Potable Water Master Plan

Vista Irrigation District

Committed to Supplying High Quality Water in an Economically and Environmentally Responsible Way

April 9, 2018

Contents

Tables

Figures

Appendices

Abbreviations and Acronyms

AAD	average annual demand
AF, AFY	acre feet, acre feet per year
CAFR	Comprehensive Annual Financial Report
cfs	cubic feet per second
CARS	Condition Assessment Ratings System
CIP	capital improvement program
District, VID	Vista Irrigation District
EPS	extended period simulation
ESP	Emergency Storage Project
EVWTP	Escondido-Vista Water Treatment Plant
FF	fire flow
fps	feet per second
FY	fiscal year
gdb	geodatabase
GIS	geographic information system
gpd, gpm	gallons per day, gallons per minute
GV	gate valve
HDPE	high density polyethlylene
MDD, MDD+FF	maximum day demand, maximum day demand plus fire flow
MinDD	minimum day demand
MG, MGD	million gallons, million gallons per day
MWD	Metropolitan Water District of Southern California
PHD	peak hour demand
PRS	pressure regulating/reducing station
PS	pump station
psi	pounds per square inch
SANDAG	San Diego Association of Governments
SCADA	supervisory control and data acquisition
TM	technical memorandum
UWMP	Urban Water Management Plan
VWD	Vallecitos Water District
Water Authority	San Diego County Water Authority
Weese	Robert A. Weese
WFP	Water Filtration Plant
WRF	Water Reclamation Facility
WTP	Water Treatment Plant

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Executive Summary

The purpose of this Potable Water Master Plan is to provide a comprehensive review of the Vista Irrigation District's potable water supply and distribution system and develop a structured program to identify system improvements necessary to meet existing and future demand conditions. System improvements are identified through a condition assessment of existing facilities and distribution system hydraulic analyses. This effort includes an updated and calibrated hydraulic model that accurately reflects the current distribution system demands and operating parameters.

Service Area and Water Demands

The District's service area encompasses property within the City of Vista, the City of San Marcos, and the County of San Diego. Each of these agencies has adopted a General Plan document that is incorporated into a regional planning database. This database is utilized in this Master Plan for understanding water usage based on land-use and developing unit demand factors for estimating future water demands.

The District's historical water use has varied significantly over the past 30 years, reaching a peak in 2004, with current demands dropping below those seen in 1986. The downward trends over the past 10 years can be attributed to a number of factors ranging from economics, weather, adoption of increased water conservation measures, and mandated restrictions. Due to these factors, the build-out demand projection in this Master Plan is 25 percent less than that estimated in the 2000 Master Plan; and as a result, very little expansion based projects are identified and the Capital Improvement Program instead focuses on system reliability and redundancy, in addition to pipeline replacements.

Water Supply Reliability

The District maintains capacity rights from two sources, raw water treated at the Escondido-Vista Water Treatment Plant located at Lake Dixon and multiple treated water connections along the San Diego County Water Authority's aqueducts. Due to reduced costs, the District typically maximizes the locally treated water supply and relies on the 11-mile Vista Flume for conveyance into the District. During a planned 10-day shutdown along the Second Aqueduct, the District is dependent on the Vista Flume. With the Flume approaching its useful life, this Master Plan reviews and outlines a number of recommended alternative projects for further study that can add redundancy, reliability, and operational flexibility to offset the Flume being out of service either short term or long term.

Pipeline Condition Assessment and Replacement Strategy

A detailed pipeline condition assessment is presented in this Master Plan that provides an overall system risk assessment along with several investment scenarios that estimate how various funding levels will impact future service levels. This assessment provides a tool for the District to strike the appropriate balance between affordability and sustaining desired service levels and also focus those investments to ensure ratepayers realize the greatest return on their investment.

Reservoir Condition Assessment

Condition assessment inspections of 10 of the District's 12 potable water reservoirs were completed to document the current condition of the civil site, corrosion, and structural aspects of the reservoirs. The findings of the inspection of the District's reservoirs were used to recommend and prioritize improvements for the rehabilitation or replacement of reservoir equipment and identify any additional assessments required.

Capital Improvement Program

An updated Capital Improvement Program has been developed based on redundancy or replacement and rehabilitation improvements for the existing distribution system and an ultimate system based on projected buildout demands. The recommended projects are shown in **Figure ES-1**, and estimated costs are provided in **Table ES-1**.

Figure ES-1. Recommended Capital Improvement Projects

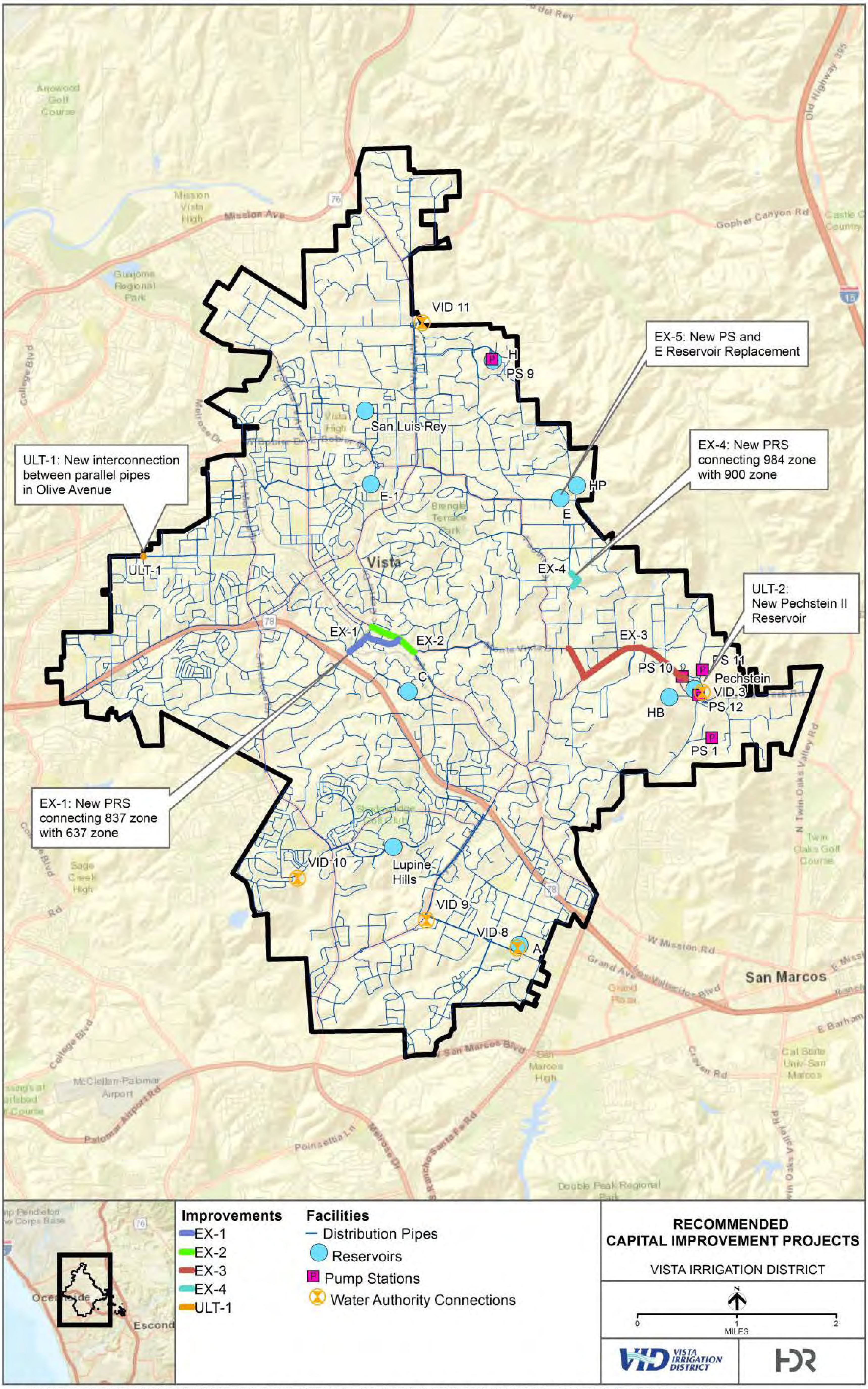

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Table ES-1. Probable Cost Opinion for Recommended Capital Improvement Program Projects

Project Number	Type	Description	Unit (Linear Feet unless otherwise specified)	Size	Unit Cost ($/Unit)	Low Range CIP Cost ($)	High Range CIP Cost (S)
EX-1	PRS	Construct new 637 zone PRS along Civic Center Drive	1 PRS	1,000 gpm peak flow	250,000	250,000	250,000
	Pipeline	New 12-inch pipe in Postal way from E43 PRS to Civic Center Drive and southwest down Civic Center Drive to new 637 PRS	3,211	12-inch	300	963,300	963,300
	Pipeline	Parallel 8-inch pipe in Civic Center Drive from new 637 zone PRS to Phillips Street	241	8-inch	250	60,250	60,250
EX-2	Pipeline	Parallel 12-inch pipe in South Santa Fe Avenue from Monte Vista Drive to E43 PRS and continuing to Civic Center Drive	2,665	12-inch	300	799,500	799,500
	PRS	Upsize E43 PRS	1 PRS	1,200 gpm peak flow	250,000	250,000	250,000
EX-3	Pipeline	New 30-inch pipe from Pechstein Reservoir to PS 10	645	30-inch	700	451,500	451,500
	Pipeline	New 24-inch pipe parallel to existing 26-inch pipe from PS 10 to Sugarbush Drive parallel to Buena Creek Road	3,386	24-inch	560	1,896,160	1,896,160
	Pipeline	New 24-inch pipe in Buena Creek Road from Sugarbush Drive to Monte Vista Drive	3,126	24-inch	560	1,750,560	1,750,560
	Pipeline	New 24-inch pipe replacing existing 12- and 10-inch pipe in Monte Vista Drive from Buena Creek Road to La Rueda Drive	1,759	24-inch	560	985,040	985,040

Table ES-1. Probable Cost Opinion for Recommended Capital Improvement Program Projects

Project Number	Type	Description	Unit (Linear Feet unless otherwise specified)	Size	Unit Cost ($/Unit)	Low Range CIP Cost ($)	High Range CIP Cost (S)
EX-4	PRS	Construct new PRS connecting 976/984 zone and 900 zone between San Clemente Way and Huntalas Lane	1 PRS	600 gpm	250,000	250,000	250,000
	Pipeline	New 8-inch pipe connecting 976/984 zone and 900 zone via new 900 PRS	1,006	8-inch	250	251,500	251,500
EX-5	PS	New PS at E Reservoir	1 PS	2000 to 7,000 gpm (3 to 10 MGD)	1 million/ MGD	3,000,000	10,000,000
	E Reservoir Replacement	Replace Existing E Reservoir, at same location	1 Reservoir	2 to 4 MG	1.50 to 1.25 per MG	3,000,000	5,000,000
	Pipeline	New pipe connecting E Reservoir PS to 976/984 zone	1,000	16 to 24-inch	400 560	400,000	560,000
ULT-1	Pipeline and Valve	Installation of 10-inch -diameter interconnection between 8-inch and 12-inch parallel pipes in Olive Avenue at the intersection of Grapevine Road	40 1 Valve	10 inch	280 5,000 per valve	11,200 5,000	11,200 5,000
ULT-2	New Pechstein II Reservoir	Construct new Pechstein II Reservoir adjacent to Pechstein Reservoir on District owned land	1 Reservoir	4 to 20 MG	1.25 to 1.00 per MG	5,000,000	20,000,000
Total Cost of Recommended Improvement Projects (Rounded to nearest $1,000)						19,324,000	43,484,000

EX - Existing System Improvement; ULT – Ultimate System Improvement; gpm - gallons per minute; CIP – capital improvement program; MG - million gallons; MGD - million gallons per day; PRS - pressure regulating/reducing station; PS – pump station

1 Introduction

This report presents the findings and recommendations of the Potable Water Master Plan (Master Plan) for the Vista Irrigation District (VID or District). The report provides a comprehensive review of the potable water supply and distribution system and identifies capital improvements necessary to adapt to current and future conditions while providing reliable and economical service to the existing customers.

1.1 District Overview

The District encompasses a service area of over 21,000 acres in North San Diego County, approximately 30 miles north of downtown San Diego. It serves water to the entire City of Vista, unincorporated areas of the County of San Diego and a small portion of the City of San Marcos. These municipal boundaries and the District service area are shown on the Vicinity Map in **Figure 1-1.** The service area is comprised of a variety of land uses including residential, commercial, industrial, agricultural, and open space.

Surrounding water agencies, shown on **Figure 1-2**, include the City of Oceanside to the west, Vallecitos Water District (VWD) to the east and south, the Carlsbad Municipal Water District to the southwest and Rainbow Municipal Water District to the north. The Twin Oaks Valley area, commonly referred to as the “Boot Area” and the “Bennett Area,” is located east of the main service area. These areas fall within the VWD sphere of influence but are currently being served by the District.

The District has both local and imported water supplies. It receives imported water supply from the San Diego County Water Authority (Water Authority) aqueducts, which imports water from northern California and the Colorado River, via the Metropolitan Water District of Southern California (MWD), to serve San Diego County. In 2015, the Water Authority began operating a desalination plant in Carlsbad that provides approximately 10 percent of the County’s water supply, offsetting the region’s reliance on imported water. Treated water from the Water Authority is distributed directly from the aqueducts into the District’s water system at six locations.

Local water is derived from precipitation in the San Luis Rey River Watershed and stored in Lake Henshaw. This water is treated at the Escondido-Vista Water Treatment Plant (EVWTP) and conveyed via the Vista Flume to the Twin Oaks Valley area and the District’s Pechstein Reservoir for distribution.

The District currently serves more than 129,000 people through approximately 28,500 metered residential and business connections. A total of 14,375 acre feet (AF), or nearly 4.7 billion gallons, of water was distributed and sold within the District in fiscal year (FY) 2016. Of that amount, 70 percent was distributed for residential use, 12 percent for industrial and commercial, 10 percent for landscape irrigation, 6 percent for agriculture and 2 percent for governmental use. The District’s distribution system includes 12 reservoirs, with a total capacity of over 46 million gallons (MG), 7 pump stations, and 429 miles of pipeline, ranging in diameter from 4-inch to 42-inch.

Figure 1-1. Vicinity Map

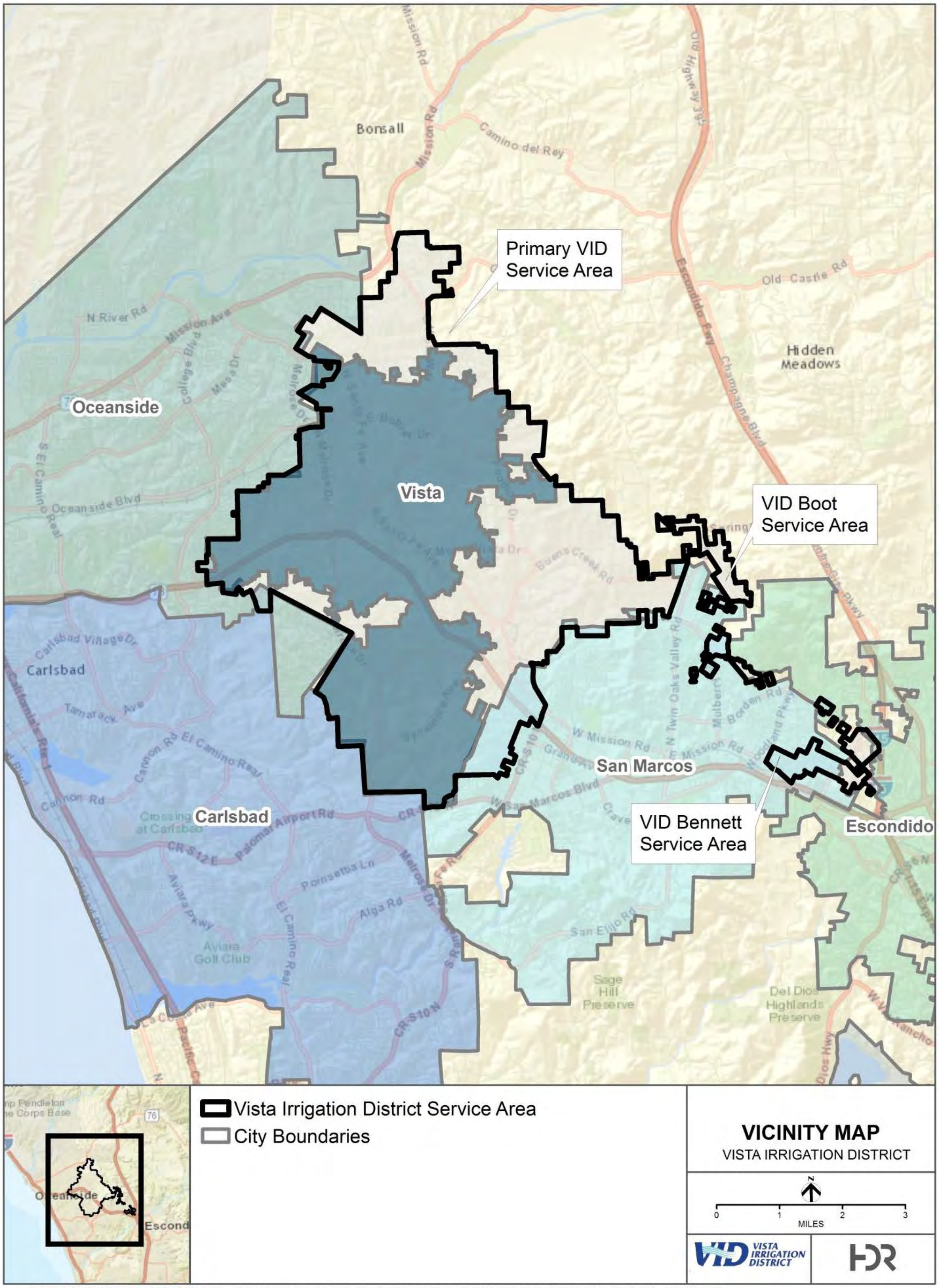
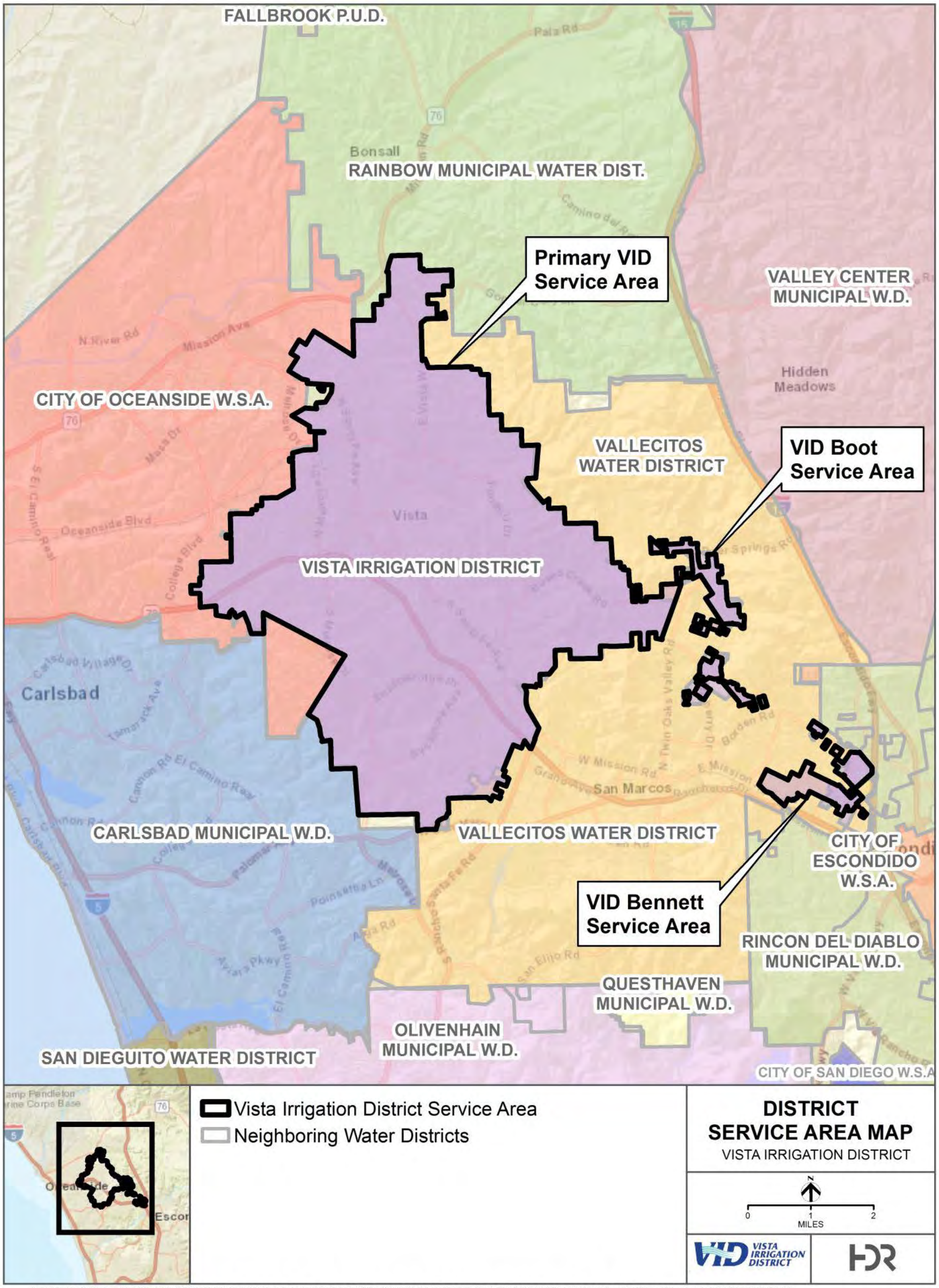


Figure 1-2. District Service Area Map



1.2 Previous Water Master Plans

Various studies were conducted throughout the 1970's and 1980's to determine peaking factors, diurnal water use patterns, the sufficiency of the District's distribution system, and adequate sizing of new facilities. The District has maintained a computer hydraulic model of the water system since the late 1970's. This model was used to develop the 1995 Potable Water Master Plan, which was adopted by the District's Board of Directors and outlined a capital improvement program (CIP) to accommodate land use development and future water demands.

The Master Plan was updated in 2000 to revise water demand projections, evaluate the potential for recycled water use, re-evaluate design criteria, and update the hydraulic model using H2ONet Version 3. The 2000 Master Plan developed a CIP for both the existing system and ultimate buildout facilities, based on the current land use plans.

1.3 Purpose of the Master Plan Update

The District contracted with HDR to prepare an updated Potable Water Master Plan based on information available representing the District's system circa 2017. This Master Plan includes an updated hydraulic model in the Innovyze InfoWater software platform that reflects the current geographic information system (GIS) information and operating parameters of the potable water distribution facilities. Future water demand projections are based on adopted land uses, as defined in the City of Vista 2030 General Plan and the current land use plans of other jurisdictions within the District's service area boundaries.

This Master Plan also establishes a structured program of system improvements. CIP projects are identified through condition assessment of existing facilities and a hydraulic modeling and capacity analysis under existing and future conditions.

1.4 Report Organization

This Master Plan is organized as follows:

- **Chapter 2** provides an overview of the District's topography and climatological setting, as well as current and future land use plans.
- **Chapter 3** reviews the District's water demand history and projects future water demand based on current water use patterns and future land use plans.
- **Chapter 4** provides a description of the District's water supply sources.
- **Chapter 5** outlines planning and design criteria for the analysis of the distribution system.
- **Chapters 6** through **8** provide a description of the existing facilities, the development of the new hydraulic model and the analysis of the existing and future water distribution system capacity to meet demands.
- **Chapter 9** provides a summary of recommended improvements and a CIP with associated planning level opinions of cost.

2 Service Area Description

The layout and sizing of potable water system facilities are greatly influenced by topography, climate and land use. Topography defines system pressure requirements. Climate impacts the seasonal use of water for irrigation. Land uses and associated water demands primarily drive the distribution system layout and capacity.

This chapter discusses the topography, climate and current and future land uses of the service area.

2.1 Topography and Terrain

The western edge of the service area is approximately 5 miles inland from the Pacific Ocean. It extends east another 10 miles to the foothills of the San Marcos Mountains. Elevations range from 300 feet above sea level toward the west and up to 1,000 feet above sea level to the east. Most of the businesses are located in the flatter areas in the center of the service area, and residences populate the surrounding hillsides. In undeveloped areas, the natural vegetation types include chaparral brushland, oak-sycamore woodland, riparian-woodland, and oak-grass savanna.

2.2 Climate

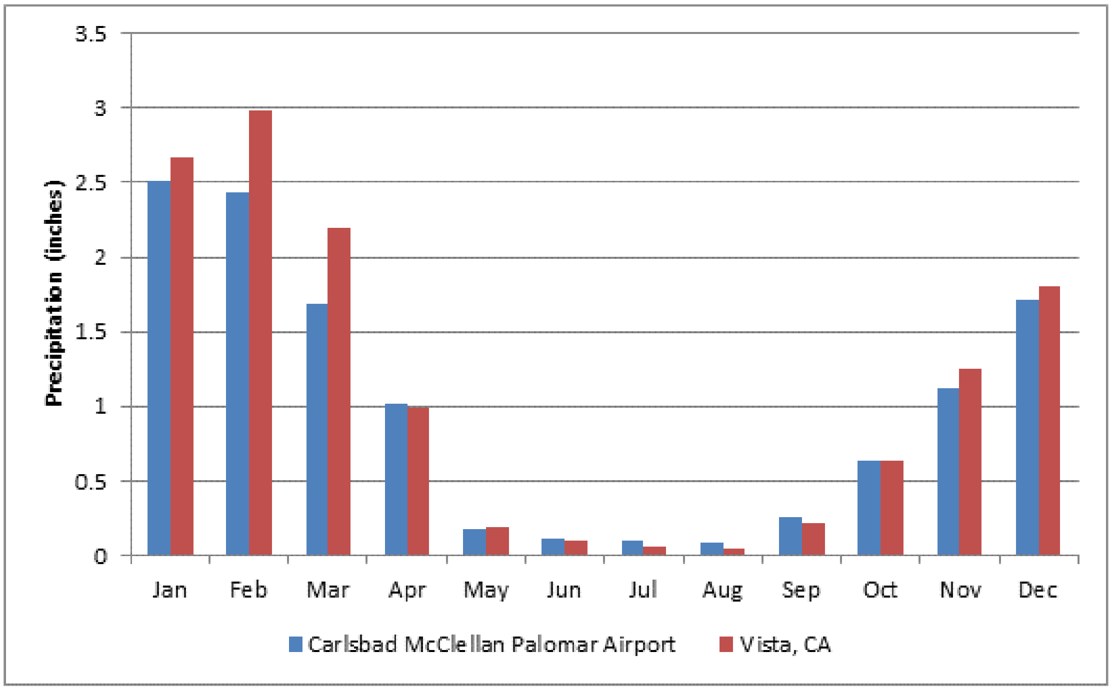
The climate in the District's service area is mild, varying from a low mean daytime temperature of 69 degrees Fahrenheit in the winter to a high mean daytime temperature of 86 degrees Fahrenheit in the summer. The average annual rainfall for Vista is approximately 13.4 inches and occurs primarily from October through April, as shown in **Figure 2-1**. Two rain gauge sites are represented on the figure:

- The Carlsbad McClellan Palomar Airport rain gauge is located just west of the southern end of the District's service area, and;
- The Vista, California, rain gauge is located near downtown Vista.

Rainfall is higher in the San Marcos Hills on the eastern edge of the District, up to 20 inches per year. The moderate climate has made Vista and surrounding areas a center of the plant nursery industry. Under normal conditions, water demand for outdoor uses is far greater during the warm, dry summer months.

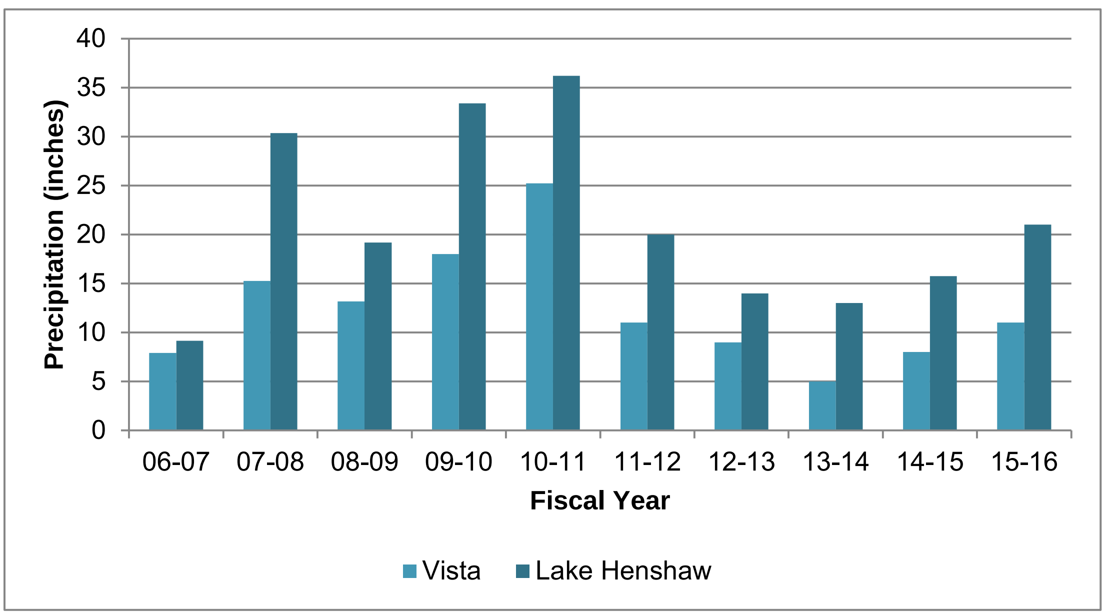
California experienced its fifth year of drought in 2015, as shown in **Figure 2-2**. Both Vista and Lake Henshaw basins, where the District draws its local water supply, exhibited below normal annual precipitation rates from 2011 through 2016. A state-wide drought emergency was declared and water conservation targets were assigned to water agencies throughout California. The District instituted mandatory water restrictions, including assigned watering days and limited irrigation run times in June 2015 to meet its reduction target of 20 percent. As the northern part of the state received much needed precipitation in the following years, water supply reservoirs began to fill.

Figure 2-1. Monthly Normal Rainfall



Source: 1981-2010 Climate Normals - National Climatic Data Center, National Oceanic and Atmospheric Administration

Figure 2-2. Local Annual Rainfall

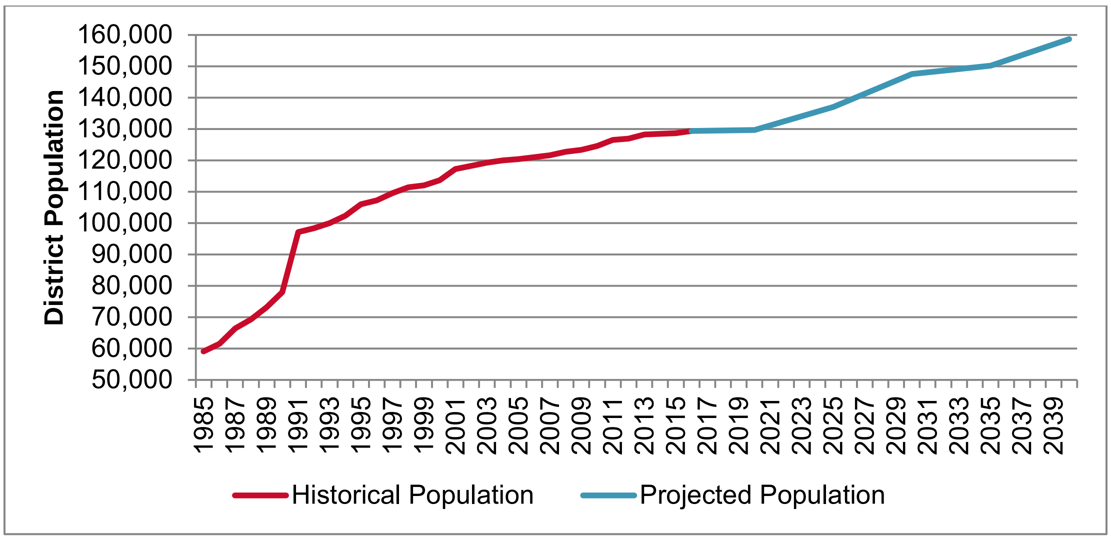


As the imported water supplies began to recover, the state revised its emergency water conservation regulations to allow water agencies to adjust their conservation targets by taking into account local climate, water efficient growth, and investments in alternative water supplies. The District was able to reduce its conservation target from 20 percent to 12 percent by taking into account the new regional supply, conveyed by the Water Authority, from the Claude "Bud" Lewis Carlsbad Desalination Plant. In June 2016, the District declared an end to mandatory water use restrictions when the state allowed for a "stress test" approach. This approach allowed urban water agencies, including the District, to determine their individual conservation target based on each agency's verifiable supplies.

2.3 Population

Population within the District's service area has increased at an average rate of 0.7 percent per year over the past 10 years, as reported in the District's 2016 Audited Comprehensive Annual Financial Report (CAFR) and shown in **Figure 2-3**. According to the San Diego Association of Governments (SANDAG) Series 13 Regional Growth Forecast, the region's population will grow at a steady rate and is expected to increase to 158,627 by 2040.

Figure 2-3. Historical and Projected Population Growth



2.4 Land Use

Land use designations are a convenient means of evaluating, organizing, and projecting water demands. For example, land use categories can be used to group current water customers with similar demand patterns for the analysis of existing system capacity. Local and regional agencies have adopted planning documents that designate the allowable types of land uses within their jurisdiction. The District's boundary encompasses property within the City of Vista, the City of San Marcos, and the County of San Diego. Each of these agencies has adopted a General Plan document that is subsequently incorporated into a regional planning database that is periodically updated.

The following documents and land use data were used for this Master Plan:

- SANDAG Series 13: 2050 Regional Growth Forecast
- City of Vista Downtown Specific Plan, dated September 2015

2.4.1 City of Vista

The City of Vista, which constitutes the majority of the District's service area, has experienced considerable growth over the past 20 years, with the addition of 24,000 new residents and construction of new industrial and commercial development. Although the City is approaching buildout, it is expected to add nearly 14,775 residents by 2030. The majority of this growth is anticipated to be accommodated by infill of vacant sites and redevelopment of underutilized sites.

The Public Safety, Facilities, and Services Element of the Vista General Plan 2030 also addresses the City's potable water supply goals, which include coordinating with the District to ensure that adequate, safe and reliable water is available to meet existing and planned needs of the community. The Public Safety, Facilities, and Services specific goals and policies associated with water supply are provided in the text box below.

Water Supply

PSFS Goal 11: Continue to ensure that the City has an adequate, safe, and reliable water supply to meet the existing and planned needs of the community.

PSFS Policy 11.1: Coordinate with the Vista Irrigation District (VID) to update its Urban Water Management Plan during the regular update cycle.

PSFS: 11.2: Coordinate with VID to conduct assessments of water supply to determine if water supplies are adequate to serve the demand generated by projects.

PSFS Policy 11.3: Promote water conservation programs and use of recycled water to reduce Vista's demand for potable water.

2.4.2 Outside the City of Vista

The area along the District's eastern boundary lies within the County of San Diego. This area consists primarily of low density residential developments, slightly increasing in densifications in areas directly adjacent to State Route 78.

The 670-acre Boot Area is located primarily in the County of San Diego, and within the City of San Marcos and VWD spheres of influence. The area is largely agricultural and residential. As this area develops, individual parcels have been and will continue to be annexed into VWD for both water and sewer service.

The 470-acre Bennett Area is located within the City of San Marcos and VWD sphere of influence, and primarily includes single family homes. It is anticipated that the individual parcels within this area will eventually be annexed into VWD for water service.

2.4.3 Summary of Land Use Data

The District's existing and future land use categories and corresponding acreages, including the Boot and Bennett areas, are listed in **Table 2-1** and are illustrated in **Figure 2-4** and **Figure 2-5**, respectively. These land uses form the basis for calculating current unit demand factors and projected water demand in **Chapter 3**.

Table 2-1. Existing and Future Land Use

SANDAG Land Use Code Designation [1]	Land Use Category [1]	Current Area (Acres)[1]	2030 Projected Area (Acres)[2]
10_ _	Rural Residential	2,854	5,060
11_ _	Single Family Residential	7,565	8,321
12_ _	Multi-Family Residential	740	858
13_ _	Mobile Home Park	326	281
14_ _	Other Group Quarters Facility	126	134
15_ _	Hotel/Motel (Low-Rise)	14	11
21_ _	Industrial	1,102	1,166
41_ _	Utilities	2,466	2,445
50_ _	Commercial	597	561
60_ _	Office	99	125
61_ _	Public Services	237	240
65_ _	Health Care	45	44
68_ _	Education	375	320
72_ _	Golf Course	184	166
76_ _	Park	1,057	1,035
80_ _	Agriculture	1,438	14
91_ _	Vacant and Undeveloped Land	1,839	-
95_ _	Under Construction	96	-
97_ _	Mixed Use	<1	377
	Total	21,158	21,158

[1] Source: San Diego Association of Governments (SANDAG) Series 13 Existing and Planned Land Use, Group Codes Represent Groupings of SANDAG Land Use Codes. Area represents values for entire service area including Boot and Bennett areas.

[2] 2030 SANDAG Projections were updated to include Downtown Vista Specific Plan Land Uses

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Figure 2-4. Existing Land Use

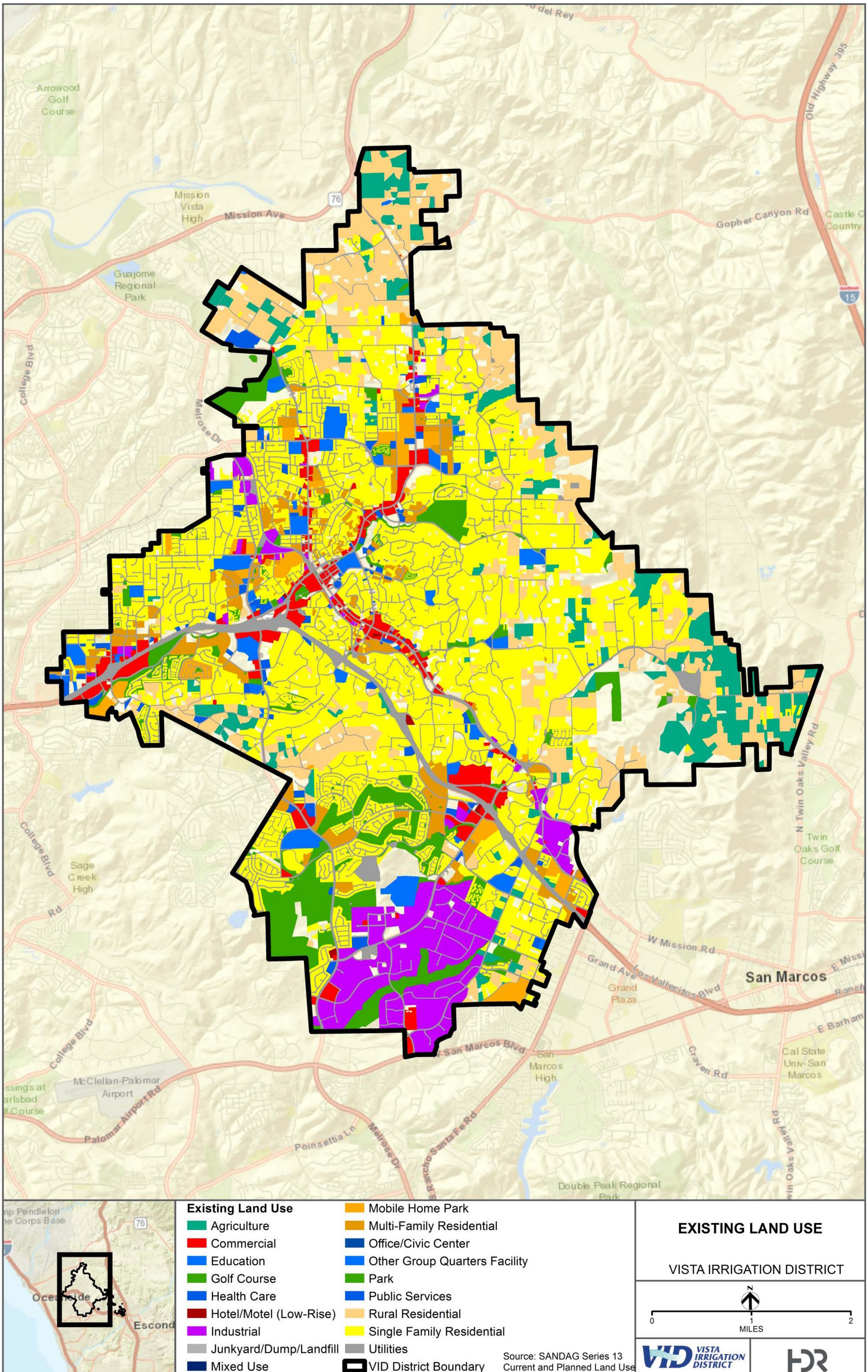

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Figure 2-5. Buildout Land Use Map 2030 General Plan

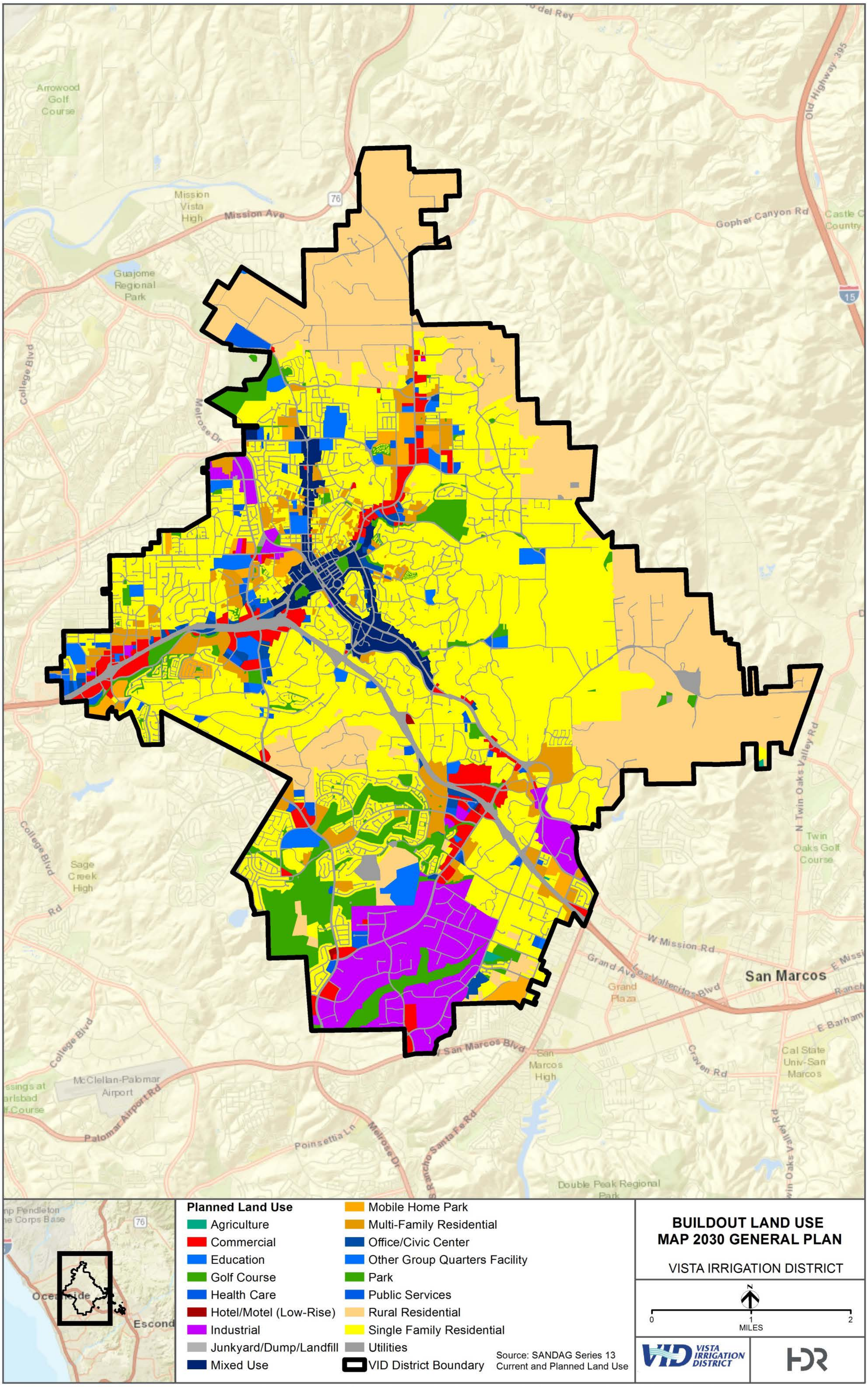

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Figure 2-6 indicates which areas within the District are anticipated to contribute to future water demands, including the Vista Downtown Specific Planning area. These areas include currently vacant parcels that are expected to be developed and areas specifically slated for redevelopment. Redevelopment in Downtown Vista is planned to include mixed use, commercial, and public service land uses.

2.5 Fire Hazard Areas

Rural residential and undeveloped areas lie within high hazard fire zones as designated by the California Department of Forestry and Fire Prevention and adopted by the Vista Fire Protection District. A hazardous fire area is defined as any geographic area mapped by the State or local jurisdiction as a high, or very high fire hazard area, or as set forth by the fire authority having jurisdiction, that contains the type and condition of vegetation, topography, weather, and structure density to potentially increase the possibility of vegetation conflagration fires shall be considered a hazardous fire area. The designated high hazard areas in the vicinity of the District's service area are shown in **Figure 2-7.**

In Ordinance 2013-23, the Vista Fire Protection District found that:

> *"The topography of the Vista Fire Protection District presents problems in delivery of emergency services, including fire protection. Hilly terrain with narrow, winding roads with poor circulation prevents rapid access and orderly evacuation. Many of these hills are covered with highly combustible natural vegetation. In addition to access and evacuation problems, the terrain makes delivery of water extremely difficult. Some hill areas are served by water pump systems subject to failure in fire, high winds, earthquake, and other power failure situations. This would only allow domestic gravity feed water from tanks, and not enough water for fire fighting."*

As such, the Vista Fire Protection District has determined that new development in wildland-urban interface fire areas will require a higher fire flow than residential areas in lesser hazardous areas. These requirements are discussed in detail in **Chapter 5**, Planning and Design Criteria.

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Figure 2-6. Future Densification Areas

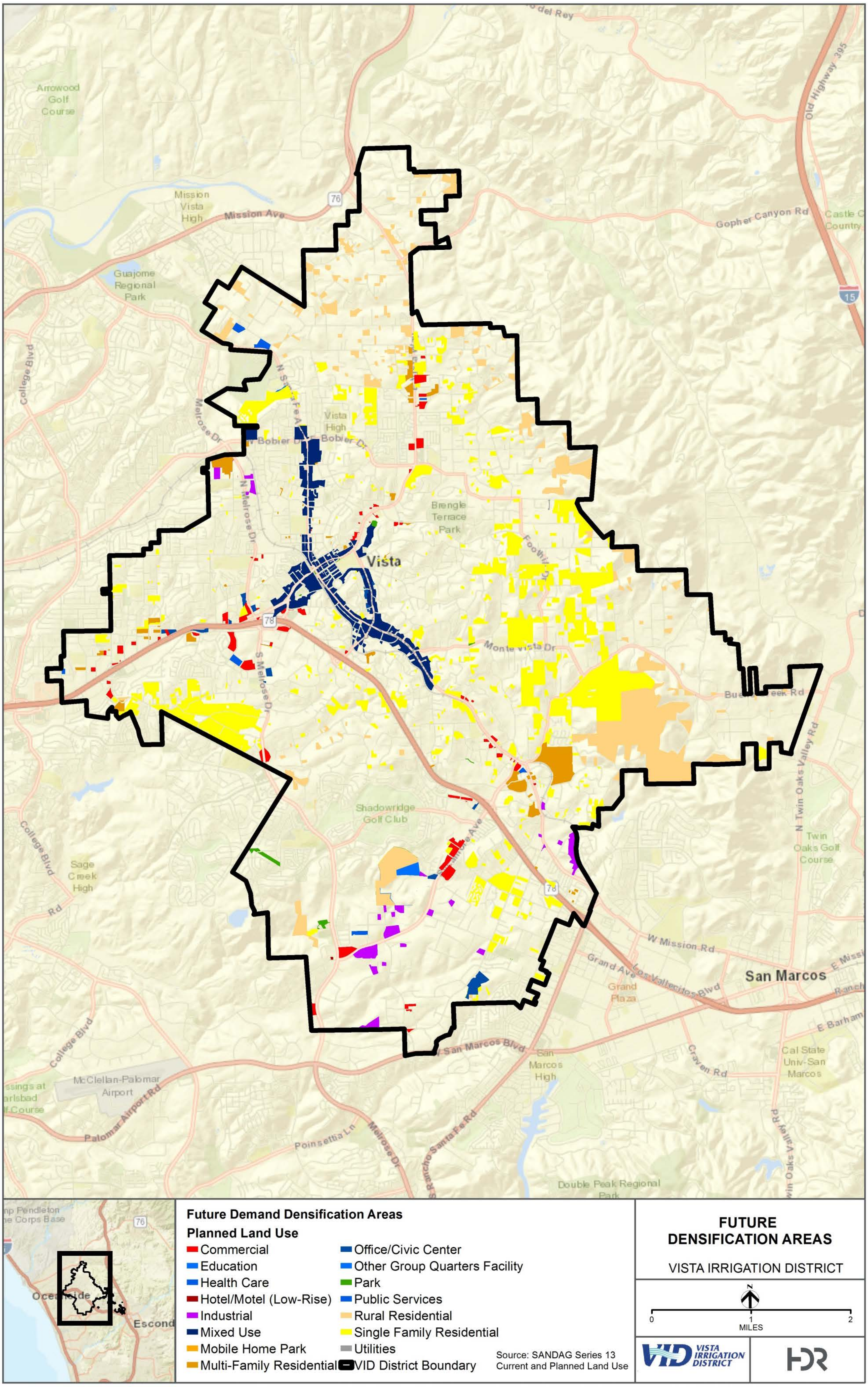

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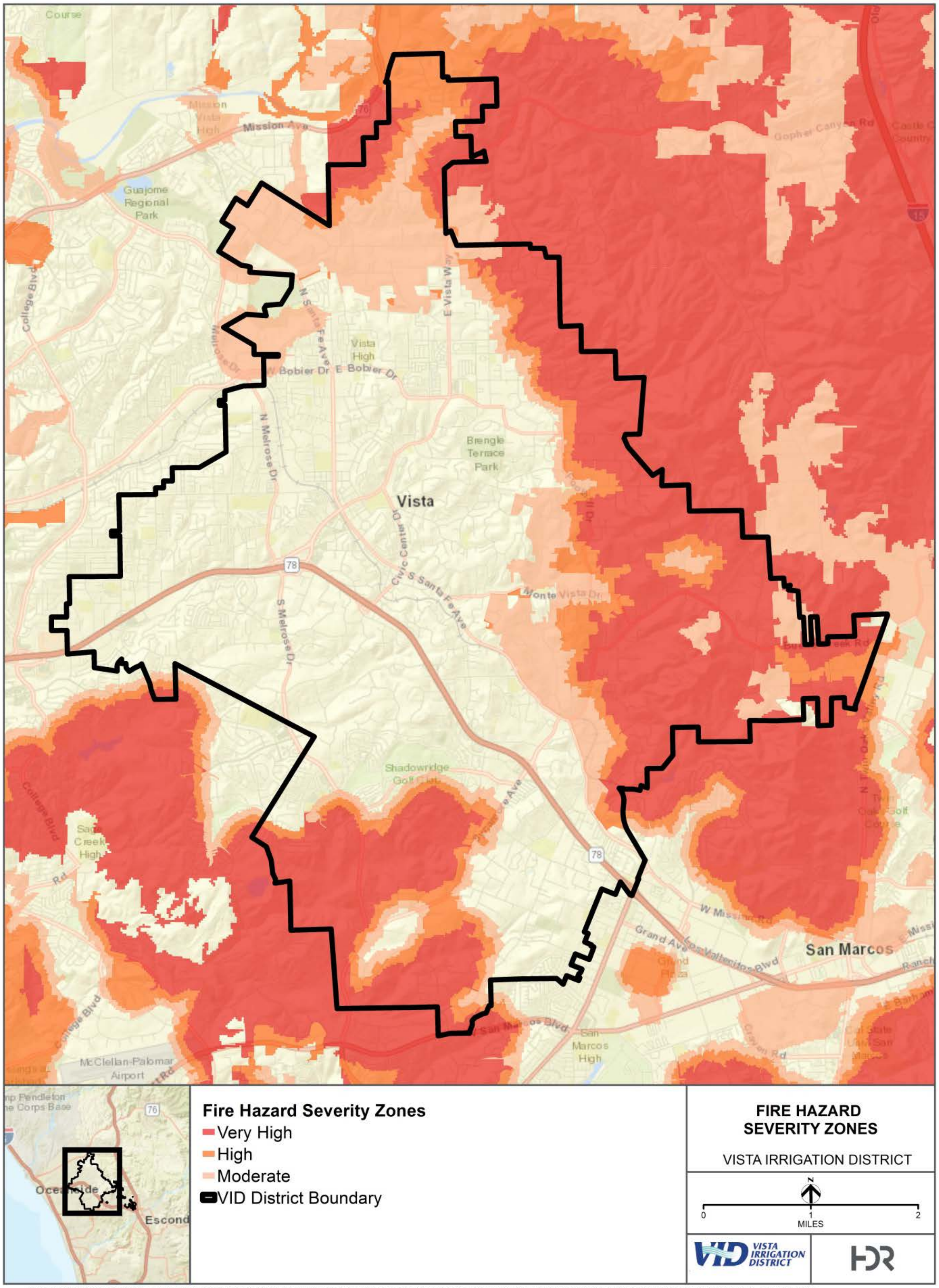
Figure 2-7. Fire Hazard Severity Zones



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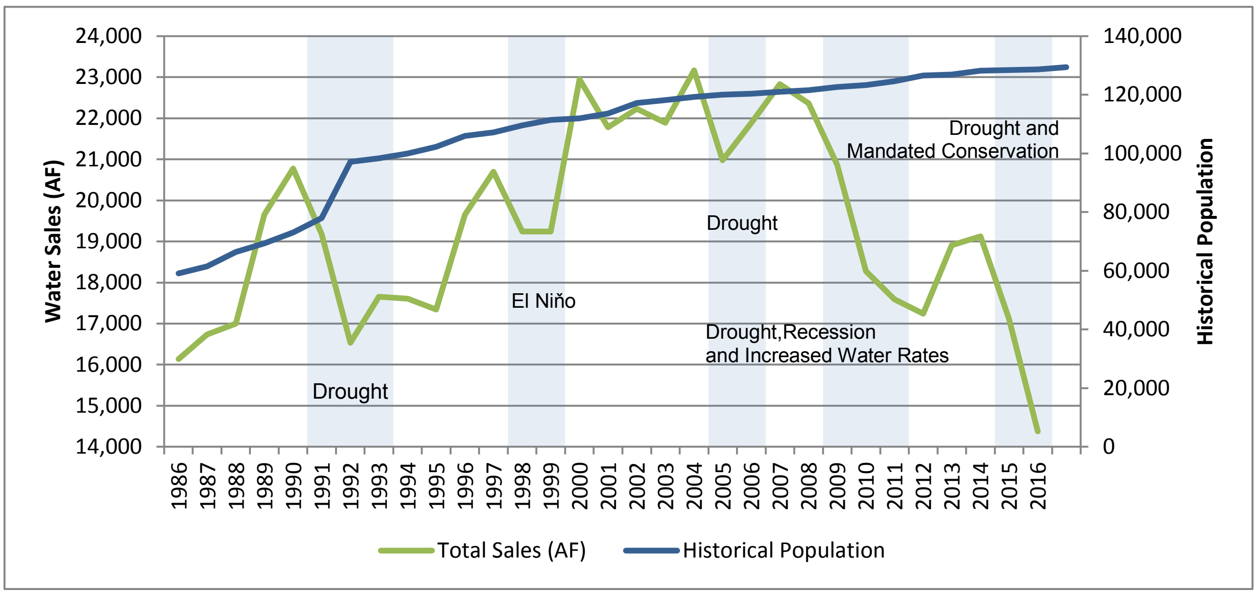
3 Water Demands

This chapter describes the basis for estimating current water demands, evaluates daily demand patterns and peaking factors, and projects future water demands. This data will be used to evaluate potential capacity deficiencies in the existing system and the need for improvements to accommodate future water demands.

3.1 Historical Water Use

Figure 3-1 shows the District's historical water sales on a FY basis dating back to 1986. In FY 1991 water demands decreased from over 20,000 AF to just over 17,700 AF in FY 1992. This decrease was attributable to a number of factors, including lingering drought impacts and the implementation of aggressive water conservation measures. Since that time, water demands remained relatively constant through 2006, taking into consideration weather, water supply conditions, and population growth.

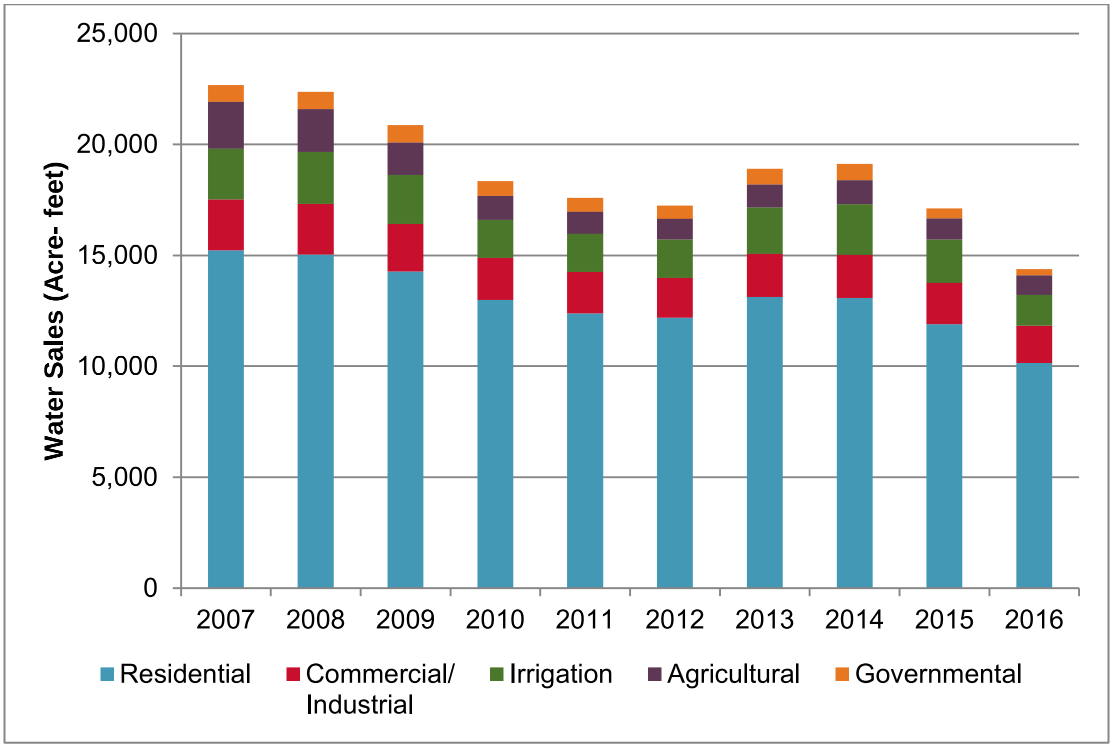
Figure 3-1. Historical Water Sales, Fiscal Year 1986 - 2016



Source: 2015 Urban Water Management Plan (UWMP)

Regional drought began gripping the American Southwest in 2002, and North San Diego County has experienced below-average rainfall for most of the past decade. In addition, the region experienced an economic recession in the 2009 to 2011 period, essentially halting anticipated development and causing an increase in local unemployment, which in turn resulted in a decrease in water use. The region experienced record warm years in 2015 and 2016, which would typically result in water use increases. However, statewide mandatory conservation in response to statewide drought conditions contributed to a significant decrease in water use during those years. Detailed water sales for the past 10 years are shown in **Figure 3-2**.

Figure 3-2. Detailed Water Sales, Fiscal Year 2007-2016

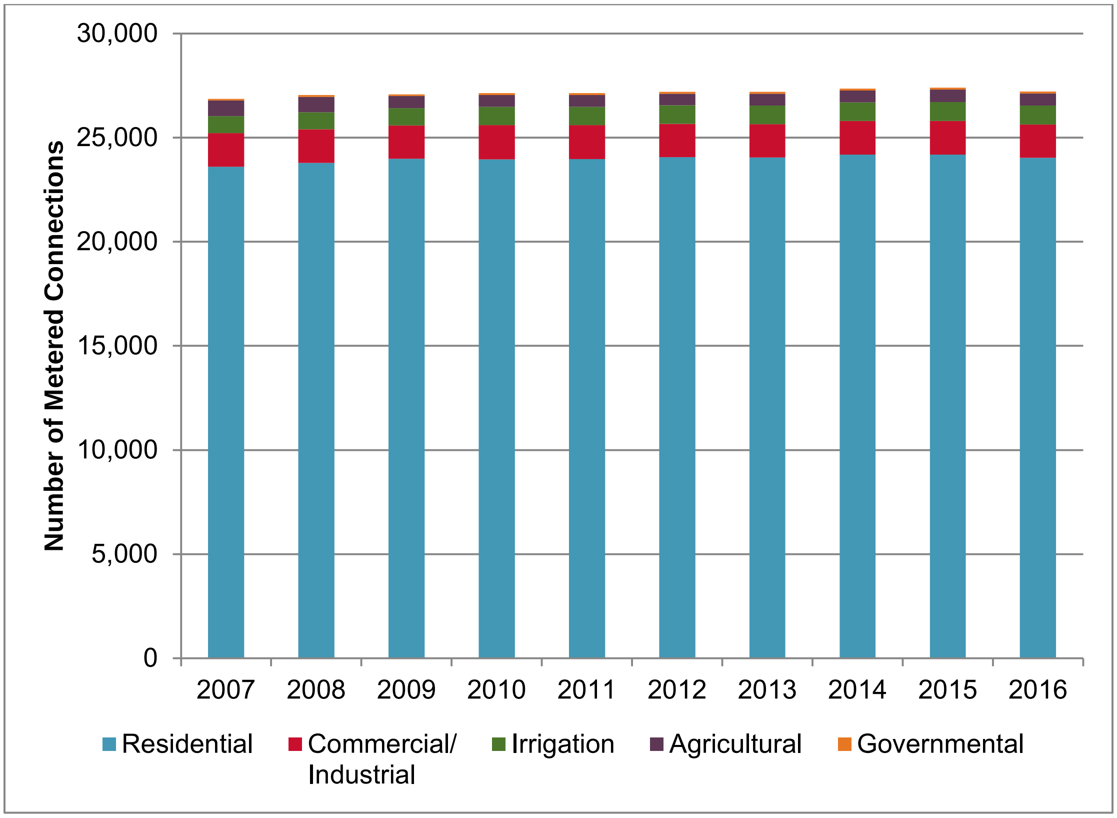


Source: 2016 Audited Comprehensive Annual Financial Report (CAFR)

Note that over the past 10 years, the District's Tier 1 Domestic Water charge has doubled (increasing from $1.98 per hundred cubic feet in 2007 to $4.04 per hundred cubic feet in 2016). Although this rate increase is less than other agencies in the region, it is likely to also have contributed to the decrease in water use within the District. Irvine Ranch Water District estimated a 10 percent reduction in water use by single family residences due to rate changes associated with a new tiered rate structure, similar to what the District instituted in September 2009. As local unemployment rates doubled in 2009 through 2012 (from 5 percent to over 11 percent), it is evident that the combination of the economic recession and changes in the water rate structure significantly impacted water use during this time period.

Although water sales have changed over the years due to economic recession, rate structure changes, and drought conditions, the total number of metered connections has stayed relatively the same over the past 10 years, as shown in **Figure 3-3**.

Figure 3-3. Historical Metered Connections



Source: 2016 Audited Comprehensive Annual Financial Report (CAFR)

All agencies experience some water loss as an ordinary part of operation. Water loss is "unaccounted for" water which can include leaks, line breaks, unmetered uses, meter inaccuracies, fire flow, or theft. **Table 3-1** illustrates the occurrence of District water loss over the past 5 years. Water loss is calculated by subtracting average annual demand (AAD) based on District billing data from the District's metered water supply data.

The District's 2015 Urban Water Management Plan (UWMP) estimated non-revenue (unmetered plus metered unbilled) water to be 836 AF and water loss to be 606 AF for July 2014 through July 2015, using the American Water Works Association Water Audit worksheet. Unavoidable Annual Real Losses were estimated to be 729 acre feet per year (AFY). This compares with 716 AFY in FY 2015, as calculated from the District's 2016 audited CAFR data in **Table 3-1**.

Table 3-1. Water Loss Estimates

FY	Annual Supply (AFY)	Annual Demand (AFY)	Difference (AFY)	Calculated Water Supply Loss (%)
2011	17,916	17,590	326	1.8
2012	18,901	17,241	1,660	8.8
2013	19,481	18,904	577	3
2014	20,134	19,128	1,006	5
2015	17,833	17,117	716	4

Source: 2016 Audited Comprehensive Annual Financial Report (CAFR)
AFY – acre feet per year; FY – fiscal year

3.2 Unit Demands

Average annual unit water demands are developed for specific land use types or water billing account types to project future water usage. The amount of water required by a given land use can vary widely, so for planning purposes the District's unit demand factors are developed based on existing water use specific to customers within the District. Because the type, location, and number of water customers change with time, typically the most recent 5 years of billing data are used for unit demand calculations. Ideally, as the basis of planning for future demand conditions, unit demands should represent current District demands independent of any short term trends. With this in mind, the District's water use history was taken into consideration in determining the basis for determining current unit demands.

As discussed in **Section 3.1**, several factors affected District water use in the decade from 2007 through 2016. Increased water rates were adopted in 2007 that may have contributed to decreased demands through the decade. The economic recession from 2009 through 2011, with some recovery from 2012 through 2014, followed by mandatory water conservation requirements in 2015 through 2016, greatly impacted water use trends. Although 2016 is the most recent calendar year with complete billing data, water demands during this period do not represent typical water use within the District for future planning purposes.

For this Master Plan, water demands from calendar year 2014 (17.29 million gallons per day [MGD]) were determined to be a reasonable baseline. Demands in 2014 appear to represent current District demands affected by the long term trend due to increased water charges, adoption of water efficient appliances and voluntary water conservation efforts, but independent of the short term factors of economic cycles and mandatory water use restrictions due to drought.

Existing land use data is based on the SANDAG Series 13 Regional Growth Forecast published in 2013, and the City of Vista Downtown Specific Plan, dated 2015, and summarized in **Table 2-1**. Unit demands were calculated for each land use type based on 2014 billing data for meters spatially located within each land use category for the District's entire service area (including the Boot and Bennett areas). To account for water loss in the unit demands, an estimated District-wide water loss of 4 percent was distributed proportionately among all the land use groups.

For reference, **Table 3-2** compares the calculated 2014 unit demand factors with the factors from the 2000 Master Plan (note that there are land use categories that currently exist that were not listed in the 2000 Master Plan). Most of the unit factors have decreased since the 2000 Master Plan. All non-residential unit demands have decreased, with public services, education, and parks showing significant decreases. Single family residential unit demands slightly increased, and rural residential and multifamily residential unit demands decreased. Mixed use developments (blended residential and non-residential) have increased significantly.

For planning purposes, calculated unit demand factors are typically rounded up to a particular magnitude so as to be more user-friendly and slightly conservative. For this Master Plan, the raw unit factors calculated based on 2014 billing data were rounded up to the nearest 50 value. The resulting 2017 Master Plan unit water demand factors are used to project future water demands, as described in **Section 3.4**.

Table 3-2. Average Annual Demand Unit Water Demand Factors

SANDAG Group Code[1]	Land Use Group	AAD Unit Water Demand Factors (gpd/Acre)		
		Calculated from 2014 Demands[2]	2000 Master Plan	2017 Master Plan[3]
10 _ _	Rural Residential	411	650	450
11 _ _	Single Family Residential	1,081	1,020	1,100
12 _ _	Multi-Family Residential	3,635	4,100	3,650
13 _ _	Mobile Home Park	1,202	-	1,250
14 _ _	Other Group Quarters Facility	2,237	-	2,250
15 _ _	Hotel/Motel (Low-Rise)	3,127	-	3,150
21 _ _	Industrial	1,009	2,020	1,050
41 _ _	Utilities	225	0-200	250
50 _ _	Commercial	1,425	2,020	1,450
60 _ _	Office	1,274	2,020	1,300
61 _ _	Public Services	569	2,020	600
65 _ _	Health Care	1,865	-	1,900
68 _ _	Education	840	2,020	850
72 _ _	Golf Course	109	-	150
76 _ _	Park	483	1150-1250	500
80 _ _	Agriculture	445	-	450
97 _ _	Mixed Use	3,386	2,020	3,400

1 Source: San Diego Association of Governments (SANDAG) Series 13 Current Land Use shapefiles

2 Source: Vista Irrigation District (District) Water Billing Account History 2014 adjusted for 4 % Water Loss based on San Diego County Water Authority (Water Authority) and Filter Plant Supply Data

3 Based on Existing Area acres (**Table 2-1**) and 2014 average annual demand (AAD) values for District service area rounded up to nearest 50 value

gpd – gallons per day

3.3 Peaking Factors

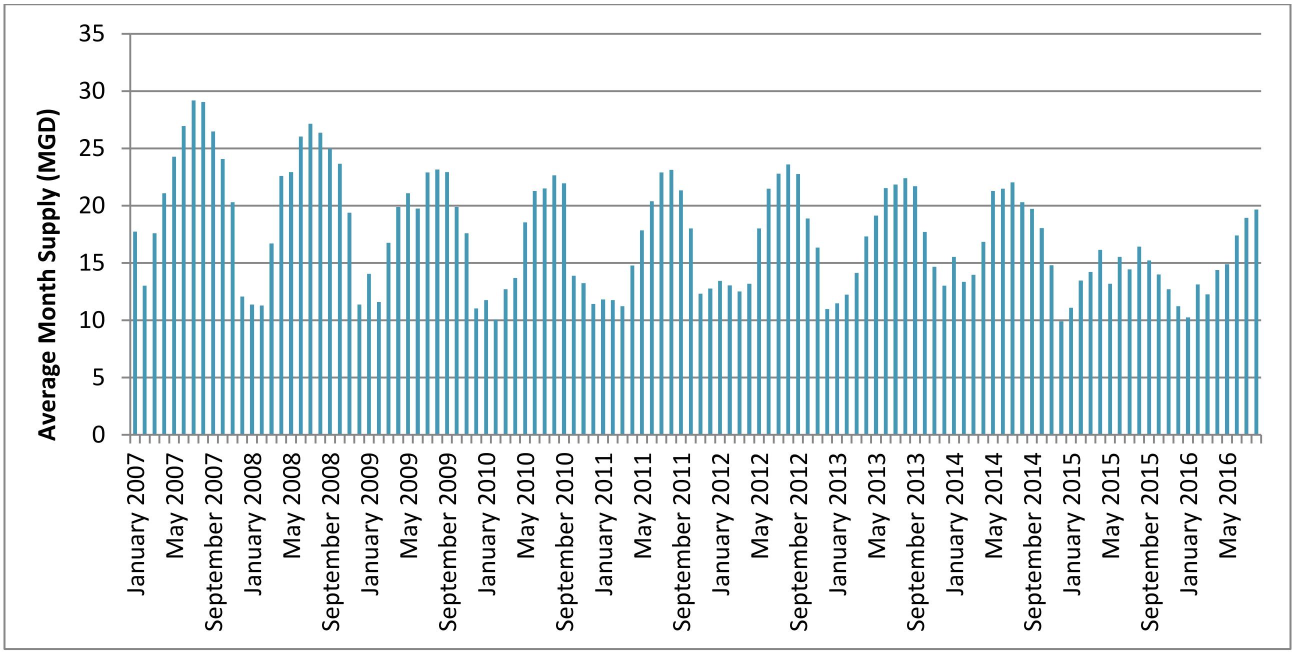
The water demands discussed in the previous sections are based on average annual water consumption. Actual water demand patterns, however, vary daily, hourly and seasonally. Flow variations are commonly expressed in terms of peaking factors, which are multipliers used to express the magnitude of variation from the AAD. The peaking factors that are most important in the development and the analysis of how a water system corresponds to the maximum day use and peak hour use. Peaking factors within the distribution system will typically decrease as the total system demand increases. Therefore peaking factors for the entire system may be less than the peaking factors for individual pressure zones.

Water consumption varies throughout a 24-hour period creating diurnal patterns. These patterns typically vary from weekdays to weekend days. These patterns can also vary within individual pressure zones based on land use factors. For instance, in an area that is primarily industrial or commercial, the diurnal pattern will vary from a predominantly residential area. Peak hourly demands and diurnal curve patterns for the overall District are included in this chapter. Diurnal patterns for individual pressure zones are discussed in **Chapter 6**.

3.3.1 Maximum and Minimum Day Peaking Factors

Water demands vary seasonally and daily. Seasonal variations arise primarily from seasonal differences in weather conditions and resulting irrigation needs, with water demands higher in the summer months and lower in the winter months. Likewise, day-to-day variations arise primarily from weather conditions, with maximum demand days typically occurring during hot Santa Ana wind days in late summer and early fall, and minimum demand days occurring on rainy days in the wintertime. **Figure 3-4** displays monthly potable water deliveries for 2007 through mid 2016 in MGD and illustrates these seasonal fluctuations. Historically, the single day with the maximum water use typically occurs during a dry, hot day between July and September when outdoor use for irrigation is peaked. The minimum day has historically occurred in January or February, during rainy periods when irrigation is not needed.

Figure 3-4. Average Month Supply



Source: Actual End of Month Deliveries Reported by CWA / Filter Plant, 2007 - 2016

However, over the past decade these maximum and minimum patterns have shifted. The current data, shown in **Table 3-3**, indicates water use can peak earlier than August (as early as March) at levels from 150 percent to 180 percent of the annual average, and typically reaches a low in December at levels from 40 to 50 percent of the annual average.

Figure 3-5 graphically presents the data provided in **Table 3-3**, illustrating the declining trend of the District's average day supply over the past 10 years, the consistency of the annual minimum day supply and the variability of the annual maximum day supply.

The 2000 Master Plan used a 2.0 peaking factor for maximum day demands (MDD) as a conservative projection for planning purposes. As is evident in **Table 3-3**, there has been a downward trend in both water use and peaking factors, with peaking factors ranging from 1.42 to 1.84. This is likely a result of successful outdoor water conservation programs reducing demands during summer months.

As previously discussed, data from 2014 appears to provide a reasonable baseline for future planning purposes. The maximum day peaking factor in 2014 was 1.84. As this peaking factor is relatively close to the 2000 Master Plan criteria of 2.0, to be consistent and conservative, a maximum day peaking factor of 2.0 and a minimum day factor of 0.5 is recommended for future planning purposes.

Table 3-3.Historical Peaking Factors

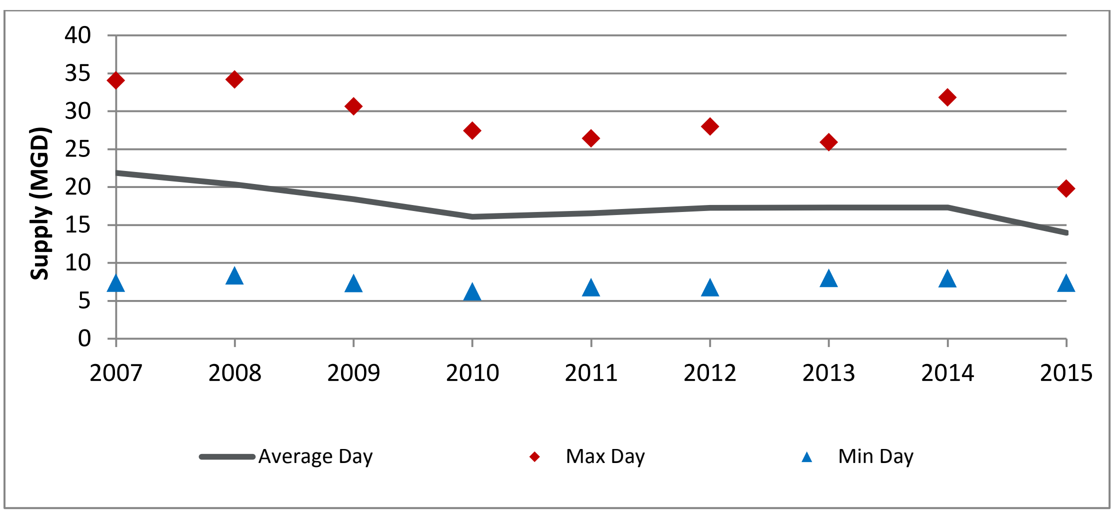
Year	Average Day Supply (MGD)	Maximum Day Supply (MGD)	Maximum Day Date	Maximum Day Peaking Factor	Minimum Day Supply (MGD)	Minimum Day Date	Minimum Day Peaking Factor
2007	21.87	34.05	July 27, 2007	1.56	7.37	December 4, 2007	0.34
2008	20.33	34.16	August 26, 2008	1.68	8.34	January 12, 2008	0.41
2009	18.42	30.64	August 27, 2009	1.66	7.33	January 1, 2009	0.40
2010	16.09	27.43	September 28, 2010	1.70	6.23	December 22, 2010	0.39
2011	16.55	26.40	July 9, 2011	1.59	6.77	January 12, 2011	0.41
2012	17.26	27.98	August 6, 2012	1.62	6.77	December 18, 2012	0.39
2013	17.29	25.90	June 28, 2013	1.50	8.00	December 7, 2013	0.46
2014	17.29	31.79	May 14, 2014	1.84	7.97	December 21, 2014	0.46
2015	13.97	19.77	March 14, 2015	1.42	7.36	December 23, 2015	0.53

Source: Actual End of Month Deliveries Reported by CWA / Filter Plant, 2007 – 2016.

Note Max Day Supply includes non-demand contributions to storage and interagency flow exchanges.

MGD - million gallons per day

Figure 3-5. Historical Average, Maximum and Minimum Day Water Supply

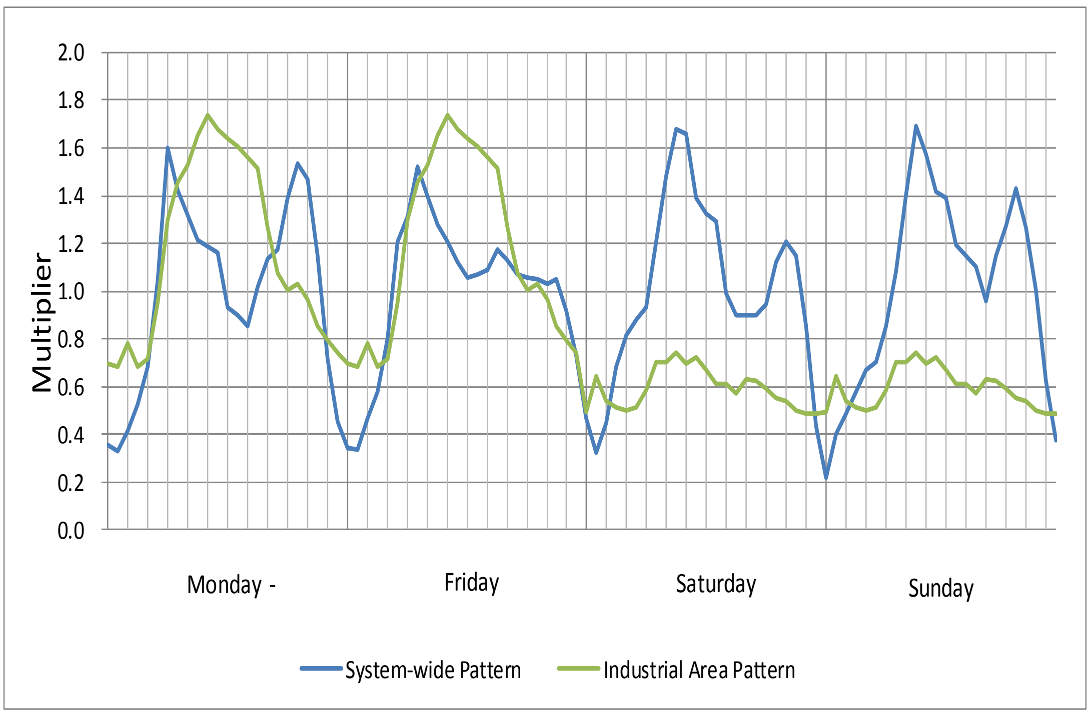


3.3.2 Diurnal Patterns

In operating the District's water distribution system, daily water consumption patterns govern the pumping and storage requirements needed to maintain water supply to District customers. The time of day diurnal demand curve is a series of 24 hourly demand factors that define how water usage varies over the course of a day. This demand pattern is used in the hydraulic model to calculate hourly demands, peak flows, and operational storage needs.

Based on supervisory control and data acquisition (SCADA) data for 2016, diurnal patterns were developed for system wide demands and industrial demands. The multiplier of 1.0 on the vertical axis represents the current day's total demand. The horizontal scale is divided into 2-hour increments, covering a 24-hour day. The curves, displayed in **Figure 3-6**, illustrate the variation in water use over the course of the day, relative to the total day's demand.

Figure 3-6. District-Wide Diurnal Curve



Note that the weekday patterns for Monday through Thursday are fairly uniform, with peak usage in the early morning and dinner time. The afternoon and evening patterns on Friday differ greatly from the other weekdays, with little or no variation from noon to 8 p.m. The weekends have higher peak hour usage in the mornings and smaller evening peak uses in the evenings than typical weekday patterns. These curves vary slightly on a seasonal basis. The curves shown represent winter months, with little outdoor water use. In summer months, during maximum days when more water is being used in the evenings or early morning hours for irrigation, the diurnal curve peaks are slightly smaller. For this Master Plan, the diurnal patterns shown in **Figure 3-6** were used.

3.4 Projected Water Demands

District population forecasts in 5-year increments through 2040 were reported in the District's 2015 UWMP and illustrated in **Figure 2-3**. Based on projections presented, the population in the District's service area is expected to increase approximately 24 percent from 2015 to 2040, for an average of just less then 1 percent annually.

Assuming that water use demographics in the future remain similar to the 2014 baseline (17.29 MGD) and future water demand would correspond proportionately with population growth, a 24 percent increase in water demand in 2040 would result in a total water demand projection of 21.44 MGD. Alternatively, applying 125 gallons per capita per day, which the 2015 UWMP estimates is the current usage, the anticipated additional water demand would be 3.87 MGD, which would result in a similar projected demand of 21.16 MGD (3.87 MGD over the baseline demand of 17.29 MGD).

However, more detailed methods for projecting future water demand are available and are presented in the following sections. It should be noted that no projection is assured of accuracy in an environment where changing economic and climate conditions and growth rates influence water consumption. Demand projections in this Master Plan assume that future unit demand water consumption will be similar to the 2014 baseline.

3.4.1 Land Use Based Projections

Land use based future AAD projections are based on the land use data discussed in **Chapter 2**, including SANDAG Series 13 Regional Growth Forecast and the Vista Downtown Specific Plan, and the unit water demand factors shown in **Table 3-2**.

Two approaches were considered. Demand projections using each of these methods for the District's entire service area are shown in **Table 3-4**.

In one approach, the new unit demand factors were applied to all parcels within the District's service area, using the buildout land use category. This results in a projected demand of 20.3 MGD for planned land use.

In the second approach, we considered that existing land use may have a higher demand now than if the property was redeveloped under the buildout land use category. For instance, there is approximately 5 acres of commercial property that is designated as Golf Course under buildout conditions. The commercial property uses quite a bit more than the golf course might. Demands for each parcel were calculated based on both current and planned land use type using the unit water demand factors. To be conservative, the larger of the two calculated values was used as the projected future demand. The resulting demand projection for the District's entire service area using this maximum land use methodology totals 20.9 MGD. This includes 0.3 MGD and 0.6 MGD for the Boot and Bennett areas, respectively.

By comparison, these projections are approximately 25 percent less than the 2000 Master Plan, which projected an ultimate demand of 30,500 AFY or 27.2 MGD. This decrease is likely due to the District's water conservation efforts over the past 16 years, a significant reduction in non residential unit demands and reductions in densities for ultimate buildout associated with more recent land use planning documents.

