provided as nominal sizes (i.e., 8-inch diameter). The inside diameters of the pipe were updated to the known diameter from standards for ductile iron pipe. The inside diameter of ductile iron pipe is slightly larger than the nominal size and will affect the model results.
- Finished water flow rates, system pressures, and reservoir storage levels were collected from the SCADA system in 5-minute increments during the flow testing period for use in model calibration. Monthly data was provided for the various seasonal demands for diurnal demand curve development.
- Elevation data were downloaded from the United State Geological Survey, which includes Light Detection and Ranging (LiDAR) data gathered in 2016. These data were used to define junction and hydrant elevations throughout the distribution system.

The pipe model included all-pipes in the District's system, which includes water mains, hydrant leads, and hydrants but excludes service lines.

5.2 Demand Allocation

The consumption rates were spatially distributed using InfoWater Demand Allocator®. This InfoWater module uses GIS technology to assign geocoded consumption data to a designated location within the water distribution system. For each meter record, algorithms in the software were used to distribute the water demands to the closest pipe. The water demands were then allocated proportionally to the nodes at each end of the pipe. For each node within the model, all the contributing water demands were summed to represent the total demand imposed on that particular node.

5.2.1 Base Demand

A meter layer was provided in the GIS geodatabase, which defined the location and account number for each meter in the system. The billing records provided the EQR value for each meter and aided in the development of the water demand factors, as discussed in Chapter 4.0. The GIS meter layer was updated with the account's EQR value and resulting demand flow rate by incorporating the water demand factor for the account type.

A GIS shapefile layer was developed which shows the point of use and EQR value of additional EQRs throughout the system. The water demand factors were used with the EQR values to provide the additional demand flow rates spatially distributed throughout the system. The demand flow rates are an average rate of water use throughout the day; the flow rates fluctuate during an extended period simulation according to the diurnal demand patterns described in the next subsection.

5.2.2 Diurnal Demand Pattern

Water usage for any distribution system is highly variable over the course of a day due to fluctuations in water demand types. In municipal systems, there will typically be a morning and an evening peak in customer water use. However, the fluctuation in water use throughout the day can be impacted by seasonal and climatic conditions (winter vs. summer, precipitation events, etc.), restriction on water use such as irrigation, and demands from large users such as industrial or commercial businesses. Diurnal demand patterns define the hourly water usage on a 24-hour cycle and are used in the hydraulic model for extended period simulations.

Diurnal demand curves were developed for the system using SCADA data provided by the District. For a water distribution system, a flow balance simply indicates that the water that enters the distribution system must be equal to the water that exits the distribution system, plus or minus any changes to the volume contained in water storage facilities.

The following steps were taken to develop the diurnal demand patterns:

- A flow balance was constructed using 5-minute SCADA data containing production source flows, intra-system transfer facility flows or status, and storage tank levels in the distribution system.
- The data were then averaged into hourly increments to define the diurnal pattern over the entire day for the entire distribution system.
- An average diurnal demand pattern analysis was completed for the system for the three seasonal scenarios; winter, shoulder, and summer season.
  - Summer demand patterns were developed using July 2019 data
  - Winter demand patterns were developed using February 2019 data
  - Shoulder demand patterns were developed using late April 2019 and October 2019 data.
- The average diurnal demand pattern was adjusted slightly, so the average of the values equals 1.0, and some corrections were made for incomplete SCADA data.

Figure 5-1 through Figure 5-3 shows the diurnal demand patterns for winter, summer, and shoulder seasons, respectively. The figures show the individual day patterns in the data period, along with the average pattern that is used in the hydraulic model. The pattern values are provided in Table 5-1.

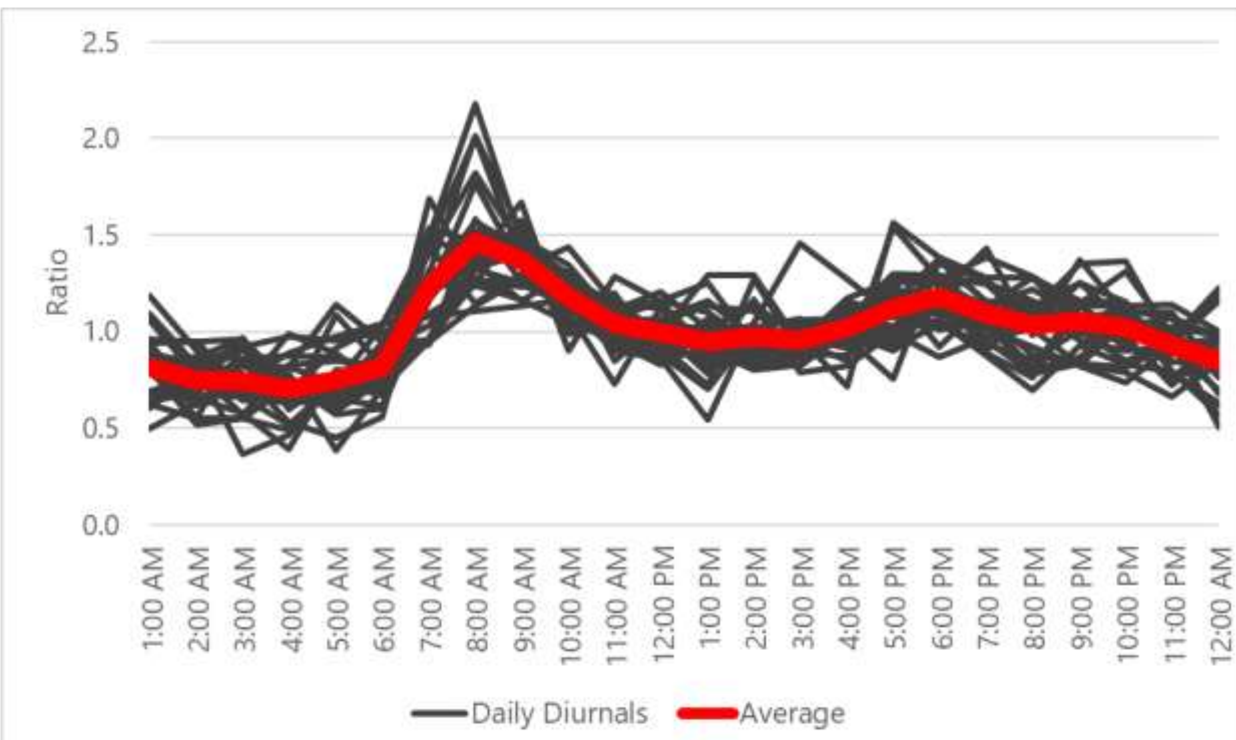


Figure 5-1 – Winter Season Diurnal Demand Pattern

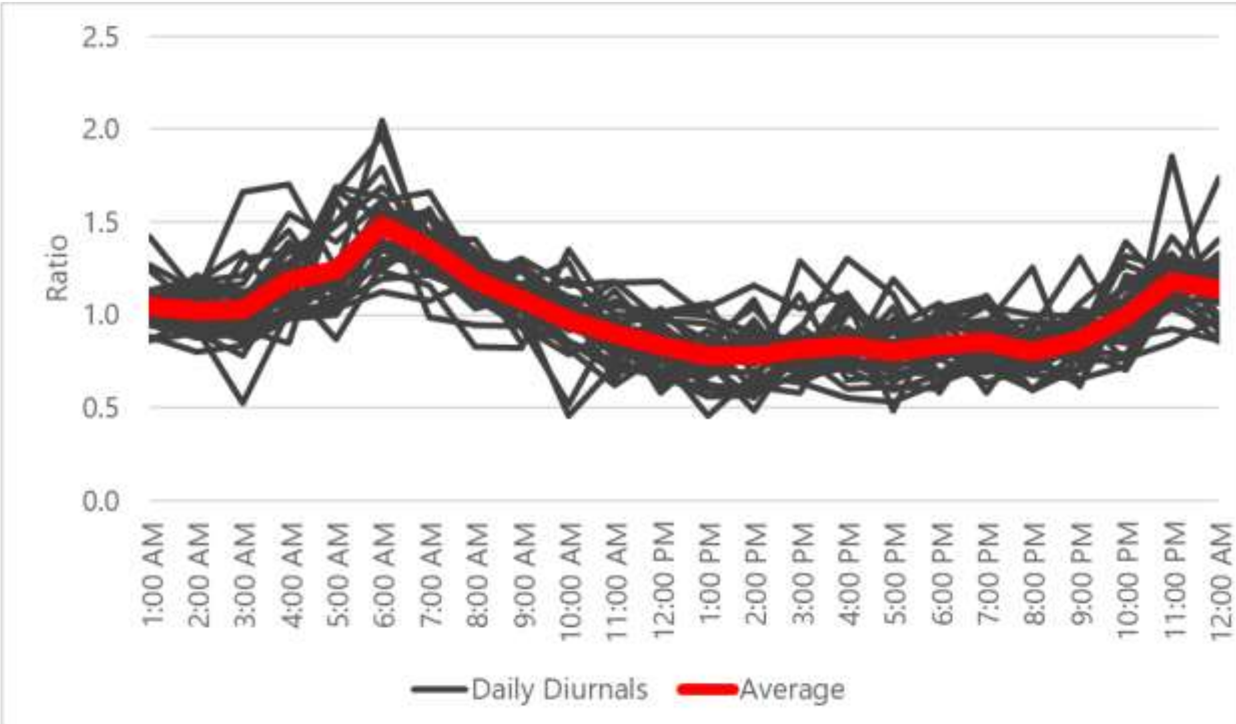


Figure 5-2 – Summer Season Diurnal Demand Pattern

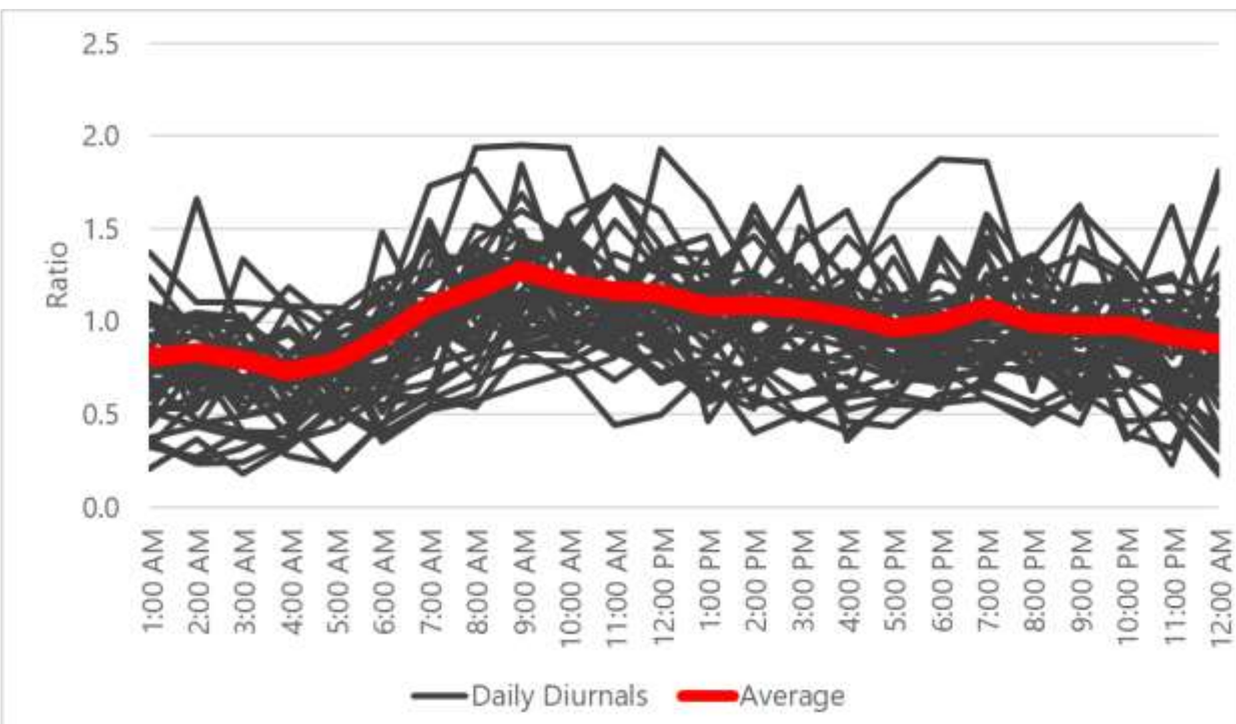


Figure 5-3 – Shoulder Season Diurnal Demand Pattern

Table 5-1 – Diurnal Demand Pattern Values

Time of Day	Summer Season	Shoulder Season	Winter Season
1:00 AM	1.02	0.84	0.75
2:00 AM	1.01	0.80	0.74
3:00 AM	1.18	0.73	0.70
4:00 AM	1.24	0.78	0.75
5:00 AM	1.48	0.92	0.81
6:00 AM	1.35	1.08	1.23
7:00 AM	1.23	1.17	1.46
8:00 AM	1.08	1.28	1.37
9:00 AM	0.97	1.21	1.18
10:00 AM	0.90	1.16	1.04
11:00 AM	0.84	1.15	0.99
12:00 PM	0.78	1.08	0.95
1:00 PM	0.78	1.09	0.97
2:00 PM	0.81	1.07	0.95
3:00 PM	0.83	1.03	1.02
4:00 PM	0.80	0.97	1.12
5:00 PM	0.83	0.99	1.18
6:00 PM	0.86	1.08	1.09
7:00 PM	0.80	1.00	1.04
8:00 PM	0.87	0.98	1.06
9:00 PM	1.00	0.98	1.02
10:00 PM	1.18	0.92	0.93
11:00 PM	1.14	0.89	0.85
12:00 AM	1.05	0.81	0.82

5.3 Field Testing & Data Collection

The objective of creating a model is to generate a tool for predicting the distribution system network's behavior within an acceptable range of accuracy. To generate an accurate model, a robust calibration process must be conducted. Field testing to gather pressure and flow data was completed to develop and calibrate the hydraulic model.

Hydrant flow tests were performed throughout the distribution system to gather flow rate data and the associated pressure drop at a nearby hydrant. The field data is compared with the model simulation data to calibrate the model, as further explained in 5.4.1. Figure 5-4 shows the diffuser, hydrant pressure recorder (HPR), and data collector used for the field tests.

Figure 5-4 – Diffuser, HPR, and Data Collector

5.3.1 Fire Hydrant Flow Tests

Fire hydrant flow tests were performed through the distribution system to gather data to calibrate the hydraulic model. Ten (10) test sites were identified, and flow tests were completed at all sites. Figure 5-5 shows a hydrant during a flow test in the Mountain House area.

Figure 5-5 – Hydrant Flow Testing at Mountain House

Each test consisted of designating a pressure hydrant and two flow hydrants. A hydrant pressure recorder was installed on the pressure hydrant to observe and capture pressure data during the test. A diffuser with a pressure recorder and pitot tube was installed at each flow hydrant to capture the flow rates during the test. The National Fire Protection Association (NFPA) recommends a goal of at least a 25 percent drop in pressure in their NFPA-291 standard, these were deemed excessive for this system where pressure can be over 100 psi. Therefore, a pressure drop goal of 10 psi was used for the testing. The testing followed the sequence below:

1. Install recorder on pressure hydrant and observe an initial “static” pressure.
2. Open one flow hydrant and observe pressure drop at the pressure hydrant until pressure is steady. The pressure when a hydrant is flowing is termed “residual” pressure.

3. If the pressure drop was less than 10 psi, open the other flow hydrant to try to achieve at least a 10 psi drop at the pressure hydrant. If the pressure drop is greater than 10 psi with one hydrant, then the flow hydrants were operated individually.
4. Once a period of steady (residual) pressure was observed at the pressure hydrant with both hydrants flowing, the first hydrant was closed, and the residual pressure was observed.
5. Gather a period of data with only the second hydrant flowing, then close the flow hydrant.
6. Observe pressures at the pressure hydrant and verify the return to initial "static" pressure.

To properly calibrate the model, the following information was recorded at the time of the fire hydrant flow test:

1. Time and date;
2. Hydrant location;
3. Flow rate of hydrant being flowed;
4. Duration of the hydrant flow test;
5. Static and residual pressures at the corresponding test hydrant location. The results of the fire hydrant flow tests are discussed in subsection 5.4.2; and
6. Simultaneous information from the SCADA system on water storage levels, pump operation, and metered flow rates.

Figure 5-6 shows an example of the completed hydrant flow test sheet with the designated pressure hydrant and flow hydrants. Fire flow testing location and individual field testing sheets are included in **Error! Reference source not found.**.

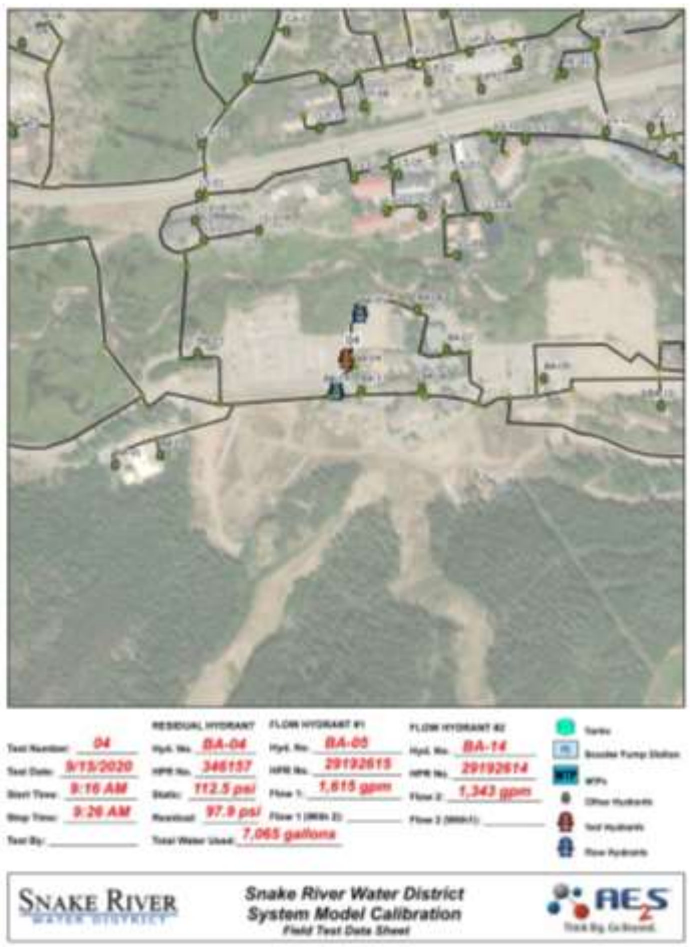


Figure 5-6 – Hydrant Flow Test Sheet

5.4 Model Calibration

The guidelines presented below by the authors of *Water Distribution Modeling*[4] provide some numerical guidelines for calibration accuracy:

> *"The model should accurately predict hydraulic grade line (HGL) to within five to 10 feet at calibration data points during fire flow tests and to the accuracy of the elevation and pressure data during normal demands. It should also reproduce water storage level fluctuations to within three to six feet for EPS runs and match treatment plant/pump station flows to within 10 to 20 percent."*

The above guideline is not definitive but is a good gauge of a model's accuracy. The more accurate the model, the more confidence there can be in both existing and future model simulations.

5.4.1 Calibration Process

A robust effort was made to allocate demands by meter location throughout the water distribution system, as described in subsection 5.2.1. The primary focus of the calibration effort was on pipe roughness coefficients used in the model. The coefficients were adjusted to more closely match field data collected during the fire hydrant flow tests. The calibration process can be summarized in the following steps:

- System operational data such as water storage levels, pump and control valve operation, meter data, and estimated system demands were entered into the model for each individual flow test.
- After the background data was entered, and the fire flow test was simulated, model results were compared with field measurements.
- When model results varied from the observed field measurements, the pipe roughness coefficients were adjusted.
- Adjustments were made to various pipe diameters and pipe materials until the model results matched the field measurements within an acceptable tolerance. District staff was consulted regarding pipe size, condition, and operational settings such as pressure reducing valve settings that may impact the results of the calibration.

---

[4] Walski, T. M., Chase, D. V., & Savic, D. (2001). Water distribution modeling. Waterbury, CT, U.S.A.: Haestad Press.

5.4.2 Calibration Results

Static Pressure Test Calibration Results

Static pressures were recorded at the pressure hydrant before opening the flow hydrants. Initial comparison of static pressures from field test results with simulated hydraulic model results showed that all instances were within 5 feet (≈2.2 psi) of the field measurement. These results confirm that the tank elevations and LiDAR data used to determine hydrant elevations were accurate to hydraulic modeling standards.

Residual Pressure Test Calibration Results

Comparison of residual pressures from field test results with simulated hydraulic model results showed that 5 of the 11 tests were within 5 feet (≈2.2 psi) of the field measurement, and 8 of 11 tests were within 10 feet (≈4.3 psi) of the field test measurement. The calibration results for static and simulated pressures are shown in Table 5-2. The difference in residual results for test No. 10A and 10B are likely due to a partially closed valve or other obstruction in the line; operations staff were unable to find a closed valve or obstruction during this master planning effort. The difference in residual pressure at test No. 8 may be due to the test occurring on new hydrants and inaccurate hydrant lead information resulting in additional headloss in the simulated results. Overall, the hydraulic model calibration results were satisfactory.

Table 5-2 – Calibration Results

Test No.	Pressure Hydrant						Flow Hydrant No. 1	Flow Hydrant No. 2	Material
	Measured Static Pressure (psi)	Measured Residual Pressure (psi)	Simulated Static Pressure (psi)	Simulated Residual Pressure (psi)	Pressure Difference		Flow (gpm)	Flow (gpm)	
					Static Pressure (psi)	Residual Pressure (psi)			
1	79.2	71.9	80.3	68.6	-1.0	3.3	1,278	1,335	DIP
2	51.0	40.5	51.5	41.5	-0.5	-1.0	853	9,33	DIP
3	87.2	73.0	87.6	72.8	-0.4	0.2	1,405	1,295	DIP
4	112.5	97.9	113.4	96.6	-0.8	1.3	1,615	1,343	DIP
5	101.1	85.0	101.7	81.3	-0.7	3.7	1,498	1,108	DIP
6	54.7	40.5	54.0	38.2	0.6	2.3	948	942	DIP
7	101.2	80.0	101.6	81.2	-0.3	-1.3	1,102	-	DIP
8	66.0	52.0	66.1	47.0	-0.1	5.0	1,057	982	DIP
9	90.3	68.1	90.7	67.7	-0.4	0.4	1,403	1,313	DIP
10A	118.2	90.8	118.9	102.6	-0.7	-11.9	1,574	-	DIP
10B	118.2	89.9	118.9	106.4	-0.7	-16.5	-	1,416	DIP

6.0 Design Parameters and Evaluation Criteria

Design parameters identify the features and performance requirements of distribution system infrastructure and provide the standard against which system performance is assessed. The design parameters and criteria presented within this chapter were used to evaluate the performance of the existing water distribution system and to conceptualize system improvements (water mains, storage, flow control, and pumping facilities) necessary to maintain system reliability and accommodate future growth and development of the system.

Design parameters and evaluation criteria are established herein for water system pressures, distribution system storage, pumping facilities, transmission and distribution piping, and fire protection. The criteria were established based on industry standards, Colorado Department of Public Health and Environment (CDPHE) regulations and standards, existing SRWD rules and regulations, and engineering judgment.

6.1 Water System Pressure

6.1.1 Maximum Pressure

Maximum pressure refers to the maximum pressure a customer will experience at their service connection. High pressures within distribution systems can be problematic, resulting in various issues such as increased wear on system components, more frequent leaks and breaks, extreme pressure variations, and issues with operating fire hydrants. Additionally, water main breaks quickly become catastrophic, creating excessive damage to the surrounding area, and creating a safety risk for both the community and operations staff.

SRWD has established rules and regulations for new water main piping within its service area. The allowable materials and minimum pressure ratings of each are included in Appendix C of the regulations last updated on February 11, 2020.

6.1.2 Minimum Pressure

The CDHPE requires a minimum pressure of 20 psi at ground level at all points in the system under all flow conditions per paragraph 8.2.1 of the Design Criteria for Potable Water Systems (CDPHE Design Criteria)[5]. Maintaining a minimum of 20 psi aligns with the NFPA recommended minimum pressure during fire flow events. The CDPHE Design Criteria also states normal working pressure in the distribution system must be at least 35 psi and should be approximately 60 to 80 psi. The

[5] Design Criteria for Potable Water Systems (2017). Denver, CO: Colorado Department of Public Health and Environment.

CDPHE also recognizes that pressure will be low near water storage tanks and expects water system design to mitigate these low pressures as required.

The *Computer Modeling of Water Distribution Systems, AWWA Manual M32*[6], recommends that minimum pressures of 40 to 50 psi be maintained during peak hour demand (PHD) to help ensure that there is adequate pressure to the second story fixtures within a property. The *AWWA Manual M32* also notes that where residential fire sprinkler systems are required by legislation, the minimum acceptable pressure is 50 psi for the fire sprinklers to operate correctly.

6.1.3 Pressure Difference

A third criterion to evaluate system performance is to compare maximum and minimum pressure experienced at any one location in the system. When a new system is being designed, the pressure fluctuation goal should be 20 psi or less[7]. Based on these guidelines, the pressure fluctuation performance criterion established for the system is 20 psi or less.