Table 3-4. Projected Water Demands

SANDAG Group Code	Planned Land Use Group	Unit Factor (gpd/Acre)[1]	Buildout Area (Acres)[2]	Projected Demand based on Planned Land Use (gpd)[3]	Projected Demand based on Maximum Land Use (gpd)[4]
10 _ _	Rural Residential	450	5,060	2,277,000	2,776,100
11 _ _	Single Family Residential	1,100	8,335	9,168,900	9,176,500
12 _ _	Multi-Family Residential	3,650	859	3,136,800	3,136,800
13 _ _	Mobile Home Park	1,250	281	351,200	351,200
14 _ _	Other Group Quarters Facility	2,250	135	304,200	304,200
15 _ _	Hotel/Motel (Low-Rise)	3,150	11	34,300	34,300
21 _ _	Industrial	1,050	1,166	1,224,700	1,226,100
41 _ _	Utilities	250	2,448	611,900	614,900
50 _ _	Commercial	1,450	564	817,600	844,100
60 _ _	Office	1,300	120	156,400	161,000
61 _ _	Public Services	600	248	148,600	148,600
65 _ _	Health Care	1,900	44	84,500	84,500
68 _ _	Education	850	338	286,900	287,000
72 _ _	Golf Course	150	174	26,100	33,600
76 _ _	Park	500	1,019	509,400	514,600
80 _ _	Agriculture	450	14	6,400	6,400
97 _ _	Mixed Use	3,400	341	1,160,600	1,167,600
Total		N/A	21,157	**20,305,000**	**20,867,500**

1 Per **Table 3-2**

2 Source: San Diego Association of Governments (SANDAG) Series 13 Planned Land Use shapefiles, including Boot and Bennett areas; Note Area is rounded to nearest whole acre.

3 Based on application of unit factors to actual acres per planned land use type for the entire service area.

4 Based on application of unit factors to actual acres per maximum land use type for the entire service area.

gpd – gallons per day

3.4.2 Urban Water Management Plan Projections

The District's 2015 UWMP projected water demands are consistent with SANDAG's Series 13 Regional Growth Forecast for future population and projected per capita water demands, as developed by the Water Authority. The normal year projections are based on the Water Authority's statistical evaluation of relevant data such as climate, rainfall/run-off, population growth, water demands, and the relationship between household income and response to the price of water, per the Water Authority's UWMP. The District estimated that hot-dry weather years (absent mandatory water use restrictions) may generate 10 percent greater demands than during normal years, and this percentage was utilized to calculate single-dry and multiple-dry year demands. Both normal and dry year projections are shown in **Table 3-5**.

Both passive and active conservation measures were considered in the 2015 UWMP. Based on the California Department of Water Resources' 2015 Plan Guidebook, the Water Authority developed estimated water savings for each of its member agencies, including the District, using the Alliance for Water Efficiency Water Conservation Tracking Tool. Passive conservation savings are based on appliance standards, plumbing code changes, and conversion of active savings to passive as the useful life of devices are reached. Estimated savings from the 2015 Model Water Efficient Landscape Ordinance are included in this category. Compliance from new residential development was set at 80 percent, and a majority of this savings was assumed to continue over the Plan's 2040 planning horizon. Additionally, passive conservation includes savings from landscape conversions at existing single family homes.

Future active conservation was set at the 2015 level of participation in program offerings and estimated savings for each year over the planning horizon. Active conservation includes activities, such as indoor and outdoor incentives, landscape classes, and irrigation checkups. The District incorporated estimated water savings shown in **Table 3-5** in its 2015 UWMP projected future demands for planning purposes. Under these varying conditions the District's projected demands in 2040 range from 19.75 MGD to 23.72 MGD.

Table 3-5. Urban Water Management Plan Demand Projections

Water Demand (MGD)						
Year	**2015**	**2020**	**2025**	**2030**	**2035**	**2040**
2015 UWMP	15.92	17.63	19.04	20.20	20.82	21.56
2015 UWMP Under Single and Multi Dry Year Conditions	NA	19.39	20.94	22.22	22.90	23.72
Estimated Water Conservation Savings	NA	(2.77)	(3.38)	(3.51)	(3.74)	(3.97)
2015 UWMP with Dry Year Conditions and Conservation Savings	NA	16.62	17.56	18.72	19.16	19.75

MGD – million gallons per day; UWMP - Urban Water Management Plan

3.4.3 Potential Demand Projection Variables

As illustrated in the section above, climate conditions and water conservation measures, as well as economic conditions, are likely to be the key drivers for future variations in actual water demand and consumption. An assessment of these potential variables and their impact on water demand was conducted to create a planning envelope, defining High and Low water demand projections to bracket the Baseline or Medium water demand projection.

In the sections above, water demand projections for the 2040 planning horizon were estimated using both future population and land use projections.

- Based on SANDAG population growth projections and a 125 gallons per capita per day demand factor (21.16 MGD)
- Based on planned ultimate land use and unit demand factors (20.30 MGD)
- Based on maximum land use and unit demand factors (20.87 MGD)
- Based on 2015 UWMP normal year projections (21.56 MGD)
- Based on 2015 UWMP dry year projections (23.72 MGD)
- Based on 2015 UWMP dry year conditions plus water conservation savings (19.75 MGD)

These estimates are represented by the dotted lines in **Figure 3-7**. All but the population growth projection use the 15.92 MGD baseline that was assumed in the District's 2015 UWMP.

In **Table 3-6**, three planning variables were considered and factors associated with low, medium, and high projections were developed to evaluate additional scenarios to the projections listed above. For the baseline variable, dry year conditions and potential rebound from recent drought conditions were considered. For the economic growth variable, plus and minus 10 percent of maximum land use projections were considered. For water conservation savings, achievement of 100, 50, and 25 percent of the UWMP estimates were considered. The results are shown in **Table 3-6** and displayed as the solid line projections in **Figure 3-7.**

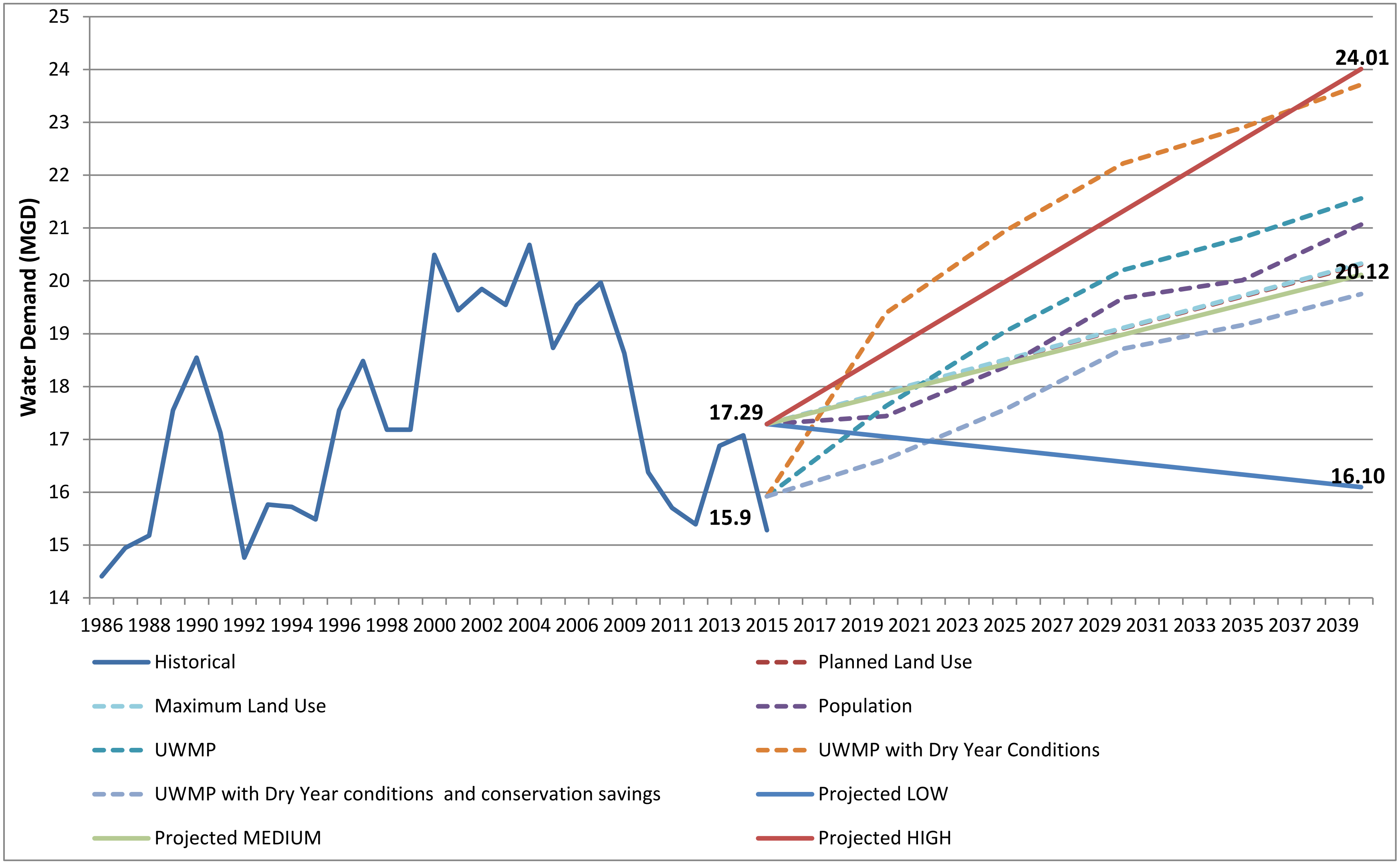
The medium water demand projection of 20.12 MGD aligns well with the land use based projections and the 2015 UWMP Dry Year conditions with conservation savings. The high and low projections are 20 percent above and below, respectively, of the medium projection. Given the wide swing in demand over the past 10 years it is challenging to look backward for a trend. For planning purposes, using the medium projection appears a reasonable approach and should be reassessed in 5 years to track and respond to any significant variance.

Table 3-6. Planning Variables for Water Demand Projections in 2040

Planning Variable	Low Projection		Medium Projection		High Projection	
Baseline (2014)	2014 Baseline Demand	17.29 MGD	Baseline with 10% Increase under Dry Year Conditions	19.02 MGD	Baseline with 25% Increase due to Dry Year Conditions and Rebound to Pre-2007 Demands	21.60 MGD
Economic Growth	10% Decrease in Medium Projection	2.77 MGD	Anticipated Increase based on Maximum Land Use Projections	3.08 MGD	10% Increase in Median Projection	3.39 MGD
Water Conservation Savings	UWMP's Estimate for Active Conservation Savings of 3.97 MGD by 2040	3.97 MGD	Achievement of 50% of UWMP Estimate	1.98 MGD	Achievement of 25% of UWMP Estimate	0.99 MGD
	Total	16.10 MGD		20.12 MGD		24.01 MGD

MGD – million gallons per day; UWMP - Urban Water Management Plan

Figure 3-7. Water Demand Projections

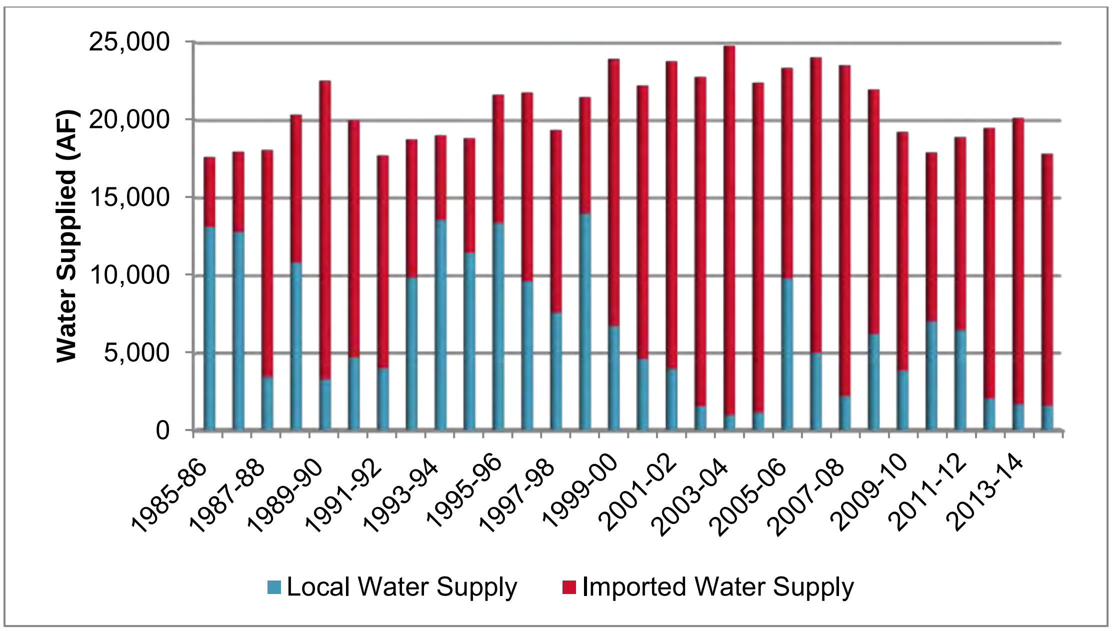


4 Water Supply

The District's water supply originates from two sources: local water and imported water from the Water Authority. Local water from the San Luis Rey River watershed is stored on a seasonal basis in the Lake Henshaw and Lake Wohlford reservoirs. Principal water storage and conveyance facilities include the Warner Basin aquifer, Lake Henshaw, Warner Ranch Well Field, Escondido Canal, Lake Wohlford, Dixon Lake, Bear Valley Pipeline, and EVWTP. A portion of the San Luis Rey River is also used for conveyance. Local water is shared with Escondido and provides approximately 30 percent of the District's average water demand.

The District's use of water from Lake Henshaw dates back to 1926. The lake was purchased by the District, along with the 43,000 acre Warner Ranch, in 1946. Drought conditions and population growth in the late 1940's and early 1950's prompted the District to look for additional sources of water. In 1954, the District became a member of the Water Authority to gain access to water imported from the Colorado River and Northern California. During years when rainfall is significantly below average and the availability of local water is limited, well over 90 percent of the District's water supply can come from imported sources. The historical use of imported water, measured in AF, is illustrated in **Figure 4-1.**

Figure 4-1. Water Supply to Vista Irrigation District



As a result of a long term drought in the Colorado River Basin and environmental constraints associated with the delivery of water from Northern California, the Water Authority was prompted to expand its local water portfolio. The Water Authority began receiving and distributing desalinated seawater from the Claude "Bud" Lewis Carlsbad Desalination Plant to its member agencies, including the District, in December 2015.

This chapter describes the District's water supply system and local water supply facilities.

4.1 Local Supply

Water released from Lake Henshaw flows downstream in the San Luis Rey River channel to the intake of the Escondido Canal, which diverts water from the river. The Escondido Canal conveys water to Lake Wohlford, where it is stored and released through the Bear Valley Pipeline to the EVWTP at Lake Dixon. Treated water from the EVWTP is conveyed via the Vista Flume to the District service area. **Figure 4-2** shows the location of these local storage and conveyance facilities, which are further described below.

4.1.1 Warner Basin Aquifer

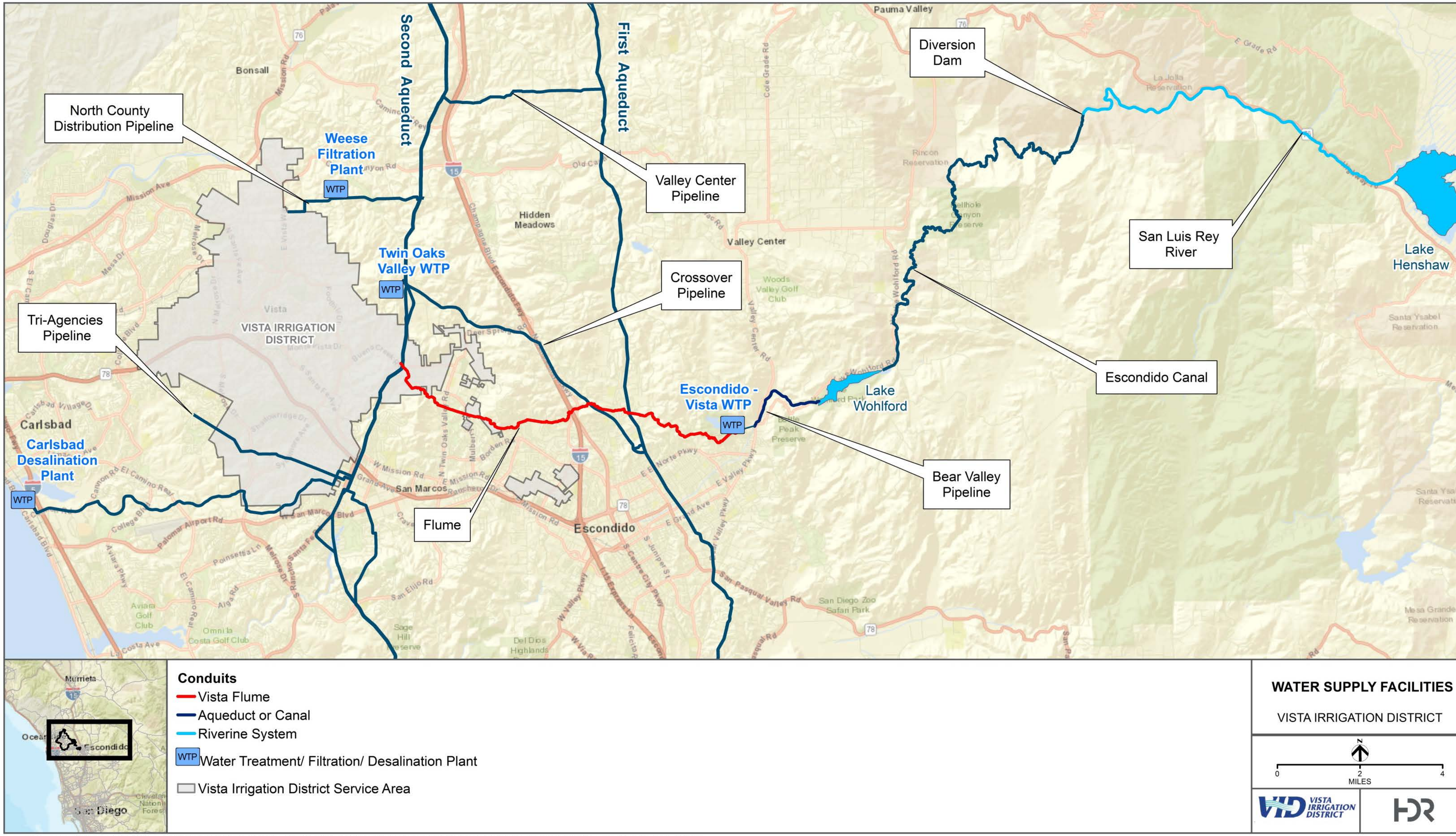
The Warner Basin aquifer is a developed groundwater basin located 50 miles east and north of the District. Total usable storage in the aquifer is estimated to be 400,000 AF; 150,000 AF of active storage volume is located in the aquifer where extraction is feasible using currently operating District wells. The District has 16 production wells that pump from depths of 150 to 350 feet, depending on rainfall and length and extent of pumping operations. Since 1960, the District's median groundwater production has been 7,728 AFY. This water is pumped into Lake Henshaw for surface water storage and subsequent delivery to the District and the City of Escondido.

In dry years, groundwater is pumped from the well field to Lake Henshaw and released as needed. The wells vary in capacity from 300 to 2,000 gallons per minute (gpm). Water is conveyed to Lake Henshaw through about 8 miles of pipeline and 12 miles of lined, open ditches. In wet years, the surface water supply is used and pumping operations cease, permitting the basin to recharge and groundwater levels to rise. Thus, the groundwater basin acts as a water bank, allowing deposits in wet years and withdrawals in dry years.

To date, the Warner Basin aquifer has not been adjudicated nor has it been identified as being in overdraft. In September 2014, the Sustainable Groundwater Management Act was signed into law. The law provides new tools and authorities for local agencies to manage groundwater resources within their jurisdictions to achieve a sustainable use of those resources within a 20-year implementation period. While Sustainable Groundwater Management Act provides specific mandates only for those groundwater basins deemed by the State to be "medium" or "high" priority groundwater basins, the law encourages the formation of "Groundwater Sustainability Agencies" and the preparation of "Groundwater Sustainability Plans" (GSPs) for all groundwater basins, even those deemed "low" and "very low" priority basins.

The California Department of Water Resources has classified the Warner Basin as a "very low" priority basin. Nevertheless, the Warner Basin represents a significant water source for the District. The District continues to investigate groundwater resources in the Warner Basin and the cost/benefit of forming a Warner Valley Groundwater Sustainability Agency.

Figure 4-2. Regional Water Supply Facilities



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4.1.2 Lake Henshaw

In 1946, the District purchased the Warner Ranch, which included Henshaw Dam and Lake Henshaw. Lake Henshaw was the District's sole supply of water until the formation of the Bueno Colorado Municipal Water District in 1954. The dam and reservoir are owned and operated by the District, and the City of Escondido maintains storage rights. About one third of the 200 square mile watershed is owned by the District and is managed to protect water quality. Lake Henshaw receives, on average, about 30 inches of rain per year. The undeveloped character of the watershed and the District's management activities contribute to the high quality of this local water supply.

Lake Henshaw is a 52,000 AF capacity water supply reservoir located on the San Luis Rey River, about 25 miles east of the District's service area. Lake Henshaw Dam was completed in 1922, enlarged in 1927, and modified in 1981 to comply with California State Division of Safety of Dams requirements. The dam is a zoned hydraulic-fill embankment with an overflow weir spillway on the right abutment.

Both natural runoff developed above Lake Henshaw and groundwater pumped from the Warner Basin are held as surface water in Lake Henshaw. The water is delivered to the District, the City of Escondido, and the Rincon Band of Indians under terms of several governing contracts. While the amount of water delivered to each party is dependent on annual hydrologic conditions, the median local water delivery to the District since 1960, including groundwater production and surface water runoff, is 5,062 AFY.

4.1.3 San Luis Rey River

About 9.5 miles of the natural channel of the San Luis Rey River is used to convey water from Lake Henshaw Dam to the intake of the Escondido Canal. The river is enclosed by steep canyon walls and has no maximum conveyance limitations, nor any minimum flow requirements. It is estimated that there is very little seepage from the river, although about 2,500 AFY is absorbed by riparian vegetation or evaporates. On the average, the river catches about 10,000 AFY of additional runoff from adjacent watersheds.

The District has recently resolved litigation initiated in 1969 pertaining to its use of the waters of the San Luis Rey River, including both its Lake Henshaw and Warner Basin groundwater supplies. This litigation, involving the District, the City of Escondido, five local Indian Bands, and the federal government, was resolved when the Settlement Agreement approved by the parties became effective on May 17, 2017. Under the Settlement Agreement, Escondido and the District are allowed to develop, divert, and use the waters of the San Luis Rey River basin (Local Water) substantially as they have in the past. Under the Settlement Agreement, the federal government has agreed to furnish 16,000 AFY of water conserved from the lining of the All American and Coachella Canals (referred to as Supplemental Water) to the Settlement Parties (the District, the City of Escondido, and the five Indian Bands). Other agreements provide for the wheeling of Supplemental Water through facilities owned by MWD and the Water Authority for use either on the reservations of the five Bands, or within the service areas of the District or the City of Escondido.

Under the Settlement Agreement, the District and the City of Escondido continue to pay for and enjoy the benefits of Local Water and the five Bands pay for and enjoy the benefits of the Supplemental Water. Any Supplemental Water that is surplus to the needs of the five Bands will be delivered in equal measure to the District and the City of Escondido, which are required to take delivery of such water and pay the Bands what they would otherwise have paid the Water Authority for that same quantity of water. Additionally, any of the five Bands may elect to exchange an acre-foot of Local Water delivered from the local water system operated by Escondido and the District for an acre-foot of Supplemental Water delivered to Escondido and the District. This last measure provides for water delivery to Bands' reservations that may not have access to imported water, or who may prefer the delivery of untreated water.

4.1.4 Escondido Canal

The Escondido Canal was first constructed in 1895. A small diversion dam routes water in the San Luis Rey River into the Escondido Canal for delivery to Lake Wohlford, about 14 miles distant. The canal and diversion dam were improved and enlarged in 1924 to take advantage of increased deliveries made possible by the construction of Henshaw Dam. The diversion dam is a 16 foot high concrete gravity structure with an integral canal intake facility at the left end. There is an ungated overflow weir with 13,000 cubic feet per second (cfs) of capacity.

Current operational capacity of the Escondido Canal is 50 cfs. The canal is owned and operated by the City of Escondido, although the District has capacity rights. The canal traverses about 14 miles of rugged terrain and consists of 11.1 miles of shotcreted canal, 1.6 miles of pipeline, 0.7 mile of tunnel, and 0.1 mile of metal flume. It terminates in Escondido Creek at the north end of Lake Wohlford.

4.1.5 Lake Wohlford Dam and Reservoir

In 1895, a 2,800 AF impoundment was created by the construction of a rock-fill dam on Escondido Creek, originally called the Bear Valley Dam, to receive the waters delivered through the Escondido Canal. In 1924, in conjunction with the construction of Henshaw Dam and the enlargement of the Escondido Canal, the dam was completely rebuilt as a hydraulic-and-rock-fill structure with a maximum storage capacity of 6,460 AF.

In 2007, the Federal Energy Regulatory Commission began requiring that Lake Wohlford water level be maintained at least 20 feet below the spillway crest level for dam safety purposes, thus limiting the capacity to 2,800 AF. The City of Escondido has completed several studies for the Lake Wohlford Dam Replacement Project and plans to replace the existing dam structure with a new roller compacted concrete dam to utilize the full storage capacity.

Most of the water released from Lake Wohlford passes through the 75 cfs capacity Wohlford Penstock to the Bear Valley Hydroelectric Generation Facility, which has a capacity of 50 cfs. The District maintains a bypass line to directly divert the excess 25 cfs when necessary. Lake Wohlford is also used as a recreational facility.

4.1.6 Bear Valley Pipeline

The Bear Valley Pipeline was originally constructed as two 43 cfs pipelines, one each for the City and the District. In the early 1990s, these pipelines were partially replaced with a single 54-inch diameter pipeline from the Bear Valley Hydroelectric Plant to the intersection of Lake Wohlford Road and Foxley Lane.

4.1.7 Lake Dixon Dam and Reservoir

Lake Dixon Dam was completed in 1970 and is a zoned earth-fill embankment. With a total capacity of 2,610 AF, Lake Dixon Reservoir is primarily used to store imported water. There is no significant delivery of local water to Lake Dixon.

4.1.8 Escondido-Vista Water Treatment Plant

The EVWTP treats raw water from wholesale and local sources before it is delivered to District customers. Water flows by gravity from Lake Dixon at a maximum instantaneous flow rate of 80 MGD and enters the EVWTP through a 54-inch pipeline. Water may also enter from the 42-inch Water Authority Crossover Pipeline with a fluctuating flow. Maximum inflow from Lake Wohlford is approximately 50 MGD. Local water is blended with imported water prior to treatment.

The EVWTP was completed in 1975 and expanded in 1984. Designed for 90 MGD, the EVWTP is currently permitted to produce 75 MGD due to restrictions placed by the Department of Health on the plant’s filtration system. Treatment includes coagulation, sedimentation, filtration, and disinfection to ensure drinking water quality. Bacteriological, physical, and chemical tests are performed on water samples to assure that safe water for customers is being produced and maintained in the distribution system. Treated water is delivered either to the EVWTP Clearwell and then to Escondido’s distribution system or to the Vista Flume for delivery to the District. The District owns capacity rights for treatment of 18 MGD; Escondido owns the remainder.

4.1.9 Vista Flume

The District’s portion of treated water from the EVWTP is conveyed to the District’s Pechstein Reservoir via an 11-mile conduit that includes both flume and siphon conveyance systems. The Vista Flume is owned, operated, and maintained by the District. The flume portion of the alignment totals 5.5 miles in length and consists of 11 bench sections. The siphon system is 5.75 miles in length and is comprised of five riveted steel sections, three concrete sections, one high density polyethlylene (HDPE) section, and a 0.25-mile-long hard rock tunnel (Big Tunnel) section.

The flume was constructed with a very uniform vertical grade approximating 1 percent throughout. The horizontal bending of the flumes is often quite severe to match the terrain needed to obtain the uniform vertical grade and includes numerous compound and compound reverse curves of minimal radius. Gravity flow through the existing bench sections that are lined on the floor and walls with a HDPE sheet lining system can currently convey approximately 20 MGD.

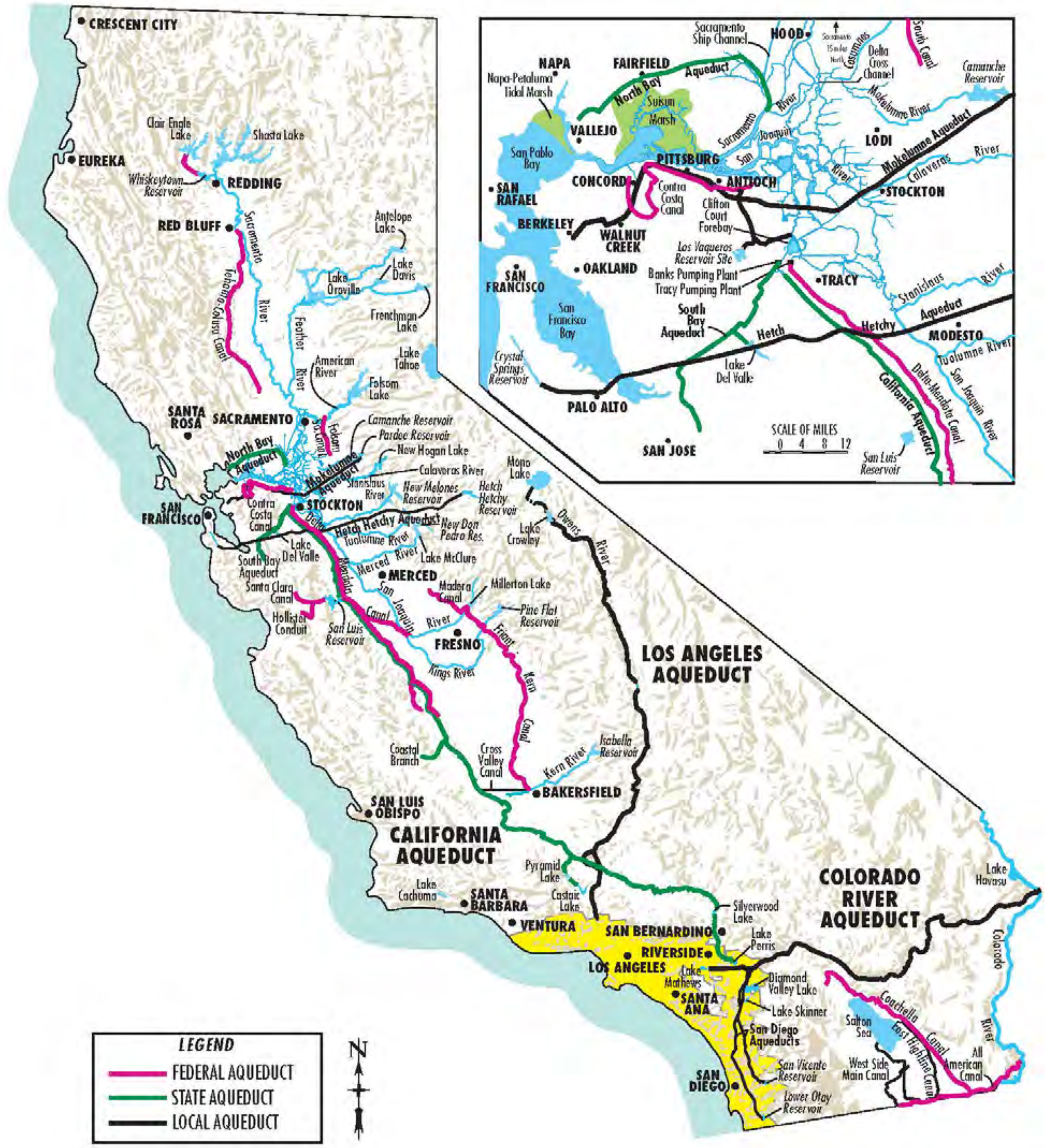
4.2 Water Authority Supply

Depending on the availability of local water, the District obtains as much as 90 percent of its potable water supply from the Water Authority. The Water Authority is one of the largest of 26 member agencies of MWD. MWD was formed in 1928 to develop, store, and provide wholesale distribution of supplemental water in southern California for domestic and municipal purposes. MWD's supplies come from two primary sources, the State Water Project, owned and operated by the California Department of Water Resources, and the Colorado River, via the Colorado River Aqueduct, as shown in **Figure 4-3.** Historically, the Water Authority has relied on imported water supplies purchased from MWD to meet the needs of its 24 member agencies.

After experiencing severe shortages from MWD during the 1987–1992 drought, the Water Authority began aggressively pursuing actions to diversify the region's supply sources. To reduce its dependency on MWD and diversify its supplies, the Water Authority undertook several initiatives, including the following:

- **Carlsbad Seawater Desalination Water Purchase Agreement:** To further help diversify regional supplies, the Water Authority entered into a Water Purchase Agreement under which it agrees to purchase up to 56,000 AFY of desalinated water from the Claude "Bud" Lewis Carlsbad Desalination Plant, which became operational in December 2015.
- **Imperial Irrigation District Transfer:** The Water Authority signed a Water Conservation and Transfer Agreement with Imperial Irrigation District in 1998. Through the transfer agreement, the Water Authority is purchasing water from Imperial Irrigation District at volumes that will gradually increase year to-year, reaching 200,000 AFY in 2021. The water is physically delivered to San Diego via MWD's Colorado River Aqueduct.
- **All-American and Coachella Canal Lining Conserved Water:** In 2003, as part of the execution of the Quantification Settlement Agreement on the Colorado River, the Water Authority was assigned rights to 77,700 AFY of conserved water from projects to line the All-American and Coachella Canals. These canal lining projects are now complete and the Water Authority is receiving this water. As with the Imperial Irrigation District transfer water, the water is physically delivered to San Diego via the Colorado River Aqueduct.
- **Water Transfer and Banking Programs:** In addition to the above, the Water Authority has entered into water transfer and water banking arrangements with Central Valley area agricultural agencies and groundwater storage interests. These projects are designed to make additional water available to the Water Authority during dry-year supply shortages from MWD.

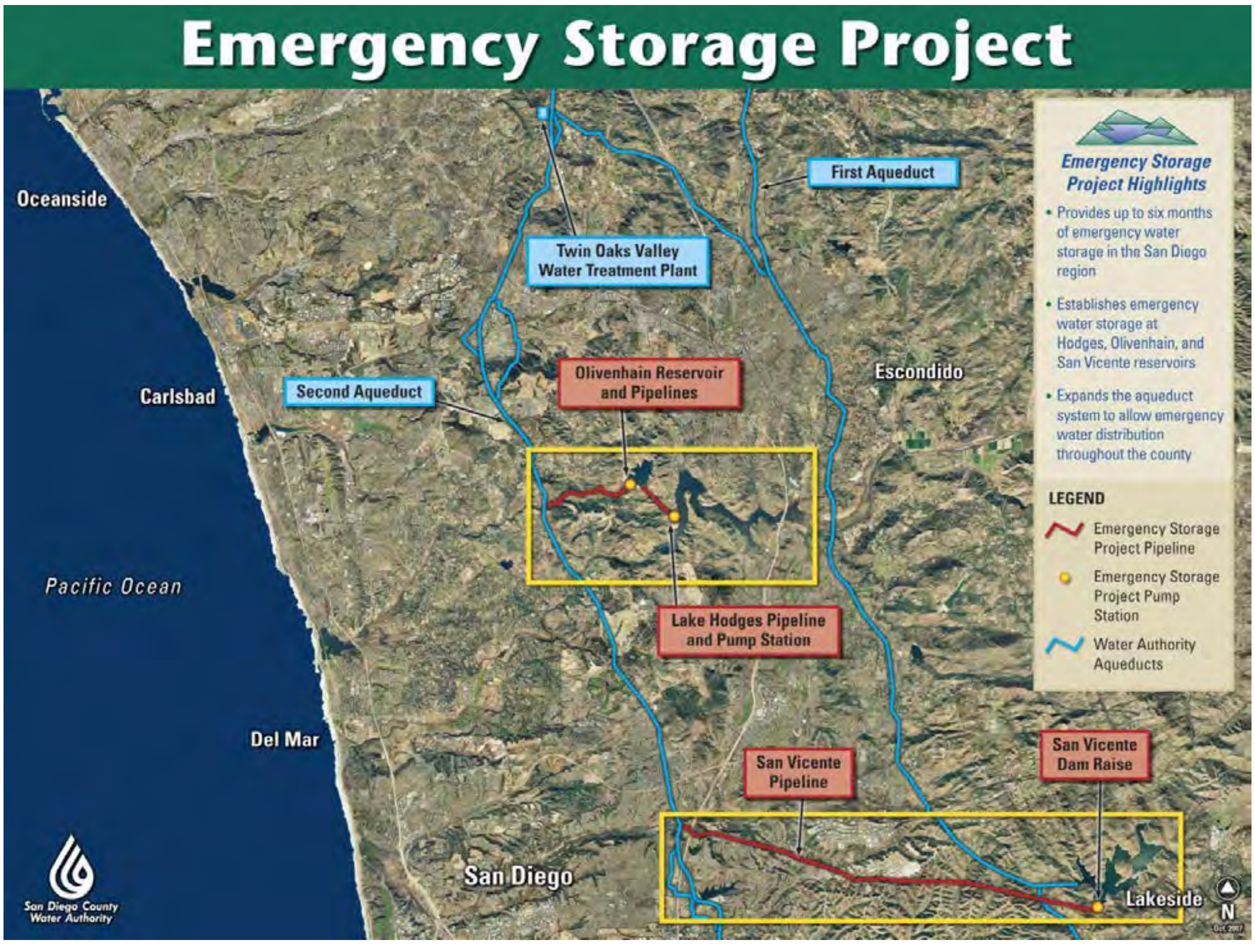
Figure 4-3. Major Water Conveyance Facilities in California



Source: Metropolitan Water District of Southern California (MWD). 2015.
http://www.mwdh2o.com/Who%20We%20Are%20%20Fact%20Sheets/6.4.2_Maps_Major_Water_Conveyance.pdf

In the early 1990s, recognizing the potential for a large earthquake or other emergency condition to cause a sustained outage of the pipelines, the Water Authority initiated the Emergency Storage Project (ESP) to safeguard against this risk. The primary objective of the ESP is to develop an emergency storage and delivery system able to provide 75 percent of 2-month peak water demand for all water users in the service area. This is referred to as the "2-month" emergency event. The major facilities of the ESP include the Olivenhain Reservoir and pipeline, the Hodges-Olivenhain Connection, the San Vicente Dam enlargement, and San Vicente–Miramar Pipeline, as shown in **Figure 4-4**. The largest components of the ESP facilities are now completed.

Figure 4-4. Emergency Storage Project



Source: The Water Authority (2017) https://www.sdcwa.org/sites/default/files/images/projects-facilities-ops/esp/esp-county-map-2.jpg.

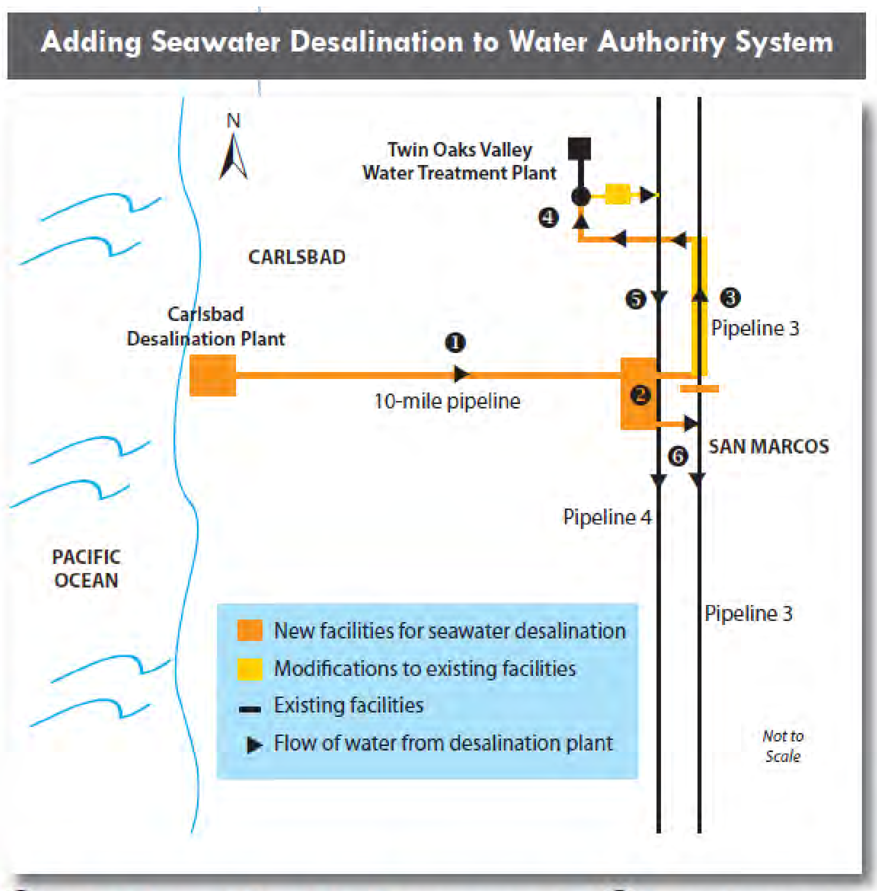
The Water Authority delivers treated and raw water from the State Water Project and the Colorado River into San Diego County through five large diameter pipelines, located in two principal corridors known as the First and Second San Diego Aqueducts. The system has evolved over time to serve the growing needs of the region. The aqueduct pipelines connect to both filtered and raw water feeds from MWD facilities at Lake Skinner, in southern Riverside County.

The First Aqueduct, Pipelines 1 and 2, delivers filtered water to the northeastern portion of San Diego County. Prior to 1992, Pipelines 1 and 2 provided raw water to the City of

Escondido. In March 1992, the Water Authority converted the northerly portion of Pipelines 1 and 2 to deliver filtered water, and connected the southern portion of Pipelines 1 and 2 to a different raw water supply: Pipeline 5, via a cross-over pipeline, which provides a source of supply for the EVWTP. Currently, delivery of filtered water from Pipelines 1 and 2 ends at the delivery points to the District and Rincon del Diablo Municipal Water District and the Hubbard Hill Overflow.

The Water Authority's Second Aqueduct is located west of the First Aqueduct and includes Pipelines 3, 4 and 5. Pipeline 5 delivers raw water and Pipeline 4 delivers filtered water from MWD's Lake Skinner Water Treatment Plant (WTP). As part of the incorporation of the Carlsbad Desalination Plant facilities, the Water Authority converted Pipeline 3 to convey the treated water northward to the Water Authority's regional facilities in Twin Oaks Valley. From there, this new supply blends with existing imported supplies in Pipeline 4 to enhance the reliability regionally, as shown in **Figure 4-5.** Currently there is no independent connection for the District to access desalinated water, although one is being planned for the City of Carlsbad.

Figure 4-5. Pipeline 3 Desalination Conversion



1 Desalinated seawater flows to San Marcos in new pipeline. 2 New pipeline control facilities send water north into Pipeline 3. 3 Upgraded Pipeline 3 delivers water to regional hub at Twin Oaks. 4 Improvements at Twin Oaks plant blend desalinated water with existing treated water supplies. 5 Water flows south in Pipeline 4 to control facility 2 and then continues southward into Pipelines 3 and 4. 6

Source: http://www.sdcwa.org/sites/default/files/pipeline3-desal-relining-FS.pdf

4.3 Water Authority and Interagency Connections

The District maintains six flow control facility connections to the Water Authority Aqueducts delivering filtered water, as shown on **Figure 4-6** and in **Table 4-1.**

Table 4-1. District Water Supply Connections

Connection	Aqueduct and Feed	Capacity (cfs)	Capacity (MGD)
VID 1	First Aqueduct, Pipelines 1 and 2	10	6.5
VID 3	Second Aqueduct, Pipelines 3 and 4	30	19.4
VID 8	Second Aqueduct, Tri-Agency Pipeline, Pipelines 3 and 4	5	3.2
VID 9	Second Aqueduct, Tri-Agency Pipeline, Pipelines 3 and 4	20	13.0
VID 10	Second Aqueduct, Tri-Agency Pipeline, Pipelines 3 and 4	15	9.7
VID 11	Second Aqueduct, North County Distribution Pipeline, Pipeline 4, Weese WFP	50	32.3

cfs – cubic feet per second; MGD – million gallons per day; VID – Vista Irrigation District; WFP - Water Filtration Plant

In addition to its primary supply connections to the Water Authority Aqueducts, the District also has emergency connections to neighboring water agencies. These interconnections allow the District to be supplied by its neighbors during times when its supply from the Water Authority is interrupted. In some cases, the interconnections also allow the District to reciprocate by providing water to a neighboring agency should the need arise. These connections are shown in **Figure 4-6** and are described in **Table 4-2**.

The City of Oceanside Water Utilities Department purchases imported raw water from the Water Authority and treats it at the 25 MGD Robert A. Weese (Weese) Water Filtration Plant (WFP). The District's intertie with Oceanside provides the District with a potential access to this locally treated water in the event of an emergency outage of the District's other water supplies.

VWD and the City of Carlsbad have or are planning to construct connections to the Desalination pipeline. Interagency connections with these neighboring agencies may provide the District with emergency access to desalinated water supply sources if the Water Authority Aqueducts are out of service.

Figure 4-6. Water Authority and Interagency Connections

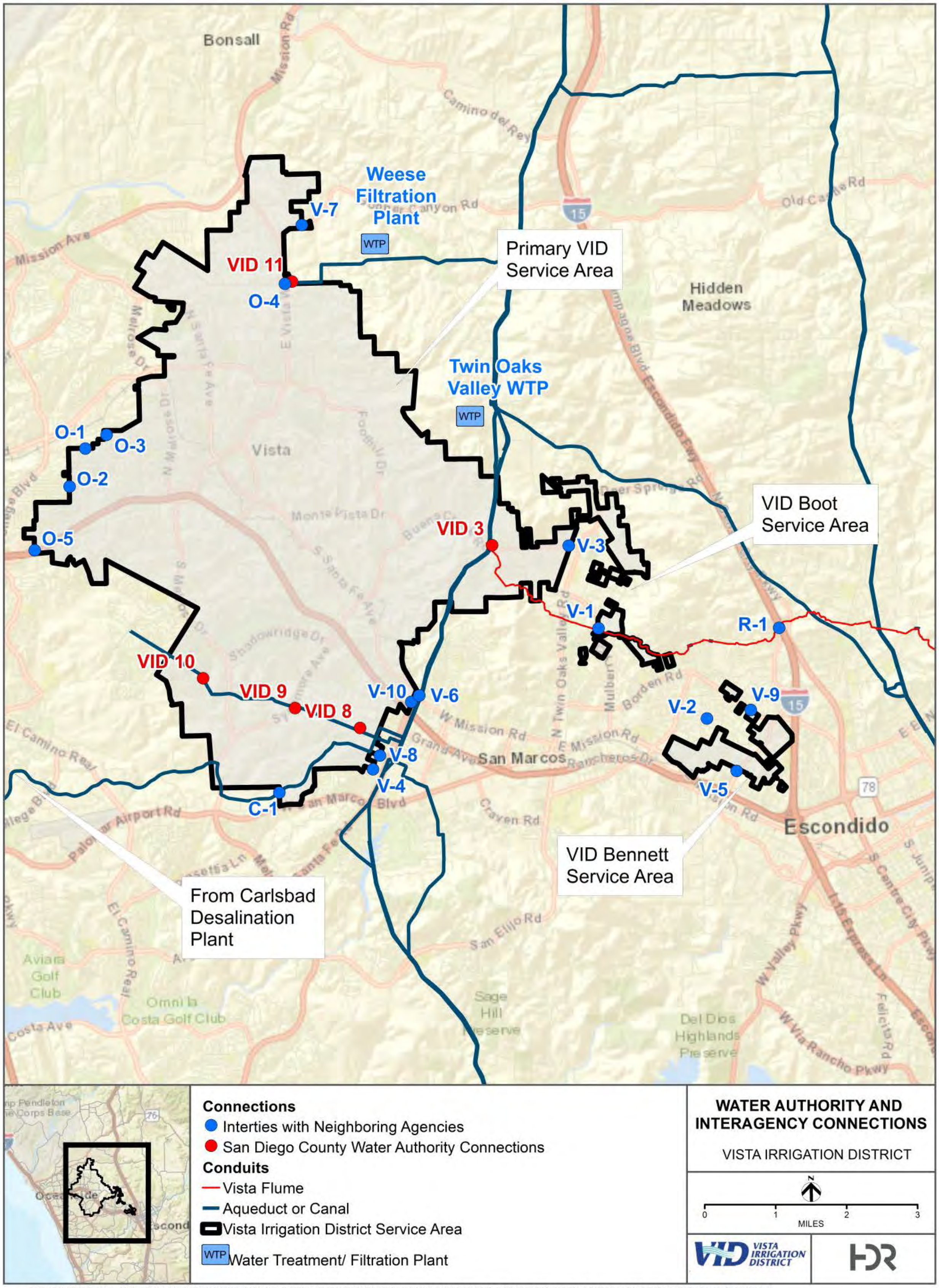


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Table 4-2. Interagency Connections

Map ID (See Figure 4-6)	Location	Connecting Agency	Type	Year Installed	Pipe Size (Inches)	Meter Size (Inches)	Service From	Service To	VID Zone	Connecting Agency Zone	Approx. Flow Rate (gpm)
O-1	Fall Place and Olive Avenue	Oceanside	Supply	1985	6	6	VID	Oceanside	565	511	500
C-1	Lionshead and Poinsettia Avenues	Carlsbad	Supplement	2005	8	6	Carlsbad	VID	707	700	750
R-1	Nutmeg Street at Caldwell Siphon	Rincon del Diablo Municipal Water District	Supplement	1993	8	8	Rincon del Diablo Municipal Water District	VID	Flume (899')	1000	500
V-1	Mulberry Drive and Woodward Street	VWD	Supplement	1995	6	6	VWD	VID	850	920	500
O-2	561 Emerald Drive	Oceanside	Emergency	1983	6	Closed GV	Oceanside	VID	486	600	500
O-3	853 Granada Drive	Oceanside	Emergency	1979	8	8	VID	Oceanside	565	526	500
O-4	Osborne Street and East Vista Way	Oceanside	Emergency	1971	10	Closed GV	VID	Oceanside	810	511	1,000
O-5	Thunder Drive and West Vista Way	Oceanside	Emergency	1963	6	Closed GV	Oceanside	VID	486	511	500
V-2	1440 Rocksprings Road	VWD	Emergency	1959	6	Closed GV	VWD	VID	980	920	500

Table 4-2. Interagency Connections

Map ID (See Figure 4-6)	Location	Connecting Agency	Type	Year Installed	Pipe Size (Inches)	Meter Size (Inches)	Service From	Service To	VID Zone	Connecting Agency Zone	Approx. Flow Rate (gpm)
V-3	215 Buena Creek Road	VWD	Emergency	1968	10	8	VWD	VID	976/ 984	1,028	2,000
V-4	3870 First Street	VWD	Emergency	1970	6	Closed GV	VWD	VID	837	855	500
V-5	851 Nordahl Road	VWD	Emergency	1990	10	Closed GV	VWD	VID	980	920	500
V-6	Capalina and Rancho Santa Fe	VWD	Emergency	1971	8	Closed GV	VWD	VID	837	855	500
V-7	Fairview Drive and Gopher Canyon Road	VWD	Emergency	2003	8	6	VWD	VID	810	900	500
V-8	Linda Vista Drive (VWD Reg. Vault)	VWD	Emergency	1995	8	6	VWD	VID	837	920	500
V-9	Rees Road and El Norte Parkway	VWD	Emergency	1995	8	6	VWD	VID	898	920	1,000
V-10	South Santa Fe and Rancho Santa Fe	VWD	Emergency	2006	8	6	VWD	VID	837	920	500

gpm – gallons per minute; GV – gate valve; VID – Vista Irrigation District; VWD - Vallecitos Water District

4.4 Supply Reliability – Water Shortage Events

The Water Authority conducts scheduled shutdowns of sections of its regional water supply pipeline for internal inspection, maintenance, and capital improvements on an annual basis. These shutdowns are typically scheduled during low demand, winter months, although early fall shutdowns do occasionally occur. The District and other member agencies receive advanced notice of these planned shutdowns so that they can be prepared to serve their customers using alternative supply sources or stored water. In 2005, shutdowns to both the First and Second Aqueducts occurred simultaneously, precluding the District from relying on its connections to either the First or Second Aqueduct.

The Water Authority recommends that its member agencies maintain 10 days of storage or alternative supply in order to be self reliant during routine maintenance on the aqueducts. With the exception of emergencies, the maintenance typically occurs during winter months when demands are low. Minimum day demands (MinDD) are typically 50 percent of the AAD. As discussed in **Chapter 3**, the District's current AAD is approximately 17 MGD, and projected buildout is 20 MGD. With a MinDD of 8.5 to 10 MGD, 10 days of storage would require the District to have 85 to 100 MG of storage capacity. The District currently has just over 40 MG of storage capacity. Note this analogy assumes all system storage could be used. In reality, 30 to 40 percent of the system storage would be needed at the end of the 10-day outage for the distribution system to operate effectively.

To offset a planned outage of the Water Authority aqueducts, the District relies on its capacity rights for 18 MGD at the EVWTP, which can be conveyed to the District via the Vista Flume. Meeting a 10-day outage of the Water Authority aqueduct by depending on the District's local supply via the Vista Flume has been sufficient to meet the Water Authority independence criteria. However, if the Vista Flume were out of service, the District would rely on its 40 MG of storage capacity, which would only provide 4 to 5 days of supply. Storage supplies could potentially be supplemented through connections with neighboring agencies that have excess storage or independent water supplies.

If the Vista Flume were out of service under buildout maximum day conditions, the District would be dependent on Water Authority for approximately 40 MGD, or 62 cfs of supply. Assuming that an outage of the Vista Flume would preclude the use of the District's connection to the First Aqueduct (VID 1), the District would depend on its five connections to the Second Aqueduct. These connections, as described in **Section 4.3**, have a total capacity to deliver over 100 cfs, which is sufficient to offset supply from the Vista Flume. The Boot and Bennett areas would need to be served by interagency connections with VWD and/or Rincon del Diablo Municipal Water District.

Table 4-3 summarizes the types of water shortage events that could affect the District, the assets currently available to the District to address the shortage event, and the consequences of each event to the District with existing assets. **Section 4.5** expands on the District's opportunities to enhance supply reliability should the Vista Flume be out of service.

Table 4-3. Summary of Potential Shortage Events and Consequences

Event Existing	Frequency	Duration	Response Assets	Consequence
1) Drought (or other prolonged reduction in imported water supplies and local resources)	Unknown (Imported delivery reliability is dependent on State, MWD, and Water Authority actions)	1 year and longer	a) State, MWD, and Water Authority response capabilities b) District drought response ordinance and rate structure	**Significant** (Cutbacks to District treated water customers at same level as Water Authority cutbacks to District)
2) ESP Event (Earthquake induced or other failure of all or most of the San Diego Aqueduct pipelines)	Low (on the order of one event per 100 years)	2 months (per ESP design criteria, based on aqueduct repair time estimates)	a) EVWTP via the Vista Flume b) Water Authority ESP facilities, Carlsbad Desalination Plant and Twin Oaks WTP c) District Treated Water Storage d) District interties with neighboring agencies e) District Water Shortage Contingency Plan	**Moderate** (No Water Authority deliveries for 4 to 7 days; thereafter deliveries at minimum 75% level of service)
3) Treated Water Shutdown of First and/or Second Aqueducts (planned event)	Annually (approximately)	10 days (typically during winter months)	a) EVWTP via the Vista Flume b) District Treated Water Storage c) District interties with neighboring agencies	**Minor** (Possible drawdown of District storage to below preferred levels)
4) Outage of EVWTP or Vista Flume	Low (on the order of one event per 50 years, assuming ongoing maintenance and rehabilitation of the Vista Flume)	2 to 6 months (based on repair time estimates)	a) Water Authority Aqueduct and ESP facilities, Carlsbad Desalination Plant and Twin Oaks WTP b) District interties with neighboring agencies c) District Water Shortage Contingency Plan	**Moderate** (No deliveries from EVWTP or First Aqueduct for duration)

ESP – Emergency Storage Project; District - Vista Irrigation District; EVWTP - Escondido-Vista Water Treatment Plant; MWD - Metropolitan Water District of Southern California; Water Authority – San Diego County Water Authority; WTP – water treatment plant

4.5 Assessment of Water Reliability Improvement Concepts

The District has connection capacity to the Water Authority aqueduct system and the EVWTP, via the Vista Flume, that significantly exceeds its current and projected AAD. This surplus capacity provides operational flexibility to accommodate peaking and to allow for one or more of the District's aqueduct connections to be off-line. However, given the age and current condition of the Vista Flume, the District has concerns regarding the long term viability of this important conveyance system.

As documented in the *Historic American Engineering Level Written Documentation - Vista Irrigation District Main Water Conveyance* prepared for the District in November 2016, the Vista Flume was originally constructed in 1926 and underwent a significant repair and maintenance program in 1947 through 1955. During that time, 7 miles of the flume were covered with a reinforced concrete arch and 4 miles of steel siphon sections were lined with concrete mortar. In the 1980s, repairs to the cover were made and HDPE liners were installed to reduce seepage. Inspections in the 1990s noted seepage at the bench sections and overall susceptibility of the flume to service interruption from rock slides or seismic activity. In 2005, upgrades were made to the bench sections and, in 2010, the District successfully conducted a pilot project to line the MW Bench with HDPE pipe.

In March of 2012, the District conducted a condition assessment of the flume, as well as a cost of water evaluation. The study concluded that rather than rehabilitation of the flume bench sections with HDPE pipe, the District's least expensive option was to internally repair the roofs with grout, which would extend the flume's life 20 to 30 years. The study's estimated cost for this work was approximately $4 million ($140/foot). The study also recommended relining all the siphons, with an estimated cost of $7 million ($230/foot).

Following the study, the District has pursued the recommended roof repairs and found it difficult to obtain bids for such work. Additionally, the repairs do not address the ongoing maintenance required on the existing HDPE liner and exterior portions of the flume, where cracking between the roof and walls is prevalent. As such, the District considers the internal roof repair recommendations to only be a partial and short-term solution, where full slip-lining or replacement would be an appropriate avenue for the long-term.

The District has recently been involved in additional flume projects, including an HDPE slip-line design for the Meyer's Siphon, relocation, and replacement construction of the Baumgartner Bench and Siphon with a new HDPE siphon, and an alternatives study for the rehabilitation/replacement of the Beehive Bench and Siphon. Based on these recent rehabilitation/replacement projects, the District has found a wide range of unit costs associated with long-term solutions for the Vista Flume. The actual cost to relocate the Baumgartner Bench and Siphon with a new 42-inch HDPE siphon, as part of a new residential development, was approximately $500/foot. Estimated costs to HDPE slip-line or epoxy line the Meyer's and Beehive Siphons are between $800 and $1,000/foot, and the range to rehabilitate or replace the Beehive Bench is between $1,500 and $1,900/foot. This all equates to an expensive price tag for a long-term rehabilitation or replacement solution for the entire remaining 10 miles of the Vista Flume (projected between $36 and $75 million).

Given the potential costs to replace the Flume, this section explores water supply reliability improvement opportunities that could potentially offset a short-term outage or permanent abandonment of the Vista Flume.

4.5.1 Opportunities to offset a 10-day Aqueduct Outage

As noted in the **Section 4.4**, meeting a planned 10-day outage of the Water Authority aqueduct systems by depending on the District's local supply via the Vista Flume has been sufficient to meet the Water Authority independence criteria. However, if the Vista Flume were out of service, the District's 40 MG of storage capacity would provide only 4 to 5 days of supply during winter (minimum) day demands. If the Vista Flume were out of service for a longer period of time, the District would be primarily reliant on Water Authority service connection VID 3 to directly serve the District's highest zones. Outage of the Vista Flume, west of the Kornhauser Bench, also precludes the District from access to the Water Authority's First Aqueduct system at VID 1.

Opportunities to mitigate outage of the Vista Flume during a planned 10-day aqueduct outage include the following:

New Water Authority Isolation Valve(s) on Second Aqueduct's Treated Water System

Pipeline 4, the sole treated water pipeline of the Second Aqueduct north of the Twin Oaks Diversion Structure, is subject to occasional planned shutdowns for inspection, maintenance, and installation of new connections. The Water Authority provides an updated Annual Operating Plan in June to reflect anticipated operational opportunities and constraints for the upcoming FY, and to evaluate performance for the prior FY. The Annual Operating Plan includes the Water Authority's anticipated operating schedules and WTP outages. The Annual Operating Plan is developed based on information received from member agencies, historical delivery/production data, capacity constraints within the Water Authority's aqueduct system, and scheduled shutdowns. For FY 2018, the Water Authority had one planned outage of the entire Second Aqueduct's treated water system, between November 5 and 14, 2017. The primary reason for the shutdown was to support activities related to asset management and warranty inspections of the Carlsbad Desalination Plant.

During Second Aqueduct treated water shutdown events, the District relies on the Vista Flume to deliver supply from the VID 1 connection to the First Aqueduct and treated water from EVWTP. In addition, VID 11 can deliver treated water from the Weese WFP, if available from the City of Oceanside.

The Water Authority's Twin Oaks WTP provides a possible additional means of supplying water to the District during a treated water aqueduct shutdown. The plant receives raw water from Pipelines 3 and 5, and then treats the water for delivery back to Twin Oaks Diversion Structure and hence into the treated water aqueduct pipelines south of the Diversion Structure. However, during a treated water shutdown, if Pipeline 4 north of the Diversion Structure is drained for inspection or maintenance, the plant cannot deliver back to the Diversion Structure without flooding Pipeline 4 to the north, and therefore cannot operate during this situation. Likewise, desalinated water from the Carlsbad Desalination Plant is conveyed to this Diversion Structure prior to being introduced to Pipeline 4, and therefore is also unavailable during a Pipeline 4 shutdown.

In FY 2008, the Water Authority considered installing an isolation valve in Pipeline 4 just north of the Twin Oaks WTP and just south of the North County Distribution Pipeline connection, to allow the Twin Oaks and Carlsbad Desalination Plants to operate during a Pipeline 4 shutdown. This isolation valve would also allow treated water in Pipeline 4 to continue to serve the North County Distribution Pipeline, including VID 11, in the event that maintenance on the Carlsbad Desalination and Twin Oaks facilities required shut down.

Another potential option would be to install isolation valves between Twin Oaks WTP and the Carlsbad Desalination Plant. This could give the Water Authority additional operational flexibility and allow VID 3 and/or the Tri-Agencies Pipeline to remain in service.

Any of these valve options could greatly reduce the consequences of a Second Aqueduct treated water system shutdown. Because of the important benefit that this would provide the District, as well as other Water Authority member agencies, it is recommended that the District encourage the Water Authority to pursue any aqueduct or treatment facility projects that provide operational flexibility and eliminate the need to shut down the entire treated water system.

Additional District Storage

The District currently operates 40 MG of potable water storage, which complies with the District's storage criteria, described in **Chapter 5**. To be completely independent during a planned 10-day outage in winter months, the District would require approximately 85 MG, increasing to 100 MG at projected buildout. The current deficit is approximately 45 MG, increasing to 60 MG at projected buildout.

The District owns approximately 16 acres of property along Buena Creek Drive, adjacent to the 20 MG Pechstein Reservoir site. The District purchased this property having anticipated additional District storage at this elevation may someday be necessary. **Figure 4-7** provides an aerial view of the site, and illustrates the availability of space within the property boundaries to locate three additional 20 MG tanks, which would fully address the projected 10-day emergency storage deficit of 60 MG. This assumes all storage could be used, when additional storage would actually be required for the system to operate effectively by the end of the 10th day. To prepare for a planned outage, the District would fill these tanks prior to the planned event, via the VID 3 connection.

At a planning level cost of $1.50 per gallon, these additional tanks would require an investment of $90 million. While advantageous during an aqueduct outage, day to day use of all three tanks could cause water age and quality issues. However, investment in one new 20 MG tank (Pechstein II) would partly mitigate the District's need for storage during annual aqueduct shutdowns by providing an additional 2 days of storage. This new tank would also provide complete redundancy for the existing Pechstein Reservoir, so that it could be taken out of service for operation and maintenance activities.

Figure 4-7. Pechstein Reservoir Site Property Map

Local Interagency Connections - Oceanside

In 2013, the District entered into an agreement with the City of Oceanside for the sale of water from the Weese WFP. The agreement allows the District to purchase up to 5 MGD from November 1 to April 30 and up to 2.5 MGD from May 1 through October 31, totaling up to 4,150 AFY. The current treatment cost is $141.75/AF and is escalated on July 1 of each year by the consumer price index. The agreement is automatically renewed each year unless terminated by Oceanside or the District by giving 6 months advanced notice. Built in 1983, the plant is capable of treating up to 25 MGD and delivering that supply to the Water Authority's North County Distribution Pipeline. The North County Distribution Pipeline can be operated independent of treated water aqueduct Pipeline 4, as long as there is a supply of raw water to the Weese WFP. The District's VID 11 connection draws its supply from this pipeline. During winter months the Weese WFP has excess capacity.

In July 2017, the City of Oceanside presented a staff report to its Utilities Commission, proposing an amendment to the 2013 agreement that would allow the District to purchase up to 3.3 MGD from November 1 to April 30 and up to 5 MGD from May 1 through October 31, totaling up to 4,440 AFY. As Oceanside expands its plans for recycled water production and use, Oceanside Pubic Utilities staff noted that additional capacity at the Weese WFP may become available. Oceanside estimates that the cost of the treated water would be at least $180 per AF. As delivery would be through the Water Authority's regional facilities, the District would be billed directly by the Water Authority for the cost of untreated water. Similar to the current agreement, this new agreement would be renewable on a year to year basis and can be cancelled by either agency with 6 months advanced notice.

This local supply would offset 33 MG (3.3 MGD x 10 days) of the estimated 60 MG of new storage needed for the District to be completely independent during a 10-day treated water aqueduct outage. Even if this supply were only used during the 10-day outage, this almost $20,000 annual purchase ($180 per AF x 33 MG x 3.07 AF/MG = $18,235) would certainly be advantageous over the construction of 33 MG of new storage at $1.50 per gallon or $49.5 million.

In the event that the Vista Flume outage becomes long term, this alternative allows the District to transfer a portion of its purchase of raw water from the Water Authority that is currently sent to the EVWTP, but would be inaccessible if the Flume was out of service, to the Weese WFP. At a cost of $180 per AF, the annual cost for 4,440 AFY of local water would be $0.8 million per year. This cost may be offset if the District were to sell or lend its capacity at the EVWTP, which it would no longer be using; however, a long-term water purchase agreement with Oceanside would be required.

Local Interagency Connections – Vallecitos Water District

The VWD lies east and south of the District and shares 10 emergency service connections with the District's system. Only one connection, V-3 on Buena Creek Road, is located such that it could serve the District at an equivalent elevation to the District's VID 3 connection to the Water Authority aqueduct. The V-3 connection allows flows up to 2,000 gpm or 2.88 MGD. As VWD has approximately 30 MGD of potentially excess storage capacity in its Twin Oaks Reservoirs, in the event of a planned 10-day outage of the aqueduct system it is possible that the District could negotiate access to that capacity, assuming VWD has the capability to deliver the water from the Twin Oaks Reservoirs to V-3.

This local emergency supply source would offset 28 MG (2.88 MGD x 10 days) of the estimated 60 MG of new storage needed by the District to be completely independent during 10-day treated water aqueduct outage. Similar to the Oceanside supply source, this opportunity is likely to be significantly less costly than constructing new storage facilities.

4.5.2 Opportunities to Provide Redundancy to Vista Irrigation District 3 Connection

With a long term outage of the Vista Flume, the VID 3 Connection is the sole connection to the Water Authority aqueduct that can directly feed the Pechstein Reservoir, which serves the 837/810 zones. VID 8, VID 9, and VID 11 all feed this zone; however, because of the distance from Pechstein, a significant amount of pressure is required to account for the elevation difference and head loss that occurs when trying to deliver water to the Pechstein Reservoir.

Two alternatives were considered to serve Pechstein Reservoir from VID 9 or VID 11, if both the Flume and VID 3 were out of service. These alternatives are discussed in the paragraphs below.

E Reservoir Expansion and New Pump Station

VID 11 serves the 837/810 zone, which in turn feeds the E Reservoir and the 752 zone. Although they serve different pressure zones, the E Reservoir is located in close proximity to the HP 5.0 MG Reservoir, which serves the 984/976 zone and has recently been rehabilitated. Shown in **Figure 4-8**, the E Reservoir is located on a 1.55 acre parcel adjacent to Edgehill Road. The 1.5 MG concrete tank is below ground, oval in shape, 96 feet wide and 244 feet long. Built in 1929, the tank is scheduled for near term replacement. In 1995, the District conducted an initial environmental assessment for replacing the E Reservoir with a 146-foot diameter, 35 feet deep prestressed concrete reservoir with a capacity of 4.4 MG.

Replacement of the E Reservoir and the addition of a pump station feeding the higher zones (e.g., HP Reservoir) would provide a redundant means of getting 30 cfs of water from VID 11 to Pechstein Reservoir, in the event that VID 3 was out of service. This opportunity also provides a means of delivering any additional supply from the Weese WFP via VID 11 to the District's higher pressure zones. The hydraulic requirements and facilities needed to implement this opportunity are discussed in detail in **Chapter 8**.

Figure 4-8. E Reservoir Property Map

In the 1995 *Master Plan and Program Environmental Impact Report*, the 5 MG E Reservoir replacement project included raising the high water level of the tank by 25 feet to an overall height of 38 feet, with the west side of the tank being entirely above grade. Along the east side, 23 feet of the tank would be subterranean. The program environmental impact report noted the existence of sensitive habitat along the northwest corner of the site, potentially requiring mitigation. Based on the storage capacity evaluation conducted in **Chapter 7**, increasing storage in this location would be beneficial in serving the 752 zone; however, the site has limited room for expansion and close neighbors such that raising the height of the reservoir may be challenging. The new site plan would also need to include space for a pump station.

Delivery from Vista Irrigation District 11 and Vista Irrigation District 9 to Pechstein Reservoir

A hydraulic analysis was conducted to determine the capacity of the existing system to offset the VID 3 supply of 30 cfs to Pechstein through a balance of supply from VID 11 and VID 9. The goal of the iterative analysis was to maximize flow from VID 9 into the 837/810 zone with sufficient pressure to reach Pechstein, but without creating high pressures in the system and then allowing the balance of the 30 cfs flow to come from VID 11, again without creating high pressures in the system. The hydraulic requirements and facilities needed to implement this opportunity are discussed in detail in **Chapter 8**.

4.5.3 Recommended Opportunities for Further Study

It has been noted that complete rehabilitation of the Flume and the alternative of construction of 60 MG of new storage facilities are both quite costly, in excess of $36 to $75 million and $90 million, respectively. This section identified a number of alternatives that, when combined, provide sufficient supply redundancy to offset the Flume being out of service either short term or long term.

The following opportunities for adding redundancy, reliability, and operational flexibility are recommended for further detailed study.

1. Continue to advocate for the installation of new isolation valves on the Second Aqueduct treated water system or other operational flexibility projects with the Water Authority.
2. Construct a new 20 MG storage tank at the Pechstein Reservoir site to provide 2 days additional storage and operational redundancy to the existing tank.
3. Enter into a long-term agreement with the City of Oceanside to gain access to excess treated water capacity at the Weese WFP for at least 3.3 MG during winter months and 5 MG during summer months.
4. Negotiate an agreement with VWD for access to excess storage capacity (up to 28 MG) during a 10-day Water Authority planned outage.
5. Maximize use of capacity within the existing system to allow supply from VID 11 and/or VID 9 to reach Pechstein Reservoir.

4.6 Recycled Water Coordination

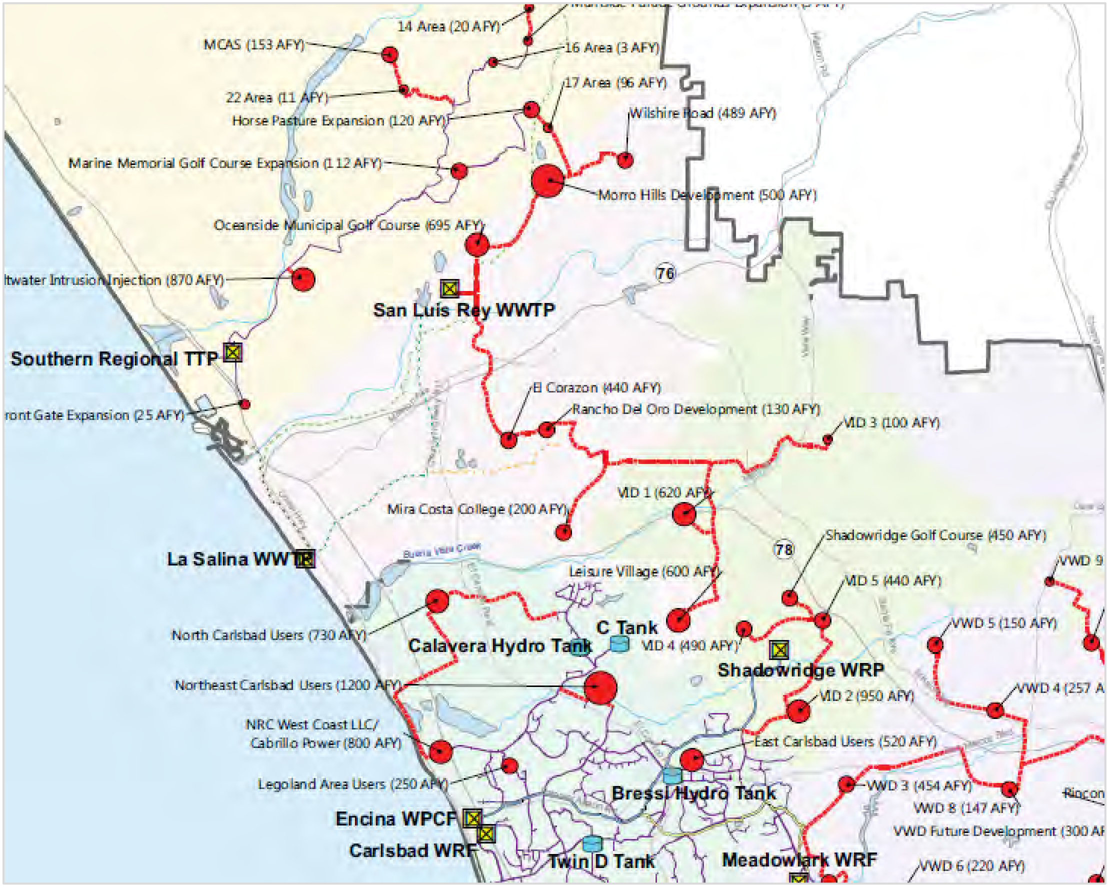
The Shadowridge Water Reclamation Facility (WRF) was built in 1986 to provide wastewater treatment for the Shadowridge development and recycled water service for golf course irrigation. The facility was owned and operated by Buena Sanitation District. For failsafe capacity, the Buena Outfall was constructed. In August 1995, the District's Board of Directors approved a Water Reclamation Master Plan, with a goal of reducing potable water demand by providing recycled water to certain targeted customers. The Water Reclamation Master Plan identified approximately 2,200 AF of recycled water demand that could be available for distribution within the District's service area on an annual basis. This plan required significant investments in treatment, storage and distribution infrastructure by the City of Vista and the Buena Sanitation District, and was never implemented.

In 2003, the Shadowridge WRF was decommissioned, as treatment capacity became available at Encina, and it was no longer financially feasible to operate the WRF. Currently, there is no recycled water being delivered to customers in the District's service area. A study prepared in August 2010 estimated that the capital cost to renovate/expand the mothballed Shadowridge WRF to 2.0 MGD and make the plant operational would cost approximately $17.9 million.

In June 2010, the District joined with the Olivenhain Municipal Water District, Carlsbad Municipal Water District, VWD, Santa Fe Irrigation District, City of Oceanside, Leucadia Wastewater District, City of Escondido, Rincon Del Diablo Municipal Water District and the San Elijo Joint Powers Authority to form a coalition (the North San Diego Water Reuse Coalition) to investigate the expanded use of recycled water within north San Diego County. The Coalition has had an engineering report prepared that analyzed existing and proposed recycled water facilities and evaluated each of the participating agencies ability to interconnect and maximize the use of recycled water within their combined service areas.

The 2013 North County Regional Recycled Water Facilities Plan identified a potential recycled water demand of 1,840 AFY (including the Shadowridge Golf Course) and considered using the Shadowridge WRF failsafe outfall as a conduit for delivering recycled water from the City of Carlsbad to the District. The long term potential recycled water demand was estimated to be over 3,000 AFY. The facilities required included significant investment in pipeline facilities to reach the proposed recycled water customers, as shown in **Figure 4-9**.

Figure 4-9. Proposed Regional Recycled Water Facilities



Source: 2013 North County Regional Recycled Water Facilities Plan

The option presented included extension of the recycled water distribution system from the Oceanside San Luis Rey Wastewater Treatment Plant to two potential groups of recycled water customers within the District: VID 3 (100 AFY) and VID 2 (620 AFY). A second pipe extension was proposed from the Carlsbad recycled water system to serve the Shadowridge Golf Course (450 AFY), VID 2 (950 AFY), VID 4 (490 AFY) and VID 5 (440 AFY). Based on a rough estimate of cost sharing, the District would be responsible for as much as 40 percent of the 10-mile Oceanside system extension, serving only 720 AFY, and 100 percent of the 8-mile Carlsbad system extension, serving 2,330 AFY.

Assuming these pipes were on average 10-inch diameter pipes, at a cost of $325 (including engineering design costs and contingencies), that the District would pay their share of the pipeline extensions and pay their neighboring agencies for retail recycled water rates, and subsequently charge their customers the potable rate of water until the investment was paid off, the payback rate was estimated to range from 12 to 28 years. This return on investment calculation is provided in **Table 4-4**.

The return on investment to serve these same customers, who in 2016 are only using 10 percent of the amount of water they were in 2014, would be increased ten-fold.

In 2015, an alternative proposal was presented to the District Board to construct a recycled water pipeline from Oceanside's El Corazon WRP to the Ocean Hills Golf

Course, and possibly extending it to the Shadowridge Golf Course, as well as other potential irrigation customers along the Melrose Drive corridor. The Shadowridge Golf Course recently drilled a groundwater well on their property and removed turf in order to reduce its demand on potable water, which negatively impacted the economic feasibility of the project. The District subsequently agreed to allow transfer of Round 2 Proposition 84 construction grant funding for this proposed project to the City of Oceanside.

Given the significant drop in water use for the District's potable water customers that were being considered for conversion to recycled water, in addition to the Shadowridge Golf Course going to well water, the Board's decision appears to have been a prudent one.

Table 4-4. North County Recycled Water Project Return on Investment

Recycled Water Project	Cost Share to VID (%)	Length of Pipe (Miles)	Size of Pipe (Inches)	Unit Cost ($/foot)	Total VID Cost (Millions)	Recycled Water Served (AF)	Unit Cost of Recycled Water ($/Hundred Cubic Feet)	Unit Cost of Recycled Water ($/AF)	VID Potable Water Cost ($/AF)	Difference in Unit Cost ($/AF)	Years Required to Recoup Pipeline Investment
Oceanside Recycled Water Pipe Extension	40	10	10	325	6.864	720	2.42	1,054	1,812	758	12.58
Carlsbad Recycled Water Pipe Extension	100	8	10	325	13.728	2,330	3.69	1,607	1,812	205	28.74

AF – acre feet; VID – Vista Irrigation District

5 Planning and Design Criteria

The District's planning and design criteria for potable water facilities are based on past criteria used by the District, criteria obtained from the 2000 Master Plan and current industry and area standards.

Planning and design criteria include standards for peaking factors, pressure, velocity, storage, and fire flow. These criteria are the basis for evaluating water system performance and determining facility requirements to serve future development. **Table 5-1** displays the system design criteria summary for the District's water facilities. The following sections expand on these criteria.

5.1 System Pressure Criteria

The range of water pressures experienced at any location is a function of hydraulic grade and the service elevation. Within a specific pressure zone the hydraulic grade is affected by the reservoir water level and/or pressure reducing valve setting and the headloss in the distribution system. The maximum desired pressure is 150 pounds per square inch (psi). This criteria limits pressures in the distribution system and deliveries to customers for operational and maintenance purposes.

The criteria for minimum desired pressure in residential areas is 40 psi under peak hour flow conditions and 20 psi at a fire flow location during a fire occurring under maximum day flow conditions. The minimum pressure in the distribution system must be 20 psi based on Health Department guidelines and the ability to provide adequate pressures for fire flows.

5.2 Pipeline Criteria

Criteria for pipeline sizing are based on keeping fluid velocities low to minimize wear on valves and scouring of interior coatings, and limiting headloss in the distribution system. Water distribution mains should supply peak flows at velocities below 8 feet per second (fps) and headloss within pipelines should not exceed 10 feet per 1,000 feet of pipe. During fire flow situations pipeline velocities should not exceed 16 fps.

Looping is highly desirable in a distribution system and long, dead-ended pipelines should be avoided where possible due to reliability and water quality concerns. Although 4-inch diameter is the minimum pipe size, new pipelines supplying a fire hydrant are recommended to be a minimum of 8-inches in diameter to provide the minimum required fire flow rate.

Hydraulic water system models use the Hazen-Williams equation to determine headloss in a pipeline for a given flow rate. The Hazen-Williams coefficient or "C" factor in the equation is a function of the diameter, material, and age of the conduit. If detailed information is not available, a global "C" factor of 130 should be used in hydraulic models for all pipelines.

Table 5-1. System Planning and Design Criteria Summary

Category	Planning and Design Criteria
Unit Demands	See **Chapter 3, Table 3-2.**
Demand Peaking Factors	Minimum Day/AAD Ratio = 0.5 Maximum Day/AAD Ratio = 2.0 Peak Hour/AAD Ratio = 3.0 See **Figure 5-1**
System Pressure	40 psi - minimum desired pressure at peak flow 20 psi - minimum allowable pressure at peak flow 20 psi - minimum allowable pressure with MDD+FF 150 psi - maximum desired pressure
Velocity	8 fps - maximum velocity with peak hour flows 16 fps - maximum FF velocity
Headloss	10 feet per1,000 feet maximum desired headloss at peak flow
Diameter	4-inch diameter minimum 8-inch diameter for new pipelines supplying a fire hydrant
Fire Flow	Rural Residential 1,000 gpm, 2-hour duration (2,500 gpm in High and Very High Fire Hazard Areas) Single Family Residential 1,500 gpm, 2-hour duration Multi-Family Residential 2,000 gpm, 2-hour duration Schools 2,500 gpm, 2.5-hour duration Commercial 3,000 gpm, 3-hour duration Industrial 3,500 gpm, 3.5-hour duration
Storage	Capacity equal to: 0.1 x MDD Operational Storage plus the greater of 2 x AAD Emergency Storage or Minimum Required FF x Minimum Required Duration Note: Emergency Storage may be located in a higher pressure zone if the stored water can be delivered by gravity.
Pump Station (Zones with Reservoirs)	MDD + 150 gpm Fire Storage replenishment Minimum Number of Pumps – Three (two duty + one standby) Pumping Period - During San Diego Gas & Electric off-peak and semi-peak rates is preferable Standby Power - Generator in building and in separate room
Hydropneumatic Pump Station (Zones without Reservoirs)	Peak Hour (or) MDD + FF, whichever is greater Minimum Number of Pumps - Four (one duty + one standby for domestic use plus one duty + one standby for FF) Pumping Period - 24 hours Standby Power - Generator in building and in separate room

AAD - average annual demand; FF – fire flow; fps – feet per second; gpm – gallons per minute; MDD - maximum day demand; MDD+FF – maximum day demand plus fire flow; PS – pump station; psi - pounds per square inch

5.3 Fire Flow Criteria

Water must be available not only for domestic use, but also for emergency fire fighting situations. This fire flow must be sustainable for a specific duration at a minimum pressure of 20 psi. General standards establishing the amount of water for fire protection purposes are set by the Insurance Services Office, and these general standards are applied by local fire jurisdictions such as the City of Vista Fire Department. Based on discussions with the Vista Fire Department, the standards are specific to a particular building based on a number of considerations such as type of occupancy, type of construction and construction materials, distance from other structures, and additional factors. Those standards are available in the 2016 California Fire Code, Part 9, Appendix B Tables B105.1(1), B105.1(2), and B105.2, which are used by developers with specific building design projects. It should be noted that many of the older areas in the District were originally designed with less stringent requirements.