6.2 Distribution System Storage

Water distribution system storage is provided to ensure the reliability of supply, maintain pressure, equalize pumping and treatment rates, reduce the size of transmission mains, and improve operational flexibility and efficiency. Storage facilities should be sized to provide for the following:

1) Operational/Equalization Storage – Provide storage to meet peak-hour demands and pressure equalization;
2) Fire Protection Storage – Supply storage for fire flow demands; and
3) Emergency Storage – Provide water reserves for contingencies such as system failures, power outages, emergencies, and operational flexibility/reliability (e.g., flooding, earthquake, ability to remove the tank for maintenance without adverse consequence to customers, etc.).

Figure 6-1 depicts storage requirements, inclusive of situations where sufficient capacity exists for winter (low use) adjustment:

[6] Computer modeling of water distribution systems (Manual M32) (2012). Denver, CO: American Water Works Association.

[7] Cesario, L. (1995). Modeling, analysis, and design of water distribution systems. Denver, CO: American Water Works Association.

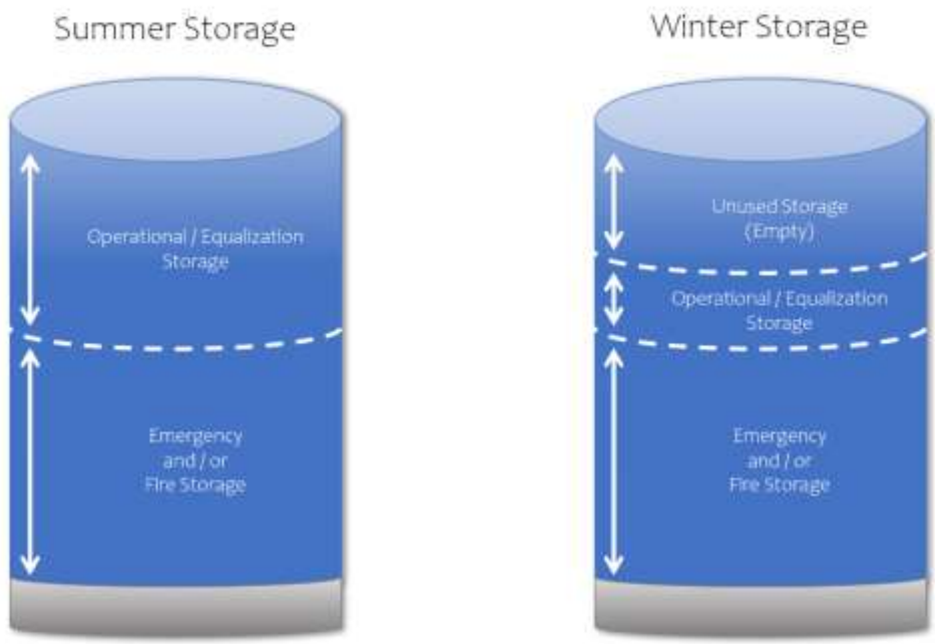


Figure 6-1 – Storage Requirements Overview

Tank sizing criteria is provided in the CDPHE Design Criteria. Paragraph 7.0.1 of the Design Criteria states the following:

> *Storage tanks must have sufficient capacity, as determined from engineering studies, to meet domestic demands, and where fire protection is provided, fire flow demands.*
>
> > *a. The minimum storage capacity (or equivalent capacity) for systems not providing fire protection must be equal to the average daily consumption. This requirement may be reduced when the source and treatment facilities have sufficient capacity with standby power to supplement peak demands of the system.*
> >
> > *b. Excessive storage capacity should be avoided to prevent potential water quality deterioration problems.*
> >
> > *c. Fire flow requirements established by the appropriate state Insurance Services Office should be satisfied where fire protection is provided.*
> >
> > *• Non-community systems are not required to meet items a, b, and c.*

Additionally, the CDPHE Design Criteria provides some direction for mitigating water age issues. Paragraph 7.0.6 of the manual states the following:

> *a. All water storage tanks must have adequate controls to provide tank turnover to maintain finished water quality. Control design must facilitate turnover of water in the finished water*

storage to minimize stagnation and/or stored water age. Demonstration of "adequate" may require a control narrative showing how turnover will occur.

i. Use of the finished water tank overflow as a control mechanism is not considered adequate.

b. Consideration should be given to piping configurations that are reflective of the storage tank geometry and promote mixing of the finished water storage tank contents.

c. The finished water tank design must consider all factors that affect water quality and freezing.

• Tanks less than 11,000 gallons are exempt from Item 7.0.6(a).

The CDPHE Design Criteria does not provide exact storage requirements such as 100% of fire volume or 100% of average day demand. Therefore, the following subsections will discuss design parameters established for the evaluation of the distribution system storage facilities to identify potential deficiencies in the system.

Article 7.3 of the CDPHE Design Criteria also provides requirements for level controls within storage tanks. Paragraph 7.3.3 states the following:

Adequate controls must be provided to maintain levels in the distribution system finished water storage tanks.

Level indicating devices must be provided, accessible at a central location.

a. *Pumps should be controlled from tank levels with the signal transmitted by telemetry equipment when any appreciable head loss occurs in the distribution system between the source and the storage structure.*
b. *Altitude valves or equivalent controls may be required for secondary and subsequent structure of the system.*
c. *Overflow and low-level warnings or alarms must be provided and able to notify water system staff.*

- *Tanks located at non-community water systems that have a total volume of less than 11,000 gallons are not required to meet items a, b, and c above.*

Chapter 7 of the CDPHE Design Criteria also provides requirements for other tank appurtenances such as protection from contamination and trespassing, drains, overflows, silt stops, cathodic protection systems, access, and vents. Chapter 7 also provides guidance on location of facilities and proximity to floodplain and sanitary sewer infrastructure. The scope of this master plan does not cover resolving deficiencies of appurtenances or proximity to floodplains and sewer infrastructure. In general, issues with these appurtenances would be identified and resolved during periodic inspections by District staff or during sanitary surveys by the CDPHE. Evaluation of tank capacity and operations is the main scope of this master plan.

6.2.1 Operational/Equalization Storage

Operational storage enables the source, treatment, and pumping facilities to operate at a predetermined rate, depending on the utility's preference. Additionally, operational storage is generally less expensive than increased capacities of treatment and booster pump stations beyond that required to meet the MDD. Consequently, the source, treatment, and pumping facilities should be sized to serve the water needs up to the MDD of their service area and operational storage provided for meeting peak instantaneous water demands.

Operational storage requirements can vary by system due to differing pump capacities, pipe capacities and water demand characteristics. The volume of water that must be stored during a maximum day demand scenario depends on the individual utility, system configuration and operational procedures. The diurnal demand pattern analysis provided in 5.2.2 shows the need for this operational storage. When demand in a zone exceeds the available supply, operational storage volume is required to maintain system functionality as well as fire/emergency volume. The operational storage value is typically between 20-30% of maximum day demands (MDD). The recommendation and criteria for operational storage is shown below.

- A minimum of 30% of MDD for all zones is recommended for operational and equalization storage.

6.2.2 Fire Storage

Fire storage volume was determined by multiplying the required maximum fire flow rate by the required duration of time. Section 6.5 discusses the development of fire storage volume requirements in greater detail. In addition to fire storage volume requirements, the following criteria are recommended for planning purposes:

- Sufficient storage must exist for the worst-case fire within a pressure zone served by gravity storage.
- Total storage to be provided is based on **one fire** occurring within a 24-hour period.
- Most of the pressure zones can use stored water from the zone above or below them via pressure sustaining valves or pump station, respectively. The ability to use water from the lower zone should be limited to the firm capacity of the pump station. The ability to download water from the higher zone should be limited to the size of the pressure reducing/sustaining valve designated to provide fire flow. The Cla-Val literature provides suggested maximum flow for their model 90-01 (full-port) valves. The fire flow PRVs in the

distribution system are 6-inch or 8-inch in size and their suggested maximum flow rates in manufacturer literature are bulleted below.

- 6-inch PRV – 1,800 gpm
- 8-inch PRV – 3,100 gpm

6.2.3 Emergency Storage

Emergency storage provides water for domestic consumption during unforeseen events such as transmission or distribution main failures, raw water contamination events, extended power outages, failure of raw water transmission facilities, failure of treatment facilities, or a natural disaster.

No industry-standard formula exists for determining the volume of emergency storage required by a utility. Emergency storage requirements are typically policy decisions that are based on an assessment of the perceived vulnerability of the utility's water supply, risk of failures, and the desired degree of system reliability.

If a utility has redundant sources and treatment facilities with auxiliary power, or power supplied from multiple sources, the need for emergency storage may be relatively small. However, enough emergency storage should be available to handle a catastrophic pipe break that cannot be isolated easily. If a utility has a single source without auxiliary power and a relatively unreliable distribution system, a significant volume of emergency storage may be prudent.

Based on a review of the reliability of the water supply the following emergency storage criteria are recommended:

- Emergency storage shall be equal to at least 0.75 days of average summer demands for all zones.

For emergency situations, the District should implement water use restrictions and rationing to reduce system demands until the issues causing the emergency are resolved.

6.2.4 Total Storage

The District's recommended total water storage capacity should be the greater of the following:

1) The sum of operational/equalization storage plus fire flow storage; or

2) The sum of operational/equalization storage plus emergency storage.

The amount of total system storage and system facility capacity required to meet these criteria will change over time as the area continues to grow and water usage increases. Storage parameters should be met in each pressure zone. If a pressure zone is unable to meet these requirements, zones with excess storage capacity may supplement the deficiency, provided the necessary infrastructure is in place (i.e., Booster Stations, Transmission Main, PRVs). Table 6 1 presents the water distribution system storage criteria used for master planning purposes.

Table 6-1 – Storage Criteria Recommendations

Storage Capacity	Criteria
Operational / Equalization Storage	Minimum of 30% of MDD for all zones
Fire Storage	Fire storage to be provided is based on the single worst-case fire occurring within a 24-hr period
Emergency Storage	Equal to at least 0.75 days of average summer demands for all zones.
Total Water Storage Capacity	Total storage should be the greater of: 1. The sum of operational storage plus fire flow; or 2. The sum of operational storage plus emergency storage

6.3 Pumping Facility Capacity

The CDPHE Design Manual provides some criteria for distribution booster pumps in Paragraph 6.4 which states the following:

Distribution booster pumps must be located or controlled so that:

a. *They will not produce negative pressure in their suction lines.*

b. *Pumps installed in the distribution system must maintain inlet pressure as required in Item 8.2.1 under all operation conditions (exclusive of pumps connected to transmission piping).*

c. *Systems designed to operate in an automatic mode have automatic shutoff or a low-pressure controller to maintain at least 20 psi (140 kPA) in the suction line under all operating conditions, unless otherwise acceptable to the department. Pumps taking suction from ground storage tanks and designed to operate in an automatic mode must be equipped with automatic shutoffs or low-pressure controllers as recommended by the pump manufacturer.*

d. *Automatic control devices must have a range between the start and cutoff pressure which will prevent excessive cycling.*

The CDPHE Design Criteria states it was developed based on the Recommended Standards for Water Works, 2012 Edition and modified to meet the needs of the State of Colorado. These standards are referred to as the "10 States Standards" with the most recent edition released in 2018. The latest edition includes the following standards for pumping facilities which are not included in the CDPHE Design Criteria:

6.4.1 Duplicate pumps

Each booster pumping station shall contain not less than two pumps with capacities such that peak demands can be satisfied with the largest pump out of service.

6.4.2 Metering

All booster pump station shall be fitted with a flow rate indicator and totalizer meter.

6.4.3 Inline booster pumps

In addition to the other requirements of this section, inline booster pumps shall be accessible for servicing and repairs.

Ensuring the pressure zone on the suction side of the pumps maintains 20 psi at ground level per Paragraph 8.2.1 during all pump operating conditions is a suitable parameter to evaluate. Maintaining adequate net positive suction head available for the pumps to prevent cavitation is a suitable parameter to evaluate locally at the pumps.

Appropriate pumping facility capacity should be provided to meet the following conditions within the water system:

1) The firm capacity of the pump station should be 25% greater than the MDD for its service area.

Pump station capacity guidelines are based on firm capacity, which is defined as the capacity of the system with the largest pump out of service. In the future, consideration could be given to installing on-site backup power for all pumping facilities considered critical. Less critical facilities could similarly be equipped with a receptacle to allow for a quick connection to a portable generator.

6.4 Transmission and Distribution Mains

The CDPHE Design Criteria manual states in paragraph 8.2.2 that water mains providing fire protection and serving fire hydrants must be a minimum of 6-inch diameter. The entire system is designed to provide fire protection so each main will fall under the criteria. Paragraph 8.2.3 of the manual also states systems designed for fire protection should be in accordance with the appropriate regulatory authority. In most cases, the regulatory authority is the Insurance Service Office (ISO). The design criteria from ISO is provided in section 6.5.

6.4.1 Velocity and Headloss Criteria

Pipelines are sized to meet maximum flow conditions, which generally occur during maximum day plus fire flow or peak hour demand conditions. Pipelines are expected to carry water from sources, including water tanks, reservoirs, and pump stations, to the customer without excessive pressure loss.

Piping within the water distribution system was generalized into two categories for this study:

1) transmission pipelines and
2) distribution pipelines.

Transmission pipelines are large pipes that carry water long distances and branch off to feed the distribution pipelines. Distribution pipelines are generally referred to as pipelines in the street to which fire hydrants and customer service leads are connected.

Establishing a maximum permissible velocity in a pipe must also consider headloss, as velocity is only indirectly the limiting factor in evaluating pipe sizes for a distribution system. Headloss provides a better indication of the capacity of pipelines in that this performance criterion considers the roughness coefficient (C-factor) of the pipeline and the associated velocities within the pipeline. Pipeline velocities also have a direct effect on hydraulic surges and water hammer created in pipelines. As a result, criteria for both maximum permissible velocity and headloss were established for evaluating the performance of the distribution system:

- Transmission Pipelines (10-inch and larger) = less than 3 fps
- Distribution Pipelines (8-inch and smaller) = less than 5 fps
- Transmission Pipelines (10-inch and larger) = less than 2 feet/1,000 feet
- Distribution Pipelines (8-inch and smaller) = less than 5 feet/1,000 feet

Headloss guidelines are used to identify potential problems associated with the hydraulic capacity of water mains to move water from the pumping facilities to water storage. Existing pipelines that exceed these criteria will not be replaced unless there is an existing problem within the distribution system. However, if new pipelines are planned to replace old deteriorating pipelines, then these new pipelines will be sized appropriately to meet these guidelines. As with the velocity guidelines for dedicated transmission pipelines, headloss within dedicated transmission pipelines may exceed the guidelines presented herein but should be evaluated on a case-by-case basis.

6.5 Fire Protection

The decision to provide water for fire protection requires careful consideration of fire flow requirements when sizing pipelines, pumps, and storage tanks because it results in higher capital and operation and maintenance (O&M) costs. However, provisions for fire flows provide a valuable public service by reducing the potential loss of human life and property and improving fire insurance ratings within the community which can reduce insurance costs.

6.5.1 Methods for Calculating Fire Flow Requirements for Structures

This subsection summarizes the four commonly used methods of calculating fire flow requirements for structures in the United States. Later subsections describe the concepts of needed fire flows (NFF), fire flow duration, and discuss the provisions established for evaluating the system.

As described in the *AWWA Manual M31*[8], there are three generally accepted methods for calculating fire flow requirements:

1) Iowa State University (ISU);
2) Illinois Institute of Technology Research Institute (IITRI); and
3) Insurance Services Office (ISO).

Although not identified within the *AWWA Manual M31*, a fourth method of calculating fire flow requirements is the International Fire Code (IFC).

Iowa State University Method

The ISU method is the oldest of the four methods. It addresses the quantity of water required to extinguish a fire and considers the effect of a range of application rates. The equation used to calculate the fire flow under this method is relatively simple, equal to the volume of building space in cubic feet divided by 100. The drawback to this method is the fact that for non-compartmentalized buildings, such as warehouses, the calculated flow would be quite large, as the equation assumes the entire structure is involved in the fire. This method assumes water is supplied in an ideal manner and maximum effectiveness is achieved.

Illinois Institute of Technology Research Institute (IITRI) Method

The IITRI method was developed based on statistics obtained from 134 actual fires of varying magnitude. Water application rates were calculated using the documented length and diameter

[8] Distribution system requirements for fire protection (Manual M31) (2008). Denver, CO: American Water Works Association.

of fire hose and the nozzle pressures. From this data, formulas for fire flows for residential and nonresidential occupancies were developed through a curve fitting analysis. These equations consider the actual area of the fire and, of the three methods described herein, this method generally projects the highest fire flow requirement.

Insurance Services Office (ISO)

The ISO method is the most commonly used of the three methods described in *AWWA Manual M31* and develops or determines the rate of flow considered necessary to control a major fire within a specific structure. This method was derived as a tool for use by the insurance industry in establishing fire insurance rates for individual properties based on the community's fire defenses. The results calculated using this method are generally consistent with those calculated using the ISU method, although slightly higher due in part to the fact that the ISO method accounts for the need to protect the adjacent buildings as well.

The NFF is described as the specific amount of water necessary to control a major fire in a specific building. This value is based on the size of the burning structure, construction materials, combustibility of the contents, proximity of nearby buildings and if fire suppression systems (sprinkler system) are included in the building. The NFF is expressed in units of gpm at a pressure of 20 psi for a range of two to four hours. The minimum NFF for a single building as identified by the ISO is 500 gpm at 20 psi for one hour[9].

According to ISO, fires requiring 3,500 gpm or less are referred to as receiving "Public Fire Suppression," while those requiring greater than 3,500 gpm are classified as receiving "Individual Property Fire Suppression." Therefore, the public classification applies to properties with a needed fire flow of 3,500 gpm or less.

The Fire Suppression Rating Schedule is the manual ISO uses in reviewing the firefighting capabilities of individual communities. The schedule measures the major elements of a community's fire-suppression system and develops a numerical grading called a Public Protection Classification (PPC). ISO assigns a PPC from 1 to 10 and is shown as #/#X or #/#Y. The first number applies to properties within 5 road miles of the responding fire station and within 1,000 feet of a creditable water supply. The second number applies to properties within 5 road miles of the responding fire station but beyond 1,000 feet from a creditable water supply.

Class 1 represents the best protection, and Class 10 indicates no recognized protection. ISO classification ratings are based on the three following areas:

[9] Guide for determination of needed fire flow (2014). Jersey City, NJ. Insurance Services Office, Inc.

- Fire Department - 50 percent of the score looks at the local fire department, including staffing, training, geographic distribution of firehouses and adequacy of the fire equipment.
- Water Supply System - 40 percent of the score considers the community's water supply, including the placement and condition of fire hydrants and the amount of water available to put out fires.
- Fire Alarm and Communication System - 10 percent of the score measures the efficiency of emergency communications, such as the 911 system and the number of emergency dispatchers.

To determine the rate of flow the water mains provide, ISO observes fire-flow tests at representative locations in the community. The ISO Fire Suppression rating affects insurance costs for properties with NFF of 3,500 gpm or less. The private and public protection at properties with larger NFF should be individually evaluated.

International Fire Code

Table 6-2 – Fire Flow Requirements for Buildings Table from 2018 IFC

FIRE-FLOW CALCULATION AREA (square feet)					FIRE-FLOW (gallons per minute)[b]	FLOW DURATION (hours)
Type IA and IB[a]	Type IIA and IIIA[a]	Type IV and V-A[a]	Type IIB and IIIB[a]	Type V-B[a]		
0-22,700	0-12,700	0-8,200	0-5,900	0-3,600	1,500	2
22,701-30,200	12,701-17,000	8,201-10,900	5,901-7,900	3,601-4,800	1,750	
30,201-38,700	17,001-21,800	10,901-12,900	7,901-9,800	4,801-6,200	2,000	
38,701-48,300	21,801-24,200	12,901-17,400	9,801-12,600	6,201-7,700	2,250	
48,301-59,000	24,201-33,200	17,401-21,300	12,601-15,400	7,701-9,400	2,500	
59,001-70,900	33,201-39,700	21,301-25,500	15,401-18,400	9,401-11,300	2,750	
70,901-83,700	39,701-47,100	25,501-30,100	18,401-21,800	11,301-13,400	3,000	3
83,701-97,700	47,101-54,900	30,101-35,200	21,801-25,900	13,401-15,600	3,250	
97,701-112,700	54,901-63,400	35,201-40,600	25,901-29,300	15,601-18,000	3,500	
112,701-128,700	63,401-72,400	40,601-46,400	29,301-33,500	18,001-20,600	3,750	
128,701-145,900	72,401-82,100	46,401-52,500	33,501-37,900	20,601-23,300	4,000	4
145,901-164,200	82,101-92,400	52,501-59,100	37,901-42,700	23,301-26,300	4,250	
164,201-183,400	92,401-103,100	59,101-66,000	42,701-47,700	26,301-29,300	4,500	
183,401-203,700	103,101-114,600	66,001-73,300	47,701-53,000	29,301-32,600	4,750	
203,701-225,200	114,601-126,700	73,301-81,100	53,001-58,600	32,601-36,000	5,000	
225,201-247,700	126,701-139,400	81,101-89,200	58,601-65,400	36,001-39,600	5,250	
247,701-271,200	139,401-152,600	89,201-97,700	65,401-70,600	39,601-43,400	5,500	
271,201-295,900	152,601-166,500	97,701-106,500	70,601-77,000	43,401-47,400	5,750	
295,901-Greater	166,501-Greater	106,501-115,800	77,001-83,700	47,401-51,500	6,000	
—	—	115,801-125,500	83,701-90,600	51,501-55,700	6,250	
—	—	125,501-135,500	90,601-97,900	55,701-60,200	6,500	
—	—	135,501-145,800	97,901-106,800	60,201-64,800	6,750	
—	—	145,801-156,700	106,801-113,200	64,801-69,600	7,000	
—	—	156,701-167,900	113,201-121,300	69,601-74,600	7,250	
—	—	167,901-179,400	121,301-129,600	74,601-79,800	7,500	
—	—	179,401-191,400	129,601-138,300	79,801-85,100	7,750	
—	—	191,401-Greater	138,301-Greater	85,101-Greater	8,000	

For SI: 1 square foot = 0.0929 m^2, 1 gallon per minute = 3.785 L/m, 1 pound per square inch = 6.895 kPa.
a. Types of construction are based on the *International Building Code*.
b. Measured at 20 psi residual pressure.

The IFC is a model code regulating minimum fire safety requirements for new and existing buildings, facilities, and storage areas. As stated in the IFC, the minimum fire flow required for one- and two-family dwellings that do not exceed 3,600 square feet and do not have an automatic sprinkler system is 1,000 gpm. For one- and two-family dwellings exceeding 3,600 square feet, and for all buildings other than one- and two-family dwellings, the minimum fire flow, and flow durations, are presented in Table 6-2. The minimum fire flow for these types of structures ranges from 1,500 gpm to 8,000 gpm, over durations from two to four hours. The IFC does provide criteria that allows the needed fire flow to be reduced by up to 75% if fire sprinklers are installed in the building.

6.5.2 Fire Flow Requirements

The District's Rules and Regulations provide some guidelines for fire flow rate availability in their pressure zones and that rates may vary at specific sites. These general guidelines are provided below:

- Base 1: 2,500 gpm
- Base 2, 3 and 4: 3,000 gpm

The most recent ISO reports for the fire protection district provided some indication of the needed fire flows above 1,500 gpm at various structures throughout the service area. Both the District's Rules and Regulations along with the flow requirements from the ISO report will be used in the fire flow evaluation. The maximum flow rates shown in the ISO report for each pressure zone is shown below:

- Base 1: 1,500 gpm
- Base 2: 4,000 gpm
- Base 3-4: 4,000 gpm

The ISO report indicated a requirement of 8,000 gpm in Base 2 which was reviewed with the local fire marshal. The marshal indicated that the building this flow rate is attributed to does have a fire sprinkler system and therefore the flow rate can be reduced by 50% to 4,000 gpm.

6.5.3 Fire Flow Availability

The available fire flow established for this master plan shall be at least 1,500 gpm and shall be greater than needed fire flow at structures requiring flow greater than 1,500 gpm. Additionally, the system shall have the capacity to provide the fire flow volume for the duration of the fire flow event. The fire flow rate of the building defines the duration of the event as shown in Table 6-2 and ultimately the volume of water required. These criteria will be evaluated in Chapter 7.0.