For planning purposes minimum fire flows and durations for general building categories, in conformance with the 2016 California Fire Code, are included in **Table 5-2**. The minimum fire flows for different land uses range from 1,000 gpm to 3,500 gpm.

Table 5-2. Fire Flow Criteria

Land Use	Minimum Required Fire Flow (gpm)	Minimum Required Duration (Hours)
Rural Residential	1,000	2
Single Family Residential	1,500	2
MultiFamily Residential	2,000	2
All Residential Areas in High and Very High Fire Hazard Areas	2,500	2
Schools	2,500	2.5
Commercial	3,000	3
Industrial	3,500	3.5

gpm – gallons per minute

The Vista Fire Protection District's Ordinance 2013-23 requires higher fire flows for new subdivisions in wildland-urban interface areas. Wildland-urban interface areas are geographical areas identified by the state as "Fire Hazard Severity Zones." High and Very High Fire Hazard Severity Zones are located along the northern, eastern and southern boundaries of the District, as shown on **Figure 2-7.**

Ordinance 2012-23 states,

> Section 507.2: “In setting the requirements for fire flow, the fire code official shall follow section 507.3 or Appendix B of the County Fire Code, or the standard published by the Insurance Services Office, "Guide for Determination of Required Fire Flow."
>
> Section 507.3: “In wildland-urban interface fire areas, as defined in Appendix B, the main capacity for new subdivisions shall be not less than 2,500 gpm unless otherwise approved by the Fire Chief.”

5.4 Storage Criteria

Storage of the District’s potable water is provided by 12 reservoirs that serve specific pressure zone areas. Of the 12 reservoirs, 10 are located in the main service area, and 2 are in the Boot and Bennett area, east of the main service area. The reservoirs provide operational storage, emergency storage and fire flow storage. Operational storage refers to the peak hour fluctuations above MDD and is further discussed in **Section 5.4.1**. Emergency storage criteria in **Section 5.4.2** is developed to ensure water is available during a wide range of emergency events. Emergency storage might be necessary in the event of pipeline or reservoir failure, as well as planned and unplanned outages of water supply service. Significant outages of the Vista Flume and Water Authority Aqueducts are explored in **Section 4.4**. Fire flow storage requirements are discussed in **Section 5.4.3**.

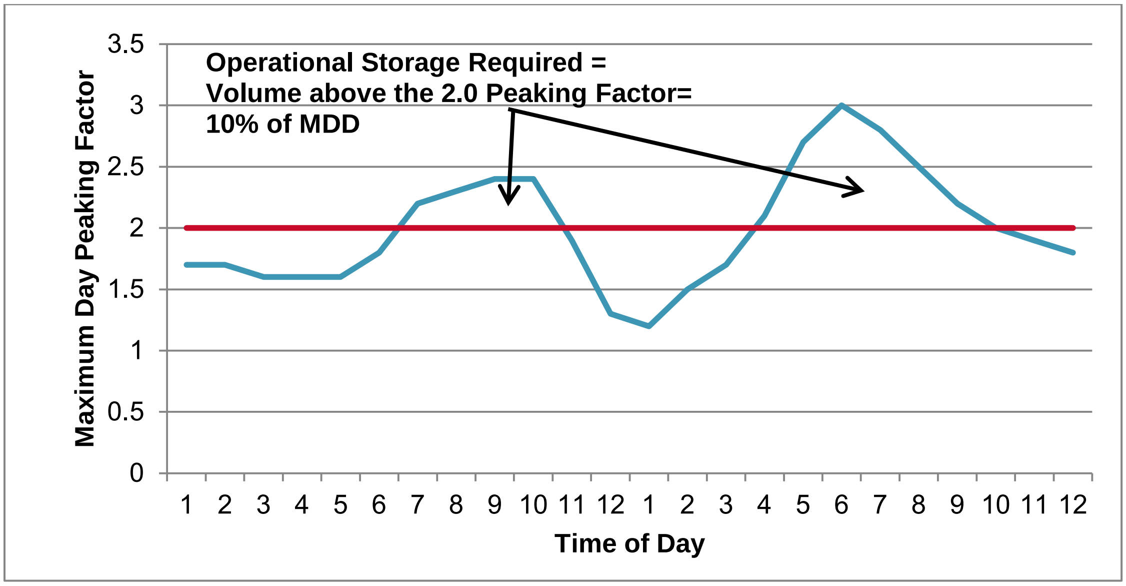
5.4.1 Operational Storage

While the Water Authority Aqueduct connections and the Vista Flume generally supply a constant 24-hour flow rate, additional flows to supply peak demand periods must be satisfied by drawing on water stored in the District’s reservoirs. Providing operational storage within a zone allows transmission mains for the pressure zone to be sized for maximum day, rather than higher peak hour flows.

For this Master Plan, the operational storage requirement is calculated in a similar manner to the 2000 Master Plan. Operational storage is equal to the volume of water used during the maximum day in excess of the 24-hour average for the maximum day.

Figure 5-1 displays the calculated 24-hour MDD curve which is based on hourly demand data obtained during model calibration. The required operational storage (volume of demand above the maximum day peaking factor of 2) is equivalent to approximately 10 percent of the MDD or 20 percent of the AAD. This assumes that the available incoming water supply is equal to the MDD; otherwise more operational storage would be required.

Figure 5-1. Operational Storage Volume



Operational storage criteria in other San Diego County water agencies range from 15 to 30 percent of MDD, indicating that their peak hour demand (PHD) may be higher than the District's, requiring additional storage.

5.4.2 Emergency Storage

The District's emergency storage criterion is 2 days of AAD. By comparison, emergency storage criteria for other San Diego County water agencies range from 1 to 3 days of AAD. The amount of emergency treated water storage necessary is based on an assessment of the risk and the degree of system reliability desired. Since the District has partial ownership of the EVWTP, access to substantial raw water reserves, emergency interconnects to other water districts, and multiple Water Authority filtered water connections; it has more options during an emergency than many other water purveyors in San Diego County.

As discussed in **Section 4.4**, the Water Authority recommends that its member agencies maintain 10 days of storage or alternative supply in order to be self reliant during routine maintenance on the aqueducts. With the exception of emergencies, the maintenance typically occurs during winter months when demands are low. MinDDs are typically 50 percent of the AAD demand. As discussed in **Chapter 3**, the District's current AAD is approximately 17 MGD and projected buildout AAD is 20 MGD. Ten days of storage during minimum demand days would require the District to have 85 to 100 MG of storage capacity if all storage was used. The District currently has just over 40 MG of storage capacity.

To offset a planned outage of the Water Authority aqueducts, the District relies on its capacity rights for 18 MGD at the EVWTP, which can be conveyed to the District via the Vista Flume. Meeting a 10-day outage of the Water Authority aqueduct by depending on the District's local supply via the Vista Flume has been sufficient to meet the Water Authority independence criteria. Water supply reliability was further assessed in **Section 4.4**.

To address local emergencies, such as District main breaks or short term power outages, storage should be provided by reservoirs located in the same pressure zone or at a higher elevation. This will ensure that each zone could still receive water when pumped water is unavailable and stored water can be delivered by gravity. In the 2000 Master Plan, it was determined that having approximately 50 percent of the District's MDD in the 837 zone, where the District's largest reservoir, the 20 MG Pechstein Reservoir, is located, was acceptable since this water is available by gravity to the District's main service area. Similar to the District, the City of Escondido also allows for emergency storage to be located at upper zones when water can be fed by gravity to lower zones in an emergency.

Water quality regulations are becoming more stringent, and system operators are finding it more difficult to maintain the water quality in reservoirs, especially in those that do not have good turnover rates. In special circumstances, the District may elect to reduce or eliminate the emergency storage component for a specific zone if there are multiple supply sources, delivery locations and a well-looped transmission system within the zone. This option was added to the storage criteria in this master plan update to address potential water quality concerns and the lack of suitable storage sites, and to allow for alternative improvements to bring in new sources of water in lieu of constructing additional storage.

5.4.3 Fire Flow Storage

Fire flow storage is established to ensure that each reservoir serving the District is able to supply enough water to extinguish the worst case fire that is likely to occur within its service area. Each reservoir should contain adequate fire flow storage for a single fire based on the most intensive fire flow demand in that pressure zone, or service area if the reservoir serves more than one pressure zone. Fire flow criteria, shown in **Table 5-2**, range from 1,000 gpm for 2 hours, requiring 120,000 gallons in storage, to 3,500 gpm for 3.5 hours, requiring 750,000 gallons in storage.

Figure 2-7 displays the very high, high, and moderate fire hazard severity zones within the District's service area. Since the previous District master planning effort, California Department of Forestry and Fire Protection has designated significant parts of the District to be within Fire Severity Zones, requiring increased fire flows and emergency storage to fight potential wildfires. These areas require 2,500 gpm for 2 hours or 300,000 gallons of storage.

If the fire flow storage requirement is greater than the emergency storage requirement, then the fire flow volume should be used to determine emergency storage capacity requirements.

5.5 Pump Station Criteria

Pump stations boost the water pressure so that service may be provided to users at a higher elevation. Pump stations may supply water to an "open system" or to a "closed system." An open system is a service area with its own storage reservoir. A closed system is an area without a storage reservoir. Pump stations supplying a closed system must regulate pressures utilizing multiple pumps, variable speed drives, and/or a hydropneumatic tank.

Design criteria for pump stations supplying an open system require the pumps to provide capacity equal to the MDD plus an amount adequate to replenish fire storage in a reasonable period, usually 150 gpm. This replenishment is called recharge. The minimum number of pumps is three (two duty and one standby).

Design criteria for pump stations supplying water to a closed system require the pumps to provide capacity for either PHD or MDD plus fire flow (MDD + FF), whichever is greater. The minimum number of pumps is four (one duty, one standby for domestic demand, one duty, and one standby for fire flows).

If the pump station is for back up supply, or tank out of service scenarios, the pump station may need to be sized for peak or fire flows, on a case by case basis.

The District's distribution system is essentially supplied by gravity, and the existing pump stations are primarily operated to supply water to the 984/976/900 zones via the Vista Flume and/or Pechstein Reservoir. The pump stations also increase system reliability by providing a redundant supply. To reduce electricity costs, pumping during San Diego Gas & Electric off-peak and semi-peak rates is preferable. Typically, back up generators in a building and in separate room are recommended, however; the District has portable 150 and 400 kilovolt amp units that can be deployed.

5.6 Pressure Regulating Station Criteria

A pressure regulating/reducing station (PRS) is used to convey water from a higher pressure zone to a lower pressure zone and maintains a desired downstream grade. A pressure sustaining feature can be incorporated to ensure that the pressure upstream does not drop below a desired pressure. A valve with both of these features is called a combination pressure reducing/sustaining valve, or combination PRS. The District has 17 of these combination PRSs that switch between the two modes throughout the day. Combination PRSs can make system operations challenging but are necessary to control flow rates from higher to lower zones to prevent "robbing" supply from the upper zone. Generally, supply to a zone remote from a reservoir should operate primarily in the pressure reducing mode, while supply near a reservoir may be controlled on reservoir water levels and would operate in the sustaining mode while a reservoir is filling. To ensure system reliability, there should be at least two PRSs supplying each major zone.

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6 Existing System

This chapter presents a summary of the District's existing water distribution system, including an overview of the major facilities and a description of system operations on a zone by zone basis.

6.1 Distribution System Facilities Overview

The District currently provides service to three separate service areas including the primary Vista service area and two smaller service areas to the east, respectively referred to as the Boot and Bennett service areas. The majority of the District's customers, infrastructure, and demands are located in the primary Vista service area. A map of the District's existing distribution system major facilities and pressure zones is provided in **Figure 6-1.**

The District currently operates the Vista service area distribution system as 14 distinguishable pressure zones. A water system schematic of the District's distribution system, illustrating how these zones are connected is provided in **Figure 6-2**.

Pressure zones in the primary service area are supplied from:

- Water Authority Second Aqueduct connections (707, 810, 837, and 976/984 zones) (The locations of these connections are shown on **Figure 4-6**.)
- EVWTP via the Vista Flume (837 zone)
- PRSs (486, 550, 565, 630, 637, 668, 707, 752, 900 and 976/984 zones)
- Pump stations (976/984 and 1070 zones)
- VWD Metered Connection (1360)

Flows from the Vista Flume are delivered directly to Pechstein Reservoir or conveyed to the 976/984 zone by pump station. Nine of the 14 zones in the primary Vista service area contain their own reservoir storage (550, 565, 637, 707, 752, 810/837, and 976/984 zones). The 837/810 zones operate as a single zone, as does the 976/984 zones. The five zones in the primary Vista service area that do not contain storage (486, 630 668, 900, and 1070 zones) are smaller service areas that are supplied via pump stations or PRSs.

The Boot and Bennett service areas are supplied from District's connection to the Water Authority's First Aqueduct and the EVWTP via the Vista Flume. The Boot service area is split into two pressure zones (850, 870). The Bennett service area is also split into two pressure zones (898, 980). The 898 zone is served by two reservoirs. Local pump stations convey flow to the 980 zone.

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Figure 6-1. Major Facilities and Pressure Zones

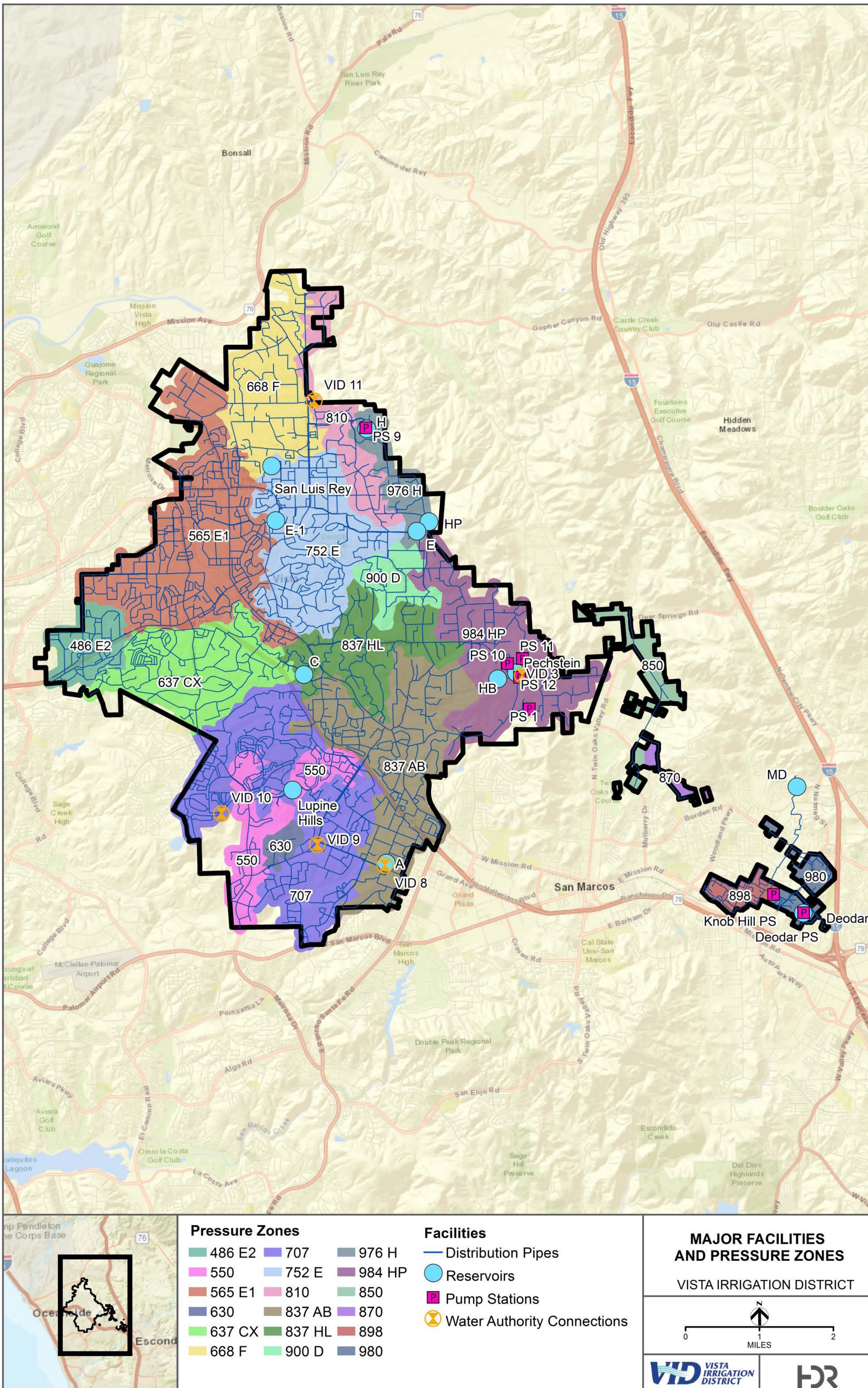

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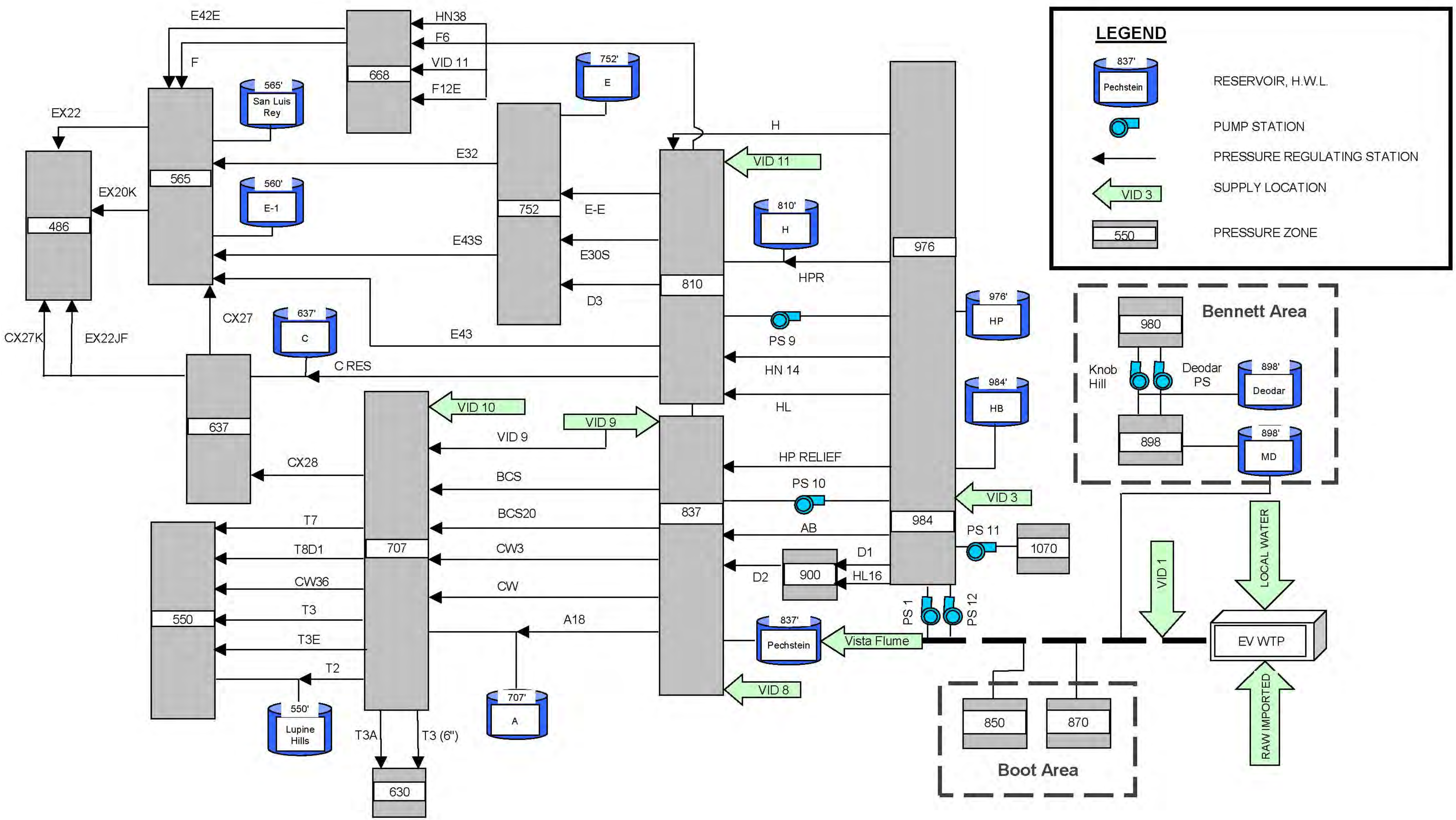
Figure 6-2. Existing System Hydraulic Schematic



Source: Vista Irrigation District 2017

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6.2 Pipelines

The District's transmission and distribution network includes over 429 miles of pipelines, owned and maintained by the District, and 10 miles of privately-owned, District-maintained properties. The materials, size or capacity and lengths are listed in **Table 6-1** and **Table 6-2**. **Appendix C** includes a large scale map showing pipe material and diameter and a similar map showing pipe age.

In 1995, the Board of Directors initiated an on-going Main Replacement Program with the goal of replacing aging pipelines before they reach the end of their useful life and become a maintenance liability. The Main Replacement Program allows pipe replacements to be prioritized based on the age of the line, leak history, and pipe material, as well as a number of factors related to site conditions. Since its inception, 30 miles of older pipe ranging in size from 4 to 20 inches have been replaced.

As part of this Master Plan, the current system performance and deterioration rates were analyzed and recommendations for improvements to the current prioritization process were developed. Those findings are summarized in **Section 6.7**.

Table 6-1. Distribution Pipeline Inventory

Pipeline Material	Size Range (Inches)	Length of Pipe (Miles)
Asbestos Concrete	4 to 12	250
Asbestos Concrete	14 to 36	17
Polyvinyl Chloride	4 to 12	91
Polyvinyl Chloride	14 to 24	3
Steel	4 to 12	38
Steel	14 to 36	24
All other Materials	>4	6
	Total	**429**

Table 6-2. Transmission Pipeline Inventory

Transmission Facility	Carrying Capacity (cfs)	Length of Pipe (Miles)
Escondido Canal and Intake	70 (District has rights to 2/3 of capacity)	14
Vista Main Canal (Flume)	33 (based on 2017 assessment of Baumgartner Siphon carrying capacity)	12

cfs - cubic feet per second

6.3 Reservoirs

Reservoir storage for the primary Vista service area is provided by the 20 MG Pechstein Reservoir and nine additional reservoirs, ranging in size from 0.7 to 5.4 MG. The Bennett service area is served by two reservoirs, the MD and Deodar Reservoirs.
Table 6-3 summarizes the capacity, elevations, and dimensions of the District's reservoirs.

Condition assessment of the District's reservoirs was conducted in November 2016 as part of this Master Plan update effort. Those findings are summarized in **Section 6.7.2.** Two of the 12 reservoirs were not inspected.

In late 2016, the HP Reservoir was out of service while it was undergoing retrofits. The HP Reservoir is a 4.7 MG pre-stressed concrete reservoir constructed in 1962. The rehabilitation improvements included replacement of pre-stressing wires, seismic retrofit, new aluminum dome roof and interior/exterior staircases, and inlet/outlet piping upgrades.

The E Reservoir is currently scheduled for near term replacement and, therefore, was also not inspected.

Table 6-3. Storage Reservoir Summary

Reservoir Name	Pressure Zone	Operating Capacity (MG)	Actual Capacity (MG)	Bottom Elevation (Feet)	HWL Elevation (Feet)	Interior Dimensions (Feet)	Construction Year	Reservoir Type			Reservoir Roof Type
								Buried/ Above Ground	Shape	Material	
Lupine Hills	550	3.00	3.40	537	568	137	1987	Partially Buried	Circular	Prestressed Concrete	Reinforced Concrete
A	707	0.60	0.80	695	708	100	1926	Partially Buried	Circular	Cast-in-place Reinforced Concrete	Wood Rafter and Girder System
Pechstein	837	18.50	20.00	810	837	355	1978	Partially Buried	Circular	Prestressed Concrete	Wood Rafter and Girder System
HB	984	4.05	4.50	951	981	160	1964	Above Ground	Circular	Prestressed Concrete	Tapered Reinforced Concrete Dome
HP (Upon Rehabilitation)	976	4.05 (4.30)	4.50 (4.70)	943	973 (975)	160	1962	Above Ground	Circular	Prestressed Concrete	Tapered Reinforced Concrete Dome (Aluminum)
C	637	0.60	0.80	625	638	100	1926	Above Ground	Circular	Cast-in-place Reinforced Concrete	Wood Rafter and Girder System
E	752	1.20	1.50	741	753	96 x 244	1929	Buried	Oval	-	-
E1	565	0.50	0.60	546	559	90	1925	Above Ground	Circular	Cast-in-place Reinforced Concrete	Wood Rafter and Girder System
San Luis Rey	565	2.70	3.10	540	565	156 x 136	1978	Buried	Rectangular	Cast-in-place Reinforced Concrete	Reinforced Concrete
H	810	5.00	5.40	774	810	160	1997	Partially Buried	Circular	Prestressed Concrete	Reinforced Concrete
MD	898	0.19	0.20	886	896	55	1926	Partially Buried	Circular	Cast-in-place Reinforced Concrete	Wood Rafter and Girder System
Deodar	898	1.10	1.30	869	899	86	1978	Partially Buried	Circular	Prestressed Concrete	Wood Rafter and Girder System

Source: Vista Irrigation District (District) Water Supply Permit, February 2016
MG – million gallons

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6.4 Pressure Regulating Stations

The majority of the primary Vista distribution system is supplied by gravity and supported either directly and/or indirectly via PRSs. Each of the pressure zones receives supply from two to six separate reducing and/or sustaining PRSs. Eleven PRSs are controlled remotely by SCADA. Information for the manually controlled PRSs, including existing control settings is summarized in **Table 6-4**. Information for the SCADA controlled PRSs is summarized in **Table 6-5.**

Table 6-4. Manually Controlled Pressure Regulating Station Summary

Pressure Regulator (ID and Diameter [Inches])		Location	Pressure Zone (Source/Control)	Set Point (psi)		Elevation (Feet)
				Sustaining	Reducing	
A18	6	770 Virginia Place	837 / 707	34/58	10	690
AB	12	2107 Esplendido Avenue	984 / 837	86	26	750
BCS20	3	921 Grand Avenue	837 / 707	118	80	505
BCS20	8	921 Grand Avenue	837 / 707	118	78	505
CW	3	1932 Watson Way	837 / 707	145	113	445
CW	8	1932 Watson Way	837 / 707	145	110	445
CW3	3	358 Mar Vista Drive	837 / 707	90	63	565
CW3	10	358 Mar Vista Drive	837 / 707	90	61	565
CW36	8	Sycamore and Thibido	707 / 550	125	65	390
CX27K	3	Hacienda Drive and Evelyn Lane	637 / 486	NA	88	280
CX27K	8	Hacienda Drive and Evelyn Lane	637 / 486	NA	86	280
D1	4	2450 San Clemente Avenue	976 / 900	NA	90	695
D1	10	2450 San Clemente Avenue	976 / 900	NA	87	695
D2	6	1783 Sunrise Drive	900 / 837	128	86 / 98	620
D3	8	1946 Alta Vista Drive	837 / 752	NA	83	555
EX22JF	6	Cottonwood Drive	637 / 486	130	78	285
E42E	6	W. Knapp and W. Bobier Drive	668 / 565	72	45	450
E43	8	1034 South Santa Fe Avenue	837 / 565	NA	73	380
E43S	6	239 Terrace Way	752 / 565	135	55	410

Table 6-4. Manually Controlled Pressure Regulating Station Summary

Pressure Regulator (ID and Diameter [Inches])		Location	Pressure Zone (Source/Control)	Set Point (psi)		Elevation (Feet)
				Sustaining	Reducing	
EX20K	8	1331 West Vista Way	565 / 486	107	86	280
EX22	6	705 Emerald Drive	565 / 486	80	45	355
F	6	402 Osborne Street	668 / 565	125	84	350
F6	6	2728 East Vista Way	810 / 668	NA	40	550
F12E	8	Lower Taylor Street	810 / 668	135	78	465
F-Reg at VID 11	12	E. Vista Way and Osborne	810 / 668	NA	65	488
H-Reg at VID 11	8	E. Vista Way and Osborne	810 / 837	NA	130	488
H	10	1910 Camino Loma Verde	976 / 810	NA	68	625
HL16	6	2305 Catalina Avenue	976 / 900	NA	78	795
HN	3	Vista Grande Drive	837 / 810	NA	62	680
HN	8	Vista Grande Drive	837 / 810	NA	60	680
HN-14	3	1755 Kings Road	976 / 837	NA	62	670
HN-14	8	1755 Kings Road	976 / 837	NA	60	670
HN38	3	304581/2 Montratchet Street	810 / 668	125	82	440
HN38	8	304581/2 Montratchet Street	810 / 668	125	78	440
T3	6	Sycamore Avenue	707 / 630	NA	78	455
T3	12	Sycamore Avenue	707 / 550	NA	40	455
T3A	4	Business Park Drive	707 / 630	NA	84	440
T3A	8	Business Park Drive	707 / 630	NA	82	440
T3E	4	Park Center Drive	707 / 550	NA	52	460
T3E	8	Park Center Drive	707 / 550	NA	50	460
T7	3	1940 Live Oak Road	707 / 550	105	43	440
T7	6	1940 Live Oak Road	707 / 550	103	41	440
T8D1	6	1051 Chaparral Drive	707 / 550	132	65	400

psi - pounds per square inch

Table 6-5. Supervisory Control and Data Acquisition Controlled Pressure Regulating Station Summary

Pressure Regulator (ID and Diameter [Inches])		Location	Pressure Zone (Source/Control)	Predominant Control Mode	Set Point (psi)				Elevation (Feet)
					Operator[1]		Control[2]		
					Sustaining	Reducing	Sustaining	Reducing	
BCS	6	400 3/4 Sycamore Avenue	837 / 707	Pressure	150	127	150	130	415
BCS	10	400 3/4 Sycamore Avenue	837 / 707	Pressure	150	120	150	130	415
CX27	6	Melrose and W. Vista Way	637 / 565	Flow	136	110	130	124	300
CX27	10	Melrose and W. Vista Way	637 / 565	Pressure	135	98	130	124	300
CX28	6	1099 S. Melrose Drive	707 / 637	Flow	148	122	138	125	330
CX28	10	1099 S. Melrose Drive	707 / 637	Pressure	151	98	138	125	330
C (Res.)	6	1301 Summit Terrace	837 / 637	Level / Flow	71	5	74	5	625
C (Res.)	10	1301 Summit Terrace	837 / 637	Level / Flow	74	5	72	5	625
E30S	6	1070A Taylor Street	810 / 752	Level / Flow	115	103	110	104	520
E30S	16	1070A Taylor Street	810 / 752	Level / Flow	113	102	112	104	520
E32	8	761 East Bobier Drive	752 / 565	Level / Flow	N/A	38	103	42	465
E32	12	761 East Bobier Drive	752 / 565	Level / Flow	N/A	40	103	42	465
E-E	8	2330 Edgehill Road	837 and 810 / 752	Pressure	N/A	5	20	8	740
HP-HL	12	2330 Edgehill Road	976 / 837	Pressure	N/A	26	80	42	740
HPR	12	2082 Pleasant Heights Drive	976 / 810	Level / Flow	N/A	N/A	74	11	775
HP-Rel.	10	3733 Bluebird Canyon Road	984 / 837	Relief	65	N/A	65	N/A	835

Table 6-5. Supervisory Control and Data Acquisition Controlled Pressure Regulating Station Summary

Pressure Regulator (ID and Diameter [Inches])		Location	Pressure Zone (Source/Control)	Predominant Control Mode	Set Point (psi)				Elevation (Feet)
					Operator[1]		Control[2]		
					Sustaining	Reducing	Sustaining	Reducing	
HP-Rel.	16	3733 Bluebird Canyon Road	984 / 837	Relief	67	N/A	67	N/A	835
T2	6	2450 Lupine Hills Drive	707 / 550	Level / Flow	N/A	3	73	22	536
T2	12	2450 Lupine Hills Drive	707 / 550	Level / Flow	N/A	3	70	22	536

1 Operator set-points are common set-points that are routinely adjusted.

2 Control set-points are limit points for the valve.

cfs - cubic feet per second; psi - pounds per square inch

6.5 Pump Stations

The primary Vista service area includes five pump stations which either convey flows to higher pressure zones within the service area or convey flows from the Vista Flume to the 976/984 zone. Three of the five pump stations convey flow from lower pressure zones to higher pressure zones including Pump Station (PS) 9 (from 810 to 976/984), PS 10 (from 837 to 976/984), and PS 11 (from 976/984 to 1070). Additionally, two pump stations convey flow from the Vista Flume to the 976/984 zone including PS 1 and PS 12. Depending on the volume of flow being conveyed by either the Vista Flume or the Water Authority connections, the five pump stations in the Vista service area can be used to distribute required flows to the service area's three highest zones (900, 976/984, and 1070). In addition to the Vista service area, the Bennett service area is served by two pump stations, Knob Hill and Deodar, which can be used to convey flow from the 898 zone to the 980 zone. A summary of pump station data is provided in **Table 6-6**.

Table 6-6. Pump Station Summary

PS Name	Location	Year Constructed	Zones Served	Pump Capacity
PS 1	1852 Robinhood Road	-	Flume to 984	One pump at 550 gpm
PS 9	2082 Pleasant Heights	-	810 to 976	Two pumps at 1,500 gpm each
PS 10	3733 Bluebird Canyon	-	837 to 984	One pump at 1,300 gpm One pump at 1,600 gpm
PS 11	-	-	984 to 1070	Two pumps at 200 gpm each
PS 12	3874 Bluebird Canyon	-	Flume to 984	Three pumps at 1,600 gpm each
Knob Hill PS (PS 3)	1833 Knob Hill Road	-	898 to 980	One pump at 300 gpm Two pumps at 600 gpm each
Deodor PS (PS 4)	969 Deodor Road	-	898 to 980	Four pumps at 300 gpm each

gpm - gallons per minute; PS – pump station

6.6 Pressure Zones

The primary Vista service area distribution system is comprised of 14 distinguishable pressure zones as displayed in **Figure 6-1** and shown schematically in **Figure 6-2**. The Boot and Bennett service areas include two pressure zones each.

The high number of pressure zones in the primary distribution system is due to the topography of the service area, which generally slopes downhill from east to west. Currently, water is supplied at connection points in the eastern portion of this distribution system to high elevation pressure zones. As the system is currently operated, water from these high elevation pressure zones flows downgradient through a series of PRSs to serve lower elevation zones. The District operations staff is responsible for maintaining the balance of pressure in the higher pressure zones while allowing flow to lower zones via the PRSs.

Conceptually, the primary Vista service area is divided into pressure zones in order to maintain acceptable pressures for customers. A hydraulic grade and AAD summary for each of the major pressure zones is shown in **Table 6-7**. The recommended low service elevation for a zone is calculated by subtracting 350 feet from the hydraulic grade of the zone. Any elevation lower than this would result in static pressures greater than 150 psi. The high service elevation for a zone can only be approximated because the actual minimum residual pressures are a function of elevation and headloss in the distribution system during peak demands.

Table 6-7. Major Pressure Zone Demand Summary

Pressure Zone/ Hydraulic Grade (Feet)	Hydraulic Grade Control [1]	AAD [2]		
		(AFY)	(gpm)	(MGD)
486	PRS	847	525	0.76
550	Lupine Hills Reservoir	1,497	928	1.34
565	San Luis Rey and E1 Reservoirs	3,704	2,296	3.31
630	PRS	60	37	0.05
637	C Reservoir	1,470	911	1.31
668	PRS	872	541	0.78
707	A Reservoir	3,001	1,861	2.68
752	E Reservoir	2,361	1,464	2.11
810	H Reservoir	536	332	0.48
837	Pechstein Reservoir	2,816	1,746	2.51
900	PRS	254	157	0.23
976/984	HP Reservoir	269	167	0.24
	HB Reservoir	996	617	0.89
	Totals [2]	18,683	11,582	16.68

[1] Zones with reservoir hydraulic grade control typically represent tank high water level. The exception is Lupine Hills Reservoir, which has a high water level of 568 feet.

[2] Demands do not include the Boot and Bennett service areas.

AFY - acre feet per year; gpm – gallons per minute; MGD - million gallons per day; PRS – pressure regulating station

Major features and existing system operations of each pressure zone are described in detail below. The information is based on previous master plans and studies, site visits, and numerous discussions with field personnel.

6.6.1 984 and 976 Zones

The 984 and 976 zones operate as a single pressure zone, and the actual hydraulic grade depends on the service areas of the HB and HP Reservoirs. This zone is supplied primarily by a combination of Water Authority water at VID 3 and water from the Vista Flume via PS 1 and PS 12. PS 10 is able to supplement the 984 zone from the 837 zone. The 4.5 MG HB Reservoir provides storage for the 984 zone.

The 984 zone borders several zones including 837, 900 and 1070. The AB PRS provides a connection from the 984 zone to the 837 zone. The 900 zone is served from PRSs D1 and HL16. Additionally, water is conveyed from the 984 zone to the 1070 zone via PS 11.

The 4.7 MG HP Reservoir is supplied from the 984 zone and provides storage for the 976 zone. PS 9 provides a backup supply from the 810 zone. There are four connections to the 810 zone via PRSs including H, HPR, HN14, and HL.

6.6.2 1070 Zone

The 1070 zone is a small pressure zone which serves five customers. This zone receives water from the 976/984 zone via PS 11.

6.6.3 900 Zone

The 900 zone is a smaller zone which receives water from the 976/984 zone via the D1 and HL16 PRSs. This zone also feeds the 837 zone via the D2 PRS.

6.6.4 810 and 837 Zones

The 810 and 837 zones are operated as a single pressure zone or as separate pressure zones at the District's discretion. The zones are separated by a valve that can be closed remotely to isolate the two systems. Combined, the 810 and 837 zone is the largest zone spatially, extending from the San Luis Rey River at the northern boundary of the District all the way to the southern boundary of the District, south of Highway 78. If the 810/837 interconnecting valve is open, headloss though the distribution system is sufficient to qualify 810 and 837 as separate pressure zones, although they are not hydraulically separated by a pressure reducing facility.

The actual grade of the combination 810 and 837 zone varies between approximately 810 in the north and south and 837 in the vicinity of Pechstein Reservoir. The service area of the H Reservoir is often referred to as the 810 zone, although hydraulically is a part of the combination 810 and 837 zone. Due to headloss through the distribution system, the northern part of this zone is referred to as the 810 zone. The H Reservoir in the north has an operational high water grade of 806, instead of 837.

The 837 zone contains the largest storage volume, with 20 MG at Pechstein Reservoir. The primary supply to the 837 zone is the EVWTP via the Vista Flume. The

VID 1 connection to the Water Authority's First Aqueduct is another supply to the District via the Vista Flume, which terminates at Pechstein Reservoir. VID 8 and VID 9 are other Water Authority connections that serve the southern part of the 837 zone from the Tri-Agency Pipeline. PRSs that supply the 837 zone include HL from the 976/984 zone, D2 from the 900 zone, and HP Relief and AB from the 976/984 zone.

Along with gravity flow from the Pechstein Reservoir, the Water Authority supplies the 810 zone via the VID 11 connection. The 5.4 MG H Reservoir provides additional storage for the 810 zone. PRSs that supply the 810 zone include H, HPR, HL, and HN14 from the 976/984 zone.

Given the north to south spatial coverage of the combined 810 and 837 zones, as well as its large storage capacity, number of aqueduct connections, and high zone elevation, it follows that this zone has the largest number of PRSs of any other zone. These PRSs facilitate the supply to the lower zones. There are three PRSs to the 668 zone (F6, HN38, and F12E), three to the 752 zone (E30S, E-E, and D3), one to the 565 zone (E43), one to the 637 zone (C RES), and five to the 707 zone (CW3, CW, BCS, BCS20, and A18), for a total of 13 PRSs.

6.6.5 752 Zone

The 752 zone is in the central portion of the District, and is bordered by five other zones. The E Reservoir provides 1.5 MG of storage for the zone. Three PRSs supply the 752 zone from the 810 and 837 zone including E-E, D3, and E30S. There are two PRSs that feed the 565 zone from the 752 zone including E32 and E43S. The E32 PRS is the primary supply to the 565 zone, and the water level of E Reservoir is affected by the operation of the E32 and E30S PRSs. The E32 PRS has four flow control settings, which are routinely changed by SCADA.

6.6.6 707 Zone

The 0.8 MG A Reservoir provides operational storage for the 707 zone. The primary supply to the zone is from Water Authority connections VID 9 and VID 10, off the Tri-Agency Pipeline. There are five PRSs from the 810 and 837 zone that supply the 707 zone: CW3, CW, BCS, BCS20, and A18, all of which have combination pressure sustaining/reducing controls. Nine PRSs supply lower zones from the 707 zone.

Two PRSs, T3 and T3A, serve the small, reduced 630 zone. The T3 PRS consists of two valves with one serving the 630 zone and the other serving the 550 zone. Six PRSs - T2, T8D1, CW36, T7, T3E, and T3 – feed the 550 zone.

The CX28 PRS supplies the 637 zone. This PRS is hydraulically critical in the event that the 707 zone becomes over pressurized by aqueduct turnout flows, which must be delivered at a constant flow throughout the day. If the 707 and 550 zones cannot utilize the flow ordered at VID 9 and 10, and the CX28 valve is closed, the flow at the turnouts will be rejected. Thus, the CX28 PRS is utilized to relieve excess flow to the 637 zone. Additionally, the T2 PRSs at Lupine Hills Reservoir have a "high" pressure override that allows the PRSs to feed during a high pressure event, given the reservoir is set to a water level less than 25 feet.

6.6.7 630 Zone

As discussed in **Section 6.6.6**, the 630 zone is a small, reduced zone supplied by the 707 zone via two PRSs, T3, and T3A. The T3 includes two valves, with one valve serving the 550 zone and the second valve, a 6-inch pressure reducing valve, serving the 630 zone. T3A consists of two pressure reducing valves, including a 4-inch and an 8-inch, which serve the 630 zone from the 707 zone.

6.6.8 668 Zone

The 668 zone is in the north end of the District and is supplied from the 810/ 837 zone via three PRSs including F6, HN38, and F12E. An additional 12-inch PRS is able to supply the zone from the VID 11 Water Authority connection. Two PRSs – E42E and F – deliver water from the 668 zone to the 565 zone.

6.6.9 637 Zone

The 637 zone is supplied from two PRSs, CX28 from the 707 zone and the C RES PRS from the 837 zone. The 0.8 MG C Reservoir provides operational storage for the zone. Three PRSs convey water from the 637 zone to lower zones: CX27 to the 565 zone and CX27K and EX22JF to the 486 zone.

6.6.10 565 Zone

The 565 zone has the highest demands in the District. Two reservoirs serve the zone, the 0.6 MG E1 Reservoir and the 3.1 MG San Luis Rey Reservoir. There are five PRSs that feed the zone from four higher zones including F and E42E from the 668 zone, E32 and E43S from the 752 zone, E43 from the 837 zone, and CX27 from the 637 zone. There are two PRSs that supply the lower 486 zone: EX20K and EX22.

6.6.11 550 Zone

The 550 zone is supplied exclusively from the 707 zone through six PRSs: T2, T8D1, CW36, T7, T3E, and T3. Storage is provided by the 3.4 MG Lupine Hills Reservoir. The reservoir is supplied from the T2 PRS via the 707 zone and is controlled via SCADA.

6.6.12 486 Zone

The 486 zone is the lowest zone in the District, situated in the most western part of the District. This zone does not include storage and does not supply any other zones. The zone is suppliedby four PRSs: CX27K and EX22JF from the 637 zone and EX22 and EX20K from the 565 zone.

6.6.13 Boot and Bennett Areas

The Boot and Bennett areas are satellite District service areas supplied by the Vista Flume and located to the east of the main service area. The Boot area is located adjacent to the primary service area and relies on the flume to maintain service pressures. The Boot area is split into two pressure zones, the 870 and 850 zones, each connecting to the flume at different locations resulting in the 20 foot difference in head between the two zones.

The Bennett area is located east of the Boot area, south of the Vista Flume. Like the Boot service area, the Bennett service area is split into two pressure zones, the 980 and the 898. However, the Bennett area is more complex and includes two reservoirs and two pump stations. The 0.20 MG MD Reservoir is located in close proximity to the flume, which sets the grade in the reservoir and the 898 zone. The 1.30 MG Deodar Reservoir is located south of the MD Reservoir in the Bennett service area and provides head for the Knob Hill PS (PS 3) and Deodar PS (PS 4), which lift flow to the 980 zone.

6.7 Condition Assessment Summary

As part of this Master Plan, condition assessment of the District's pipelines and reservoirs was conducted. The assessment of the pipelines is based on a review of the District's datasets and workshops with District staff. This information was used to develop an approach to repairing or replacing aging infrastructure.

The assessment of the reservoirs is based on field investigation of 10 of the District's 12 reservoirs. Two reservoirs, HP and E Reservoirs, were not inspected. HP Reservoir was out of service, undergoing rehabilitation due to corroded and failing prestressed wire wrap. E Reservoir was in service, but did not require inspection since it is scheduled for replacement

The detailed findings of the condition assessments are provided in **Appendix A -** Water Pipeline Condition Assessment and **Appendix B** - Reservoir Condition Assessment Technical Memorandum (TM). A brief summary of those assessments is provided in the following sections and recommendations are included in the Capital Improvement discussion in **Chapter 9**.

6.7.1 Pipeline Condition Assessment

The District owns 429 miles of water main infrastructure and manages an additional 10 miles of privately owned water main infrastructure. As the system continues to age and deteriorate, one of the District's primary goals is to cost effectively sustain desired service levels. To accomplish this, the District has initiated this effort to continuously improve the way distribution main assets are managed. The three primary objectives of this project are to:

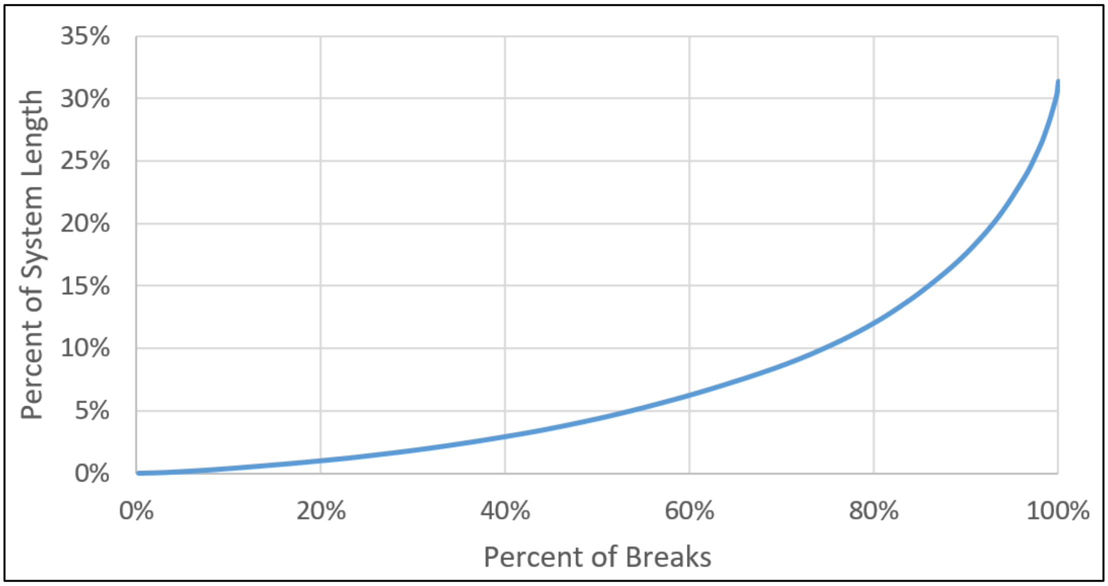
1. Establish prudent, transparent, and defensible investment levels that will enable the District to sustain desired levels of service as the system continues to age and deteriorate.
2. Focus those investments to ensure ratepayers realize the greatest return on their investment.
3. Optimize existing practices.

For distribution mains, the District has break data going back to 1992. The District has documented 2,230 breaks from 1992 through January of 2017, of which 839 were classified as occurring on a mainline (as opposed to a service, valve, or other appurtenance) and were used in this analysis. The data is of sufficient quantity and quality to build risk and investment models that meet the three objectives of this project.

Industry experience tells us that pipeline performance and useful life can vary significantly from one construction project to the next. Construction project data provided insight regarding the relative quality of the material used, transport, and handling procedures, installation quality, backfill quality, and construction management quality. Analysis of the District's break data validates that District pipeline performance varies significantly by project.

Figure 6-3 summarizes project number performance by cumulative breaks and lengths. As shown, a small percentage of system piping is responsible for most of the breaks (e.g., 80 percent of all breaks have occurred on projects that represent only 12 percent of the entire system length). The relationship between project number and performance was found to be significant, thus construction project numbers were used as the basis for sizing and prioritizing renewal investments.

Figure 6-3. Small Percentage of Pipe is Responsible for Most Breaks



Based on the data, historic break count was also found to be a good indicator of performance, as the percent of projects that broke again increased as the break count increased, and the duration between subsequent breaks became shorter.

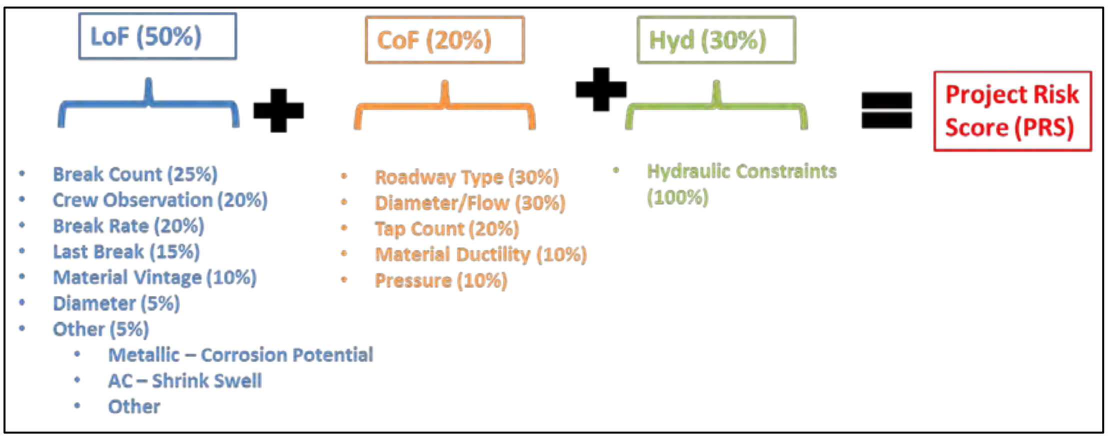
To better understand how various investment levels will impact future service levels, a break forecasting model was developed. This model applies prudent, transparent, and reproducible methods to District data to forecast how many breaks will occur in each year over the planning horizon (through 2040). Three investment scenarios were modeled:

- Scenario 1 – Sustain Existing Investment Levels
- Scenario 2 – Sustain Existing Service Levels
- Scenario 3 – Double Existing Investment Levels

It is anticipated that these scenarios, in conjunction with engineering and operational judgment, will enable the District to make informed renewal decisions, with confidence that desired levels of service will be maintained. Selection of the appropriate investment level should be made by District management and should strike the appropriate balance between desired long term service level goals and the associated cost to achieve that service level.

The next objective was to focus those investments to ensure ratepayers realize the greatest return on their investment. A consistent, transparent, efficient, prudent, and defensible approach was defined to select an appropriate investment level through the identification and prioritization of water pipeline replacement projects. To accomplish this, a project risk score was developed that quantifies relative risk on a scale of zero (lowest risk) to 100 (highest risk). This methodology considers the consequence of failure (CoF), the likelihood of failure (LoF), and hydraulic limitations, as shown in **Figure 6-4**.

Figure 6-4. Risk Calculation Method



This methodology was applied to the District's distribution mains. The resulting risk map is provided in **Figure 6-5**.

Historically, the District has typically used the open-trench replacement method. Based on regulatory challenges, useful life extension uncertainty, additional research needed, and limited economies of scale, it is recommended the District continue to use open-trench replacement as the primary renewal method. However, the viability of alternative renewal solutions should be evaluated on a project specific basis, particularly where the integrity of the host pipe can be cost effectively determined and site-specific factors lend themselves to alternative renewal solutions.

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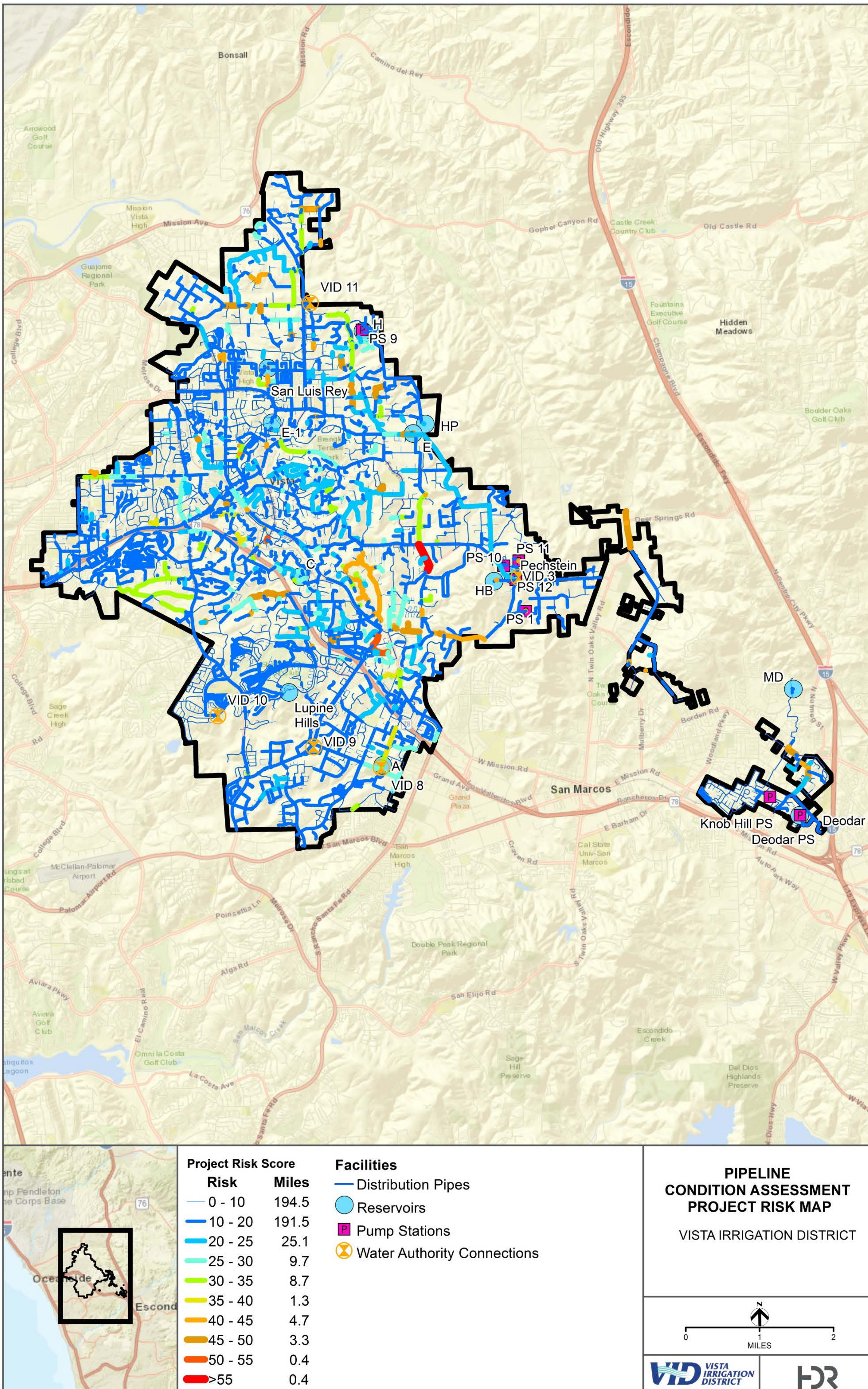
Figure 6-5. Pipeline Condition Assessment Project Risk Map



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6.7.2 Reservoir Condition Assessment

Condition assessment inspections of 10 of the District's 12 potable water reservoirs were completed in November 2016. Two reservoirs, HP and E Reservoirs, were not inspected. HP Reservoir was out of service, undergoing rehabilitation because of corroded and failing prestressed wire wrap. E Reservoir was in service but did not require inspection since it is scheduled for replacement. Confined space entry of the reservoirs was not conducted; however, visual inspection of the reservoir's interior from access hatches was attempted when it was deemed safe to do so.

The exterior inspections were intended to document the current condition of the civil site, corrosion, and structural aspects of the reservoirs. Field activities completed during these field visits included:

- Perimeter, site, and drainage inspection
- Structural inspection
- Exterior coatings inspection
- Reservoir climb and roof inspection
- Non entry, visual hatch inspection

The findings of the inspection of the District's reservoirs were used to recommend and prioritize improvements for the rehabilitation or replacement of reservoir equipment and identify any additional assessments required. The overall approach and detailed inspection with photographic documentation are included in **Appendix B** - Reservoir Condition Assessment TM.

The HDR standardized Condition Assessment Ratings System (CARS) was utilized to guide the inspection team while conducting the reservoir inspections. CARS promotes consistency from site to site to facilitate proper prioritization of the reservoirs civil/site, corrosion and structural aspects.

The criteria specified in the CARS are grouped into four categories, as follows:

1. Structural
2. Site (non-reservoir)
3. Aesthetic (reservoir only)
4. Safety/Security

The civil/site and corrosion and structural recommendations listed for each reservoir address the deficiencies noted during the field inspections. The civil/site, corrosion, and structural recommendations pertain to ongoing monitoring, minor maintenance, and repair work. The recommendations for further investigation include potentially larger scale improvements and recommendations, such as interior cleaning and inspection or seismic evaluations.

Each criterion was scored on a scale or listed as not applicable. The scoring criteria are displayed in **Table 6-8**.

Table 6-8. Reservoir Condition Scoring Criteria

Score	Description	Phasing
0	No action required	-
1	Minor (7 plus years)	Long-Term
3	Moderate (2 to 6 years)	Mid-Term
5	Immediate (0 to 2 years)	Near-Term
N/A	Not Applicable	-

Each reservoir received a score for Civil/Site components, Civil/Corrosion components and Structural components. Each category of components was first normalized to a 100-point scale and then weighted based on potential risk. Site and civil/corrosion were weighted at 20 percent each and structural was weighted at 60 percent. Weighting the structural components at a higher value allowed for a more accurate prioritization of the projects to address safety and reliability concerns first.

The scoring components, rankings, and recommendations for each inspected reservoir are provided in **Table 6-9.** Detailed recommendations are provided in **Appendix B**. The top three reservoirs in the most need for near term repair and/or replacement, based on the rankings, are Deodor, Pechstein and A Reservoirs. All three require additional internal inspections to determine the potential need for complete replacement. The recommended capital improvement projects for all of the reservoirs are discussed further in **Chapter 9**.

Table 6-9. Reservoir Condition Findings and Recommendations

	Rank	1	2	3	4	5	6	7	8	9	10
	Reservoir	**Deodar**	**Pechstein**	**A**	**HB**	**Lupine Hills**	**E1**	**MD**	**C**	**H**	**San Luis Rey**
Recommended Improvements	Access Road	🟡	🟢		🟢	🟢		🟡	🟢	🔴	
	Fences and Gates	🔴		🔴	🔴			🔴	🔴		🟡
	Trees and Vegetation			🔴		🟡	🔴	🔴		🔴	
	Signage and Safety Signage	🟢	🟢	🔴	🔴		🔴	🟢	🟢	🟢	🔴
	Drainage		🟡								
	Site Piping and Appurtenances			🔴	🟢						
	Roof Hatch	🔴	🟡	🔴	🟡	🟡	🟡	🟡	🟢	🟡	🟡
	Roof	🔴	🔴	🔴	🟡	🔴	🟡	🟡	🟡	🟡	🟢
	Handrails, Ladders, and Stairs	🟢	🟢		🔴	🔴	🔴	🟡	🔴	🟡	
	Hatches and Doors		🔴		🟡						
	Overflow Pipe	🟡								🟡	
	Reservoir Exterior Wall	🟡	🟡	🟢	🟡	🟢	🟢	🟢	🟢	🟢	
	Vent		🔴		🟡	🟡	🟡		🟡		🟢
	Stability/ Geotechnical/ Foundation	🔴			🟡			🔴			
	Interior Structure		🔴	🟡			🟡				🟢
	Further Investigation	🔴	🔴	🔴	🔴	🔴	🟡	🟡	🟡	🟡	🟡

🔴 Near Term Improvements (0 to 2 years)
🟡 Mid Term Improvements (2 to 6 years)
🟢 Long Term Improvements (7 plus years)

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7 Existing System Analysis and Recommendations

This chapter briefly summarizes the software selection process and validation of the new hydraulic model, and subsequently describes hydraulic analyses of the existing system. Based on the evaluation criteria provided in **Chapter 5**, existing system deficiencies were determined and projects to improve system performance were recommended. Recommended improvements are proposed to improve basic operations, bring the water system in compliance with hydraulic evaluation criteria, and increase system reliability.

7.1 Model Selection, Development, and Validation

The computational hydraulic model of the District's distribution system has passed through three main phases in its historical development. In its original phase, the hydraulic model was constructed by the District using Cybernet. This original model included information relevant to the distribution system at the time, including pipeline data (alignment, length, diameter, and roughness coefficient), node data (AADs and elevations), reservoir dimensional data, and valve data (location, type, and size).

In the second phase of model development, as part of the 2000 Master Plan effort, the model was converted from Cybernet to H2ONET Version 3 by Innovyze. As part of this process, the model was also updated to represent the District's distribution system at the time. In addition, the model was verified using a combination of field and SCADA data collected over a 24-hour period from November 9-10, 1999. As part of the model update, special attention was given to the modeling of combination PRSs (e.g., pressure reducing/sustaining valves), which were not offered as a standard control valve option in H2ONET Version 3.

The third phase of model development was conducted as part of this Master Plan. Between the 2000 Master Plan and this Master Plan, the H2ONET version of the hydraulic model was maintained and updated by the District. As part of this Master Plan, the District's model was converted to a new modeling software and updated based on available information (e.g., GIS, operations information, billing data, and supply data). The updated model was then validated based on current operations information, SCADA data, and hydrant tests. The validated model was then used to conduct an analysis of the capacity and reliability of the existing and future distribution systems based on the criteria developed in **Chapter 5.**

7.1.1 Model Conversion to InfoWater

As part of this Master Plan, the District's existing H2ONET model was converted to InfoWater Version 12.2 by Innovyze. InfoWater includes features that were more desirable to District staff than H2ONET, including the ability to run the modeling software in ArcMap. Prior to converting the model to InfoWater, a comparison of seven of the leading water distribution system modeling software packages was developed for the District's review, and it was determined that InfoWater was the best fit for the District's needs. H2ONET and InfoWater are both distributed by Innovyze, and InfoWater includes the ability to automatically import H2ONET models, so conversion between the two software packages was streamlined. A summary of the model software selection process is included in **Appendix D -** Hydraulic Model Software Selection TM.

As part of the model conversion to InfoWater, the two smaller H2ONET models of the Boot and Bennett systems were also imported into the new InfoWater model.

7.1.2 Model to Geographic Information System Relationship

As part of this Master Plan, the District's hydraulic model was updated based on the latest GIS information in the District's geodatabase (gdb). As part of this process, the District explored the practicality of establishing a one-to-one model-to-GIS relationship where the modeled facilities could be linked to the gdb facilities via a unique identification number. A cost, benefit analysis was preformed and it was determined that the costs of establishing a one-to-one relationship with the GIS would outweigh the benefits. The existing hydraulic model is skeletonized and represents the distribution system with fewer primary distribution pipes than are included in the gdb. Increasing the number of pipes in the model could make the model unnecessarily complex leading to increased errors, longer run times, and resulting in a model that is more difficult to manage. Additionally, matching the model pipes with the GIS would require an upgraded license to accommodate the large number of pipes in the gdb. A summary of the cost/benefit analysis of establishing a one-to-one relationship between the model and the District's GIS is included in **Appendix E** - Hydraulic Model GIS Integration TM.

Maintaining a relationship between the model and the District's GIS information is a priority, even if the relationship is not a one-to-one facility relationship with the gdb. The selection of InfoWater as the software for the model conversion allows for a visual comparison of the relationship between the gdb and the model in ArcMap. In ArcMap, the gdb facilities can be overlaid on the modeled facilities allowing for a quick comparison.

7.1.3 Modeling Combination Regulators

In addition to converting the District's H2ONET model to InfoWater, the approach to modeling the combination PRSs was enhanced as part of this Master Plan. A review of the ability of InfoWater to represent combination PRSs is provided in **Appendix D.**

At the time of the 2000 Master Plan, the District operated 17 combination PRSs. Combination PRSs have the ability to modulate between pressure reducing and pressure sustaining modes by throttling the flow to achieve the desired pressure settings upstream and/or downstream. Since valves in H2ONET are either pressure reducing/regulating valves or pressure sustaining valves and not both, a pressure reducing/regulating valve and a pressure sustaining valve were modeled in parallel to represent the combination PRS. Logic controls were used to open one and close the other, or vice versa, and then switch if necessary, depending on pressures upstream and downstream of the valve.

For this Master Plan, the model was updated to represent the combination PRSs as a pressure sustaining valves and a pressure reducing/regulating valve in series (up-gradient to down-gradient) with no logic controls. When reviewing options of modeling software for the model conversion, the ability of the software to properly represent combination PRSs was a primary concern.

7.1.4 Operations

Operations information in the hydraulic model was updated based on information provided by District Operations staff including facility settings, SCADA data, and conceptual information about how the distribution system is operated.

As discussed in the previous section, the operational control information in the hydraulic model was updated to incorporate combination PRSs without logic controls. PRS settings were updated in the model based on set points provided by Operations staff. Settings were included for manually adjusted PRSs and PRSs controlled by the SCADA system. PRS settings included in the model are listed in **Chapter 6**.

7.1.5 Demands

Model demands were developed based on calendar year 2014 billing and supply information as discussed in **Chapter 3**. Billing data were provided as bimonthly water use volumes. Billing accounts were linked to a meter GIS layer from the District's gdb, which provided the spatial location of each meter. Water supply data were also provided by the District and were used to estimate water loss. Operations SCADA data were used to develop updated diurnal patterns. The development of updated existing system demands are discussed in detail in **Chapter 3**.

7.1.6 Model Validation

Following the existing system model update, the model was validated to demonstrate that the updated model represents the real world distribution system. Hydraulic model validation consisted of two main stages including macro level verification and micro level calibration. Model validation is discussed in more detail in **Appendix F** - Hydraulic Model Validation TM.

Macro level verification consisted of adjusting the model for demand distribution, diurnal patterns, water loss, and system operations. The goal of macro level verification is to demonstrate that the model represents system demands and behavior during extended period simulation (EPS) in a qualitative comparison with SCADA data. Model verification was performed for both summer and winter demand conditions based on supply, demand, and SCADA data for August 2016 and February 2015, respectively. A qualitative comparison assessment was performed based on tank levels for the EPS model output and available SCADA data for each of the verification scenarios. Comparing the model results with the SCADA data indicated that the model acceptably represents the real world system operations for both the summer and winter verification scenarios. Comparison graphs are included in **Appendix F**.

Micro level calibration consisted of comparing model results with system response to hydrant tests. The goal of micro level calibration is for model results to replicate hydrant test field data for static and residual hydrant pressures in a quantitative comparison. Hydrant tests were performed in July 2017, over a 2 day period, and consisted of 21 individual tests. Each test consisted of a flow hydrant and two residual hydrants. The hydrant tests are discussed in more detail in **Appendix G** - Fire Flow Test Report. Based on comparisons of model output with the field data collected as part of the hydrant tests, it was determined that the modeled hydrant test results match the field data to within 10 percent accuracy for each of the tests. Therefore, the updated model was considered calibrated for the purposes of this Master Plan. A comparison table of field data and model output for the hydrant tests is included in **Appendix F** - Hydraulic Model Validation TM.

7.2 Existing System Analysis

The updated and calibrated InfoWater model was used to analyze the District's existing distribution system, based on the planning and design criteria defined in **Chapter 5** to identify potential deficiencies. **Chapter 5** indicates three primary system conditions for applying the criteria for system evaluation including PHD, MDD +FF, and MinDD. MinDD simulations were run to identify high pressures and to evaluate water age.

7.2.1 Maximum Day and Peak Hour Demand

Maintaining required pressures under high demand conditions is the District's primary concern with regard to system performance evaluation. The District's main distribution system is extremely dynamic. Changes in water supply or demand in one pressure zone can affect hydraulic conditions in all other areas of the system due to the large number of interzone connections, such as PRSs and pump stations. Because of the complex interrelationship between pressure zones, steady state model runs would not represent the temporal changes that occur as the distribution system adjusts to MDD conditions leading up to a PHD event. In order to account for this, EPS model runs were used to represent PHD conditions in the model. MDD EPS model runs were performed, and the peak model result values were used to represent PHD conditions. MDD EPS model runs were also performed to evaluate reservoir drain/fill operations.

Model valve settings were based on existing system settings as described in **Chapter 6**, **Table 6-4,** and **Table 6-5**. Water Authority flows were adjusted to provide the average supply to meet demand and maintain system operations. Flows were introduced at all turnouts except for VID 8, as VID 8 is not widely used and was recommended to be abandoned in the 2000 Master Plan. Flow from the Vista Flume was also included in the model. Modeled supply flows for the MDD EPS scenario are listed in **Table 7-1**.

Table 7-1. Maximum Day Demand Supply Summary

Supply Location	Pressure Zone	Flow (under MDD settings)	
		(gpm)	(cfs)
Vista Flume	837	7,207	16.1
VID 3	984	3,742	8.3
VID 8	837	-	-
VID 9	837/707	5,468	12.2
VID 10	707	1,841	4.1
VID 11	810/668	6,621	14.8
	Total	24,878	55.4

cfs - cubic feet per second; gpm – gallons per minute; VID – Vista Irrigation District

Model results for the existing system PHD scenario indicate some high elevation, low-static pressure areas, primarily in the 565 zone. Model results also indicate that a few relatively high elevation locations on the periphery of the 984 zone may experience pressures below the minimum required pressure criteria of 40 psi under modeled PHD conditions.

Existing system model results also indicate that no pipelines exceed the high velocity criteria during PHD. The results indicate that some pipes exceed the high headloss criteria throughout the distribution system at various locations throughout the system including in the 565, 668, 837 zones, and in the 707 zone near the A Reservoir. Model results for the existing PHD scenario are shown in **Figure 7-1**.

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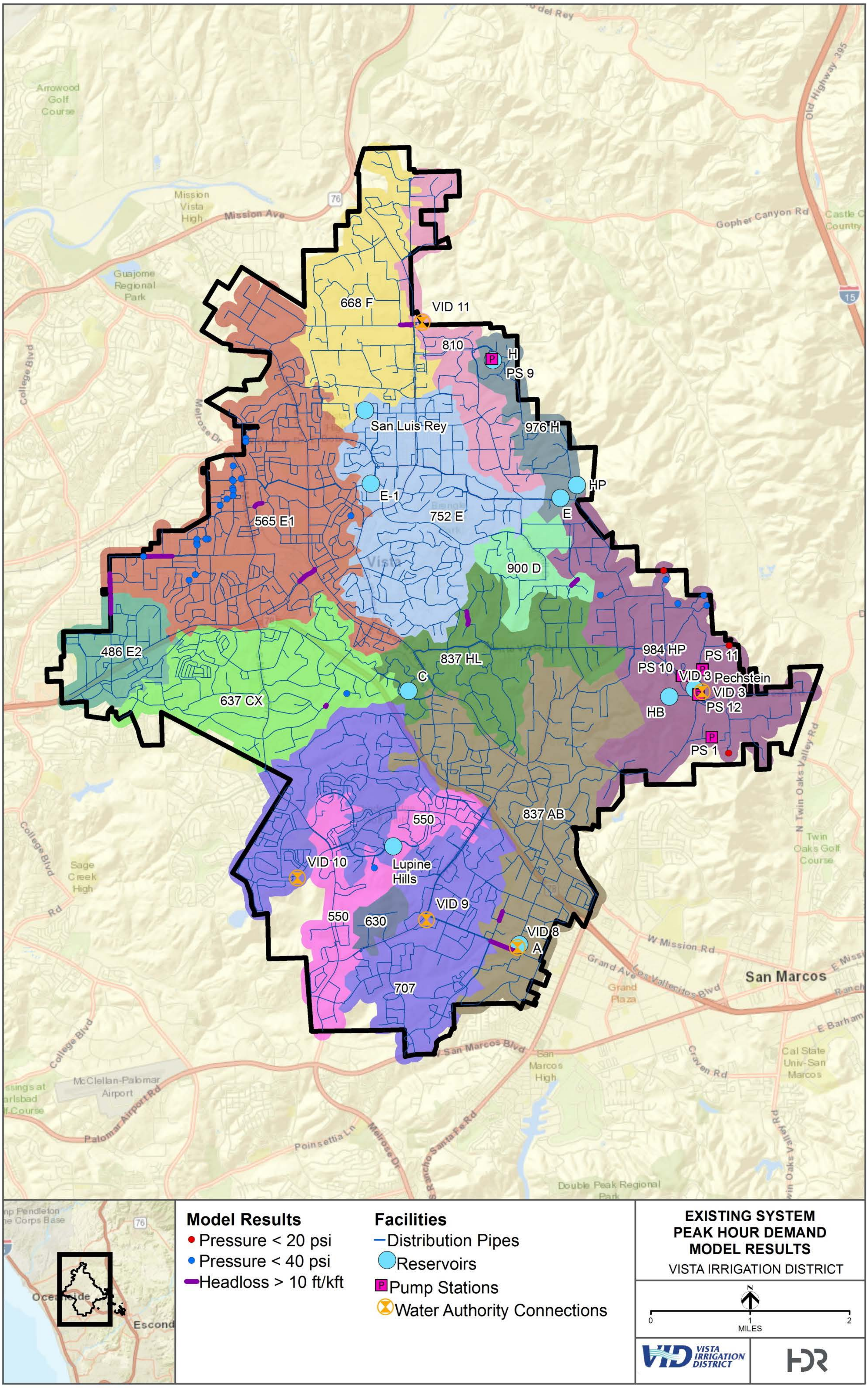
Figure 7-1. Existing System Peak Hour Demand Model Results



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7.2.2 Maximum Day Demand plus Fire Flow

Fire flow simulations were run using the InfoWater Fireflow simulation module. Required fire flows were loaded to the model based on the planning criteria presented in **Chapter 5,** and the existing land use presented in **Chapter 2**. Fire flow criteria include minimum residual pressure and maximum velocity limits of 20 psi and 16 fps, respectively, during MDD+FF conditions. Fire flow deficiencies within the existing system are primarily located at hydrants on small diameter, dead-end pipes. Under the District's rehabilitation program, these pipelines will be considered for upsizing when condition assessment indicates a need for replacement.

7.2.3 Minimum Day Demand

Maximum system pressures and water age were analyzed using MinDD EPS. These model simulations were run with flow supplied by the Vista Flume and the Water Authority connections, similar to the MDD scenario but scaled back to meet the average MinDD. The results are illustrated in **Figure 7-2**. Based on the evaluation criteria of 150 psi for maximum desired pressure, the model results indicate potentially high system pressures primarily in the 837, 752, and 707 zones. Additionally, results show high pressures in the northern area of the system in the 668 and 810 zones. The model results suggest local elevations primarily contribute to the pressures exceeding the evaluation criteria (e.g. located near the borders of the lower zones). Most of the high pressure model nodes are located in the 837 zone.

MinDD model simulations were also run to assess water age. Model results, as shown in **Figure 7-3**, indicate under these conditions, some areas of the system experience water age older than 10 days. Primarily, the northern portions of the 565 zone and the 707 zone show older water age. Additionally, the western area of the 637 zone shows higher water age. These areas of the system do not have source water connections and rely on gravity flows from other zones, which increases the age of the water used to satisfy local demands. Recommendations for improving water age during low demand conditions include operational adjustments limiting the amount of water stored in reservoirs to the minimum required by the storage requirements discussed in **Chapter 4,** which would accelerate turnover within the reservoirs and improve water movement within the system.

As system demands seasonally decrease, the required volume of operational storage decreases. Based on demands and system performance, reservoirs could also be taken offline during periods of low demand. For example, the E1 Reservoir provides redundant storage in the 565 zone that may not be needed during low demand periods. Reservoirs are also controlled via SCADA and can be operated at lower levels to optimize water quality and desired storage levels. However, during low demand periods when planned shutdown of Water Authority Aqueduct connections occur, these reservoirs are critical to providing supply to District customers and should not be taken off line.

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Figure 7-2. Existing System Minimum Day Demand Pressures

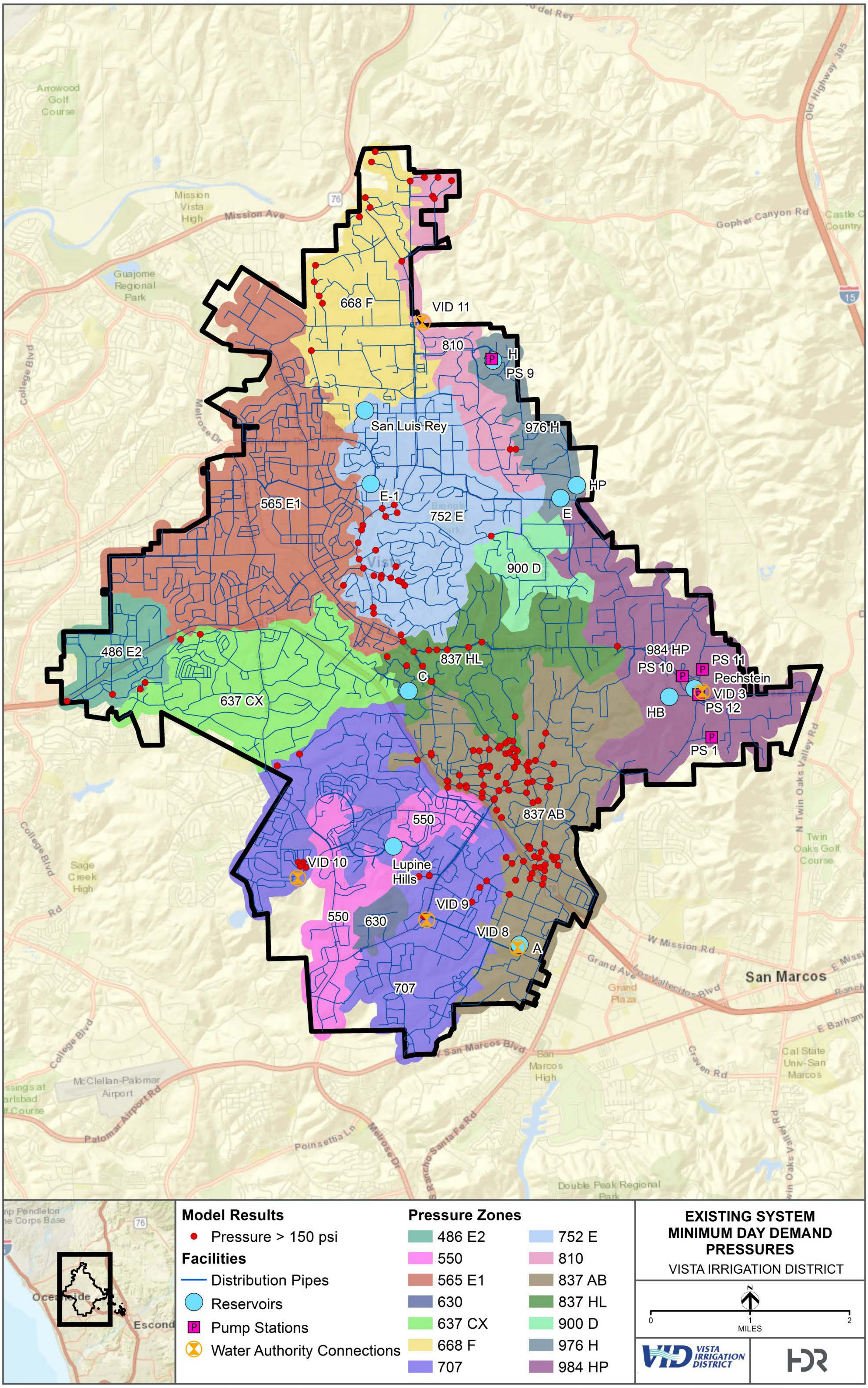

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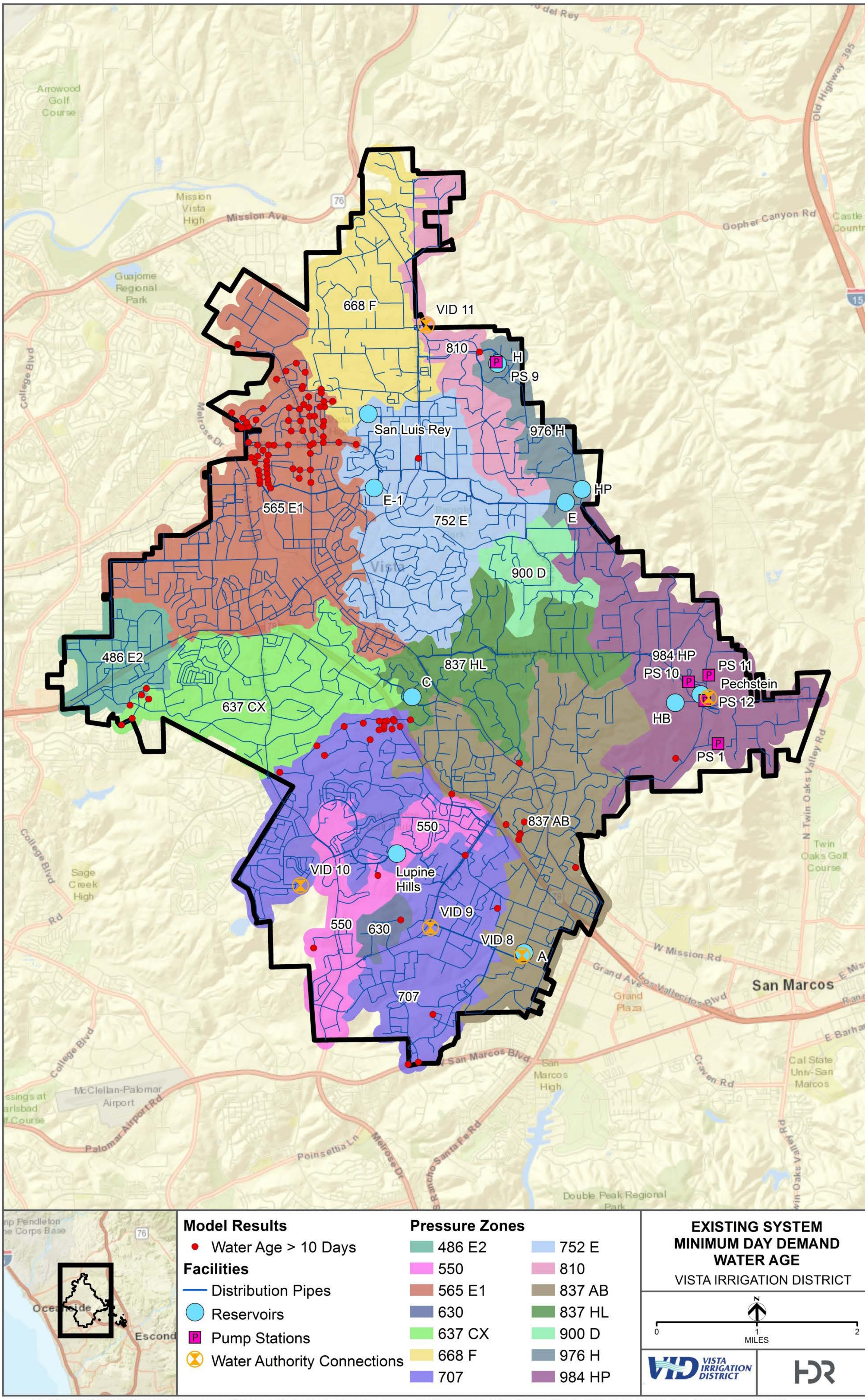
Figure 7-3. Existing System Minimum Day Demand Water Age



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7.3 Recommended Existing System Improvements

The recommended existing system improvements are summarized in **Table 7-2** and their locations are shown in **Figure 7-4**. These recommended improvements were identified in the analysis with the InfoWater hydraulic model, as well as discussions with District staff. Some of the projects developed as part of this analysis had also been recommended as part of the 2000 Master Plan. For reference, the corresponding 2000 Master Plan project numbers are included in **Table 7-2**. The 2000 Master Plan recommended improvement project descriptions are included in **Appendix H**, for reference**.**

A review of the existing system hydraulic model results indicates that there are no hydraulic deficiencies identified based on current demand conditions that warrant improvement projects. Areas of the system, primarily relatively high elevation areas of the 565 and 976/984 zones, already have static pressures or experience periodic operating pressures below the desired 40 psi criteria at PHD. While operating pressures below 40 psi are not ideal, the pressures are due to local high elevations and do not drop below the 20 psi criteria, and therefore do not warrant improvement projects.

Additionally, model results indicate that some pipes in the existing system may experience headloss higher than the desired 10 feet per 1,000 feet at PHD. However, the headloss in these pipes are not attributed to low system pressures, and model results indicate that velocity in these pipes does not exceed the 8 fps criteria at PHD. Therefore, these pipes do not negatively affect system operation and do not warrant system improvements.

As the existing system model results did not indicate the need for hydraulic deficiency improvement projects, the focus of the existing system assessment was shifted to improving system redundancy.

The first two improvement projects (EX-1 and EX-2) listed in **Table 7-2** address the addition of a third PRS providing flow to the 637 zone and takes advantage of the robust transmission system along Santa Fe Avenue. Project EX-1 includes the construction of a PRS to convey flows from the 837 zone to the 637 zone. Project EX-2 provides additional capacity to relieve high velocities resulting from the construction of EX-1. EX-1 was included in the 2000 Master Plan as part of ULT-5 and ULT-20. EX-2 was included in the 2000 Master Plan as part of ULT-1.

Project EX-3 consists of a large diameter pipe alignment to provide redundant supply out of Pechstein Reservoir. The alignment parallels an existing large diameter pipe connecting Pechstein Reservoir to the 837 zone in Buena Creek Road with additional new pipe in Buena Creek Road and Monte Vista Drive, relocating the cross country alignment recommended in the 2000 Master Plan as EX-6.

Project EX-4 consists of providing a third PRS feed to the 900 zone. The project would connect the 976/984 zone to the 900 zone via new pipe and a PRS between San Clemente Way and Huntalas Lane.

Project EX-5 consists of constructing a new pump station at E Reservoir that would allow flow to be conveyed from the 752 zone to the 976/984 zone, adding operational reliability and flexibility to the existing system.

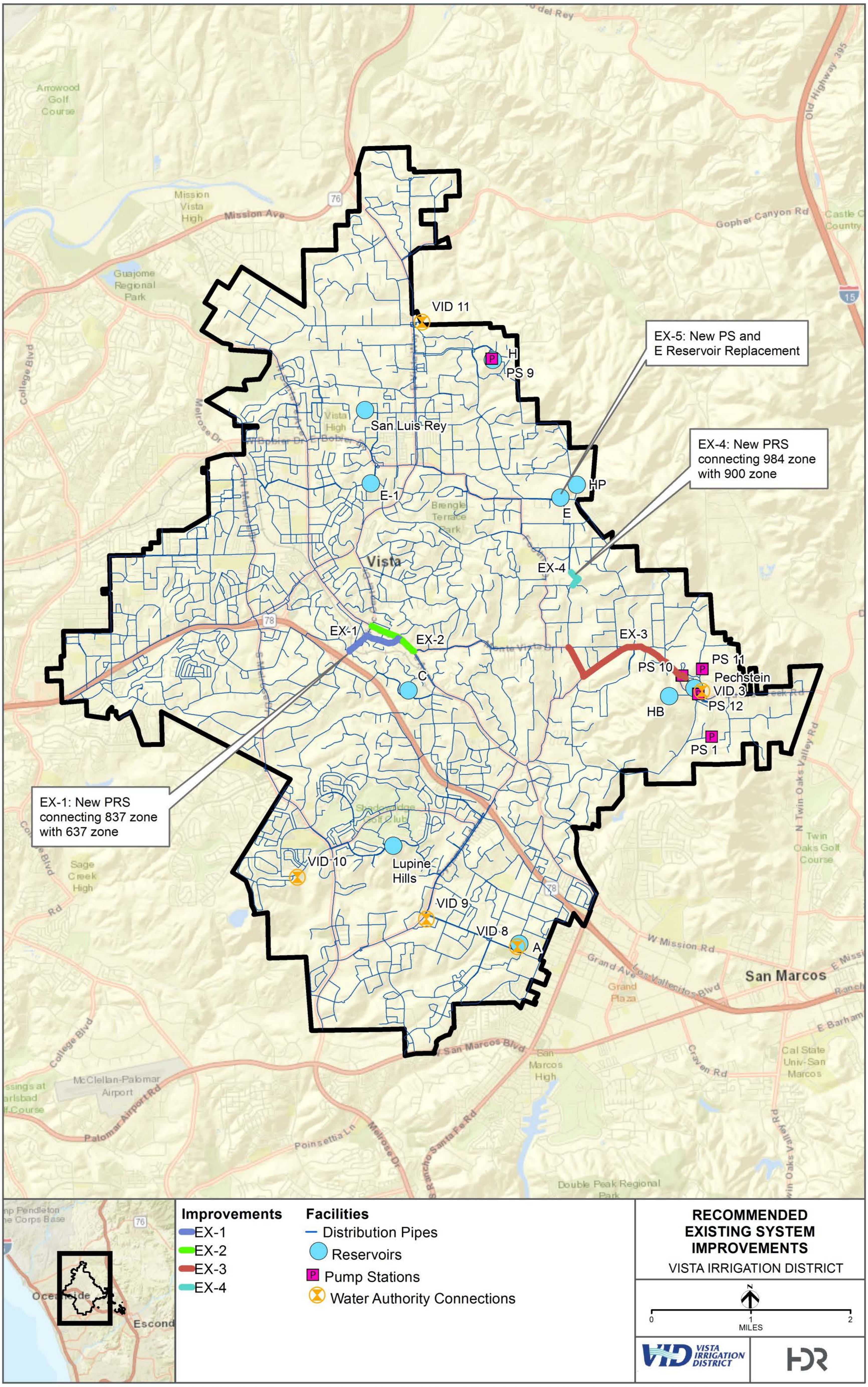
Detailed descriptions of the individual recommended projects are provided in the following paragraphs and project locations are indicated in **Figure 7-4.**

Table 7-2. Recommended Existing System Improvements

Job Number	Description	Diameter (Inches)	Length (Feet)	Reason	2000 Master Plan Project Number(s)
EX-1	Construct new 637 zone PRS along Civic Center Drive	N/A	N/A	Redundant connection to the 637 zone	ULT-5 ULT-20
	New 12-inch pipe in Postal way from E43 PRS to Civic Center Drive and southwest down Civic Center Drive to new 637 PRS	12	3,211		
	Parallel 8-inch pipe in Civic Center Drive from new 637 zone PRS to Phillips Street	8	241		
EX-2	Parallel 12-inch pipe in South Santa Fe Avenue from Monte Vista Drive to E43 PRS and continuing to Civic Center Drive	12	2,665	High velocities pipes in South Santa Fe Avenue, resulting from the addition of EX-1, and increasing capacity to 18-inch pipe installed in South Santa Fe Avenue at Civic Center Drive	ULT-1
EX-3	New 30-inch pipe from Pechstein Reservoir to PS 10	30	645	Redundant feed out of Pechstein to 837 zone	EX-6 (variant)
	New 24-inch pipe parallel to existing 26-inch pipe from PS 10 to Sugarbush Drive parallel to Buena Creek Road	24	3,386		
	New 24-inch pipe in Buena Creek Road from Sugarbush Drive to Monte Vista Drive	24	3,126		
	New 24-inch pipe replacing existing 12- and 10-inch pipe in Monte Vista Drive from Buena Creek Road to La Rueda Drive	24	1,759		
EX-4	Construct new PRS connecting 976/984 zone and 900 zone between San Clemente Way and Huntalas Lane	N/A	N/A	Redundant connection to the 900 zone	
	New 8-inch pipe connecting 976/984 zone and 900 zone via new 900 PRS	8	1,006		
EX-5	New PS at E Reservoir	N/A	N/A	Provides flexibility for conveying water from the 752 zone to 837 and 976/984 zones	
	New pipe connecting E Reservoir PS to desired zone(s)	Up to 24	up to 1,000		

PRS - pressure regulating/reducing station; PS – pump station

Figure 7-4. Recommended Existing System Improvements



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7.3.1 Third Supply to 637 Zone (EX-1)

Project EX-1 should be a fairly high priority for the District. This project provides a third supply to the 637 zone. The CX28 PRS and the C Reservoir PRS are the only supplies to the zone and the C Reservoir is not large enough to supply the PHD. Project EX-1 provides an additional supply to the 637 zone from the large diameter pipe alignment in Monte Vista Drive and Santa Fe Avenue, to a new PRS near the intersection of Civic Center Drive and Phillips Street. New 16-inch diameter pipe in Postal Way and Civic Center Drive is recommended to convey flow from the existing pipe alignment in Santa Fe Avenue, just upstream of the E43 PRS, to the new EX-1 PRS.

Additional pipe is also recommended parallel to existing pipes in Civic Center Drive to avoid the potentially high velocities introduced by the new PRS. Alternatively, the existing pipe in Civic Center Drive could be replaced with a new larger diameter pipe in order to increase capacity.

The location of the new PRS with regard to the system schematic is shown in **Figure 7-5** with the PRS labeled EX-1. It is recommended that the valve be sized to convey a 1,000-gpm peak flow. Approximately half of the 637 zone AAD demand (500 gpm) would be conveyed through the valve in the case of an outage of one of the two existing PRS feeds to the 637 zone; however, model results indicate that demand and reservoir fluctuations could result in approximately 1,000 gpm under PHD conditions. The pressure setting in the model was set to reduce pressures to 97 psi at an elevation of 400 feet.

7.3.2 Additional Capacity in Santa Fe Avenue Upstream of E43 Pressure Regulating Station (EX-2)

In the existing distribution system, two large 30-inch diameter pipes converge to a relatively small 10-inch diameter pipe at the intersection of Monte Vista Drive and Santa Fe Avenue. According to model simulations, the addition of project EX-1 potentially increases the peak hour velocities in the existing 10-inch pipe significantly enough to trigger the 8 fps evaluation criteria. The recommended solution is to install new 12-inch diameter pipe parallel to existing pipe from the intersection of Monte Vista Drive and Santa Fe Avenue to the E43 PRS. Alternatively, the existing pipe in Santa Fe Avenue could be replaced with new larger diameter pipe in order to increase capacity. This project is similar to the project ULT-1 in the 2000 Master Plan.

7.3.3 Second Feed out of Pechstein Reservoir (EX-3)

Pechstein Reservoir is the largest reservoir in the system with a capacity of 20.0 MG. The reservoir outlet is currently limited to a single 26-inch main connecting to the 837 zone. A redundant feed out of Pechstein Reservoir to the 837 zone was recommended in the 2000 Master Plan and is also recommended in this Master Plan. The 2000 Master Plan identified two potential alignments, the first (referred to as EX-6) being a feed to the south of the reservoir location paralleling the existing AB line, connecting the 976/984 zone to the AB PRS through a cross country alignment. However, this cross country alignment requires tunneling, is hard to access, and may result in environmental permitting issues.

Therefore, the District would prefer the second, northern alignment, which consists of a 24-inch feed from Pechstein Reservoir paralleling the existing large diameter feed along Blue Bird Canyon Road to PS 10. The 24-inch pipe would continue to parallel the existing feed down Buena Creek Road to the intersection of Sugarbush Drive. From this location the new 24-inch pipe would deviate from the alignment of the existing feed and continue down Buena Creek Road to the intersection of Monte Vista Drive. From the intersection of Buena Creek Road and Monte Vista Drive, the new redundant feed would continue north in Monte Vista Drive to existing pipe in La Rueda Drive.

7.3.4 Third Supply to 900 Zone (EX-4)

The 900 zone is currently supplied by only two PRSs: D1 and HL16. Additionally, the zone has no storage and relies on PRS flows to satisfy demands in all conditions. District staff indicated a redundant connection to the 900 zone as a priority.

Project EX-4 recommends adding an additional PRS connecting the 900 zone to the 976/984 zone. EX-4 consists of new 8-inch pipe parallel to existing pipe alignment connecting the existing 976/984 zone pipe at the southern end of San Clemente Way and existing 900 zone pipe in Huntalas Lane. The new PRS would be installed at an accessible location along this alignment.

The location of the new PRS with regard to the system schematic is shown in **Figure 7-5,** with the PRS labeled EX-4. It is recommended that the valve be sized to convey 32 gpm, approximately half the average demand of the 900 zone. The pressure setting in the model was set to reduce pressures to 70 psi at an elevation of 744 feet.

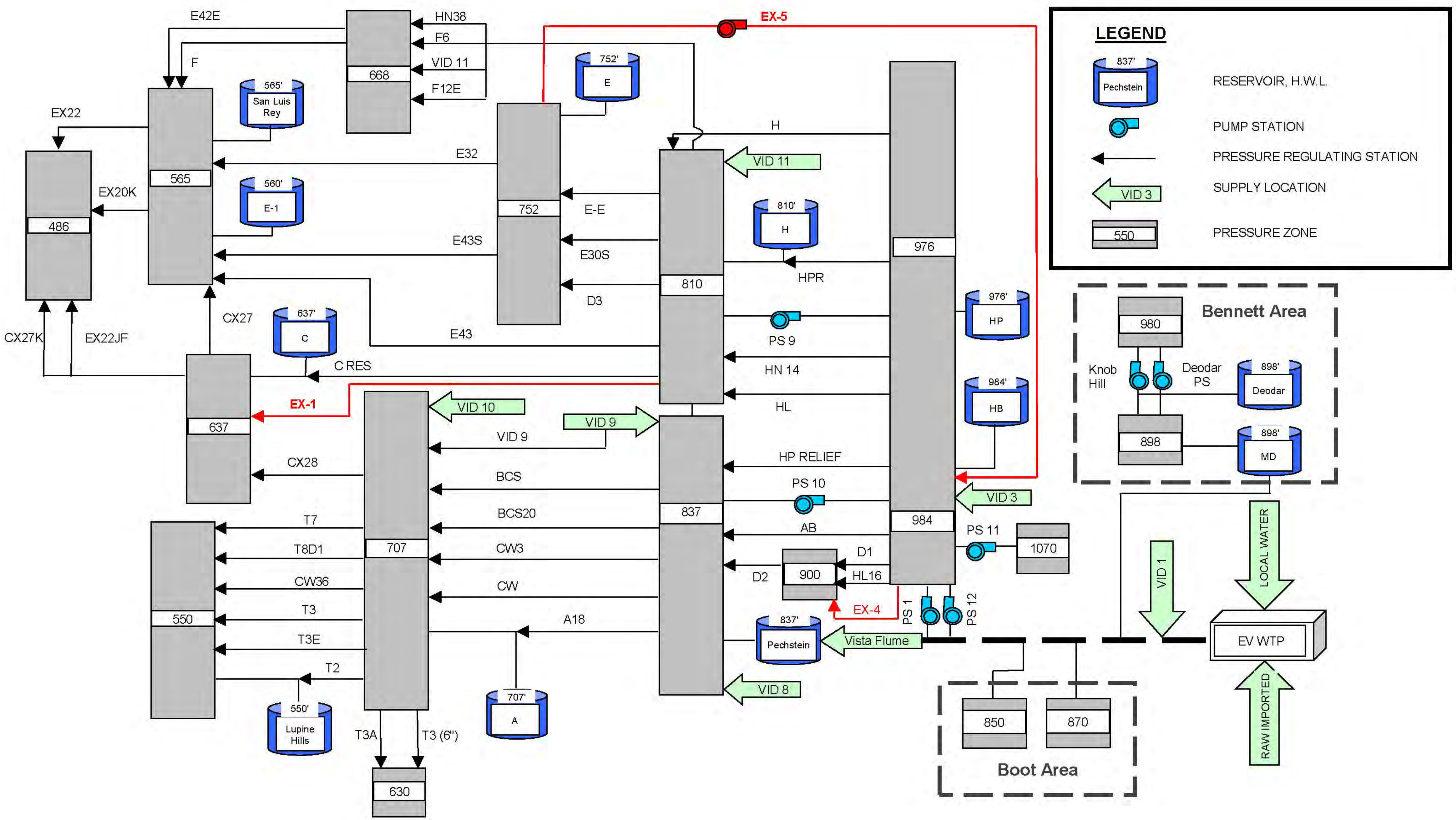
7.3.5 E Reservoir Pump Station (EX-5)

District staff indicated that a new pump station located at the E Reservoir would increase system reliability and potentially provide redundancy for certain supply interruptions by allowing the District to convey flows from the 752 zone to the 976/984 zone, all of which converge near the E Reservoir. The Water Authority connection VID 11 currently feeds into the 752 zone via the E30S PRS. In the event that the primary supplies to Pechstein, including the Vista Flume and VID 3, are offline, a pump station at E Reservoir could give the District more options for moving water from VID 11 to the higher zones of the system (976/984 zone).

However, preliminary model runs, with the proposed E Reservoir PS pumping into the existing system and reversing flow direction in the distribution system, indicate that this scenario may cause high pressure issues near the proposed pump station. These issues, and strategies for redundant supply and distribution when the Vista Flume and VID 3 are offline, are discussed further in **Chapter 8**.

Capacity of the proposed E Reservoir PS could be as much as 7,000 gpm under MDD conditions with no supply from the Flume or VID 3, but could be less, depending on the level of supply reliability the District wishes to achieve. For the purposes of the existing proposed improvements list, infrastructure related to the E Reservoir PS includes sufficient lift capacity to convey flows to the 976/984 zone, which is the highest head zone near E Reservoir. Head required to convey flow to the HB Reservoir in the 976/984 zone is approximately 700 feet with a flow rate of 7,000 gpm. Pipe required to connect to the 976/984 zone and convey this flow would be approximately 1,000 feet of 24-inch diameter pipe.

Figure 7-5. Existing System with Improvements Schematic



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7.4 Existing System Storage Assessment

The required reservoir storage based on existing system demands and the storage criteria defined in **Chapter 4** is presented in **Table 7-3**. The storage criteria require the reservoirs to have sufficient capacity to provide 10 percent of MDD plus either the fire flow requirements within that zone, or 2 days of AAD, whichever is larger. The storage assessment is based on existing demands and available storage for each zone. Demands were estimated using the methodology discussed in **Chapter 5** and allocated to pressure zones based on meter location. It was also assumed that higher zones with excess capacity, such as H Reservoir, HB Reservoir, and HP Reservoir, would supplement storage deficiencies in lower zones.

Based on the required storage calculations, the existing system currently has 3.47 MG of excess storage capacity. However, individually, the 707, 637, 752, 565, and 486 zones all have insufficient storage based on current demands. The higher elevation zones have excess capacity, notably the 837 zone has excess storage capacity due to Pechstein Reservoir. In the case of fire or emergency, the deficient lower pressure zones could be supplied by the higher zones with excess storage via gravity.

It should be noted that the storage assessment presented in **Table 7-3** does not account for Water Authority aqueduct shutdowns. As discussed in **Chapter 4**, planned aqueduct shutdowns can last for up to 10 days during which the District must rely on its own local water supply and storage reserves to meet demands. Currently, the District relies on local water treated at the EVWTP and conveyed via the Flume during shutdowns. Scenarios and responses to planned or emergency outages are addressed in **Chapter 4.**

Table 7-3. Existing System Storage

Major Pressure Zone	Zone Grade (Feet)	AAD		MDD[1] (MGD)	Storage Criteria[2]						Reservoir	Existing Operational Storage (MG)	Surplus (Deficit) (MG)
		(gpm)	(MGD)		Operational (Gallons) +	Fire (Gallons)	or	Emergency (Gallons)	=	Total (MG)			
HB Zone	984, 900	687	0.99	1.98	197,811	300,000		1,978,109		2.18	HB	4.05	1.87
HP Zone	976	148	0.21	0.43	42,582	300,000		425,822		0.47	HP	4.05[3]	3.58
AB/HL Zone	837	1,631	2.35	4.70	469,612	540,000		4,696,121		5.17	Pechstein	18.50	13.33
810, F Zone	810, 668	779	1.12	2.24	224,387	540,000		2,243,870		2.47	H	5.00	2.53
707 Zone	707	1,506	2.17	4.34	433,674	735,000		4,336,739		4.77	A	0.60	(4.17)
CX Zone	637	1,024	1.48	2.95	295,055	540,000		2,950,553		3.25	C	0.60	(2.65)
E Zone	752	1,508	2.17	4.34	434,278	540,000		4,342,779		4.78	E	1.20	(3.58)
550 Zone	550	684	0.98	1.97	196,905	735,000		1,969,049		2.17	LH	3.00	0.83
E-1, E-2 Zone	565, 486	3,629	5.23	10.45	1,045,227	735,000		10,452,266		11.50	SLR, E1	3.20	(8.30)
Totals		11,596	16.70	33.40	3,339,531	4,965,000		33,395,307	36.73			40.20	3.47

[1] MDD= 2.0 x AAD

[2] Operational = 0.1 x MDD
Fire = Max fire flow demand and duration within the zone per the fire flow requirements in **Table 4-3**, including 2,500 gpm for 2 hours (300,000 gallons) in wild fire interface areas.
Emergency = 2.0 x AAD
Total = Operational + the larger of fire or emergency storage criteria

[3] HP storage volume, prior to rehabilitation in 2017

AAD – average annual demand; gpm - gallons per minute; MG – million gallons; MDD – maximum day demand; MGD – million gallons per day

8 Ultimate System Analysis and Recommendations

This chapter addresses potential deficiencies in the District's potable water distribution system under ultimate demand conditions. As discussed in **Chapter 3**, ultimate system demands are based on a combination of land use demand values and planned land uses representing the service area at ultimate buildout.

This chapter also discusses projects recommended to improve system performance under ultimate demand conditions. Water distribution system facility improvements are proposed to improve basic operations, meet hydraulic evaluation criteria, and increase system reliability.

8.1 Ultimate System Analysis

Ultimate system analyses were based on the existing system hydraulic model, updated to include the recommended improvements discussed in **Chapter 7,** and loaded with the projected ultimate demands discussed in **Chapter 3**. Model simulations of PHD and MDD+FF were run to identify high pipe headloss, pipe high velocity, and low system pressures.

8.1.1 Maximum Day and Peak Hour Demand

MDD EPS model runs were performed to identify peak hour deficiencies, similar to the model analysis runs performed on the existing system and discussed in **Chapter 7.**

Similar to the existing system analysis, Water Authority flows were introduced at all turnouts except for VID 8, and flows were adjusted to provide the average supply to meet demand and maintain system operations. Flow from the Vista Flume was also included in the model. Modeled supply flows for the MDD EPS scenario are listed in **Table 8-1**.

Model results for the ultimate system PHD scenario indicate pressures below the 40 psi preferred low pressure criteria occur as demands increase, primarily in the western central area of the 565 zone. Model results also indicate that a few relatively high elevation locations on the periphery of the 984 zone may experience pressures below the minimum required pressure criteria of 20 psi. These are the same locations identified in the existing system analysis and discussed in **Section 7.3**. With slight operational adjustments, lower pressures that occur in the 486 zone under ultimate conditions can be alleviated by increasing the reducing pressure settings on the PRS feeding into the zone (EX22, EX20K, EX22JF, and CX27K) by 1 to 2 psi.

Model results also indicate some pipes exceeding the high headloss criteria throughout the distribution system, including large stretches of pipe in the 668 zone and pipes near the A Reservoir. Additionally, two reaches of pipe indicate velocity higher than the 8 fps criteria located in the 486 and 565 zones. Model results for the existing PHD scenario are shown in **Figure 8-1**.

Table 8-1. Maximum Day Demand Supply Summary

Supply Location	Pressure Zone	Flow (under MDD conditions)	
		(gpm)	(cfs)
Vista Flume	837	11,600	25.8
VID 3	984	4,075	9.1
VID 8	837	-	-
VID 9	837/707	5,177	11.5
VID 10	707	1,747	3.9
VID 11	810/668	6,651	14.8
	Total	29,251	65.2

cfs - cubic feet per second; gpm - gallons per minute; MDD – maximum day demand; VID – Vista Irrigation District

8.1.2 Maximum Day Demand plus Fire Flow

Fire flow simulations were run using the InfoWater Fireflow simulation module. Required fire flows were loaded to the model based on the planning criteria presented in **Chapter 5** and the existing land use presented in **Chapter 2**. Fire flow criteria include minimum residual pressure and maximum velocity limits of 20 psi and 16 fps respectively during MDD+FF conditions. The majority of existing system fire flow deficiencies consist of hydrants located on dead-end pipes. Likewise, fire flow deficiencies within the ultimate system are primarily located at hydrants on small diameter, dead-end pipes. Under the District's rehabilitation program, these pipelines will be considered for upsizing when condition assessment indicates a need for replacement.

8.1.3 Minimum Day Demand

Maximum system pressures and water age were analyzed using MinDD EPS. These model simulations were run with flow supplied by the Vista Flume and the Water Authority connections. Because there is more demand under ultimate conditions, water age improves throughout the system. Based on the evaluation criteria of 150 psi for maximum allowable pressure, the model results indicate potentially high system pressures primarily in the 837, 752, and 707 zones. These results are similar to the existing system model results, with some additional nodes within the vicinity of the areas currently identified as having high pressures. Additionally, results show high pressures in the northern area of the system in the 668 and 810 zones. The model results suggest local elevations primarily contribute to the pressure issues as defined by the evaluation criteria. Most of the high pressure model nodes are located in the 837 zone.
Figure 8-2 displays the MinDD high pressure locations under ultimate conditions.

Figure 8-1. Ultimate System Peak Hour Demand Model Results

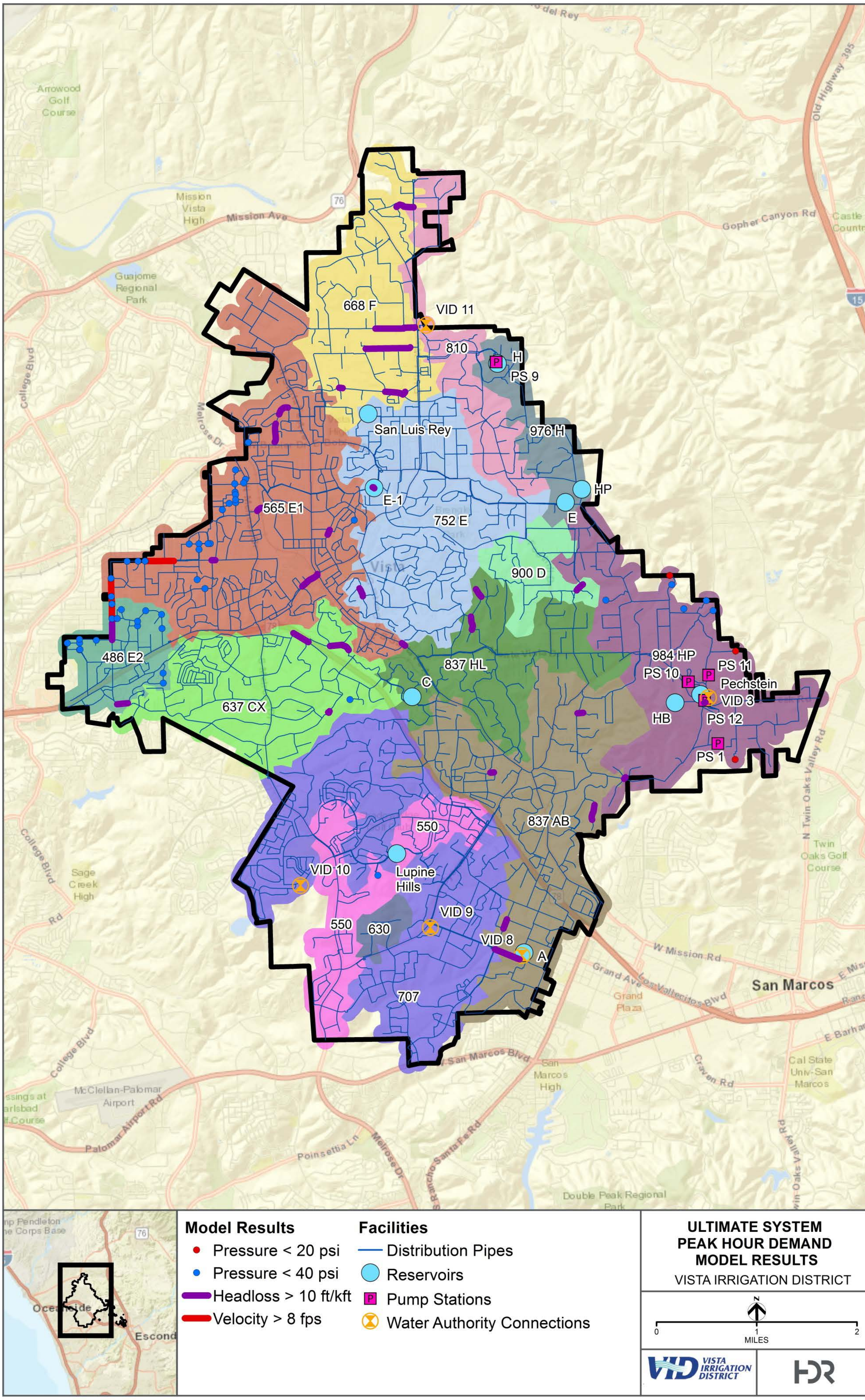

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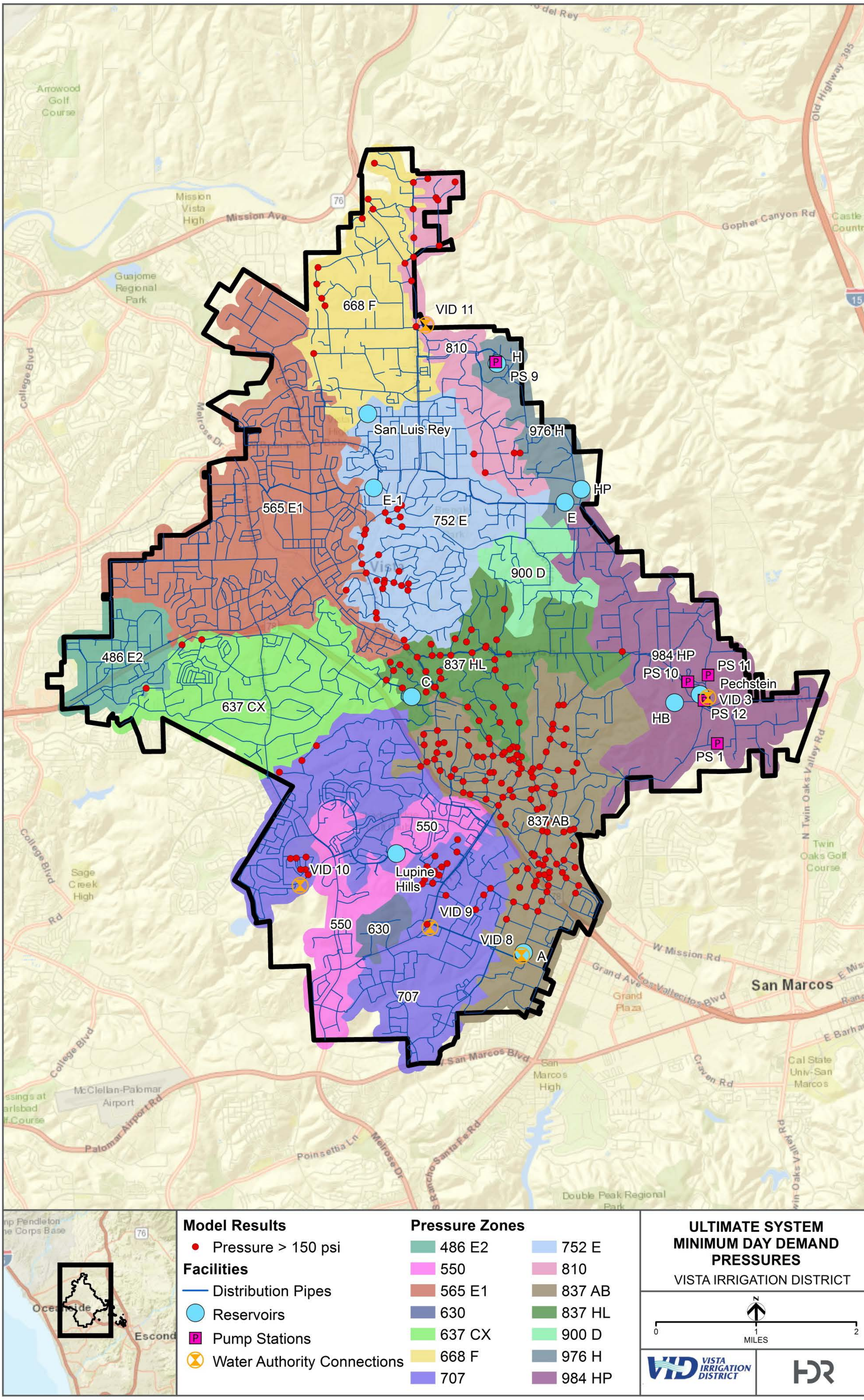
Figure 8-2. Ultimate System Minimum Day Demand Pressures



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8.2 Recommended Ultimate System Improvements

The recommended ultimate system improvements are summarized in **Table 8-2**. **Figure 8-3** shows the locations of the improvements in the distribution system. Improvements were identified based on analysis with the InfoWater hydraulic model and the evaluation criteria discussed in **Chapter 5**.

Ultimate system model runs were developed using the existing system infrastructure, plus the recommended existing system improvement projects discussed in **Chapter 7,** run with ultimate demands. Recommended ultimate system improvement projects are limited to one project addressing high velocities in pipes in the 565 zone. This project is described in more detail in the paragraph below. Further discussion on alternative strategies to increase the District's ability to move water from VID 9 and VID 11 to Pechstein Reservoir in the event of outages of the Vista Flume and/or VID 3 is also provided.

Table 8-2. Recommended Ultimate System Improvements

Project Number	Description	Diameter (Inches)	Length (Feet)	Reason
ULT-1	Installation of 10-inch diameter interconnection between 8-inch and 12-inch parallel pipes in Olive Avenue at the intersection of Grapevine Road	10	30	High velocities in pipes in Olive Avenue

8.2.1 High Velocity Pipes in Olive Ave (ULT-1)

The 565 zone has the largest total demand in the existing and ultimate systems. As a result, this zone can experience high local flows and associated high velocities. Model results indicate that one reach of 8-inch pipe in Olive Avenue, from Grapevine Road to Burke Road, may experience velocities exceeding the evaluation criteria under PHD conditions. This reach of 8-inch pipe is paralleled by a reach of pipe varying in diameter from 10 to 12 inches.

Project ULT-1 is recommended to alleviate the high velocities in the 8-inch pipe. ULT-1 consists of installing a 10-inch interconnection between existing parallel pipes in Olive Avenue at the intersection of Grapevine Road. Additionally, it is recommended that existing interconnections between parallel pipes in Olive Avenue at the intersection of Brookins Lane be open to allow flow in all directions. Model results indicate that these changes would allow parallel flow within the existing parallel pipes in Olive Avenue reducing the velocity in the 8-inch pipe to an acceptable level. Project ULT-1 is displayed in **Figure 8-3.**

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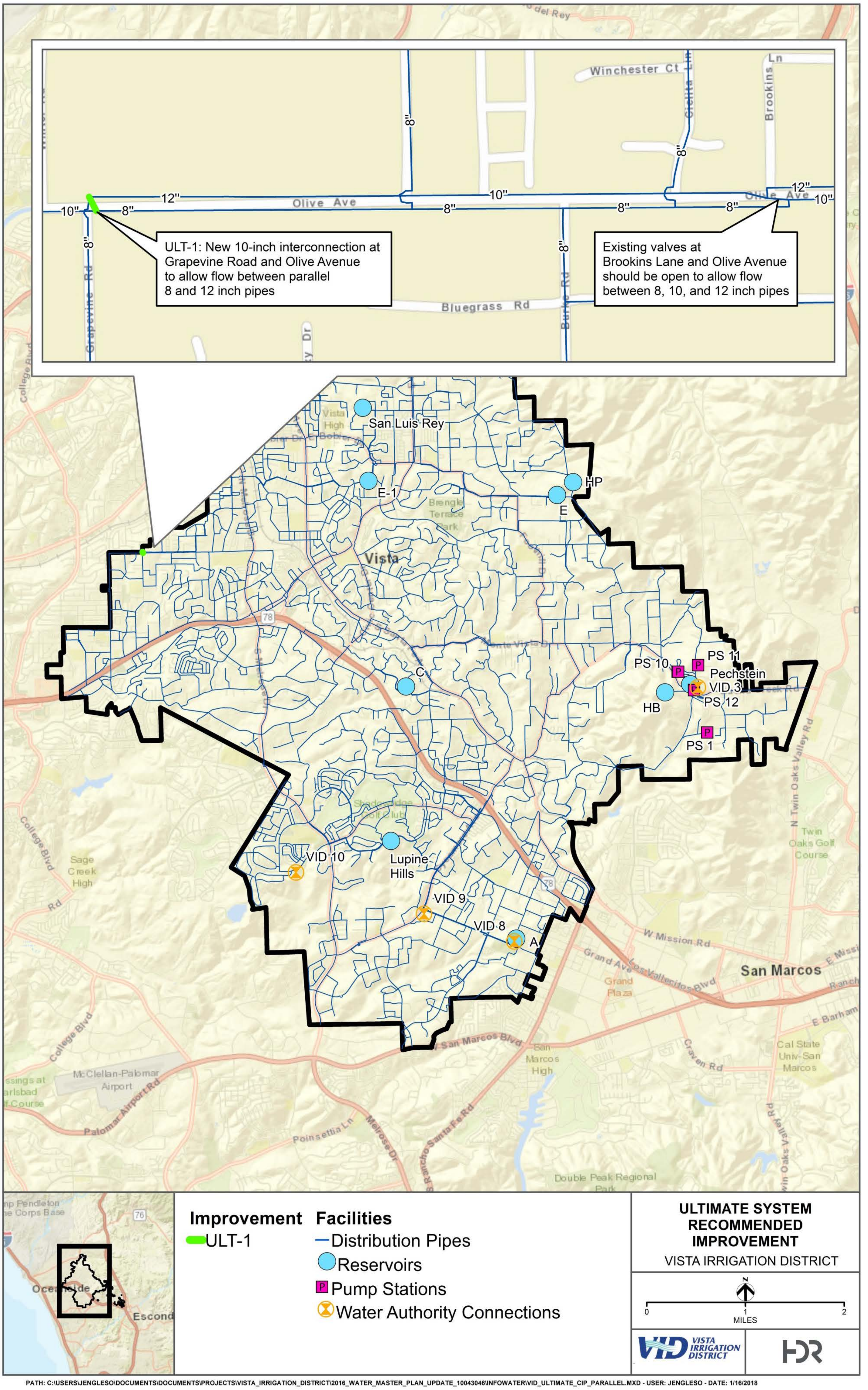
Figure 8-3. Recommended Ultimate System Improvements



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8.2.3 Redundant Water Supply Strategies

Water reliability improvement concepts were discussed in **Section 4.5**, including providing redundancy to VID 3 connection in conjunction with a long-term outage of the Vista Flume. A hydraulic model analysis was conducted for this scenario. The goal of this exercise was to find a way to convey water from both VID 9 and VID 11 to Pechstein, relying as much as possible on existing infrastructure, and satisfying evaluation criteria. The benefit of the District's ability to convey water from VID 9 and/or VID 11 to Pechstein is increased operational flexibility and the ability to operate the system using existing Water Authority connections in the event the Vista Flume and VID 3 are both offline.

The base scenario in this redundant alternatives analysis included existing infrastructure, plus existing and ultimate recommended improvement projects, and ultimate demands. MDDs were used to represent the system operating at full capacity. This represents a worst case condition, and the District may wish to evaluate facility requirements further under average or winter demand conditions. Both the VID 9 and VID 11 redundant supply concepts were included in a single scenario. **Figure 8-4** illustrates the location of facilities relevant to the redundant supply concepts.

Vista Irrigation District 9 Supply Strategy

The VID 9 redundant supply concept consists of extending an existing large diameter pipe from the VID 9 connection north along Sycamore Avenue and along Buena Creek Road. Currently, an existing 20-inch pipe extends from VID 9 northeast to the BCS PRS near the intersection of Sycamore Avenue and Highway 78. Approximately 6,600 feet of new 20-inch pipe would need to be added to extend the pipeline to its terminus at the intersection of Buena Creek Road and Canyon Drive. Additionally, with the exception of two existing turnoffs to PRS feeding the 707 zone (VID 9 PRS and BCS PRS), the large diameter pipeline would not connect to the distribution system until it reaches its new terminus at the intersection of Buena Creek Road and Canyon Drive. Currently, at least two connections from the existing 20-inch pipe to the existing 837 distribution system would need to be closed to isolate the flows. Isolating the pipeline would allow flow from VID 9 to enter the 837 zone nearer the Pechstein Reservoir.

The strategy for the VID 9 redundant supply concept is to avoid high distribution system pressures by introducing the flow at a relatively high elevation near the Pechstein Reservoir. This approach (1) decreases the pressure head at the point the flow is introduced to the distribution system, and (2) takes advantage of headloss across the 837 zone to decrease pressures in lower elevation areas of the zone. The VID 9 connection can be supplied by a total head value near 1,000 feet. By conveying the VID 9 flows to the elevation of 510 feet at the intersection of Buena Creek Road and Canyon Drive, the change in elevation head, minus headloss in the transmission main, bring the effective pressure below 170 psi at this location under ultimate MDD conditions.

On the other hand, the 837 zone is designed for water to feed the zone from Pechstein Reservoir. As the system is currently operated, headlosses across the distribution system decrease total head as water flows downhill from Pechstein. The result is a balance between headloss gradient and elevation head gradient that results in pressure head that is acceptable, although relatively high, at the lower elevations of the pressure zone. Introducing VID 9 flows nearer Pechstein Reservoir takes advantage of this system design, and allows the increased head due to larger VID 9 inflow to be decreased by the headloss gradient across the distribution system to the lower elevation areas of the zone.

Vista Irrigation District 11 Supply Strategy

The overall VID 11 supply strategy is to convey water from VID 11 to the higher elevation zones of the District's distribution system, specifically to Pechstein, in order to facilitate typical system operation in the case that the Vista Flume and VID 3 are offline. For the purposes of this Master Plan study, both the VID 9 and VID 11 supply strategies are assumed to be operational concurrently.

An E Reservoir PS was included in the existing improvements project list in **Chapter 7**. E Reservoir is located at the intersection of four major pressure zones: the 752, 810, 837, and 976/984. An E Reservoir PS (752 zone) could conceivably provide flow to any of the four higher zones in the immediate area. Because the distribution system was not designed to pump flows from E Reservoir to these adjacent zones, existing hydraulic capacity in the respective zones may not be sufficient to handle significant flows in a reversed direction from an E Reservoir PS.

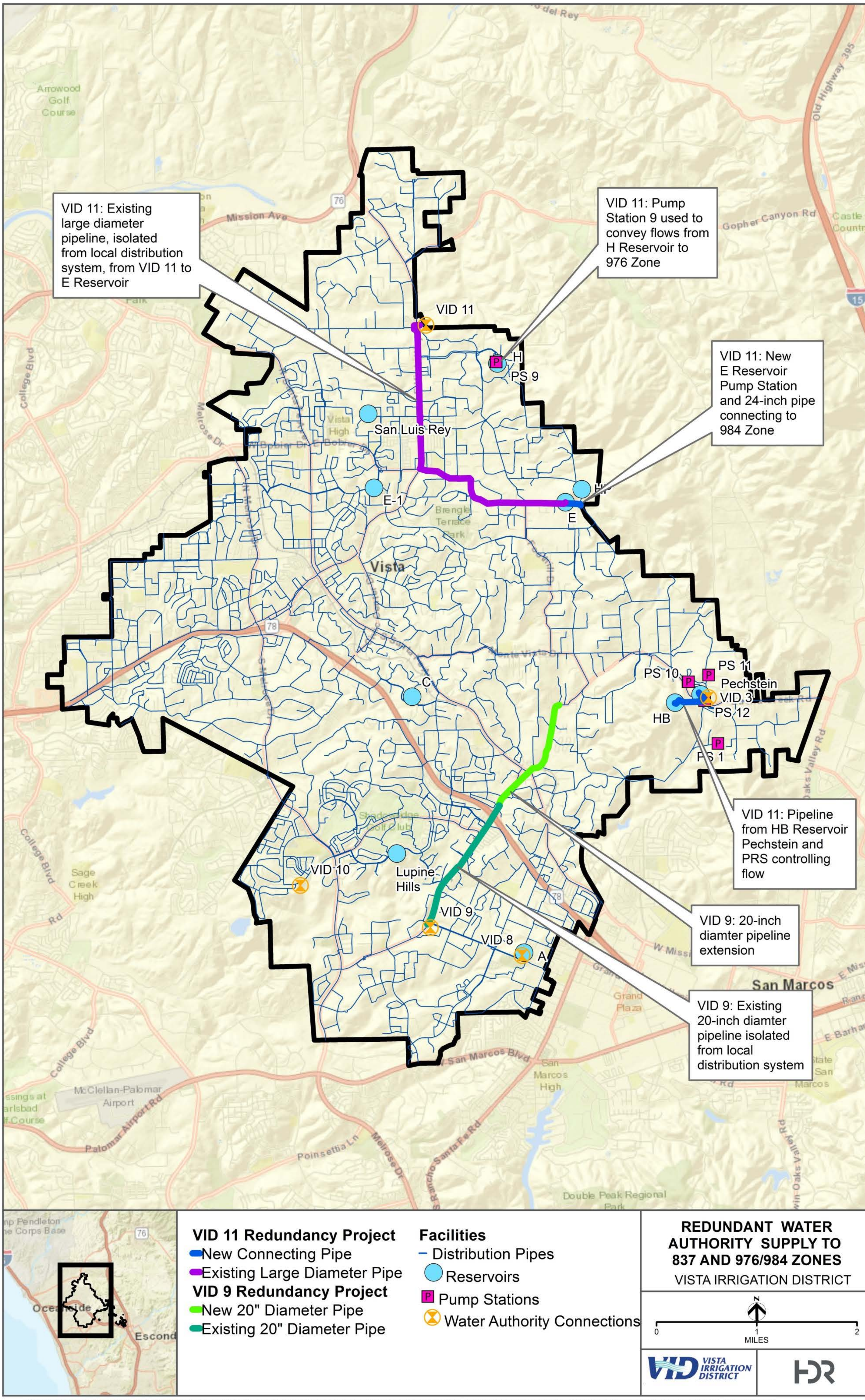
The VID 11 redundant supply concept consists of pumping flows from the E Reservoir to the 976/984 zone in order to fill the Pechstein Reservoir. The VID 11 Water Authority connection currently has a capacity of 50 cfs. In the current distribution system, VID 11 flows are split between the 668, 752, and 810 zones. Flows to the 810 zone are primarily fed into the H Reservoir. Flows to the 668 zone are reduced at PRSs and conveyed to lower zones. Flows to the 752 zone are reduced at the E30S PRS and conveyed to the E Reservoir via recently installed large diameter pipe ranging from 30 to 24 inches in diameter.

For the purposes of the VID 11 redundant supply strategy, this large diameter feed from E30S to the E Reservoir would be converted to an isolated, dedicated pipeline conveying flows to the E Reservoir at a pressure higher than the surrounding 752 distribution system. The pipeline would connect to the existing distribution system at Bobier Drive and Vista Way in order to convey flows to the E32 PRS. Additionally, the pipeline would connect to the distribution system at Foothill Drive and Edgehill Road in order to convey flows to the 752 zone. The existing E30S PRS would be adjusted from a setting of 102-103 psi to 132-133 psi to increase the head in the dedicated pipeline and provide increased flow to the E Reservoir.

The new E Reservoir PS would convey flow from the E Reservoir directly to the 976/984 zone. The 976/984 zone would be isolated from the adjacent zones near the E Reservoir, so that flows from the E Reservoir PS are conveyed to the HB Reservoir. The 976/984 zone would be supplied by the HP Reservoir and existing PS 9 from the H Reservoir, which is fed by VID 11. Flows from the E Reservoir PS to the HB Reservoir would be used to fill the Pechstein Reservoir via the existing HP relief valve.

The strategy for conveying flows from the E Reservoir to the 976/984 zone is to avoid the high pressure issues resulting from pumping flow directly into the 837 zone at the E Reservoir location. By conveying flow to the 976/984 zone, pressures in the 837 zone are not affected and Pechstein can be filled from the higher zone.

Figure 8-4. Redundant Water Authority Supply to 837 and 976/984 Zones



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Resulting System Deficiencies

Implementing the VID 9 and VID 11 redundant water supply alternatives results in acceptable operating pressures but also creates pipe velocities above the evaluation criteria of 8 fps under ultimate PHD conditions. Further study is required to assess specific demand conditions and mitigation measures that could alleviate these high velocities. Pipes experiencing high velocities include the following.

- 18-inch diameter pipe in Edgehill Road
- 20-inch diameter pipe Mango Glen to Catalina Heights Way
- Various pipes in Buena Creek Road
- 14-inch feed into HB Reservoir

8.3 Storage Assessment

The required reservoir storage based on ultimate system demands and the storage criteria defined in **Chapter 4** is presented in **Table 8-3**. The storage assessment is based on ultimate demands and storage for each zone. Projected ultimate demands were estimated using the methodology discussed in **Chapter 3** and allocated to pressure zones based on land use type. It was also assumed that zones with excess capacity would supplement storage deficiencies in other zones. As with the existing storage assessment discussed in **Chapter 7**, the ultimate system storage assessment presented in **Table 8-3** does not account for storage required during Water Authority aqueduct shutdowns.

Based on the required storage calculations, the ultimate system is projected to have a storage deficit of 3.88 MG. As with the existing system storage assessment, the 707, 637, 752, and 565 zones are projected to have insufficient storage based on projected demands. The remaining zones have excess capacity, notably the 837 zone has significant excess storage capacity in Pechstein Reservoir.

The 2000 Master Plan recommended the construction of a 20 MG Pechstein II Reservoir to address the projected ultimate system deficiency and additional emergency storage. The proposed Pechstein II location, adjacent to the existing Pechstein Reservoir location, is advantageous based on the availability of District owned land to accommodate such a large reservoir, and its elevation. This would also allow the District to take the existing Pechstein Reservoir off line for rehabilitation. Additional storage serving the 837/810 zone would provide flows to all the lower zones projected to have storage deficiencies in the ultimate system. Any additional storage would need to have an operational capacity of at least 3.88 MG in order to offset the projected ultimate system storage deficiency.

Reservoir E is being considered for near term replacement. In 1995, the proposed replacement project consisted of a 146-diameter, 38-foot-high, 4.4 MG prestressed concrete reservoir, as discussed in **Chapter 4**. This reservoir would enhance emergency supply within the E zone, which requires 4.98 MG in the ultimate system. However, this site is significantly constrained by neighboring residences and sensitive habitat. Alternatively, the District’s total storage deficit would be offset with the addition of a Pechstein II Reservoir project.

Table 8-3. Ultimate System Storage

Major Pressure Zone	Zone Grade (Feet)	AAD[1]		MDD[2] (MGD)	Storage Criteria[3]						Reservoir	Existing Operational Storage (MG)	Surplus (Deficit) (MG)
		(gpm)	(MGD)		Operational (Gallons) +	Fire (Gallons)	or	Emergency (Gallons)	=	Total (MG)			
HB Zone	984, 900	1,233	1.78	3.55	355,029	300,000		3,550,286		3.91	HB	4.05	0.14
HP Zone	976	212	0.31	0.61	61,098	300,000		610,980		0.67	HP	4.30[4]	3.63
AB/HL Zone	837	2,770	3.99	7.98	797,722	540,000		7,977,218		8.77	Pechstein	18.50	9.73
810, F Zone	810, 668	1,136	1.64	3.27	327,179	540,000		3,271,790		3.60	H	5.00	1.40
707 Zone	707, 630	1,890	2.72	5.44	544,197	735,000		5,441,972		5.99	A	0.60	(5.39)
CX Zone	637	1,237	1.78	3.56	356,209	540,000		3,562,086		3.92	C	0.60	(3.32)
E Zone	752	1,571	2.26	4.52	452,444	540,000		4,524,438		4.98	E	1.20	(3.78)
550 Zone	550	711	1.02	2.05	204,855	735,000		2,048,550		2.25	LH	3.00	0.75
E-1, E-2 Zone	565, 486	3,154	4.54	9.08	908,438	735,000		9,084,379		9.99	SLR, E1	3.20	(6.79)
	Totals	13,914	20.04	40.07	4,007,170	4,965,000		40,071,700		44.08		40.45	(3.63)

[1] Buildout demands based on SANDAG Series 13 Planned Land Use and Unit Demand Factors rounded up to the nearest 50. Projected demands represent increased demand density compared with existing demands.

[2] MDD = 2 x ADD

[3] Total = Operational + larger of Fire or Emergency Storage Criteria'
Operational = 0.1 x MDD
Fire = Fire flow and duration per requirements in **Table 4-3**, including 2,500 gpm for 2 hours (300,000 gallons) in wild fire interface areas.
Emergency = 2 x AAD

[4] HP Reservoir volume, as rehabilitated in 2017.

AAD – average annual demand; MDD – maximum day demand; gpm - gallons per minute; MG – million gallons; MGD – million gallons per day

9 Capital Improvement Program Recommendations

This chapter summarizes the recommended CIP projects for the District's potable water distribution system based on the findings of this master plan. The recommended projects address redundancy or replacement and rehabilitation improvements for the existing distribution system and an ultimate system based on projected buildout demands. An estimate of probable construction cost is provided for each improvement project. These are planning level assessments; therefore, it is in the best interest of the District to conduct feasibility or preliminary engineering evaluations before embarking on a major capital investment.

9.1 Unit Costs

The opinions of probable construction costs are developed based on costs obtained from industry manufacturers and HDR's experience on similar water master planning projects. Some key cost assumptions are:

- All cost assumptions are based on 2017 U.S. Dollars (December 2017 Engineering News Record/Construction Cost Index 11935.82) and are consistent with the American Association of Cost Engineers guidelines for developing reconnaissance-level estimates (Class 5).
- 20 percent of construction costs for contingency is included in the cost estimates.
- 30 percent of construction cost for the engineering, administration, and legal costs is included in the cost estimates. The engineering, administration, and legal costs also include typical services such as inspection, materials testing and construction management.
- Escalation, land acquisition, environmental documentation, permits and easements costs are not included.

These estimates are based on representative available data at the time of this report; however, since prices of materials and labor fluctuate over time, new estimates should be obtained at or near the time of construction of proposed facilities.

A scaling factor is included on a project by project basis to account for pipeline projects that are relatively short in distance or have more significant environmental or construction challenges.

9.1.1 Distribution System Pipe Unit Costs

Base unit costs for replacement of distribution system pipe includes pipeline material and installation, repaving and system appurtenances that, collectively, constitute principal elements of the water distribution system facilities, are provided in **Table 9-1.**

Table 9-1. Pipeline Unit Costs

Diameter (Inches)	Cost per Inch Diameter ($/Inch/Linear Foot)	Construction Unit Cost ($/Linear Foot)	20% Contingency ($/Linear Foot)	Sub Total	30% Engineering, Legal and Admin ($/Linear Foot)	Total Unit Cost ($/Linear Foot)
8	20	160	32	192	58	250
10	18	180	36	216	65	280
12	16	192	38	230	69	300
16	16	256	51	307	92	400
18	15	270	54	324	97	420
20	15	300	60	360	108	470
24	15	360	72	432	130	560
30	15	450	90	540	162	700
36	15	540	108	648	194	840
42	15	630	126	756	227	980
48	15	720	144	864	259	1,120

9.1.2 Reservoir Replacement Unit Costs

Unit costs for new or replacement reservoirs are included in **Table 9-2**. Reservoir replacement is not recommended until detailed condition assessment of the existing reservoir's interior is completed.

Table 9-2. Reservoir Unit Costs

Volume (Gallons)	Unit Cost ($/Gallons)
< 1,000,000	2.00
1,000,000 - 3,000,000	1.50
3,000,000 - 6,000,000	1.25
6,000,000 – 20,000,000	1.00

9.1.3 Pump Station and Pressure Regulating/Reducing Station Unit Costs

Booster pump station costs typically include site preparation, including earthwork, paving and site piping, booster pumps installation, electrical and SCADA, and housing. Planning level unit costs can be as high as $1 million per MGD of capacity.

Planning level unit cost for a new PRS is estimated to be $250,000 per station.

9.2 Opinions of Probable Cost for Recommended Improvement Projects

Recommended improvement projects include those related to improving system redundancy and hydraulic performance (discussed in detail in **Sections 7.3** and **8.2**) as well as those based on condition assessment and risk analysis (discussed in **Section 6.7**). Opinion of probable construction costs are provided for each proposed improvement project in the following sections.

9.2.1 Redundancy and Hydraulic Performance Improvement Projects

Opinions of probable cost for the five existing system and one ultimate system recommended improvement projects, discussed in detail in **Sections 7.3** and **8.2**, respectively, are provided in **Table 9-3.** These project locations are illustrated in **Figure 9-1.**

Table 9-3. Probable Cost Opinion for Recommended Capital Improvement Program Projects

Project Number	Type	Description	Unit (Linear Feet unless otherwise specified)	Size	Unit Cost ($/Unit)	Low Range CIP Cost ($)	High Range CIP Cost (S)
EX-1	PRS	Construct new 637 zone PRS along Civic Center Drive	1 PRS	1,000 gpm peak flow	250,000	250,000	250,000
	Pipeline	New 12-inch pipe in Postal way from E43 PRS to Civic Center Drive and southwest down Civic Center Drive to new 637 PRS	3,211	12-inch	300	963,300	963,300
	Pipeline	Parallel 8-inch pipe in Civic Center Drive from new 637 zone PRS to Phillips Street	241	8-inch	250	60,250	60,250
EX-2	Pipeline	Parallel 12-inch pipe in South Santa Fe Avenue from Monte Vista Drive to E43 PRS and continuing to Civic Center Drive	2,665	12-inch	300	799,500	799,500
	PRS	Upsize E43 PRS	1 PRS	1,200 gpm peak flow	250,000	250,000	250,000
EX-3	Pipeline	New 30-inch pipe from Pechstein Reservoir to PS 10	645	30-inch	700	451,500	451,500
	Pipeline	New 24-inch pipe parallel to existing 26-inch pipe from PS 10 to Sugarbush Drive parallel to Buena Creek Road	3,386	24-inch	560	1,896,160	1,896,160
	Pipeline	New 24-inch pipe in Buena Creek Road from Sugarbush Drive to Monte Vista Drive	3,126	24-inch	560	1,750,560	1,750,560
	Pipeline	New 24-inch pipe replacing existing 12- and 10-inch pipe in Monte Vista Drive from Buena Creek Road to La Rueda Drive	1,759	24-inch	560	985,040	985,040

Table 9-3. Probable Cost Opinion for Recommended Capital Improvement Program Projects

Project Number	Type	Description	Unit (Linear Feet unless otherwise specified)	Size	Unit Cost ($/Unit)	Low Range CIP Cost ($)	High Range CIP Cost (S)
EX-4	PRS	Construct new PRS connecting 976/984 zone and 900 zone between San Clemente Way and Huntalas Lane	1 PRS	600 gpm	250,000	250,000	250,000
	Pipeline	New 8-inch pipe connecting 976/984 zone and 900 zone via new 900 PRS	1,006	8-inch	250	251,500	251,500
EX-5	PS	New PS at E Reservoir	1 PS	2000 to 7,000 gpm (3 to 10 MGD)	1 million/ MGD	3,000,000	10,000,000
	E Reservoir Replacement	Replace Existing E Reservoir, at same location	1 Reservoir	2 to 4 MG	1.50 to 1.25 per MG	3,000,000	5,000,000
	Pipeline	New pipe connecting E Reservoir PS to 976/984 zone	1,000	16 to 24-inch	400 560	400,000	560,000
ULT-1	Pipeline and Valve	Installation of 10-inch -diameter interconnection between 8-inch and 12-inch parallel pipes in Olive Avenue at the intersection of Grapevine Road	40 1 Valve	10 inch	280 5,000 per valve	11,200 5,000	11,200 5,000
ULT-2	New Pechstein II Reservoir	Construct new Pechstein II Reservoir adjacent to Pechstein Reservoir on District owned land	1 Reservoir	4 to 20 MG	1.25 to 1.00 per MG	5,000,000	20,000,000
Total Cost of Recommended Improvement Projects (Rounded to nearest $1,000)						19,324,000	43,484,000

EX - Existing System Improvement; ULT – Ultimate System Improvement; gpm - gallons per minute; CIP – capital improvement program; MG - million gallons; MGD - million gallons per day; PRS - pressure regulating/reducing station; PS – pump station

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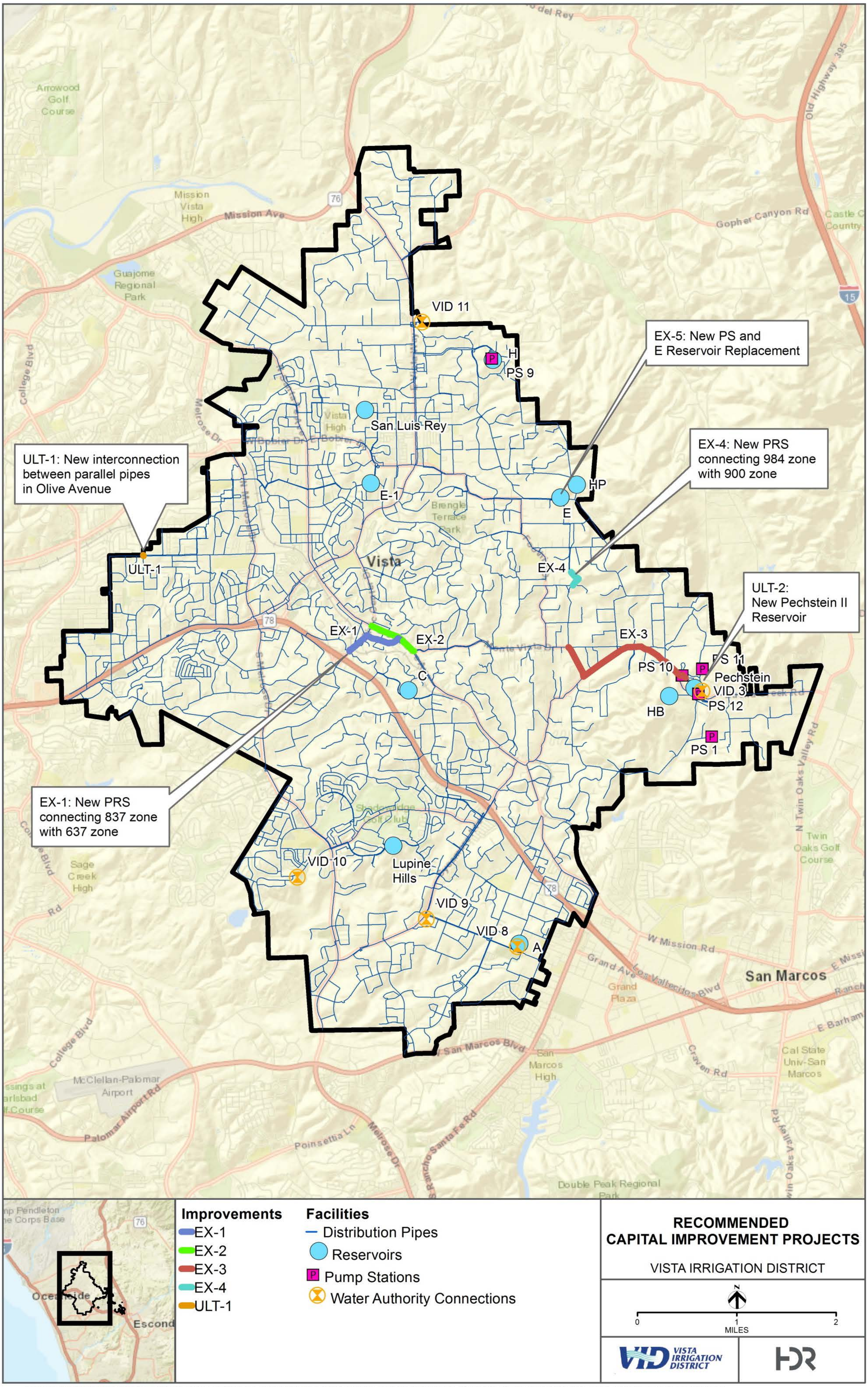
Figure 9-1. Recommended Capital Improvement Projects



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9.2.2 Reservoir Improvement Projects

In the Reservoir Condition Assessment TM (**Appendix B**) a probable opinion of cost for reservoir condition improvements for the ten reservoirs that were inspected was developed. The recommended improvements are based on external inspections only.

Rehabilitation work outlined in the cost opinion includes the civil/site, corrosion, and structural recommendations outlined in the Reservoir Condition Assessment TM. Where further investigation was recommended for seismic analysis for all ten reservoirs and an internal inspection for 7 of the 10 reservoirs, these costs are noted.

Appendix B of the TM includes reservoir roof replacement options and costs for A, Pechstein, and Deodar Reservoirs and reservoir replacement costs for A, Pechstein, H, and Deodar Reservoirs. Reservoir replacement and roof replacement is not recommended until further detailed condition assessment of the reservoirs interior is completed.

Table 9-4 displays the summary of the total probable cost opinions for near term and mid-term phases. The priority for improvements at each reservoir is based on rank, as discussed in detail in **Section 6.7.2**. The Deodor, Pechstein, and A Reservoirs are in the most need of near term repairs.

9.2.3 Potential Long Term Water Supply Reliability Projects

Although a number of long term water supply alternatives that would provide additional reliability to the District were presented in **Chapter 4**, the proposed CIP is based on the assumption that the Flume will remain in service.

To further assess, compare, and prioritize long term water supply projects, the District may want to proceed with a Water Supply Planning Study. This study would help to assess the best options for overcoming the 4 MG storage deficit under ultimate demand conditions, which could lead to an economy of scale decision to build a larger tank at Pechstein II. In addition, the study would provide guidance on moving forward with an extension of the Buena Creek Pipeline from Santa Fe, as discussed in **Section 8.2.3**. This Water Supply Planning Study would define appropriate levels of service goals and evaluate and prioritize the supply alternatives to help the District achieve those goals and make prudent investment decisions.

Table 9-4. Probable Cost Opinions for Recommended Reservoir Improvements

Rank	Reservoir Name	Near-Term (0-2 years) ($)	Mid-Term (2-6 years) ($)	Total Direct Costs (Near-Term and Mid-Term) ($)	Total Indirect Costs (Fees and Contingency) ($)	Total Probable Cost Options (Rounded) ($)	Further Investigation Assessments Near-Term (0-2 years) ($)	Further Investigation Assessments Mid-Term (2-6 years) ($)
1	Deodar Reservoir	42,000	9,964	51,964	25,807	78,000	57,000	-
2	Pechstein Reservoir	45,850	1,222	47,072	23,446	71,000	81,000	-
3	A Reservoir	26,508	7,749	34,257	17,033	52,000	31,000	-
4	HB Reservoir	28,200	28,892	57,092	28,353	86,000	61,000	-
5	Lupine Hills Reservoir	27,392	3,831	31,222	15,540	47,000	61,000	-
6	C Reservoir	12,850	11,015	23,865	11,892	36,000	-	10,000
7	E1 Reservoir	10,450	11,915	22,365	11,052	34,000	-	10,000
8	MD Reservoir	4,300	10,551	14,851	7,348	23,000	-	16,000
9	H Reservoir	9,000	27,468	36,468	18,128	55,000	-	61,000
10	San Luis Rey Reservoir	5,150	1,803	6,953	3,404	11,000	-	61,000
					Total	**493,000**	**291,000**	**158,000**

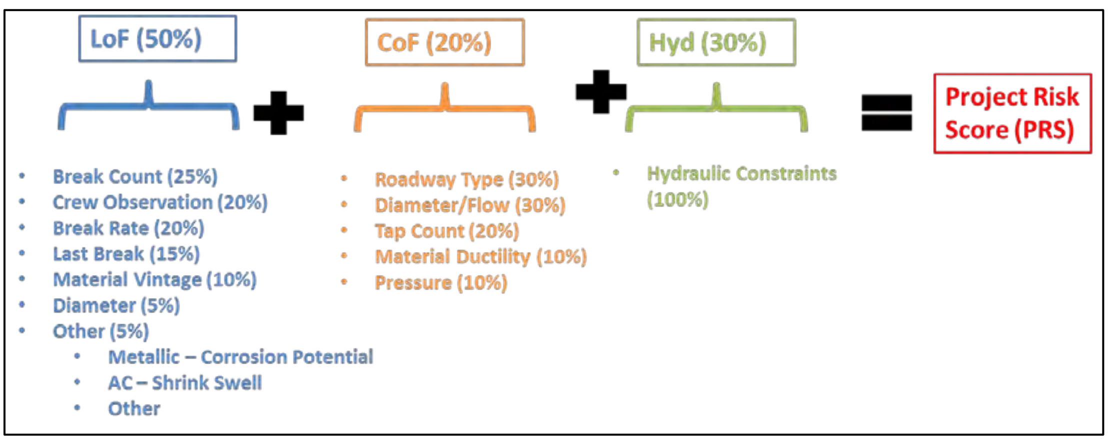
9.3 Pipeline Repair and Replacement Investment Scenarios

The District water main infrastructure replacement cost is roughly $600 million dollars. As the system continues to age and deteriorate, one of the District's primary goals is to cost effectively sustain desired service levels. To accomplish this, the District has initiated this effort to continuously improve the way distribution main assets are managed. The Pipeline Condition Assessment TM (**Appendix A**) applied a prudent, transparent, and reproducible method to District data to accomplish the three following primary objectives of this effort:

1. Establish prudent, transparent, and defensible investment levels that will enable the District to sustain desired levels of service as the system continues to age and deteriorate
2. Focus those investments to ensure ratepayers realize the greatest return on their investment
3. Optimize existing practices

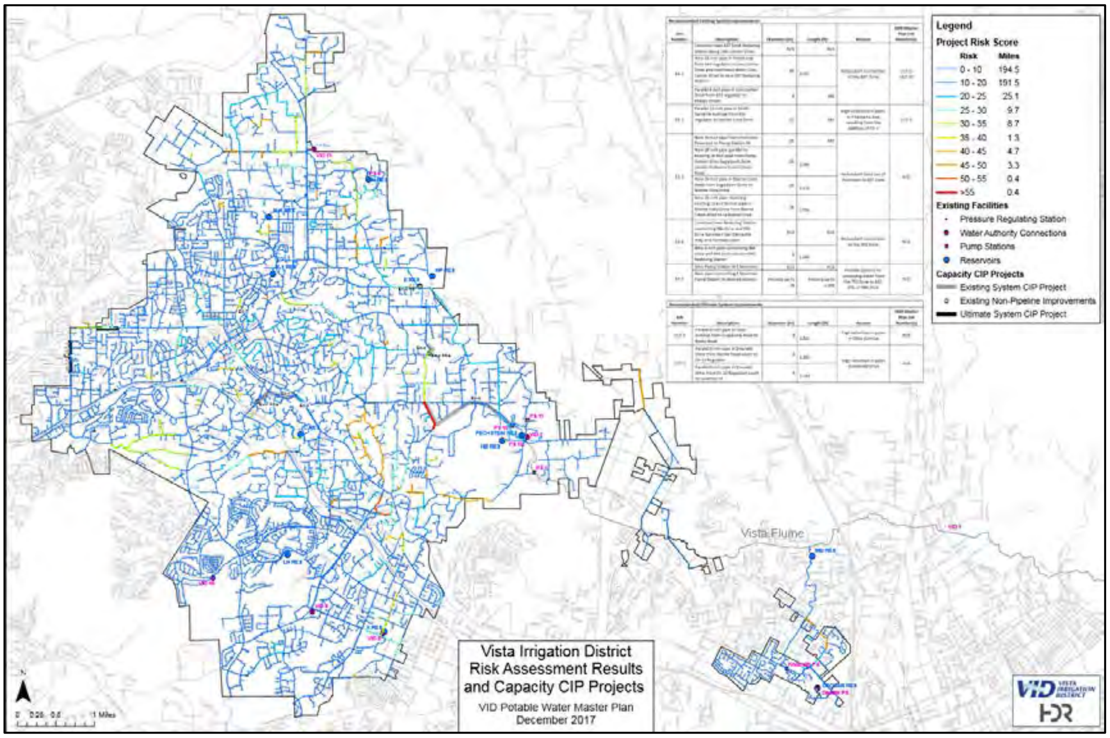
To accomplish Objective #2, a Project Risk Score (PRS) was developed that quantifies relative risk on a scale of zero (lowest risk) to 100 (highest risk). This methodology considers the consequence of failure (CoF), the likelihood of failure (LoF), and hydraulic limitations, as shown in **Figure 9-2**.

Figure 9-2. Risk Calculation Method



This methodology was applied to the District's distribution mains. The resulting risk map is provided in **Figure 9-3**.

Figure 9-3. Project Risk Map



To better understand how various investment levels will impact future service levels (Objective #1), a break forecasting model was developed in **Section 2** of the **Appendix A**. This model applies prudent, transparent, and reproducible methods to District data to forecast how many breaks will occur in each year over the planning horizon (through 2040). Three investment scenarios were modeled ordered from lowest to highest investment level:

- Scenario 1 – Sustain Existing Investment Levels
- Scenario 2 – Sustain Existing Service Levels
- Scenario 3 – Double Existing Investment Levels

The District's current program is focused on replacing sub-standard Nipponite pipe because it fails more often and more catastrophically than other materials, and approximately 9 miles of it remains in the system. Based on review of the PRS model, the majority of the system has a PRS score of 20 or less, and only a very small percentage scores higher than 40 (approximately 9 miles). The District should consider inclusion of these high risk pipelines in the near term replacement program based on engineering and operational judgment while striking the appropriate balance between affordability and sustaining desired service levels.

While decisions made in the near term have long-term consequences, course corrections are allowed and encouraged. As additional data are gathered and technologies advance, it is appropriate to apply new data collected, verify forecasting accuracy, and refine forecasting, as well as to revisit desired risk tolerances, service levels, and cost targets. As more data are collected, the accuracy of these long-term renewal projections will increase.

9.3.1 Format of Scenario Results

Figure 9-4 shows the impact of the current investment level on the number of years required to replace the entire system. The green bars summarize the investment level in terms of miles replaced per year (secondary y-axis). To provide context, the red circles summarize historic system replacement rate cycles. For example, in 2000, the District operated 415 miles of pipe and replaced 3 miles of pipe. At that rate, it would take approximately 140 years to replace the entire system. The blue circles forecast future system replacement rates based on the current size of the system, estimated growth, and future replacement levels. In this scenario, because replacement levels are flat yet the system is assumed to grow, the number of years to replace the system is forecast to increase from 221 years to 268 years by 2040.

Figure 9-4. Sustainability – Years to Replace Entire System

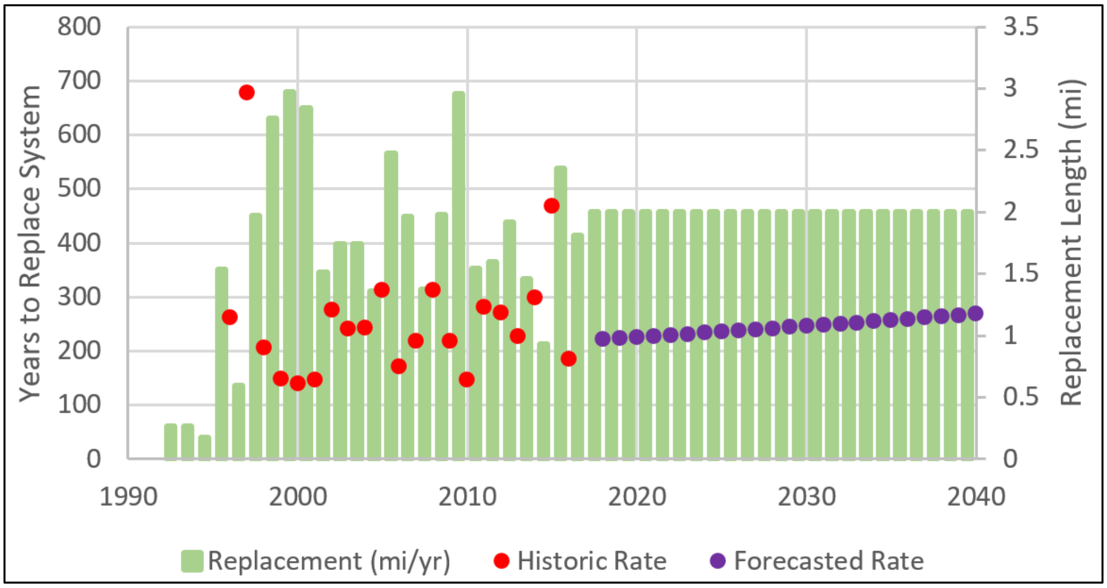


Figure 9-5 shows the impact of the current investment level on service levels. The green bars summarize the investment level in terms of miles replaced per year (secondary y-axis). The primary y-axis shows the annual break rate in terms of annual breaks per 100 miles of pipe in service. The red circles show the District's historic break rate and is associated with the primary y-axis. The blue circles show the District's forecasted break rate and is also associated with the primary y-axis. In this scenario where investment levels are held constant, service levels deteriorate from a break rate of approximately seven to a break rate of approximately eight by 2040.

Figure 9-5. Forecasted Service Levels

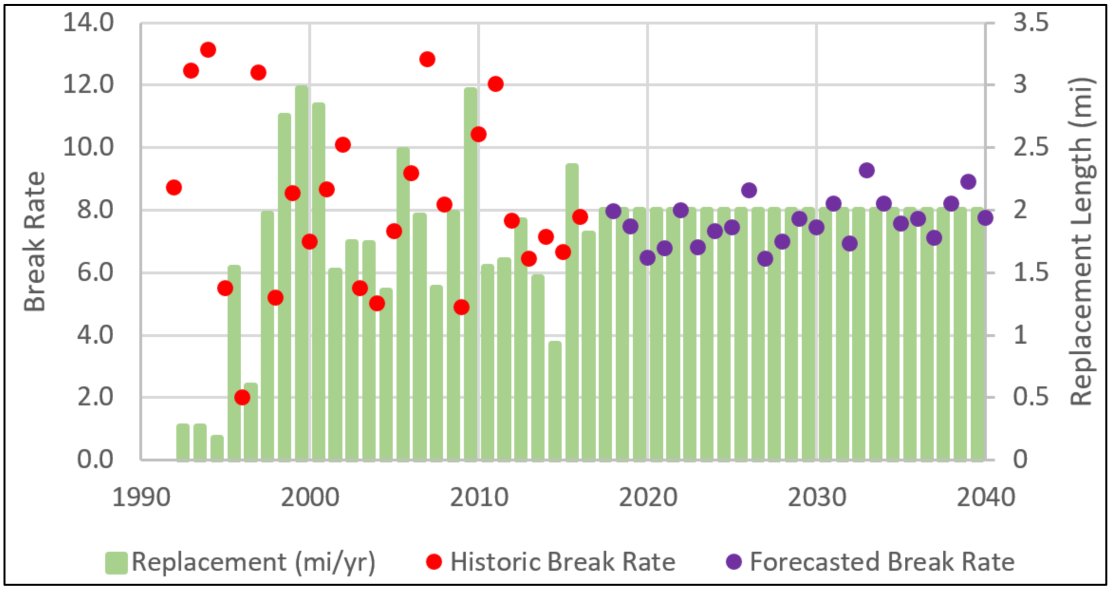


Figure 9-6 shows the impact of the current investment level on the number of breaks that occur per year in the system. This figure is intended to communicate the implications to District staffing levels to respond to the breaks. The green bars summarize the investment level in terms of miles replaced per year (secondary y-axis). The primary y-axis shows the count of breaks. The red circles show the District's historic break count and is associated with the primary y-axis. The blue circles show the District's forecasted break count and is also associated with the primary y-axis. In this scenario where investment levels are held constant, the number of breaks per year is forecasted to increase from approximately 35 (which the District is currently staffed to respond to) to approximately 45 by 2040 (an increase of roughly 30 percent under this investment scenario).

Figure 9-6. Staffing Levels – Breaks per Year

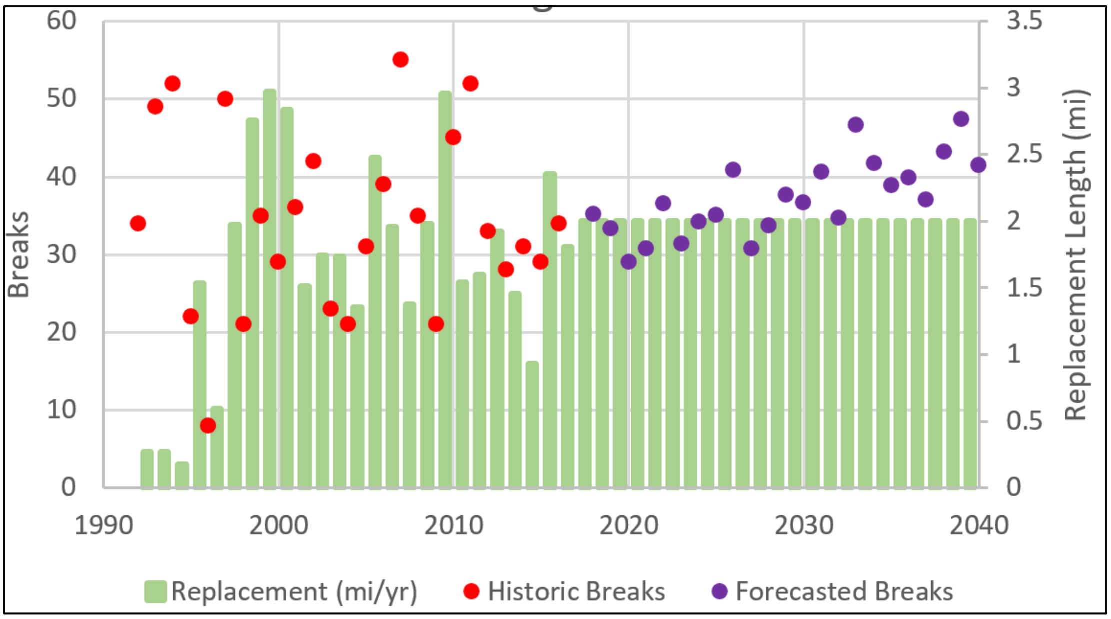
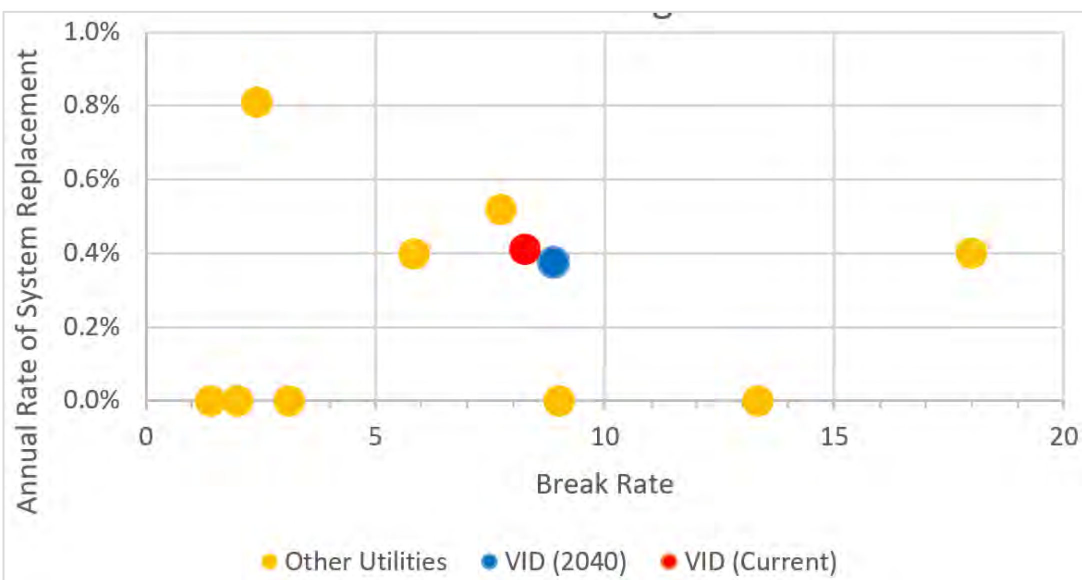


Figure 9-7 benchmarks the District's performance versus other utilities in Southern California. Utilities are benchmarked based on service levels (i.e., break rate) and annual rate of system replacement. In general, utilities with a higher break rate should also have a higher annual replacement rate, although each community must find the appropriate balance between service levels and affordability for their customers. The District's current and recommended investment levels are in line with how other Southern California utilities balance service levels and affordability for their customers. The red circle identifies the District's current state. The blue circle identifies where the District is forecasted to operate in 2040 based on the current investment level. Gold circles represent the recent performance of the following utilities:

- City of Carlsbad
- Rainbow Municipal Water District
- City of San Diego
- Helix Water District
- Padre Dam Municipal Water District
- City of San Juan Capistrano
- City of Buena Park
- Sweetwater Authority/South Bay Irrigation District
- Los Angeles Department of Water and Power

Several of the utilities that are not replacing pipe are currently evaluating implementation of a pipe replacement program. For example, the utility currently operating at a break rate of 13 and a replacement rate of zero is in the process of increasing investment levels to approximately 0.8 percent of the system per year.