6.6 Design Parameters and Evaluation Criteria Summary

Table 6-3 – Summary of Design and Evaluation Criteria

Component	Criteria	
Distribution System Pressures	Pressure (psi)	
Maximum Pressure[a]	150	
Minimum Pressure during PHD	50 psi per AWWA Manual M32 35 psi per CDPHE	
Minimum Pressure during a Fire Flow	20	
Maximum Pressure Difference	20	
Storage Capacity [b]		
Operational Storage per Zone	Minimum of 25% of MDD for all zones	
Fire Storage	Fire storage to be provided is based on a single fire occurring within a 24-hr period	
Emergency Storage	Equal to at least 0.75 days of average summer demands for zones fed only by BPS	
Total System Storage is the greater of:	1. The sum of operational storage plus fire storage; or	
	2. The sum of operational storage plus emergency storage	
Pump Station Capacity	BPS Firm capacity is greater than 125% of MDD of all zones supplied by BPS	
Water Transmission and Distribution Pipelines	Velocity (fps)	Headloss [c] (feet/1000 feet)
Transmission pipelines (12-inch and larger)	Less than 3	Less than 2
Distribution pipelines (10-inch and smaller)	Less than 5	Less than 5
Fire Flow Requirements [d]	Flow Rate (gpm)	Duration (hrs)
Minimum Fire Flow	1,500	2

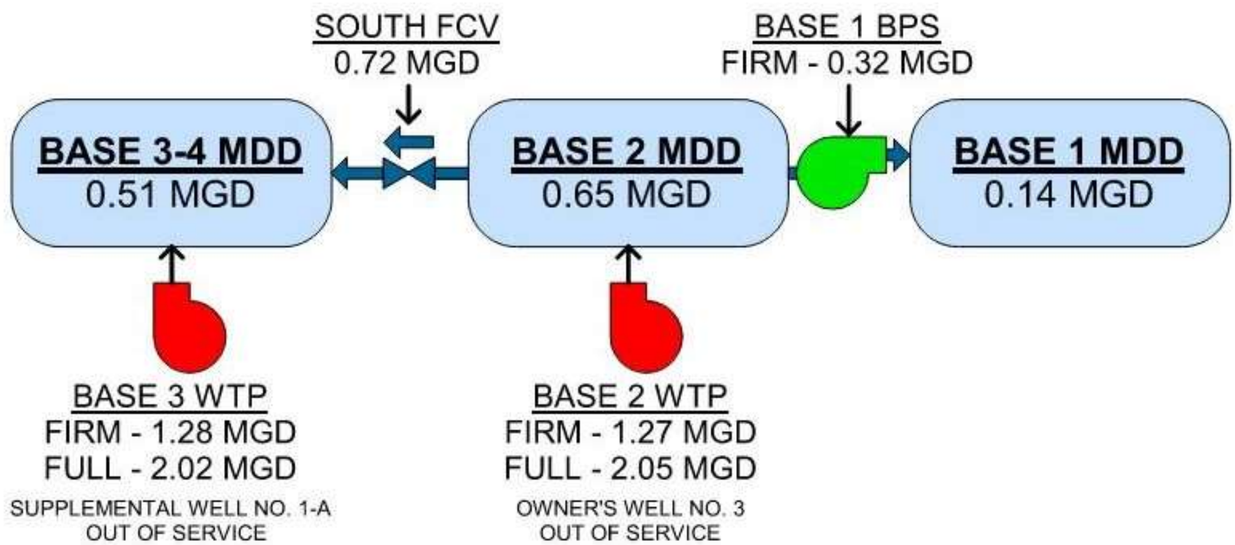
a – Maximum pressure recommendation is attributed to new systems.
b- Storage parameters should be met in each pressure zone. If a pressure zone is unable to meet these requirements, zones with excess storage capacity can supplement the deficiency provided the necessary infrastructure is in place (i.e., Booster Stations, PRVs).
c - With existing pipelines, headloss may exceed guidelines presented, but should be evaluated on a case by case basis. New pipelines should be sized appropriately to meet these guidelines.
d –The minimum fire flow requirement was selected, higher requirements within an analysis zone were provided in data received from ISO.

7.0 Existing System Evaluation

This chapter will larger compare hydraulic model results with the performance criteria established in Chapter 6.0. The evaluation will look into production, storage, and pumping capacities, system pressures and demands, and fire flow analysis.

7.1 Existing System Demands and Production

Analysis of the ability to distribute water throughout the system will be provided in this section. In summary, the firm capacity of production sources and pumping stations will be compared to the zones served by the facility. Figure 7-1 shows the production and pumping firm capacities compared to the current maximum day demand for each zone.



**Water can also be transferred through the North PRV vault; this is not shown as this transfer would be adjusted at the valve. The South Flow Control Vault (FCV) has remote operation capabilities.*

Figure 7-1 – Existing System Production and Pumping Capacity versus Existing MDD

- The firm capacity of BPS 1 is well above the current maximum day demand for Base 1 pressure zone.
- The firm capacity of both WTP is very near the overall system maximum day demand of 1.3 MGD.
- Base 1 and 2 are reliant on the Base 2 WTP as there is no way to bring water produced by the Base 3 WTP to these zones.

7.2 Water System Pressure

Maximum and minimum pressures were evaluated during peak hour demand. The maximum pressures expected in all pressure zones is around 115 psi which is well below the maximum pressure criteria established in subsection 6.1.1. There are three areas where minimum pressures do not meet the CDPHE criteria of 35 psi minimum. These areas of low pressure are well known to the District and simply driven by the elevation of the nearby residences compared to the storage tank supplying them. These areas are:

- near North Fork Reserve
- near Saints John Condos
- along Tennis Club Road near the Tennis Townhome Condos

The maximum pressure difference in the system was 3 psi which is well below the criteria of 20 psi maximum. The minimum and maximum pressures observed during maximum day demands are shown in Figure 7-2 and Figure 7-3, respectively. Pressure differences throughout the system are not shown as the differences were all below 3 psi.

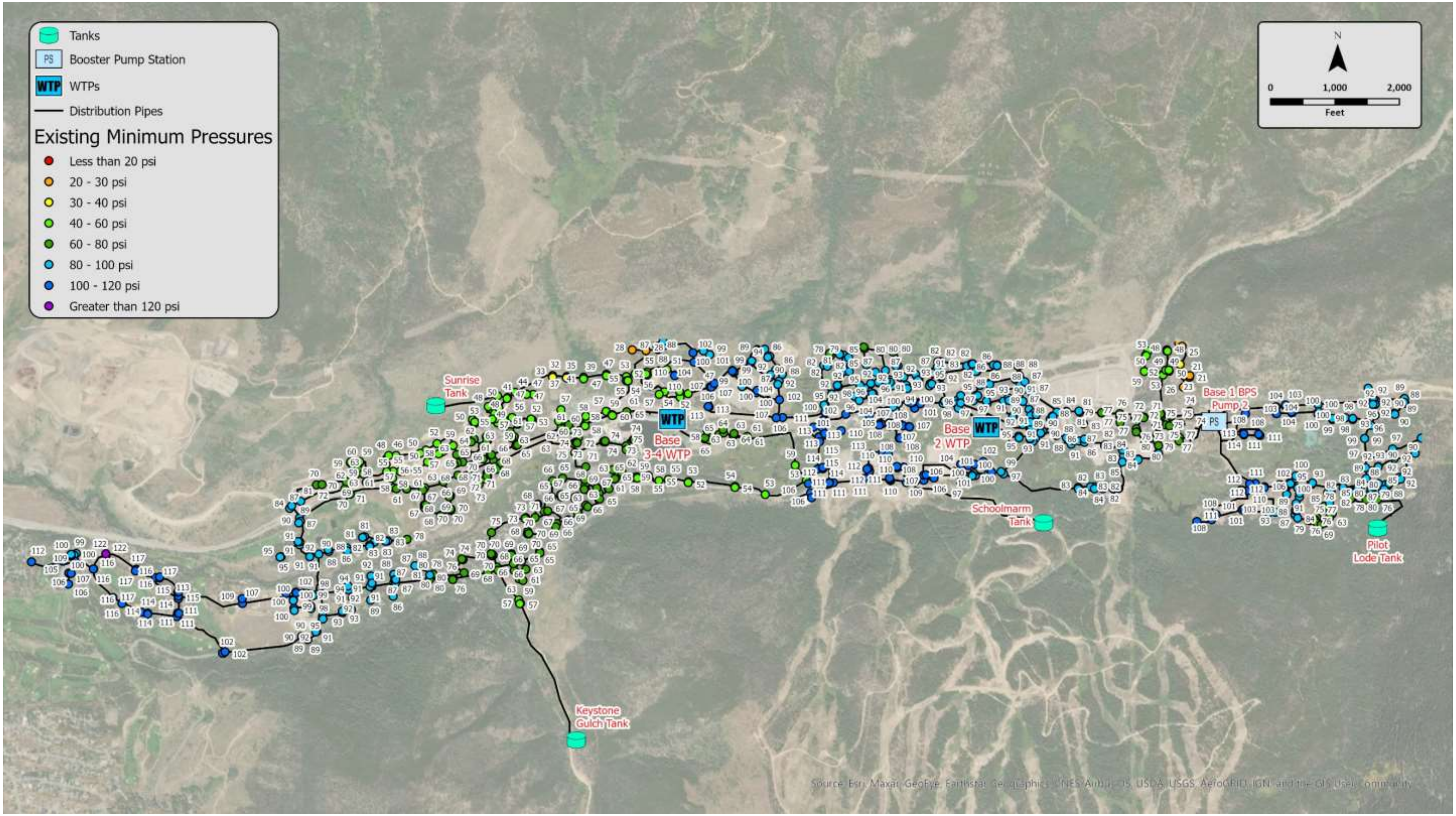


Figure 7-2 – Existing System Minimum Pressures

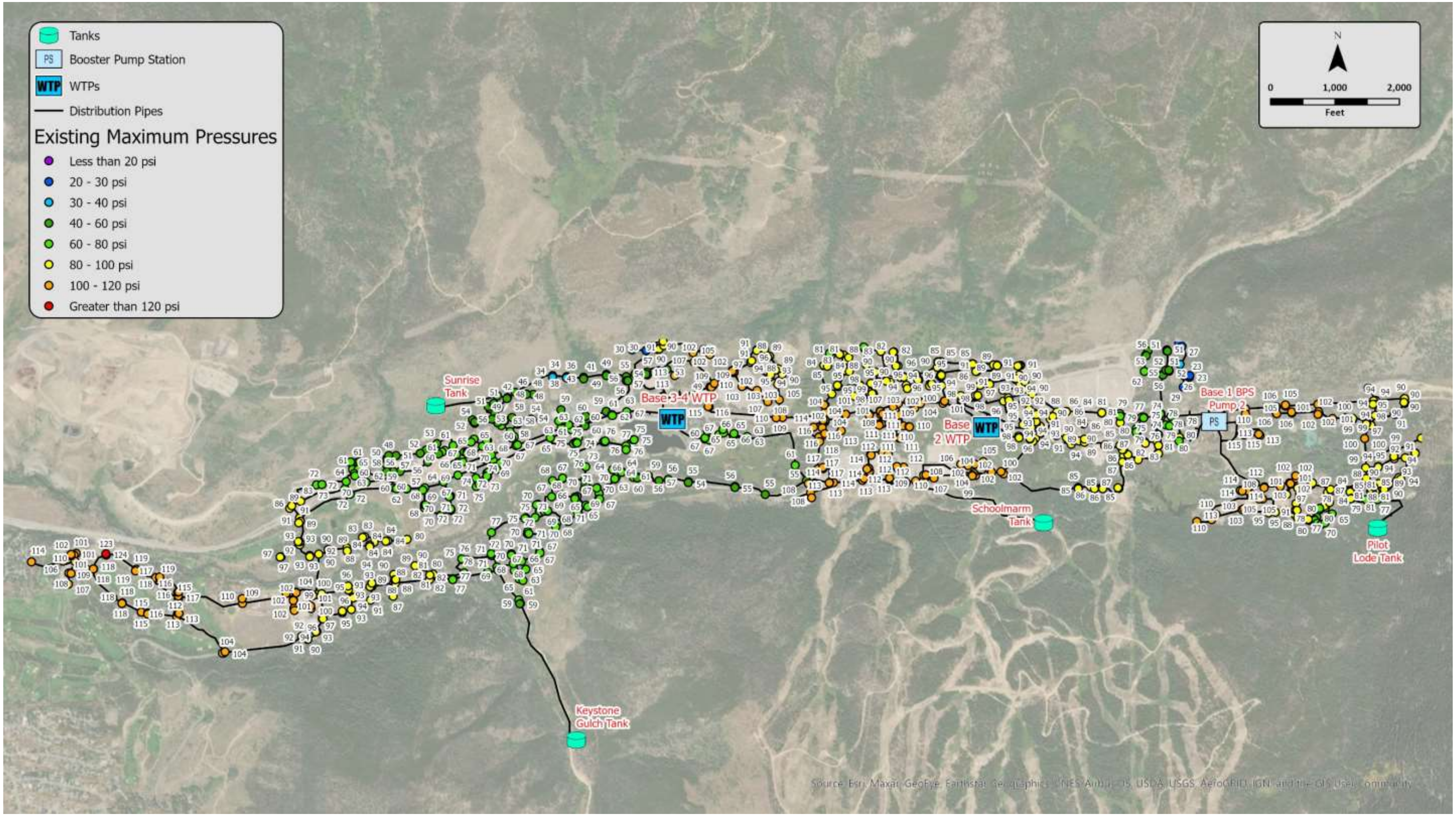


Figure 7-3 – Existing System Maximum Pressures

7.3 Distribution System Storage

Tank capacity, operations and water age are reviewed in this section.

7.3.1 Capacity

Tank capacity is evaluated based upon the storage requirements reviewed in section 6.2. Recall that the requirement is the greater of the fire plus emergency storage or operational plus emergency storage. In all instances, the fire storage requirements are greater than the operational storage requirements, so fire protection is the driving criteria. Table 7-1 provides the storage requirements, existing tank capacities and storage surplus or deficit for each zone.

Table 7-1 – Existing Storage Capacity Analysis

Pressure Zone	Tank Capacity (Gallons)	Storage Requirement (gallons)				Storage Surplus / (Deficit)	% of Avg Summer Demand in Emergency	% of MDD in Operational / Equalization
		Fire	Emergency	Operational / Equalization	Total Required			
Base 1	750,000	180,000	57,300	41,500	**221,500**	528,500	75%	30%
Base 2	1,000,000	960,000	308,900	196,500	**1,156,500**	(156,500)	75%	30%
Base 3-4	2,250,000	960,000	231,800	151,300	**1,111,300**	1,138,700	75%	30%

If the District's fire flow rate of 2,500 gpm is used for Base 1 from the District's Rules and Regulations, then the storage requirement increases to 341,500 gallons which drops the excess capacity to 408,500 gallons; still very adequate for the pressure zone. Cumulatively, the District has adequate storage for all demands. There is an apparent storage deficit in Base 2 off 156,500 gallons, however, the system does have the ability to download water from Base 1 to Base 2 if needed to supplement fire demand events. This analysis is based off full storage tanks capacity which rarely occurs as tank levels will fluctuate with demand and pump operations.

7.3.2 Operations

A general goal in tank operations is to achieve a 90% mixed tank during refill with requirements based on the low tank water level. Achieving tank mixing needs to be balanced with maintaining minimum tank volumes for fire protection. Tank mixing analysis is based on the tank dimensions and start and stop fill levels. Based on SCADA data and discussion with operation staff, the tanks were historically operated a 2-foot interval between high and low water level. The required and actual volumetric exchange rates with these approximate 2-foot intervals is shown in Table 7-2. Recommended setpoint changes are also shown which were based off minimum tank levels from the previous capacity review section.

Table 7-2 – Historical and Recommended Setpoints

Operations	Tank	Start Fill Level (ft)	Stop Fill Level (ft)	Required Volume Exchange	Actual Volumetric Exchange	Required Exchange Achieved
With Historical Setpoints	Pilot Lode Tank	26	28	20%	8%	38%
	Schoolmarm Tank	15	17	18%	14%	75%
	Keystone Gulch Tank	20	22	20%	10%	52%
	Sunrise Tank	20	22	16%	10%	64%
With Recommended Setpoints	Pilot Lode Tank	20	28	22%	41%	184%
	Schoolmarm Tank	14	17	19%	22%	119%
	Keystone Gulch Tank	17	22	21%	30%	144%
	Sunrise Tank	17	22	17%	30%	179%

The recommended start fill level setpoints is based on providing 360,000 gallons for use in a Base 2 fire flow event through the Base 1 BPS PRV. This volume allows Base 1 to supplement up to 1,500 gpm for the four-hour event thus using 600,000 gallons stored in the Schoolmarm Tank.

7.3.3 Water Age

Water age can be a cause for concern as the potential for increased levels of disinfection by-products (DBP) and nitrification can occur with increase water age. The formation of DBP is highly dependent on water quality, chlorine concentrations and water temperatures; formation will increase at higher temperatures, higher chlorine concentrations and with increased precursors for development such as organic carbons. The District has a good quality water source with low DBP precursors and water temperatures in the system remain relatively low. The results of annual DBP testing have been very low compared to the regulatory limits which reflects the high-quality source and treated water.

Water age analysis results using the shoulder demand scenario from the hydraulic model and using the historical setpoints in Table 7-2 are shown in Figure 7-4. The highest water age areas occur near the Pilot Lode tank, Keystone Gulch Tank, and very west end of the Base 3 Pressure Zone. The location of large demands can heavily influence the water age results. Water age results using the recommended setpoints from Table 7-2 and during shoulder demand scenario are shown in Figure 7-5. In general, water age is reduced by about 25% in the previously mentioned high water age areas with the use of the recommended setpoints.

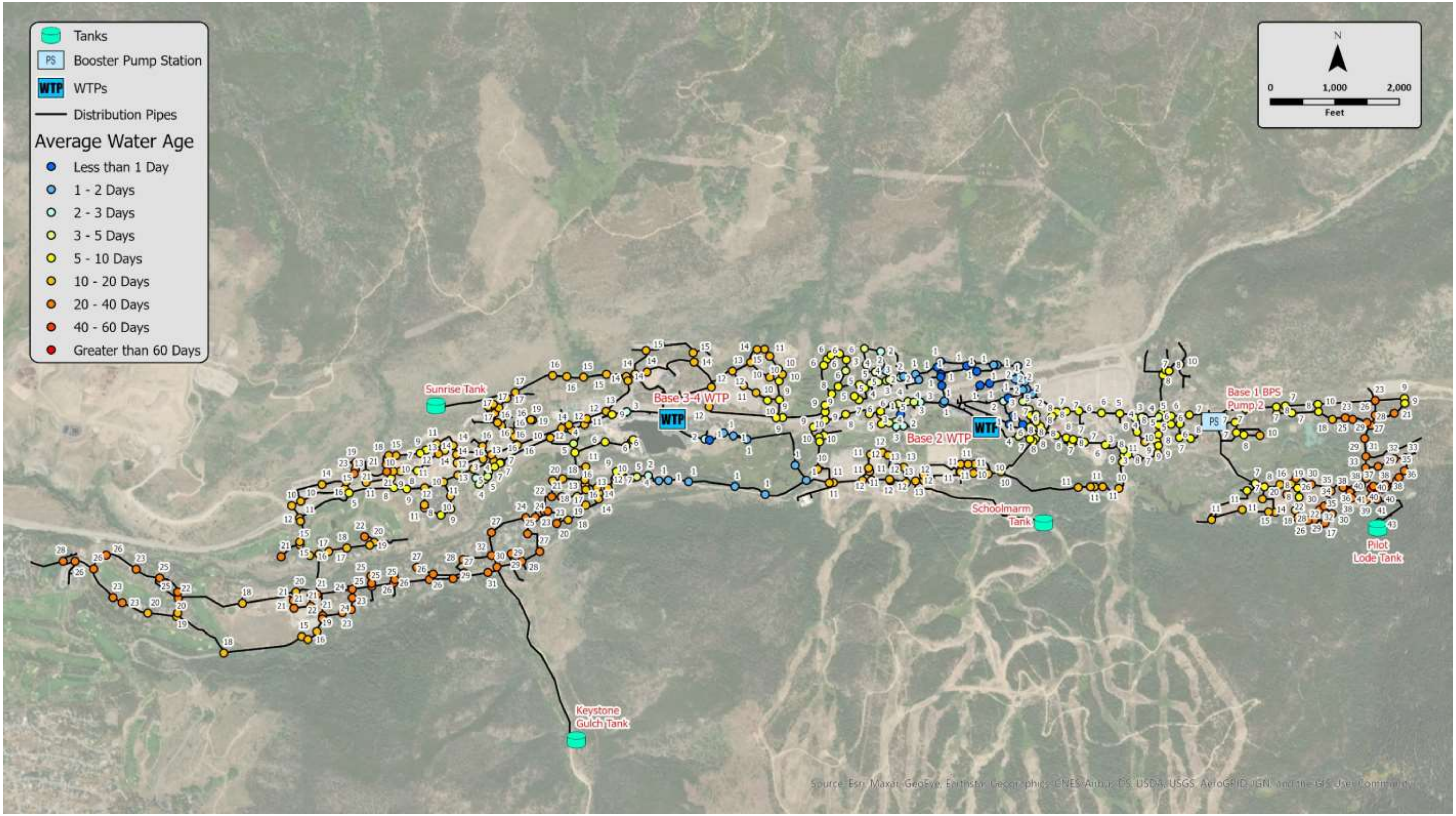


Figure 7-4 – Water Age with Historical Setpoints

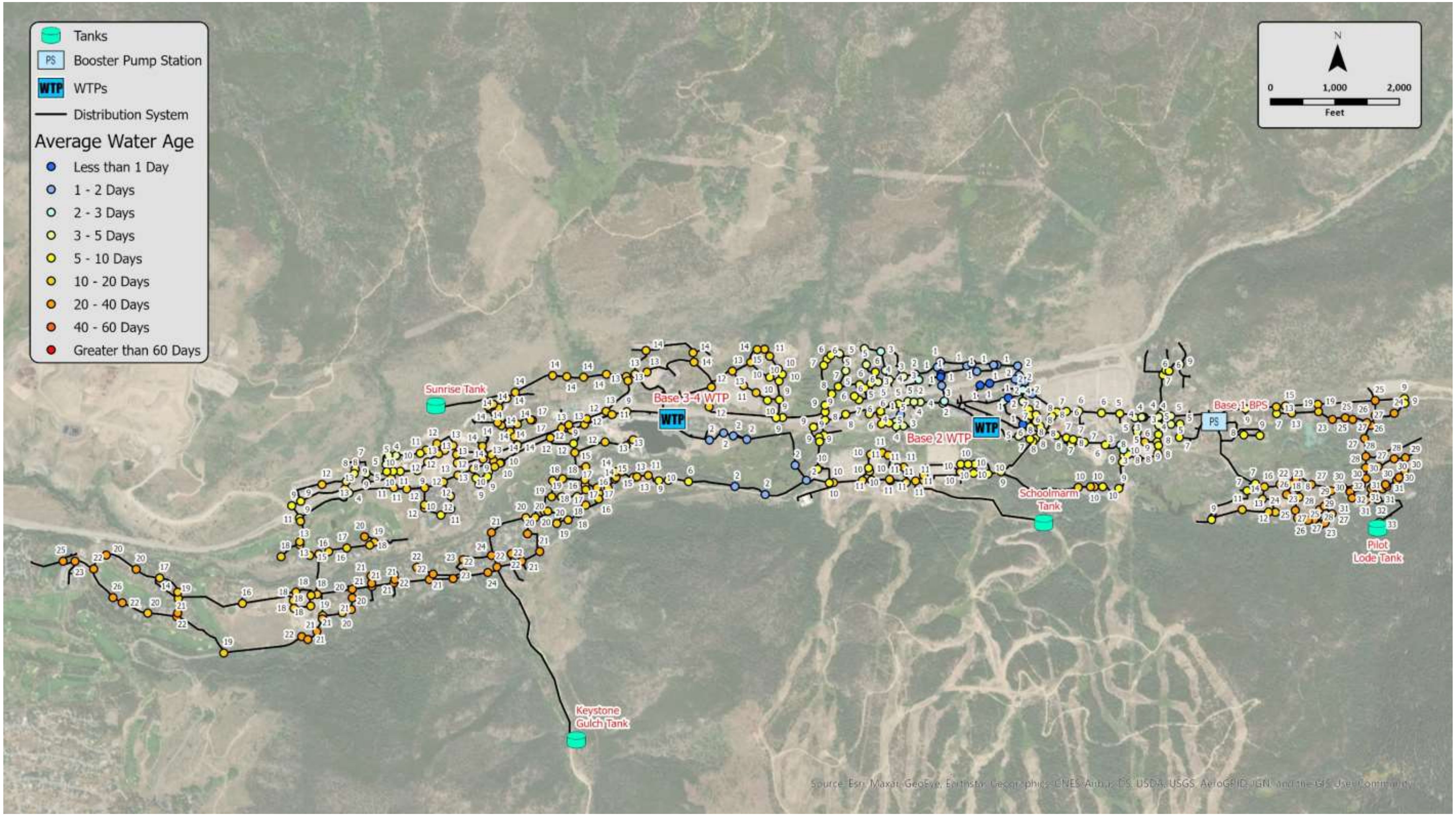


Figure 7-5 – Water Age with Recommended Setpoints

7.4 Distribution System Pumping

7.4.1 Capacity

The Base 1 BPS has a full capacity of 525 gpm and a firm capacity of 225 gpm. The current maximum day demand for the Base 1 pressure zone is 138,500 gallons which equates to 96 gallons per minute of flow over a 24 hours period. Therefore, BPS 1 has a firm capacity about 2.34 times greater than the current maximum day demands and sufficiently sized for current demands.

7.4.2 Pumping Efficiency

District operating staff asked about why there is a pressure reducing valve located on the suction side of the pumps as shown in Figure 7-6. The pump curves, suction and discharge pressures were reviewed which shows that this PRV reduces suction pressure in order for the pumps to operate on their best efficiency point. Reducing the pressure is wasting energy in the water and therefore requiring large motors than necessary.

Figure 7-6 – Base 1 BPS Photo

The pressure reducing valve lowers pressure from 70 psi down to 40 psi on the suction side of the pumps while the discharge pressure is near 112 psi. This results in the pumps operate near 170 feet of total dynamic head which is very near their best efficiency point. If the suction pressure remained near 70 psi the pumps would operate to the right of their curve which will produce more flow, however, the pumps will become less efficient. The operating point without the PRV would be near 100 ft TDH. Additional concerns include cavitation in the impeller due to increased net positive suction pressure requirements. The inefficiency of this setup was analyzed to quantify the cost of the reducing suction pressure then immediately adding energy via pumping.

Table 7-3 – BPS Operating Cost

	HP Required	kW Required	Energy Consumption Annual Cost
With PRV	11.05	8.23	$ 1,170.89
Without PRV	6.45	4.80	$ 683.02
	Electrical Cost Difference		$ 487.87

Table 7-3 provides a comparison of annual energy consumption costs and energy demand costs with and without the PRV in operation. The results in the table assume the following:

- 1,673 annual operating hours at 200 gpm to meet average day demand of 55,000 gallons
- Pump head requirement with the PRV is 166 ft TDH and without is 97 ft TDH
- Pump efficiency is 76%
- Consumption charge is $0.085 /kWh based on Xcel rate schedule C (Commercial)

Based on the assumptions above, the use of the PRV to keep the pumps operating at their best efficiency point increases operating cost by approximately $500 annually.

7.5 Transmission and Distribution Main Capacity

The section will review the transmission and distribution system capacity based on headloss and velocity as defined in subsection 6.4.1.

7.5.1 Velocity and Headloss

The velocity and headloss analysis used the maximum day demand period as this scenario requires the most production and pumping capacity on the system. Figure 7-7 shows the headloss in the system which is largely under 1 foot per 1,000 feet of pipe which is well below the performance criteria. The headloss results also show that velocities in the system are very low so a figure of velocity results is not shown for the existing system.

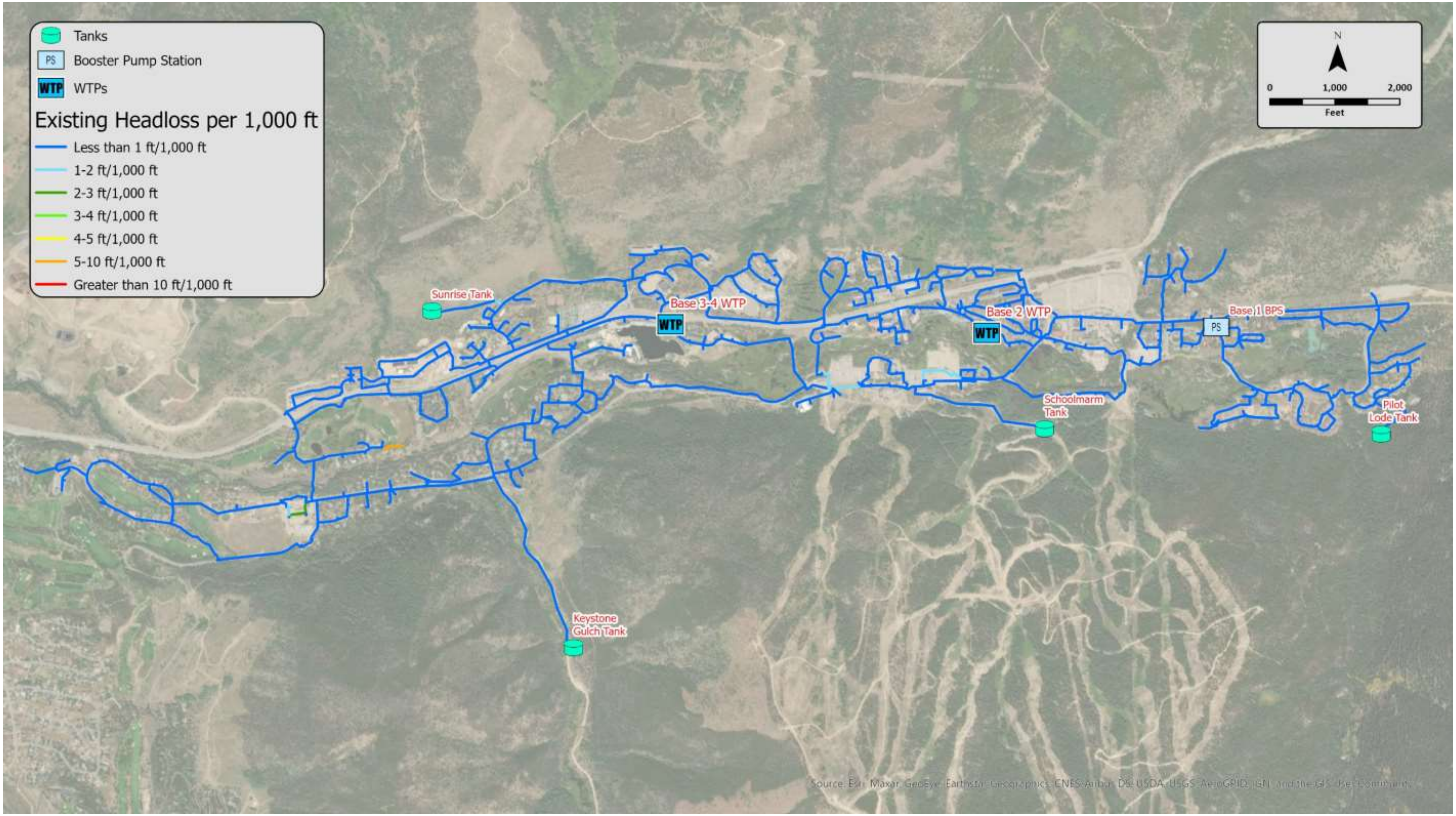


Figure 7-7 – Existing System Headloss Results

7.5.2 Break and Leak History

The District provide GIS data detailing the location, date, and pertinent notes about watermain breaks and leaks throughout their system. AWWA provides utility benchmarking data which is developed from survey of water providers throughout North America. One metric they evaluate is breaks and leaks per 100 miles of pipe. A leak is defined as a continuous discharge or water while a break is an abrupt disruption to service. Additionally, issues on service lines in the system are not counted in this metric, only issues on the system-owned infrastructure are counted. Figure 7-8 shows the yearly value of breaks per 100 miles based on the available data along with the median and 75th percentile value for this benchmark. On average, the District is within the top 75th percentile for this metric.

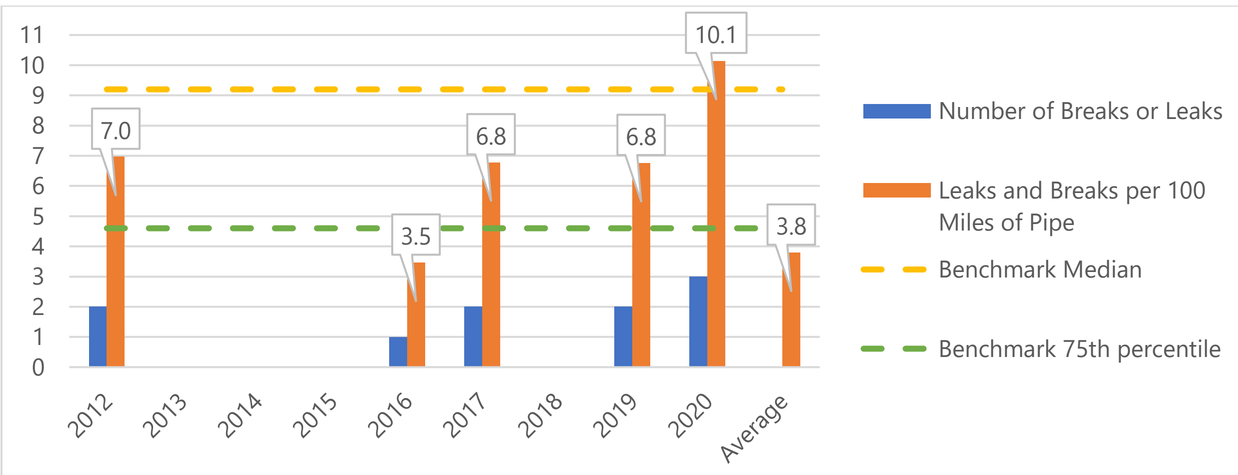


Figure 7-8 – Leak and Break History Benchmark

7.6 Fire Flow Analysis

The available fire flow (AFF) rates at hydrants throughout the system were simulated using the hydraulic model. The model predicts the flow rates at each hydrant flowing individually while maintaining a minimum of 20 psi at the hydrant per IBC requirements. Figure 7-9 shows the "hydrant design flow" rates at the hydrants throughout the system. Hydrant design flow provides conservative fire flow rate values compared to the IBC requirement as 20 psi is maintained through the entire system rather than 20 psi at the hydrant. This method ensures all customers maintain pressure during the fire flow event. The Base 1 PRV is set at 40 psi in this instance. There are a few instances where AFF is less than 1,500 gpm. These are at high elevation of North Fork area, in Loveland Pass Village on Razor Drive and near Saints John Condos. The deficiency in North Fork area is elevation driven while Razor Drive has 4-inch diameter piping in a branch system causing the low AFF. The Saints John Condos AFF deficiency is also elevation driven, however, there are nearby hydrants fed by Base 3 pressure zone that provide adequate AFF.

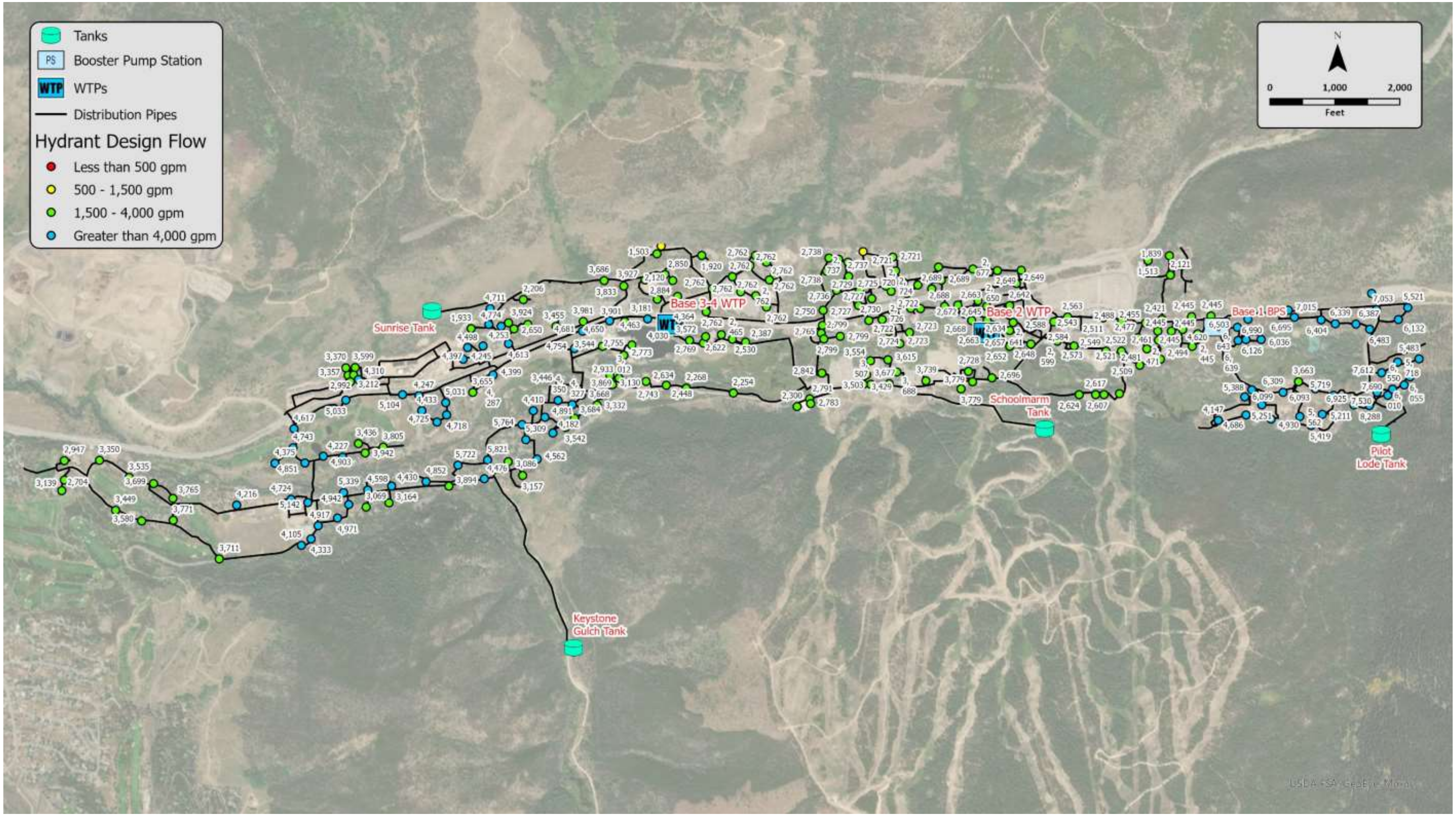


Figure 7-9 – Existing System Design Fire Flow

The operation of the PRV in Base 1 BPS was reviewed in the hydraulic model. A 4,000 gpm demand was placed on Hydrant RR-08 in the River Run Village to simulate a fire flow event. Figure 7-10 shows the flow through the Base 1 BPS PRV using this 4,000 gpm demand at various PRV opening pressure setpoints. Based on the review of tank capacity in subsection 7.3.1, it is recommended to adjust the PRV opening setpoint near 54 psi to help supplement fire flow rates and volumes in the Base 2 Pressure Zone.

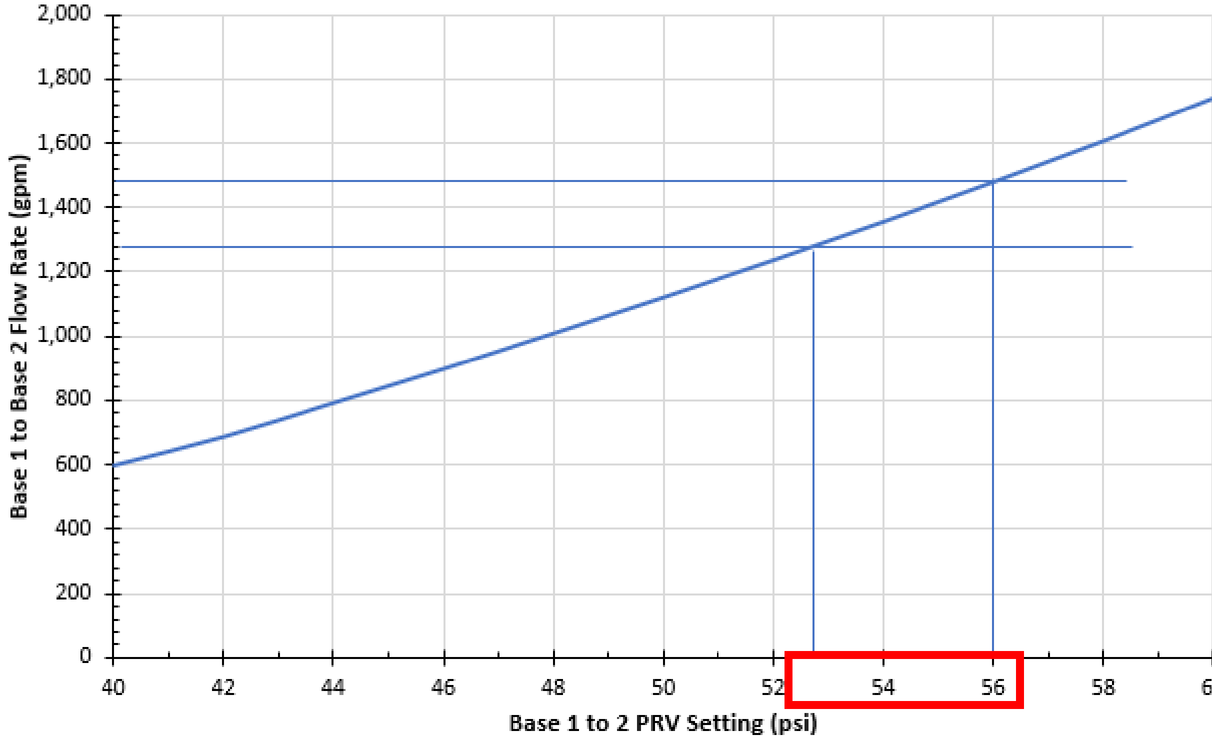


Figure 7-10 – Base 1 BPS PRV Flow Curve

7.7 Summary of Existing System Evaluation

Overall, the performance of the Districts system meets the evaluation criteria in a majority of the system.

- The capacity of treatment and pumping facilities are adequate for current maximum day demands.
- There are a few local instances of low pressure due to the elevation profile of the system; these areas are North Fork and near Saints John Condos.
- The storage capacity for the Base 2 Pressure Zone is below the storage requirement, which is larger affected by fire storage volume, however, the Pilot Lode tank can supplement volume to the fire flow event.
- Changes to tank fill setpoints can help reduce water age in the system while maintaining adequate storage for fire flow. The changes can also help increase turnover and promote better mixing in the tanks to help with water age and water quality issues.
- A pumping inefficiency could be resolved at Base 1 BPS when pumps are replaced at the end of their useful life.
- There are no headloss or velocity concerns in the system.
- Simulated AFF results appear to be adequate to meet the fire flow requirements from the latest ISO evaluation.

8.0 Water System Risk Assessment

This chapter reviews the framework for condition and risk assessment for assets in the District system. Individual criteria and criteria weight will be review for each asset type that ran through assessment process using InfoAsset Planner® software.

8.1 Condition and Risk Assessment Framework

8.1.1 Develop Criteria

The first step of the risk assessment was to select the criteria to be included in the assessment. The developed criteria were grouped into two categories – Consequence of Failure (COF) and Likelihood of Failure (LOF). These two categories form the basis of risk and are important to fully understand how asset risk levels are determined.

Figure 8-1 shows a simplified equation for how risk was calculated for each asset type. It's simplified because it does not show all the specific COF and LOF criteria included when calculating risk. The 'union' symbol between COF and LOF represents the combination of the data sets in both categories; COF and LOF can be multiplied or added together when calculating risk within InfoAsset Planner®. For this assessment, COF and LOF were multiplied together to calculate risk for all asset types.

Figure 8-1 – Simplified Equation for Calculating Asset Risk

Consequence of Failure (COF) Criteria

COF is defined as the consequence or impact a system would experience during a negative event (most commonly an infrastructure failure). While all water system components are important, some are more critical than others based on a variety of factors including but not limited to customers served, critical facilities, physical location, etc. For example, a 24-inch transmission main, with no redundancy that provides service to 50% of the customer base is more critical than an 8-inch water main with redundancy that only provides service to a few homes. Both water system components are important, but the consequence of losing the 24-inch transmission main is much greater than losing the 8-inch water main. For these reasons, COF criteria were developed in order to understand the consequence magnitude for all water system components. The COF criteria developed for each asset type are provided in each respective asset section outlined later in this chapter.

Likelihood of Failure (LOF) Criteria

LOF is defined as the likelihood or probability that a negative event (most commonly an infrastructure failure; could be either a structural or performance failure) will occur. All water system components will eventually experience issues as time progresses. The LOF side of the risk equation provides an understanding of which water system assets are most likely to fail and why. For example, water mains that have previously experienced breaks are more likely to experience another break versus a water main that has never experienced a failure. By evaluating multiple relevant LOF criteria sets, the approach to determining which assets are most likely to fail is practical and defendable. The LOF criteria developed for each asset type are provided in each respective asset section outlined later in this chapter.

8.1.2 Weighting Factors

Not all criteria for this assessment are considered equal; like water system components, some of the COF and LOF criteria developed are more important than others. A workshop meeting was conducted with District staff to determine appropriate weighting factors for the COF and LOF criteria. Project team members individually ranked criteria based on importance using a scale of 1-10, with 1 representing "minimally important" and 10 representing "critically important." The weighting factors assigned to the criteria served as scaling factors to specify the relative importance of each criterion

Table 8-1 – Criteria Importance Weights

Criteria Weight	Description
10	Critically Important
7 to 9	Very Important
5 to 6	Moderately Important
3 to 4	Somewhat Important
1 to 2	Minimally Important

Table 8-1 provides the criteria weights that were used and assigned to each criterion for each respective asset type risk assessment. Each of the ranking tables presented in this chapter represent the combined input from both the engineering and operations groups within the District. The COF and LOF criteria and weights included for each asset type risk assessment are included within each respective asset section outlined later in this chapter.

8.1.3 Conduct Risk Assessments

Once the criteria are established and weighed, the risk assessment can be performed. Each asset receives an LOF and a COF score based on the criteria established and the respective criteria weight. Those LOF and COF scores can be analyzed independently to determine which assets are most likely to fail or bear the largest consequences of failure but are most powerful when combined and screened within a risk matrix.

	LOF Low	LOF M. Low	LOF Medium	LOF M. High	LOF High
COF High	Medium	Medium	High	Extreme	Extreme
COF M. High	Medium	Medium	High	Extreme	Extreme
COF Medium	Low	Medium	Medium	Extreme	Extreme
COF M. Low	Negligible	Low	Medium	High	High
COF Low	Negligible	Negligible	Low	Medium	Medium

Figure 8-2 – Risk Matrix

A matrix, provided in Figure 8-2, considers the combination of LOF and COF scores and illustrates the risk grade assigned to each component of the water distribution system. The risk grades shown in the matrix correspond to a risk ranking shown in the tables throughout the report. The grades and their respective risk scores are:

- Negligible Risk = 1
- Low Risk = 2
- Medium Risk = 3
- High Risk = 4
- Extreme Risk = 5

The InfoAsset Planner® software application allows for multiple risk scenarios to be created efficiently. Multiple risk scenarios were developed for each respective asset to perform a sensitivity analysis. The results from the risk scenarios were compared and discussed with the District to validate the risk assessment process. The matrix boundaries utilized for the risk assessments presented in this report are shown in Table 8-2.

Table 8-2 – Risk Matrix Boundaries

Consequence of Failure		Likelihood of Failure	
Low	<30	Low	<30
Medium – Low	30 – 40	Medium – Low	30 – 40
Medium	40 – 50	Medium	40 – 50
Medium – High	50 – 60	Medium – High	50 – 60
High	>60	High	>60

8.1.4 Develop Project Recommendations

The final step in the risk assessment process was to develop project recommendations. This is more straightforward for vertical assets such as PRVs, BPSs, and Storage Tanks because vertical assets are visible and can be accessed but is much more complex for buried assets like water mains.

The water main system is comprised of thousands of pipes all containing unique asset identifications. Each unique water main asset received a LOF score, a COF score, a risk score, and a risk grade. One corridor may contain multiple water mains all with unique identifications and associated risk grades. Therefore, project recommendations were developed because not every high or extreme risk asset warranted a project.

For example, assume a corridor contains one water main but it's comprised of six sections ranging in risk grade such as negligible-extreme-negligible-extreme-extreme-low. What type of project should be conducted? Is it more appropriate to conduct a point repair, full water main replacement, or a rehabilitation project? In some cases, an asset may be considered high risk because it has a high COF score, but it may still be in functioning condition. These instances may warrant a condition assessment project rather than a replacement or rehabilitation project.

Every scenario is unique, therefore high and extreme risk assets were reviewed and project recommendations were provided based on engineering judgement, constructability, and feasibility concerns. The forthcoming sections in this chapter provide the details and results for each risk assessment conducted for each asset type.

8.2 Water Main

Although water mains are entirely out of sight, they are never out of mind for the District. Water main breaks are never pleasant for any utility, but the mountainous terrain and elevated water pressures most of the District operates under can quickly change a minor break into a dangerous situation. Preventative measures are the best protection against catastrophic pipe failure.

Each of the LOF and COF criteria used to assess the risk of each water main are explained in the following subsections.

8.2.1 Likelihood of Failure Criteria and Weighting Factors

The following tables summarize how the pipe LOF criteria were evaluated for water main which include:

- Pipe Installation Year
- Pipe Material
- Soil Corrosivity to Steel
- Pipe Velocity
- Maximum Pipe Pressure
- Pipe Breaks

Pipe installation year is used as one of the LOF criteria as older water main generally experiences more failures when compared to newer water main. For the District's water main, the highest LOF score was assigned to the older pipes and then number sequentially by decade for the year of installation, as shown in Table 8-3.

Table 8-3 – Pipe Installation Year LOF

Pipe Installation Year	LOF Score
Pre 1975	10
1975 – 1984	8
1985 – 1994	6
1995 - 2004	4
2005 - 2019	2

The District currently uses Ductile Iron Pipe for all new pipe installations. Some of the older pipe throughout the system is Cast Iron Pipe. The LOF scores assigned based on pipe material are shown in Table 8-4.

Table 8-4 – Pipe Material LOF

Pipe Material	LOF Score
Cast Iron Pipe	10
Ductile Iron Pipe	2

USDA soil data (Web Soil Survey) was utilized to assess the soil corrosivity to steel. The risk of corrosion pertains to potential soil-induced electrochemical or chemical action that corrodes or weakens uncoated steel and other metals. The rate of corrosion of uncoated steel is related to factors such as soil moisture, particle-size distribution, acidity, and electrical conductivity of the soil. The risk of corrosion is expressed as low, moderate, or high as shown in Table 8-5 with more corrosive soil yielding a higher LOF score.

Although pipe velocity does not directly cause pipe failure, higher velocities increase the risk of damage due to hydraulic transients and scouring of pipe lining over time. The LOF assigned for the maximum pipe velocity under maximum day demand is shown in Table 8-6 where pipes with higher velocities received a higher LOF score.

Results from the hydraulic model were used to assign LOF scores based on the maximum operating pressures experienced under maximum day demand. These scores are shown in Table 8-7 where pipes with higher pressure receive a higher LOF score.