Figure 9-7. Replacement Program Benchmarking

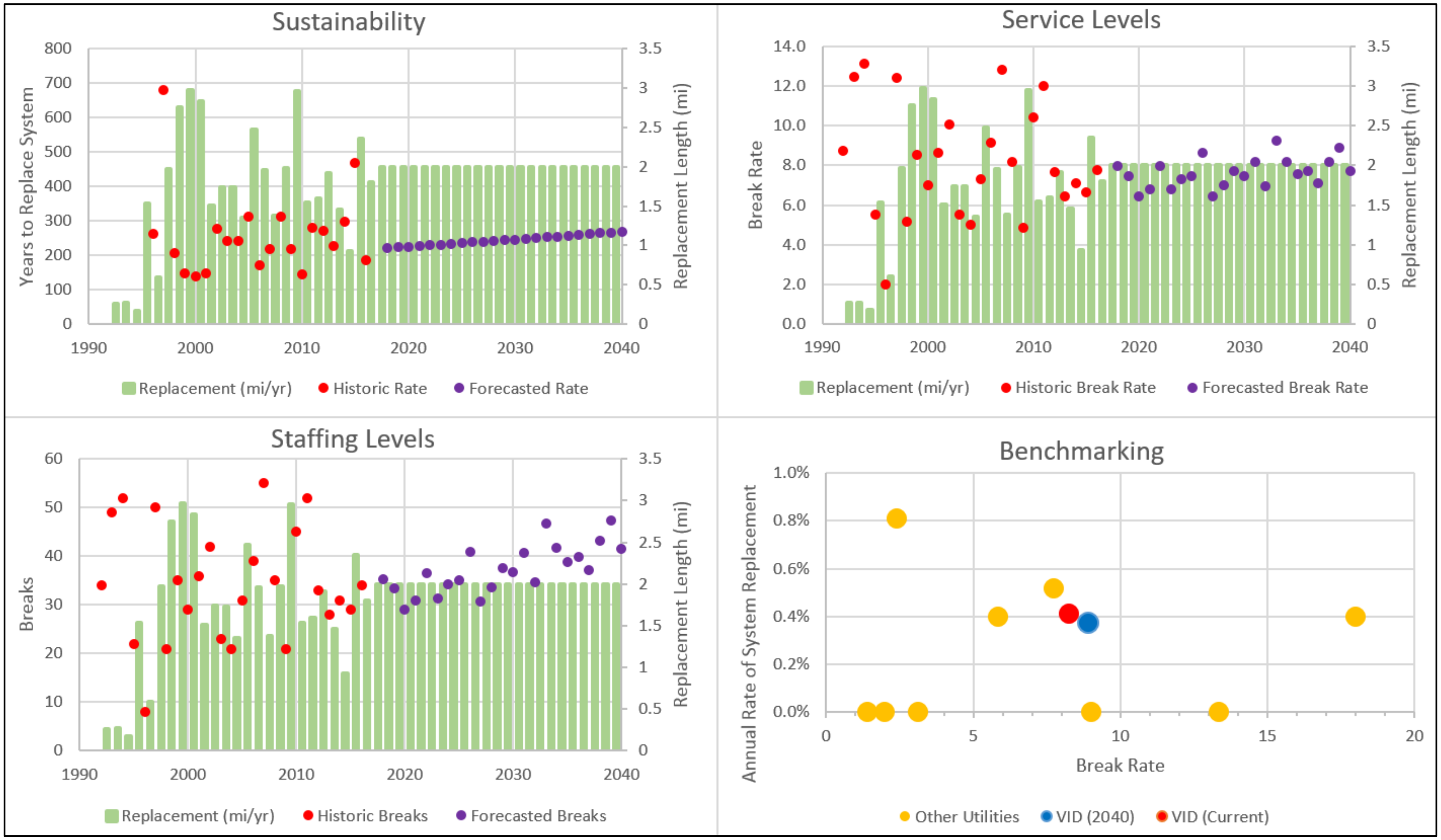


Sections 9.3.2 through 9.3.4 summarize the three investment scenarios that were modeled. It is anticipated that the District Board will evaluate and select one of these three investment approaches on an annual or long term basis.

9.3.2 Investment Scenario 1 – Sustain Existing Investment Levels

In this scenario, existing investment levels are held constant through 2040.

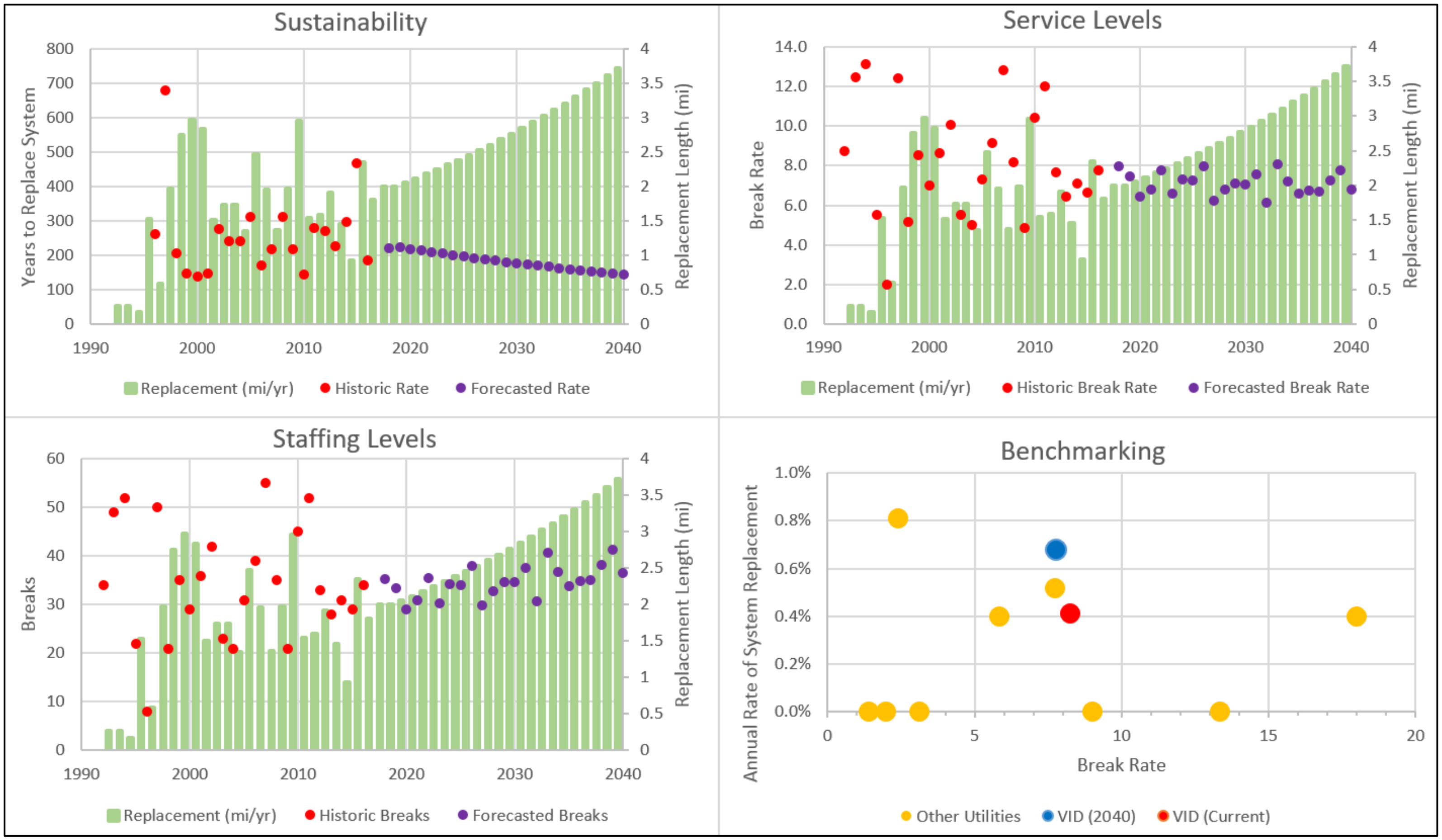
Assumptions		
Current renewal level (mi/yr) = 2	Minimum Replacement Length (ft) = 2000	Annual System Growth1 = 0.88%
Funding Increase Begins = 2020	Annual % Increase = 0%	New Pipe Break Rate = 2



9.3.3 Investment Scenario 2 – Sustain Existing Service Levels

In this scenario, existing service levels are held constant through 2040.

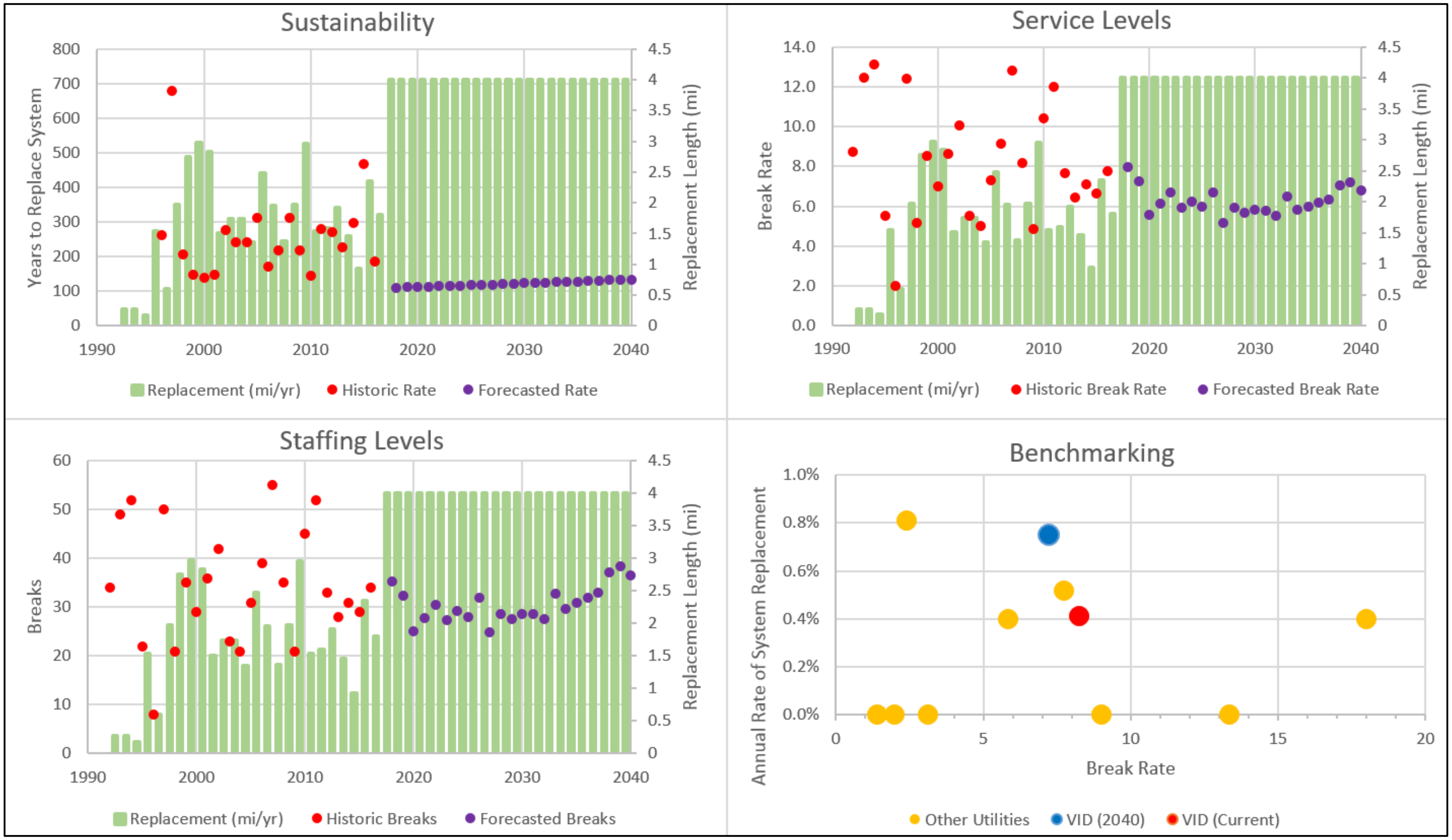
Assumptions		
Current renewal level (mi/yr) = 2	Minimum Replacement Length (ft) = 2000	Annual System Growth1 = 0.88%
Funding Increase Begins = 2020	Annual % Increase = 3%	New Pipe Break Rate = 2



9.3.4 Investment Scenario 3 – Improve Existing Service Levels

In this scenario, historic investment levels are doubled. Service levels are expected to improve.

Assumptions		
Current renewal level (mi/yr) = 4	Minimum Replacement Length (ft) = 2000	Annual System Growth1 = 0.88%
Funding Increase Begins = 2020	Annual % Increase = 0%	New Pipe Break Rate = 2



Appendix A. Water Pipeline Condition Assessment Technical Memorandum

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Vista Irrigation District

Water Pipeline Condition Assessment

Technical Memorandum

Vista, California

January 10, 2018

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Contents

Tables

Figures

Appendices

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Executive Summary

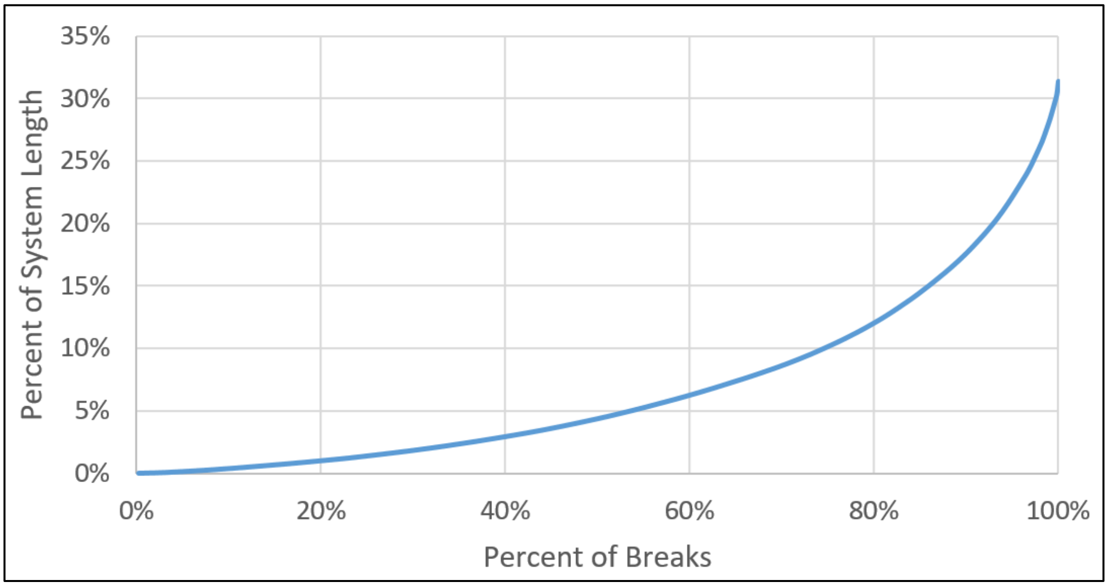
Vista Irrigation District (District) owns 429 miles of water main infrastructure and manages an additional 10 miles of privately owned water main infrastructure. As the system continues to age and deteriorate, one of the District's primary goals is to cost effectively sustain desired service levels. To accomplish this, the District has initiated this effort to continuously improve the way distribution main assets are managed. The three primary objectives of this project are to:

1. Establish prudent, transparent, and defensible investment levels that will enable the District to sustain desired levels of service as the system continues to age and deteriorate.
2. Focus those investments to ensure ratepayers realize the greatest return on their investment.
3. Optimize existing practices.

For distribution mains, the District has break data going back to 1992. The District has documented 2,230 breaks from 1992 through January of 2017, of which 839 were classified as occurring on a mainline (as opposed to a service, valve, or other appurtenance) and were used in this analysis. The data, summarized in Section 1, is of sufficient quantity and quality to build risk and investment models that meet the three objectives of this project.

Industry experience tells us that pipeline performance and useful life can vary significantly from one construction project to the next. Construction project data provided insight regarding the relative quality of the material used, transport and handling procedures, installation quality, backfill quality, and construction management quality. Analysis of the District's break data validates that District pipeline performance varies significantly by project. Figure ES-1 summarizes project number performance by cumulative breaks and lengths. As shown, a small percentage of system piping is responsible for most of the breaks (e.g., 80 percent of all breaks have occurred on projects that represent only 12 percent of the entire system length). The relationship between project number and performance was found to be significant, thus construction project numbers were used as the basis for sizing and prioritizing renewal investments.

Figure ES-1. Small Percentage of Pipe is Responsible for Most Breaks



Based on the data, historic break count was also found to be a good indicator of performance, as the percent of projects that broke again increased as the break count increased, and the duration between subsequent breaks became shorter.

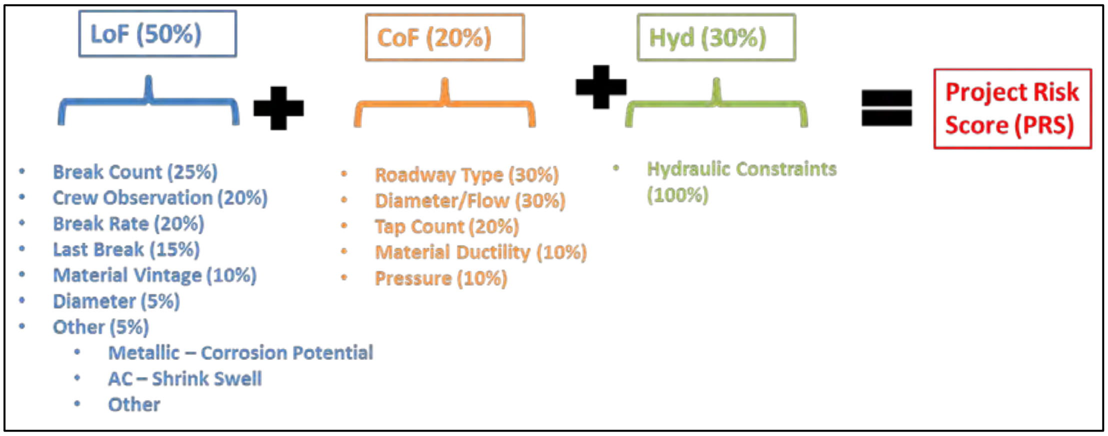
To better understand how various investment levels will impact future service levels, a break forecasting model was developed in Section 2. This model applies prudent, transparent, and reproducible methods to District data to forecast how many breaks will occur in each year over the planning horizon (through 2040). Three investment scenarios were modeled:

- Scenario 1 – Sustain Existing Investment Levels
- Scenario 2 – Sustain Existing Service Levels
- Scenario 3 – Double Existing Investment Levels

It is anticipated that these scenarios, in conjunction with engineering and operational judgment, will enable the District to make informed renewal decisions, with confidence that desired levels of service will be maintained. Selection of the appropriate investment level should be made by District management and should strike the appropriate balance between desired long term service level goals and the associated cost to achieve that service level.

The next objective was to focus those investments to ensure ratepayers realize the greatest return on their investment. Section 3 defines a consistent, transparent, efficient, prudent, and defensible approach to selecting an appropriate investment level through the identification and prioritization of water pipeline replacement projects. To accomplish this, a Project Risk Score (PRS) was developed that quantifies relative risk on a scale of zero (lowest risk) to one-hundred (highest risk). This methodology considers the consequence of failure (CoF), the likelihood of failure (LoF), and hydraulic limitations, as shown in Figure ES-2.

Figure ES-2. Risk Calculation Method



This methodology was applied to the District's distribution mains. The resulting risk map is provided in Figure 3-3 and Appendix C.

Historically, the District has typically used the open-trench replacement method. Based on regulatory challenges, useful life extension uncertainty, additional research needed, and limited economies of scale; it is recommended that the District continue to use open-trench replacement as the primary renewal method. However, the viability of alternative renewal solutions should be evaluated on a project specific basis, particularly where the integrity of the host pipe can be cost effectively determined and site-specific factors lend themselves to alternative renewal solutions. Section 4 provides more detail regarding the evaluation of main renewal strategy alternatives for the District.

Recommendations for continuous improvement in managing aging distribution infrastructure are included in Section 5.

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1 Summary of Data

Prudent and data driven investment decisions rely on high quality data. This section describes the District's break data, system data, and data limitations.

1.1 Pipe Data

The District's infrastructure database of record is their Geographic Information System (GIS). The District provided readily available GIS files in October of 2016. This dataset was limited to asset with a subtype of Distribution Main or Transmission Main and an owner of District, Private, or null. Active pipes were assumed to be in a status of Existing, Constructing, or Unknown. Inactive pipes were assumed to be in a status of Abandoned, Inactive, or Removed. A summary of system mileage by installation era and status is included in Table 1-1.

Table 1-1. Summary of Pipe Data

Installation Era	District		Private [1]		Total
	Inactive (mi)	Active (mi)	Inactive (mi)	Active (mi)	
Unknown	35	8	-	-	43
Pre 1950	5	5	-	-	10
1950s	18	73	-	-	91
1960s	12	60	-	-	72
1970s	3	67	-	-	70
1980s	2	116	0	3	120
1990s	0	41	0	6	48
2000s	0	41	-	0	41
2010s	-	19	-	1	19
Grand Total	**74**	**429**	**0**	**10**	**513**

[1] Indicates that pipe is privately owned but is maintained by the District at the owner's expense based on water system maintenance agreements.

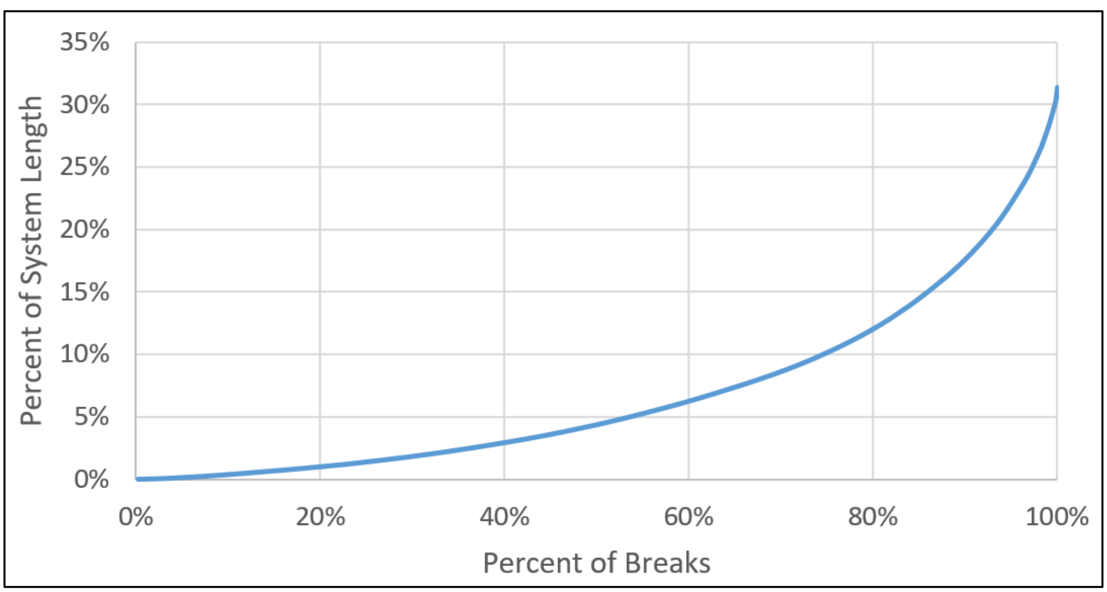
1.2 Pipe Grouping

District pipe assets are divided into small lengths at diameter changes, material changes, install date changes, valves, tees/crosses, bends, and other attributes. The median and average pipe asset length in GIS is 26 feet and 92 feet respectively. This is useful for some purposes (hydraulic modeling, cohort analysis, attribute data management, etc.); however, it is not a useful basis for renewal decision making because it is not cost effective to renew infrastructure in such small units. Therefore, it is necessary to aggregate these short pipes into more meaningful groupings so asset specific information can be determined such as break count, pipe performance, renewal budgeting, probability of failure, consequence of failure, and project identification and prioritization.

The District manages two fields in GIS (“INSTALLNUM” and “WORKORDERTYPE”) that identify the unique project under which the pipe was constructed. Construction project data can provide insight regarding the relative quality of the material used, transport and handling procedures, installation quality, backfill quality, and construction management quality. Industry experience tells us that pipeline performance and useful life can vary significantly from one construction project to the next. Analysis of the District’s break data validates that District pipeline performance varies significantly by project.
Figure 1-1 summarizes project number performance by cumulative breaks and length. For example:

- Twenty percent of all breaks have occurred on projects that represent 1 percent of the entire system by length
- Forty percent of all breaks have occurred on projects that represent 3 percent of the entire system by length
- Sixty percent of all breaks have occurred on projects that represent 6 percent of the entire system by length
- Eighty percent of all breaks have occurred on projects that represent 12 percent of the entire system by length
- Projects that represent 69 percent of the entire system by length have never had a recorded break

Figure 1-1. Small Percentage of Pipe is Responsible for Most Breaks



Because the relationship between project number and performance is strong and this pipe grouping better supports renewal decision making (the median and average pipe length by INSTALLNUM is 540 feet and 1,105 feet respectively), INSTALLNUM was used as the basis for sizing and prioritizing renewal investments.

Note some pipe did not have an INSTALLNUM. For those pipes with more than two breaks (~2.25 mi), a manual review of install year, material, diameter, and spatial location was performed to approximate the extent of a project. Five projects were generated (HDR1, HDR2, HDR3, HDR4, and HDR5). A list of pipes associated with these five project numbers is included in Appendix B.

1.3 Break Data

The District has break data dating back to 1992. The data collected with each break has remained relatively consistent since 1997 when the District created a paper form to document breaks. The form is still populated by break response crews. The breaks are summarized in an Excel spreadsheet managed by operations supervisors. The data is then entered into the District's CMMS by office staff. Then the paper form is provided to the GIS group to add each break to a GIS layer.

An initial assessment of the District's break data showed significant discrepancies between the various sources of information. To address these issues, District staff evaluated the various data sources, cleansed the historic data, generated a break database of records in GIS, and associated those breaks to the asset that broke.

HDR performed a brief review of the break association and in general, the association looked valid. However, issues were discovered on 46 main breaks that were manually updated to the appropriate pipe. A list of these updates is included in Appendix A.

As a result of this effort, the District now has data of sufficient quality and quantity to develop a prudent, defensible, and data driven asset renewal program. On May 15, 2017, the District provided HDR with 2,230 break records through January of 2017. Eight hundred and thirty-nine of those breaks were classified as occurring on a mainline (as opposed to a service, valve, or other appurtenance) and were used in this analysis.

1.4 Limitations

A lack of break data prior to 1992 limits the ability to measure system performance in age ranges and break counts where pipes were active prior to 1992. For example, a pipe that was installed in 1960 and has six recorded breaks may have experienced one or many breaks between installation and the date when recorded break data became available. Therefore, the total number of breaks each pipe has experienced is unknown; the only known data is that this pipe has experienced six breaks since 1992. This limitation can significantly impact results if there are only several years of break data available. However, the limitation for the District is significantly mitigated because over 26 years of break data is available for analysis. Still, it is important to recognize this data limitation exists and that it will continue to be mitigated as more and more break data is collected and a larger proportion of the complete break history for every pipe becomes available.

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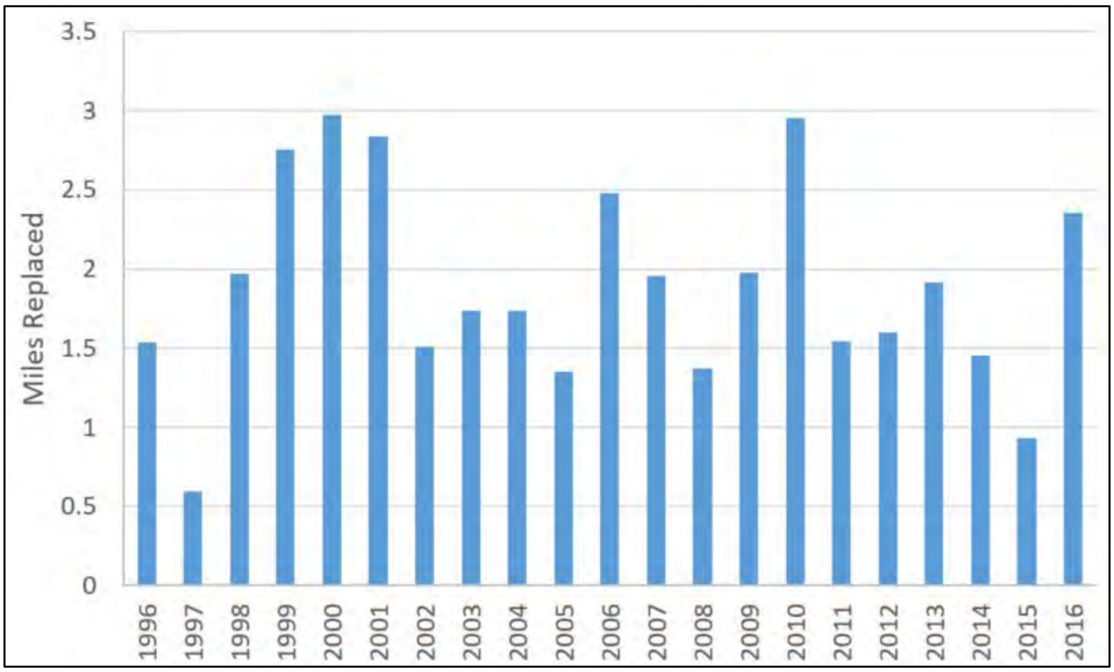
2 Aging Infrastructure Investment Levels

2.1 Historic Replacement Program

In 1995, the Board of Directors initiated an ongoing Main Replacement Program with the goal of replacing aging pipelines before they reached the end of their useful life and became a maintenance liability. Since that time, the District has replaced approximately 2 miles of pipe per year. Figure 2-1 summarizes the miles replaced per year based on the data in GIS.

Figure 2-1. Historic Replacement Program



2.2 Project Performance

In this analysis, the relationship between break count and performance for a particular project[1] is measured by calculating the proportion of projects that break again and the average duration between subsequent breaks. If historic break count is a good indicator of performance, one would expect that the percent of projects that break again would increase as the break count increases, and the duration between subsequent breaks would become shorter. Figure 2-2 summarizes the results of this analysis. The blue points indicate the proportion of the projects that broke again (associated to the primary y-axis). For example:

- Projects that have experienced one break and have broken a second time: 39 percent.

[1] For the purposes of this analysis, INSTALLNUM was used to identify projects.

- Projects that have experienced two breaks and have broken a third time: 57 percent.
- Projects that have experienced 10 breaks and have also experienced their eleventh break: 83 percent.

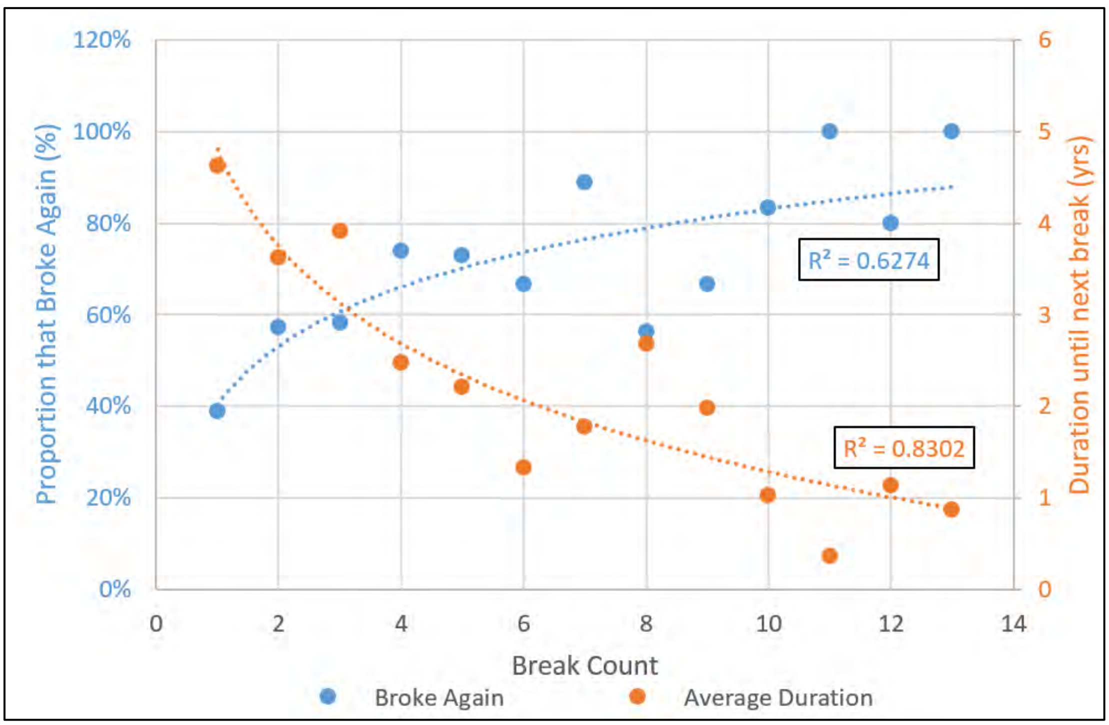
This data indicates that as a project experiences more breaks, it is more likely to experience another break.

The orange points (associated to the secondary y-axis) in Figure 2-2 summarize the average duration between subsequent breaks. For example, for projects that have at least:

- **Two breaks**: the average duration between the first and second break is **4.6 years.**
- **Three breaks:** the average duration between the second and third break is **3.6 years.**
- **Ten breaks**: the average duration between the tenth and eleventh break is **1.0 year.**

The data trend is best described by the orange line, which has a strong coefficient of determination[2] (R^2 = 0.83). This data indicates that as a project experiences more breaks, the duration until the next break becomes shorter. Both trends support the theory that historic break count is a good indicator of future performance of a project.

Figure 2-2. Project Performance Curve



[2] R^2 is a statistic that will give some information about the goodness of fit of a model. In regression, the R^2 coefficient of determination is a statistical measure of how well the regression line approximates the real data points. An R^2 of 1 indicates that the regression line perfectly fits the data.

2.3 Break Forecasting Curves

The project performance curves documented in the previous section is a simple method to validate whether historic break count can be used to estimate future breaks. However, it cannot be used to accurately forecast future breaks due to several shortcomings:

- Analysis doesn't account for the performance since the last break.
  - For example it is fairly common for a project to break several times and then go 10+ years without a break.
- Analysis doesn't account for the length of each pipe, which is highly variable.
- Analysis doesn't account for the performance between installation or the start of break data collection and the first recorded break.

To address these limitations, the basis for estimating performance should be changed from duration (measured in years) to break rate (measured in annual breaks per 100 miles of pipe). This analysis accounts for when each project was installed, when break data began to be collected and associated to projects, when breaks occurred, when each project transferred between states (e.g., from two historic breaks to three historic breaks), if and when a project was abandoned, and the length of the project. So, for example, if one 0.1-mile-long project was installed on January 4, 2000, and had breaks on February 7, 2010, August 1, 2012, and April 28, 2013, and was then abandoned on December 17, 2014, the break rate for each break count is summarized in Table 2-1. When a project transfers states, the "break count" field describes the break count at the beginning of the state. So, for example, the break rate between the first and second break would be shown in the row with the break count equal to one.

Table 2-1. Example Break Rate by Break Count Calculation

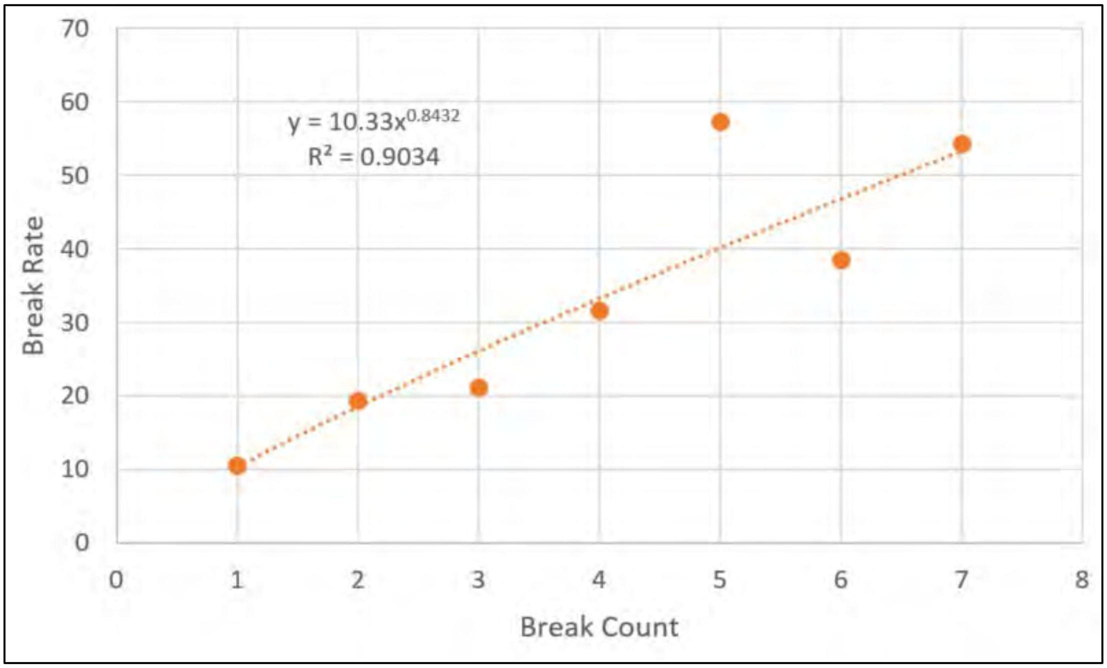
Break Count	Miles	Start	End	Duration (Years)	Break Rate
0	0.1	January 4, 2000	February 7, 2010	10.1	99
1	0.1	February 7, 2010	August 1, 2012	2.5	403
2	0.1	August 1, 2012	April 28, 2013	0.7	1353
3	0.1	April 28, 2013	December 17, 2014	1.6	0

This methodology was applied to the 1,867 unique active and abandoned projects in the District's system. Figure 2-3 shows the break rate by historic break count. For example, for all projects that have had at least:

- One break, the average break rate until the second break was 10.6 annual breaks per 100 miles of pipe.
- Two breaks, the average break rate until the third break was 19.4 annual breaks per 100 miles of pipe.
- Three breaks, the average break rate until the fourth break was 21.2 annual breaks per 100 miles of pipe.

- Seven breaks, the average break rate until the fourth break was 54.4 annual breaks per 100 miles of pipe.

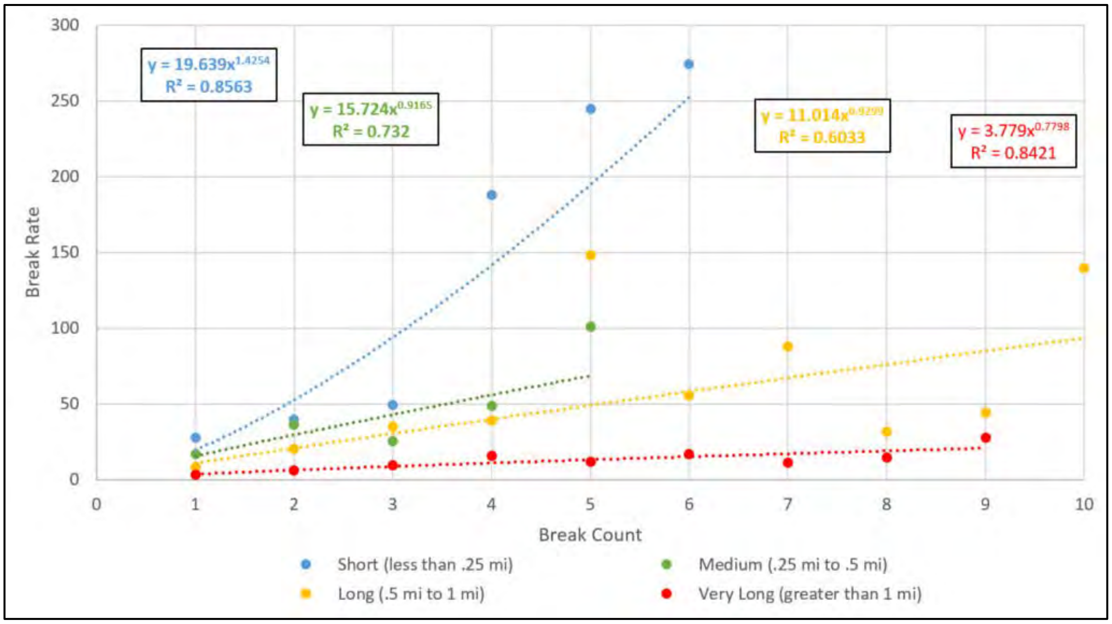
Figure 2-3. Preliminary Project Break Forecasting Curve



Further analysis was conducted to determine whether characteristics (e.g., material, installation era, pressure, etc.) significantly influence the duration until the next break. In addition to break count, the primary factor driving performance was pipe length. Other utilities such as Phoenix and Des Moines have had similar experiences. In theory, this is because if a short pipe (i.e., 200 feet) has three breaks, it is very likely that pipe is in poor condition and will break again soon. Conversely, if a very long pipe (i.e., 4,000 feet) has broken three times, those breaks may be more random in nature and not indicative that the entire pipe has deteriorated and is likely to break again in the near future.

Figure 2-4 summarizes the curves used to forecasting future breaks.

Figure 2-4. Final Project Break Forecasting Curves



2.4 Break Forecasting

This section describes the basis for forecasting future breaks. Future break forecasting will be used as the primary basis for determining the size of the replacement program. The break forecasting model is the sum of four components:

- Existing projects with historic breaks
- Existing projects without historic breaks
- Pipe not associated to a project
- Projects to be constructed in the future

In this section, each component will be described in more detail.

2.4.1 Existing Project with Historic Breaks (Component 1 of 4)

This section describes the approach for estimating future breaks on projects that have broken in the past. The length and count of breaks was summarized for each active project. Based on the length of each project, the appropriate equation from Figure 2-4 was applied to estimate when future breaks would occur on those projects. The equations to calculate the next break on each project with a historic break were:

For projects less than a quarter mile in length (i.e., “short”):

$$\text{Forecasted Break} = 19.639 * (\text{“Historic Break Count”} + n - 1)^{1.4254} + \text{“Last Break Date”}$$

For projects between a quarter mile and a half mile in length (i.e., “medium”):

$$\text{Forecasted Break} = 15.724 * (\text{“Historic Break Count”} + n - 1)^{0.9165} + \text{“Last Break Date”}$$

For projects between a half mile and a mile in length (i.e., “long”):

$$\text{Forecasted Break} = 11.014 * (\text{“Historic Break Count”} + n - 1)^{0.9299} + \text{“Last Break Date”}$$

For projects greater than a mile in length (i.e., “very long”):

$$\text{Forecasted Break} = 3.779 * (\text{“Historic Break Count”} + n - 1)^{0.7798} + \text{“Last Break Date”}$$

Where n is the forecasted break number and the first Forecasted Break Date must be greater than the Break Forecast Start Date

Note: while the model produces an exact forecasted break date, such precision is not realistic. The appropriate usage of such output is to summarize the results system-wide by year to understand breaks saved.

Break data was available through January 2017. Therefore, the break forecast start date was assumed to be February 1, 2017 for all projects. So, for example, project number 831 has the following characteristics:

- Status = Active
- Length = 0.67 miles (i.e., “long”)
- Historic Break Count = 3
- Last Break Date = 12/17/2016

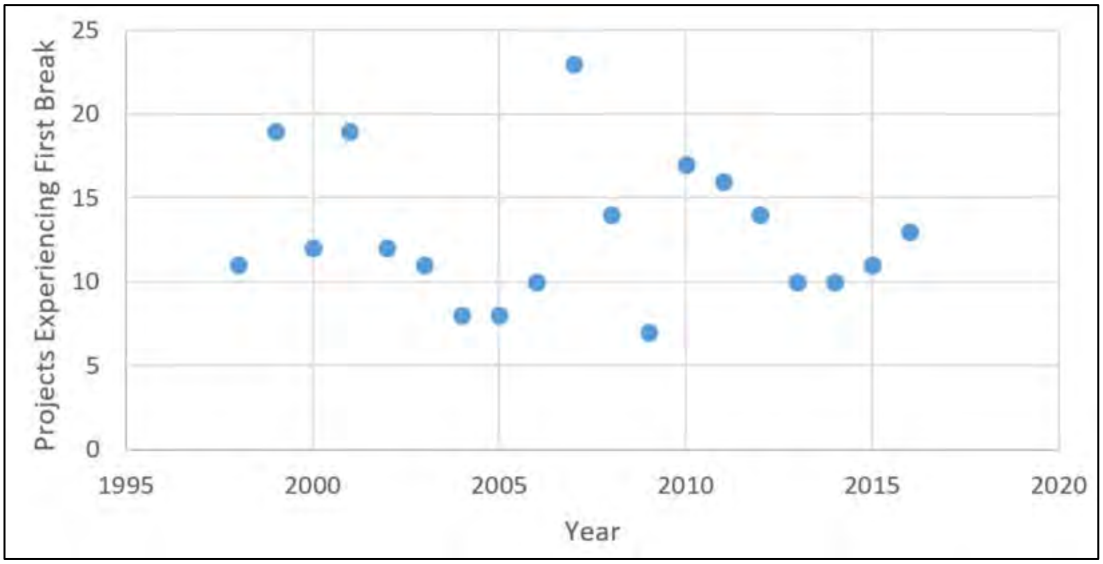
Therefore, the forecasted breaks are calculated as:

$$\text{First Forecasted Break} = 11.014 * (3 + 1 - 1)^{0.9299} + 12/17/2016 = 11/1/2021$$
$$\text{Second Forecasted Break} = 11.014 * (3 + 2 - 1)^{0.9299} + 11/1/2021 = 7/26/2025$$
$$\text{Third Forecasted Break} = 11.014 * (3 + 3 - 1)^{0.9299} + 7/26/2025 = 8/5/2028$$

2.4.2 Existing Projects without Historic Breaks (Component 2 of 4)

The second of four components in the break forecast model is to estimate the number of breaks that will occur on existing projects that have not had a recorded break.
Figure 2-5 summarizes the number of projects that experienced their first break over the past 18 years.

Figure 2-5. Projects Experiencing First Break

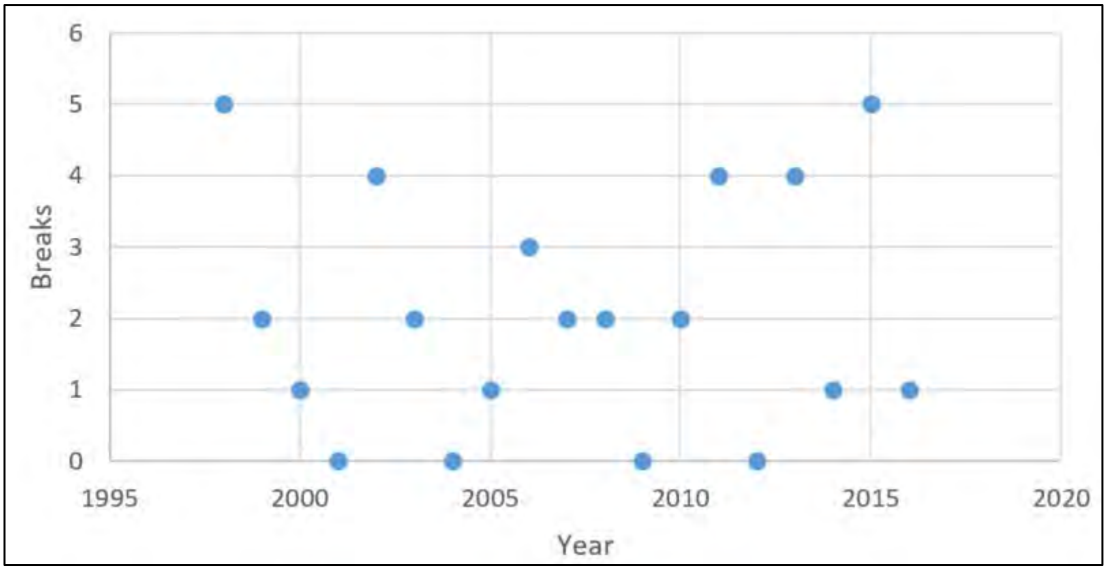


An average of 13 breaks per year occurs on projects that had never previously experienced a break. Therefore, it is assumed that 13 projects will break for the first time each year over the forecasted period. The equation in Figure 2-3 was used to estimate additional breaks that would occur on these projects over the forecasted period.

2.4.3 Pipe Not Associated to a Project (Component 3 of 4)

A small portion of the system is not assigned to a project. Figure 2-6 summarizes the number of breaks that have occurred on these pipes over the past 18 years.

Figure 2-6. Breaks on Pipes Not Assigned to a Project



An average of two breaks per year occurs on pipes not assigned to a project. Therefore, it is assumed that two breaks will occur per year on pipes not assigned to a project.

2.4.4 Projects to be Constructed in the Future (Component 4 of 4)

The final component in the break forecast model is to estimate the number of breaks that will occur on projects that will be constructed in the future. The assumed annual system growth rate of 0.88 percent is based on SANDAG Series 13 Growth Forecast for population growth in the VID service area between 2015 and 2040. While newly installed pipes generally break less often, breaks will occur on newly installed pipe over the forecast period. An annual break rate of two breaks per 100 miles was assumed.

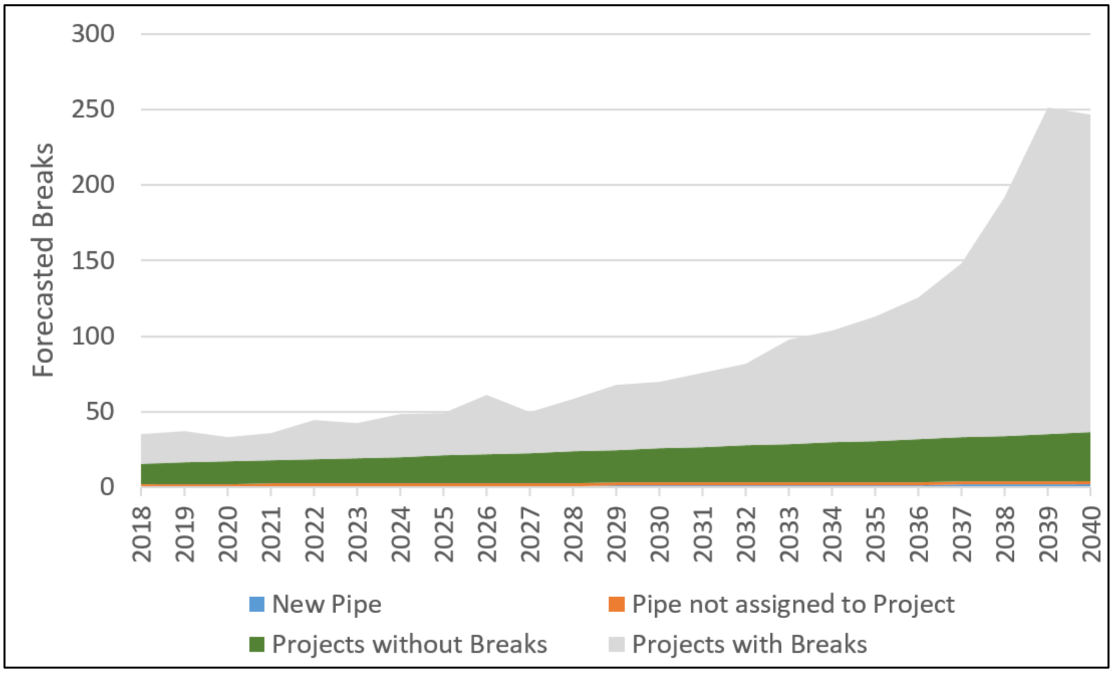
2.4.5 Summary of Forecasted Breaks

Figure 2-7 summarizes the annual count of breaks by summing the forecasted breaks in each of the four components:

- Existing projects with historic breaks
- Existing projects without historic breaks
- Pipe not associated to a project
- Projects to be constructed in the future

The vast majority of forecasted breaks come from the first two components while the last two components represent a relatively small portion of the forecasted breaks.

Figure 2-7. Summary of Forecasted Breaks by Component



Note: this figure does not account for breaks avoided through the pipe replacement program. This will be accounted for in the next section.

2.5 Investment Scenarios

The purpose of this section is to apply prudent, transparent, and reproducible methods to District data to estimate how various funding levels will impact future service levels. Three investment scenarios were modeled ordered from lowest to highest investment level:

- Scenario 1 – Sustain Existing Investment Levels
- Scenario 2 – Sustain Existing Service Levels
- Scenario 3 – Double Existing Investment Levels

It is anticipated that these forecasts, in conjunction with engineering and operational judgment, will enable the District to strike the appropriate balance between affordability and sustaining desired service levels.

While decisions made in the near term have long-term consequences, course corrections are allowed and encouraged. As additional data are gathered and technologies advance, it is appropriate to apply new data collected, verify forecasting accuracy, refine forecasting, as well as to revisit desired risk tolerances, service levels, and cost targets. As more data are collected, the accuracy of these long-term renewal projections will increase.

2.5.1 Assumptions

The investment forecasting model includes six primary assumptions that can be modified as shown in Table 2-2. Each assumption is described in more detail below. All scenarios assume a planning period through 2040.

Table 2-2. Adjustable Model Assumptions

Assumptions	
2	Current renewal level (mi/yr)
2020	Funding Increase Begins
2000	Minimum Replacement Length (ft)
0.0%	Annual Replacement Increase (%)
0.88%	Annual System Growth (%)
2	New Pipe Break Rate (annual break per 100 miles)

- **Current renewal level (mi/yr)** – The miles assumed to be replaced in 2018.
- **Funding Increase Begins** – Near term investment levels are constrained based on existing resources (e.g., budgets, staff, etc.). This assumption determines the year in which the percentage increases are first implemented.
- **Minimum Replacement Length (feet)** – This assumption determines, at a planning level, the typical minimum length of contiguous pipe that the District will mobilize a contractor to replace. In general, it is not cost effective to mobilize a construction resource to replace short lengths of pipe. Therefore, the District will typically expand the project boundaries beyond the target project to obtain a more contiguous pipe based on an assessment of the risk of adjacent pipes and project constructability considerations (e.g., traffic control, valve location, surface features, outage area, etc.).
- **Annual Replacement Increase (percent)** – This assumption determines how quickly investment levels increase or decrease from the current renewal level. A positive number represents an increase in investment levels while a negative number represents a decrease in investment levels.
- **Annual System Growth** – This assumption estimates the rate at which the overall system will grow over the forecast period. The assumed annual system growth rate of 0.88 percent is based on SANDAG Series 13 Growth Forecast for population growth in the VID service area between 2015 and 2040. This assumption will be used in conjunction with the new pipe break rate assumption to estimate the impact of growth on future system performance.
- **New Pipe Break Rate** – This assumption estimates the rate at which new pipe installed in the system will break over the forecast period. While newly installed pipe generally break less often, breaks will occur on newly installed pipe.

2.5.2 Format of Scenario Results

The results of each scenario are provided both in graphical and descriptive format. A description of the graphs is included below.

Figure 2-8 shows the impact of the current investment level on the number of years required to replace the entire system. The green bars summarize the investment level in terms of miles replaced per year (secondary y-axis). To provide context, the red circles summarize historic system replacement rate cycles. For example, in 2000, the District operated 415 miles of pipe and replaced 3 miles of pipe. At that rate, it would take approximately 140 years to replace the entire system. The blue circles forecast future system replacement rates based on the current size of the system, estimated growth, and future replacement levels. In this scenario, because replacement levels are flat yet the system is assumed to grow, the years to replace the system is forecast to increase from 221 years to 268 years by 2040.

Figure 2-8. Sustainability – Years to Replace Entire System

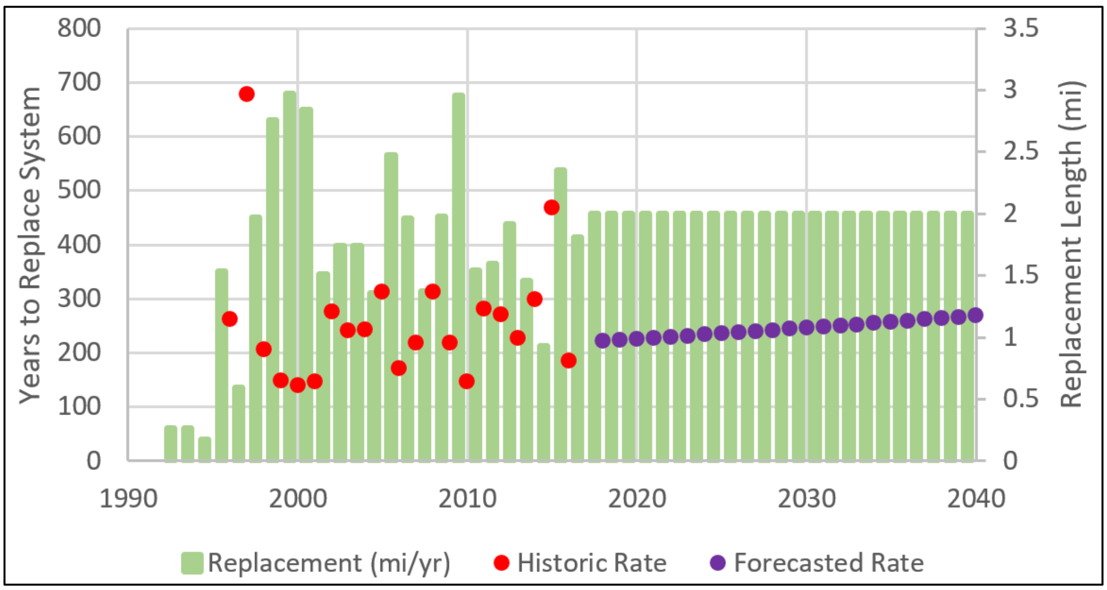


Figure 2-9 shows the impact of the current investment level on service levels. The green bars summarize the investment level in terms of miles replaced per year (secondary y-axis). The primary y-axis shows the annual break rate in terms of annual breaks per 100 miles of pipe in service. The red circles show the District's historic break rate and is associated with the primary y-axis. The blue circles show the District's forecasted break rate and is also associated with the primary y-axis. In this scenario where investment levels are held constant, service levels deteriorate from a break rate of approximately seven to a break rate of approximately eight by 2040.

Figure 2-9. Forecasted Service Levels

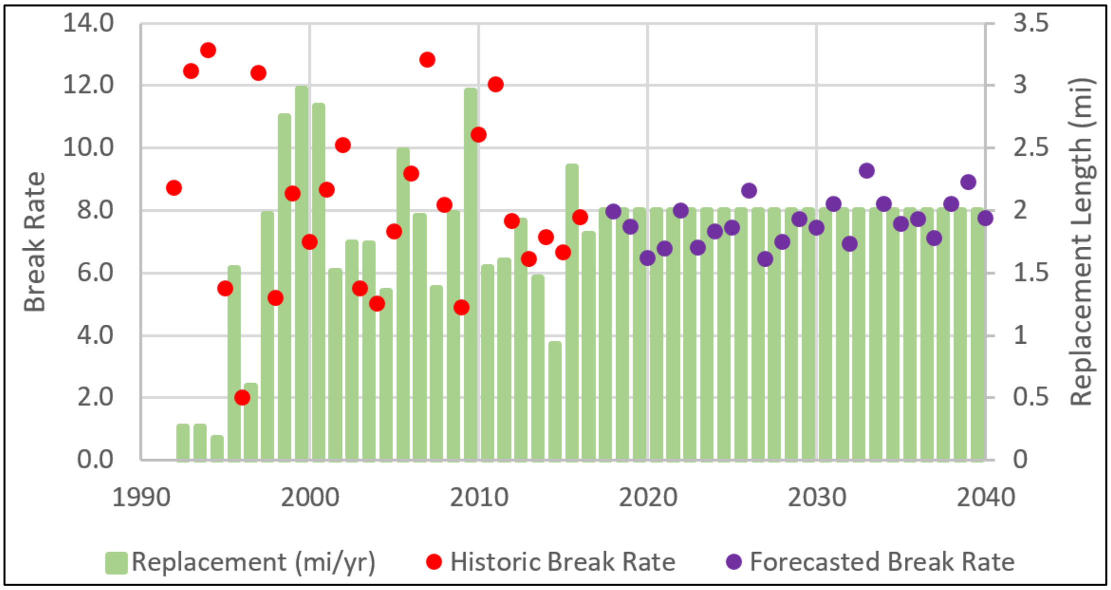


Figure 2-10 shows the impact of the current investment level on the number of breaks that occur per year in the system. This figure is intended to communicate the implications to District staffing levels to respond to the breaks. The green bars summarize the investment level in terms of miles replaced per year (secondary y-axis). The primary y-axis shows the count of breaks. The red circles show the District's historic break count and is associated with the primary y-axis. The blue circles show the District's forecasted break count and is also associated with the primary y-axis. In this scenario where investment levels are held constant, the number of breaks per year is forecasted to increase from approximately 35 (which the District is currently staffed to respond to) to approximately 45 by 2040 (an increase of roughly 30 percent under this investment scenario).

Figure 2-10. Staffing Levels – Breaks per Year

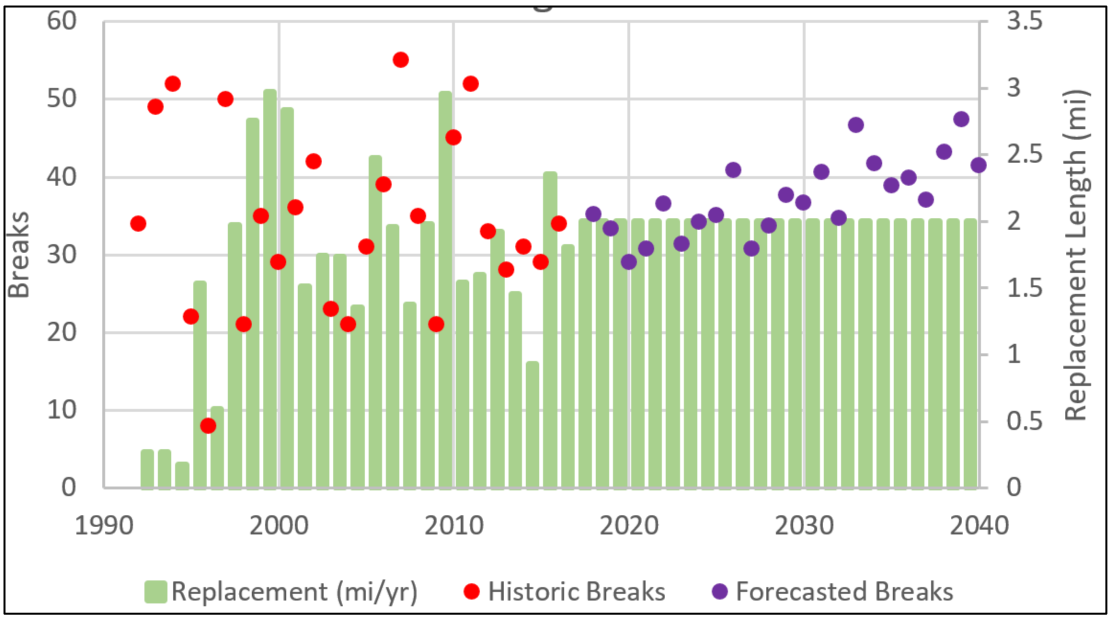
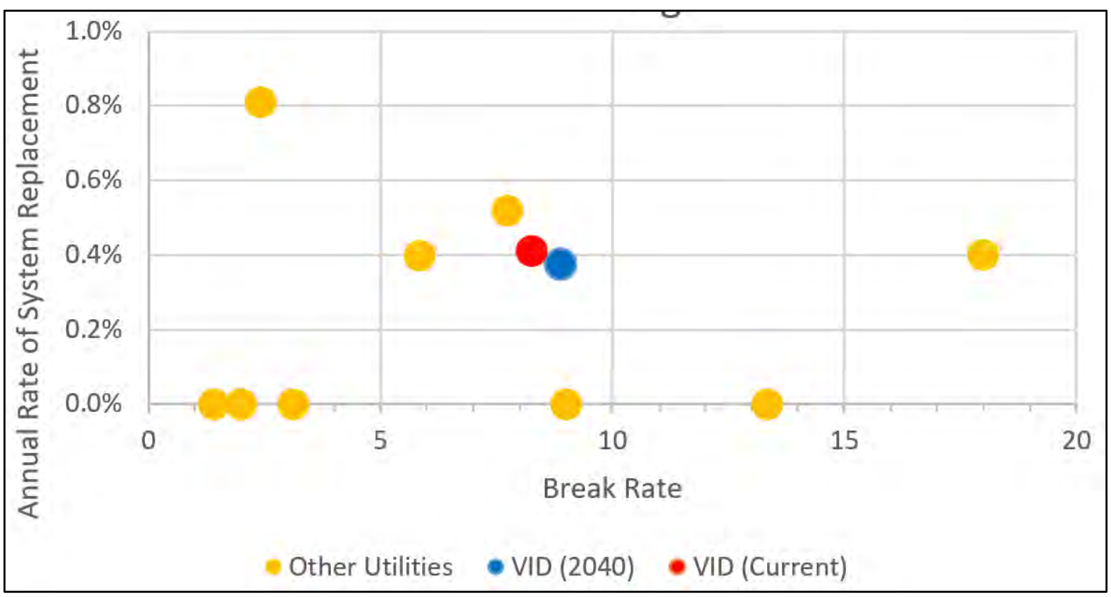


Figure 2-11 benchmarks the District's performance versus other utilities in Southern California. Utilities are benchmarked based on service levels (i.e., break rate) and annual rate of system replacement. In general, utilities with a higher break rate should also have a higher annual replacement rate, although each community must find the appropriate balance between service levels and affordability for their customers. The red circle identified the District's current state. The blue circle identifies where the District is forecasted to operate in 2040 based on the current investment level. Gold circles represent the recent performance of the following utilities:

- City of Carlsbad
- Rainbow Municipal Water District
- City of San Diego
- Helix Water District
- Padre Dam Municipal Water District
- City of San Juan Capistrano
- City of Buena Park
- Sweetwater Authority/South Bay Irrigation District
- Los Angeles Department of Water and Power

Note: several of the utilities that are not replacing pipe are currently evaluating implementation of a pipe replacement program.

Figure 2-11. Replacement Program Benchmarking

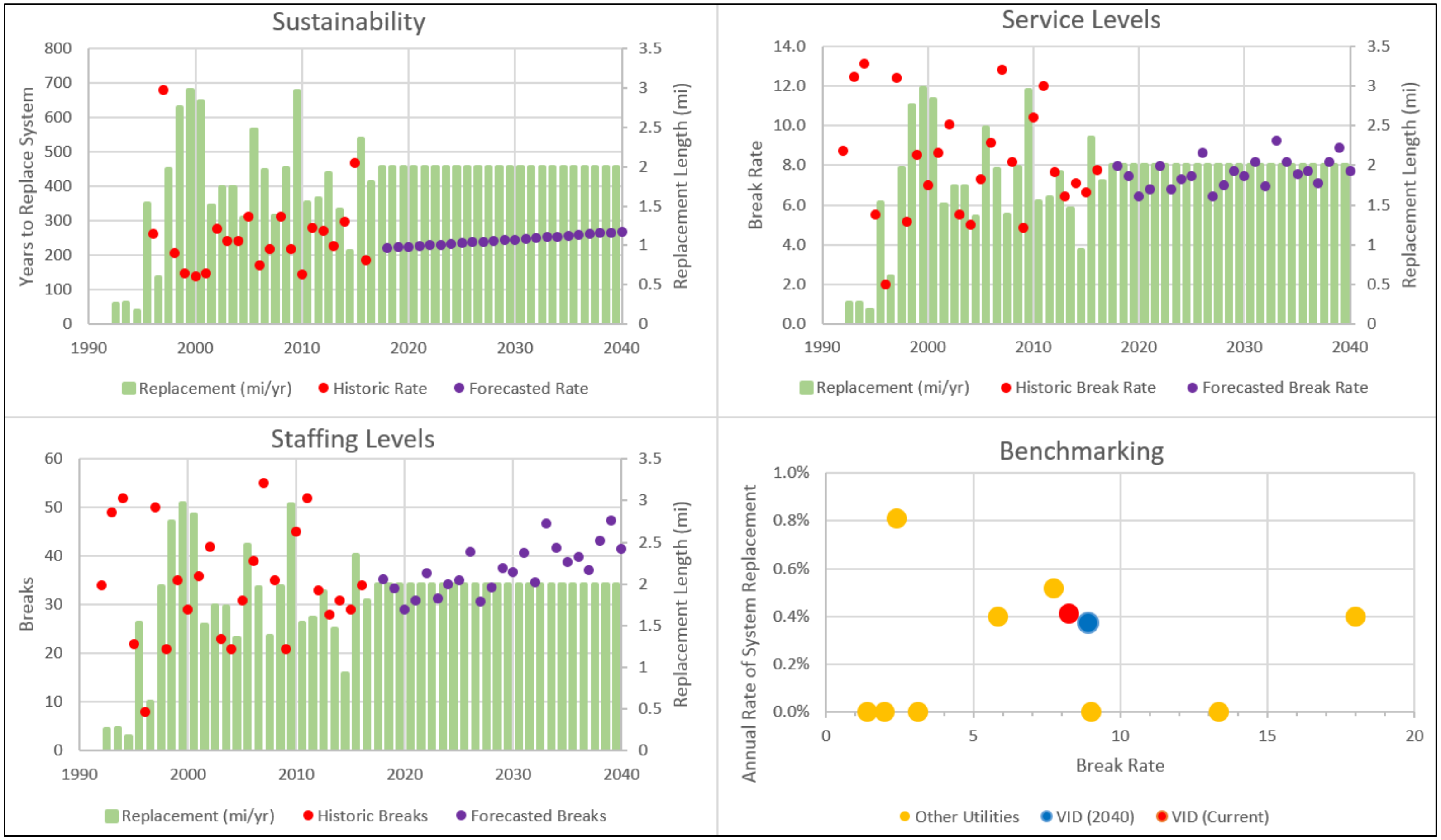


Sections 2.5.3 through 2.5.5 summarize the three investment scenarios that were modeled.

2.5.3 Investment Scenario 1 – Sustain Existing Investment Levels

In this scenario, existing investment levels are held constant through 2040.

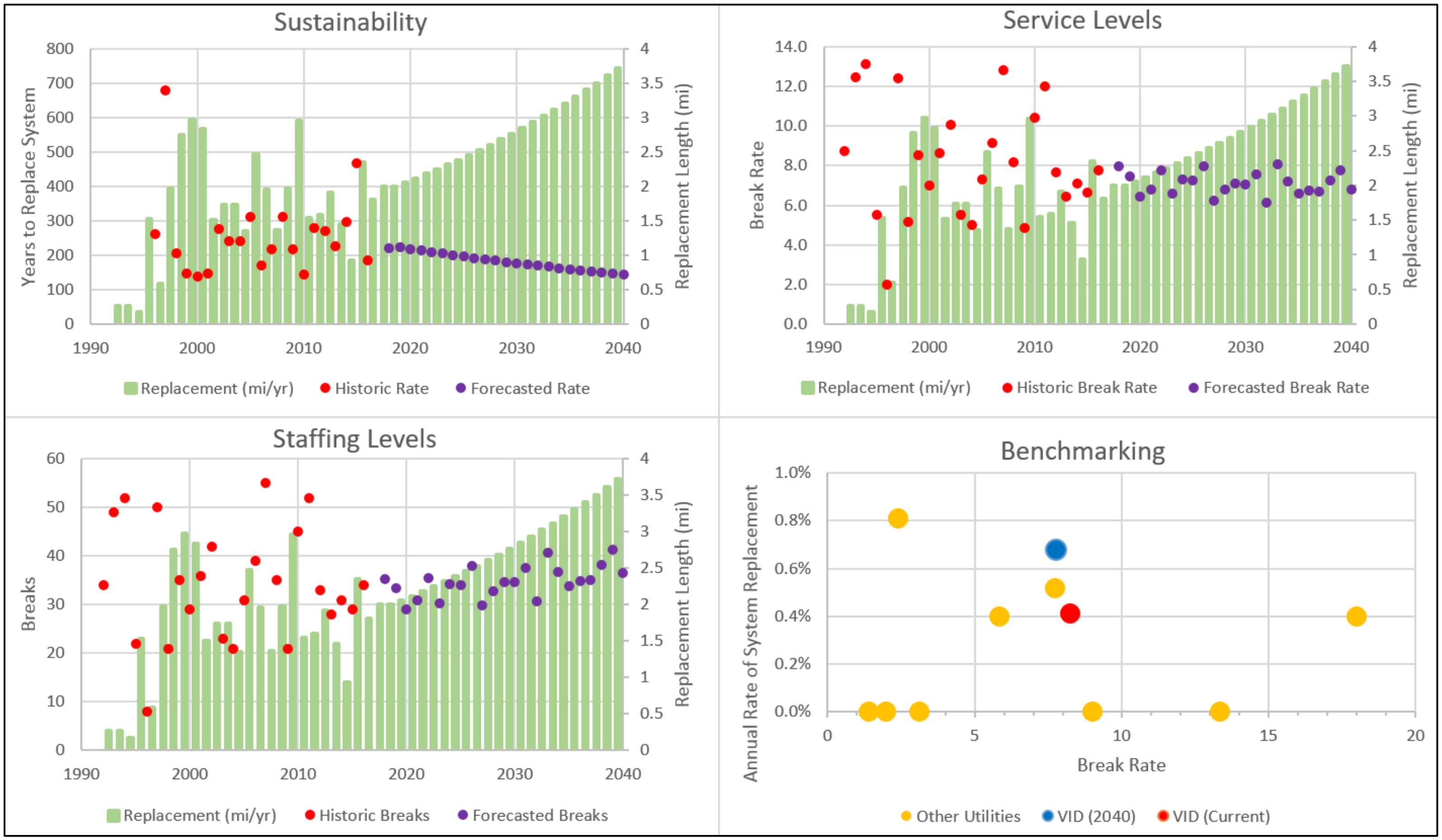
Assumptions		
Current renewal level (mi/yr) = 2	Minimum Replacement Length (ft) = 2000	Annual System Growth1 = 0.88%
Funding Increase Begins = 2020	Annual % Increase = 0%	New Pipe Break Rate = 2



2.5.4 Investment Scenario 2 – Sustain Existing Service Levels

In this scenario, existing service levels are held constant through 2040.

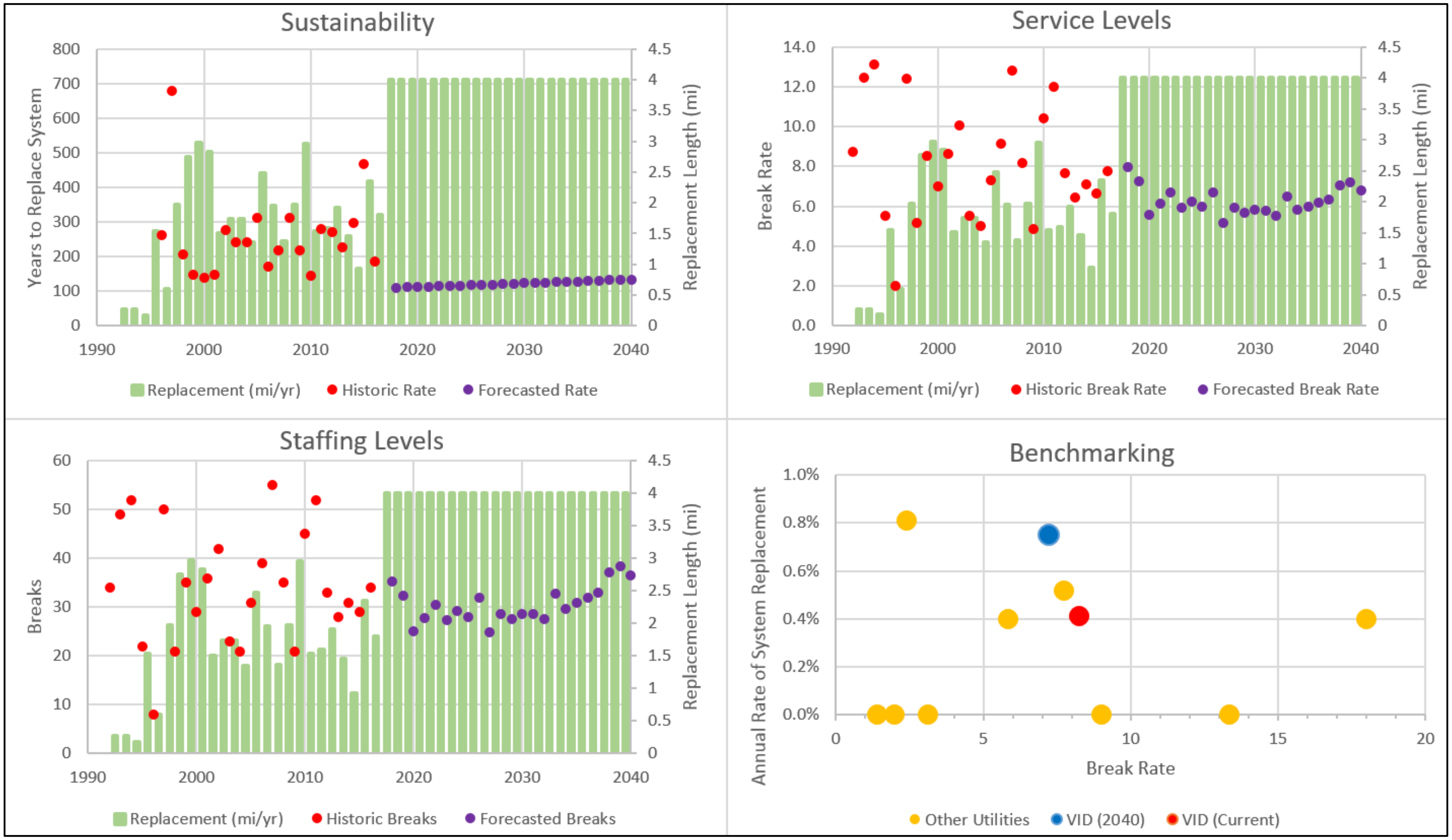
Assumptions		
Current renewal level (mi/yr) = 2	Minimum Replacement Length (ft) = 2000	Annual System Growth1 = 0.88%
Funding Increase Begins = 2020	Annual % Increase = 3%	New Pipe Break Rate = 2



2.5.5 Investment Scenario 3 – Improve Existing Service Levels

In this scenario, historic investment levels are doubled. Service levels are expected to improve.

Assumptions		
Current renewal level (mi/yr) = 4	Minimum Replacement Length (ft) = 2000	Annual System Growth1 = 0.88%
Funding Increase Begins = 2020	Annual % Increase = 0%	New Pipe Break Rate = 2



3 Renewal Identification and Prioritization

3.1 Quantification of Deterioration

3.1.1 Purpose

In this report, the term “cohort” refers to a subset of the water main systems that has a specific characteristic. In general, the intent of the cohort analysis is to better understand broad infrastructure performance trends that will be used to identify and prioritize renewal investments, assess possible break mitigation strategies, and optimize replacement specifications based on deterioration rates. In this section, readily available District pipe data are analyzed at a macro level to:

- Validate that pipe deteriorates over time as infrastructure ages (i.e., do pipes generally break more often as they get older?)
- Determine whether deterioration is non-homogenous (i.e., do cohorts deteriorate at different rates?)
- If deterioration over time is nonhomogeneous, quantify which factors drive deterioration and useful life.

3.1.2 Analysis Method

For the purposes of this study, pipe deterioration rates are measured as a function of infrastructure age verses break rate (in terms of annual breaks per 100 miles of pipe in service). The break rate calculation is shown below where the “number of breaks” is the count of breaks that occurred at a particular pipe age and the “miles of main” is the length of active pipe at a particular age when break data were collected.

Break Rate = (100 * Number of Breaks) / (Miles of Main)

For example, Table 3-1 summarizes all of the breaks in the system that occurred when the pipes that broke were 68 years old.

Table 3-1. Summary of Breaks on 68-year Old Pipe

Installation Year	Break Year	Age When Break Occurred
1939	2007	68
1939	2007	68
1942	2010	68
1943	2011	68

For this study, full year break data was available between 1992 and 2016. Table 3-2 shows miles of pipe that have ever been 68 years old while break data were collected.

Table 3-2. Summary of Pipes 68-years Old when Break Data were Collected (1992-2016)

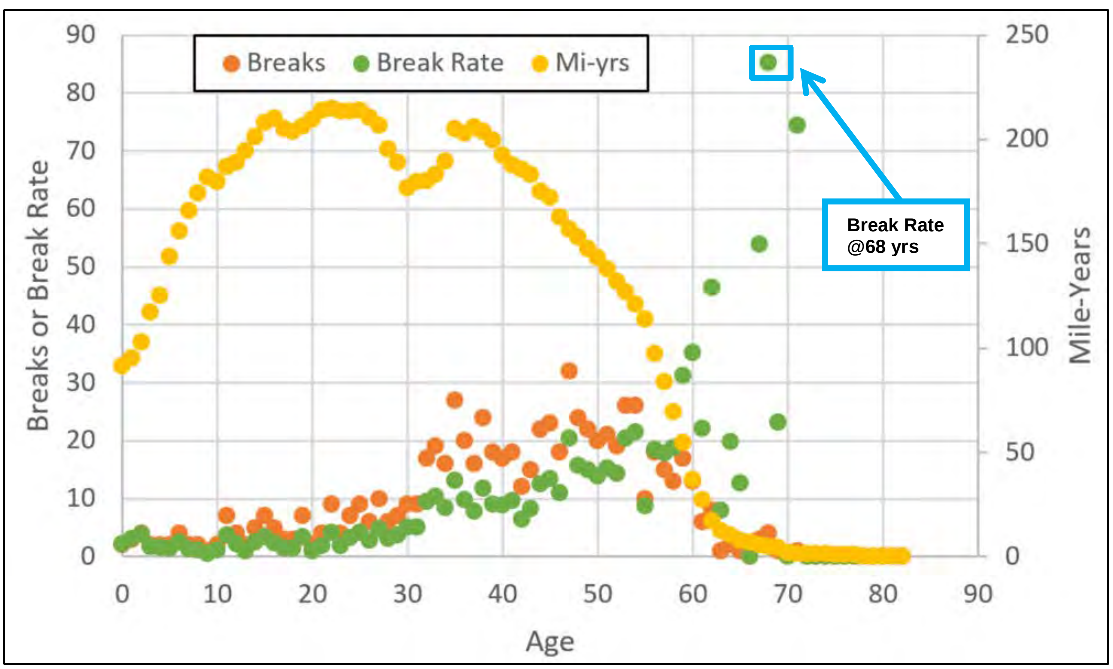
Age of Pipe	Installation Year	Year Pipe Was 68 Years Old	Pipe Length (mi)
68	1929	1997	0.16
68	1939	2007	0.71
68	1942	2010	0.43
68	1943	2011	0.04
68	1946	2014	0.48
68	1947	2015	2.55
68	1948	2016	0.32
		Total	**4.70**

Based on Table 3-1 and Table 3-2, at age 68, the number of breaks is equal to 4, and the miles of main are equal to 4.7. Therefore, the break rate at age 68 is calculated as:

Break Rate = (100 * 4) / (4.7) = 85 annual breaks per 100 miles

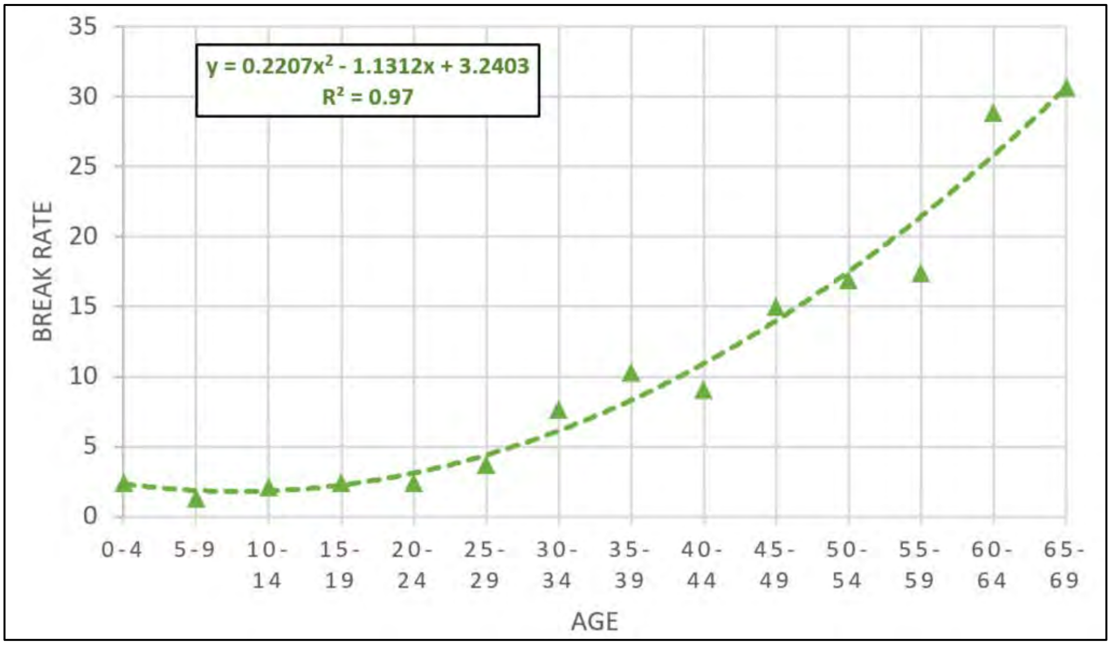
Figure 3-1 shows a scatter graph of the system size in terms of mile-years of data, count of breaks, and break rate by age.