Work order history provides valuable insight into the assets that may be more likely to fail in the future based on history of pipe breaks/leaks in the area. Table 8-8 summarizes how the break history is used to assess likelihood of failure. Pipes with a higher count of work orders receive a higher LOF score.

Table 8-5 – Soil Corrosivity LOF

Soil Corrosivity	LOF Score
High	10
Moderate	5
Low	1

Table 8-6 – Pipe Velocity LOF

Pipe Velocity (under MDD)	LOF Score
>5 ft/s	10
3-5 ft/s	8
1-2.9 ft/s	6
0.6-1 ft/s	3
<0.5 ft/s	1

Table 8-7 – Pipe Maximum Pressure LOF

Pipe Maximum Pressure (under MDD)	COF Score
>110 psi	10
90-110 psi	7
70-89 psi	5
50-69 psi	3
<49 psi	1

Table 8-8 – Pipe Break LOF

Count of "Emergency Repair" Work Orders	LOF Score
>3	10
3	8
2	6
1	3
0	0

LOF criteria and their respective weighting factors for the Water Mains are summarized in Table 8-9. The weighting factor is a multiplier for the LOF score of each asset within the respective category. For example, if a pipe received a LOF score of 5 for the soil corrosivity, the multiplier of 9 would be applied and the overall LOF contribution of soil corrosivity for that pipe would be 45 (9 x 5 = 45).

Table 8-9 – Water Main LOF Criteria Weight

Likelihood of Failure Criteria	Weighting Factor
Number of Leaks	10
Pipe Material	4
Installation Year	6
Soil Corrosivity to Steel	8
Slope Stability	9
Maximum Velocity	1
Maximum Pressure	2

8.2.2 Consequence of Failure Criteria and Weighting Factors

The following tables summarize how the pipe COF criteria were evaluated which include:

- Pipe Diameter
- Pipe Maximum Flow Rate
- Number of Customers Connected (address points)
- Critical Facilities Connected
- Pipe Accessibility for Repairs
- Pipe Redundancy

Larger pipe diameters generally correspond to transmission mains that serve as the backbone of the distribution system. As shown in Table 8-10, the larger the pipe diameter, the large the assumed consequence of failure.

Table 8-10 – Pipe Diameter COF

Pipe Diameter	COF Score
Greater than 12-inch	10
12-inch	8
10-inch	6
8-inch	5
6-inch	2
Less than 6-inch	1

Closely related to the pipe diameter is the maximum flow rate experienced in the pipe. Results from the hydraulic model were used to assign COF scores based on the maximum flow rate expected under maximum day demand. These scores are shown in Table 8-11 where pipes with higher flow receive a higher COF score. Velocity was used as a likelihood of failure criteria and flow was used in consequence of failure criteria as velocity is related to pipe size and flow is dependent upon nearby pumping and demands.

Table 8-11 – Pipe Maximum Flow Rate COF

Pipe Maximum Flow Rate (under MDD)	COF Score
>700 gpm	10
500-699 gpm	9
400-499 gpm	8
300-399 gpm	7
200-299 gpm	6
100-199 gpm	5
50-99 gpm	4
25-49 gpm	3
10-24 gpm	2
<10 gpm	1

The number of individual customer connections to an individual pipe (based on a count of address points) was used as a consequence of failure criteria. As shown in Table 8-12, pipes with a higher number of customers connected received a higher COF score. Although the count of connected customers provides less weighting to single customer high volume users (resorts), the following criteria

Table 8-12 – Connected Customer COF

Number of Customers Connected to Pipe	COF Score
>100	10
80-100	9
60-79	8
40-59	7
20-39	6
10-19	5
5-9	4
<5	2

Proximity to critical facilities was used to assign the pipe COF criteria shown in Table 8-13. Pipes within 1,000-feet of critical facilities received a higher COF score. Critical facilities were broadly defined as:

- Medical Facilities;
- Fire Stations;
- Economic Hubs.

Table 8-13 – Critical Facilities COF

Distance to Critical Facilities	COF Score
Medical Facilities	10
Fire Stations	5
Economic Hubs	5

Pipe accessibility has a tremendous impact on how quickly a pipe can be repaired/replaced when needed. Pipes that cross under Highway 6, the Snake River, or County Road 5 (approaching Keystone Resort) would be significantly more difficult to access for repairs. Additionally, winter emergency repairs are difficult due to snow removal, deep frost, and extreme weather conditions. The COF scores selected for the accessibility criteria are shown in Table 8-14.

Table 8-14 – Pipe Accessibility for Repairs COF

Accessible for Repairs	COF Score
Highway 6	10
Snake River	9
Snow Piling Locations	8
County Road 5	7

Pipe redundancy is a measure of whether a break on each individual segment of pipe would result in a severe service interruption, or if sufficient looping exists to maintain service to customers. A COF score of 10 was given to all pipes lacking redundancy, while those with redundancy were given a score of 0. This is shown in Table 8-15.

Table 8-15 – Pipe Redundancy COF

Does Pipe Have Redundancy	COF Score
No	10
Yes	0

Each of the water main COF criteria previously explained were assigned a weighting factor, as summarized in Table 8-16. The weighting factor is a multiplier for the COF score of each asset within the respective category. For example, if a pipe received a COF score of 5 for the maximum flow rate, the multiplier of 7 would be applied and the overall COF contribution of flow rate for that pipe would be 35 (7 x 5 = 35).

Table 8-16 – Water Main COF Criteria Weight

Consequence of Failure Criteria	Weighting Factor
Pipe Diameter	1
Maximum Flow Rate	2
Number of Customers Connected	5
Proximity to Critical Facilities	2
Accessibility for Repairs	5
Redundancy	4

8.2.3 Pipe Risk Results

The pipe risk results shown by size and risk category are shown in Table 8-17. Overall, less than 10-percent of the system fall in the extreme or high-risk category. The pipe risk result map is shown in Figure 8-3 and Figure 8-4.

Table 8-17 – Pipe Risk Analysis Data

Pipe Diameter	Risk Category				
	Extreme	**High**	**Medium**	**Low**	**Negligible**
4-inch	-	-	1,446	562	667
6-inch	-	214	2,633	9,562	11,694
8-inch	2,958	2,088	14,374	24,800	33,019
10-inch	159	1,224	3,674	8,638	5,158
12-inch	4,108	3,901	11,428	8,729	4,143
16-inch	-	-	1,020	-	-
Length per Category (feet)	**7,225**	**7,427**	**34,575**	**52,291**	**54,681**
Percent of System Length per Category	4.6%	4.8%	22.1%	33.5%	35.0%

8.2.4 Water Main Capital Replacement Projects

Based on the results of the risk analysis, the replacement of all the extreme and high risk pipe was used to develop a budget for the water main replacement program. This results in the replacement of approximately 14,500 feet of pipe over the next 10-years. This would replace just under 10-percent of the existing piping. Based on the risk results in Figure 8-3 most of the replacement projects will be in the Mountain House Area. The projects may be larger or smaller than the annual budget shown in the capital improvement plan in section 10.4 based on scope of each replacement project.

8.3 Hydrants

Existing fire hydrants risk was evaluated to prioritize hydrants for replacement as part of an annual replacement program. Hydrants provide a vital function for fire-fighting events while also allowing operators to flush water through the system.

Each of the LOF and COF criteria used to assess the risk of each water main are explained in the following subsections.

8.3.1 Likelihood of Failure Criteria and Weighting Factors

The following tables summarize how the pipe LOF criteria were evaluated for water main which include:

- Distance to Critical Facilities
- Hydrant Age

The District GIS data included a field to identify if each hydrant "Needs Attention" which can be various maintenance items such as gasket replacement, internal component replacement, painting, etc. This data was either yes" or no" so the scoring was simple as shown in Table 8-18

Table 8-18 – Hydrant Maintenance LOF

Needs Attention	LOF Score
Yes	10
No	0

Hydrant age is used as one of the LOF criteria as reliability of the assets will decrease as age increases. For the District's hydrants, the highest LOF score was assigned to the older hydrants and then number sequentially in 5-year increments for the year of installation, as shown in Table 8-19.

Table 8-19 – Hydrant Age LOF

Hydrant Installation Year	LOF Score
Pre-1975	10
1975 - 1980	9
1981 - 1985	8
1986 - 1990	7
1990 - 1995	6
1996 - 2000	5
2001 - 2005	4
2006 - 2010	3
Post-2010	1

LOF criteria and their respective weighting factors for the hydrants are summarized in Table 8-20. The weighting factor is a multiplier for the LOF score of each asset within the respective category. For example, if a hydrant received a LOF score of 8 for the age, the multiplier of 3 would be applied and the overall LOF contribution of age for the hydrant would be 24 (8 x 3 = 24).

Table 8-20 – Hydrant LOF Criteria Weight

Likelihood of Failure Criteria	Weighting Factor
Hydrant Maintenance	4
Hydrant Age	3

8.3.2 Consequence of Failure Criteria and Weighting Factors

The only consequence of criteria used in the hydrant risk analysis is distance to critical facilities. Since the primary purpose of the hydrant is to provide quick access for fire-fighting the critical facility criteria was used.

Table 8-21 shows the distance range and resulting score for the criteria. The weight of this criteria was a value of 2. While there is only one consequence criterion, the weighting does affect the overall score when paired with the likelihood of failure criteria.

Table 8-21 – Distance to Critical Facilities COF

Distance Range	COF Score
Less than 400 feet	10
401to 500 feet	9
501 to 600 feet	8
601 to 1,000 feet	7
1,001 to 2,000 feet	6
2,001 to 4,000 feet	5
4,001 to 6,000 feet	4
6,001 to 8,000 feet	3
8,001 to 100,00 feet	2
Greater than 10,000 feet	1

8.3.3 Hydrant Risk Results

Table 8-22 provides the hydrant risk results by category and installation year. Overall, less than 2-percent of the hydrants fall in the extreme risk category with medium and low risk results constituting over 78% of the results. A map of the hydrant results by risk category is provided in Figure 8-5.

Table 8-22 – Hydrant Risk Results

Installation Year	Risk Category				
	Extreme	**High**	**Medium**	**Low**	**Negligible**
Pre-1980	0	5	39	4	0
1980-1989	1	20	47	10	6
1990-1999	4	1	17	52	21
2000-2009	0	1	6	34	0
2010-Present	0	1	6	16	4
Total	**5**	**28**	**115**	**116**	**31**
Percent of Hydrants per Category	1.7%	9.5%	39.0%	39.3%	10.5%

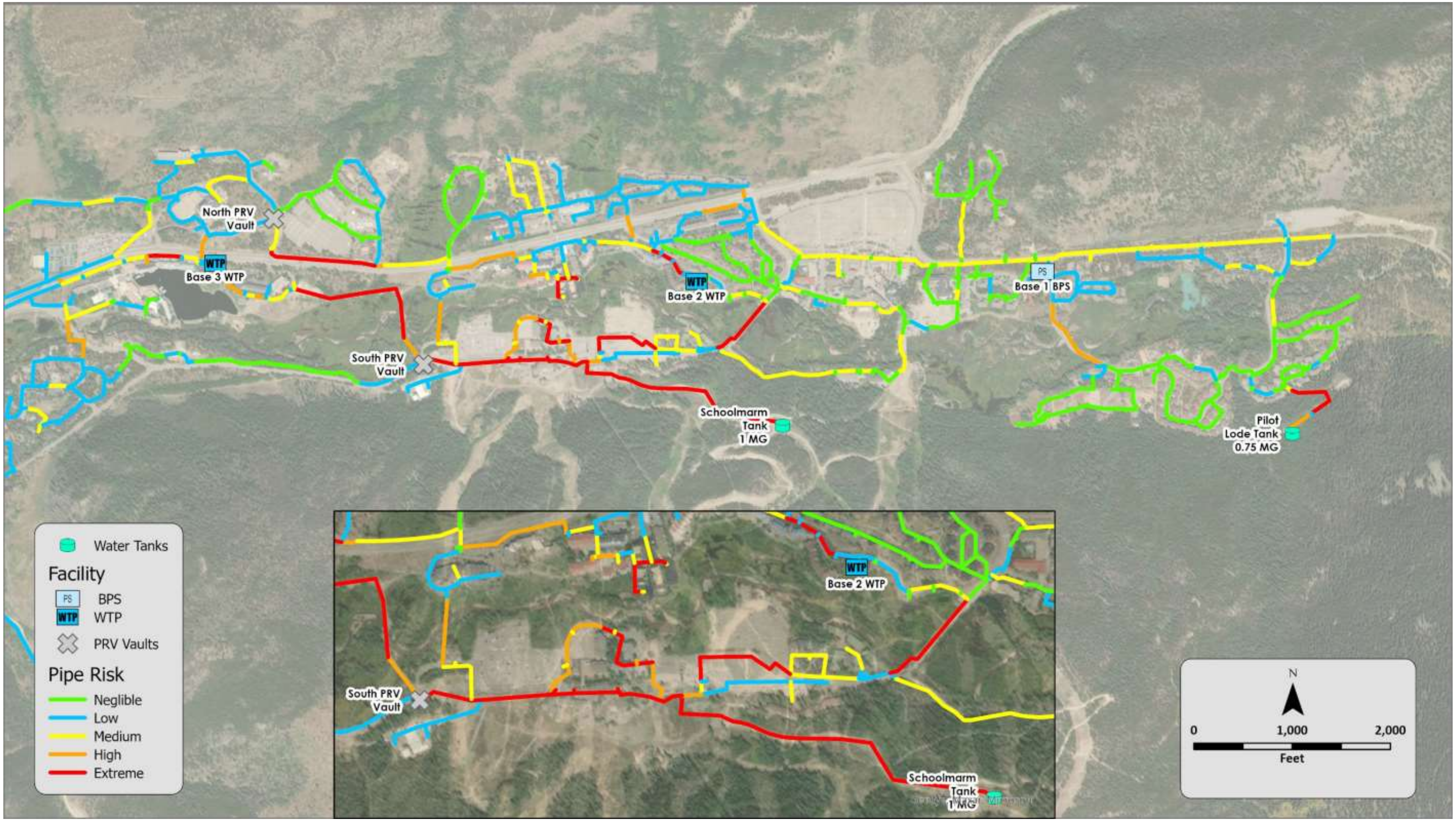


Figure 8-3 – Base 1 and 2 Pipe Risk Results

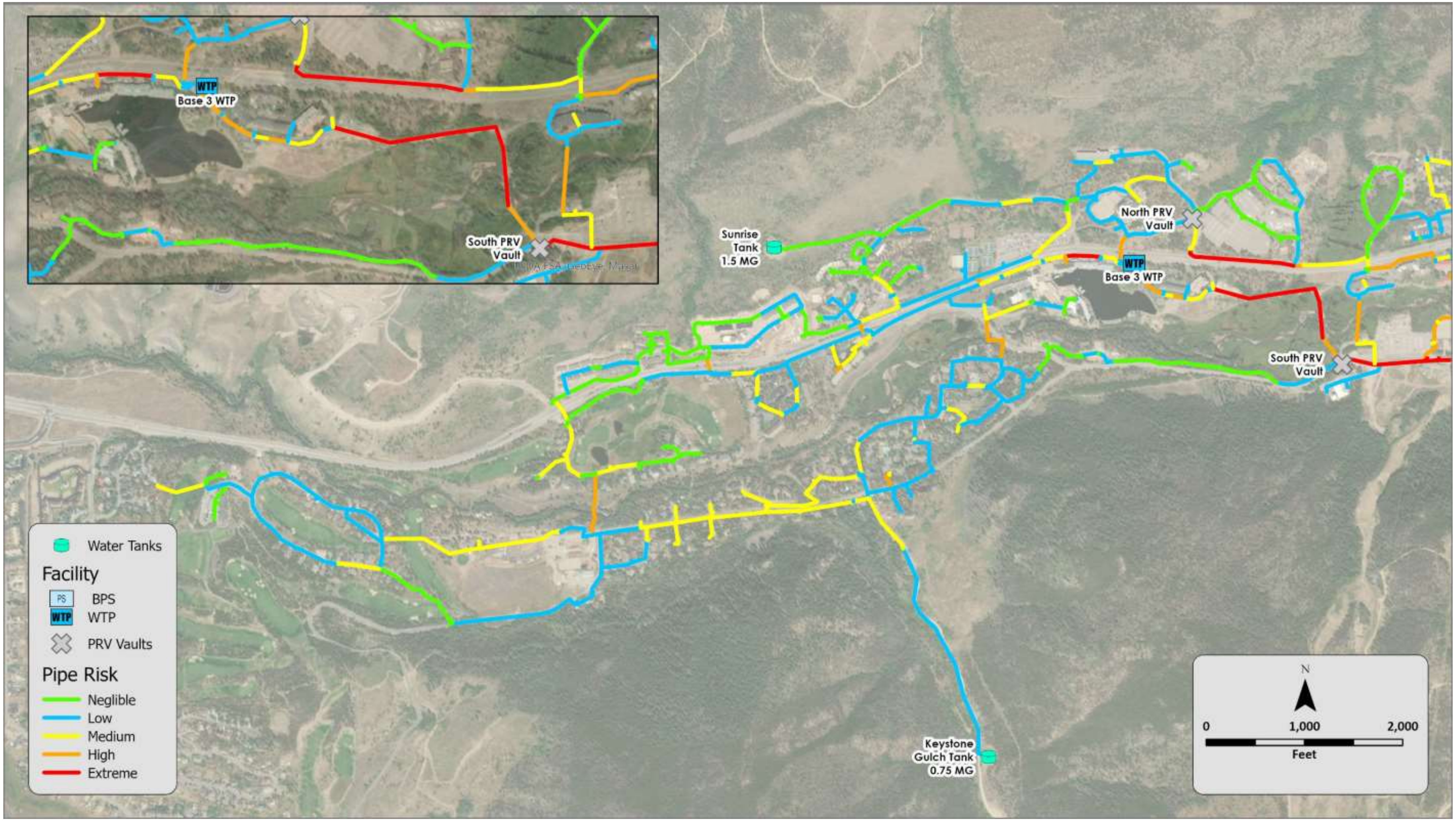


Figure 8-4 – Base 3-4 Pipe Risk Results

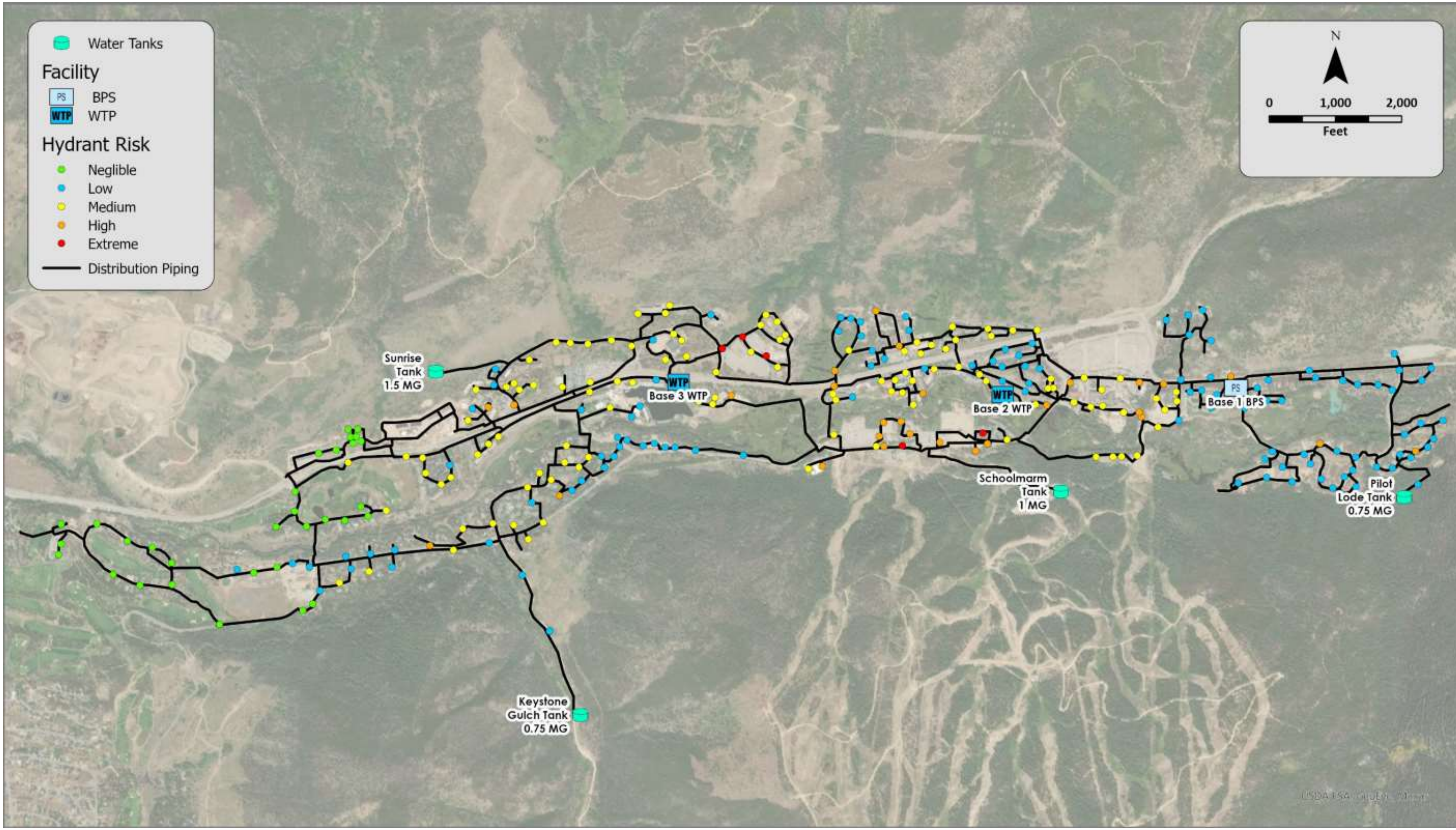


Figure 8-5 – Hydrant Risk Results

9.0 Future System Evaluation

9.1 Future System Modeling Scenarios

The future maximum day demand scenario was the primary scenario used to evaluate future system performance. The future MDD values were presented in subsection 4.3.1 with demands spatially allocated as discussed in Chapter 5.0.

9.2 Future System Demands and Production

Figure 9-1 shows the future maximum day demands for each pressure zone along with the existing production and pumping capacities. The system has ample production capacity as the Base 2 WTP firm capacity can meet Base 1 and Base 2 maximum day demands while the Base 3 WTP can meet the Base 3-4 maximum day demands. The District also should be able to fully supplement the Base 3-4 demands through the South flow control valve as long as the Base 2 WTP is a full capacity. Operations staff can also transfer water manually via the North pressure reducing valve not shown in Figure 9-1.

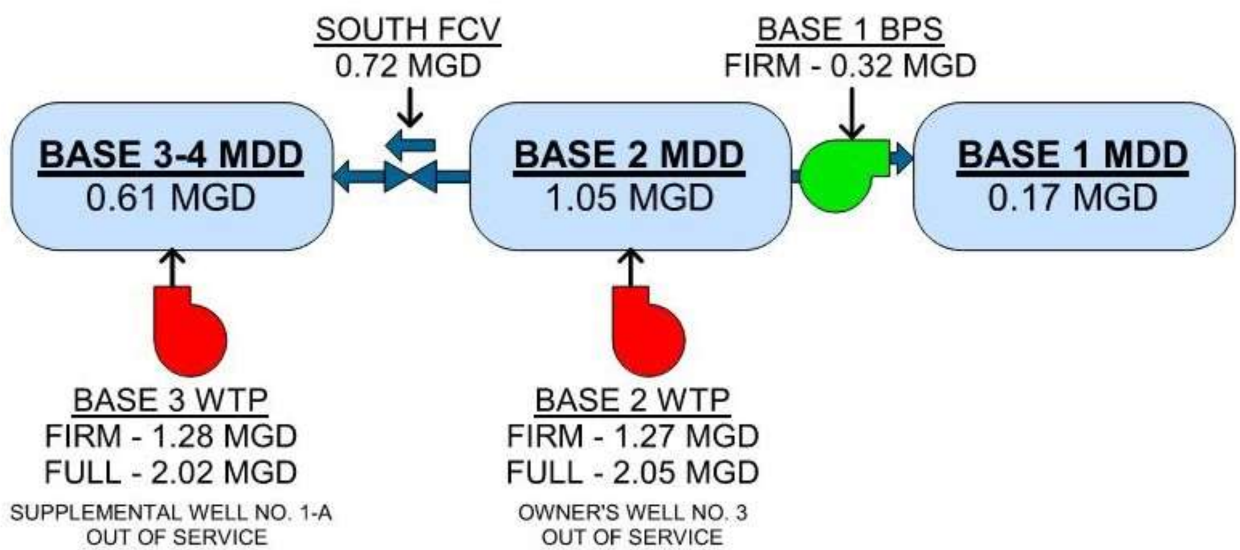


Figure 9-1 – Future System Production and Pumping Capacity versus Future MDD

9.3 Future System Pressure Evaluation

Pressure in the water system with future MDD did not vary by more than 3 psi from the existing evaluation results. The addition of a new Base 3 to Base 2 BPS marginally increased pressures in Base 2 near the discharge of the pump station. Minimum pressure in the North Fork area increase by approximately 2 psi as the tank eliminated headloss in the pipe feeding the North Fork area.

9.4 Future System Storage Evaluation

9.4.1 Tank Capacity

A new 1-million gallon tank was added in the North Fork area to provide additional storage for the Base 2 pressure zone. Table 9-1 shows the resulting capacity evaluation with the future demands incorporated. The fire storage requirement is not anticipated to increase as any large development in the county will require fire sprinklers effectively reducing the fire flow event to 4,000 gpm for 4 hours. Adding a "twin" tank to Base 2 provides adequate storage capacity and will allow for inspection, maintenance, and potential construction improvements to the Schoolmarm Tank to mitigate slope instability.