Figure 3-1. Summary of Data Used to Calculate Break Rates



To obtain more statistically relevant information, Figure 3-2 groups data into 5-year age ranges. Age ranges with less than 10 mile-years of data have been filtered out to limit statistically insignificant points. Slightly elevated break rates over the first 5 years of operation may be a result of some minor construction issues. Over the first 25 years of life, the system performed relatively well at a break rate of roughly 2. However, as the pipe ages, break rates increase quickly to a rate of roughly 30 by the time the pipe is 70 years old. While this figure gives us a broad understanding of deterioration, the District's system is comprised of many different pipe cohorts that deteriorate at different rates. These cohorts are explored in more detail in the subsequent sections.

Figure 3-2. System Deterioration Rate



3.1.3 Analysis Approach

While the system-wide deterioration rate confirms that the District's pipes are generally deteriorating as they age, there are likely cohorts of pipes that are deteriorating much faster or much slower than the composite rate. Based on industry experience and institutional knowledge from District staff, the following deterioration factors were analyzed. Factors in bold were found to have a strong correlation with break rates in the District's system.

- **Material Vintage** – Includes pipe material and significant changes in manufacturing, installation, and corrosion protection quality.
- Proximity to Booster Stations
- Owner (District vs. Private)
- **Status**
- **Diameter**
- Soils Characteristics

  - Concrete Corrosion Potential
  - **Steel Corrosion Potential**
  - **Soil Shrink Swell Potential**

The results are presented by factor in the following sections. Each section:

- Describes the theory regarding why a relationship may exist.
- Describes how and why infrastructure was grouped into cohorts.
- Summarizes the system by cohort.
- Quantifies the relationship between the factor and deterioration rate.
- Identifies whether this factor will be considered in determining final risk factors.

3.1.4 Material Vintage

Material vintage seeks to establish a relationship between deterioration rates and a combination of pipe material and installation era, which may indicate significant changes in manufacturing, installation, and corrosion protection quality.

Infrastructure and breaks were grouped into cohorts based on observed changes in infrastructure performance in the District's system, industry guidelines regarding the timing of significant advances in manufacturing and installation practices, and the development of statistically relevant cohorts. Figure 3-3 quantifies the deterioration rate of each material vintage cohort. A description of each cohort and performance observations are included below. Table 3-3 summarizes the miles of active pipe in each material vintage.

- Metallic (1995 or older) and Metallic (Post 1995) – In metallic pipe where the service is not isolated, the electrochemical potential between the copper service and the metallic main will accelerate corrosion of the main in the immediate vicinity of the connection. For wrapped iron mains, complete coverage at the services is not always achieved, which can further concentrate corrosion near the connection to the service. Since 1996, the District has required anodes be attached on all services that are connected to metallic mains. Additionally, in 1996 District staff began attaching anodes when responding to breaks on copper services. At the time, these measures were implemented because in theory, they would slow deterioration rates on metallic pipe and extend useful life. Figure 3-3 quantifies the performance difference between metallic pipe before and after this change was implemented. Metallic pipe that was installed prior to 1996 is the worst performing cohort and will generally have the shortest useful life. Meanwhile, metallic pipe installed after 1995 is one of the best performing cohort and is likely to experience a much longer useful life. The data shows that the District's decision to attach anodes (at a relatively small cost relative to pipeline construction), was prudent and will significantly extend the useful life of metallic pipe installed after 1995.
- Nipponite or Pre-1963 JM – Staff report that poorly performing asbestos cement pipe, called Nipponite, performs much worse than other asbestos cement pipe. Initial analysis shows that pipe classified by staff as Nipponite or asbestos cement pipe manufactured by JM prior to 1963 performs worse than other asbestos cement pipe.

Currently, the District manages 71 miles of this pipe. Figure 3-3 shows that Nipponite is the second worst performing cohort in the system.

- Asbestos Cement (AC) – Staff report that most asbestos cement pipe is performing well. This cohort includes all asbestos cement pipe that is not classified as Nipponite nor manufactured by JM prior to 1963. Currently, the District manages 199 miles of this pipe. Figure 3-3 shows that asbestos cement is performing better than Nipponite and in general is expected to have a longer life.
- PVC (2 or 2.5 inch) – The District reported that they have historically had break issues with 2-inch and 2.5-inch PVC pipe. Currently, the District manages 4 miles of 2-inch or 2.5-inch PVC pipe. This is not enough pipe to perform an age based analysis. However, the data does show that this cohort breaks more than five times as often as the rest of the system even though it is relatively young. This cohort will be identified as having a high likelihood of failure but will be excluded from subsequent analysis to better identify other potential risk factors that may be masked by this poor performing pipe.
- PVC (Excl. 2 or 2.5 inch) – With the exception of 2-inch and 2.5-inch PVC pipe, District staff report other PVC mains generally perform well. This cohort includes all PVC pipe that is not 2-inch or 2.5-inches in diameter. Currently, the District manages 98 miles of this pipe.
- Other – The District manages approximately 2 miles of pipe material not included above (copper, permastran, unknown). This is not enough pipe to perform an age based analysis. This pipe is performing well and will be categorized as a relatively low likelihood of failure.

Figure 3-3. Deterioration Rate by Material Vintage

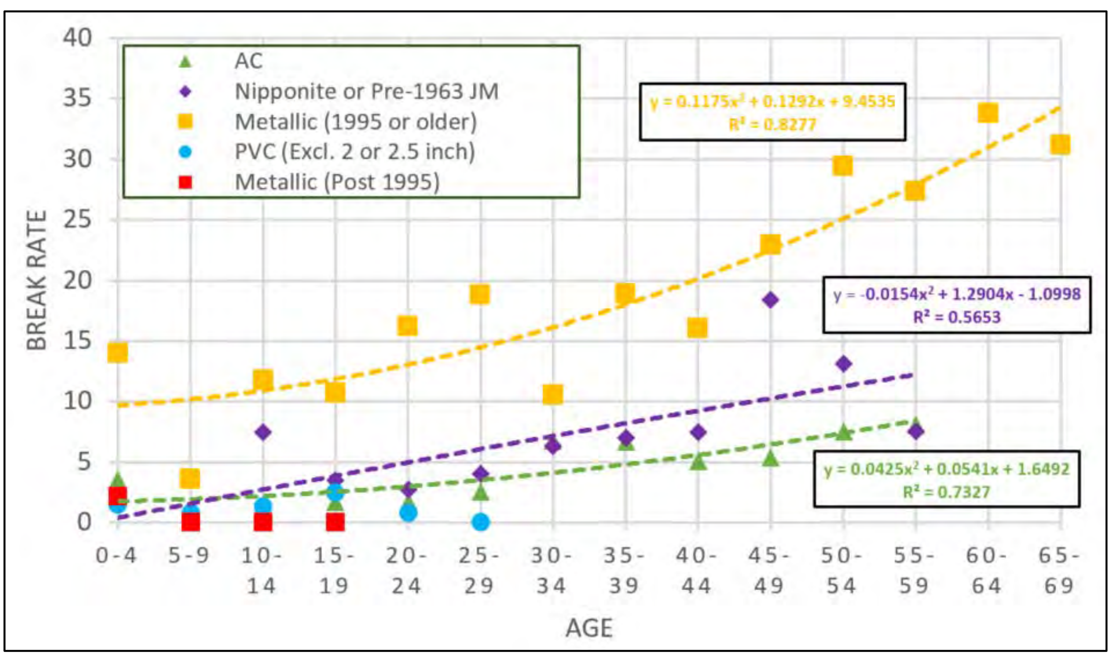


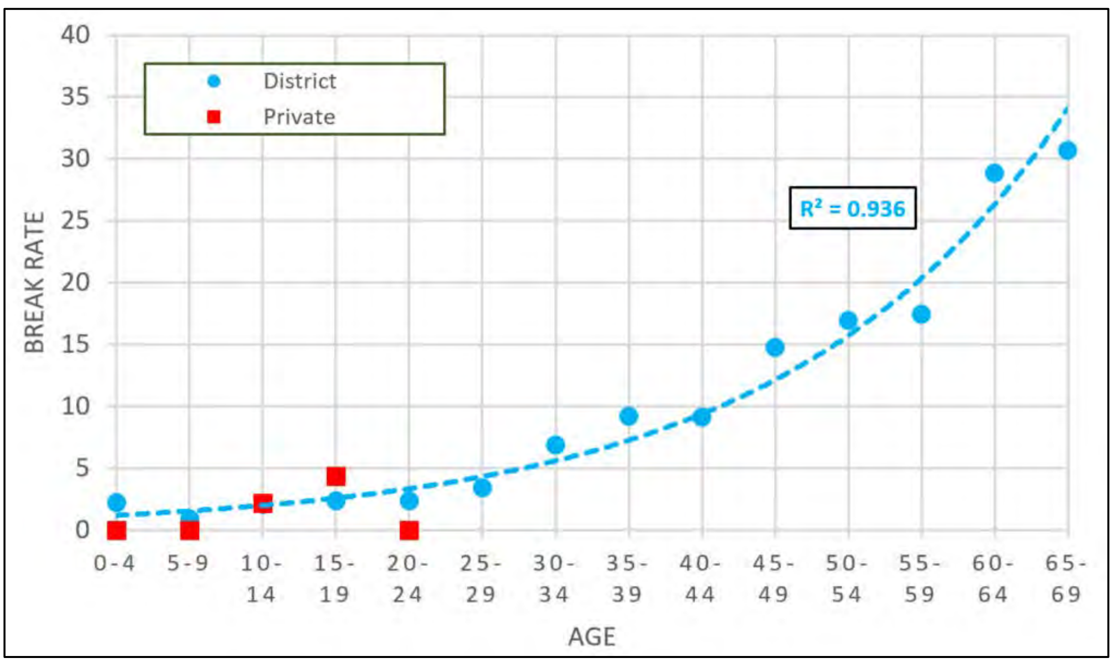
Table 3-3. System Mileage by Material Vintage

Material Vintage	Miles
Metallic (Post 1995)	9.2
PVC (not 2 or 2.5 inch)	98.3
AC	198.7
Other	2.2
Nipponite or Pre-1963 JM	70.6
Metallic (1995 or older)	56.3
PVC (2 or 2.5 inch)	4.0

3.1.5 Owner

The District owns 429 miles of active mainline and also manages 10 miles of private mainline. Figure 3-4 quantifies the deterioration rate of each owner. Since the quantity of privately owned infrastructure is relatively low, there is less confidence in long-term performance estimates. However, readily available data does not indicate that there is a significant difference in performance by owner. Therefore, owner will not be considered in the risk model.

Figure 3-4. Deterioration Rate by Owner



3.1.6 Diameter

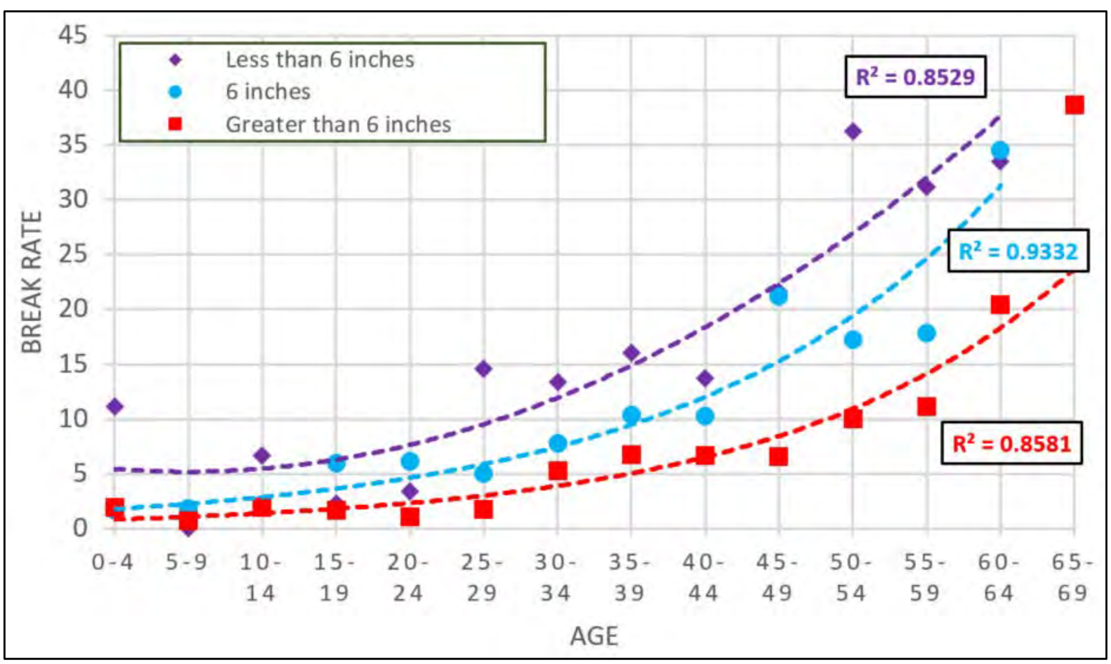
From industry experience, small diameter pipe deteriorates at a more rapid rate than large diameter pipes. In theory, this is related to thinner pipe walls and the generally smaller section modulus associated with smaller diameter pipe. Additionally, large diameter pipes commonly have a more stringent application of design, installation, testing, and construction inspection, which lead to longer lives. Diameters in the District's system are as large as 36 inches. However, the majority of infrastructure has a diameter of between 4 and 12 inches. To develop statistically relevant data sets, infrastructure and breaks were grouped into cohorts based on observed changes in infrastructure performance and development of statistically relevant cohorts. In the District system, 8-inch through 12-inch pipe perform similarly while 6-inch pipe breaks at a much higher rate. Table 3-4 summarizes the asset groups assessed and summarizes the mile of pipes in each group.

Table 3-4. Miles by Diameter Classification

Diameter Class (in)	Pipe Length (mi)
Less than 6	36
6	113
Greater than 6	290

Figure 3-5 quantifies the deterioration rate of each diameter cohort. Even at the same age, smaller diameter pipe performs worse than larger pipe. Because District data supports the theoretical relationship between diameter and performance, this data will be used to assess risk.

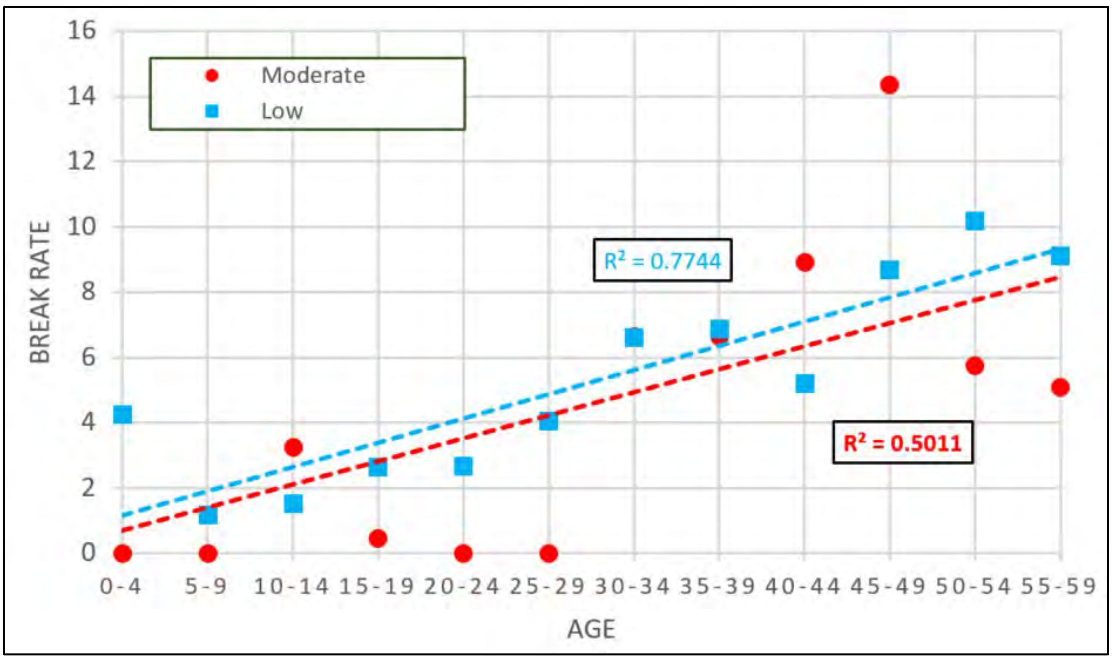
Figure 3-5. Deterioration Rate by Diameter



3.1.7 Asbestos Cement Corrosion Potential

From industry experience, concrete soil corrosion potential typically does not influence deterioration rates for asbestos cement as the primary driver for deterioration is calcium leaching when exposed to groundwater. Readily available concrete corrosion potential data were obtained from the Web Soil Survey (SSURGO database). The data were prepared by the USDA's Natural Resources Conservation Service (NRCS) Soil Survey Staff. District data summarized in Figure 3-6 verifies that concrete soil corrosion potential does not influence deterioration rates. Therefore, this factor will not be used for risk assessment.

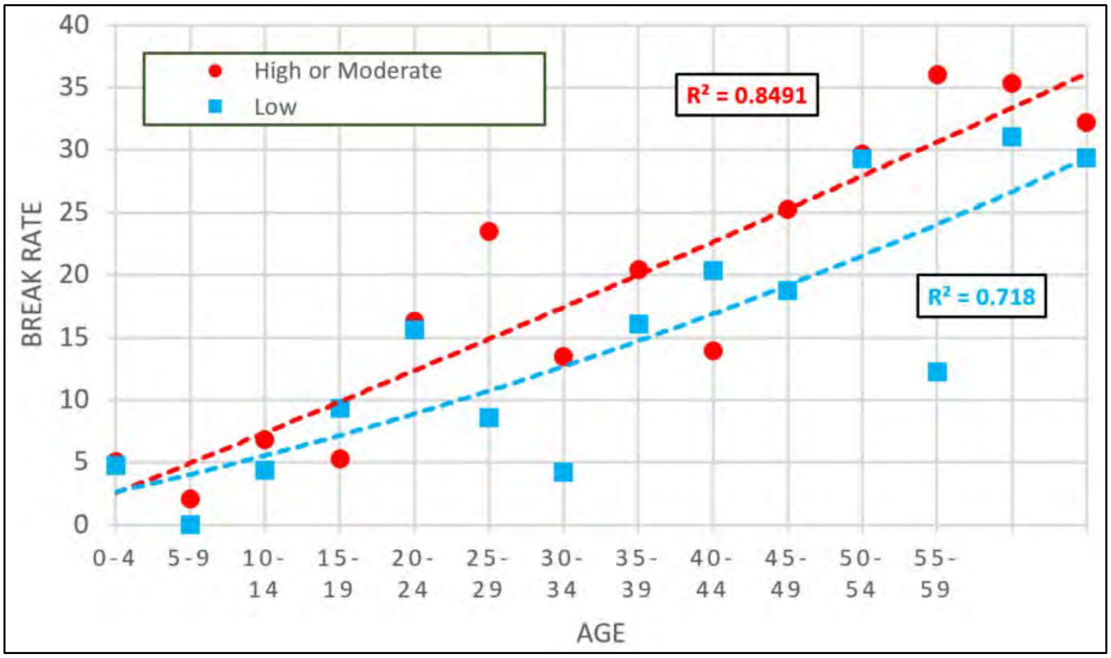
Figure 3-6. Deterioration Rate by Asbestos Cement Corrosion Potential



3.1.8 Metallic Corrosion Potential

From industry experience, metallic soil corrosion potential typically has a significant influence on deterioration rates for metallic pipe as external corrosion is often a primary deterioration source. Readily available metallic corrosion potential data were obtained from the Web Soil Survey (SSURGO database). The data were prepared by the USDA's Natural Resources Conservation Service (NRCS) Soil Survey Staff. District data summarized in Figure 3-7 verifies that metallic soil corrosion potential has a moderate influence deterioration rates. Therefore, this factor will be used for risk assessment.

Figure 3-7. Deterioration Rate by Metallic Corrosion Potential



3.1.9 Shrink-Swell Potential

From industry experience, pipes exposed to larger fluctuations in soil shrinkage and swelling will deteriorate faster. In theory, this deterioration is related to material fatigue and stresses imposed on the pipe during soil shrinkage and swelling. Shrink swell potential typically has a greater impact on brittle pipe (concrete, asbestos cement, etc.) than on ductile pipe (steel, ductile iron, etc.). The severity of this loading is dependent upon the relative variability of moisture content in the system and a soil property called linear extensibility. Linear extensibility refers to the change in length of an unconfined clod as moisture content is decreased from a moist to a dry state. It is an expression of the volume change between the water content of the clod at 1/3- or 1/10-bar tension (33 kPa or 10 kPa tension) and oven dryness. The volume change is reported as percent change for the whole soil. The amount and type of clay minerals in the soil influence volume change. A higher linear extensibility value generally leads to increases in cyclic loadings and a shorter useful life.

Readily available linear extensibility data were obtained from the Web Soil Survey (SSURGO database). The data were prepared by the USDA's Natural Resources Conservation Service (NRCS) Soil Survey Staff. The linear extensibility percentage

(LEP) used for this study is based off of a depth weighted average of all available layers. Each pipe in the system was associated to the nearest linear extensibility value.

Linear extensibility percentages in this system range from 1.5 to 9.6. To develop statistically relevant data sets, infrastructure and breaks were grouped into cohorts based on observed changes in infrastructure performance and development of statistically relevant cohorts. Table 3-5 summarizes the asset groups assessed and summarizes the miles of pipe and breaks in each group. Note that the shrink-swell potential is a measurement of how the volume of the soil will fluctuate when exposed to varying moisture contents. It does not specify for a particular area the frequency or severity of moisture variations.

Table 3-5. Miles by Linear Extensibility Potential Classification

LEP Class	Pipe Length (mi)
High (>7.5%)	118
Low (<7.5%)	321

Figure 3-8 and Figure 3-9 summarize the relationship between shrink-swell potential and performance for asbestos cement and metallic pipe respectively. District data validates industry experience that shrink swell potential is a primary driver for asbestos cement pipe and negligible for metallic pipe. Therefore, this factor will only be used for the risk assessment of asbestos cement pipe.

Figure 3-8. Deterioration Rate by Asbestos Cement Shrink-Swell Potential

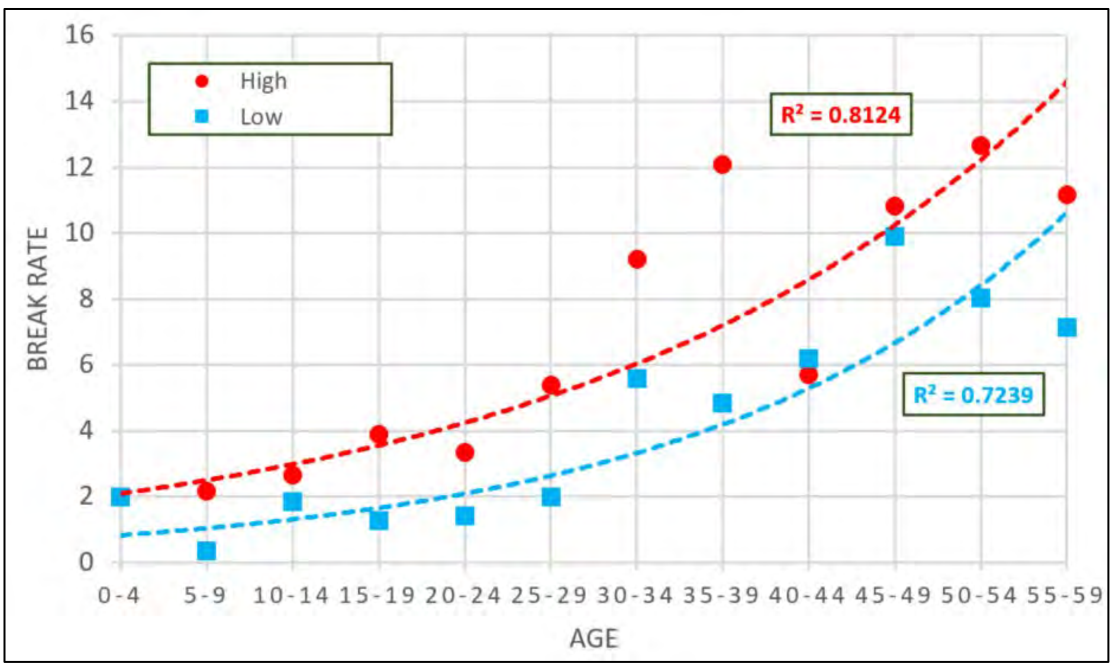
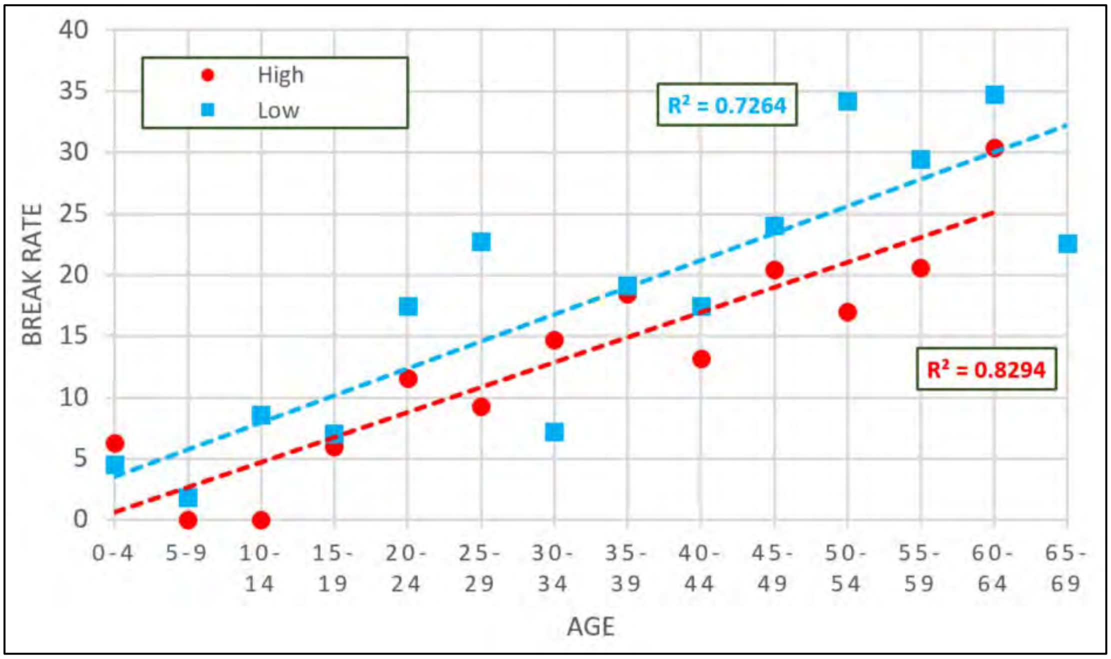


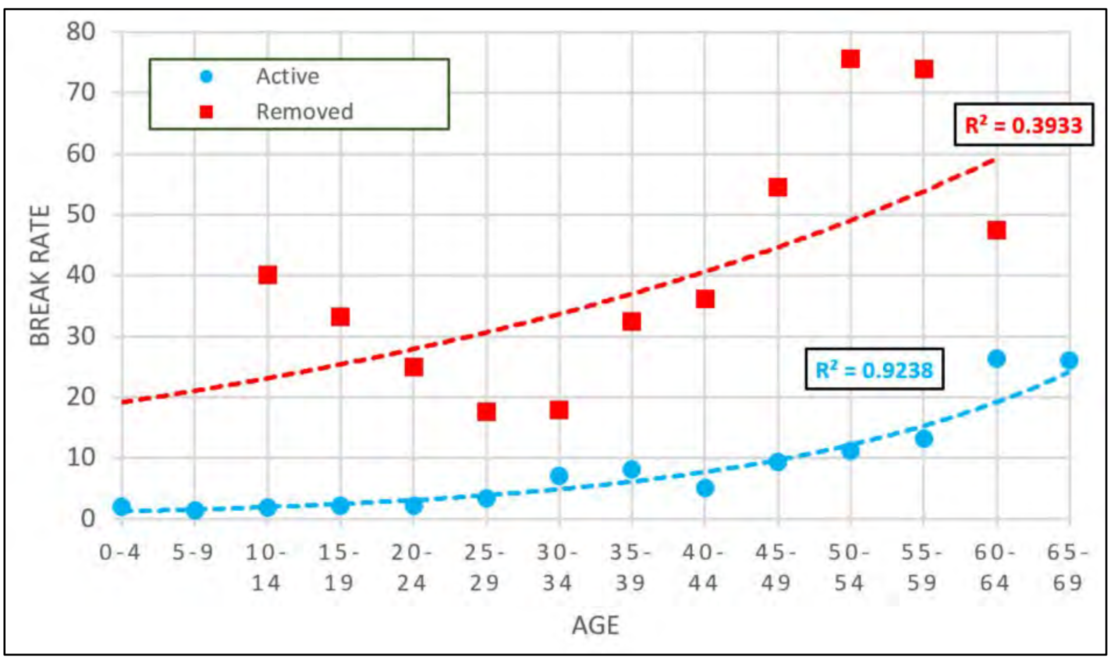
Figure 3-9. Deterioration Rate by Metallic Shrink-Swell Potential



3.1.10 Status

Figure 3-10 summarizes the performance of active pipe and pipe that has been removed from service. The data indicates that the District has been effective in targeting pipe replacement on poor performing pipe.

Figure 3-10. Deterioration Rate by Status



3.2 Risk Assessment

The District manages approximately 439 miles of water pipeline infrastructure. As the system continues to age and deteriorate, the District has and will continue to identify and prioritize pipe replacement projects for the purpose of cost effectively sustaining desired service levels. To accomplish this, a Project Risk Score (PRS) was developed. The PRS quantifies relative risk on a scale of zero (lowest risk) to one hundred (highest risk). This methodology considers the consequence of failure (CoF), the likelihood of failure (LoF), and hydraulic limitations.

The purpose of this section is to describe the methodology for calculating the PRS. The PRS should be updated regularly to account for new data such as break history. As the program continues to mature, it is anticipated that the PRS calculation methodology will adapt to changing drivers, experiences, and readily available information.

3.2.1 Basis of PRS

District pipes are divided into small lengths at diameter changes, material changes, install date changes, valves, tees/crosses, bends, and other attributes. This is useful for some purposes (hydraulic modeling, cohort analysis, attribute data management, etc.) but it is not a useful basis for renewal decision making because it is not cost effective to renew infrastructure in such small units. Therefore, it is necessary to aggregate these short pipes into more meaningful groupings so asset specific information can be determined such as break count, pipe performance, renewal budgeting, probability of failure, consequence of failure, and project identification and prioritization.

The District manages two fields in GIS (“INSTALLNUM” and “WORKORDERTYPE”) that identify the unique project under which the pipe was constructed. Construction project data can provide insight regarding the relative quality of the material used, transport and handling procedures, installation quality, backfill quality, and construction management quality. Industry experience tells us that pipeline performance and useful life can vary significantly from one construction project to the next. Analysis of break data validates that District pipeline performance varies significantly by project.

The PRS is calculated for each project in the system.

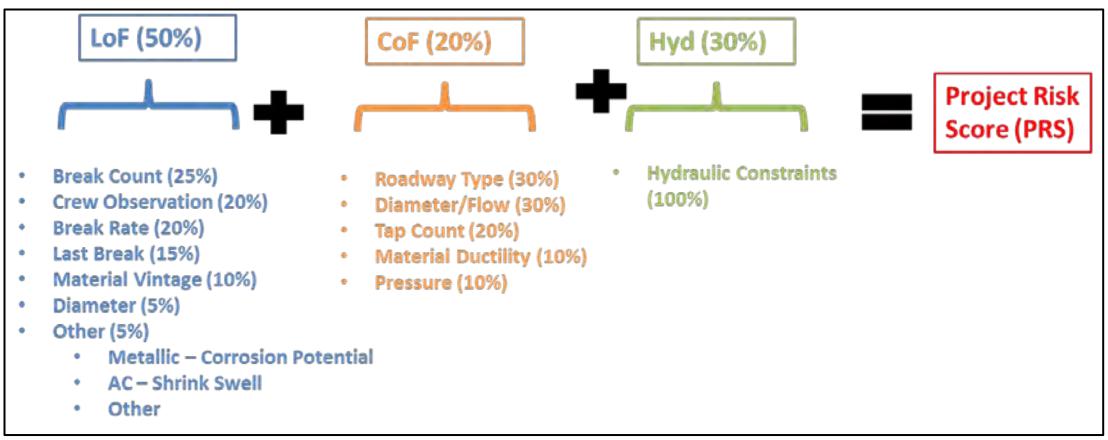
It should be noted that projects are not meant to constrain the extent on which asset-specific decisions must be made. Rather, the intent is to group short pipes in GIS in a way that more directly aligns with the extents of which asset-based decisions will be made.

3.2.2 PRS Calculation Methodology

The PRS quantifies relative risk on a scale of zero (lowest risk) to one hundred (highest risk). Figure 3-11 summarizes the PRS calculation methodology. The PRS is calculated as a weighted summation of the LoF, CoF, and hydraulic constraints. This weighted summation method is one of two methods commonly used in the industry to assess risk (the other being multiplication of LoF, CoF, and hydraulic). The weighted summation method was selected because it allows additional flexibility in weighting various factors based on their importance, which ultimately allows for more meaningful and useful results. For example, when planning for replacement projects, most utilities find that LoF is more important than CoF, particularly for smaller diameter pipes. In part, this is because a renewal project will typically mitigate LoF but will rarely mitigate CoF, which is typically driven by the operational context of the pipe (e.g., number of customers served, types of customers served, surface features, etc.). Therefore, most mature pipeline risk models will weight LoF higher than CoF.

Based on discussions with staff and evaluation of initial results, the initial LoF weighting was set at 50 percent, the CoF weighting was set at 20 percent, and the hydraulic weighting was set at 30 percent. Because the PRS is on a scale of zero to one hundred, LoF can contribute up to 50 points, CoF can contribute up to 20 points, and hydraulic constraints can contribute up to 30 points.

Figure 3-11. Risk Calculation Method



Each of the PRS criterion (LoF, CoF, and hydraulic) are made up of one to many factors. For example, the roadway type is a CoF factor because a failure under a freeway is often more consequential than a failure under a minor street. Each factor was scored on a zero to one-hundred scale (Factor Score) where zero represents the lowest risk and one-hundred represents the highest risk. Each factor contributes to the PRS based on the following equations:

$$PRS = \sum \ Factor\ Score * Factor\ Weight * Criterion\ Weight$$

For example, if a pipe has a roadway type of "Freeway" it gets the highest factor score of one hundred. Roadways have a Factor Weight of 30% of the CoF. CoF has a criterion weight of 20% of the PRS. Therefore, a pipe near a freeway will contribute six points to the PRS:

Freeway contribution to PRS = 100 * 30% * 20% = 6

The following subsections describe the method for quantifying LoF, CoF, and hydraulic scores. Each factor has a summary table which includes the miles of active pipe, the factor score, and the contribution to the overall PRS.

3.2.2.1 Likelihood of Failure

Based on the analysis performed in Sections 2 and 3 of this technical memorandum and institutional knowledge, the following LoF factors are used:

- Break Count (25 percent)
- Crew Observation (20 percent)
- Break Rate (20 percent)
- Last Break (15 percent)
- Material Vintage (10 percent)
- Diameter (5 percent)
- Other (5 percent)
    - Metallic – Corrosion Potential
    - AC – Shrink Swell Potential
    - Other

Break Count

Industry experience and District data show that leveraging historic project break data is the best indicator of LoF. Therefore, this factor is weighted the highest. As shown in Figure 2-2, there is a strong relationship between break count and the duration to the next break. A pipe that has broken in the past is more likely to break again in the future; multiple breaks on a single line increase that likelihood even more. A weighting of 25 percent was assigned to this factor. Table 3-6 summarizes the factor by miles of pipe, the factor score, and the contribution of this factor to the overall PRS. The summation of all PRS contribution scores provides the overall PRS.

Table 3-6. Break Count Factor Scoring

Break Count	Miles	Factor Score	PRS Contribution
0	318.0	0	0
1	67.0	10	1.25
2	20.6	20	2.5
3	9.1	30	3.75

Break Count	Miles	Factor Score	PRS Contribution
4	8.2	40	5
5	0.9	50	6.25
6	6.7	50	6.25
7	0.3	70	8.75
8	1.1	80	10
9	0	90	11.25
10 or more	7.1	100	12.5

Annual Break Rate

The annual breaks per 100 miles (i.e., break rate) are calculated for the project. For pipes installed after break data collection started in 1992, the install date and the last data of readily available break data were used (May 28, 2016). For pipes installed prior to break data collection, the duration used was between the start of break data collection and the last data of readily available break data were used (May 28, 2016). This method places additional emphasis on shorter pipe where more bang for the buck may be realized. A weighting of 20 percent was assigned to this factor. Table 3-7 summarizes the factor by miles of pipe, the factor score, and the contribution of this factor to the overall PRS. The summation of all PRS contribution scores provides the overall PRS.

Table 3-7. Annual Break Rate Factor Scoring

Break Rate	Miles	Factor Score	PRS Contribution
0	318.0	0	0
0-10	56.1	10	1
10-20	28.6	20	2
20-30	14.7	40	4
30-40	6.2	60	6
40-50	1.7	80	8
Greater than 50	13.7	100	10

Years since Last Break

Industry experience and District data show that projects that have recently broken are more likely to break again. In other words, if two projects otherwise have equal risk but Project A last broke 4 years ago and Project B broke 3 months ago, Project B has the greater LoF. Therefore, projects with a more recent break were assigned a higher factor score. Industry experience suggests this factor is a good indicator of LoF but not as good as the break count or break rate. Therefore, this factor is weighted at 15 percent of the LoF score. Table 3-8 summarizes the factor by miles of pipe, the factor score, and the contribution of this factor to the overall PRS. The summation of all PRS contribution scores provides the overall PRS.

Table 3-8. Years since Last Break Factor Scoring

Years	Miles	Factor Score	PRS Contribution
Never	304.4	0	0
More than 9	59.3	10	0.75
8-9	5.8	20	1.5
7-8	5.8	30	2.25
6-7	10.8	40	3
5-6	6.7	50	3.75
4-5	1.2	60	4.5
3-4	6.9	70	5.25
2-3	8.8	80	6
1-2	16.9	90	6.75
Less than 1	0.6	100	7.5

Material Vintage

The deterioration analysis documented in Chapter 2 has shown that some cohorts deteriorate more rapidly than others. The material vintage is currently assigned a weight of 10 percent. Table 3-9 summarizes the factor by miles of pipe, the factor score, and the contribution of this factor to the overall PRS. The summation of all PRS contribution scores provides the overall PRS.

Table 3-9. Material Vintage Factor Scoring

Material Vintage	Miles	Factor Score	PRS Contribution
Metallic (Post 1995)	9.2	0	0
PVC (not 2 or 2.5 inch)	98.3	0	0
AC	198.7	30	1.5
Other	2.2	30	1.5
Nipponite or Pre-1963 JM	70.6	50	2.5
Metallic (1995 or older)	56.3	80	4
PVC (2 or 2.5 inch)	4.0	100	5

Diameter

The deterioration analysis documented in Chapter 2 has shown that some cohorts deteriorate more rapidly than others. The diameter is currently assigned a weight of 5 percent. Table 3-10 summarizes the factor by miles of pipe, the factor score, and the contribution of this factor to the overall PRS. The summation of all PRS contribution scores provides the overall PRS.

Table 3-10. Diameter Factor Scoring

Diameter (inches)	Miles	Factor Score	PRS Contribution
Greater than 6 inches	290.3	0	0
6 inches	113.3	50	1.25
less than 6 inches	35.5	100	2.5

Other

The deterioration analysis documented in Chapter 2 has shown that metallic pipe deterioration is influenced by soil corrosion potential and that AC deterioration is influenced by shrink swell. Other materials did not have such an influence. This factor is currently assigned a weight of 5 percent. Table 3-11 summarizes the factor by miles of pipe, the factor score, and the contribution of this factor to the overall PRS. The summation of all PRS contribution scores provides the overall PRS.

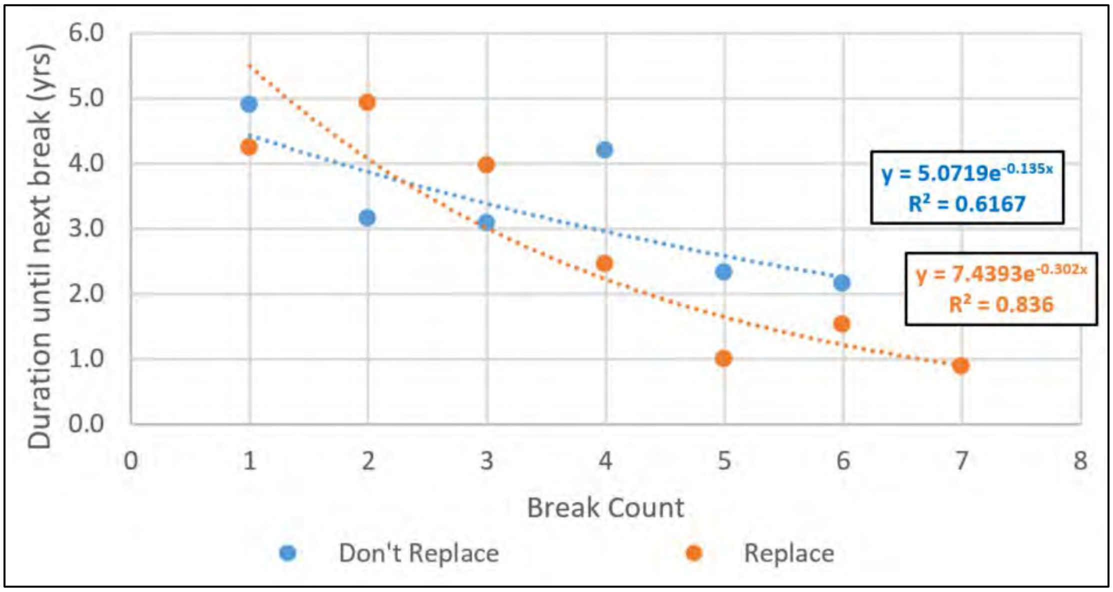
Table 3-11. Other Factor Scoring

Other Description	Miles	Factor Score	PRS Contribution
AC - High Shrink-Swell	70.8	100	2.5
AC - Low Shrink-Swell	198.4	0	0
Metallic - Low Corrosion	24.4	0	0
Metallic -High or Moderate Corrosion	41.0	100	2.5
Other	104.5	50	1.25

Crew Observation

Currently, crews identify whether they would replace the pipe based on the actual observed condition of a pipe. To quantify whether crews can visually observe and predict likelihood of failure, the duration until next break described in Section 2.4 was applied to breaks where the crew recommended the pipe be replaced or not replaced. Any breaks where the crew documented that they weren’t sure if the pipe should be replaced were excluded. Figure 3-12 summarizes the results of this analysis. On average, breaks where the crew thought the pipe should not be replaced lasted 20 percent longer before the next break than pipes the crew thought should be replaced. Based on this analysis, this factor was assigned a score of 20 percent.

Figure 3-12. Crew Observation Can Support Risk Assessment



The number of times a crew recommended that each project be replaced was counted. Table 3-12 summarizes the factor by miles of pipe, the factor score, and the contribution of this factor to the overall PRS. The summation of all PRS contribution scores provides the overall PRS.

Table 3-12. Crew Observation Factor Scoring

"Replace It" Count	Miles	Factor Score	PRS Contribution
0	402.7	0	0
1	26.3	50	5
2	4.9	70	7
3 or more	5.3	100	10

3.2.2.2 Consequence of Failure

The CoF is primarily a desktop analysis that focuses on the impact the failure would have on the service provided by the system or the risk for financial expenditures the District would incur due to the failure. The following CoF criteria are considered in the risk assessment:

- Roadway (30 percent)
- Diameter (30 percent)
- Tap Count (20 percent)
- Material Ductility (10 percent)
- Pressure (10 percent)

Diameter

In general, breaks on larger diameter pipes are more consequential because:

- More water is released, increasing the potential for damage
- Larger pipelines are more likely to affect system operations over a larger service area
- Large pipeline repairs typically require a larger excavation, more expensive materials, and often take more time to fix

Based on industry experience and input from Staff, this factor is weighted at 30 percent of the CoF score. Table 3-13 summarizes the factor by miles of pipe, the factor score, and the contribution of this factor to the overall PRS. The summation of all PRS contribution scores provides the overall PRS.

Table 3-13. Diameter Factor Scoring

Diameter (inches)	Miles	Factor Score	PRS Contribution
4 inches and less	35.3	0	0
6 inches	113.3	20	1.2
8 inches	160.4	40	2.4
Unknown	0.2	40	2.4
10 to 12 inches	85.9	60	3.6
14 to 21 inches	33.9	80	4.8
24 inches and greater	10.2	100	6

Material Ductility

A brittle pipe that fractures generally damages more property and is more difficult to repair than an equally-sized ductile pipe that merely “leaks.” Of the materials that dominate the District system, AC and PVC pipe are the only ones that typically fracture. Steel is least likely to leak. A weighting of 10 percent was assigned to this factor.

Table 3-14 summarizes the factor by miles of pipe, the factor score, and the contribution of this factor to the overall PRS. The summation of all PRS contribution scores provides the overall PRS.

Table 3-14. Material Ductility Factor Scoring

Description	Miles	Factor Score	PRS Contribution
Metallic	66.6	0	0
Unknown	0.2	20	0.4
AC	198.7	60	1.2
Plastic	103.2	100	2
Nipponite	70.6	100	2

Tap Count

In general, breaks on a pipe with a higher tap count are more consequential because they put more customers out of service while the break is being repaired. The District has a record of taps in GIS. A count of these taps were summarized by Project. Based on industry experience and input from staff, this factor is weighted at 20 percent of the CoF score.

Table 3-15 summarizes the factor by miles of pipe, the factor score, and the contribution of this factor to the overall PRS. The summation of all PRS contribution scores provides the overall PRS.

Table 3-15. Project Tap Count Factor Scoring

Project Tap Count	Miles	Factor Score	PRS Contribution
0	42.3	0	0
1-10	230.6	40	1.6
11-20	83.4	60	2.4
21-30	41.6	70	2.8
31-40	14.6	80	3.2
41-50	9.7	90	3.6
>50	16.9	100	4

Roadway

Breaks near or under significant roadways can result in impacts to health and safety, economics, public disruption, and District reputation. Road classifications used in this study were obtained from the US Census Bureau TIGER/Line GIS dataset. Pipes were associated to the nearest roadway type. Based on industry experience and input from staff, this factor is weighted at 30 percent of the CoF score.

Table 3-16 summarizes the factor by miles of pipe, the factor score, and the contribution of this factor to the overall PRS. The summation of all PRS contribution scores provides the overall PRS.

Table 3-16. Roadway Factor Scoring

Roadway Description	Miles	Factor Score	PRS Contribution
Minor	408.7	0	0
Secondary Road	19.6	60	3.6
Rail	3.9	90	5.4
Freeway	7.0	100	6

Pressure

High pressure breaks can result in greater impacts to health and safety, economics, and disruptions. Maximum system pressure data was obtained from each hydraulic model node. Each pipe was associated to the pressure of the closest node. Based on industry experience and input from District staff, this factor is weighted at 10 percent of the CoF score.

Table 3-17 summarizes the factor by miles of pipe, the factor score, and the contribution of this factor to the overall PRS. The summation of all PRS contribution scores provides the overall PRS.

Table 3-17. Pressure Factor Scoring

Pressure (psi)	Miles	Factor Score	PRS Contribution
<60	31.1	0	0
60-70	36.9	10	0.2
70-80	45.7	20	0.4
80-90	48.7	30	0.6
90-100	48.1	40	0.8
100-110	48.3	50	1
110-120	45.9	60	1.2
120-130	37.3	70	1.4
130-140	34.0	80	1.6
140-150	24.6	90	1.8
>150	38.6	100	2

3.2.2.3 Hydraulic Constraints

The distribution system was evaluated under existing and projected ultimate demand conditions as part of the hydraulic model analyses described in Chapters 7 and 8 of the 2017 Water Master Plan. The system was reviewed for potential deficiencies by comparing model output with evaluation criteria for model scenarios including peak hour system operation and fire flow. With regard to the evaluation criteria, no critical system pressures were identified during peak hour demand scenarios. However, pipes experiencing high headloss and high velocity during peak hour demand were identified. Based on direction from District staff, three existing system improvement projects, EX-1 through EX-3, were recommended to introduce redundancy at key locations and increase system reliability. Additionally, two ultimate system projects, ULT-1 and ULT-2, were recommended to address potential high velocities in the distribution system under ultimate demand conditions. High headloss pipes were considered a low priority and not addressed as part of the recommended improvement projects. The recommended improvements for both existing and projected ultimate demand scenarios are displayed in Table 7-2 and Table 8-2 in the 2017 Master Plan. Pipes related to fire flow deficiencies were also identified using the hydraulic model. Most of these deficiencies are due to small diameter dead-end pipes restricting flow to particular hydrants. Pipes related to fire flow deficiencies for the existing and ultimate system are represented by the Existing Fireflow and Ultimate Fireflow categories.

Table 3-18 summarizes the factor by miles of pipe, the factor score, and the contribution of this factor to the overall PRS. The summation of all PRS contribution scores provides the overall PRS.

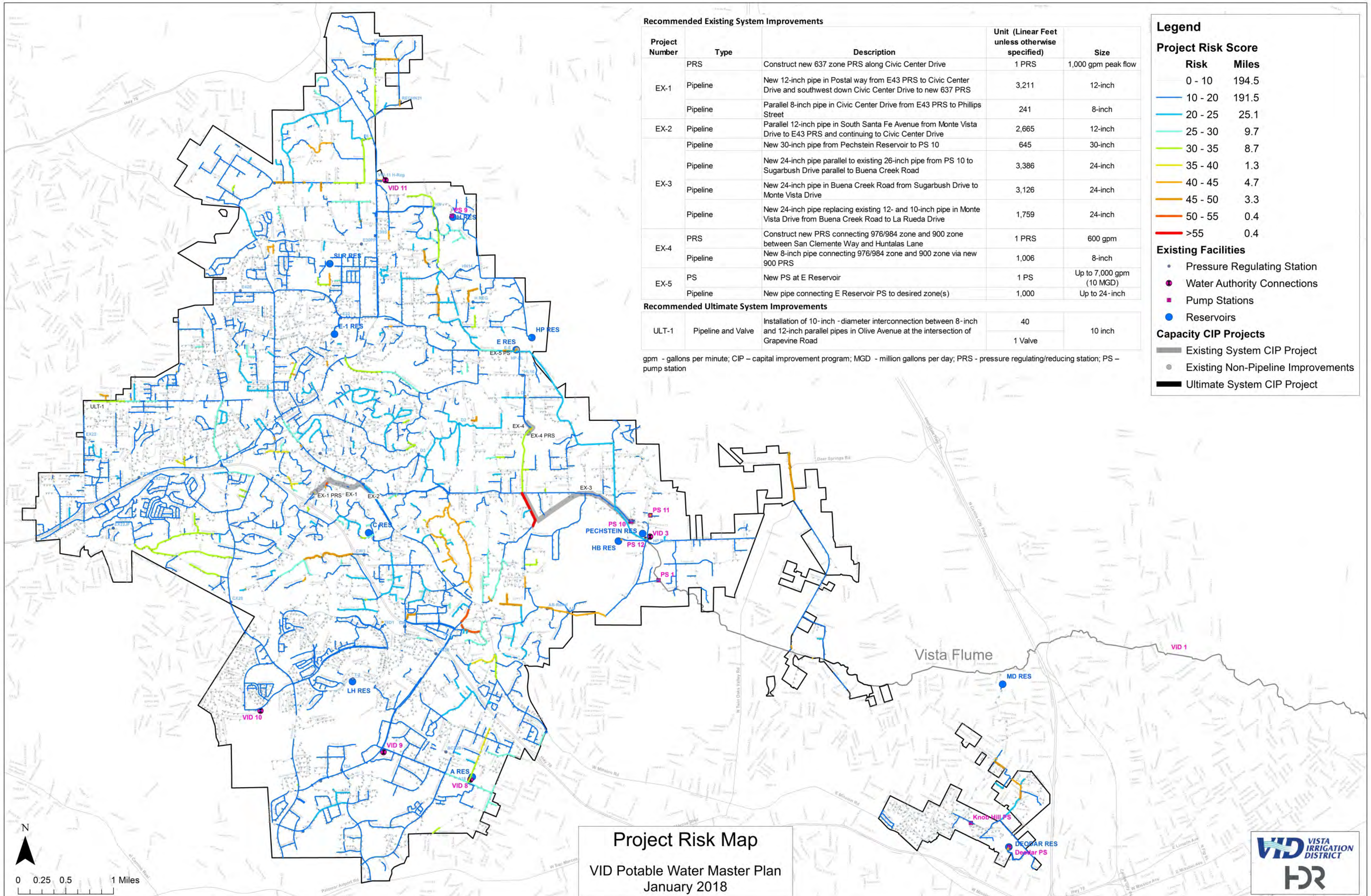
Table 3-18. Hydraulic Constraint Factor Scoring

Hydraulic Constraints	Miles	Factor Score	PRS Contribution
None	425.2	0	0
Ultimate Fireflow	0.6	10	3
Existing Fireflow	11.9	25	7.5
ULT-1	0.5	50	15
ULT-2	0.4	50	15
EX-1	0.0	100	30
EX-2	0.1	100	30
EX-3	0.4	100	30

3.3 Results

Figure 3-13 and Appendix C shows the result of the risk analysis on a red to blue scale where red is high risk pipes and blue is low risk pipes. Recommended system improvements from the hydraulic analysis are also summarized. A large scale map is included in Appendix C.

Figure 3-13. Project Risk Map



Recommended Existing System Improvements

Project Number	Type	Description	Unit (Linear Feet unless otherwise specified)	Size
EX-1	PRS	Construct new 637 zone PRS along Civic Center Drive	1 PRS	1,000 gpm peak flow
	Pipeline	New 12-inch pipe in Postal way from E43 PRS to Civic Center Drive and southwest down Civic Center Drive to new 637 PRS	3,211	12-inch
	Pipeline	Parallel 8-inch pipe in Civic Center Drive from E43 PRS to Phillips Street	241	8-inch
EX-2	Pipeline	Parallel 12-inch pipe in South Santa Fe Avenue from Monte Vista Drive to E43 PRS and continuing to Civic Center Drive	2,665	12-inch
EX-3	Pipeline	New 30-inch pipe from Pechstein Reservoir to PS 10	645	30-inch
	Pipeline	New 24-inch pipe parallel to existing 26-inch pipe from PS 10 to Sugarbush Drive parallel to Buena Creek Road	3,386	24-inch
	Pipeline	New 24-inch pipe in Buena Creek Road from Sugarbush Drive to Monte Vista Drive	3,126	24-inch
	Pipeline	New 24-inch pipe replacing existing 12- and 10-inch pipe in Monte Vista Drive from Buena Creek Road to La Rueda Drive	1,759	24-inch
EX-4	PRS	Construct new PRS connecting 976/984 zone and 900 zone between San Clemente Way and Huntalas Lane	1 PRS	600 gpm
	Pipeline	New 8-inch pipe connecting 976/984 zone and 900 zone via new 900 PRS	1,006	8-inch
EX-5	PS	New PS at E Reservoir	1 PS	Up to 7,000 gpm (10 MGD)
	Pipeline	New pipe connecting E Reservoir PS to desired zone(s)	1,000	Up to 24-inch

Recommended Ultimate System Improvements

Project Number	Type	Description	Unit	Size
ULT-1	Pipeline and Valve	Installation of 10-inch -diameter interconnection between 8-inch and 12-inch parallel pipes in Olive Avenue at the intersection of Grapevine Road	40 1 Valve	10 inch

gpm - gallons per minute; CIP – capital improvement program; MGD - million gallons per day; PRS - pressure regulating/reducing station; PS – pump station

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4 Main Renewal Strategies

The traditional method of water main renewal entails constructing a parallel pipeline and abandoning the existing pipeline (i.e., open-trench replacement). The new pipeline can be larger, if needed, and constructed of one of several materials (generally PVC, ductile iron, or steel). Historically, the District has typically used the open-trench replacement method. However, water main rehabilitation is gaining in popularity, as it affords several advantages over traditional, open-trench replacement. Generally, community impacts are lower, projects are completed faster, and costs are sometimes less. This section evaluates the applicability of main renewal strategy alternatives to traditional, open-trench replacement.

Table 4-1 lists the common water main rehabilitation technologies. The rehabilitation techniques listed here are methods that have proven their effectiveness in water main rehabilitation. Many other techniques are promoted, but not all are effective, efficient, or durable. Method selection depends on many site-specific factors, including the structural integrity of the host pipe, the locations and numbers of valves, laterals, and connections, future system plans, cost, and the owner's preferences. A more detailed discussion of the water main rehabilitation process is included in Appendix D. All materials in contact with water should be tested and certified in accordance with ANSI/NSF 61 requirements.

Table 4-1. Common Water Main Renewal Methods

Description	Advantages	Limitations
Cement mortar lining, spray-applied, in situ (ANSI/AWWA Standard C602)	• Low cost • Time-tested protection against internal corrosion • Service reconnection not required	• "Non-structural"—not recommended if pipe is structurally deficient • Not recommended where water is soft
Polymer lining, 1 mm thick (epoxy, polyurethane, or polyurea), spray-applied, in-situ (ANSI/AWWA Standard C620)	• Low cost • Time-tested protection against internal corrosion • Service reconnection not required • Rapid set-up of some linings may allow same-day return to service (avoiding bypass system costs)	• "Non-structural"—not recommended if pipe is structurally deficient [1]
Polymer lining, 3 to 8 mm thick (epoxy, polyurethane, or polyurea), spray-applied, in-situ	• Moderate cost • "Semi-structural"—proven ability to span holes and gaps. • Service reconnection not required • Rapid set-up of some linings may allow same-day return to service (avoiding bypass system costs)	• Not likely to survive fracturing of the pipe • Ability to serve as fully structural system has not been confirmed
Cured-in-place pipe lining, reinforced with fiberglass, polyester or carbon fibers	• Fully or semi-structural • May be capable of surviving pipe fracture [2] • Robotic service restoration is possible in many cases	• More costly than spray-applied linings • Service reconnections are required, but many can be performed by in-pipe robot • Long-term performance of some products not proven

Table 4-1. Common Water Main Renewal Methods

Description	Advantages	Limitations
Tight-fit HDPE slip lining, using roll-down, swage, or deformed methods	• Semi- or fully structural • Capable of surviving pipe fracture • Design criteria and properties are well established	• More costly than spray-applied linings • Service reconnections are required • Limited wall thicknesses available
Pipe bursting replacement	• Fully structural • Some upsizing possible • Design criteria and properties are well established • Compared to tight-fit lining, pipe materials should be more easily procured (less critical sizing requirements and different materials can be used)	• More costly than most other methods, although competitive market exists (not proprietary) • Service reconnections are required • Long-running cracks have occurred with fused PVC, but HDPE is very crack resistant
Cathodic Protection Retrofit	• Can economically extend the lives of water mains • Low-dig methods are available, using vacuum excavation and “keyhole” tools • Can be used in conjunction with in-pipe NDE to target corroded pipe	• Where mains are electrically discontinuous, protection is limited

[1] Testing will soon be conducted at the Trenchless Technology Center of Louisiana Tech University.
[2] Per testing performed at the Trenchless Technology Center of Louisiana Tech University.

Currently, CIPP and pipe bursting are the most common rehabilitation methods. A more detailed discussion of these methods are included in Appendix E and F respectively. While cured-in-place pipe lining systems have been around for many decades, the vast majority have been used for non-pressure pipe applications. As such, the life expectancy of lined water mains is less certain than for main replacement which adds additional uncertainty into any cost-benefit analysis.

Table 4-2 summarizes the District’s pipeline infrastructure by material type and diameter range. In general, PVC pipe is not in need of renewal in the near term. Rehabilitation of asbestos cement pipe is more challenging due to regulatory issues with pipe bursting (see Appendix F) and the unknown ability of CIPP to withstand fracture which is a common failure mechanism for asbestos cement pipe (see Appendix E). In metallic pipes, alternative rehabilitation methods are most cost effective where cost effective direct condition assessment data can be collected. Typically, this occurs when pipes can be proactively assessed without excavation through a hydrant (pipe 6-inches or 8-inches) or in large, high consequence of failure pipes where constructing access can be justified. This constitutes approximately 10% of the overall system which doesn’t offer the economies of scale typically required to implement a cost effective alternative main renewal program.

Table 4-2. Miles of Active Pipe by Material and Diameter

Material	Diameter						Total Miles
	4-inch or less	6-inch	8-inch	10 to 12 inch	14 to 16 inch	18 to 36 inches	
AC	20	84	106	43	10	7	269
PVC	6	12	47	33	1	1	100
Metallic	8	17	7	9	5	20	65
Other	2	0	0	-	-	-	2
Concrete	-	-	-	-	0	0	0
Total Miles	**35**	**113**	**159**	**85**	**16**	**28**	**437**

Historically, the District has typically used the open-trench replacement method. Based on regulatory challenges, useful life extension uncertainty, additional research needed, and limited economies of scale; it is recommended that the District continue to use open-trench replacement as the primary renewal method. However, the viability of alternative renewal solutions should be evaluated on a project specific basis, particularly where the integrity of the host pipe can be cost effectively determined and site-specific factors lend themselves to alternative renewal solutions.

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5 Continuous Improvement Recommendations

This section documents recommendations for continuous improvement in managing aging distribution infrastructure

1. This study developed new data that may be useful for managing aging infrastructure (e.g., pipe grouping, shrink swell potential, soil corrosivity, pressure, risk scores). Consider which data should be migrated into the District's database of record and perform that work.
2. Develop and implement a process to continuously update the District GIS break database with new records. The break database should include the unique asset identifier.
3. Figure 5-1 and Figure 5-2 summarize the number of breaks in the District's GIS and the number of breaks reported to the State in the annual report for Main breaks and other breaks (e.g., services, valves, hydrants) respectively. In general there is good correlation between the reported values but there are discrepancies which can expose the District to increased regulatory risk in the future. Develop and implement processes that will ensure that the number of breaks annually reported to the State align with the District's break database of record (GIS).

Figure 5-1. Main Break Comparison between Annual State Report and District Database of Record

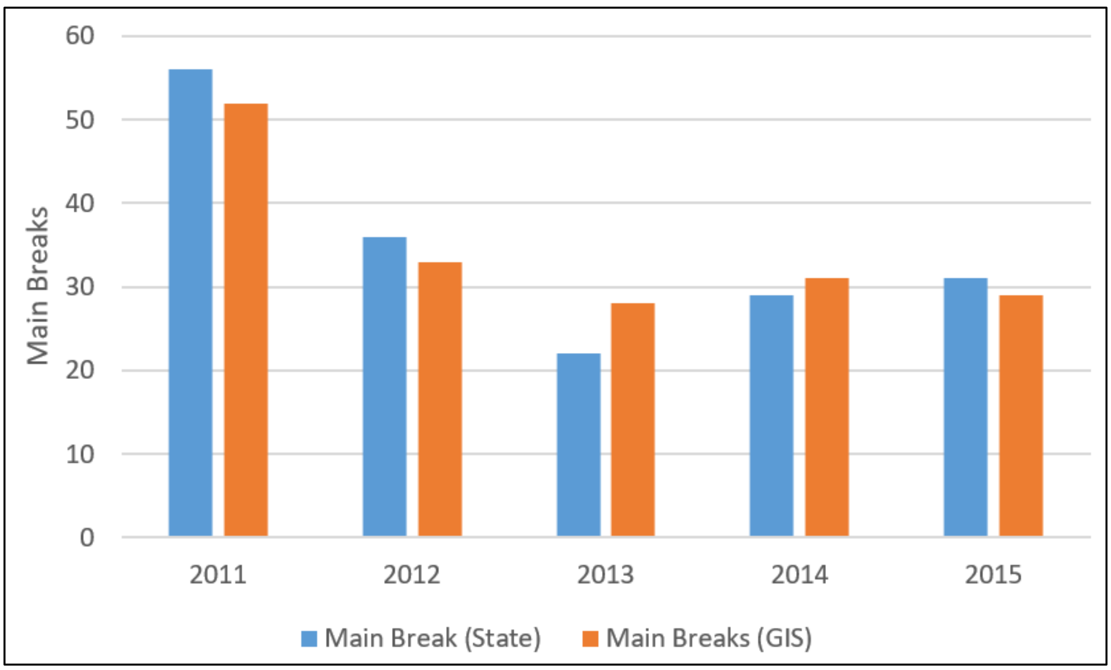
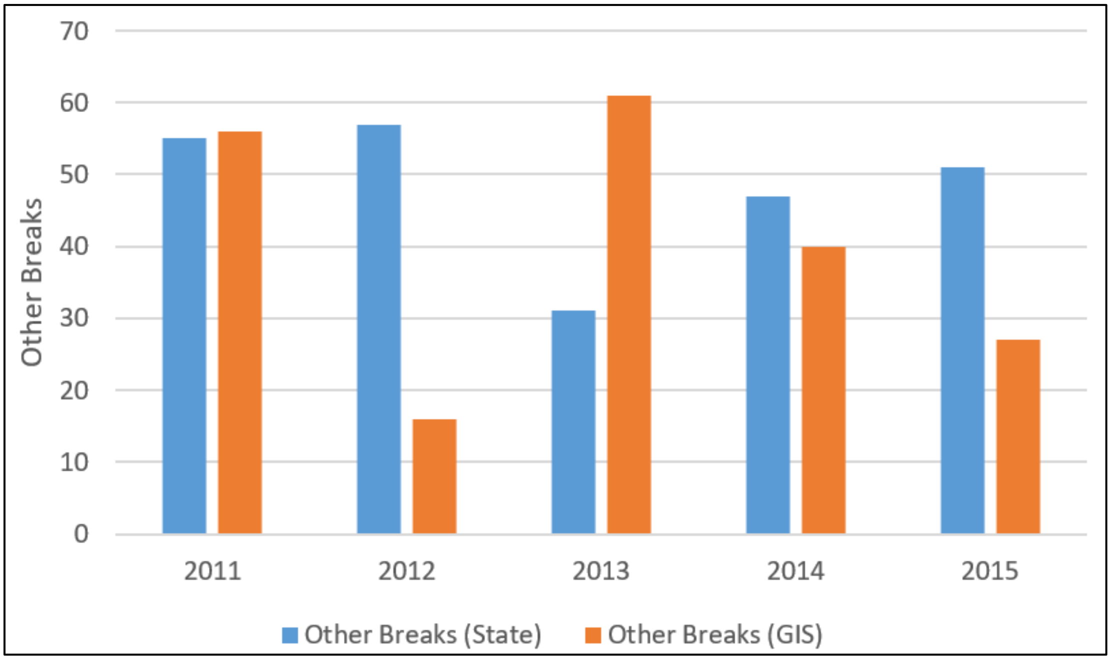


Figure 5-2. Other Break Comparison between Annual State Report and District Database of Record



4. When a pipe is exposed (e.g., break response, new tap installation, pipe renewal, and appurtenance renewal), it provides a unique opportunity to cost effectively gather condition assessment data (e.g., photos, soil samples, pipe samples, failure characteristics) that can be crucial in making effective pipe management decisions. Develop and implement an Opportunity Condition Assessment Program.
5. Develop and implement a proactive condition assessment program for pipes with elevated consequence of failure that should not be run to failure.
6. Determine which analyses in this report should be updated as new data becomes available. Determine who will update the analysis, how it will be done, and the frequency of analysis update which may vary. For example, it may be prudent to update the investment level sizing analysis once every five years while the renewal identification and prioritization analysis may need to be updated annually.
7. Consider incorporating service breaks and inoperable valves into the renewal identification and prioritization analysis.
8. Consider using insulating bushings when making a repair at the interface of a copper service and a metallic main to limit corrosion.
9. Communicate to the Board that eliminating breaks is not feasible. Communicate District's service levels in the context of national, regional, and local benchmarks.
10. Currently, services are typically only replaced as part of a main replacement project if the material is poly. In some cases, copper services will be replaced if they can easily be moved into the right-of-way. While copper services typically perform better, copper services with historic breaks will likely recur in the future. Consider developing a decision making guideline for when to replace non-poly services when

the main is being replaced. Update internal performance metrics to account for services replaced.

11. Monitor all existing pipeline test stations by performing an annual potential survey and compare to previous years' data if available.

Appendix A. Re-associated Main Breaks

The table below lists the main breaks that were re-associated to a different pipe based on a manual review of the comments. The Leak ID and Facility ID provide unique numbers to update District records. Note, where the Facility ID is 0, it is believed the break is associated with a pipe that was abandoned but is not captured in GIS.

Facility ID	Leak ID
83946	172
82394	446
87385	666
99257	1434
0	2009
0	2010
92852	2133
91641	237
84338	305
86144	520
87800	695
0	790
84338	852
0	1933
0	2053
86952	2085
87401	2138
91965	2153
80060	184
0	422
92694	637
0	665
76087	737
91131	828

Facility ID	Leak ID
0	865
79680	1051
79020	1116
76517	1119
81348	1133
0	1205
79456	1261
92644	1596
79456	1597
76087	1647
0	1800
78104	1976
0	2014
0	2015
76484	2037
0	2116
79689	2118
92475	2147
78916	2185
84370	115
83325	829
93045	1662

Appendix B. INSTALLNUM Added

Some pipe did not have an INSTALLNUM. For those pipes with more than two breaks (~2.25 mi), a manual review of install year, material, diameter, and spatial location was performed to approximate the extent of a project. Five projects were generated (HDR1, HDR2, HDR3, HDR4, and HDR5). A list of pipes by Facility ID and new INSTALLNUM is included below.

Facility ID	Install Number
75296	HDR1
75686	HDR1
75977	HDR1
85558	HDR3
85609	HDR4
85611	HDR4
85691	HDR4
85850	HDR4
76087	HDR5
76134	HDR5
76218	HDR5
76219	HDR5
76232	HDR5
76279	HDR5
85604	HDR4
91229	HDR2
98479	HDR3

Appendix C. Project Risk Map

Recommended Existing System Improvements

Project Number	Type	Description	Unit (Linear Feet unless otherwise specified)	Size
EX-1	PRS	Construct new 637 zone PRS along Civic Center Drive	1 PRS	1,000 gpm peak flow
	Pipeline	New 12-inch pipe in Postal way from E43 PRS to Civic Center Drive and southwest down Civic Center Drive to new 637 PRS	3,211	12-inch
	Pipeline	Parallel 8-inch pipe in Civic Center Drive from E43 PRS to Phillips Street	241	8-inch
EX-2	Pipeline	Parallel 12-inch pipe in South Santa Fe Avenue from Monte Vista Drive to E43 PRS and continuing to Civic Center Drive	2,665	12-inch
EX-3	Pipeline	New 30-inch pipe from Pechstein Reservoir to PS 10	645	30-inch
	Pipeline	New 24-inch pipe parallel to existing 26-inch pipe from PS 10 to Sugarbush Drive parallel to Buena Creek Road	3,386	24-inch
	Pipeline	New 24-inch pipe in Buena Creek Road from Sugarbush Drive to Monte Vista Drive	3,126	24-inch
	Pipeline	New 24-inch pipe replacing existing 12- and 10-inch pipe in Monte Vista Drive from Buena Creek Road to La Rueda Drive	1,759	24-inch
EX-4	PRS	Construct new PRS connecting 976/984 zone and 900 zone between San Clemente Way and Huntalas Lane	1 PRS	600 gpm
	Pipeline	New 8-inch pipe connecting 976/984 zone and 900 zone via new 900 PRS	1,006	8-inch
EX-5	PS	New PS at E Reservoir	1 PS	Up to 7,000 gpm (10 MGD)
	Pipeline	New pipe connecting E Reservoir PS to desired zone(s)	1,000	Up to 24-inch

Recommended Ultimate System Improvements

Project Number	Type	Description	Unit	Size
ULT-1	Pipeline and Valve	Installation of 10-inch -diameter interconnection between 8-inch and 12-inch parallel pipes in Olive Avenue at the intersection of Grapevine Road	40	10 inch
			1 Valve	

gpm - gallons per minute; CIP – capital improvement program; MGD - million gallons per day; PRS - pressure regulating/reducing station; PS – pump station

Legend

Project Risk Score

Risk	Miles
0 - 10	194.5
10 - 20	191.5
20 - 25	25.1
25 - 30	9.7
30 - 35	8.7
35 - 40	1.3
40 - 45	4.7
45 - 50	3.3
50 - 55	0.4
>55	0.4

Existing Facilities

- Pressure Regulating Station
- Water Authority Connections
- Pump Stations
- Reservoirs

Capacity CIP Projects

- Existing System CIP Project
- Existing Non-Pipeline Improvements
- Ultimate System CIP Project

Project Risk Map

VID Potable Water Master Plan
January 2018

VISTA IRRIGATION DISTRICT
HDR

Appendix D. How Water Mains are Rehabilitated

Because water mains are not equipped with manholes and are pressurized, their rehabilitation involves removing them from service and digging holes to gain access. Following excavation, a portion of the pipe is removed so the interior can be accessed. Often, these excavations occur at elbows, tees, crosses or valves, enabling lining to proceed in more than one direction from one excavation.

The pipe is cleaned using a variety of processes. The amount of cleaning effort depends upon the amount of scale and sediment. Unlined cast iron pipes generally require the use of scrapers or other mechanical cleaning methods, whereas AC pipes and mortar-lined pipes may often be cleaned with swabs and squeegees alone. The picture below shows an access hole, with a drag scraper and squeegee about to be pulled into the pipe. A steel plate lies on the pavement. Such plates are used to cover holes when construction is not underway. Along the curb in the background is bypass piping, used for keeping customers supplied while the rehabilitation process is underway.

Following cleaning, the main is lined using various methods. Spray-applied linings function as Class I and Class II linings. CIPP linings, as noted in Appendix E (and discussed below) function as Class III and perhaps Class IV linings. There are also methods of inserting a close-fitting HDPE pipe, which can function as a Class III or Class IV lining. More details regarding these methods can be found in Appendix A of WRF Report 4473, which can be downloaded here: http://www.waterrf.org/Pages/Projects.aspx?PID=4473. The pipe bursting method of main replacement can also be employed, as described below.

For most rehabilitation projects, keeping customers supplied is a necessity. This is typically done using temporary pipes laid in gutters on each side of the street as shown in the picture below. The temporary pipes are generally 2 to 4 inches in diameter and are supplied from a fire hydrant, but the pipes can range up to 12-inches. Sometimes a tap or connection to an adjacent main is required.

Short pieces of hose are used to connect this “bypass{ XE "bypass" }” pipe to the service pipes at the meter. To make these connections at the meter, the meters must be removed, and are either reinstalled laying on the ground, or are simply removed completely, and the customer’s water use is estimated for the duration of the project. Where the bypass pipe crosses driveways, cold asphalt mix is mounded over the pipe to permit vehicle passage or preformed rubber ramps are used, as shown in the picture above.

Rehabilitation contractors often have crews that specialize in installation and removal of these bypass piping systems, and the work can be a project in itself. Among the details to be addressed:

- Assuring adequate disinfection{ XE "disinfection" }, bacterial testing{ XE "bacterial testing" }, and flushing.
- Sizing the pipe to serve large customers or to replace large mains. Where bypass piping exceeds 4 inches in diameter, trenching is required where the pipe crosses driveways and alleys.
- Assuring customers are supplied from the correct gradient zone, where two mains exist in the street.

- Assuring that an adequate number of hydrants remain in service, and that they are adequately supplied.
- Avoiding undue hazards to vehicles, cyclists, and pedestrians from the pipes and hoses on the ground.
- Keeping the water from getting too hot in the summer (customers complain).

Because the work involved in constructing, maintaining, and removing bypass systems is considerable, substantial savings can result if bypassing is avoided. Cleaning, lining, and returning a main to service within a workday is certainly possible from a process standpoint. Those rehabilitation systems which don't require extended cure times are usually capable of achieving this goal, provided that work is well planned. The hurdle is being able to *safely* place the main in service, without super chlorination and bacterial testing, and gaining permission from health department authorities to do so. This requires coordination and cooperation between the utility, the rehabilitation contractor, health officials, and customers.[3]

In the UK, same-day return to service after spray-on polymer lining has become routine, and health officials there are now so confident in the processes, that confirmation bacterial testing is not generally required. Same-day return to service is now also common in parts of Canada, and was recently demonstrated in the U.S. for spray-applied polymer lining. Same-day return to service has also been accomplished with pipe bursting in various utilities in the U.S., where pre-chlorinated pipes have been used. However, same-day return to service is generally not applicable for CIPP linings, because the cure times needed for the resins are too long. For CIPP lining of most mains, a bypass system is needed.

Open-trench construction is well known and its impacts are well understood. Even on a quiet residential street, considerable disruption occurs, generally for several weeks. A rehabilitation project is markedly less disruptive, with small, isolated excavations and less space occupied by equipment, materials, and spoils. On busy streets, excavations are plated during busy times, with work proceeding when traffic subsides.

Impacts for rehabilitation projects vary, depending primarily on the number of excavations and the speed at which work is accomplished. By avoiding bypass piping, as described above, impacts can be lessened. Methods that don't require excavations for service reinstatements (as described below) also reduce impacts.

Spray-applied linings have the advantage that little to no effort is needed to re-establish the service connections. For cement mortar lining, a small blast of air down the lateral, before the lining sets up, clears the opening. For polymer linings, the openings are seldom blocked.

[3] Some of the lining methods are inherently sanitary, and risks should be minimal, provided that strict work procedures are followed. In the UK and in Canada, the ability to line pipe without jeopardizing health has been demonstrated. In the US, bypass piping could be avoided by distributing bottled water to and issuing a "boil water advisory" until experience and confidence in the process is gained. A US project utilizing same-day return to service was recently completed in Oswego, NY (Folgherait, Rogers, and Kirsch, 2013).

Some of the CIPP companies have developed in-pipe robotic tools that are capable of finding many of the laterals and re-establishing the openings. Because the laterals are small, this is a more difficult task than in CIPP lining of wastewater sewers. Generally, excavations are still required at some of the services, but success in doing the work robotically has been steadily improving. Currently, Sanexen claims 90 to 100 percent of services are reinstated robotically.

When using any method that is intended to be structural or semi-structural, re-establishing the lateral opening alone is not sufficient. There has to be a positive (leak free) connection between the lateral and the lining, otherwise water leakage into the annulus between the pipe and lining will result in equal pressures on both sides of the lining, and the negation of any structural benefit. Sanexen accomplishes this with their Aquapipe product by using epoxy resin to bond the liner to the pipe at the corporation stop and possibly to the ferrule, if it protrudes. However, such adhesion of the lining to the pipe also reduces the lining’s ability to survive host pipe fracture, which raises concerns.

Appendix E. Cured-in-Place Pipe Lining of Water Mains

Cured-in-place pipe (CIPP) lining is a common, well-established technique and its popularity for water mains has been quickly growing. Traditionally, CIPP was used primarily in the wastewater industry to rehabilitate non-pressurized pipe. In the CIPP method, a collapsed, resin-impregnated fabric tube is inserted into the host pipe, expanded, and then cured using steam, hot water, or UV light. The fully cured material forms a plastic pipe that fits tightly in the existing pipe. The photo to the right shows a resin-impregnated fabric tube being pulled into a water main in the East Bay Municipal Utility District.

There are two methods for inserting the fabric tube insertion:

1. Inversion involves pushing the fabric tube down the pipe using water or air, while the tube is turned inside out. Only one access point is required.
2. Direct pulling of the liner into the main. Two access points are required.

Because CIPP linings fit snugly and have relatively smooth surfaces, the loss of hydraulic capacity is usually minor (if any). If the lining is used in unlined cast iron pipe, where heavy scales exist, capacity will be increased. However, in some pipes, a loss of capacity may occur. For mortar-lined iron and AC pipes that are 8-inches or smaller, the potential loss of capacity should be evaluated before employing CIPP lining.

AWWA's Manual M28, "Rehabilitation of Water Mains" describes four classes of water main rehabilitation, ranging from non-structural to fully-structural. Class I, non-structural linings, provide corrosion protection to the inner surface and improve flows and water quality. Class II and III, semi-structural linings, additionally span over holes, gaps and other small weaknesses in the main, but still rely on the existing "host" pipe for some strength. Class IV, fully-structural, linings are intended to provide roughly the equivalent of a new pipe, without significant reliance on the host pipe.

The current definitions of these classifications are open to a great deal of interpretation, and the result has been that some linings have claimed to be Class IV, when in fact they are not. In the specific case of CIPP linings, it has not been well established which products are truly Class IV.

Table E1 describes the requirements for Class IV linings, and why uncertainty exists. This table is based on how the new standard will be written.

Table E1. Caption Requirements for Class IV Linings

Lining requirement	Why this is important	Why uncertainty exists
Ability to Resist Hoop Stresses on a Long-Term Basis	This demonstrates the ability to withstand sustained internal pipe pressure, without aid from the host pipe	CIPP products have not generally undergone the long-term testing needed to require long-term strengths. CIPP designs are usually based on the assumption that long-term strength is 50% of short-term strength, but this may not be true. As a fiber-reinforced plastic material, CIPP linings are difficult to test. Because variations in composition could affect test results, many expensive tests would be needed.
Ability to create a water-tight envelope	Leakage of water into the annulus between the host pipe and the lining negates nearly all the value of the lining. If leakage occurs, the lining neither holds pressure nor provides corrosion protection to the host pipe.	For CIPP and spray-applied linings, leakage is typically prevented by bonding the lining to the host pipe. The integrity of this bonding has not been investigated very well, particularly for real water mains lined in-situ. Moreover, a well-adhered lining is likely to fail when the host pipe fractures, as discussed below (and in Appendix A).
Ability to survive failure of the host pipe	If the lining tears or breaks when the host pipe cracks, it has limited value as a "structural" lining.	If a lining is well bonded to the host pipe, it experiences extremely high strains when a crack opens in the host pipe. Even if the adhesion is poor, water pressure in the pipe creates a frictional bond that may cause the lining to tear or crack.

The last requirement was not well understood until several years after publication of Water Research Foundation Report 4095, "Global Review of Structural Spray-on Lining Technologies" (Ellison, et al., 2010), which first published a method to test this criterion.

Table E2 comes from another WRF study (Ellison, et al, 2015), showing the applicability of various lining systems. Highlighted in yellow are the criteria important to CIPP linings. Note that CIPP linings are shown as both Class III and Class IV, reflecting uncertainty regarding their capabilities.

Table E2. Capabilities and Limitations of Current Rehab Technologies

	Class I Linings	Class II/III Linings	Class IV Rehab	Comments
Structural Capabilities	Some hole spanning, but considered "non-structural"	Spanning of weak areas in the host pipe (holes and gaps)	Fully structural, considered equal to constructing a new main	Refer to Manual M28 for details regarding methods
Applicability	Unlined cast iron mains (pre-1940) with little external corrosion	Mains susceptible to rust holes and leaks at joints	Deteriorated mains at risk of fracturing	Class III linings may also be applied to mains at risk of circumferential (beam) breaks, but not longitudinal splits
Adhesion and Coverage Criteria	Full lining coverage with bond to interior surfaces	Class II – full coverage and bond Class III – Water-tight envelope needed	Water-tight envelope required	Water-tight envelope requires positive connections to each lateral and at all other discontinuities

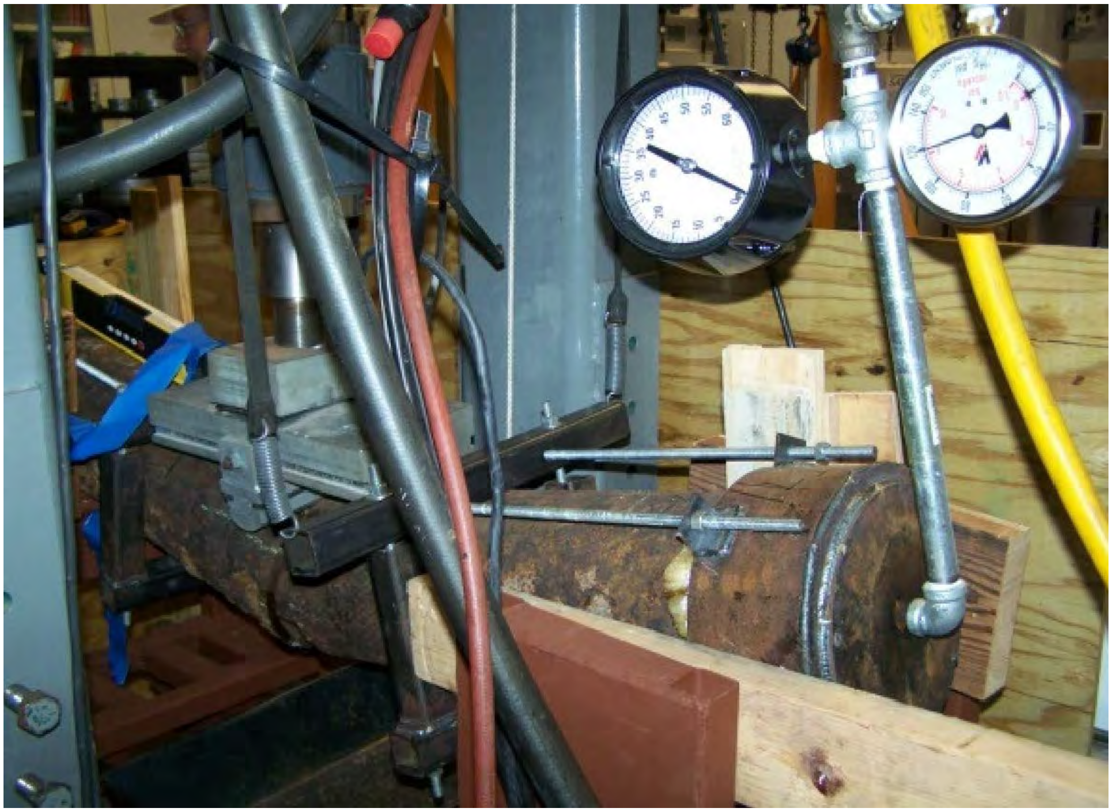
Table E2. Capabilities and Limitations of Current Rehab Technologies

	Class I Linings	Class II/III Linings	Class IV Rehab	Comments
Tear resistance upon host pipe fracture	Lining will tear where pipe fractures	Class II – tearing of lining expected at fractures Class III - may resist tearing if not bonded to host pipe	Tear resistance is essential to be considered fully structural	Tear-resistance testing is recommended for large-scale programs, using samples extracted from owner's system
Design	AWWA Standards: cement mortar (ANSI/AWWA 602) epoxy lining (ANSI/AWWA C620)	Design for maximum sized hole/gap and maximum pressure, using long-term material properties	Design for maximum operating pressure, maximum test pressure, and expected surge pressures	Apply parameters derived from other AWWA standards, particularly regarding long-term strength and applicable safety factors
Linings and Methods Commonly Available	Cement mortar lining Thin (1mm) linings of spray-applied polymers: • Epoxy • Polyurethane • Polyurea	Class II: Thick (3mm and above) of spray-applied epoxy and polyurea Class III: • Reinforced cured-in-place pipe linings • Tight-fit HDPE slip linings	Reinforced cured-in-place pipe linings (tested for tear resistance) HDPE and PVC pipes installed through pipe bursting, slip lining, or tight-fit slip lining	Also to be considered: cathodic protection retrofits Spot repairs
Advantages	Long history of use Reconnection of services is usually not required Non-proprietary	Class II is an upgrade of Class I, with similar advantages CIPP has a well-developed market, with multiple suppliers and in-pipe robotics	There are many pipe bursting options: sizes, materials, contractors. Installation of HDPE through pipe bursting and slip lining provides proven performance.	
Disadvantages	Should not be used where extensive external corrosion is evident	Some manufacturer claims about structural benefits have not been validated	Some manufacturer claims about structural benefits have not been validated	

Sanexen, the manufacturer of Aquapipe CIPP lining has conducted a laboratory test to assess the lining's ability to withstand host pipe failure. The photo below shows a reinforced CIPP liner being tested which appears to demonstrate the ability to survive the fracturing of the host pipe while withstanding 120psi of internal pressure without leakage. However, there are two concerns regarding the test which was performed:

1. The tests were not performed on a sample lined in place; and
2. According to the professor who oversaw the testing, special measures were taken to prevent the lining from bonding to the host pipe in the vicinity of the simulated fracture.

More testing is needed to substantiate the structural value of the lining.



Very recently, health concerns have been expressed about fumes produced by CIPP lining. These concerns apply mostly to the workers installing the lining and perhaps to passerbys who are briefly exposed. However, the styrene-based resins that are the source of these concerns are not used in potable water mains. If CIPP lining is used in the District, applicable specification language needs to be included that addresses the issue of worker and passerby exposure to fumes. Additionally, like other materials used in water mains, CIPP linings must be certified per NSF61.

The major difference between pressure-pipe CIPP and traditional CIPP is the fabric tube that is used. For pressure pipes, a woven jacket made from polyester, fiberglass, Kevlar® or carbon fibers is used instead of simple felt material. The types and amount of fabric reinforcement are determined by liner loading requirements. Pressure pipe CIPP liners are commonly available in pressures up to 150 psi, and can be custom-designed for higher pressures. Three companies provide CIPP lining for water mains, often through licensed-local contractors.

An advantage of CIPP lining vs. other Class III and Class IV methods, is that reinstatement of the services (as discussed earlier) can often be accomplished robotically, without the need for additional excavations.

While cured-in-place pipe lining of sewers is offered by dozens of contractors and many suppliers, CIPP for potable water systems is much more limited. In addition to Aquapipe, Insituform Blue and NordiPipe are products that have appropriate drinking-water certifications per NSF61. Both Aquapipe and Instituform operate their own crews in Southern California, while the NordiPipe lining is offered by contractors licensed by the manufacturer. Aquapipe also licenses its product to some installers. Insituform generally does their own installations.

Often, the CIPP contractor does not install the bypass piping or perform the excavations needed for pipe access. The lining contractor's work may be limited to cleaning and lining the main, and re-establishing the services. A pipeline general contractor (or the utility owner) does the other work.

While cured-in-place pipe lining systems have been around for many decades, the vast majority have been used for non-pressure pipe applications. As such, the life expectancy of lined water mains is less certain than for main replacement which adds additional uncertainty into any cost-benefit analysis. According to most manufacturers, CIPP systems are designed for 50-year lives, but they could last longer, since the host pipe does in fact provide loading assistance. While no material is perfect, and no pipe is constructed without defects, the number of leaks and repairs on a conservatively-designed, well-constructed new water main, made with modern materials, should stay within reasonable limits for well over 100 years.

Appendix F. Pipe Bursting

Pipe bursting looks very similar to slip lining (i.e., inserting a smaller pipe inside a bigger one), except the new pipe is not necessarily smaller. In fact the new pipe can be slightly larger than the one being replaced. This is accomplished by fracturing or splitting the old pipe as the new pipe is installed. The pipe fragments are pushed outward into the soil, creating a sufficient opening for the new pipe. High-density polyurethane (HDPE) is the material that is generally recommended for pipe busting installations, due to its flexibility and ductility, but several other materials have been used successfully.

In the last decade, the use of pipe bursting for general water main replacement has grown tremendously, as more contractors have gained experience and more owners have seen its effectiveness and cost benefits. As an example, WaterOne, the utility that serves several communities in the Kansas City suburbs, decided to try pipe bursting for routine water main replacement, using their own construction crews. The utility hoped that pipe bursting would produce cost saving of about 15 percent, by reducing the amount of repaving that would be required. In reality, the cost savings approached 25 percent, because more work could be completed each day (Ellison, 2014). Consolidated Mutual Water Company, which serves Lakewood, Colorado and several other communities, reports even greater savings using pipe bursting as its primary replacement method.

At this point, pipe bursting, pipe reaming, and similar methods involving the destruction and disposal of the pipe in place are not recommended for AC pipe, due to regulatory issues. The EPA has addressed replacement of AC pipe using the pipe bursting method. In a letter issued July 17, 1991, the EPA stated its position that “the crushing of asbestos cement pipe with mechanical equipment would cause this material to become ‘regulated asbestos containing material’ (RACM)” and “. . . the crushed asbestos cement pipe in place would cause these locations to be considered active waste disposal sites and therefore, subject to the requirements of §61.154 (NESHAP).” Furthermore, in this same letter, the EPA goes on to advise that “[i]n order to avoid the creation of a waste disposal site which is subject to the Asbestos NESHAP, the owners or operators of the pipe may want to consider other options for dealing with the abandoned pipe.”

While not technically “illegal,” pipe bursting of AC pipe thus creates an active hazardous waste site that, after one year of no further construction activity, becomes an inactive hazardous waste site. If the owner of the pipe location (i.e., the street) is willing to designate the area as a hazardous waste site and perform the required testing, monitoring, and reporting (and potential cleanup), pipe bursting is feasible. The City of Casselberry, Florida, has performed AC pipe bursting projects with EPA acknowledgement and acquiescence. Concerns that property owners will object to having a hazardous waste site adjacent to their properties have made other municipalities more reticent to use pipe bursting for AC pipe.

A recent project co-funded by the EPA and the Water Research Foundation investigated whether pipe bursting creates a hazardous situation that merits regulation (Matthews and Stone, 2013), and was not able to detect fiber releases into the air. This study could be a first step in what would be a long process of deregulating the use of the pipe bursting for replacing AC pipe.

Because undamaged AC pipe is considered non-friable (the asbestos does not readily become airborne) and not RACM, abandoning intact AC pipe in a public right of way generally does not require special permission. Thus when replacing an AC main using open-trench construction, there generally is no reason to remove the pipe from the ground, unless needed to make room for the new pipe or to satisfy the requirements of whomever is granting the easement.

Appendix B. Reservoir Condition Assessment Technical Memorandum

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Reservoir Condition Assessment

Vista Irrigation District

October 26, 2017

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Table of Contents

Tables

Figures

Appendices

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Executive Summary

The Vista Irrigation District (VID) Reservoir Condition Assessment documents the existing condition of VID's reservoir sites and identifies recommended improvements for the civil site, corrosion and structural components of the reservoirs. Condition assessment inspections of 10 of VID's 12 potable water reservoirs were completed in November 2016. Two reservoirs, HP and E Reservoirs, were not inspected. HP Reservoir was out of service, undergoing rehabilitation due to corroded and failing prestressed wire wrap. E Reservoir was in service, but did not require inspection since it is scheduled for replacement. Reservoir construction dates range from 1925 through 1997. All reservoirs were in generally good condition and it was observable from the field inspections that operation and maintenance is periodically conducted by VID staff on the reservoirs maintaining them in good condition. Confined space entry of the reservoirs was not conducted; however visual inspection of the reservoir's interior from access hatches was attempted when it was deemed safe to do so.

The exterior inspections were intended to document the current condition of the civil site, corrosion, and structural aspects of the reservoirs. Field activities completed during these field visits included:

- Perimeter, site and drainage inspection
- Structural inspection
- Exterior coatings inspection
- Reservoir Climb and roof inspection
- Non entry, visual hatch inspection

The findings of the inspection of VID's reservoirs were used to recommend and prioritize improvements for the rehabilitation or replacement of reservoir equipment and identify any additional assessments required. The overall approach and detailed inspection with photographic documentation are included in this technical memorandum.

The HDR standardized Condition Assessment Ratings System (CARS) was utilized to guide the inspection team while conducting the reservoir inspections. CARS promotes consistency from site to site to facilitate proper prioritization of the reservoirs civil site, corrosion and structural aspects.

The criteria specified in the CARS are grouped into four categories as follows:

1. Structural
2. Site (non-reservoir)
3. Aesthetic (reservoir only)
4. Safety/Security

The civil/site and corrosion and structural recommendations listed for each reservoir address the deficiencies noted during the field inspections. The civil/site, corrosion, and structural recommendations pertain to ongoing monitoring, minor maintenance, and repair work. The recommendations for further investigation include potentially larger scale improvements and recommendations, such as interior cleaning and inspection or seismic evaluations. A detailed condition assessment for reservoir interior is recommended for seven of the ten reservoirs, roof replacement evaluation is recommended for three of those seven, and a seismic evaluation is recommended for all ten reservoirs.

Each criterion was scored on a scale or listed as Not Applicable. The scoring criteria are displayed in Table ES-1.

Table ES-1. Reservoir Condition Scoring Criteria

Score	Description	Phasing
0	No action required	--
1	Minor (7 plus years)	Long-Term
3	Moderate (2 to 6 years)	Mid-Term
5	Immediate (0 to 2 years)	Near-Term
N/A	Not Applicable	--

Each reservoir received a score for Civil/Site components, Civil/Corrosion components and Structural components. Each category of components was first normalized to a 100-point scale and then weighted based on potential risk. Site and civil/corrosion were weighted at 20 percent each and structural was weighted at 60 percent. Weighting the structural components at a higher value allowed for a more accurate prioritization of the projects to address safety and reliability concerns first.

The scoring components, rankings and recommendations for each inspected reservoir are provided in Table ES-2. Detailed recommendations are provided in Section 12. CIP costs were developed for all near-term, mid-term, and long-term costs in addition to costs associated with minor improvement and maintenance as well as recommended additional assessments. Table ES-3 displays the overall reservoir ranking and cost summary. Deodar and Pechstein reservoirs ranked first and second and H and San Luis Rey reservoirs ranked ninth and tenth. Overall total values were based on individual rankings for each civil, corrosion and structural category displayed in Section 12.

Sections 12.1 through 12.2 and Appendix A detail the rankings and recommendations for each reservoir. Section 12.3 and Appendix B detail the unit costs and total probable cost opinions for each reservoir.

Table ES-2. Reservoir Condition Findings and Recommendations

	Rank	1	2	3	4	5	6	7	8	9	10
	Reservoir	Deodar	Pechstein	A	HB	Lupine Hills	E1	MD	C	H	San Luis Rey
Recommended Improvements	Access Road	Mid	Long		Long	Long		Mid	Long	Near	
	Fences and Gates	Near		Near	Near			Near	Near		Mid
	Trees and Vegetation			Near		Mid	Near	Near		Near	
	Signage and Safety Signage	Long	Long	Near	Near		Near	Long	Long	Long	Near
	Drainage		Mid								
	Site Piping and Appurtenances			Near	Long						
	Roof Hatch	Near	Mid	Near	Mid	Mid	Mid	Mid	Long	Mid	Mid
	Roof	Near	Near	Near	Mid	Near	Mid	Mid	Mid	Mid	Long
	Handrails, Ladders, and Stairs	Long	Long		Near	Near	Near	Mid	Near	Mid	
	Hatches and Doors		Near		Mid						
	Overflow Pipe	Mid								Mid	
	Reservoir Exterior Wall	Mid	Mid	Long	Mid	Long	Long	Long	Long	Long	
	Vent		Near		Mid	Mid	Mid		Mid		Long
	Stability/ Geotechnical/ Foundation	Near			Mid			Near			
	Interior Structure		Near	Mid			Mid				Long
	Further Investigation	Near	Near	Near	Near	Near	Mid	Mid	Mid	Mid	Mid

● Near Term Improvements (0 to 2 years) ● Mid Term Improvements (2 to 6 years) ● Long Term Improvements (7 plus years)

Table ES-3 - Reservoirs Ranking and Cost Summaries

Tank Name	Overall Priority Score Total	Rank	Timeline	Total Probable Cost Opinions for Recommended Minor Improvements (Rounded) ($)	Recommended Additional Assessments	
					Type*	($)**
Deodar Reservoir	83.33	1	Near-Term	$78,000	S,I,R	$57,000
Pechstein Reservoir	76.41	2	Near-Term	$71,000	S,I,R	$81,000
A Reservoir	73.00	3	Near-Term	$52,000	S,I,R	$31,000
HB Reservoir	72.54	4	Near-Term	$86,000	S,I	$61,000
Lupine Hills Reservoir	64.08	5	Near-Term	$47,000	S,I	$61,000
E1 Reservoir	57.69	6	Mid-Term	$34,000	S	$10,000
MD Reservoir	57.18	7	Mid-Term	$23,000	S	$16,000
C Reservoir	51.95	8	Mid-Term	$36,000	S	$10,000
H Reservoir	51.03	9	Mid-Term	$55,000	S,I	$61,000
San Luis Rey Reservoir	18.99	10	Long-Term	$11,000	S,I	$61,000

*S = Seismic, I = Interior, R = Roof System
**Costs associated with roof systems are not included in the Recommended Additional Assessment costs. Roof system options and associated costs are provided in Appendix B.

1 Introduction

Vista Irrigation District (VID) contracted HDR Engineering (HDR) to conduct the condition assessment inspections of VID's potable water reservoir inventory. Inspections of a total of ten VID reservoirs were completed in November 2016; reservoir inspection schedule is displayed in Table 1-1. The locations of these reservoirs within the District are shown on Figure 1-1. Two of VID's reservoirs, HP and E Reservoirs were not inspected. HP Reservoir was out of service since it was drained for immediate rehabilitation due to corroded and failing prestressed wire wrap. E Reservoir was in service but did not require inspection since it is scheduled for replacement. Additional information on E Reservoirs replacement including sizing will be provided in the VID Master Plan. The inspections were intended to document the current condition of the civil site, corrosion, and structural aspects of the reservoirs from exterior inspections. Confined space entry of the reservoirs was not required under the scope of work; however visual inspection of the reservoir's interior from access hatches was attempted when it was deemed safe to do so.

Table 1-1. VID Reservoir Inspection Schedule

VID Reservoir Inspections		
Date	**No.**	**Reservoir**
11/14/2016	1	Lupine Hills Reservoir
	2	A Reservoir
11/15/2016	3	Pechstein Reservoir
	4	HB Reservoir
	5	C Reservoir
	6	E1 Reservoir
11/16/2016	7	San Luis Rey Reservoir
	8	H Reservoir
	9	MD Reservoir
	10	Deodar Reservoir

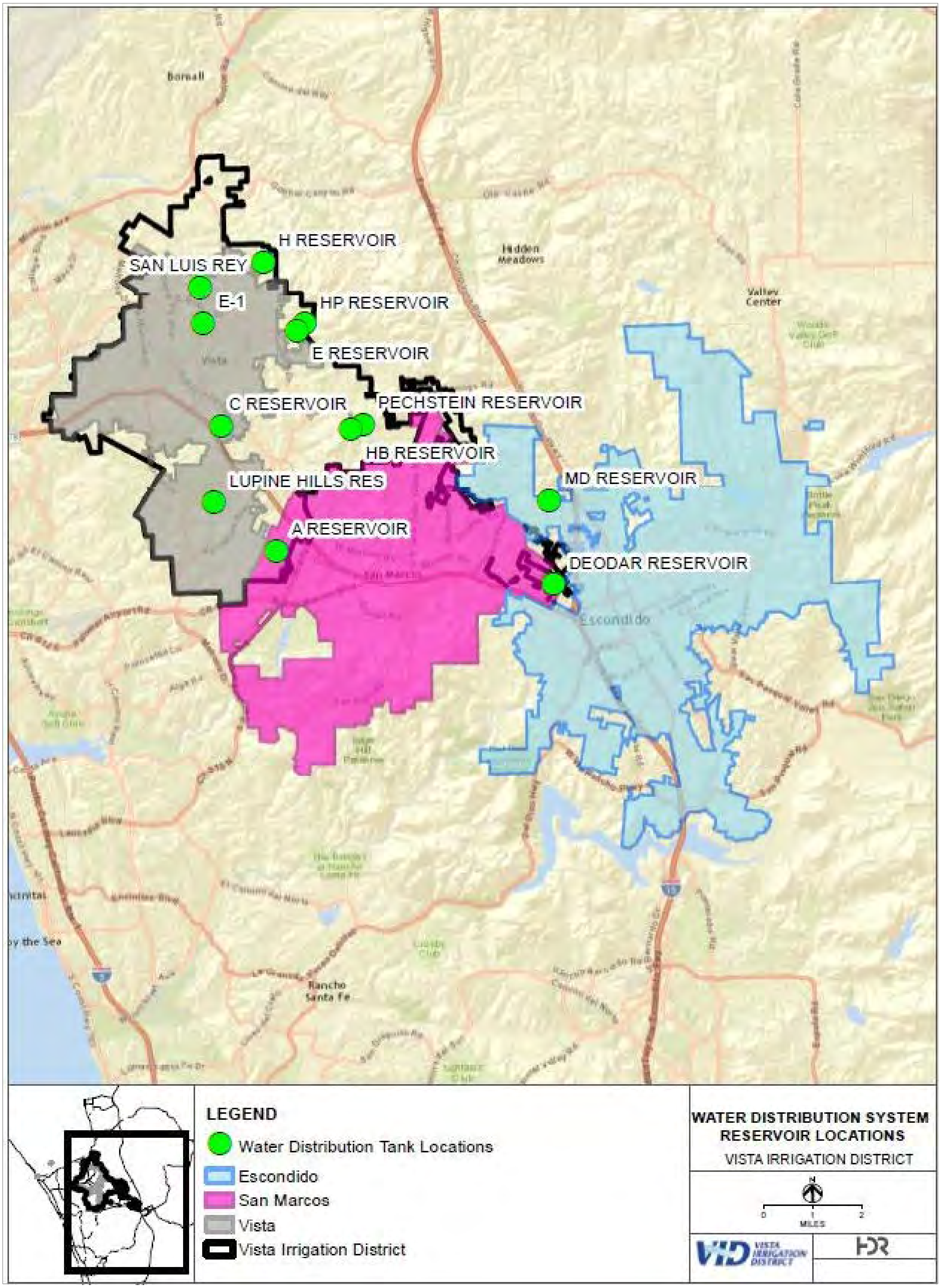
The objective of the reservoir condition assessment inspections is to provide an overall condition of the reservoir inventory and prioritize the sites for recurrent inspections, rehabilitation or replacement of reservoir equipment. The reservoir prioritization was developed based on age and overall civil site, corrosion and structural conditions at the time of inspections.

Section 1 of this report provides an overview of general information of ten VID reservoir sites inspected.

Sections 2 through 11 provide the detailed information, criteria, and photographs for each inspected reservoir. Each section provides civil site, corrosion and structural field observations and assessments.

Section 12 provides a summary of the overall results, which include site survey results, near-term priority recommendations, maintenance schedule, and budgetary-level opinion of cost summary for inclusion in the District's Capital Improvement Plan.

Figure 1-1. VID Reservoir Location Map



1.1 Overview of Inspection Approach

The reservoir site inspections were accomplished in approximately two hours per site and multiple sites were visited each working day. HDR deployed a team of three specialists, including a National Association of Corrosion Engineers (NACE) Certified cathodic protection technician, a structural engineer, and a civil engineer. The team was accompanied by two VID staff members during each of the visits.

A pre-inspection meeting was held with VID staff on November 14th, prior to the field events, to discuss the specifics of each reservoir.

The scope of the Reservoir Condition Assessment Task consisted of the following:

5. Conducted field visits of VID's 10 reservoir sites
6. The field visit required 3 staff that spent an average of 2 hours at each site. Three (3) to four (4) sites were visited per day.
7. A safety plan was provided along with personal protection equipment to safely conduct the field work.
8. District staff members accompanied field personnel to provide site and reservoir access.
9. Field activities completed during these field visits included:
    a. Perimeter, site and drainage check
    b. Structural check
    c. Exterior coatings
    d. Reservoir Climb and roof inspection
    e. Non entry, visual hatch inspection

1.2 Reservoir Summary

General information for each of the ten reservoirs inspected is provided in Table 1-2.

Table 1-2. VID General Reservoir Information

Reservoir Name	Operating Capacity (MG)	Actual Capacity (MG)	Bott Elev. (ft)	HWL Elev. (ft)	Interior Dimensions (ft)	Construction Year	Reservoir Type			Reservoir Roof
							Buried/Above Ground	Shape	Material	
Lupine Hills	3.00	3.40	537	568	137	1987	Partially Buried	Circular	Prestressed Concrete	Reinforced Concrete
A	0.60	0.80	695	708	100	1926	Partially Buried	Circular	Cast-in-place Reinforced Concrete	Wood Rafter and Girder System
Pechstein	18.50	20	810	837	355	1978	Partially Buried	Circular	Prestressed Concrete	Wood Rafter and Girder System
HB	4.05	4.50	951	981	160	1964	Above Ground	Circular	Prestressed Concrete	Tapered Reinforced Concrete Dome
C	0.60	0.80	625	638	100	1926	Above Ground	Circular	Cast-in-place Reinforced Concrete	Wood Rafter and Girder System
E1	0.50	0.60	547	560	90	1925	Above Ground	Circular	Cast-in-place Reinforced Concrete	Wood Rafter and Girder System
San Luis Rey	2.7	3.10	540	565	156 x 136	1978	Buried	Rectangular	Cast-in-place Reinforced Concrete	Reinforced Concrete
H	5.00	5.40	774	810	160	1997	Partially Buried	Circular	Prestressed Concrete	Reinforced Concrete
MD	0.19	0.20	886	900	55	1926	Partially Buried	Circular	Cast-in-place Reinforced Concrete	Wood Rafter and Girder System
Deodar	1.10	1.3	869	899	86	1978	Partially Buried	Circular	Prestressed Concrete	Wood Rafter and Girder System

Source: VID Water Supply Permit, February 2016

1.3 Inspection and Assessment Methodology

The following sections describe the methods used by the HDR inspection team. The project team consisted of the core group of HDR engineers accompanied by two District personnel. District personnel were present for all reservoir inspections and provided valuable information regarding each reservoir's operation and maintenance history.

1.3.1 General

The HDR inspection team reviewed existing data including reservoir as built drawings prior to conducting reservoir inspections. A visual inspection of each site was completed and documented at the time of arrival. Once on site, the inspection team assessed the civil site, corrosion, and structural reservoir conditions and documented their observations. Photographs of each reservoir site were taken to document the existing conditions and display specific site features and areas of improvement.

1.3.2 Civil Site Assessment Approach

The inspection team started the civil site assessment by performing a site perimeter walk around the reservoirs to note the current condition of the access road, security fence, access gates, and signage. Conditions of all aspects of the visual inspection were noted. Site appurtenances not directly attached to the reservoirs were also noted, but not evaluated during the site inspections. Trees and vegetation located on site was assessed in regards to interference with site operation or maintenance, site security, and overall site cleanliness.

The civil site assessment continued with a walk around of the entire reservoir boundary. Side and roof access hatches were checked for condition. Presence of reservoir venting systems and their conditions were noted in the field. Reservoir venting systems were not evaluated for adequacy to move draw or flow-through air. Safety systems including exterior ladders, guardrails and fall protection were assessed. The presence of safety and fall protection systems were noted along with general distances between ladders and platforms for hazard issues. The current conditions of ladders and guardrails were noted.

The presence of and the condition of signage along both the reservoir and the surrounding security fence and access gates were noted. Civil site observations for all VID reservoirs were compared and inconsistences between reservoirs were noted.

1.3.3 Corrosion Assessment Approach

Each reservoir was examined for signs of corrosion in metal components and concrete reinforcement. Metal components were inspected for coating degradation, surface rusting, galvanic coupling, pitting, and general metal loss. Concrete components of each reservoir were inspected for cracking, rust bleed and efflorescence, seepage, spalling, and other signs of distress. Notable locations and signs of distress at each reservoir were photographed and entered into the field notes.

Exterior corrosion assessment was performed by walking around each reservoir and climbing onto the roof using the fixed ladder or stairway. Rooftop assessments were limited to the areas near the ladder secured with guardrails and the interior area of the roof at least 10 feet away from the edge. In the case of the Pechstein Reservoir, due to extensive wood rot and uncertain footing along the ridgeline, only the area between the roof walkway and the center vent was traversed.

Interior corrosion assessment was limited to visual examination of the areas which could be seen from the reservoir access hatch or other vantage points on the exterior. At the Pechstein Reservoir, a permanent access door in the side of the reservoir and interior observation platform allowed for viewing of the interior structure without confined space entry.

1.3.4 Structural Assessment Approach

In addition to the civil site and corrosion observations, a limited structural evaluation was performed. Prior to visiting each reservoir site, the existing as-built drawings, where available, were reviewed. This review was used to determine the structural configuration of each reservoir and assist in identifying critical components requiring inspection.

Upon arrival at each reservoir site, visual inspection was conducted to determine the condition of the critical structural components and their general functionality. All observed signs of distress, possibly indicating current or future structural deficiency were photographed and noted.

The type and general condition of each roofing system was documented. Exterior of each reservoir wall was observed and obvious structural symptoms such as cracking occurrence and pattern, spalling, etc. were photographed and noted. The conditions of paint and/or coatings were also observed.

Observation of each reservoir interior was limited to a non-entry or, where possible, limited-entry visual inspection. The access hatch was opened at each site and a spotlight was used to view each reservoir interior. The general conditions of the purlins, girders, and columns located in close proximity to the hatch were noted and, where possible, photographed. Interior ladders or stairs located at the access hatch were visually evaluated.

Seismic analysis of each reservoir was outside the approved scope of services, but, where possible, observations and recommendations related to seismic code compliance were provided. The estimated condition of components was based on visual inspections and input from VID staff on all reservoir components was also considered.

Except for the interior reservoir components not accessible at the time of inspection, structural components not specifically identified in this report can be assumed to be in good condition.

1.3.5 Recommendations Approach

The civil/site and corrosion and structural recommendations listed for each reservoir address the deficiencies noted during the field inspections. The civil/site, corrosion, and structural recommendations pertain to ongoing monitoring, minor maintenance, and repair work. The recommendations for further investigation lists potentially larger scale improvements and recommendations for further investigation. An overall condition rating and prioritization of reservoir improvements is included in Section 12. Section 12 also contains a proposed maintenance schedule, recommendations for additional assessment and a budgetary level opinion of cost summary for inclusion in the District's Capital Improvement Plan.

2 Lupine Hills Reservoir

Lupine Hills Reservoir has a 3.00 MG operating capacity and is located at 2450 Lupine Hills Drive, Vista, CA 92081, as shown in Figure 2-1. Lupine Hills is a two tone reservoir that was constructed in 1987. According to VID staff, there have been no exterior repairs done on the Lupine Hills Reservoir since its original construction. A known maintenance issue with the reservoir in the past included noticeable dips in the roof, as viewed by VID staff. An exterior paint job was completed on the reservoir in the mid 1990's. HDR's inspection of Lupine Hills Reservoir was conducted on November 14, 2016, as shown in Figure 2-2.

Figure 2-1. Lupine Hills Vicinity Map

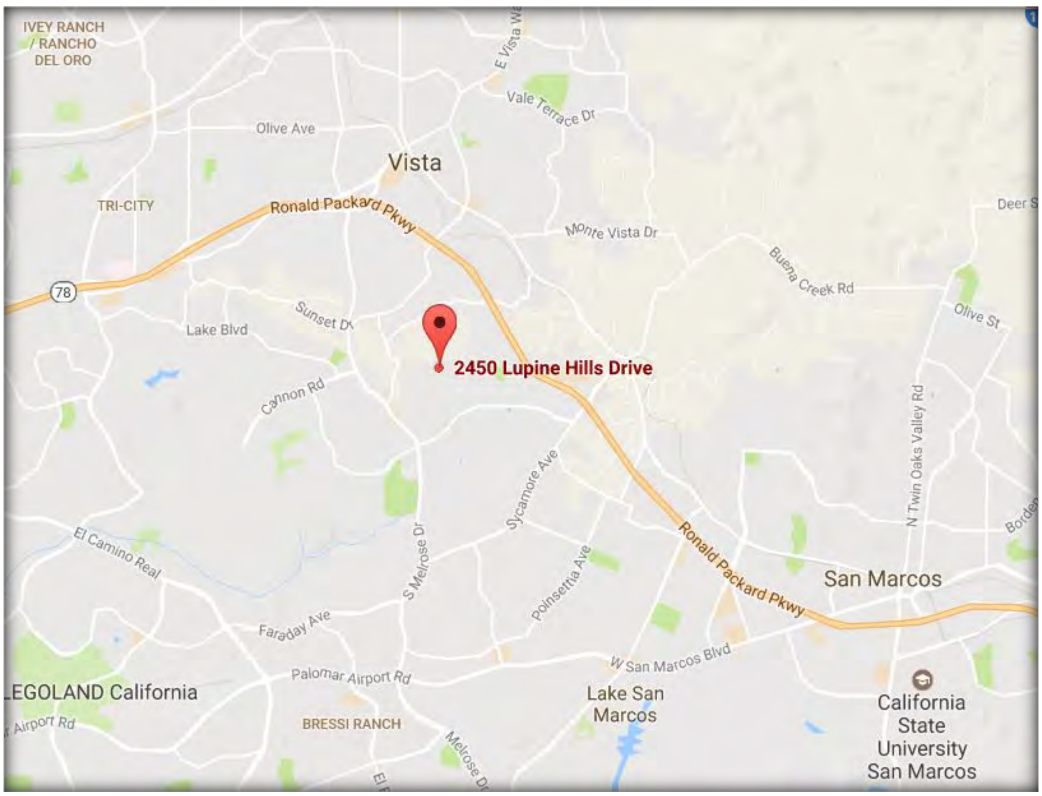


Figure 2-2. Lupine Hills Reservoir West (left) and East (right)

2.1 Typical Civil/Site and Corrosion Observations

Table 2-1. Civil/Site and Corrosion Observations

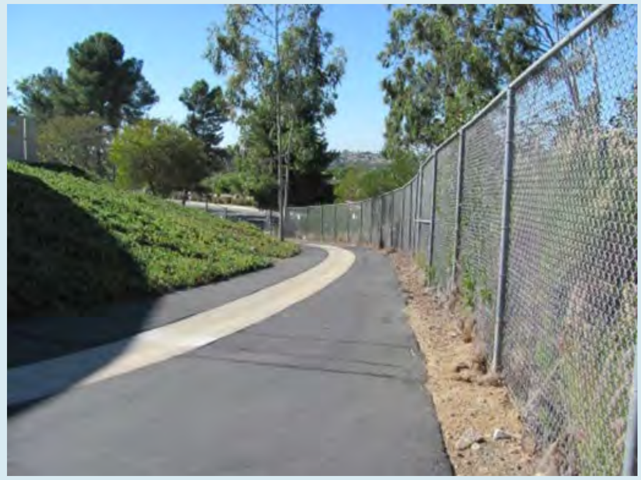
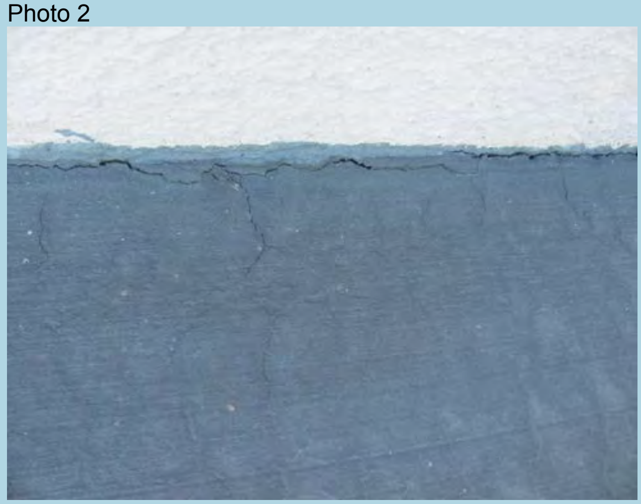
Lupine Hills Civil/Site and Corrosion Observations		Photo
Fence/Gate	Entrance contains a locked gate with VID signage. Fencing surrounding the reservoir and gate were typically in good condition. Fence height surrounding the reservoir varies due to dirt and shrubbery but is at an adequate height to prevent unauthorized entry (Photo 1).	Photo 1 
Access Road	Access road is paved and was fully accessible to the inspection teams and their vehicles. The pavement was in good condition with only signs of minor cracking surrounding the reservoir (Photo 2).	Photo 2 

Table 2-1. Civil/Site and Corrosion Observations

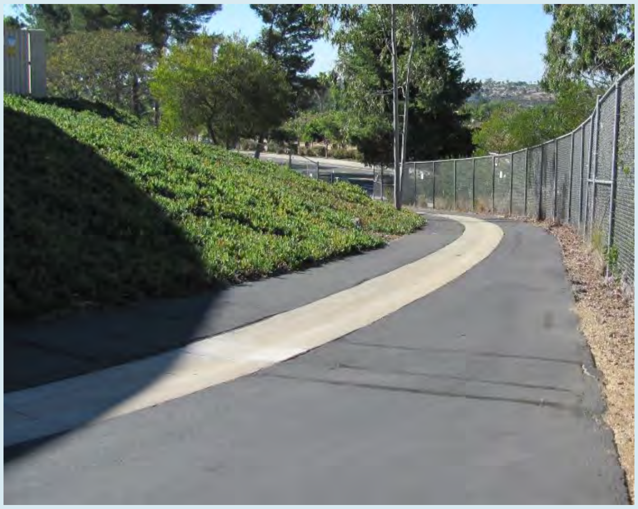
Lupine Hills Civil/Site and Corrosion Observations		Photo
Drainage	Roof drains spill runoff into gutter around perimeter of the access road. Area is clear and observed to be in good condition. Adequate drainage with good slope for runoff (Photo 3).	Photo 3 
Trees and Vegetation	Trees and shrubbery surrounded the reservoir near the perimeter fence with minimal tree growth on the fence (Photo 4).	Photo 4 
Hatches	Access to the reservoir is through the roof access hatch. Steel screws and the intrusion alarm switch were found to be corroded. The hatch was observed to be in overall good condition (Photo 5).	Photo 5 

Table 2-1. Civil/Site and Corrosion Observations

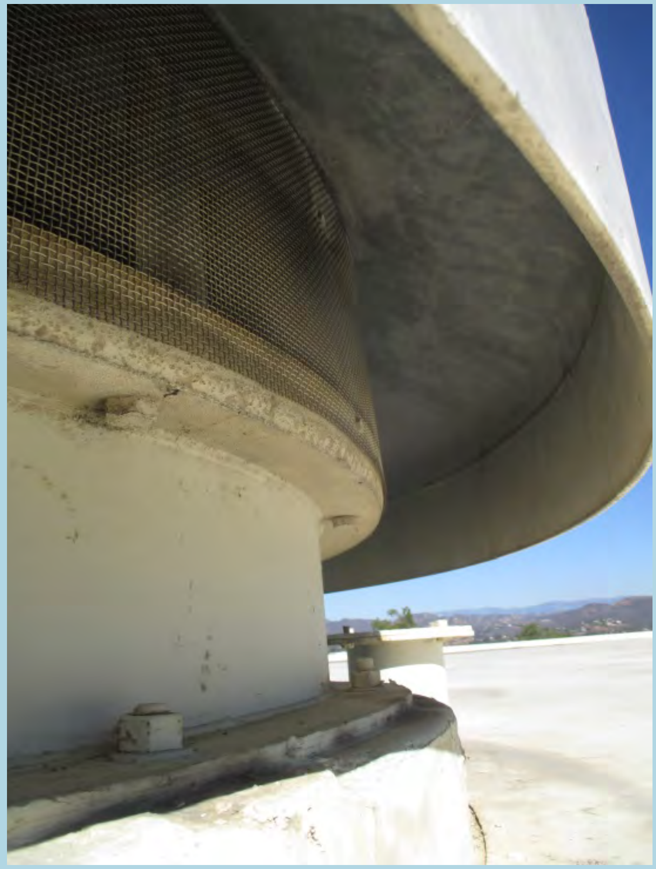
Lupine Hills Civil/Site and Corrosion Observations		Photo
Vents	One roof vent located at the center of the reservoir roof. The conduit penetration plate on top of the vent was found to be corroded. The vent was observed to be in overall good condition with minor rusting (Photo 6).	Photo 6 

Table 2-1. Civil/Site and Corrosion Observations

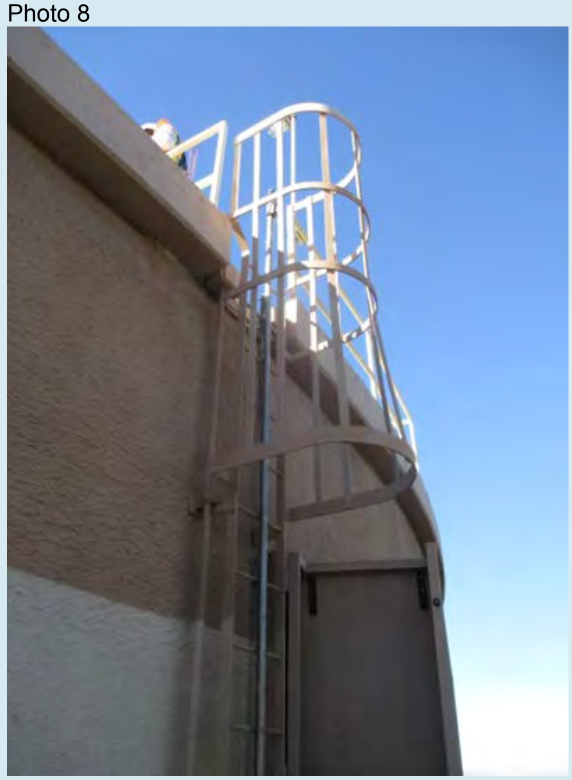
Lupine Hills Civil/Site and Corrosion Observations		Photo
Ladders	Exterior ladder is painted carbon steel with a cage and anti-climb door. The U-shaped frame on the top of the anti-climb door has coating loss and surface rusting. Anchor bolts supporting the ladder in the shotcrete on the reservoir side are intact and appear to be in good condition. Ladder top supports are welded to guardrails cast into the reservoir roof. Extensive corrosion and metal loss on the guardrail supports have resulted in failure directly above the embedment locations (Photo 7 and Photo 8).	Photo 7  Photo 8 

Table 2-1. Civil/Site and Corrosion Observations

Lupine Hills Civil/Site and Corrosion Observations		Photo
Handrails and Guardrails	Painted carbon steel guardrails are embedded in the reservoir roof slab and parapet. Previous repairs with concrete mortar at the embedment locations have not prevented corrosion and failure of the guardrail support posts at these locations. At least two of the posts have total metal loss. Guardrails at the ladder location no longer provide fall protection (Photo 9).	Photo 9 
Signage	Signage includes VID and no trespassing signs located along the fence surrounding the reservoir. On site is fenced in cellular control equipment that contains caution and danger signs (Photo 10).	Photo 10 

2.1.1 Conclusions

Based on visual inspections of the civil site and corrosion observations at Lupine Hills Reservoir at the time of the condition assessment, the following conclusions are made:

- Guardrails on the reservoir roof have failed as a result of corrosion at the embedment locations.
- The exterior ladder anti-climb device and some conduit supports have surface rusting that requires repainting.
- Surrounding trees and vegetation were out of vicinity of site operation and maintenance at the time of inspections but could potentially obstruct if not maintained periodically.
- Minimal debris build up at bottom of roof gutters could conflict with runoff flow and should be intermittently cleaned.
- The screws located on the hatches and doors were corroded. The intrusion alarm switch was also corroded.
- Gap located between the roof and ring wall. Minor cracks located in roof should be filled with concrete sealer.
- The center vent conduit plate contained rust and corroded conduit brackets.
- The minor cracking along the access road pavement and surrounding the reservoir is not of concern but should be monitored.

2.2 Structural Observations

2.2.1 As-Built Drawings Review

Based on cursory review of the as-built drawings prepared by James M Montgomery Consulting Engineers, Inc., dated August 1986, the Lupine Hills Reservoir is a partially buried, circular shaped, prestressed concrete reservoir with a 6-inch thick reinforced concrete floor and a 10-inch thick reinforced concrete core wall with an exterior gunite layer providing cover for the circumferential prestressed reinforcing. The roof consists of an 8½-inch thick reinforced concrete roof supported by 18-inch diameter reinforced concrete columns bearing on 6-foot square by 18-inch thick reinforced concrete footings. The reservoir is approximately 137 feet in diameter with a maximum water depth of approximately 31 feet at overflow.

The original reservoir design included seismic cable system at the wall to foundation connection, but further analysis will be required to determine if the reservoir is in compliance with the current seismic code.

Table 2-2. Exterior Observations

Lupine Hills Structural Exterior Observations		Photo
Roof	Typical evidence of ponding at low points between roof drains at the outside edge of the roof (approximately 10 locations) (Photo 1).	Photo 1 
Roof	Typical cracking on concrete roof, concentrated over column caps (Photo 2).	Photo 2 
Roof	Typical cracking at the corners of the concrete curb around the access hatch (Photo 3).	Photo 3 

Table 2-2. Exterior Observations

Lupine Hills Structural Exterior Observations		Photo
Roof	Typical vertical cracking on outside face of concrete roof curb (approximately 4-foot spacing) (Photo 4).	Photo 4 
Roof/Wall	Typical deterioration of joint material at roof/wall interface. Active moisture seepage with staining of exterior wall surface (Photo 5).	Photo 5 
Wall	Typical staining of exterior wall surface at deteriorated roof/wall joint material (Photo 6).	Photo 6 

Table 2-2. Exterior Observations

Lupine Hills Structural Exterior Observations		Photo
Wall	Cracking in gunite finish. This condition was only observed at the low end of the access road. The cracking is concentrated on the lower 1/3 of the wall height and does not appear to have any moisture seepage (Photo 7).	Photo 7 

Table 2-3. Interior Observations

Lupine Hills Structural Interior Observations		Photo
Roof/Wall	Light entering reservoir at deteriorated roof/wall joint locations (Photo 1).	Photo 1 
Wall	Intact repair of interior concrete wall surface (Photo 2).	Photo 2 
Access Stairs	Typical surface staining of interior metals (Photo 3).	Photo 3 

2.2.2 Conclusions

Based on inspection of the visible portions of Lupine Hills Reservoir at the time of the condition assessment, the following conclusions are made:

- The evidence of ponding observed on the roof is not a structural concern, but should be addressed to prevent long-term damage to the roof coating and concrete.
- The cracking on the roof deck and hatch curb appear to have been present since construction, but should be addressed to prevent long-term damage to the concrete and embedded steel reinforcing due to moisture infiltration.
- The cracking on the outside face of the roof deck appears to have been present since construction and is not a structural concern.
- The deterioration of the joint material at the roof/wall interface is not a structural concern, but is affecting the water-tightness of the reservoir and should be mitigated.
- The staining of the exterior wall surface is not a structural concern, but should be cleaned once the roof/wall joint material is repaired.
- The cracking of the gunite finish is not currently a structural concern, but should be monitored for indication of moisture seepage and corrosion of the embedded circumferential prestressed reinforcing.
- The repairs to the interior concrete surfaces appear to be intact and are not currently a structural concern.
- The surface staining and minor corrosion of the interior metals is not currently a structural concern, but should be monitored to ensure long-term serviceability of the metal components.

2.3 Recommendations

The following recommendations address the deficiencies noted during the field inspections. Section 2.3.1 and 2.3.2 include recommendations pertaining to minor maintenance, repair work and ongoing monitoring. Section 2.3.3 lists potentially larger scale improvements and recommendations for further investigation. An overall condition rating and prioritization of reservoir improvements is included in Section 12. Section 12 also contains proposed recommendation phasing, recommendations for additional assessment and a budgetary level opinion of cost summary for inclusion in the District's Capital Improvement Plan.

2.3.1 Civil/Site and Corrosion Recommendations

The following are civil site and corrosion improvement recommendations to be considered for the Lupine Hills Reservoir site:

- The guardrails should be replaced with bolt-down style guardrails using stainless steel anchor bolts.
- The anti-climb device on the exterior ladders should be repainted.
- Maintain trees and vegetation to ensure it doesn’t interfere with site operation or maintenance.
- Ensure roof gutters are periodically cleaned to allow for adequate runoff.
- The corroded screws on the hatches and doors should be replaced with stainless hardware. The corroded intrusion alarm switch should be replaced.
- The gap between the roof and ring wall should be caulked. Cracks in roof should be filled with concrete sealer.
- The center vent conduit plate should be cleaned of rust and repainted. The corroded conduit brackets should be repainted or replaced.

2.3.2 Structural Recommendations

The following are structural improvement recommendations to be considered for the Lupine Hills Reservoir site:

- Modify the roof slope, as required, to prevent ponding and provide proper drainage.
- Seal all cracking on the roof deck and hatch curb to prevent long-term damage to the concrete and embedded steel reinforcing due to moisture infiltration.
- Replace the joint material at the roof/wall interface and seal the joint to restore the water-tightness of the reservoir.
- Clean all staining of the exterior wall surface.
- Regularly monitor the cracking of the gunite finish for indication of moisture seepage and corrosion of the embedded circumferential prestressed reinforcing.
- Monitor all corrosion of the interior metal components.

2.3.3 Recommendations for Further Investigation

The following are potentially larger scale improvements and recommendations for further investigation for the Lupine Hills Reservoir site:

- Perform a detailed condition assessment of the reservoir interior.
- Perform a seismic evaluation of the reservoir to determine if it is in compliance with the current seismic code.

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3 A Reservoir

A Reservoir has a 0.60 MG operating capacity and is located at 770 Virginia Place, San Marcos, CA 92078, as shown in Figure 3-1. A Reservoir was constructed in 1926. According to VID staff, the beams and columns during time of inspections are the originals from the time of construction. VID staff mentioned that due to leaking, the reservoir interior was lined with CIM 1061 in December 2007. A Reservoir's roof has been resurfaced several times over the years due to minor termite damage to the roof lumber. Most recently, A Reservoir was cleaned in 2015. HDR's inspection of A Reservoir was conducted on November 14, 2016, as shown in Figure 3-2.

There is a shared site with the San Diego County Water Authority located to the east of the entrance of A Reservoir.

Figure 3-1. A Reservoir Vicinity Map

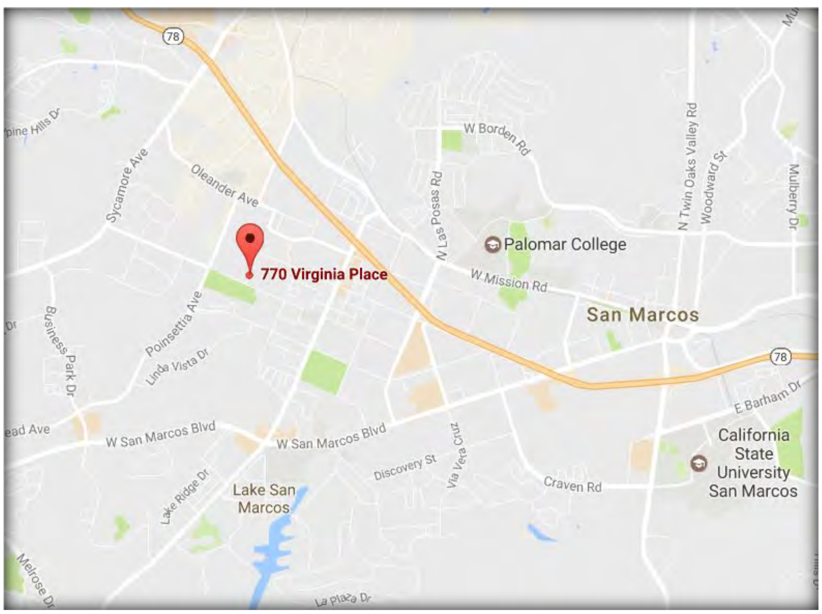


Figure 3-2. A Reservoir

3.1 Typical Civil/Site and Corrosion Observations

Table 3-1. Civil/Site and Corrosion Observations

A Reservoir Civil/Site and Corrosion Observations		Photo
Fence/Gate	Full access of the surrounding fence at A Reservoir requires entry and re-entry through two separate fencing areas. Figure 3-2 displays the additional fence on the east side entrance. Paved access road has adequate space for field crews and multiple vehicles (Photo 1).	Photo 1 
Access Road	Approximately half of the surrounding fencing is shared between the reservoir and the adjacent properties. The fence in the back area is damaged due to the neighboring homes property, displayed in Figure 3-3 (Photo 2).	Photo 2 

Table 3-1. Civil/Site and Corrosion Observations

A Reservoir Civil/Site and Corrosion Observations		Photo
Drainage	Roof drains spill runoff into gravel slope that leads down to the street below (Photo 3). VID staff mentioned past complaints from adjacent properties regarding runoff; problems have since been alleviated.	Photo 3 
Trees and Vegetation	Typical trees and vegetation from adjacent properties surrounding reservoir. Instances of minimal tree growth on the surrounding fence (Photo 4).	Photo 4 

Table 3-1. Civil/Site and Corrosion Observations

A Reservoir Civil/Site and Corrosion Observations		Photo
Hatches	Access to the reservoir is through the roof access hatch. The hatch showed surface corrosion on the lock cover but was in overall good condition (Photo 5).	Photo 5 
	The float box hatch was found to be in good condition with some paint degradation and minor surface rusting. Coating loss and pitting was observed on the support beam inside the float box (Photo 6).	Photo 6 

Table 3-1. Civil/Site and Corrosion Observations

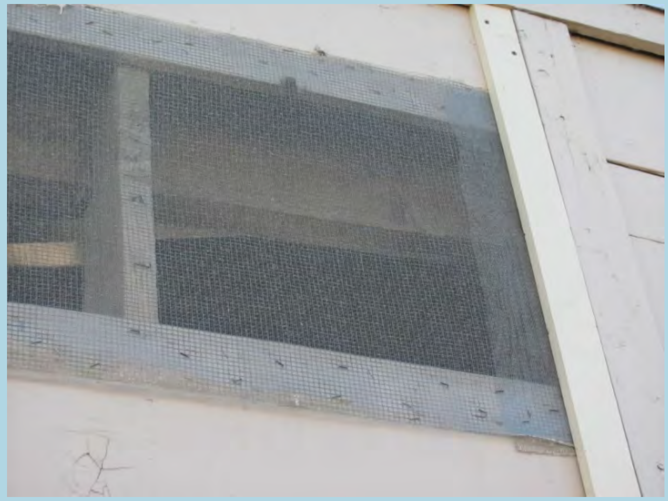
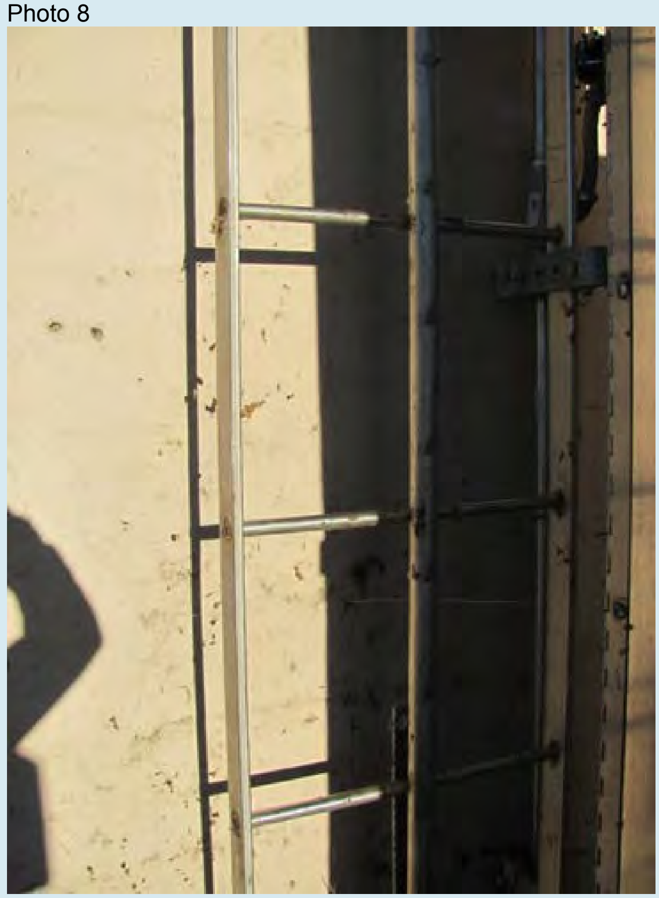
A Reservoir Civil/Site and Corrosion Observations		Photo
Vents	Side vents are located around the perimeter of the reservoir every few feet apart. Vents contain a mesh covering and wood frame on each side. Vents were observed to be in good condition (Photo 7).	Photo 7 
Ladders	Typical surface rusting observed on exterior ladder, overall good condition (Photo 8).	Photo 8 

Table 3-1. Civil/Site and Corrosion Observations

A Reservoir Civil/Site and Corrosion Observations		Photo
Handrails, and Guardrails	Aluminum guardrails located at roof hatch and float box areas were in good condition (Photo 9).	Photo 9 
Pipes and Appurtenances	Corrosion and localized metal loss observed on the exterior pipe riser (Photo 10).	Photo 10 
Signage	Signage observed to be in faded and poor condition. Replacement is necessary (Photo 11).	Photo 11 

3.1.1 Conclusions

Based on visual inspections of the civil/site and corrosion observations at A Reservoir at the time of the condition assessment, the following conclusions are made:

- Front gate was observed to be in good condition. Surrounding fence consists of both VID and adjacent properties fencing. Fence issues consist of the following:
  - In order to get full perimeter access around Reservoir A, entry and re-entry through two separate fencing areas is required. Access from the west side of the reservoir does not allow for full reservoir access, see Figure 3-3.
  - Neighboring property shared chain link fence in the back area is damaged due to the neighboring homes property, see Figure 3-4.
- Surrounding trees and vegetation out of vicinity of site operation and maintenance at the time of inspections but could potentially obstruct if not maintained periodically.
- Exterior piping riser contained areas of significant metal loss at joints.
- Signage was rusted, aging and faded. No confined space signage installed on roof hatch or overflow hatches.
- The interior support beam in the float box was found to have coating degradation and pitting.
- Roof hatch contains rust and lock cover has areas of chipped paint.
- Soft spots and termite damage on the existing roof system.
- Sill bolts on ring walls are corroded and have areas of chipped paint.

Figure 3-3. East Side Fence

Figure 3-4. Damaged Fence

3.2 Structural Observations and Conclusions

3.2.1 As-Built Drawings Review

Based on cursory review of the as-built drawings prepared by the Engineering Offices of J B Lippincott, dated August 1925, the A Reservoir is a partially buried, circular shaped, cast-in-place reinforced concrete reservoir with a 4-inch thick reinforced concrete floor and a tapered reinforced concrete wall that is 16 inches thick at the base and 8 inches thick at the top. The wall is supported by a 32-inch wide by 12-inch thick continuous reinforced concrete footing. The roof consists of a wood rafter and girder system supported by 8-inch square precast reinforced concrete columns bearing on 2-foot square by 6-inch thick reinforced concrete footings. The reservoir is approximately 100 feet in diameter with a maximum water depth of approximately 13 feet at overflow. A 10-foot by 7-foot, 4-inch by 13-foot, 8-inch high control box is located at the northwest quadrant of the reservoir.

Considering A Reservoir's date and type of construction, it is not in compliance with the current seismic code.

Table 3-2. Exterior Observations

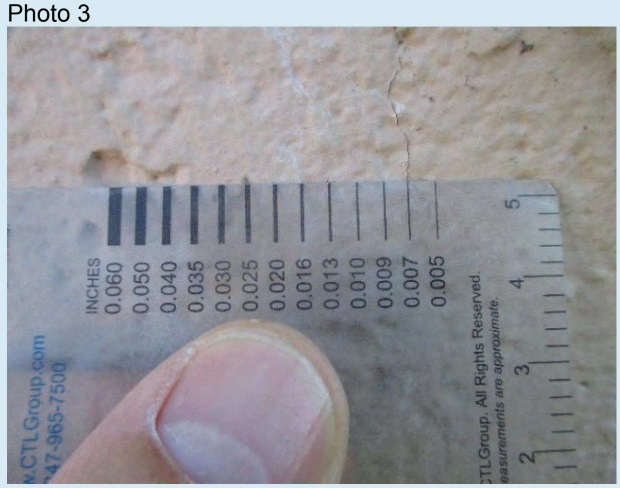
A Reservoir Structural Exterior Observations		Photo
Roof	Visible sagging of the roof paneling between the supports with evidence of ponding (throughout entire roof area). Vertical deflection of the roof system felt while walking the roof (Photo 1).	Photo 1 
Wall	Typical staining of exterior wall surface from roof/wall interface (Photo 2).	Photo 2 
Wall	Typical full-height cracking in exterior face of concrete wall. This condition was observed along the entire perimeter of the reservoir at approximately 8-foot spacing. There does not appear to be any moisture seepage (Photo 3).	Photo 3 

Table 3-3. Interior Observations

A Reservoir Structural Interior Observations		Photo
Roof	Typical condition of interior roof framing and support (Photo 1).	Photo 1 
Roof/Wall	Typical condition of roof structure connection to reservoir wall (Photo 2).	Photo 2 
Control Box Roof	Typical severe corrosion with section loss on interior roof framing. Concrete spalling with exposed corroded reinforcing above overflow opening into control box (Photo 3).	Photo 3 

3.2.2 Conclusions

Based on inspection of the visible portions of A Reservoir at the time of the condition assessment, the following conclusions are made:

- The sagging and evidence of ponding observed on the roof is expected of age and type of construction. Roof loading should be limited and roof condition monitored regularly for safety.
- The staining of the exterior wall surface is not a structural concern.
- The wall cracking appears to have been present since construction and is not a structural concern.
- Despite the age of the roof framing, it is in fair condition and appears to be functioning properly. As stated previously, the roof system is not in compliance with the current seismic code and should be monitored regularly for safety.
- The corrosion of the interior metals is not currently a structural concern, but should be monitored to ensure long-term serviceability of the control box roof.
- The spalling and exposed reinforcing above the overflow has an immediate effect on the structural stability of the wall above the overflow and should be repaired as soon as possible.

3.3 Recommendations

The following recommendations address the deficiencies noted during the field inspections. Section 3.3.1 and 3.3.2 include recommendations pertaining to minor maintenance, repair work and ongoing monitoring. Section 3.3.3 lists potentially larger scale improvements and recommendations for further investigation. An overall condition rating and prioritization of reservoir improvements is included in Section 12. Section 12 also contains a proposed recommendation phasing, recommendations for additional assessment and a budgetary level opinion of cost summary for inclusion in the District's Capital Improvement Plan.

3.3.1 Civil/Site and Corrosion Recommendations

The following are civil site and corrosion improvement recommendations to be considered for the A Reservoir site:

- Partial chain link fence surrounding parts of reservoir boundary should be replaced with a full chain link fence allowing full access to entire reservoir boundary from the reservoir entrance.
- Maintain trees and vegetation to ensure it doesn't interfere with site operation or maintenance.
- Replace corroded pipe sections on exterior pipe riser.
- Replace faded aging signage with new signage. Install confined space signage on roof hatch. Use black text on signage to prevent fading.
- Remove corrosion products and recoat steel support beam in float box.

- Remove rust and repaint lock cover on the roof hatch.
- Further inspect and monitor soft spots and termite damage on the existing roof system.
- Inspect and paint corroding sill bolts on ring walls.

3.3.2 Structural Recommendations

The following are structural improvement recommendations to be considered for the A Reservoir site:

- Modify the roof slope, as required, to prevent ponding and provide proper drainage.
- Limit roof loading to two workers and fifty pounds of equipment and regularly monitor roof condition for safety.
- Clean all staining of the exterior wall surface.
- Seal all cracking in the exterior wall to prevent long-term damage to the concrete and embedded steel reinforcing due to moisture infiltration.
- Monitor all corrosion of the interior metal components.
- Clean corrosion and coat interior anchor bolts.
- Clean corrosion and coat interior roof beams.
- Repair the spalling in the concrete beam above the overflow.

3.3.3 Recommendations for Further Investigation

The following are potentially larger scale improvements and recommendations for further investigation for the A Reservoir site:

- Perform a detailed condition assessment of the reservoir interior.
- Full reservoir roof replacement following the results of the detailed condition assessment of the reservoir interior.
- Perform a seismic evaluation of the reservoir to determine if seismic retrofit is a viable option to achieve compliance with the current seismic code.

4 Pechstein Reservoir

Pechstein Reservoir has a 18.50 MG operating capacity and is located at 3784 Bluebird Canyon Road, Vista, CA 92084, as shown in Figure 4-1. Pechstein Reservoir was constructed in 1978. According to VID staff, Pechstein Reservoir has significant exterior rot of glulam beams. A large steel girder was installed inside on 4 beams to shore up the glulam's. Most of the roofing cross members have lost the joint hangers due to decay. Despite issues with the roof failing, only standard maintenance has been performed on this reservoir. HDR's inspection, as illustrated in Figure 4-2, of Pechstein Reservoir was conducted on November 15, 2016.

Figure 4-1. Pechstein Vicinity Map

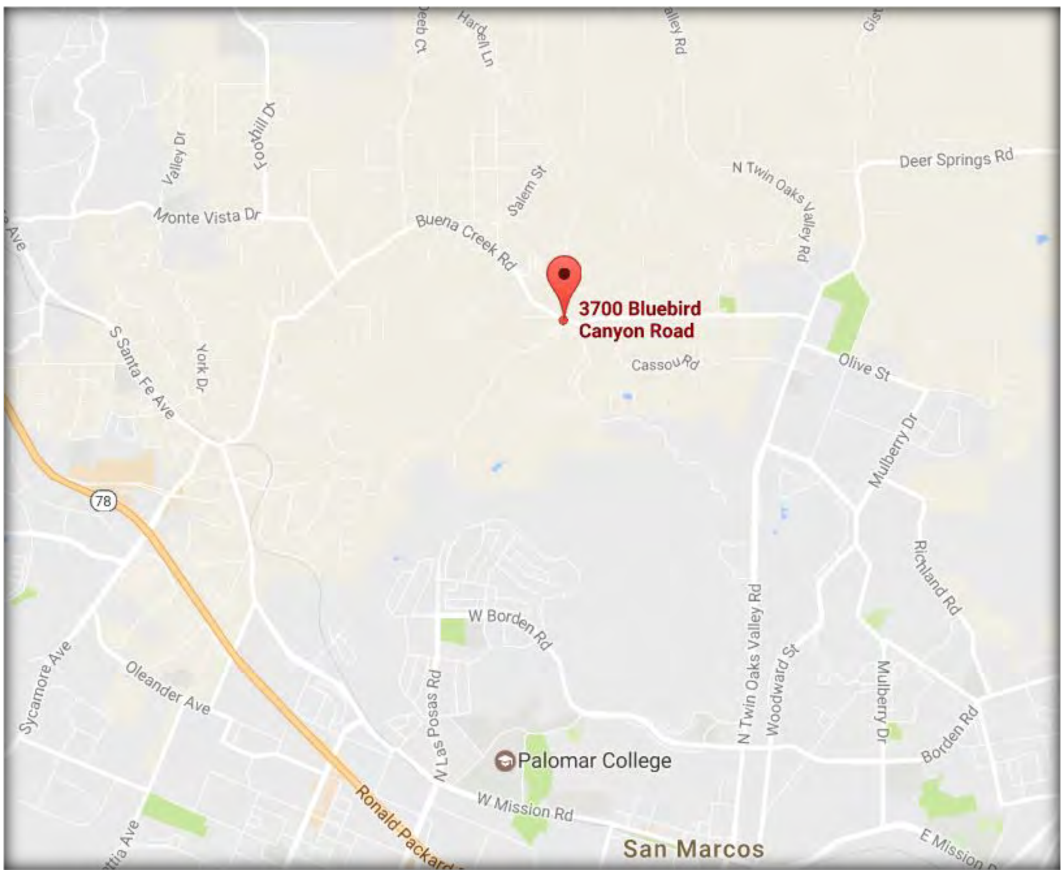
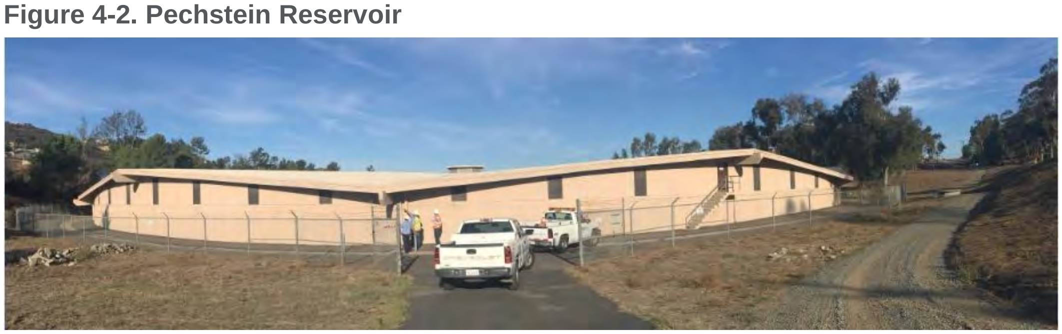


Figure 4-2. Pechstein Reservoir



4.1 Typical Civil/Site and Corrosion Observations

Table 4-1. Civil/Site and Corrosion Observations

Pechstein Reservoir Civil/Site and Corrosion Observations		Photo
Fence/Gate	The perimeter of the reservoir is enclosed by a chain link fence with 3 rows of barbed wire on top. The fence and gate were observed to be in good condition with minor rusting (Photo 1).	Photo 1 
Access Road	Paved access road provides adequate space for field crews and multiple vehicles. The access road was observed to have minor cracking, displayed in Figure 4-3, but is in overall good condition (Photo 2).	Photo 2 

Table 4-1. Civil/Site and Corrosion Observations

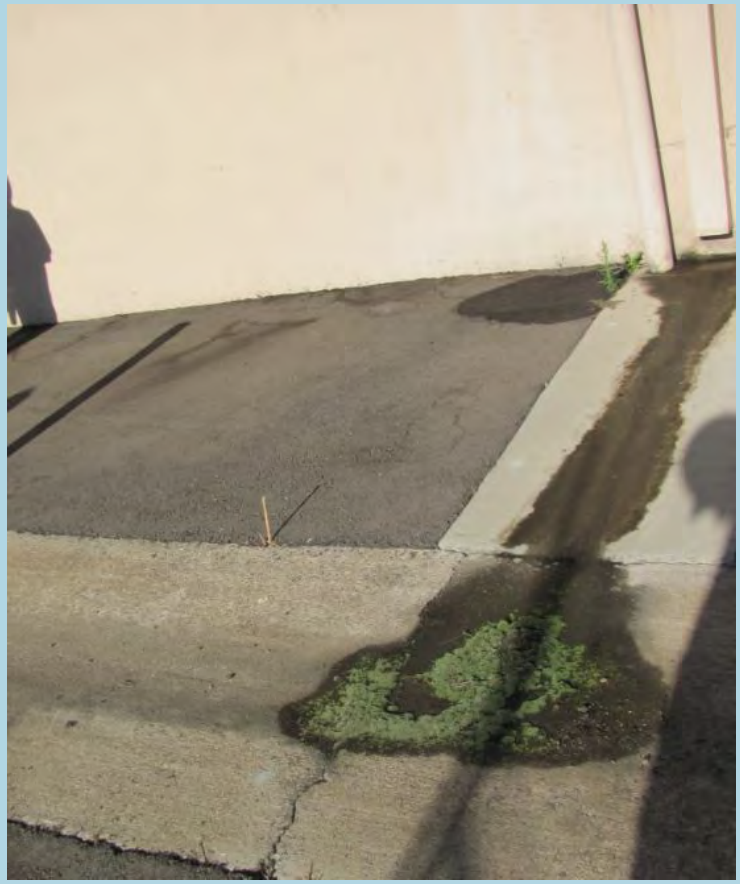
Pechstein Reservoir Civil/Site and Corrosion Observations		Photo
Drainage	Roof gutters allow runoff into surrounding channel. Ponding was observed around the perimeter of the reservoir (Photo 3).	Photo 3 
Trees and Vegetation	Typical trees and vegetation outside of the site fence (Photo 4).	Photo 4 

Table 4-1. Civil/Site and Corrosion Observations

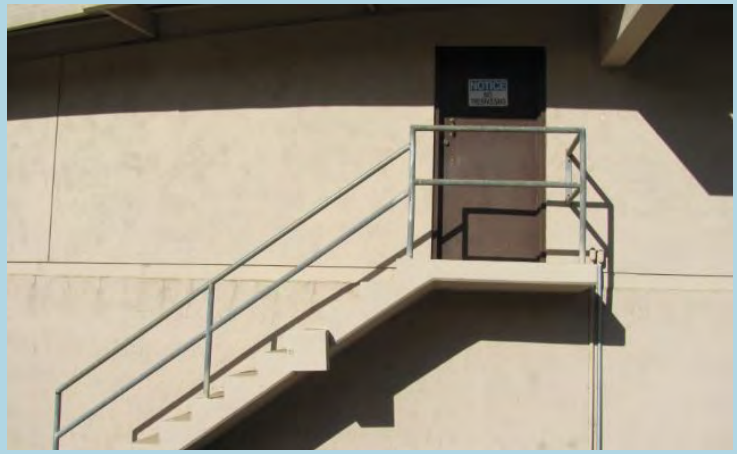
Pechstein Reservoir Civil/Site and Corrosion Observations		Photo
Entrance/Stairs	Access to the inside of Pechstein Reservoir is done through stairs located at the front entrance. Localized corrosion was observed on the interior door surface (Photo 5).	Photo 5 

Table 4-1. Civil/Site and Corrosion Observations

Pechstein Reservoir Civil/Site and Corrosion Observations		Photo
Vents	One roof vent is located at the center of the reservoir roof. The roof vent had areas of rusting and loose mesh observed from a distance (Photo 6).	Photo 6 
	Side vents are located around the perimeter every few feet apart. Vents have a mesh covering. Vents were observed to be in good condition with minor rusting (Photo 7).	Photo 7 

Table 4-1. Civil/Site and Corrosion Observations

Pechstein Reservoir Civil/Site and Corrosion Observations		Photo
Ladders	Typical surface rusting on exterior ladder, overall in good condition (Photo 8).	Photo 8 

Table 4-1. Civil/Site and Corrosion Observations

Pechstein Reservoir Civil/Site and Corrosion Observations		Photo
Handrails, and Guardrails	Guardrail and handrail located at the reservoir roof access hatch were observed to be in good condition (Photo 9).	Photo 9 
	Guardrails at reservoir roof were observed to be in good condition (Photo 10).	Photo 10 

Table 4-1. Civil/Site and Corrosion Observations

Pechstein Reservoir Civil/Site and Corrosion Observations		Photo
Signage	Site signage includes signs located on the entrance gate and along the fence. All signage typically in good condition (Photo 11).	Photo 11 

4.1.1 Conclusions

Based on visual inspections of the civil/site and corrosion observations at Pechstein Reservoir at the time of the condition assessment, the following conclusions are made:

- Continuous cracks along pavement surrounding Pechstein Reservoir is not currently of concern but should be monitored, see Figure 4-3.
- Evidence of ponding in channel surrounding Pechstein Reservoir coming from roof gutters is not currently of concern but should be monitored.
- Roof vent observed to have areas of rusting and loose mesh.
- Hatches contain rust spots and the access door interior contains areas of chipped off paint.
- No confined space signage installed at the roof hatch location.

Figure 4-3. Pavement Cracks

4.2 Structural Observations and Conclusions

4.2.1 As-Built Drawings Review

Based on cursory review of the as-built drawings prepared by James M Montgomery Consulting Engineers, Inc., dated October 1976, the Pechstein Reservoir is a partially buried, circular shaped, prestressed concrete reservoir with a 6-inch thick reinforced concrete floor and an 18-inch thick reinforced concrete core wall with an exterior gunite layer providing cover for the circumferential prestressed reinforcing. The prestressed concrete wall extends to an elevation 28 feet above the wall footing. An 8-inch thick reinforced masonry wall sits on top of the prestressed concrete wall and varies in height based on the slope of the roof. The roof consists of a wood rafter and girder system supported by reinforced concrete columns at the interior of the reservoir and the reinforced masonry wall at the exterior. The reservoir is approximately 358 feet in diameter with a maximum water depth of approximately 27 feet at overflow.

The original reservoir design included seismic cable system at the wall to foundation connection, but further analysis will be required to determine if the reservoir is in compliance with the current seismic code.

Table 4-2. Exterior Observations

Pechstein Reservoir Structural Exterior Observations		Photo
Roof	Typical deterioration of exposed wood at center vent (Photo 1).	Photo 1 
Roof	Typical condition of roof deck. Evidence of damage due to thermal movement including missing and replaced fasteners. Isolated locations where roof deck is not adequately supported due to damaged or displaced underlying roof framing members. Missing foam insulation (Photo 2).	Photo 2 

Table 4-2. Exterior Observations

Pechstein Reservoir Structural Exterior Observations		Photo
Roof Drains	Typical accumulation of debris at roof drains. Concentrated roof deck damage at these locations due to retained moisture. Evidence of prior repairs with isolated damage to adjacent roof deck due to workers walking on deck at these locations (Photo 3).	Photo 3 
Roof	Repaired valley girder at southeast quadrant (clockwise from access doorway). Active moisture with corrosion of painted metal components (Photo 4).	Photo 4 

Table 4-2. Exterior Observations

Pechstein Reservoir Structural Exterior Observations		Photo
Roof	Typical condition of wood framing at top of masonry wall. Active moisture with visible wood deterioration (Photo 5).	Photo 5 
Roof	Typical condition of valley girder. Deterioration and delamination of glu-lam beam with active moisture and staining visible (Photo 6).	Photo 6 
Roof	Typical condition of ridge girder. Deterioration and delamination of glu-lam beam with no active moisture visible (Photo 7).	Photo 7 

Table 4-2. Exterior Observations

Pechstein Reservoir Structural Exterior Observations		Photo
Wall	Typical condition of masonry wall at intermediate girder bearing (between ridge and valley girders). Cracking of stucco finish with active moisture and staining visible (Photo 8).	Photo 8 

Table 4-2. Exterior Observations

Pechstein Reservoir Structural Exterior Observations		Photo
Wall	Typical cracking in stucco finish at ridge and valley girder penetrations on masonry wall (Photo 9).	Photo 9 
Wall	Typical vertical cracking in gunite finish at concrete/masonry wall transition with continuous horizontal crack at joint between wall types (Photo 10).	Photo 10 

Table 4-2. Exterior Observations

Pechstein Reservoir Structural Exterior Observations		Photo
Wall	Typical wall staining below louvers (Photo 11).	Photo 11 

Table 4-3. Interior Observations

Pechstein Reservoir Structural Interior Observations		Photo
Roof	Typical condition of interior roof framing and support. Progressively more severe moisture accumulation (condensation) and corrosion of framing connections moving from ridge to valley (Photo 1).	Photo 1 

Table 4-3. Interior Observations

Pechstein Reservoir Structural Interior Observations		Photo
Roof/Wall	Typical condition of roof structure connection to reservoir wall (Photo 2).	Photo 2 
Roof	Girder to Girder connection in good condition at ridge girder near access platform (Photo 3).	Photo 3 

Table 4-3. Interior Observations

Pechstein Reservoir Structural Interior Observations		Photo
Access Stairs	Typical surface corrosion of interior metals (Photo 4).	Photo 4 

4.2.2 Conclusions

Due to extensive wood rot and uncertain footing along the ridgeline, only the area between the roof walkway and center vent was traversed. Based on inspection of the visible portions of Pechstein Reservoir at the time of the condition assessment, the following conclusions are made:

- The observed condition of the exposed wood at the center vent is expected of the age of its construction and does not appear to be affecting its function. All exposed wood should be recoated and, if necessary, replaced to ensure functionality of the vent structure.
- Damage due to thermal movement is typical of this type of roof deck. The deck connections to all supports should be routinely inspected and, if necessary, replaced, for safety and to ensure its functionality.
- Roof deck loading should be limited until all missing roof deck supports are replaced and deck attachments secured.
- All missing foam insulation should be replaced to restore weather-tightness of the reservoir.
- Current design of the roof drains allows accumulation of debris and moisture at the low points, leading to deterioration of the roof system at these locations. Leaking of the gutters is also contributing to the active moisture and damage observed at the valley girders. The roof drains should be redesigned to prevent these conditions.
- The major contributor to the deterioration of the roof framing is the lack of ventilation inside the reservoir. This lack of ventilation allows condensation to form on the roof components. The condensation accumulates as it travels from ridge to valley, causing progressively more severe damage with accumulation. The observed deterioration caused by this condition includes; deterioration of the rafters and valley girders with active moisture and staining (Table 4-2, Photos 4 – 6), cracking of the masonry wall stucco with active moisture and staining at the intermediate girder bearings (Table 4-2, Photo 8), corrosion and failure of wood connections at the interior of the reservoir (Table 4-3, Photos 1 – 2, and as described by VID staff). Previous repairs to the valley girders, to mitigate this deterioration, were observed. Without improvements to the ventilation, the observed deterioration will continue.
- The cracking of the stucco finish at girder penetrations is not a structural concern.
- The cracking of the gunite finish is not currently a structural concern, but should be monitored for corrosion of the embedded circumferential prestressed reinforcing.
- The staining of the exterior wall surface is not a structural concern and will be resolved with ventilation improvements.
- The surface staining and minor corrosion of the interior metals is not currently a structural concern, but should be monitored to ensure long-term serviceability of the metal components.

4.3 Recommendations

The following recommendations address the deficiencies noted during the field inspections. Section 4.3.1 and 4.3.2 include recommendations pertaining to minor maintenance, repair work and ongoing monitoring. Section 4.3.3 lists potentially larger scale improvements and recommendations for further investigation. An overall condition rating and prioritization of reservoir improvements is included in Section 12. Section 12 also contains proposed recommendation phasing, recommendations for additional assessment and a budgetary level opinion of cost summary for inclusion in the District's Capital Improvement Plan.

4.3.1 Civil/Site and Corrosion Recommendations

The following are civil site and corrosion improvement recommendations to be considered for the Pechstein Reservoir site:

- Continuous cracks along pavement along the access road is currently not of concern but should be monitored.
- Evidence of ponding in channel surrounding Pechstein Reservoir coming from roof gutters is not currently of concern but should be monitored and cleaned frequently to prevent additional vegetation growth.
- Areas of rusting on roof vent should be removed. Secure loose mesh surrounding roof vent.
- Rust spots on hatches should be removed. Chipped off paint from access door interior should be touched up.
- Install confined space signage on roof hatch. Use black text on signage to prevent fading.

4.3.2 Structural Recommendations

The following are structural improvement recommendations to be considered for the Pechstein Reservoir site:

- Repair (clean and recoat) and, if necessary, replace all deteriorated exposed wood.
- Replace all damaged or missing roof deck connections and their supports. Provide routine inspection for safety. Limit roof deck loading until all missing roof deck supports are replaced and deck attachments secured.
- Replace all missing foam insulation to restore the weather-tightness of the reservoir.
- Regularly monitor the cracking of the stucco finish for indication of moisture seepage and corrosion of the embedded circumferential prestressed reinforcing.
- Clean all staining of the exterior wall surface.
- Monitor all corrosion of the interior metal components.

4.3.3 Recommendations for Further Investigation

The following are potentially larger scale improvements and recommendations for further investigation for the Pechstein Reservoir site:

- Reconfigure the roof drains to prevent accumulation of debris and moisture at the low points and deterioration of the roof system at these locations.
- Provide ventilation improvements for the reservoir to prevent accumulation of condensation and deterioration of the roof framing and its connections.
- Perform a detailed condition assessment of the reservoir interior.
- Full reservoir roof replacement following the results of the detailed condition assessment of the reservoir interior.
- Perform a seismic evaluation of the reservoir to determine if seismic retrofit is a viable option to achieve compliance with the current seismic code.