Table 9-1 – Future Tank Capacity Evaluation

Pressure Zone	Tank Capacity (Gallons)	Storage Requirement (gallons)				Storage Surplus / (Deficit)	% of Avg Summer Demand in Emergency	% of MDD in Operational / Equalization
		Fire	Emergency	Operational / Equalization	Total Required			
Base 1	750,000	180,000	71,500	51,700	**231,700**	518,300	75%	30%
Base 2	2,000,000	960,000	456,300	285,400	**1,245,400**	754,600	75%	30%
Base 3-4	2,250,000	960,000	332,000	213,600	**1,173,600**	576,400	75%	30%

The new Base 2 tank also provides additional AFF in the Base 2 pressure zone as water can flow from in two directions to the demand; from the new tank and existing Schoolmarm tank. Figure 9-2 shows the existing AFF and AFF with the 1-MG tank in black and red text, respectively. The AFF increases throughout the River Run area with improvements also observed in the Enclave development area north of Hwy 6. Though the primary issue solved by the additional tank is storage capacity, improvements to fire flow rates is an additional benefit.

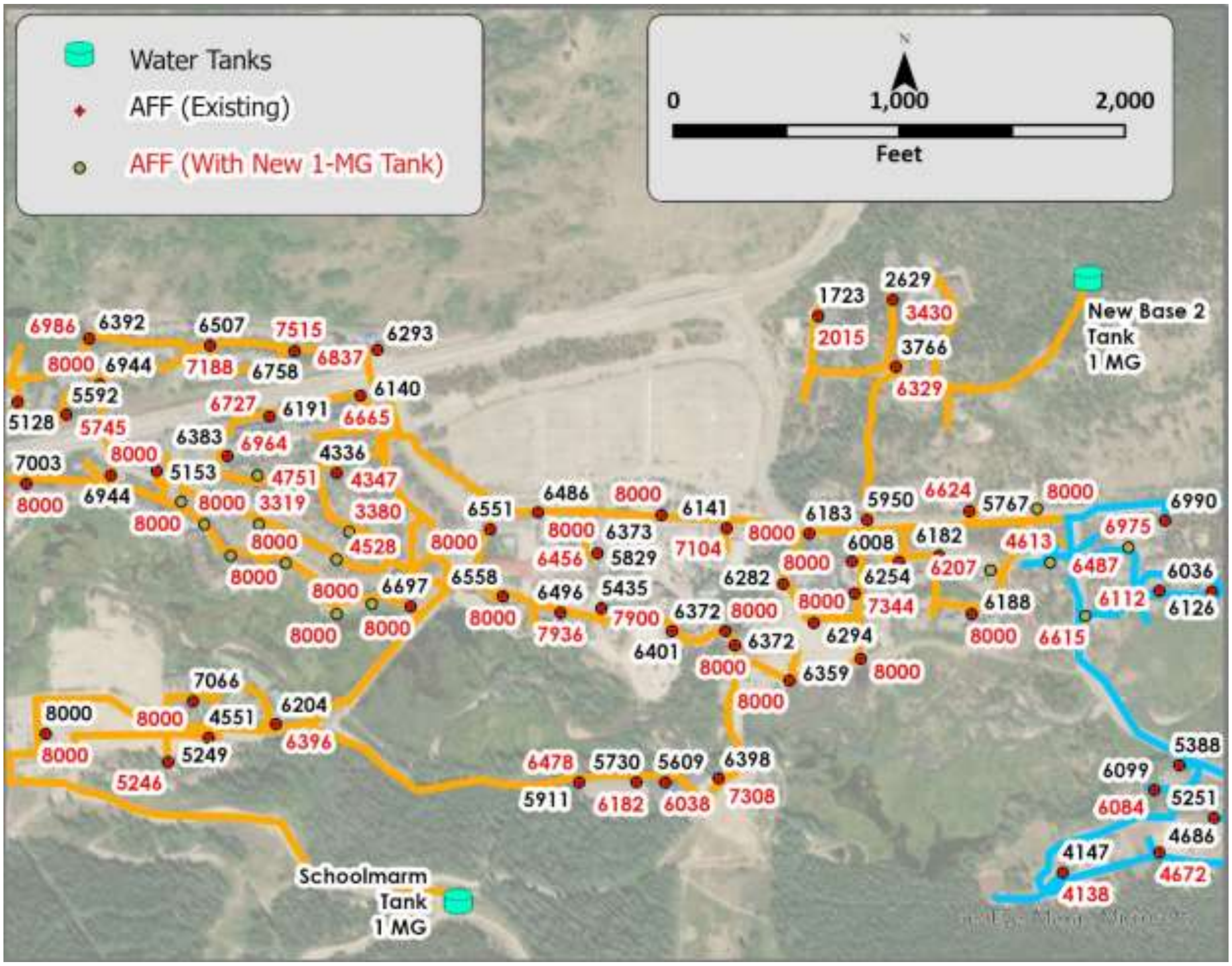


Figure 9-2 – AFF with New 1-MG Tank

9.5 Future System Pumping Capacity

The Base 1 BPS has adequate firm capacity to meet future MDD in the Base 1 pressure zone. If the pumps are replaced it is recommended to provide a firm capacity of 125% of the future MDD. Pump combinations of two 250 gpm or three 200 gpm pump would meet this criterion.

The proposed Base 3 to Base 2 pump station should have a firm capacity that can meet the combined MDD of Base 2 and Base 1 pressure zones. This MDD value is 1.22 MGD or approximately a flow rate of 850 gpm over a 24-hour period. Pump capacity configurations and considerations are further discussed in subsection 10.3.1 for the proposed pump station.

9.6 Future Transmission and Distribution Main Capacity

The existing system exhibited ample distribution main capacity under existing maximum day demand conditions. Even with the additional buildout demands, all pipes in the system meet the velocity and headloss criteria presented in Chapter 6.0. Headloss between 1-2 ft/kft is observed in the 12-inch branch piping to Schoolmarm Tank and on two pipe segments in the Mountain House area; these instances are the only increases in headloss into the 1-2 ft/kft range when compared to the existing system analysis.

9.7 Future Fire Flow Analysis

Fire flow rates do not significantly change with the future buildout demands added the system. The existing areas with potential issues identified in the existing system fire flow analysis remain. The addition of the new Base 2 storage tank provides increase available fire flow as discussed earlier in this chapter.

9.8 Summary of Future System Evaluation

Key items resulting from the future system evaluation are bulleted below:

- The firm capacity of water treatment plants has capacity to meet total system buildout demands.
- Base 1 BPS has ample firm capacity to meet buildout demands.
- Storage in Base 1 and Base 3 remains adequate with buildout demands.
- Maximum pressure criterion is met in all areas.
- Minimum pressures areas of concern remain in North Fork.
- Headloss and velocity in the distribution system is remains below the criteria.
- Available fire flow rates in the Base 2 pressure zone are increased with the new Base 2 storage tank. Areas with lower fire flow rates remain in areas identified in the existing system evaluation. As additional storage is added the tank operations will be adjusted to maintain tank turnover and minimize water age in the system.

10.0 Capital Improvement Planning

The chapter will cover recommended capital improvement projects which will cover both new assets along with asset replacement projects. The replacement projects shown are considerable cost which would generally be above a water system's typically operations and maintenance budgets. This chapter will review previously implement projects since the past master plan, provide a background of opinion of probably project costs, estimate the timing for the need of the project and summarize the spending over the next 10 years.

10.1 Previously Recommended Improvements

The 2012 Master Plan recommended eight projects as a result of the planning efforts. The District has implemented four of the projects since completion of the Master Plan. These projects include repair to the South PRV, quick connection for backup power generation at the Base 2 WTP, installation of advanced meter infrastructure for customer meters, new Base 2 WTP, and addition of SCADA system at the South PRV for remote flow control capabilities. The District has not implemented the new Base 2 storage tank, piping improvements on Rasor Dr., or piping improvements at Saints Johns condos.

10.2 Upcoming Regulatory Issues.

There are some ongoing regulatory issues going through development at the time of this master plan project. These issues are identified and discussed below.

Lead and Copper Rule Long-Term Revisions

Revision to the lead and copper rule have been in development with final rulemaking expected in 2021. The latest proposed revisions include the following requirements:

- Using science-based testing protocols to find more sources of lead in drinking water.
- Establishing a trigger level to jumpstart mitigation earlier and in more communities.
- Drive more complete replacement of lead service lines.
- Require testing in schools and childcare facilities.
- Require water systems to identify and make public the locations of lead service lines.

The rule will likely increase expenditures to comply with the revisions. The full detail and scope of expenditures is difficult to develop until the rule is finalized; therefore, it is recommended to complete a study to identify an action plan for compliance.

Perchlorate

The inclusion of perchlorate on the safe drinking water act regulations was under review since 2011. In June of 2020, the US EPA made a final determination not to regulate perchlorate. There is no indication that a perchlorate regulation will be developed at the state level.

Hexavalent Chromium

In 2010, the US EPA released a draft scientific human health assessment for hexavalent chromium (chromium-6) for public comment and peer review. Once the assessment is finalized, the EPA may move into rulemaking establishing a more stringent total chromium standard (currently 0.1 mg/L) or identify a separate standard for chromium-6. This regulation is expected to have little effect on the District's operations as current total chromium levels were not detectable during the inorganic chemical monitoring period.

PFOS and PFOA

Perfluorooctanesulfonate (PFOS) and perfluorooctanoic acid (PFAS) are fully fluorinated organic substances used in many manufacturing processes and also found in fire-fighting foam. These substances can accumulate in the human body and can affect health. In 2009, the US EPA issued a health advisory for PFOS and PFOA then issued a new health advisory targeting a limit of 0.07 mg/L in 2016. Health advisories are non-enforceable and serve to inform water providers and regulating agencies about potential impacts of these chemical.

In February of 2020, the US EPA issued a preliminary determination to regulate PFOS and PFOA. The state's primacy agency added rules in September of 2020 to limit the concentration of PFAS and PFOA at wastewater discharges but has not adopted a drinking water standard. The state's primacy agency reviewed if it has the legal authority to enforce a drinking water regulation beyond the safe drinking water act which is set at the federal level.

Manganese

Currently, manganese is included in the Secondary Drinking Water Standards for its aesthetic effects (brown staining upon oxidation). Manganese data was collected during unregulated contaminant monitoring rule (UCMR) 1 period from 2001 to 2005, but EPA determined not to regulate manganese with a Primary Drinking Water Standard at that time. However, in 2004, EPA issued a health advisory, recommending drinking water supply manganese concentration not exceed 0.3 mg/L, based on a lifetime exposure to manganese concentrations, and not exceed 1.0 mg/L for 1-day and 10-day acute exposure and recommended 0.3 mg/L be used for 10-day acute exposure for infants younger than 6 months. Preliminary health assessments indicate that excessive manganese concentrations cause adverse neurological impacts, especially in infants, although more research was needed to determine if the health impacts are sufficient to require regulation of manganese as a health-related (required) drinking water standard. The EPA will be gathering manganese concentration and occurrence data from public water supplies through the UCMR 4 period which runs from 2018 to 2020, after which a regulatory decision will be made. Although the District's treated water manganese concentrations are well below the 2004 EPA health advisory levels, the District should retain manganese removal as a treatment objective in anticipation of potential future manganese primary drinking water standards.

10.3 CIP Projects

Capital improvement planning project identify, priority and timing will be split up into new infrastructure projects and replacement of existing infrastructure projects. The intent is to balance investment into the new infrastructure while also maintaining existing infrastructure to maintain an acceptable level of service.

10.3.1 New Infrastructure Projects

There were three new infrastructure projects developed as part of this master plan. These projects include a new pump station to transfer water from Base 3 to Base 2, a new water storage tank for the Base 2 pressure zone and the potential for a new or renovated Base 2 water treatment plant. The reason, benefits and challenges of the projects will be further described below.

Pump Station

The District recently invested in a new water treatment plant at the Base 3 pressure zone which can treat up to 2.0 MGD and can meet the requirements of treating GWUDI classified source water. However, the new Base 3 water treatment plant can only distribute water to the Base 3 pressure zone which covers 38% of the District's EQRs. The new pump station would allow operations staff to pump water treated by the Base 3 WTP into the Base 2 pressure zone thus allowing Base 3 treated water to be distributed to the entire system, enhancing the recent investment into the Base 3 WTP. The project would also benefit Base 2 storage tank capacity by allowing Base 3 storage to supplement water to Base 2 pressure zone during and after a fire event.

The new pump station can also mitigate the risk of Base 2 source water becoming classified as a GWUDI source. The pump station would allow additional time to design, construction and commission a new Base 2 WTP if required due to a GWUDI source reclassification. The proposed pump station would allow shutdown time for construction and rehabilitation of the existing Base 2 WTP assets even if additional treatment for GWUDI is not required.

The recommended firm capacity of the pump station is 1,000 gpm with three pumps each rated for 500 gpm at 155 feet total dynamic head based on the system curve provided in Figure 10-1. This would result in 25 HP or 30 HP motors on each pump depending upon pump efficiencies. The pumps could be end suction horizontal type similar to the Base 1 BPS or packaged vertical turbines pumps such as the Grundfos CR line are an option.

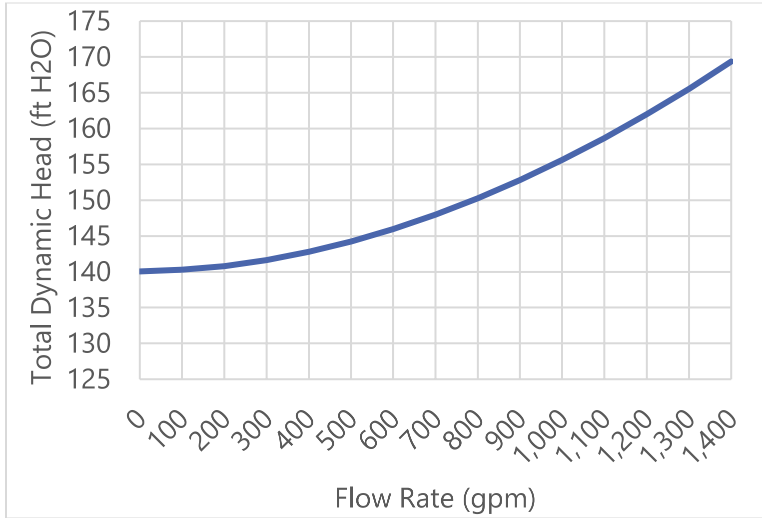


Figure 10-1 – Base 3 to Base 2 BPS System Curve

The location of the proposed BPS was near the North PRV which separates the Base 3 and Base 2 zones. There is approximately 650 linear feet of 6-inch diameter pipe that should be replaced with 12-inch diameter pipe to reduce velocities on the suction side of the pump station. The location of the proposed BPS, pipe replacement and example photos of an underground pump station vault is provided in **Error! Reference source not found.**. Either an underground vault style pump station or above ground pump station with new building are an option for the pump station. The underground vault is expected to cost less than the above ground pump station and was used in the capital improvement plan.

The total cost of the new BPS project using an underground prefabricated pump station is estimated to be $1,504,000 in 2022 dollars. The cost for an above ground pump station is estimated to be $1,655,000 in 2022 dollars.

Storage Tank

A new storage tank was identified to provide additional storage capacity for fire and operational storage in the Base 2 pressure zone. The recommended capacity of the new tank is 1.0 million gallons with an overflow elevation matching the Schoolmarm Tank. The base elevation should closely match the Schoolmarm tank though minor variance in the base elevation could be considered. Additionally, operations staff and local geotechnical experts have identified slope instability near the Schoolmarm Tank. The new storage tank would mitigate the risk of further slope movement and potential loss of the Schoolmarm Tank. The new tank also improves the fire flow rates at hydrants in the River Run area which contains large buildings with high flow rate requirements.

The total cost of the new storage tank project is estimated to be $7,578,000 in 2026 dollars.

Supply and Treatment

If the source water at the Base 2 wells is determined to be GWUDI then the District will need to provide additional treatment methods as previously discussed in subsection 2.1.2. Since the source water is low in iron and manganese both ultrafiltration membranes and convention media filters were identified as alternate treatment methods to meet filtration requirement.

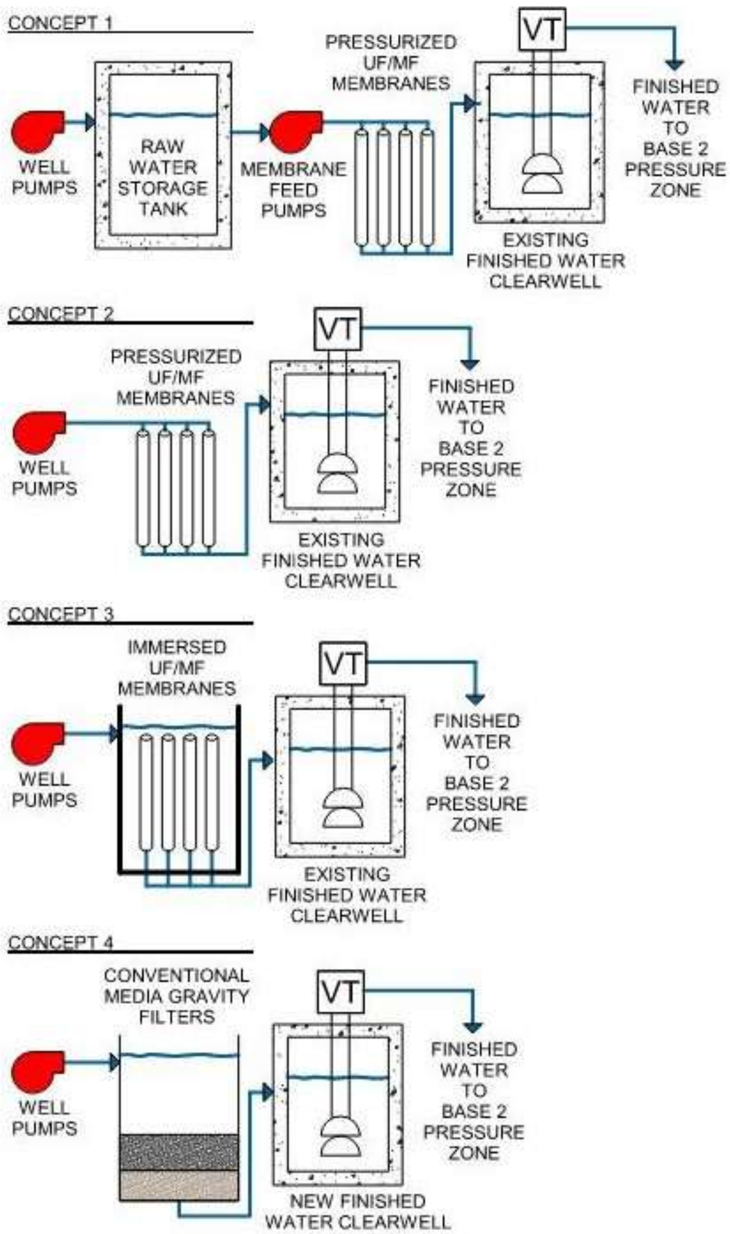


Figure 10-2 – Base 2 WTP Treatment Process Alternatives

Ultrafiltration membranes provide an advantage over conventional filters as the 3.0-log *Giardia* removal requirement can be met with the membranes. This would likely eliminate the need for an expanded clearwell as only 4.0-log virus removal will be required through chlorine disinfection; the 4.0-log virus removal requirement is already met as a groundwater source.

If conventional filters are used, then the clearwell will need to be sized to provide 1.0-log inactivation of *Giardia* via disinfection as conventional filters provide 2.0-log credits for *Giardia* removal. The required volume of the clearwell will be approximately 100,000 gallons based on a free chlorine residual of 1.4 mg/L, plant flow rate of 1,400 gpm and clearwell baffling factor value of 0.7; this is nearly 2.3 times larger than the existing clearwell.

Figure 10-2 provide simple schematics of treatment alternatives to meet GWUDI requirements; two concepts use pressurized membranes; one concept uses immersed membranes and the last concept used the conventional media filters.

Each of the treatment alternatives will need to consider space for additional processes not shown in the Figure 10-2 schematics. These processes include:

- Conventional Filters
  - Backwash Reclaim Basin
  - Reclaim Pumps
  - Backwash Blower for Air Scour
  - Backwash Flow Rates and Potential Impact on Distribution Piping
  - Space for Filter Piping
  - Control Panel
  - Clearwell expansion
- Ultrafiltration Membranes
  - Clean-In-Place Chemical Storage and Handling
  - Reclaim Basin
  - Reclaim Pumps
  - Strainers
  - Backwash Air Compressors
  - Consideration of backwash water pH and its impacts on membrane performance

The project was estimated on the cost of Concept 1 in Figure 10-2 for capital improvement planning purposes. The ability of the ultrafiltration membranes to provide the 3.0-log Giardia removal and meet filtration requirements likely results in cost savings of rebuilding a new and larger clearwell for conventional filters. The use of well pumps or dedicated membrane feed pumps along with using immersed type membranes can be further explored during preliminary design or during a pilot study in preparation for design.

The total project cost is estimated to be $11,729,000 in 2023 dollars.

A potential alternative to a new WTP is to relocate the wells to be outside the influence of surface water and remain classified as a groundwater source. This alternative will have challenges of finding physical locations for the wells with available groundwater capacity, require negotiations with private landowners, subject the change to review and approval from the water rights authority, and require installation of new raw water piping and electrical circuits to power the pump motors. The investigation of identifying alternate well locations will require upfront cost to find available well capacity that may result in excess costs if multiple locations are examined and determined to be unsuitable. The new location of the well may also impact existing wells in the area which be reviewed in the water right authority’s process.

10.3.2 New Infrastructure Project Priority and Timing

The required timeline of new infrastructure projects is largely driven by growth and regulatory process. The most significant regulatory item that will drive large capital expenditures in the next 10-year will be the GWUDI evaluation of the Base 2 wells. Near the end of the master plan project, the District was informed Owner's Well No. 3 and SRWD Well No. 1 were required to undergo GWUDI testing in 2021. Therefore, the new Base 3 to Base 2 pump station is the highest priority project recommended due to the following benefits:

- Can mitigate the impact and timing constraints if the Base 2 wells are determined to be GWUDI source by allowing Base 3 WTP to feed the entire system during improvements to meet GWUDI treatment requirement.
- Enhances the investment into the Base 3 WTP by allowing the new WTP to feed water into the entire system.
- Can allow Base 2 WTP downtime for construction purposes not related to improvements for GWUDI source. Examples include flow meter replacement, valve replacement, clearwell inspection, chemical feed improvements, and electrical equipment replacement projects.
- Can supplement water into Base 2 both during and after a high demand event such as a fire event.

If the two wells are reclassified as a GWUDI source, the Base 2 WTP would require the treatment improvements by mid-2023 for water produced by those wells. The design, construction, and commissioning of the new Base 3 to Base 2 BPS should be able to be completed well before mid-2023 and provide risk mitigation.

If the GWUDI evaluation is not required within the next 5 years, the District may consider accelerating the construction of the new Base 2 storage tank ahead of the timeline presented in this Master Plan. A timeline summary of the new improvements is presented below:

- 2022: Complete construction of new Base 3 to Base 2 BPS
- 2024: Complete construction of the new Base 2 WTP meeting GWUDI requirements
- 2026: Complete construction of the new Base 2 storage tank

10.3.3 Replacement of Existing Infrastructure Projects and Timing

Determining replacement of existing infrastructure using the age of the asset is a common method for determining replacement priority and timing. The age of the assets can be coupled with other criteria as exercised in Chapter 8.0 to prioritize asset replacement based on more than age.

Table 10-1 – Estimated Useful Life of Water System Assets

Estimated Useful Lives

Asset	Expected Useful Life (in years)
Intake Structures	35-45
Wells and Springs	25-35
Galleries and Tunnels	30-40
Chlorination Equipment	10-15
Other Treatment Equipment	10-15
Storage Tanks	30-60
Pumps	10-15
Buildings	30-60
Electrical Systems	7-10
Transmission Mains	35-40
Distribution Pipes	35-40
Valves	35-40
Blow-off Valves	35-40
Backflow Prevention	35-40
Meters	10-15
Service Lines	30-50
Hydrants	40-60
Lab/Monitoring Equipment	5-7
Tools and Shop Equipment	10-15
Landscaping/Grading	40-60
Office Furniture/Supplies	10
Computers	5
Transportation Equipment	10

Note: These numbers are ranges of expected useful lives drawn from a variety of sources. The ranges assume that assets have been properly maintained.

Where very little asset history is documented, age can be used identify replacement expenditures for future budgets. Table 10-1 provides estimated useful life of typical water system assets and is provided in a guide to asset management for small systems provided by the USEPA[10]. The priority of pipe and hydrant replacement was determined using the methods presented in Chapter 8.0. Replacement of other assets such as pump station equipment, wells and well pumps, electrical systems, computer systems, building components, and storage tanks will primarily be based on the asset age along with consideration from operations staff on asset condition and performance.

Water Main

Projects identified for the water main replacement category were determined through the hydraulic modeling and risk analysis process in Chapter 8.0. The identified projects typically consist of water main replacements in sizes from 6-inch to 12-inch diameter. The proposed water main replacements are critical to maintain both the existing and future levels of service. Figure 10-3 shows color coded pipeline replacement projects, pipe risk and priority of the projects.

[10] Asset Management: A Handbook for Small Water Systems. (2003). United State Environmental Protection Agency Office of Water

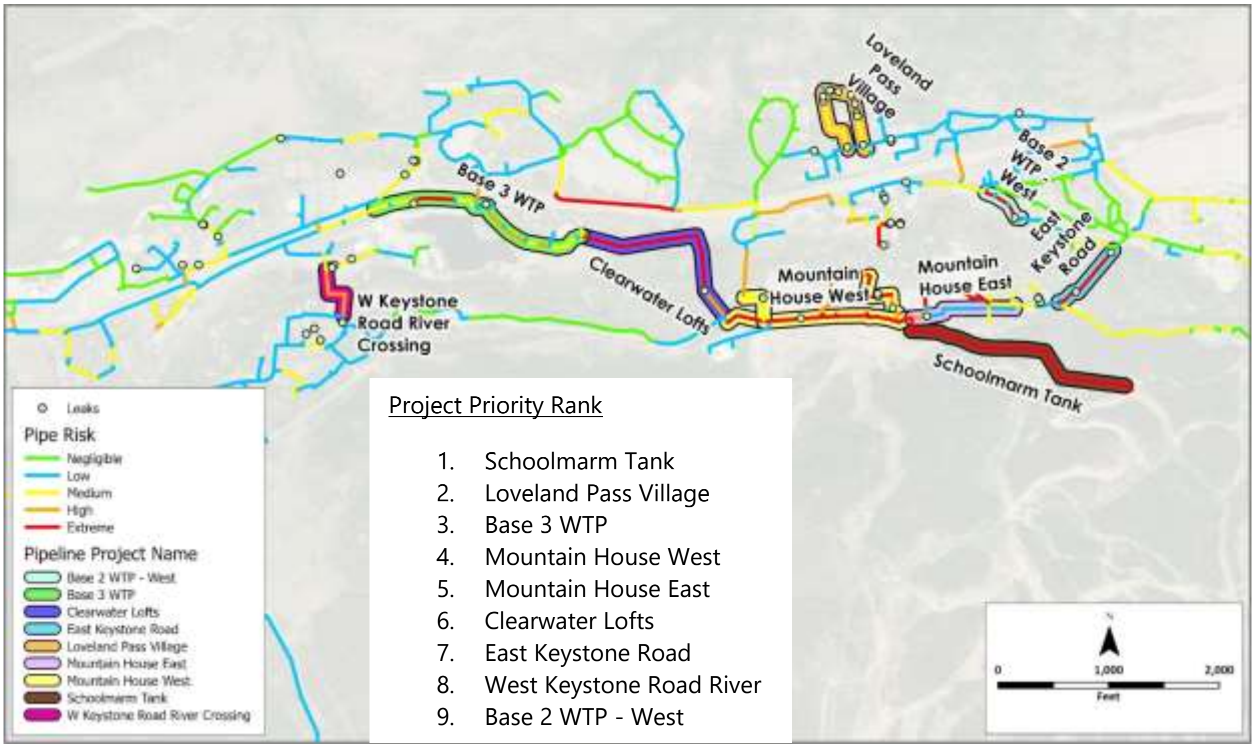


Figure 10-3 – Water Main Replacement Projects

Schoolmarm Tank: The replacement of the pipe feeding Schoolmarm Tank is the top priority water main replacement project since this pipe connects the single tank in Base 2 pressure zone. This area is prone to land movement so new pipe with restrained joints is recommended.

Loveland Pass Village: An individual pipe replacement project on Rasor Dr. (Loveland Pass Village) remains from the 2012 master plan. This area has existing 4-inch pipe that limits the fire flow availability in the area. Additionally, the 4-inch size does not meet the CDPHE criteria for minimum pipe size in systems providing fire flow and the pipe age and leak history results in a high likelihood of failure in the area.

Base 3 WTP: Replacement of water main near the Base 3 WTP is the third priority project. Multiple leaks on pipe in this area allows visual inspection of the pipe; the pipe appears to be in poor condition in this area.

Mountain House and Clearwater Lofts: These pipes are higher aged pipe feeding critical facilities in the District. There have also been multiple leaks on these pipes leading to high likelihood of failure. The area also has potential for major developments of large residential and commercial facilities similar to the River Run Area.

River Crossings: The river crossings on East and West Keystone Road are priority project in the next ten-years to replace aging infrastructure in hard to construct areas.

Hydrant

The existing hydrant assets were reviewed for age, proximity to existing facility and work required criteria from the GIS data to develop a priority list. There are currently 49 hydrants in the system whose age is within the useful life range. Using the middle useful life range of 50 years it is recommended to replace six hydrants per year for the next 10-years. A hydrant replacement program project was created in the capital improvement plan for this purpose.

An average annual expenditure of $164,400 for hydrant replacements is included in the capital improvement planning budget from 2022 to 2030. The 2021 budget was adjusted to reflect current budgeting.

Storage Tank Rehabilitation

The District has three concrete and one steel water storage tank. Steel water storage tanks are typically coated to prevent rust and require coating replacement every 10-15 years. The Pilot Lode tank is the steel tank in the system and was inspected by divers during this master planning project. The operations staff relayed that the interior coatings within the steel tank need replacement based on the recent inspection in fall of 2020. A project to replace coatings in Pilot Lode tank was added to the capital improvement plan with an estimated cost of $539,000.

Concrete tanks may need point and patch repair were concrete spalling has occurred and crack injection where significant cracks are found. Spalling can expose concrete reinforcing steel which will rust and cause further spalling and concrete deterioration. The extent of repair depends on the using proper construction methods and concrete material quality during construction. The operations staff did not indicate any immediate concerns of concrete tank condition.

PRV

The District has three pressure reducing valves in the system that are used to transfer water between pressure zones. These valves are typically operated only during high flow events such as fire flow scenarios. A modest budget was included for general maintenance of these valves in the capital improvement plan.

Well Rehabilitation and Pump Replacement.

The newest well in the District's system is approximately 25 years old. The District did not provide any well rehabilitation or inspection records during the master plan, however, did express a desire to invest in the existing wells. Additionally, the District desired to improve well pump capacities at the Base 3 WTP as the system curve changed with the addition of the pressure filters lowering the capacity of the existing wells. Investment into the wells in Base 2 should be contingent upon the GWUDI evaluation as the system curve may also change if treatment improvements are construction Additionally, documenting beneficial use of water may change conditional water rights to absolute water rights strengthening the District's water rights position.

Full scale well rehabilitation is typically suggested when the specific capacity of the well declines by 15-20%. Any further drop in specific yield may result in rehabilitation efforts only providing a recovery of 80% of the original specific capacity. Reduction in specific yield occurs over time with fine sediment migrating through soil plugging up the well screen and surrounding soil while mineral scaling can build up on the well screen.

Removing the existing pump provides better access and more options for well rehabilitation. Full removal of the pump for initial video surveillance to determine appropriate rehabilitation methods is recommended. Rehabilitation methods include chemical cleaning, backflushing, air surging, brushing, jetting, and other mechanical methods.

The estimated project costs of the Base 3 well rehabilitation is $460,000 in 2023 dollars. The Base 3 wells could be taken offline one by one to allow the treatment plant to continue production at a limited capacity. Therefore, the cost of this project may be spread out over three years. The rehabilitation of the Base 2 wells is included in the Base 2 WTP GWUDI treatment project.

SCADA and Telemetry Equipment

SCADA and telemetry equipment have a typical life of approximately 10 years. Many times, existing servers, radios, and other components will be "sunset" or no longer supported by the manufacturer. This can result in vulnerabilities as security assessments and patches are no longer provided for system software. A project budget of $317,000 was included in 2025 dollars to budget for contingent replacement of these components throughout the system.

10.4 Annual CIP Budget

This section will review multiple options and alternatives for capital improvement planning. A baseline capital improvement plan was developed for the draft master plan. After review by District staff and board members, three alternate options were developed to reduce the capital expenditures over the next 10-years. The options are based on the reclassification of the base 2 source water which should be determined by end of 2021.

10.4.1 Baseline Capital Improvement Plan

The results of the new and replacement infrastructure projects identified in the previous sections are reviewed in this section. Figure 10-4 shows the annual budgets as a result of the project estimates and anticipated years for construction. The largest years of expenditures occur in 2024 and 2026. Figure 10-5 shows the annual budget by project type; construction of a new Base 3 to Base 2 BPS increases the 2022 budget, construction of the Base 2 WTP to meet GWUDI compliance drives the large expenditures in 2023 and 2024 while construction of a new Base 2 storage tank drives the large 2026 budget. The replacement of the 4-inch pipe along Rasor Dr. increases the water main budget in 2024.

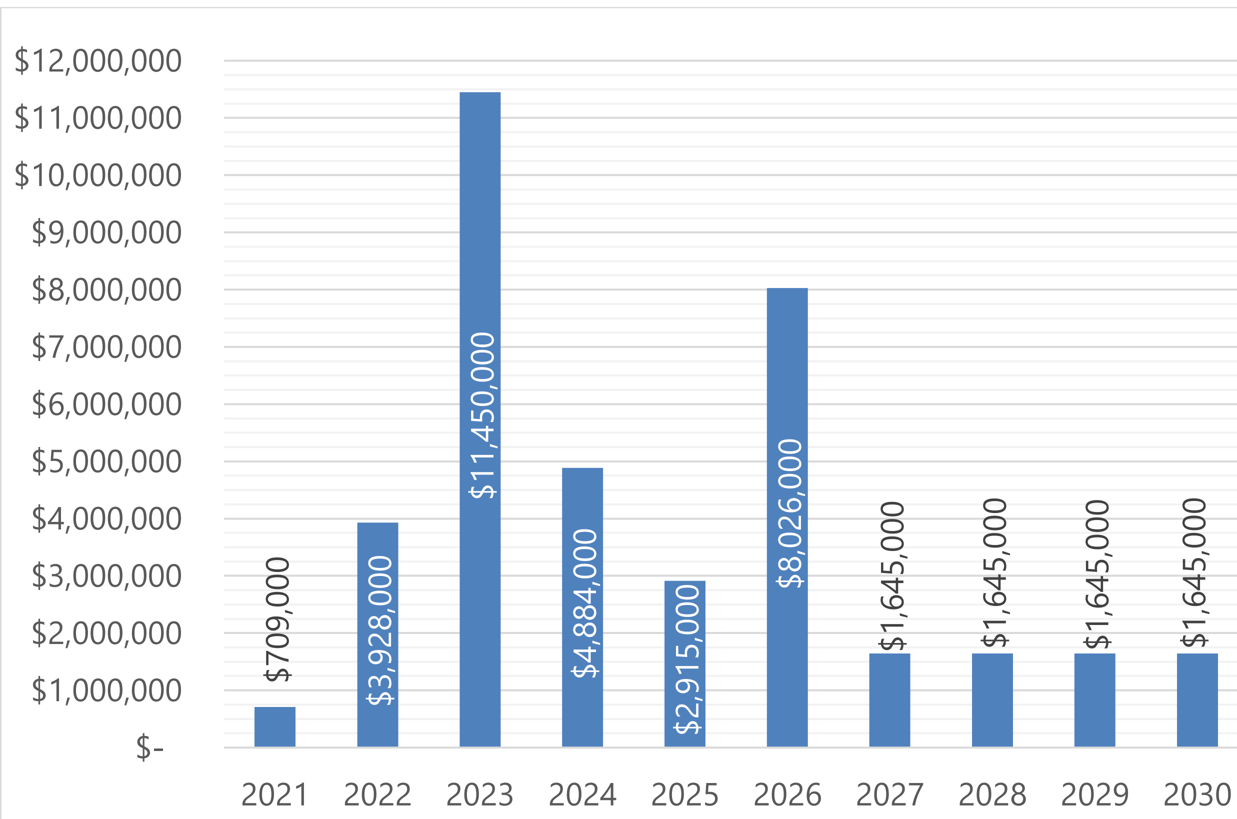


Figure 10-4 – CIP Annual Budget

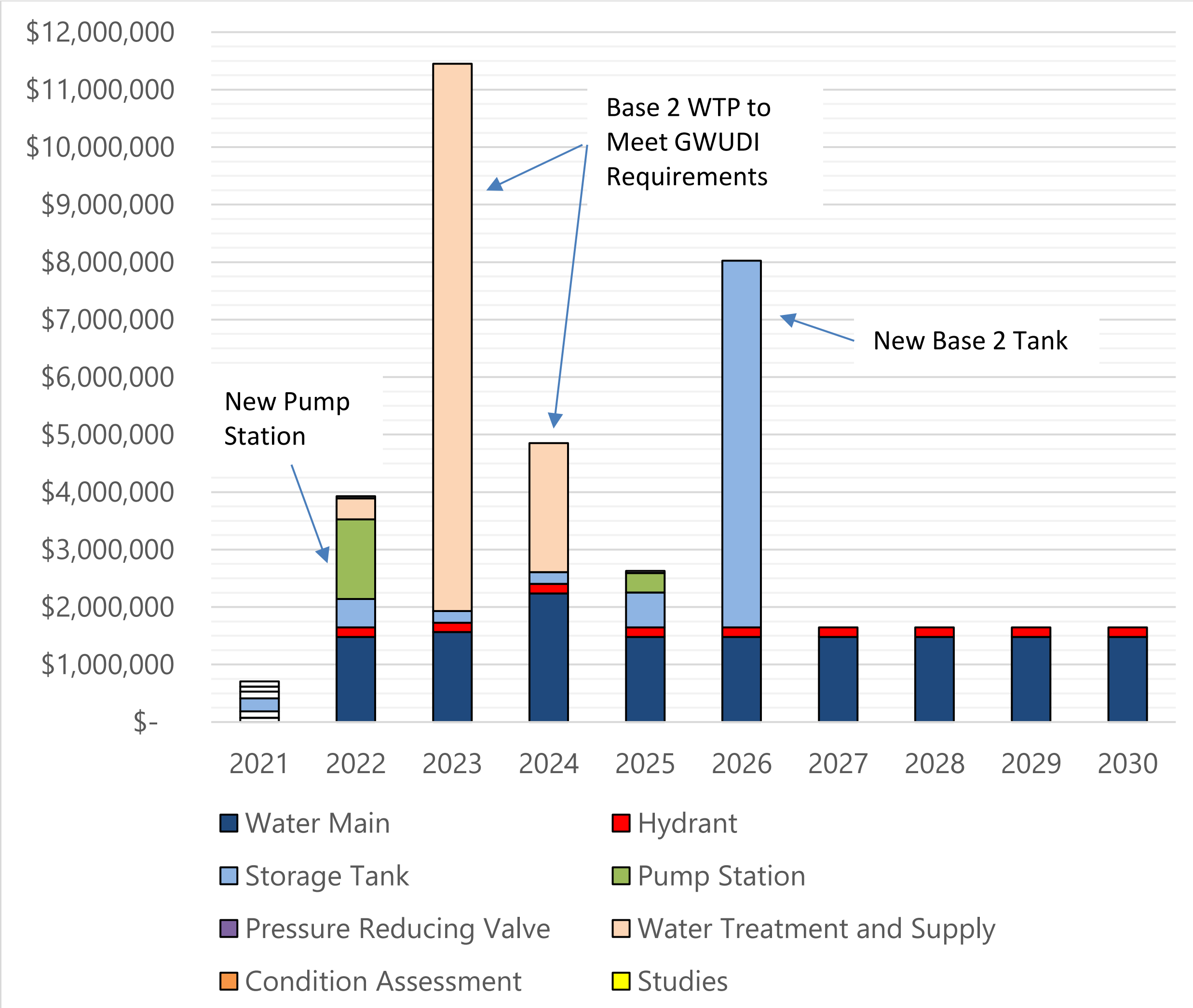


Figure 10-5 – Annual Budget by Project Type

The project names, estimate cost, year and annual budget by project are shown in Table 10-2. Many of the new infrastructure projects will have small budgets in one year followed by a larger budget representing design costs then construction costs, respectively. The hydrant and pipeline replacement programs are evenly distributed throughout each year, though these can be increased or decreased yearly depending upon project scope and financial availability.

Table 10-2 – Baseline Annual Budget by Project

Project Number	Project Name	Category	Reason	Anticipated Year	Estimated Cost	FY 2021	FY 2022	FY 2023	FY 2024	FY 2025	FY 2026	FY 2027	FY 2028	FY 2029	FY 2030
BPS-01	Base 3 to Base 2 Pump Station	Pump Station	Cond. / Risk Assessment	2022	$1,504,000	$120,320	$1,383,680								
BPS-02	Base 1 BPS Pump and Electrical Replacement	Pump Station	Cond. / Risk Assessment	2024	$341,000					$341,000					
H-01	Hydrant Replacement Program	Hydrant	Cond. / Risk Assessment	-	$1,591,000	$111,370	$164,403	$164,403	$164,403	$164,403	$164,403	$164,403	$164,403	$164,403	$164,403
M-01	Razor Drive Pipe Size Increase	Water Main	Existing Deficiency	2024	$843,000			$84,300	$758,700						
M-02	Pipeline Replacement Program	Water Main	Cond. / Risk Assessment	-	$13,399,000	$75,000	$1,480,444	$1,480,444	$1,480,444	$1,480,444	$1,480,444	$1,480,444	$1,480,444	$1,480,444	$1,480,444
PRV-01	PRV Maintenance	Pressure Reducing Valve	Cond. / Risk Assessment	2025	$38,000					$38,000					
S-01	GWUDI Evaluation of Base II Wells	Studies	Regulation	2021	$46,000	$46,000									
S-02	Revised Lead and Copper Rule Response	Studies	Regulation	2022	$48,000	$12,000	$36,000								
S-03	AWIA Compliance	Studies	Regulation	2021	$34,000	$34,000									
SC-01	SCADA and Telemetry Replacements	SCADA and Telemetry	Cond. / Risk Assessment	2025	$317,000				$31,700	$285,300					
SUP-01	Base 2 WTP Soda Ash Feed Alternatives	Water Treatment and Supply	Optimization	2021	$25,000	$25,000									
SUP-02	Base 3 WTP Well Pump Replacements	Water Treatment and Supply	Cond. / Risk Assessment	2023	$460,000	$59,800	$133,400	$133,400	$133,400						
SUP-03	Base 2 WTP GWUDI Requirements	Water Treatment and Supply	Regulation	2024	$11,792,000		$234,580	$9,383,200	$2,111,220						
T-01	New Base 2 Storage Tank	Storage Tank	Cond. / Risk Assessment	2026	$7,578,000	$90,936	$90,936	$204,606	$204,606	$606,240	$6,380,676				
T-02	Pilot Lode Tank Recoating	Storage Tank	Cond. / Risk Assessment	2021	$539,000	$134,750	$404,250								
				TOTAL	**$38,492,000**	**$709,000**	**$3,928,000**	**$11,450,000**	**$4,884,000**	**$2,915,000**	**$8,062,000**	**$1,645,000**	**$1,645,000**	**$1,645,000**	**$1,645,000**

10.4.2 Option 1 – No GWUDI Reclassification

The baseline capital improvement plan in the previous section is aggressive in both completing the three new infrastructure project and pipeline replacement over the next ten years. This option presents a capital improvement plan which:

- Does not require major treatment upgrades at Base 2 WTP as the source water remains classified as groundwater.
- Reduces investment into hydrant replacements through 2030.
    - From 60 replacements in baseline to 33 replacements in Option 1.
- Moves the new Base 2 storage tank to later years of the plan.
- Reduces investment into pipeline replacement through 2030.
    - From 16,150 LF in baseline to 12,600 LF in Option 1
- Invests in replacement Base 2 WTP well rehabilitation and pumps, finished water pumps, and other equipment since major treatment improvements are not required.

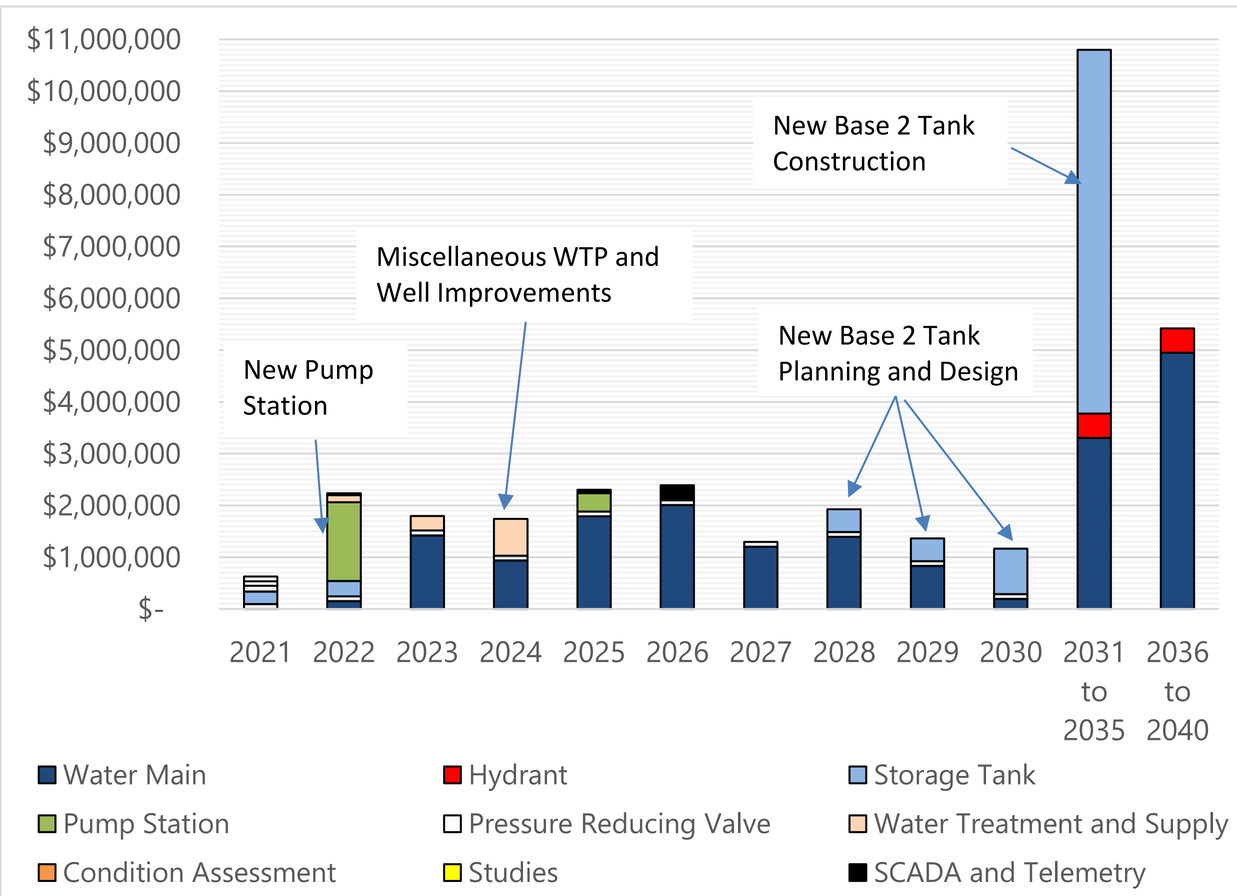


Figure 10-6 – CIP Option 1 Annual Budget by Project Type

10.4.3 Option 2 – GWUDI Reclassification

Option 2 is used to prioritize treatment improvements as a response to source water reclassification. The major changes of option 2 compared to the baseline plan include:

- Prioritizes investment into treatment improvements at the Base 2 WTP but uses reduced filtration capacity compared to the baseline plan. Capacity expansion is designed into the initial plant to reduce initials costs with expansion estimated for 2026.
- Reduces investment into hydrant replacements through 2030.
  - From 60 replacements in baseline to 33 replacements in Option 2.
- Reduces investment into pipeline replacement through 2030.
  - From 16,150 LF in baseline to 9,500 LF in Option 1
- Moves Base 2 storage tank into later years of the plan.

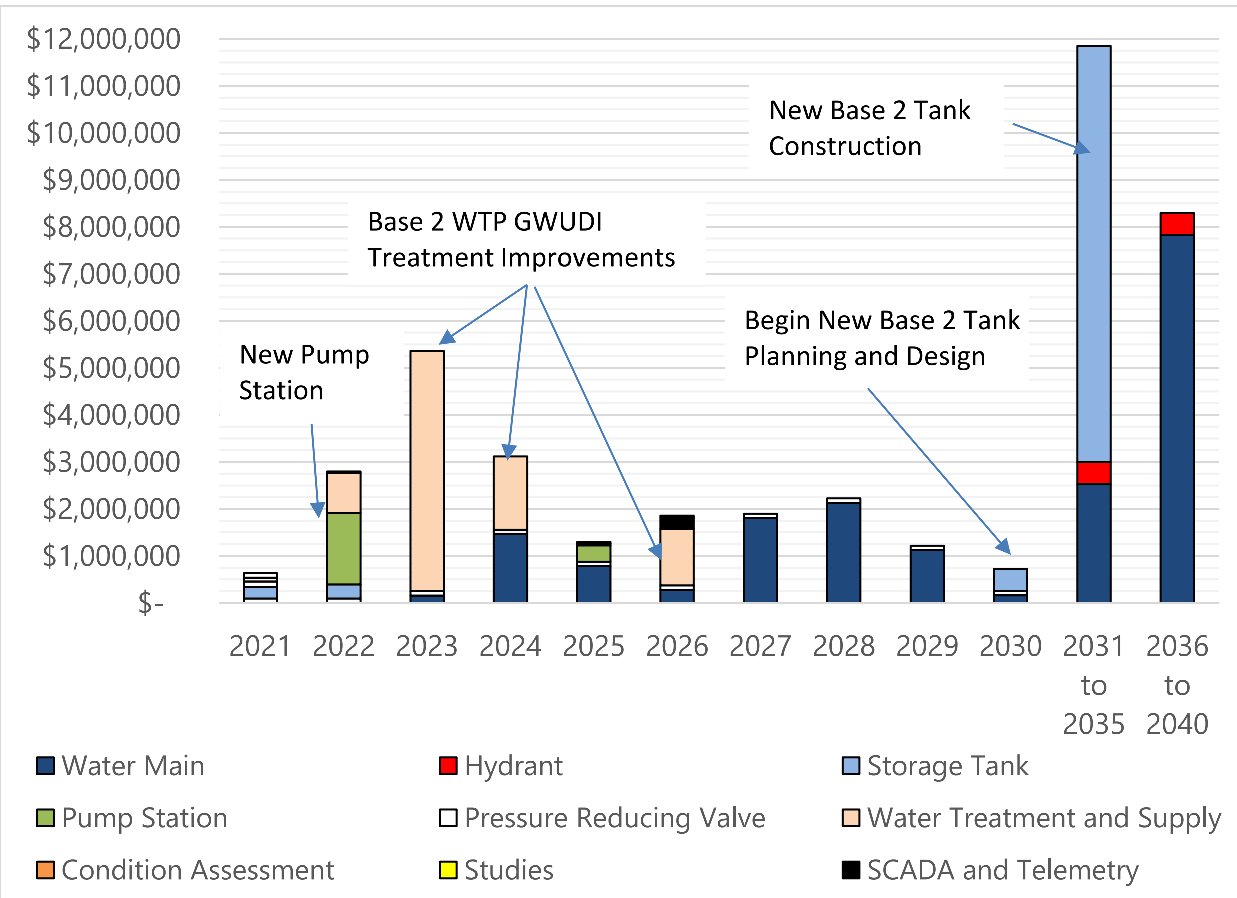


Figure 10-7 – CIP Option 2 Annual Budget by Project Type

10.4.4 Option 3 – GWUDI Reclassification with Delayed Treatment

Option 3 delays treatment improvements for GWUDI classified water and attempted to reclassify the source water as groundwater through various options. These options are included in Table 10-3.

Table 10-3 – GWUDI Mitigation Option

Option	Description	Potential Issues
Relocate Wells $1,000,000	Move the wells further away from surface water to eliminate GWUDI determination.	This may open the water rights and subject them to review which is not recommended.
Reduce Well Capacity $400,000	Reducing the flow through the well may reduce the influence of surface water so the wells remain classified as groundwater source.	Results are not guaranteed with this effort.
Reinstall Well and Casing $1,200,000	Construct new wells within the permitted area and screen interval deeper if possible	Results are not guaranteed, and deeper wells with sufficient may not be possible depending upon bedrock and geology.
Develop Keybase Well $2,450,000	Use water rights and develop the Keybase well to its full capacity. Well water will need to be treated at the well or sent to Base 3 WTP.	This is a costly alternative and the water rights are designated for use in the Base 3 area.

The major changes to the CIP for option 3 compared to the baseline include:

- Reduces investment into hydrant replacements through 2030.
  - From 60 replacements in baseline to 33 replacements in Option 3.
- Reduces investment into pipeline replacement through 2030.
  - From 16,150 LF in baseline to 12,000 LF in Option 3
- Moves Base 2 storage tank into later years of the plan.
- Anticipates a GWUDI compliant treatment plant constructed in the 2030's

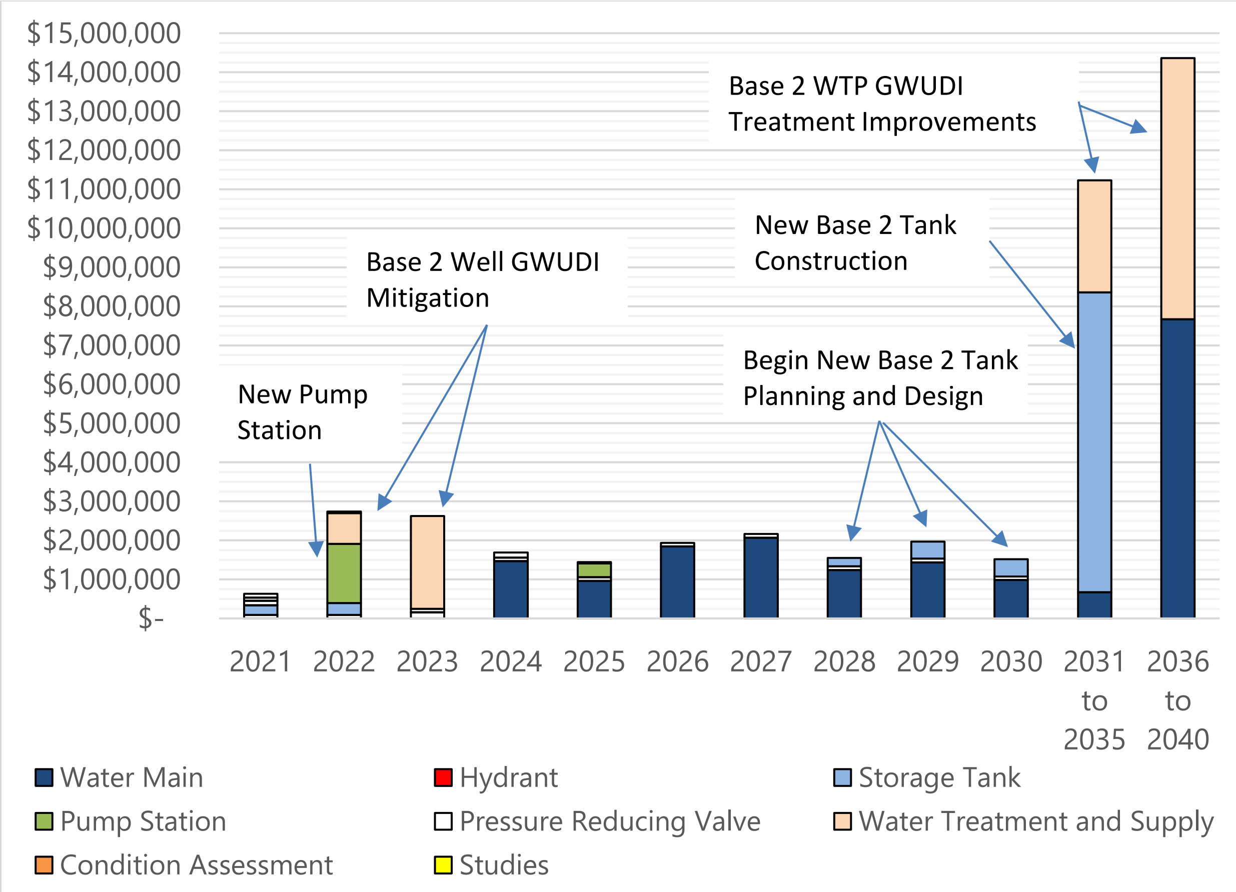


Figure 10-8 – CIP Option 3 Annual Budget by Project Type

Table 10-4 through Table 10-6 show the projects in each option that are different from the baseline CIP. Individual pipeline replacement projects are identified, and the year of construction varies between options. Projects from the baseline CIP that will remain regardless of option are not provided in the tables but included in the total values of the bottom rows.

Table 10-4 – CIP Option 1 Annual Budget by Project

Project Number	Project Name	Category	Reason	Year	Estimated Cost	FY 2021	FY 2022	FY 2023	FY 2024	FY 2025	FY 2026	FY 2027	FY 2028	FY 2029	FY 2030	2031 to 2035	Beyond 2036
H-01	Hydrant Replacement Program	Hydrant	Cond. / Risk Assessment	-	$942,000	$94,200	$94,200	$94,200	$94,200	$94,200	$94,200	$94,200	$94,200	$94,200	$94,200	$471,000	$471,000
M-02	Schoolmarm Tank Pipe	Water Main	Cond. / Risk Assessment	2023	$1,486,000		$148,600	$1,337,400									
M-03	Razor Drive Pipe Size Increase	Water Main	Existing Deficiency	2024	$843,000			$84,300	$758,700								
M-04	Base 3 WTP	Water Main	Cond. / Risk Assessment	2025	$ 1,755,000				$175,500	$1,579,500							
M-05	Mountain House Area - West Side	Water Main	Cond. / Risk Assessment	2026	$2,100,000					$210,000	$1,890,000						
M-06	Mountain House Area - East Side	Water Main	Cond. / Risk Assessment	2027	$1,176,000						$117,600	$1,058,400					
M-07	Clearwater Lofts	Water Main	Cond. / Risk Assessment	2028	$1,448,000							$144,800	$1,303,200				
M-08	East Keystone Road	Water Main	Cond. / Risk Assessment	2029	$906,000								$90,600	$815,400			
M-09	W Keystone Road River Crossing	Water Main	Cond. / Risk Assessment	2030	$141,000									$14,100	$126,900		
M-10	Base 2 WTP - West Pipe	Water Main	Cond. / Risk Assessment	2031	$651,000										$65,100	$585,900	
M-11	Remaining Extreme Risk Pipe	Water Main	Cond. / Risk Assessment	2037	$2,716,000											$2,716,000	
M-12	Remaining High Risk Pipe	Water Main	Cond. / Risk Assessment	2040	$4,950,000												$4,950,000
SUP-03	Base 2 WTP Pump Replacements	Water Treatment and Supply	Cond. / Risk Assessment	2024	$723,000			$144,600	$578,400								
T-01	New Base 2 Storage Tank	Storage Tank	Existing Deficiency	2031	$8,785,000								$439,250	$439,250	$878,500	$7,028,000	
ANNUAL TOTAL						$628,000	$2,233,000	$1,794,000	$1,740,000	$2,304,000	$2,387,000	$1,297,000	$1,927,000	$1,363,000	$1,165,000	$10,801,000	$5,421,000
CUMULATIVE TOTAL							$2,861,000	$4,655,000	$6,395,000	$8,699,000	$11,086,000	$12,383,000	$14,310,000	$15,673,000	$16,838,000	$27,639,000	$33,060,000

The following projects are not shown but included in the annual and cumulative total values: Base 3 to Base 2 Pump Station, Base 1 Pump and Electrical Replacement, PRV Maintenance, GWUDI Evaluation of Base 2 Wells, Revised Lead and Copper Rule Compliance, AWIA Compliance, SCADA and Telemetry Replacements, Base 2 WTP Soda Ash Feed Alternatives, Base 3 WTP Well Pump Replacements, Pilot Lode Tank Recoating

Table 10-5 – CIP Option 2 Annual Budget by Project

Project Number	Project Name	Category	Reason	Year	Estimated Cost	FY 2021	FY 2022	FY 2023	FY 2024	FY 2025	FY 2026	FY 2027	FY 2028	FY 2029	FY 2030	2031 to 2035	Beyond 2036
H-01	Hydrant Replacement Program	Hydrant	Cond. / Risk Assessment	-	$942,000	$94,200	$94,200	$94,200	$94,200	$94,200	$94,200	$94,200	$94,200	$94,200	$94,200	$471,000	$471,000
M-02	Schoolmarm Tank Pipe	Water Main	Cond. / Risk Assessment	2024	$1,531,000			$153,100	$1,377,900								
M-03	Razor Drive Pipe Size Increase	Water Main	Existing Deficiency	2025	$868,000				$86,800	$781,200							
M-04	Base 3 WTP	Water Main	Cond. / Risk Assessment	2027	$1,862,000						$279,300	$1,582,700					
M-05	Mountain House Area - West Side	Water Main	Cond. / Risk Assessment	2028	$2,228,000							$222,800	$2,005,200				
M-06	Mountain House Area - East Side	Water Main	Cond. / Risk Assessment	2029	$1,248,000								$124,800	$1,123,200			
M-07	Clearwater Lofts	Water Main	Cond. / Risk Assessment	2031	$1,583,000										$158,300	$1,503,850	
M-08	East Keystone Road	Water Main	Cond. / Risk Assessment	2033	$1,020,000											$1,020,000	
M-09	W Keystone Road River Crossing	Water Main	Cond. / Risk Assessment	2035	$163,000												$163,000
M-10	Base 2 WTP - West Pipe	Water Main	Cond. / Risk Assessment	2035	$755,000												$755,000
M-11	Remaining Extreme Risk Pipe	Water Main	Cond. / Risk Assessment	2040	$2,716,000												$2,716,000
M-12	Remaining High Risk Pipe	Water Main	Cond. / Risk Assessment	2024	$4,950,000												$4,950,000
SUP-03A	Base 2 WTP GWUDI 1 MGD Partial	Water Treatment and Supply	Regulation	2023	$7,116,000		$711,600	$4,981,200	$1,423,200		$1,200,000						
T-01	New Base 2 Storage Tank	Storage Tank	Cond. / Risk Assessment	2033	$9,320,000										$466,000	$8,854,000	
					ANNUAL TOTAL	$628,000	$2,796,000	$5,362,000	$3,116,000	$1,296,000	$1,859,000	$1,900,000	$2,224,000	$1,217,000	$719,000	$11,849,000	$9,055,000
					CUMULATIVE TOTAL		$3,424,000	$8,786,000	$11,902,000	$13,198,000	$15,057,000	$16,957,000	$19,181,000	$20,398,000	$21,117,000	$32,966,000	$42,021,000

The following projects are not shown but included in the annual and cumulative total values: Base 3 to Base 2 Pump Station, Base 1 Pump and Electrical Replacement, PRV Maintenance, GWUDI Evaluation of Base 2 Wells, Revised Lead and Copper Rule Compliance, AWIA Compliance, SCADA and Telemetry Replacements, Base 2 WTP Soda Ash Feed Alternatives, Base 3 WTP Well Pump Replacements, Pilot Lode Tank Recoating

Table 10-6 – CIP Option 3 Annual Budget by Project

Project Number	Project Name	Category	Reason	Year	Estimated Cost	FY 2021	FY 2022	FY 2023	FY 2024	FY 2025	FY 2026	FY 2027	FY 2028	FY 2029	FY 2030	2031 to 2035	Beyond 2036
H-01	Hydrant Replacement Program	Hydrant	Cond. / Risk Assessment	-	$942,000	$94,200	$94,200	$94,200	$94,200	$94,200	$94,200	$94,200	$94,200	$94,200	$94,200	$471,000	$471,000
M-02	Schoolmarm Tank Pipe	Water Main	Cond. / Risk Assessment	2024	$1,531,000			$153,100	$1,377,900								
M-03	Razor Drive Pipe Size Increase	Water Main	Existing Deficiency	2025	$868,000				$86,800	$781,200							
M-04	Base 3 WTP	Water Main	Cond. / Risk Assessment	2026	$1,808,000					$180,800	$1,627,200						
M-05	Mountain House Area - West Side	Water Main	Cond. / Risk Assessment	2027	$2,163,000						$216,300	$1,946,700					
M-06	Mountain House Area - East Side	Water Main	Cond. / Risk Assessment	2028	$1,211,000							$121,100	$1,089,900				
M-07	Clearwater Lofts	Water Main	Cond. / Risk Assessment	2029	$1,492,000								$149,200	$1,342,800			
M-08	East Keystone Road	Water Main	Cond. / Risk Assessment	2030	$934,000									$93,400	$840,600		
M-09	W Keystone Road River Crossing	Water Main	Cond. / Risk Assessment	2030	$141,000										$141,000		
M-10	Base 2 WTP - West Pipe	Water Main	Cond. / Risk Assessment	2032	$671,000											$671,000	
M-11	Remaining Extreme Risk Pipe	Water Main	Cond. / Risk Assessment	2037	$2,716,000												$2,716,000
M-12	Remaining High Risk Pipe	Water Main	Cond. / Risk Assessment	2040	$4,950,000												$4,950,000
SUP-03A	Base 2 WTP GWUDI 1 MGD Partial	Water Treatment and Supply	Regulation	2033	$9,564,000											$2,869,200	$6,694,800
SUP-04	Owner's Well No. 2 and 4 Rehab	Water Treatment and Supply	Regulation	2022	$317,000		$317,000										
SUP-05	New OW3 and SRWD No. 1 Wells	Water Treatment and Supply	Regulation	2023	$1,159,000		$231,800	$927,200									
SUP-06	VFDs for OW3 and SRWD Well 1	Water Treatment and Supply	Regulation	2023	$395,000			$395,000									
SUP-09	Base 2 WTP Finished Water Pumps	Water Treatment and Supply	Cond. / Risk Assessment	2023	$617,000		$61,700	$555,300									
T-01	New Base 2 Storage Tank	Storage Tank	Cond. / Risk Assessment	2031	$8,785,000								$219,625	$439,250	$439,250	$7,686,875	
ANNUAL TOTAL						$628,000	$2,695,000	$2,258,000	$1,692,000	$1,477,000	$2,223,000	$2,162,000	$1,553,000	$1,970,000	$1,515,000	$11,698,000	$14,832,000
CUMULATIVE TOTAL							$3,323,000	$5,581,000	$7,273,000	$8,750,000	$10,973,000	$13,135,000	$14,688,000	$16,658,000	$18,173,000	$29,871,000	$44,703,000

The following projects are not shown but included in the annual and cumulative total values: Base 3 to Base 2 Pump Station, Base 1 Pump and Electrical Replacement, PRV Maintenance, GWUDI Evaluation of Base 2 Wells, Revised Lead and Copper Rule Compliance, AWIA Compliance, SCADA and Telemetry Replacements, Base 2 WTP Soda Ash Feed Alternatives, Base 3 WTP Well Pump Replacements, Pilot Lode Tank Recoating

10.5 Opinion of Probable Project Cost for CIP Development

10.5.1 Estimate Classification

The American Association of Cost Engineers (AACE) provides guidelines for applying the general principles of estimate classification to project cost estimates (i.e., cost estimates that are used to evaluate, approve, and/or fund projects). The purpose for following a classification process is to align the level of estimating with the use of the information. The estimates provided in the Master Plan are classified in accordance with the criteria established by the AACE cost estimating classification system referred to as Standard Practice 18R-97.

In accordance with AACE criteria, the OPPC values are representative of Class 4 estimates. A Class 4 estimate is defined as a study or feasibility estimate. Typically, the engineering effort is from 1 to 15 percent complete. Class 4 estimates are used to prepare planning-level effort cost scopes or complete an evaluation of alternative schemes, technical feasibility, and preliminary budget approval or approval to proceed to the next stage of implementation.

Expected accuracy for Class 4 estimates typically range from -30 to +50 percent, depending on the technical complexity of the project, appropriate reference information, and the inclusion of an appropriate contingency determination. Ranges could exceed those shown in unusual circumstances.

10.5.2 Opinion of Probable Project Costs Basis

The OPPC values were based on the total capital investment necessary to complete a project from engineering design through construction. All estimates are based on engineering experience and judgment, recent bid tabulations for projects of similar scope, and input from area contractors and material suppliers. All costs are estimated in 2020 dollars then inflation is added for each CIP project based on the estimated year it will be bid or constructed.

Total estimated project costs were divided into two main components, as follows:

- Hard Costs – The actual physical construction of the project (i.e., excavation, materials, labor, restoration).
- Soft Costs – Fees that are not directly related to labor and building materials (i.e., architecture and engineering fees, subsurface utility engineering (SUE) reports for pipeline projects, permitting/environmental, contract administration, legal, property acquisition, contingencies, and inflation).

The sum of these two components is the total OPPC. The OPPC values are based on the preliminary concepts and layouts of the water system components developed as a result of the hydraulic modeling of the system, risk analysis and corresponding recommendations. The estimate is to be an indication of fair market value and is not necessarily a reflection of the lowest bid. Fair market value is assumed to be mid-range tender considering four or more competitive bids.

10.5.3 Hard Costs

Unit prices were developed for hard costs which are typically measured and contracted on a unit price basis. These items include pipe per linear foot, borings per linear foot, service connections, replacement hydrants, and connections to existing main. Prices for projects on vertical infrastructure such as storage tanks, pump stations and treatment facilities were developed on a case by case basis.

Table 10-7 – Summary of Hard Costs for Project Estimates

Cost Items	Assumptions
Paved Transmission Mains	• Earthwork ○ Trench depth of 9 feet to 10 feet to the top of pipe ○ Utility bedding for pipe and compaction of bedding in the trench ○ Full depth import backfill and compaction • Fire hydrant every 300 feet. • Two isolation valves every 300 feet (one on each side of hydrant). • Two fittings every 1,000 feet (on average). • Air release / vacuum valves every 1,000 feet (on average) • Asphalt pavement surface restoration of existing paved areas. • Hydroseeding surface restoration of unpaved areas.
Unpaved Transmission Mains	• Similar to paved transmission mains except for pavement restoration costs

Other Transmission Main Items	• Additional Pavement Restoration Cost for Colorado Department of Transportation (CDOT) Highway = $13.12 per linear foot ○ This cost is associated with increased thickness of asphalt, base, and subbase plus higher-grade asphalt mix for one square yard of restoration per linear foot of main in projects within CDOT paved right-of-way. • Hydrant and Guard Valve Replacement = $15,000 each • ¾-inch Residential Water Service Installation = $1,600 each • ¾-inch Residential Water Service Installation with Meter Pit = $3,000 each. • Water Main Connections of proposed transmission main to other mains in the system.

Markups to the hard costs to cover contractor mobilization, erosion control requirements and construction surveying and material testing are calculated as a percentage of the hard costs.

- Mobilization/demobilization/insurance/permits/bonds – 0-8 percent

Mobilization costs include the administrative costs and expenses to mobilize materials, equipment, and labor to the jobsite and demobilize upon project completion. Costs associated with contractor insurance, permits, and bonding are also included.

- Traffic Control – 0-5 percent

Traffic control was assigned to projects that occur in the public right-of-way, primarily transmission main projects, where traffic control is required.

- Erosion Control – 0-3 percent

Erosion control will likely be required for all construction projects to ensure compliance with Storm Water Pollution Prevention Plans. Projects that disturb over 1 acre of area are required to submit an erosion control plan and obtain a permit for the state's regulating agency.

- Testing and Construction Surveying – 0-5 percent

Costs associated with materials testing during construction in addition to construction surveying and staking. Common material testing for water system projects typically includes backfill compaction testing, pressure and disinfection testing, and concrete quality testing. Other specialized testing may be required for specific projects.

10.5.4 Soft Costs

Table 10-8 provides a summary of the soft costs for improvement projects. Engineering, construction administration and management, and legal costs are based on a percentage of the hard costs while contingency and inflation are based on the sum of hard and soft costs. A mountain factor percentage was added to account for labor and housing costs in the region, material availability and other factors of mountainous terrain that may cause increased project costs.

Table 10-8 – Summary of Soft Costs for Project Estimates

Cost Items	Assumptions
Soft Cost Markups	• Engineering Design – 0-20 percent of Hard Costs • Construction Administration and Management – 0-10 percent of Hard Costs • Legal and Administrative Cost – 0-5 percent of Hard Costs
Property Acquisition Costs	Property acquisition costs are associated with purchasing property and acquiring right-of-way or easements for the project. Costs normally consist of payments to landowners. This was appropriate for most of the identified CIP projects anticipated to be built outside of right-of-way.
Contingency	A contingency is an amount added to the base cost to cover both identified and unidentified risk events that occur on the project. Depending on the project type, the contingency values ranged from 10 to 30 percent. The contingency values were added to the overall project base cost (i.e., hard and soft costs) in anticipation of uncertainties inherent to the planning-level analysis completed for the Master Plan.
Inflation	Projects intended for construction several years in the future include a factor for inflationary impacts to address the general trend of cost indices, which accounts for future labor, material, and equipment cost increases beyond values at the time the estimate is prepared. For this planning-level analysis, the 2019 project costs were inflated to the construction year anticipated for each CIP project. An annual average inflation rate was generated based on historic inflation data to estimate inflation trends into the future. The construction cost index provided by Engineering News Record was reviewed and the average annual increase in the Denver area was 1.75%. An inflation of 3.0% was used as resort mountain areas have experienced higher inflation than the Denver area.
Mountain Factor	A mountain factor was added to projects due to a limited skilled labor pool, high housing costs for construction workers, limited material availability and other unknown factors of construction in the mountain terrain. This mountain factor markup is a percentage of the hard and soft costs for the project.

Summary of Estimate Markups

Table 10-9 provides a summary of the suggested hard costs markups, soft costs, and contingency rate percentages.

Table 10-9 – Total Estimate Project Markup Summary

Item	Rate Range (%)
Hard Cost Markups	
Mobilization/Demobilization/Insurance/Permits/Bonds	0-8
Traffic Control	0-5
Erosion Control	0-3
Testing and Construction Surveying	0-5
Soft Costs	
Engineering Design	0-20
Construction Administration and Management	0-10
Legal and Administrative	0-5
Other	
Property Acquisition	Unit Price
Contingency	10-30
Estimated Annual Inflation	3.0
Mountain Factor	10-20

10.5.5 Estimating Exclusions

Unless specifically identified, the following was excluded in the development of the cost estimates:

- Environmental mitigation of hazardous materials and/or disposal.
- O&M costs for the project components.

10.5.6 Opinion of Probable Cost (OPPC) Sheets

Opinion of probable costs for each project identified in the chapter are included in **Error! Reference source not found.**. The sheets identify the hard costs, soft costs, and other markup items for each project.