5 HB Reservoir

HB Reservoir has a 4.05 MG operating capacity and is located at 3791 Buena Creek Road, Vista, CA 92084, as shown in Figure 5-1. HB Reservoir contains a concrete dome roof and was constructed in 1964. According to VID staff, issues with HB Reservoir consisted of evidence of floor leaking which was discovered in 1979. A liner was recommended and installed in 1987; the liner was inspected periodically and eventually replaced in 2002. VID staff also mentioned the near term additions to HB Reservoir which include footing and stairs. HDR's inspection of HB Reservoir, illustrated in Figure 5-2 and Figure 5-3, was conducted on November 15, 2016.

Figure 5-1. HB Reservoir Vicinity Map

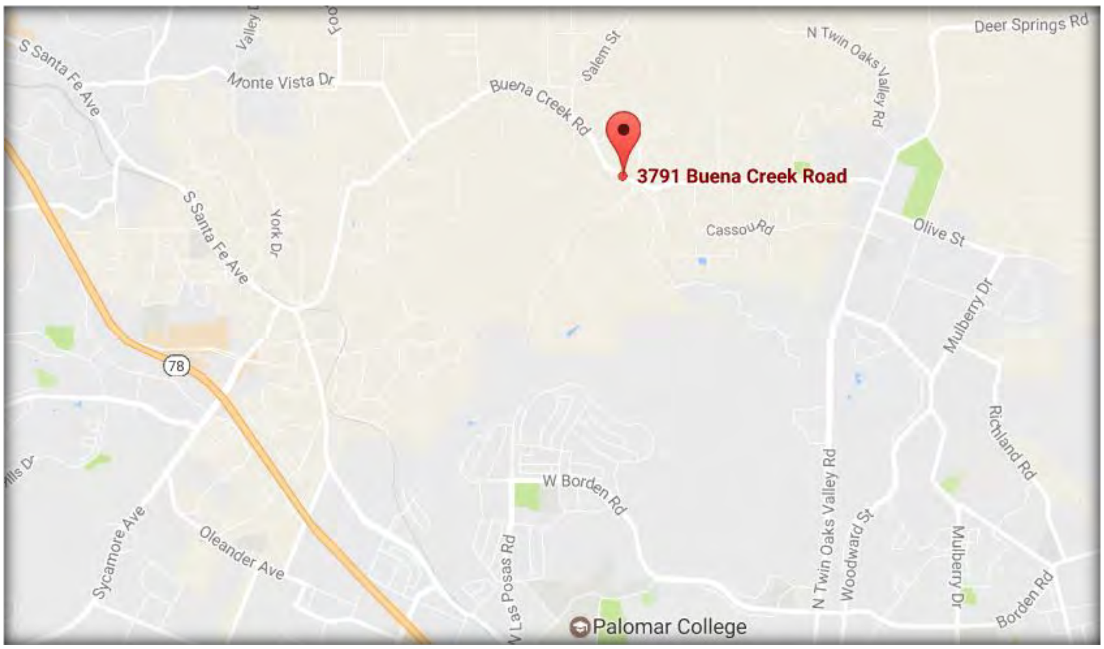


Figure 5-2. HB Reservoir

Figure 5-3. HB Reservoir Entrance

5.1 Typical Civil/Site and Corrosion Observations

Table 5-1. Civil/Site and Corrosion Observations

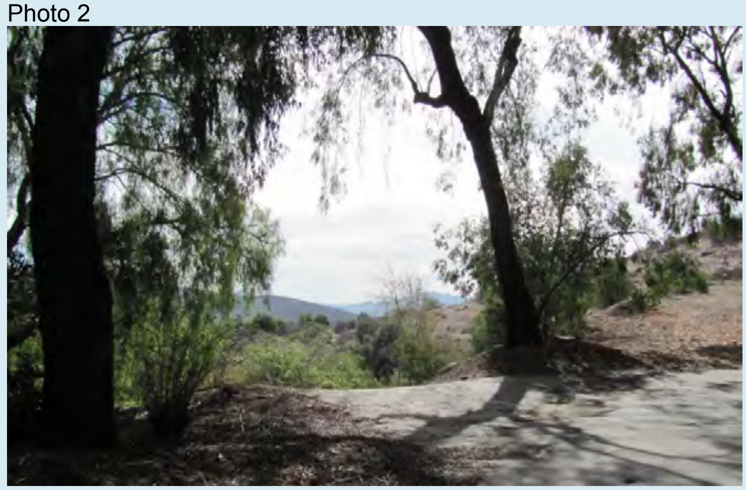
HB Reservoir Civil/Site and Corrosion Observations		Photo
Fence/Gate	Access gate consists of a single swing gate and lock. Gate is observed to be in good condition (Photo 1). No VID fence surrounding the perimeter of the reservoir. West side of reservoir contains fencing from adjacent property (Photo 5).	Photo 1 
Access Road	Paved access road leading up to and surrounding reservoir. Access road on north west side of entrance is completely accessible from the outside (Photo 2).	Photo 2 

Table 5-1. Civil/Site and Corrosion Observations

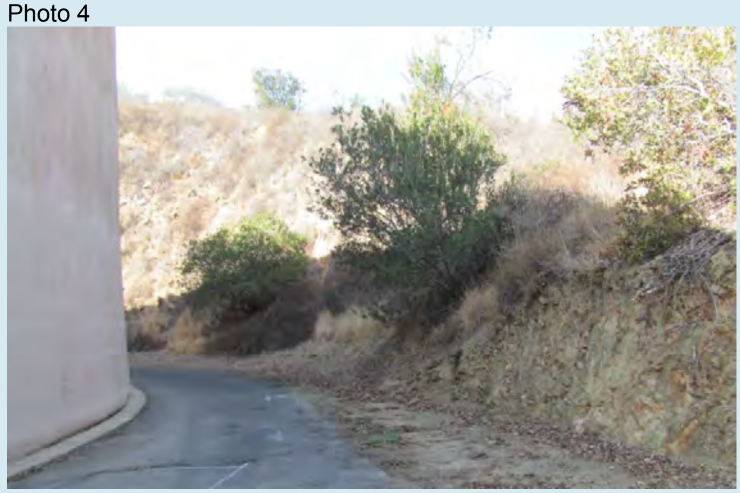
HB Reservoir Civil/Site and Corrosion Observations		Photo
Drainage	Roof drains located throughout perimeter of reservoir. Good slope for runoff into drain and surrounding vegetation (Photo 3).	Photo 3 
Trees and Vegetation	Typical trees and vegetation surrounding reservoir do not impose interference but could interfere with accessibility if not periodically maintained (Photo 4).	Photo 4 
	Trees and cactus located on west side of entrance along adjacent property (Photo 5).	Photo 5 

Table 5-1. Civil/Site and Corrosion Observations

HB Reservoir Civil/Site and Corrosion Observations		Photo
Vents	One roof vent is located at the center of the reservoir roof. Minor rusting developed near vent mesh. Roof vent observed to be in overall good condition (Photo 6 and Photo 7).	Photo 6  Photo 7 

Table 5-1. Civil/Site and Corrosion Observations

HB Reservoir Civil/Site and Corrosion Observations		Photo
Ladders	Typical exterior ladder observed to be in overall good condition (Photo 8). Exterior ladder at the time of inspections is scheduled to be replaced. Interior ladder was observed to have significant corrosion at anchor brackets due to galvanic coupling with stainless anchor bolts.	Photo 8 

Table 5-1. Civil/Site and Corrosion Observations

HB Reservoir Civil/Site and Corrosion Observations		Photo
Handrails, and Guardrails	Handrail and guardrail located at the reservoir were observed to be in good condition. Minor rusting on guardrail. (Photo 9) Handrail and guardrail at the time of inspections is scheduled to be replaced.	Photo 9 
Signage	Site signage includes signs located on the entrance gate. No signage located along the perimeter of the reservoir. Signage observed to be in good condition (Photo 10).	Photo 10 

5.1.1 Conclusions

Based on visual inspections of the civil/site and corrosion observations at HB Reservoir at the time of the condition assessment, the following conclusions are made:

- HB Reservoir has no security fence surrounding the perimeter of the reservoir. Only fencing on site consists of partial shared fencing from adjacent property on West side of reservoir. Lack of a security fence allows for easy unauthorized entry and vandalism.
- Access road on North West side of reservoir entrance is completely open and accessible from the outside.
- Appropriate signage shall be included with the installation of VID surrounding security fence.

- No confined space signage installed at the roof hatch location.
- The access road pavement is in good condition with minor cracking observed, displayed in Figure 5-4. Minor cracking is not currently a concern.
- Surrounding trees and vegetation out of vicinity of site operation and maintenance at the time of inspections but overgrown bushes could become an obstruction if not maintained periodically.
- Interior ladder has significant corrosion and metal loss at bracket locations.
- External pipe riser and blow-off in enclosure near site gate have surface rusting.

Figure 5-4. Pavement Cracking

5.2 Structural Observations and Conclusions

5.2.1 As-Built Drawings Review

Based on cursory review of the as-built drawings prepared for Vista Irrigation District, dated March 1963, the HB Reservoir is an above ground, circular shaped, prestressed concrete reservoir with a 5-inch thick reinforced concrete floor and a 10-inch thick reinforced concrete core wall with an exterior shotcrete layer providing cover for the circumferential prestressed reinforcing. The roof consists of a tapered reinforced concrete dome roof. The reservoir is approximately 160 feet in diameter with a maximum water depth of approximately 30 feet at overflow.

The original reservoir design did not include seismic cable system at the wall to foundation connection. Further analysis will be required to determine if the reservoir is in compliance with the current seismic code.

Table 5-2. Exterior Observations

HB Reservoir Structural Exterior Observations		Photo
Roof	Typical evidence of ponding at low points near roof drains at the outside edge of the roof. Invert of roof drain sits approximately 1 inch above the roof surface (Photo 1).	Photo 1 
Roof	Typical cracking on concrete roof. Horizontal and vertical cracking uniform over entire roof area (Photo 2).	Photo 2 
Roof	Typical crack width on concrete roof (Photo 3).	Photo 3 

Table 5-2. Exterior Observations

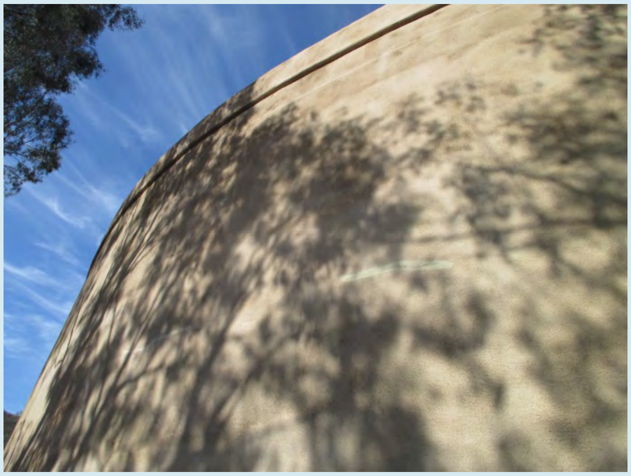
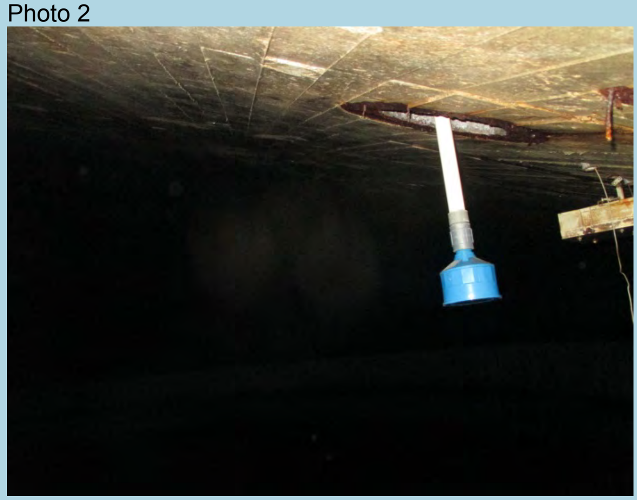
HB Reservoir Structural Exterior Observations		Photo
Roof	Exposed reinforcing at concrete curb at center vent (Photo 4).	Photo 4 
Wall	Previously repaired horizontal cracking in shotcrete finish at east side of reservoir (approximately 5 locations). No evidence of moisture seepage or corrosion of underlying prestressed reinforcing (Photo 5).	Photo 5 
Wall	Horizontal cracking in shotcrete finish with active moisture seepage at southwest and southeast reservoir quadrants (approximately 10 locations). No evidence of corrosion of underlying prestressed reinforcing (Photo 6).	Photo 6 

Table 5-2. Exterior Observations

HB Reservoir Structural Exterior Observations		Photo
Wall Foundation	Active moisture seepage at wall/foundation joint at south end of reservoir. Radial cracking in concrete foundation. This condition was observed along the entire perimeter of the reservoir at approximately 4-foot spacing (Photo 7).	Photo 7 
Foundation	Typical crack width in concrete foundation (Photo 8).	Photo 8 
Foundation	Typical scaling of top surface of concrete foundation (Photo 9).	Photo 9 

Table 5-3. Interior Observations

HB Reservoir Structural Interior Observations		Photo
Access Hatch	Exposed corroded reinforcing at roof opening (Photo 1).	Photo 1 
Roof	Typical severe corrosion of interior metals (Photo 2).	Photo 2 

5.2.2 Conclusions

Based on inspection of the visible portions of HB Reservoir at the time of the condition assessment, the following conclusions are made:

- The evidence of ponding observed on the roof is not a structural concern, but should be addressed to prevent long-term damage to the roof coating and concrete.
- The cracking of the roof concrete appears to have been present since construction, but should be addressed to prevent long-term damage to the concrete and embedded steel reinforcing due to moisture infiltration.
- The exposed reinforcing at the center vent curb appears to have been present since construction, but should be repaired to prevent long-term damage to the concrete and reinforcing due to moisture infiltration.

- The cracking of the shotcrete finish is not currently a structural concern, but should be monitored for corrosion of the embedded circumferential prestressed reinforcing. This condition likely indicates leaking in the reservoir's existing hypalon membrane liner.
- The cracking of the concrete foundation appears to have been present since construction, but should be addressed to prevent long-term damage to the concrete and embedded steel reinforcing due to moisture infiltration.
- The scaling of the concrete foundation is indicative of poor finishing methods of the concrete during construction. This is not currently a structural concern, but should be addressed to prevent more sever long-term damage to the concrete.
- The exposed corroded reinforcing in the concrete at the roof hatch opening is the result of insufficient concrete cover over the reinforcing during construction. This should be repaired to prevent long-term damage to the concrete.
- The severe corrosion of the interior metals should be addressed to maintain long-term serviceability of the metal components and the adjacent concrete.

5.3 Recommendations

The following recommendations address the deficiencies noted during the field inspections. Section 5.3.1 and 5.3.2 include recommendations pertaining to minor maintenance, repair work and ongoing monitoring. Section 5.3.3 lists potentially larger scale improvements and recommendations for further investigation. An overall condition rating and prioritization of reservoir improvements is included in Section 12. Section 12 also contains proposed recommendation phasing, recommendations for additional assessment and a budgetary level opinion of cost summary for inclusion in the District's Capital Improvement Plan.

5.3.1 Civil/Site and Corrosion Recommendations

The following are civil site and corrosion improvement recommendations to be considered for the HB Reservoir site:

- HB Reservoir has no security fence surrounding the perimeter of the reservoir. Only fencing on site consists of partial shared fencing from adjacent property on West side of reservoir. Full boundary chain link fence should be installed to prevent unauthorized entry and vandalism.
- Installation of full reservoir boundary chain link fence would prevent accessibility from the North West side.
- Install VID appropriate signage with the installation of VID security fence.
- Install confined space signage on roof hatch. Use black text on signage to prevent fading.
- Continuous cracks along pavement along the access road is currently not of concern but should be monitored.

- Trees and vegetation within vicinity of site operation and maintenance should be maintained.
- Replace corroded ladder brackets.
- Remove surface rust and repaint corroded areas on exterior piping in enclosure.

5.3.2 Structural Recommendations

The following are structural improvement recommendations to be considered for the HB Reservoir site:

- Modify the roof slope, as required, to prevent ponding and provide proper drainage.
- Seal all cracking on the roof concrete to prevent long-term damage to the concrete and embedded steel reinforcing due to moisture infiltration.
- Repair the exposed reinforcing at the center vent curb to prevent long-term damage to the concrete and reinforcing due to moisture infiltration.
- Regularly monitor the cracking of the shotcrete finish for indication of moisture seepage and corrosion of the embedded circumferential prestressed reinforcing.
- Seal all cracking and scaling of the foundation concrete to prevent long-term damage to the concrete and embedded steel reinforcing due to moisture infiltration.
- Repair the exposed corroded reinforcing in the concrete at the roof hatch opening to prevent long-term damage to the concrete.
- Monitor all corrosion of the interior metal components.

5.3.3 Recommendations for Further Investigation

The following are potentially larger scale improvements and recommendations for further investigation for the HB Reservoir site:

- Perform a detailed condition assessment of the reservoir interior.
- Perform a seismic evaluation of the reservoir to determine if seismic retrofit is a viable option to achieve compliance with the current seismic code.
- Repair leaks in reservoir liner following the results of a detailed condition assessment of the reservoir interior.

6 C Reservoir

C Reservoir has a 0.60 MG operating capacity and is located at 1301 Summit Terrace Vista, CA 92083, as shown in Figure 6-1. C Reservoir was constructed in 1926. According to VID staff, standard preventative maintenance was performed on C Reservoir. Due to wall joint leaks, it was decided in March 2014 to line the interior with CIM 1061 liner. C reservoir has had no further maintenance issues. HDR's inspection of C Reservoir, as illustrated in Figure 6-2, was conducted on November 15, 2016.

Figure 6-1. Reservoir Vicinity Map

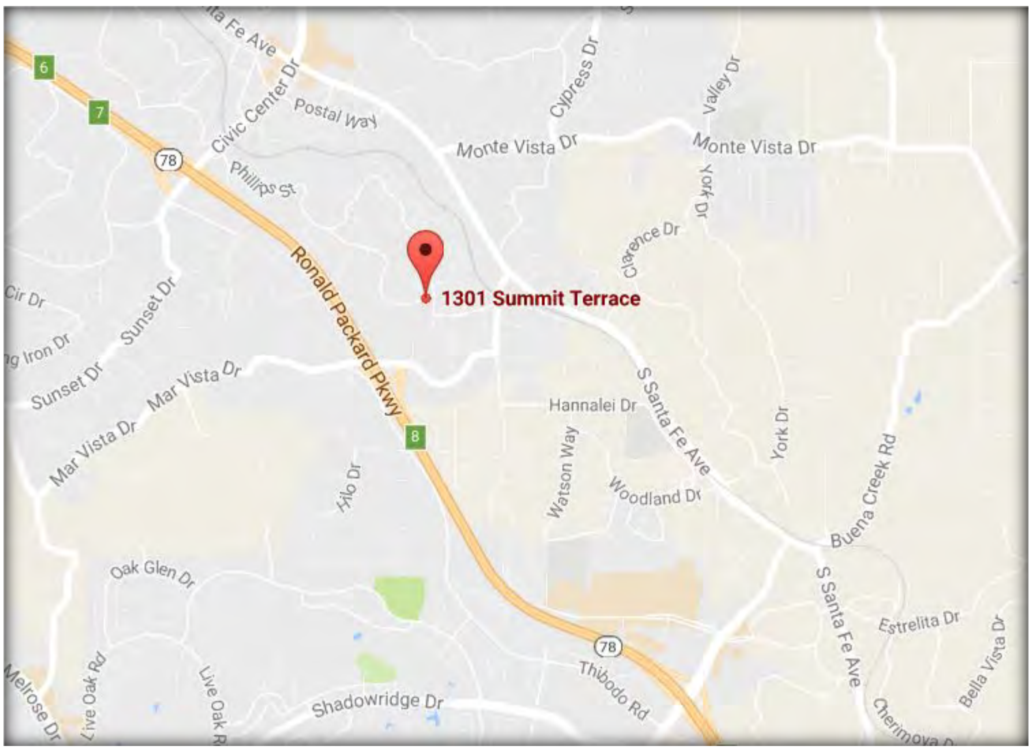


Figure 6-2. C Reservoir

6.1 Typical Civil/Site and Corrosion Observations

Table 6-1. Civil/Site and Corrosion Observations

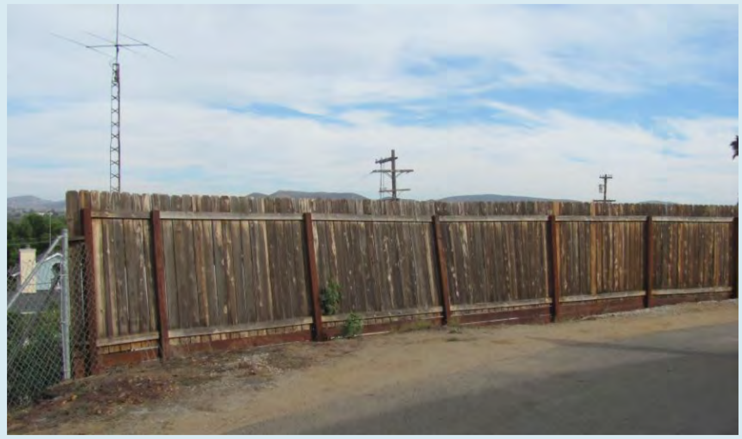
C Reservoir Civil/Site and Corrosion Observations		Photo
Fence/Gate	No VID main site access gate. VID fence does not surround entire perimeter of reservoir. Partial wood fence owned by homeowner separates adjacent properties (Photo 1). Perimeter fence would deny access to homeowners on the west side.	Photo 1 
Access Road	Paved access road provides adequate space for inspection teams and their vehicles (Photo 2). Paved access road contains areas of uneven surface and minor cracks.	Photo 2 

Table 6-1. Civil/Site and Corrosion Observations

C Reservoir Civil/Site and Corrosion Observations		Photo
Drainage	Paved gutter around perimeter of reservoir observed to be clear and good condition. Runoff slopes away from reservoir (Photo 3).	Photo 3 
Trees and Vegetation	Typical trees and vegetation along adjacent properties. No imposing accessibility interference (Photo 2). Minor shrubbery around perimeter of reservoir (Photo 4).	Photo 4 

Table 6-1. Civil/Site and Corrosion Observations

C Reservoir Civil/Site and Corrosion Observations		Photo
Hatches	Minor corrosion on interior of aluminum hatch on reservoir roof. Float box hatches found to be in good condition. Support beam inside float box found to be wrapped with tar tape (Photo 5).	Photo 5 

Table 6-1. Civil/Site and Corrosion Observations

C Reservoir Civil/Site and Corrosion Observations		Photo
Vents	One roof vent located at the center of the reservoir roof. Minor surface rusting on roof vent cap. Bug screen is loose and torn at one corner (Photo 5 and Photo 6).	Photo 5  Photo 6 
	Side vents are located around the perimeter every few feet apart. Vents have a mesh covering and wood frame. Vents were observed to be in good condition with cases of minor chipping and rusting (Photo 7).	Photo 7 

Table 6-1. Civil/Site and Corrosion Observations

C Reservoir Civil/Site and Corrosion Observations		Photo
Ladders	Typical exterior ladder in overall good condition (Photo 8).	Photo 8 
	Exterior fixed ladders are not OSHA compliant and should be further assessed (Photo 8 and Photo 9).	Photo 9 

Table 6-1. Civil/Site and Corrosion Observations

C Reservoir Civil/Site and Corrosion Observations		Photo
Handrails, and Guardrails	Guardrail is located on the reservoir roof at the hatch. Toe board is missing. The aluminum guardrails were observed to be in overall good condition (Photo 10).	Photo 10 
Signage	Site signage includes signs located on the fence surrounding ladder. No trespassing sign located along reservoir. All signage observed to be in good condition (Photo 11).	Photo 11 

6.1.1 Conclusions

Based on visual inspections of the civil/site and corrosion observations at C Reservoir at the time of the condition assessment, the following conclusions are made:

- C Reservoir has no security gate at the entrance on Summit Terrace. Currently, a partial wood fence separates the reservoir area to the surrounding properties. According to VID staff, unauthorized entry of C Reservoir was a concern in the past for surrounding property owners. Installation of a VID security gate and fence surrounding the perimeter would prevent illicit entry.
- Minor shrubbery and leaves scattered around perimeter of reservoir are not of concern but should be maintained periodically.
- Appropriate signage shall be included with the installation of the VID security gate and surrounding fence.
- Minor corrosion on the interior of the aluminum hatch on the reservoir roof.
- No confined space signage installed at the roof hatch location.
- Minor surface rusting on roof vent cap, bug screen is loose and torn at one corner.
- The fixed exterior ladders are not OSHA compliant and require further modification.
- Toe board missing, aluminum guardrail observed to be in good condition.
- The paved access road pavement contains areas of uneven surface and minor cracks.

6.2 Structural Observations and Conclusions

6.2.1 As-Built Drawings Review

Based on cursory review of the as-built drawings prepared by the Engineering Offices of J B Lippincott, dated August 1925, the C Reservoir is an above ground, circular shaped, cast-in-place reinforced concrete reservoir with a 4-inch thick reinforced concrete floor and a tapered reinforced concrete wall that is 16 inches thick at the base and 8 inches thick at the top. The wall is supported by a 32-inch wide by 12-inch thick continuous reinforced concrete footing. The roof consists of a wood rafter and girder system supported by 8-inch square precast reinforced concrete columns bearing on 2-foot square by 6-inch thick reinforced concrete footings. The reservoir is approximately 100 feet in diameter with a maximum water depth of approximately 13 feet at overflow. A 10-foot by 7-foot, 4-inch by 13-foot, 8-inch high control box is located at the southeast quadrant of the reservoir.

Considering C Reservoir's date and type of construction, it is not in compliance with the current seismic code.

Table 6-2. Exterior Observations

C Reservoir Structural Exterior Observations		Photo
Roof	Visible sagging of the roof paneling between the supports with evidence of ponding along the roof edge. Minor vertical deflection of the roof system felt while walking the roof (Photo 1).	Photo 1 
Wall	Typical staining of exterior wall surface from roof/wall interface (Photo 2).	Photo 2 

Table 6-2. Exterior Observations

C Reservoir Structural Exterior Observations		Photo
Wall	Typical full-height cracking in exterior face of concrete wall. This condition was observed along the entire perimeter of the reservoir at approximately 8-foot spacing. There does not appear to be any moisture seepage (Photo 3).	Photo 3 

Table 6-3. Interior Observations

C Reservoir Structural Interior Observations		Photo
Roof	Typical condition of interior roof deck, roof framing, and support (Photo 1).	Photo 1 
Roof/Wall	Typical condition of roof structure connection to reservoir wall (Photo 2).	Photo 2 
Control Box Roof	Typical condition of interior roof framing and underside of roof deck (Photo 3).	Photo 3 

6.2.2 Conclusions

Based on inspection of the visible portions of C Reservoir at the time of the condition assessment, the following conclusions are made:

- The sagging and evidence of ponding observed on the roof is expected of age and type of construction. Roof loading should be limited and roof condition monitored regularly for safety.
- The staining of the exterior wall surface is not a structural concern.
- The wall cracking appears to have been present since construction and is not a structural concern.
- Despite the age of the roof framing, it is in fair condition and appears to be functioning properly. As stated previously, the roof system is not in compliance with the current seismic code and should be monitored regularly for safety.

6.3 Recommendations

The following recommendations address the deficiencies noted during the field inspections. Section 6.3.1 and 6.3.2 include recommendations pertaining to minor maintenance, repair work and ongoing monitoring. Section 6.3.3 lists potentially larger scale improvements and recommendations for further investigation. An overall condition rating and prioritization of reservoir improvements is included in Section 12. Section 12 also contains proposed recommendation phasing, recommendations for additional assessment and a budgetary level opinion of cost summary for inclusion in the District's Capital Improvement Plan.

6.3.1 Civil/Site and Corrosion Recommendations

The following are civil site and corrosion improvement recommendations to be considered for C Reservoir site:

- C Reservoir's only means of security is a partial wood fence surrounding the reservoir. The fence is shared with adjacent property owners on the west side and owned by a homeowner. Perimeter fencing would deny access to homeowners on the west side.
- Trees and vegetation within vicinity of site operation and maintenance should be maintained.
- Install VID appropriate signage with the installation of VID security fence.
- Clean all staining on aluminum hatch on reservoir roof.
- Install confined space signage on roof hatch. Use black text on signage to prevent fading.
- Replace roof vents mesh covering and clean all surface rusting.
- The fixed exterior ladders are not OSHA compliant and require further modification.
- Install toe boards on guardrail system.

6.3.2 Structural Recommendations

The following are structural improvement recommendations to be considered for the C Reservoir site:

- Modify the roof slope, as required, to prevent ponding and provide proper drainage.
- Limit roof loading to two workers and fifty pounds of equipment and regularly monitor roof condition for safety.
- Clean all staining of the exterior wall surface.
- Seal all cracking in the exterior wall to prevent long-term damage to the concrete and embedded steel reinforcing due to moisture infiltration.
- Monitor all corrosion of the interior metal components.
- Clean corrosion and coat interior anchor bolts.

6.3.3 Recommendations for Further Investigation

The following are potentially larger scale improvements and recommendations for further investigation for the C Reservoir site:

- Perform a seismic evaluation of the reservoir to determine if seismic retrofit is a viable option to achieve compliance with the current seismic code.

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7 E1 Reservoir

E1 Reservoir has a 0.50 MG operating capacity and is located at 1122 Cabrillo Circle Vista, CA 92084, as shown in Figure 7-1. E1 Reservoir was constructed in 1926. According to VID staff, standard preventative maintenance was performed on E1. Due to wall and floor joint leaks, it was decided in April 2016 to line the interior with Warren Environmental S-301-01 NSF approved epoxy liner. E1 has had the roof resurfaced several times over the years due to minor termite damage to roof lumber. No further maintenance issues were noted. HDR's inspection of E1 Reservoir, as illustrated in Figure 7-2 and Figure 7-3, was conducted on November 15, 2016.

Figure 7-1. E1 Reservoir Vicinity Map

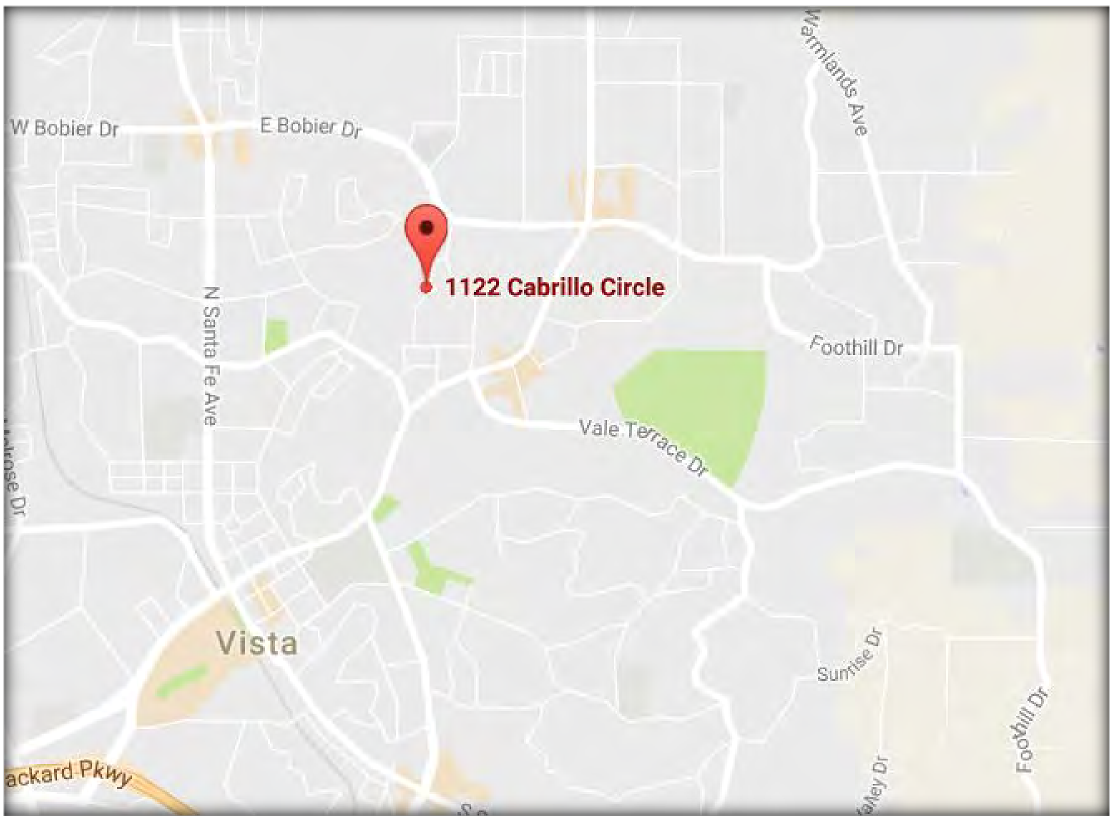


Figure 7-2. E1 Reservoir

Figure 7-3. E1 Reservoir Entrance

7.1 Typical Civil/Site and Corrosion Observations

Table 7-1. Civil/Site and Corrosion Observations

E1 Reservoir Civil/Site and Corrosion Observations		Photo
Fence/Gate	Typical VID chain link fence surrounds entire perimeter of reservoir. Fence and gate observed to be in good condition (Photo 1).	Photo 1 
Access Road	Tree growth along fence observed (Photo 2 and Photo 4). Paved access road and a set of stairs lead to the reservoir entrance. Dirt surrounds boundary of reservoir (Photo 2). Adequate space for inspection teams and their vehicles is located on paved access road.	Photo 2 

Table 7-1. Civil/Site and Corrosion Observations

E1 Reservoir Civil/Site and Corrosion Observations		Photo
Drainage	Runoff drains to surrounding dirt area. Good slope through vegetation and down to access road. No roof or boundary gutters present (Photo 3)	Photo 3 

Table 7-1. Civil/Site and Corrosion Observations

E1 Reservoir Civil/Site and Corrosion Observations		Photo
Trees and Vegetation	Tree growth over a majority of the fence all along west side. Imposing accessibility interference (Photo 4).	Photo 4 
	Two large trees located within five feet of reservoir and exterior ladder on the east side (Photo 5).	Photo 5 

Table 7-1. Civil/Site and Corrosion Observations

E1 Reservoir Civil/Site and Corrosion Observations		Photo
Hatches	Surface rusting observed on lock cover of roof hatch. Corroded screws on latch mechanism. Minor staining on interior of aluminum hatch (Photo 6).	Photo 6 

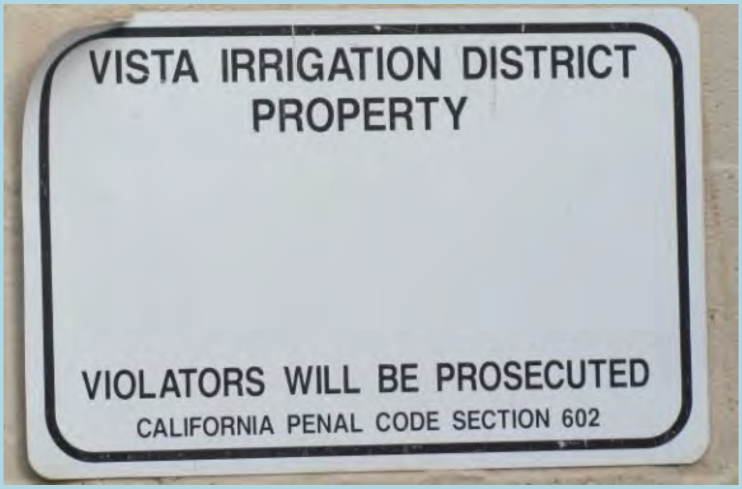
Table 7-1. Civil/Site and Corrosion Observations

E1 Reservoir Civil/Site and Corrosion Observations		Photo
Vents	One roof vent located at the center of the reservoir roof. Cracks in caulking at vent bottom. Center vent observed to be in overall good condition (Photo 7 and Photo 8).	Photo 7  Photo 8 
	Side vents are located around the perimeter every few feet apart. Vents have a mesh covering and wood frame. Vents were observed to be in good condition with cases of rusting and chipping on wood frame (Photo 9).	Photo 9 

Table 7-1. Civil/Site and Corrosion Observations

E1 Reservoir Civil/Site and Corrosion Observations		Photo
Ladders	Typical exterior ladder has minor surface rusting along brackets and hinge (Photo 10).	Photo 10 
	Exterior ladders first step bracket is placed too high (Photo 11).	Photo 11 

Table 7-1. Civil/Site and Corrosion Observations

E1 Reservoir Civil/Site and Corrosion Observations		Photo
Handrails, and Guardrails	Guardrail is located on the reservoir roof at the hatch. Missing toe boards on roof (Photo 12).	Photo 12 
Signage	Site signage includes signs located on the fence and along reservoir. Red text from signs has completely faded (Photo 13).	Photo 13 

7.1.1 Conclusions

Based on visual inspections of the civil/site and corrosion observations at E1 Reservoir at the time of the condition assessment, the following conclusions are made:

- Security fence surrounding E1 Reservoir contains significant tree growth over a majority of the fence along the west side. Continuous tree growth could impose accessibility interference.
- Two large trees are located within five feet of E1 Reservoir on the east side. Branches and leaves from the tree obstruct the top vicinity of the ladder. Recurrent growth of tree branches and leaves would obstruct the roof ladder and could ultimately cause a security concern.

- Surface rusting and corroded screws were found on the lock cover for the roof hatch.
- Minor cracking in the caulking of the roof vent bottom.
- Exterior roof ladders first step bracket is placed too high.
- Signage located surrounding reservoir observed to be aging with completely faded text.
- Toe board missing, aluminum guardrail observed to be in good condition.
- No confined space signage installed at the roof hatch location.

7.2 Structural Observations and Conclusions

7.2.1 As-Built Drawings Review

Based on cursory review of the as-built drawings prepared by the Engineering Offices of J B Lippincott, dated August 1925, the E1 Reservoir is an above ground, circular shaped, cast-in-place reinforced concrete reservoir with a 4-inch thick reinforced concrete floor and a tapered reinforced concrete wall that is 16 inches thick at the base and 8 inches thick at the top. The wall is supported by a 32-inch wide by 12-inch thick continuous reinforced concrete footing. The roof consists of a wood rafter and girder system supported by 8-inch square precast reinforced concrete columns bearing on 2-foot square by 6-inch thick reinforced concrete footings. The reservoir is approximately 90 feet in diameter with a maximum water depth of approximately 13 feet at overflow. A 10-foot by 7-foot, 4-inch by 13-foot, 8-inch high control box is located at the northwest quadrant of the reservoir.

Considering E1 Reservoir's date and type of construction, it is not in compliance with the current seismic code.

Table 7-2. Exterior Observations

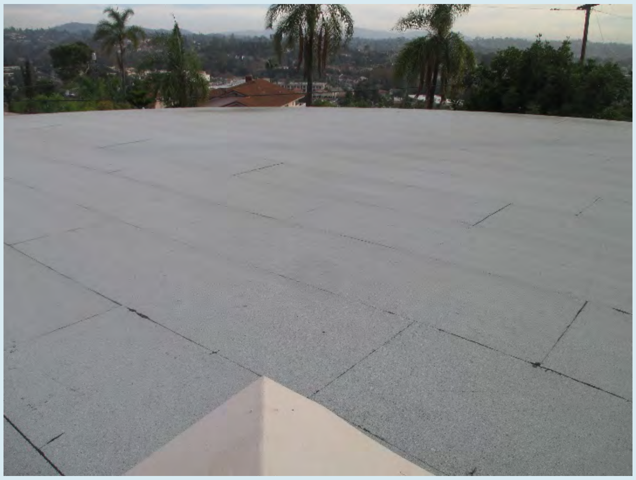
E1 Reservoir Structural Exterior Observations		Photo
Roof	Visible sagging of the roof paneling between the supports with evidence of ponding along the roof edge. Minor vertical deflection of the roof system felt while walking the roof (Photo 1).	Photo 1 
Wall	Typical staining of exterior wall surface from roof/wall interface (Photo 2).	Photo 2 
Wall	Evidence of moisture seepage from horizontal cold joint in exterior wall (Photo 3).	Photo 3 

Table 7-2. Exterior Observations

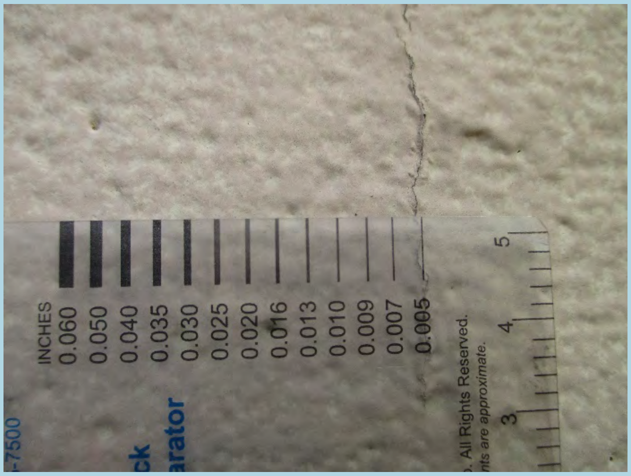
E1 Reservoir Structural Exterior Observations		Photo
Wall	Typical full-height cracking in exterior face of concrete wall. This condition was observed along the entire perimeter of the reservoir at approximately 8-foot spacing. There does not appear to be any moisture seepage (Photo 4).	Photo 4 

Table 7-3. Interior Observations

E1 Reservoir Structural Interior Observations		Photo
Roof	Typical condition of interior roof deck, roof framing, and support (Photo 1).	Photo 1 
Roof/Wall	Typical condition of roof structure connection to reservoir wall (Photo 2).	Photo 2 
Control Box Roof	Typical condition of interior roof framing and underside of roof deck. Severe corrosion with section loss on interior framing (Photo 3).	Photo 3 

7.2.2 Conclusions

Based on inspection of the visible portions of E1 Reservoir at the time of the condition assessment, the following conclusions are made:

- The sagging and evidence of ponding observed on the roof is expected of age and type of construction. Roof loading should be limited and roof condition monitored regularly for safety.
- The staining of the exterior wall surface is not a structural concern.
- VID staff indicated presence of moisture seepage in horizontal cold joint prior to installation of current liner system so it is not a structural concern.
- The wall cracking appears to have been present since construction and is not a structural concern.
- Despite the age of the roof framing, it is in fair condition and appears to be functioning properly. As stated previously, the roof system is not in compliance with the current seismic code and should be monitored regularly for safety.
- The corrosion of the interior metals is not currently a structural concern, but should be addressed to maintain long-term serviceability of the control box roof.

7.3 Recommendations

The following recommendations address the deficiencies noted during the field inspections. Section 7.3.1 and 7.3.2 include recommendations pertaining to minor maintenance, repair work and ongoing monitoring. Section 7.3.3 lists potentially larger scale improvements and recommendations for further investigation. An overall condition rating and prioritization of reservoir improvements is included in Section 12. Section 12 also contains proposed recommendation phasing, recommendations for additional assessment and a budgetary level opinion of cost summary for inclusion in the District’s Capital Improvement Plan.

7.3.1 Civil/Site and Corrosion Recommendations

The following are civil site and corrosion improvement recommendations to be considered for the E1 Reservoir site:

- Remove 2 large trees located on east side of fence and all vegetation within five feet from fence to prevent accessibility interference.
- Replace corroded screwed on lock cover of roof hatch.
- Caulk areas of cracking in the bottom area of the roof vent.
- Exterior ladder’s first step rung should be placed such that it is compliant with OSHA fixed ladder requirements.
- Replace faded signage located surrounding reservoir.
- Install toe boards on guardrail system.

- Install confined space signage on roof hatch. Use black text on signage to prevent fading.

7.3.2 Structural Recommendations

The following are structural improvement recommendations to be considered for the E1 Reservoir site:

- Modify the roof slope, as required, to prevent ponding and provide proper drainage.
- Limit roof loading to two workers and fifty pounds of equipment and regularly monitor roof condition for safety.
- Clean all staining of the exterior wall surface.
- Seal all cracking in the exterior wall to prevent long-term damage to the concrete and embedded steel reinforcing due to moisture infiltration.
- Monitor all corrosion of the interior metal components.
- Clean corrosion and coat interior anchor bolts.
- Clean corrosion and coat interior roof beams.

7.3.3 Recommendations for Further Investigation

The following are potentially larger scale improvements and recommendations for further investigation for the E1 Reservoir site:

- Perform a seismic evaluation of the reservoir to determine if seismic retrofit is a viable option to achieve compliance with the current seismic code.

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8 San Luis Rey Reservoir

San Luis Rey Reservoir has a 2.70 MG operating capacity and is located at 1700 Anza Avenue, Vista, CA 92084, as shown in Figure 8-1. San Luis Rey is an underground reservoir that was constructed in 1978. According to VID staff there has been no major maintenance since the time of construction. HDR's inspection of San Luis Rey Reservoir, as illustrated in Figure 8-2, was conducted on November 16, 2016.

Figure 8-1. San Luis Rey Vicinity Map

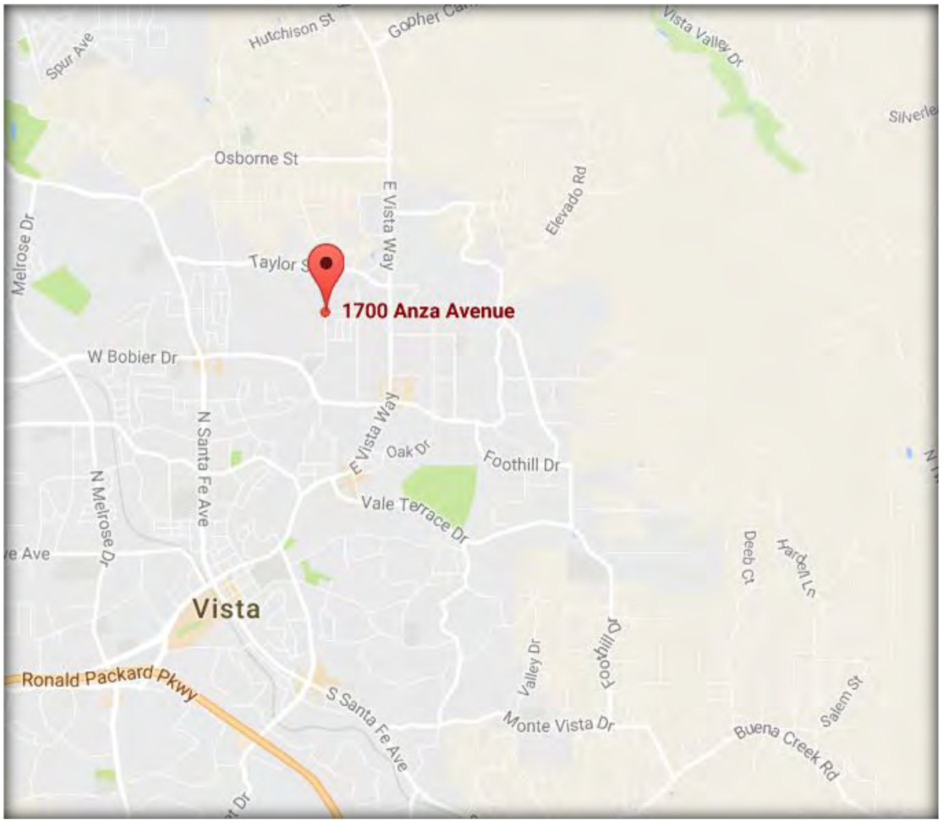


Figure 8-2. San Luis Rey Reservoir Site

8.1 Typical Civil/Site and Corrosion Observations

Table 8-1. Civil/Site and Corrosion Observations

San Luis Rey Reservoir Civil/Site and Corrosion Observations		Photo
Fence/Gate	Typical chain link gate located at entrance in good condition. Fence partially surrounding perimeter of reservoir (Photo 1).	Photo 1 
Access Road	Paved access road following entrance leads up to reservoir (Photo 2). Access road area contains sufficient space for inspection teams and multiple vehicles	Photo 2 
	Access road has areas of minor cracking along pavement but was observed to be in overall good condition (Photo 3).	Photo 3 

Table 8-1. Civil/Site and Corrosion Observations

San Luis Rey Reservoir Civil/Site and Corrosion Observations		Photo
Trees and Vegetation	Typical trees and vegetation outside of site area. Cactus off site between reservoir and adjacent property (Photo 4).	Photo 4 
	Shrubbery located on top of reservoir appears to be well maintained (Photo 5).	Photo 5 
Vents	One roof vent is located at the side of the reservoir roof. Vent contains areas of minor rusting but is in overall good condition (Photo 6).	Photo 6 

Table 8-1. Civil/Site and Corrosion Observations

San Luis Rey Reservoir Civil/Site and Corrosion Observations		Photo
Signage	Signage on site included no trespassing and caution signs (Photo 7).	Photo 7 
	Private Property sign faded and illegible (Photo 8).	Photo 8 

8.1.1 Conclusions

Based on visual inspections of the civil/site and corrosion observations at San Luis Rey Reservoir at the time of the condition assessment, the following conclusions are made:

- San Luis Rey Reservoir contains security fence partially surrounding perimeter of reservoir. Security fence is adjacent to the surrounding properties.
- Shrubbery located on top of reservoir at the time of inspections was well maintained. Erosion surrounding reservoir perimeter along access road is currently not of concern. Regular upkeep of surrounding areas would prevent potential obstruction with site access road.
- Signage on site had cases of fading and aging.
- Significant corrosion was observed on entry hatch locks and hinge hardware.
- Access road has areas of minor cracking along pavement but was observed to be in overall good condition.

8.2 Structural Observations and Conclusions

8.2.1 As-Built Drawings Review

Based on cursory review of the as-built drawings prepared by James M Montgomery Consulting Civil Engineers, Inc., dated October 1976, the San Luis Rey Reservoir is a buried, rectangular shaped, hopper-bottom, cast-in-place reinforced concrete reservoir with a 6-inch thick reinforced concrete floor and 14-inch thick reinforced concrete walls. The roof consists of an 8½ -inch thick reinforced concrete roof supported by 20-inch diameter reinforced concrete columns bearing on 7-foot square reinforced concrete footings. The reservoir is approximately 138 feet wide by 158 long with a maximum water depth of approximately 25 feet at overflow.

Further analysis will be required to determine if the reservoir is in compliance with the current seismic code.

Table 8-2. Exterior Observations

San Luis Rey Reservoir Structural Exterior Observations		Photo
General	Typical exterior view of reservoir (Photo 1).	Photo 1 
Roof	Typical cracking at the corners of the concrete curb around the access hatch (Photo 2).	Photo 2 

Table 8-3. Interior Observations

San Luis Rey Reservoir Structural Interior Observations		Photo
Access Hatch	Typical interior view of access hatch. Corrosion of abandoned anchors in concrete. Surface corrosion of interior metals. Efflorescence observed at joint between roof concrete and hatch riser (Photo 1).	Photo 1 

8.2.2 Conclusions

Based on inspection of the visible portions of San Luis Rey Reservoir at the time of the condition assessment, the following conclusions are made:

- The concrete cracking around the access hatch appears to have been present since construction and is not a structural concern.
- The corrosion of the abandoned anchors in the concrete should be addressed to prevent long-term damage to the concrete and embedded steel reinforcing due to moisture infiltration.
- The minor corrosion of the interior metals is not currently a structural concern, but should be addressed to maintain long-term serviceability of the metal components.

8.3 Recommendations

The following recommendations address the deficiencies noted during the field inspections. Section 8.3.1 and 8.3.2 include recommendations pertaining to minor maintenance, repair work and ongoing monitoring. Section 8.3.3 lists potentially larger scale improvements and recommendations for further investigation. An overall condition rating and prioritization of reservoir improvements is included in Section 12. Section 12 also contains proposed recommendation phasing, recommendations for additional assessment and a budgetary level opinion of cost summary for inclusion in the District's Capital Improvement Plan.

8.3.1 Civil/Site and Corrosion Recommendations

The following are civil site and corrosion improvement recommendations to be considered for the San Luis Rey Reservoir site:

- Consider installation of VID security fence surrounding perimeter of reservoir.
- Remove all vegetation in close proximity to the reservoir fence and signage and clear sediment and debris from the site structures.
- Replace faded VID signage. Black text signage is recommended as red text fades in due time.
- Replace corroded hardware on both hatches including prop bar, hinge screws, and lock cover screws.

8.3.2 Structural Recommendations

The following are structural improvement recommendations to be considered for the San Luis Rey Reservoir site:

- Seal all cracking in the concrete around the access hatch to prevent long-term damage to the concrete and embedded steel reinforcing due to moisture infiltration.
- Repair or remove corroding abandoned anchors to prevent long-term damage to the concrete and reinforcing due to moisture infiltration.
- Clean all corrosion of the interior metal components.

8.3.3 Recommendations for Further Investigation

The following are potentially larger scale improvements and recommendations for further investigation for the San Luis Rey Reservoir site:

- Perform a detailed condition assessment of the reservoir interior.
- Perform a seismic evaluation of the reservoir to determine if seismic retrofit is a viable option to achieve compliance with the current seismic code.

9 H Reservoir

H Reservoir has a 5.00 MG operating capacity and is located at 2082 Pleasant Heights Drive, Vista, CA 92084, as shown in Figure 9-1. H Reservoir is a prestressed concrete reservoir that was constructed in 1997. According to VID staff there has been no major maintenance since the time of construction. HDR's inspection of H Reservoir, as illustrated in Figure 9-2, was conducted on November 16, 2016.

Figure 9-1. H Reservoir Vicinity Map

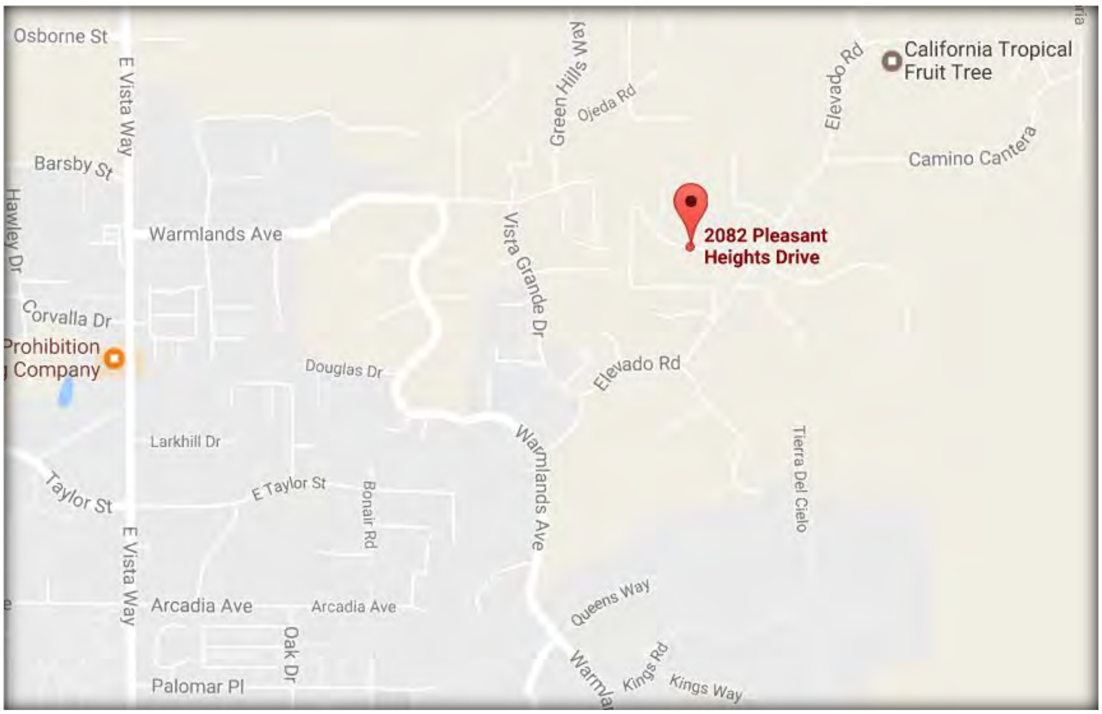


Figure 9-2. H Reservoir

9.1 Typical Civil/Site and Corrosion Observations

Table 9-1. Civil/Site and Corrosion Observations

H Reservoir Civil/Site and Corrosion Observations		Photo
Fence/Gate	Typical rolling steel gate with lock and signage located at entrance in good condition (Photo 1).	Photo 1 
Access Road	Site perimeter is enclosed by a chain link fence. The fence contains overgrown shrubbery in certain areas but is in overall good condition (Photo 2). Paved access road leads up to entrance and contains adequate space for maintenance crew. Access road surrounding reservoir has cases of uneven pavement and cracking (Figure 9-3 and Figure 9-3).	Photo 2 
Drainage	Runoff from roof drains spill into surrounding vegetation. Concrete lined ditch surrounding reservoir boundary contains minor sediments but was in overall good condition (Photo 3).	Photo 3 

Table 9-1. Civil/Site and Corrosion Observations

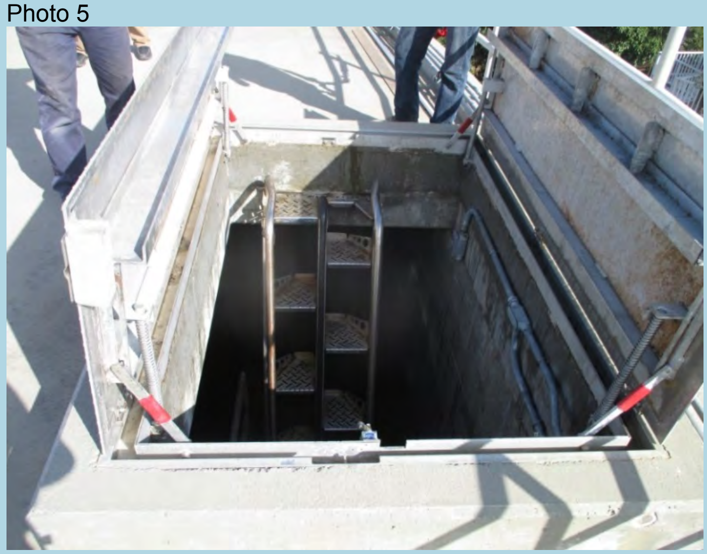
H Reservoir Civil/Site and Corrosion Observations		Photo
Trees and Vegetation	Tree growth over fence all along NE side. Shrubbery and weeds intertwined into links of fence (Photo 4).	Photo 4 
Hatches	Hatches interior conduit brackets were found to be corroded. Hatch observed to be in overall good condition (Photo 5).	Photo 5 

Table 9-1. Civil/Site and Corrosion Observations

H Reservoir Civil/Site and Corrosion Observations		Photo
Vents	One roof vent located at the center of the reservoir roof. Center vent assembly was observed to be in good condition (Photo 6 and Photo 7).	Photo 6  Photo 7 

Table 9-1. Civil/Site and Corrosion Observations

H Reservoir Civil/Site and Corrosion Observations		Photo
Ladders	Stainless steel interior ladder located in pump station room contains concrete landing prior to first step. Ladder in good condition (Photo 8).	Photo 8 

Table 9-1. Civil/Site and Corrosion Observations

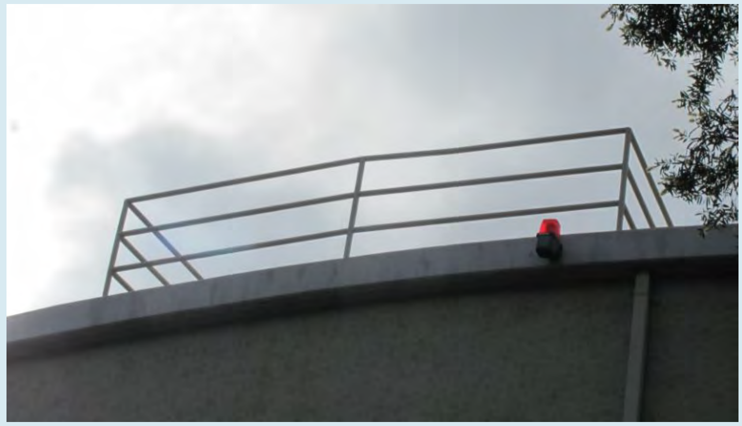
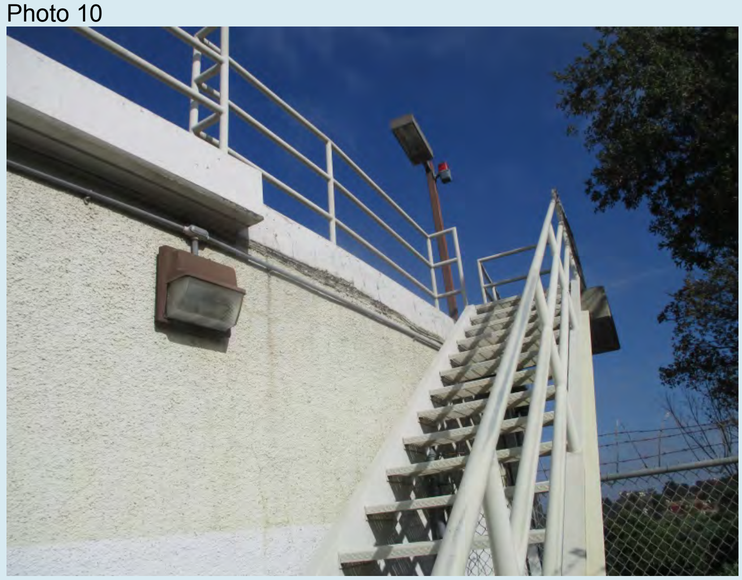
H Reservoir Civil/Site and Corrosion Observations		Photo
Handrails, and Guardrails	There are a total of three guardrails located at H Reservoir. Two located on the roof and one on landing inside (Photo 9).	Photo 9 
	There are a total of three stairways. Stairway on NE side of reservoir entrance is missing inner handrail but was observed to be in good condition overall (Photo 10).	Photo 10 

Table 9-1. Civil/Site and Corrosion Observations

H Reservoir Civil/Site and Corrosion Observations		Photo
Signage	Signage on site includes entrance, no trespassing and caution signs (Photo 11).	Photo 11 

9.1.1 Conclusions

Based on visual inspections of the civil/site and corrosion observations at H Reservoir at the time of the condition assessment, the following conclusions are made:

- The chain link fence contains tree growth over fence all along NE side of reservoir. Shrubbery and weeds are intertwined into links of fence.
- Access road surrounding reservoir has several areas of uneven pavement and cracking displayed in Figure 9-3 and Figure 9-4.
- Minor sediment and debris built up in concrete lined ditch. Clear ditch allows for adequate run off flow.
- Stairway on NE side of reservoir entrance is missing inner handrail.
- Corrosion on conduit fittings on roof and inside access hatch.
- Spot corrosion was observed on the interior overflow structure.
- No confined space signage installed at the roof hatch location.

Figure 9-3. Pavement Damage

Figure 9-4. Pavement Cracks

9.2 Structural Observations and Conclusions

9.2.1 As-Built Drawings Review

Based on cursory review of the as-built drawings prepared by John Powell & Associates, Inc., Consulting Civil Engineers, dated April 1995, the H Reservoir is a partially buried, circular shaped, prestressed concrete reservoir with a 6-inch thick reinforced concrete floor and a 12-inch thick reinforced concrete core wall with an exterior, fiber-reinforced shotcrete layer providing cover for the circumferential prestressed reinforcing. The roof consists of a 9-inch thick reinforced concrete roof supported by 24-inch diameter reinforced concrete columns bearing on 10-foot square by 20-inch thick reinforced concrete footings. The reservoir is approximately 160 feet in diameter with a maximum water depth of approximately 36 feet at overflow.

The original reservoir design included seismic cable system at the wall to foundation connection, but further analysis will be required to determine if the reservoir is in compliance with the current seismic code.

Table 9-2. Exterior Observations

H Reservoir Structural Exterior Observations		Photo
Roof	Typical cracking on concrete roof, concentrated over column caps (Photo 1).	Photo 1 

Table 9-2. Exterior Observations

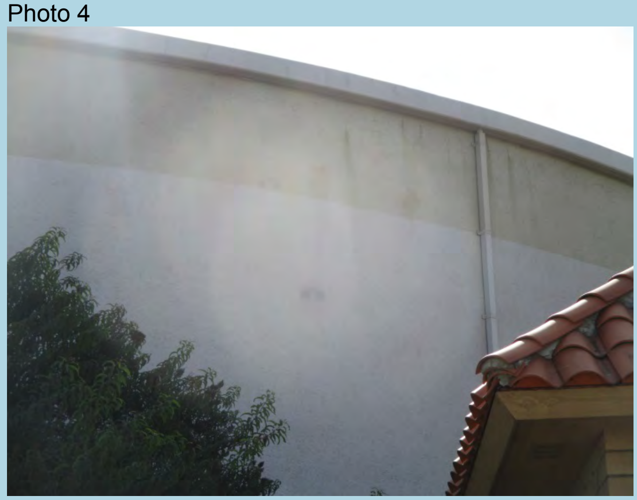
H Reservoir Structural Exterior Observations		Photo
Roof	Typical crack width on concrete roof (Photo 2).	Photo 2 
Roof	Typical vertical cracking on outside face of concrete roof curb (approximately 4-foot spacing) (Photo 3).	Photo 3 
Wall	Typical cracking of shotcrete finish near the roof/wall joint (multiple locations). Typical staining of exterior wall surface at these locations (Photo 4).	Photo 4 

Table 9-2. Exterior Observations

H Reservoir Structural Exterior Observations		Photo
Wall	Typical pattern cracking of shotcrete finish (approximately 12-inch grid) (Photo 5).	Photo 5 
Wall	Typical crack width on shotcrete finish (Photo 6).	Photo 6 

Table 9-3. Interior Observations

H Reservoir Structural Interior Observations		Photo
General	Typical condition at reservoir interior (Photo 1).	Photo 1 
Access Stairs	Typical surface corrosion of interior metals (Photo 2).	Photo 2 

9.2.2 Conclusions

Based on inspection of the visible portions of H Reservoir at the time of the condition assessment, the following conclusions are made:

- The cracking on the roof deck appears to have been present since construction, but should be addressed to prevent long-term damage to the concrete and embedded steel reinforcing due to moisture infiltration.
- The cracking on the outside face of the roof deck appears to have been present since construction and is not a structural concern.
- The cracking of the shotcrete finish is not currently a structural concern, but should be monitored for indication of moisture seepage and corrosion of the embedded circumferential prestressed reinforcing. The cracking near the top of the wall is above the high water line, so the moisture staining at these locations may be an

indication of moisture penetrating the roof/wall joint and running down the exterior face of the wall.

- The minor corrosion of the interior metals is not currently a structural concern, but should be addressed to maintain long-term serviceability of the metal components.

9.3 Recommendations

The following recommendations address the deficiencies noted during the field inspections. Section 9.3.1 and 9.3.2 include recommendations pertaining to minor maintenance, repair work and ongoing monitoring. Section 9.3.3 lists potentially larger scale improvements and recommendations for further investigation. An overall condition rating and prioritization of reservoir improvements is included in Section 12. Section 12 also contains proposed recommendation phasing, recommendations for additional assessment and a budgetary level opinion of cost summary for inclusion in the District's Capital Improvement Plan.

9.3.1 Civil/Site and Corrosion Recommendations

The following are civil site and corrosion improvement recommendations to be considered for the H Reservoir site:

- Remove all vegetation in close proximity to the reservoir, fence, and other structures. Remove all shrubbery and weeds intertwined into links of fence.
- Repair uneven pavement and pavement cracking in surrounding access road. Cracking and uneven pavement displayed in Figure 9-1 and Figure 9-3.
- Clear minor sediment and debris built up in concrete lined ditch.
- Install missing inner handrail on exterior stairs.
- Replace corroded conduit fittings on roof and inside access hatch.
- Remove corrosion and repair coating on interior overflow structure.
- Install confined space signage on roof hatch. Use black text on signage to prevent fading.

9.3.2 Structural Recommendations

The following are structural improvement recommendations to be considered for the H Reservoir site:

- Repair the roof, as required, to prevent ponding and provide proper drainage.
- Seal all cracking on the roof concrete to prevent long-term damage to the concrete and embedded steel reinforcing due to moisture infiltration.
- Regularly monitor the cracking of the shotcrete finish for indication of moisture seepage and corrosion of the embedded circumferential prestressed reinforcing.
- Monitor all corrosion of the interior metal components.

9.3.3 Recommendations for Further Investigation

The following are potentially larger scale improvements and recommendations for further investigation for the H Reservoir site:

- Perform a detailed condition assessment of the reservoir interior.
- Perform a seismic evaluation of the reservoir to determine if seismic retrofit is a viable option to achieve compliance with the current seismic code.

10 MD Reservoir

MD Reservoir has a 0.19 MG operating capacity and is located at 1961 Rockhoff Road, Escondido, CA 92026, as shown in Figure 10-1. MD Reservoir was constructed in 1926. According to VID staff, only standard preventative maintenance has occurred on MD Reservoir since the time of construction. A product called RAMNEK was used on the wall joint leaks. No further leaks occurred following use of RAMNEK. MD Reservoir has had its roof resurfaced several times since construction due to a minor case of termite damage on the roof lumber. HDR's inspection of MD Reservoir, as illustrated in Figure 10-2, was conducted on November 16, 2016.

Figure 10-1. MD Reservoir Vicinity Map

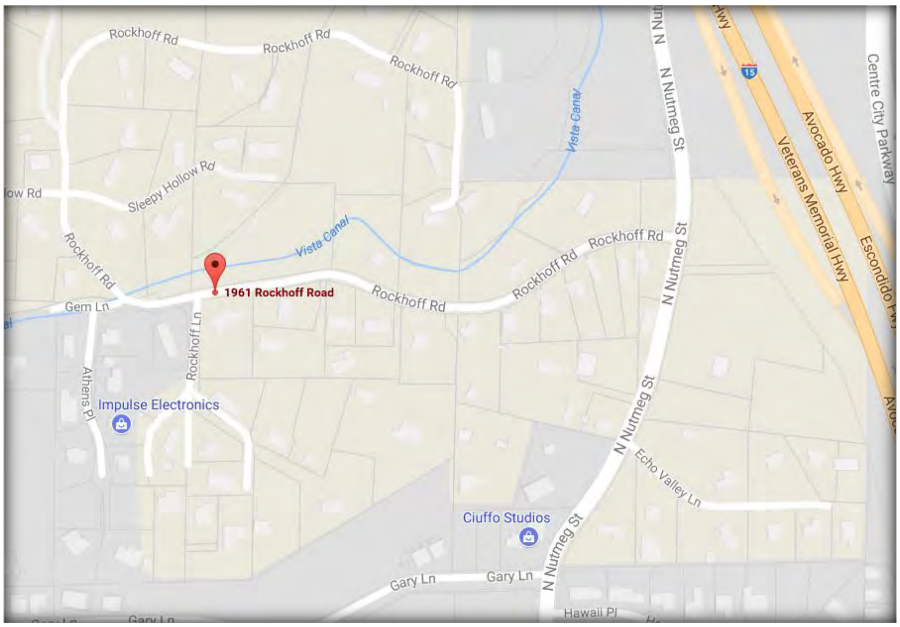


Figure 10-2. MD Reservoir

10.1 Typical Civil/Site and Corrosion Observations

Table 10-1. Civil/Site and Corrosion Observations

MD Reservoir Civil/Site and Corrosion Observations		Photo
Fence/Gate	Typical swing steel gate with barbed wire containing lock and signage located at entrance in good condition (Photo 1).	Photo 1 
Access Road	Reservoir is bounded by chain link fence. Overgrown shrub and bushes conflict with fence in certain areas. (Photo 2 and Photo 4). Paved access road leads up to entrance and gravel surrounding reservoir (Photo 2). Major case of erosion NE of entrance causing hazardous slope for maintenance personnel.	Photo 2 

Table 10-1. Civil/Site and Corrosion Observations

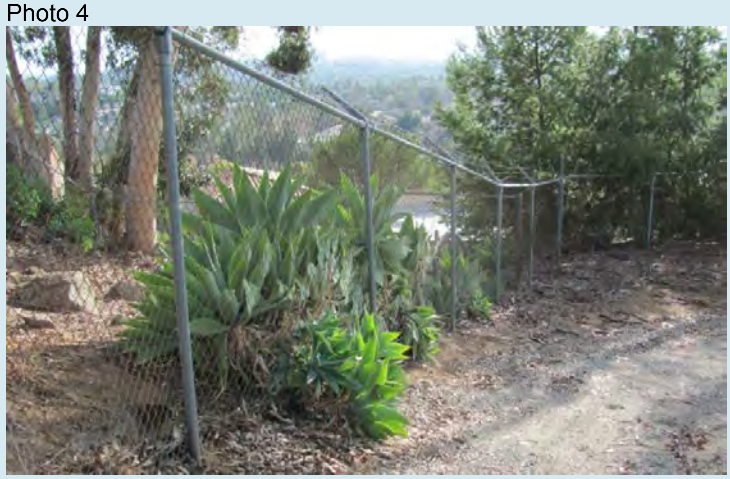
MD Reservoir Civil/Site and Corrosion Observations		Photo
Drainage	Good slope for runoff from reservoir. Drains into surrounding vegetation. Concrete lined ditch NW of entrance contains minor sediments but was in overall good condition (Photo 3).	Photo 3 
Trees and Vegetation	Excessive plant growth in multiple areas encroaches perimeter fence. This caused damage in fence (Photo 4).	Photo 4 

Table 10-1. Civil/Site and Corrosion Observations

MD Reservoir Civil/Site and Corrosion Observations		Photo
Hatches	Roof hatch was observed to have surface rusting on the lock cover. Steel screws were found corroded (Photo 5).	Photo 5 

Table 10-1. Civil/Site and Corrosion Observations

MD Reservoir Civil/Site and Corrosion Observations		Photo
Vents	Center vent was found to be in good condition (Photo 6).	Photo 6 
	New fine mesh bug screen had been added with temporary wire (Photo 7).	Photo 7 
	Side vents are located around the perimeter every few feet apart. Vents have a mesh covering and wood frame. Wood frames had cases of chipping. Visible holes near side vents were stapled with mesh covering (Photo 8).	Photo 8 

Table 10-1. Civil/Site and Corrosion Observations

MD Reservoir Civil/Site and Corrosion Observations		Photo
Ladders	Stainless steel external ladder located near reservoir entrance (Photo 9). Top of ladder does not have handrail and is therefore a safety hazard.	Photo 9 
Handrails, and Guardrails	There are a total of two guardrails located at MD Reservoir. Guardrails were found in good condition (Photo 10).	Photo 10 

Table 10-1. Civil/Site and Corrosion Observations

MD Reservoir Civil/Site and Corrosion Observations		Photo
Signage	Signage on site includes entrance, and signs located throughout reservoir. Signage was observed to be in good condition (Photo 11).	Photo 11 

10.1.1 Conclusions

Based on visual inspections of the civil/site and corrosion observations at MD Reservoir at the time of the condition assessment, the following conclusions are made:

- Overgrown shrub and bushes surrounding in close proximity to the reservoir, fence and other structures.
- Access area surrounding reservoir consists of gravel. Major case of erosion on northeast side of reservoir entrance could be unsafe for maintenance personnel and their vehicles.
- Excessive plant growth in multiple areas intrudes perimeter fence. This caused bend in fence.
- No confined space signage installed at the roof hatch location.
- Roof hatch was observed to have surface rusting on the lock cover and steel screws were found to be corroded.
- Spot corrosion and pitting observed on interior platform and ladder.
- Top of ladder does not contain a handrail which is a safety hazard.

10.2 Structural Observations and Conclusions

10.2.1 As-Built Drawings Review

Based on cursory review of the as-built drawings prepared by the Engineering Offices of J B Lippincott, dated November 1926, the MD Reservoir is a partially buried, circular shaped, cast-in-place reinforced concrete reservoir with a 4-inch thick reinforced concrete floor and a tapered reinforced concrete wall that is 15 inches thick at the base and 8 inches thick at the top. The wall is supported by a 31-inch wide by 12-inch thick continuous reinforced concrete footing. The roof consists of a wood rafter and girder system supported by 8-inch square precast reinforced concrete columns bearing on 2-foot square by 6-inch thick reinforced concrete footings. The reservoir is approximately 55 feet in diameter with a maximum water depth of approximately 14 feet.

Considering MD Reservoir's date and type of construction, it is not in compliance with the current seismic code.

Table 10-2. Exterior Observations

MD Reservoir Structural Exterior Observations		Photo
Roof	Visible sagging of the roof paneling between the supports with evidence of ponding along the roof edge. Minor vertical deflection of the roof system felt while walking the roof (Photo 1).	Photo 1 
Wall	Typical full-height cracking in exterior face of concrete wall. This condition was observed along the entire perimeter of the reservoir at approximately 8-foot spacing. There does not appear to be any moisture seepage (Photo 2).	Photo 2 

Table 10-3. Interior Observations

MD Reservoir Structural Interior Observations		Photo
Roof	Typical condition of interior roof framing and support (Photo 1).	Photo 1 
Roof/Wall	Typical condition of roof structure connection to reservoir wall (Photo 2).	Photo 2 

10.2.2 Conclusions

Based on inspection of the visible portions of MD Reservoir at the time of the condition assessment, the following conclusions are made:

- The sagging and evidence of ponding observed on the roof is expected of age and type of construction. Roof loading should be limited and roof condition monitored regularly for safety.
- The wall cracking appears to have been present since construction and is not a structural concern.
- Despite the age of the roof framing, it is in fair condition and appears to be functioning properly. As stated previously, the roof system is not in compliance with the current seismic code and should be monitored regularly for safety.

10.3 Recommendations

The following recommendations address the deficiencies noted during the field inspections. Section 10.3.1 and 10.3.2 include recommendations pertaining to minor maintenance, repair work and ongoing monitoring. Section 10.3.3 lists potentially larger scale improvements and recommendations for further investigation. An overall condition rating and prioritization of reservoir improvements is included in Section 12. Section 12 also contains proposed recommendation phasing, recommendations for additional assessment and a budgetary level opinion of cost summary for inclusion in the District's Capital Improvement Plan.

10.3.1 Civil/Site and Corrosion Recommendations

The following are civil site and corrosion improvement recommendations to be considered for the MD Reservoir site:

- Remove all vegetation in close proximity to the reservoir, fence and other structures.
- Repair fence damage where excessive plant growth intruded perimeter fence.
- Install confined space signage on roof hatch. Use black text on signage to prevent fading.
- Remove roof hatch surface rusting and repaint lock cover. Replace corroded screws with stainless hardware.
- Remove corrosion and pitting on interior platform and ladder.
- Add ladder extensions above roof level.

10.3.2 Structural Recommendations

The following are structural improvement recommendations to be considered for the MD Reservoir site:

- Repair the roof, as required, to prevent ponding and provide proper drainage.
- Limit roof loading to two workers and fifty pounds of equipment and regularly monitor roof condition for safety.
- Clean all staining of the exterior wall surface.
- Seal all cracking in the exterior wall to prevent long-term damage to the concrete and embedded steel reinforcing due to moisture infiltration.
- Monitor all corrosion of the interior metal components.
- Clean corrosion and coat interior anchor bolts.

10.3.3 Recommendations for Further Investigation

The following are potentially larger scale improvements and recommendations for further investigation for the MD Reservoir site:

- Perform a seismic evaluation of the reservoir to determine if seismic retrofit is a viable option to achieve compliance with the current seismic code.
- Investigate the stability of the erosion on the west side of the reservoir.

11 Deodar Reservoir

Deodar Reservoir has a 1.10 MG operating capacity and is located at 947 Deodar Road, San Marcos, CA 92069, as shown in Figure 11-1. Deodar Reservoir is a prestressed concrete reservoir that was constructed in 1978. According to VID staff, standard preventative maintenance that occurred on Deodar Reservoir consisted of minor maintenance to correct the glulam beams which had significant exterior rot. HDR's inspection of Deodar Reservoir, as illustrated in Figure 11-2, was conducted on November 16, 2016.

Figure 11-1. Deodar Vicinity Map

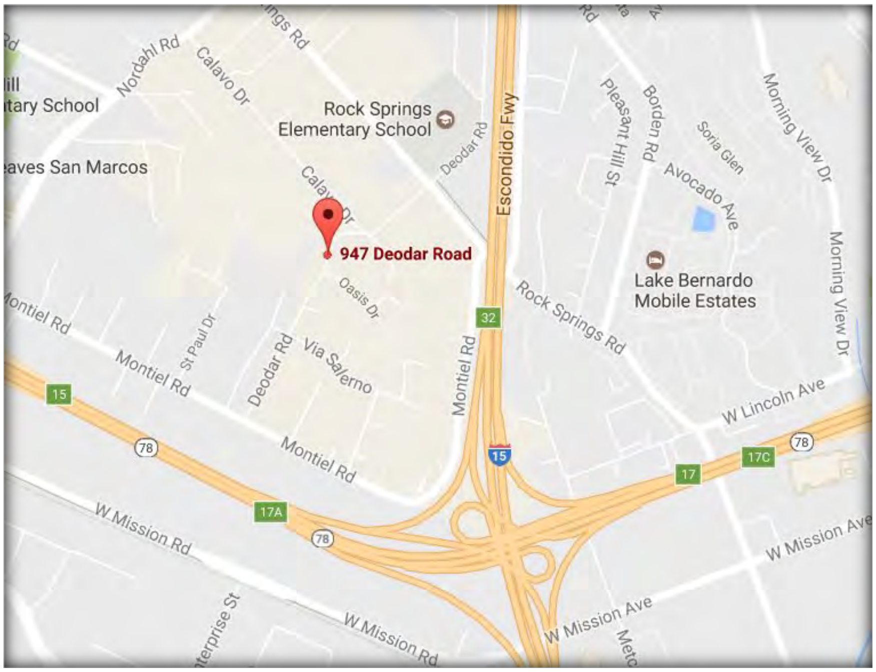


Figure 11-2. Deodar Reservoir

11.2 Typical Civil/Site and Corrosion Observations

Table 11-1. Civil/Site and Corrosion Observations

Deodar Reservoir Civil/Site and Corrosion Observations		Photo
Fence/Gate	Two typical swing steel gates containing lock and signage located at both the east and west side of the entrance. Gates observed to be in good condition (Photo 1 and Photo 2).	Photo 1  Photo 2 
Access Road	No VID site enclosure fence. Paved access road leading up to reservoir in good condition. Paved road surrounding reservoir barely accessible for one crew vehicle.	

Table 11-1. Civil/Site and Corrosion Observations

Deodar Reservoir Civil/Site and Corrosion Observations		Photo
Drainage	Roof drains surrounding reservoir. Good slope for runoff on North side of reservoir down to surrounding trees and vegetation (Photo 3).	Photo 3 
Trees and Vegetation	Trees and vegetation growth surrounding reservoir within distance does not conflict with operations. North end of reservoir consists of deep slope from end of pavement to adjacent landscape (Photo 4).	Photo 4 

Table 11-1. Civil/Site and Corrosion Observations

Deodar Reservoir Civil/Site and Corrosion Observations		Photo
Hatches	Roof hatch found to have corroded screws and hinges (Photo 5).	Photo 5 

Table 11-1. Civil/Site and Corrosion Observations

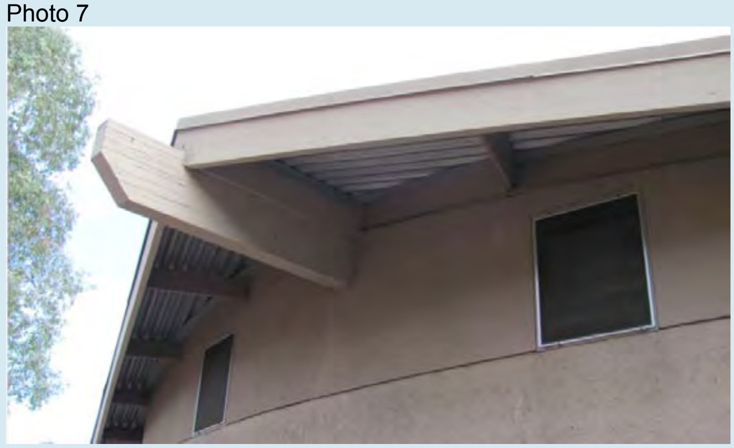
Deodar Reservoir Civil/Site and Corrosion Observations		Photo
Vents	Center vent was found to be in good condition (Photo 6).	Photo 6 
	Side vents are located around the perimeter of the reservoir. Vents have a mesh covering and metal frame. Metal frames were observed to be in good condition with only minor signs of rust (Photo 7).	Photo 7 

Table 11-1. Civil/Site and Corrosion Observations

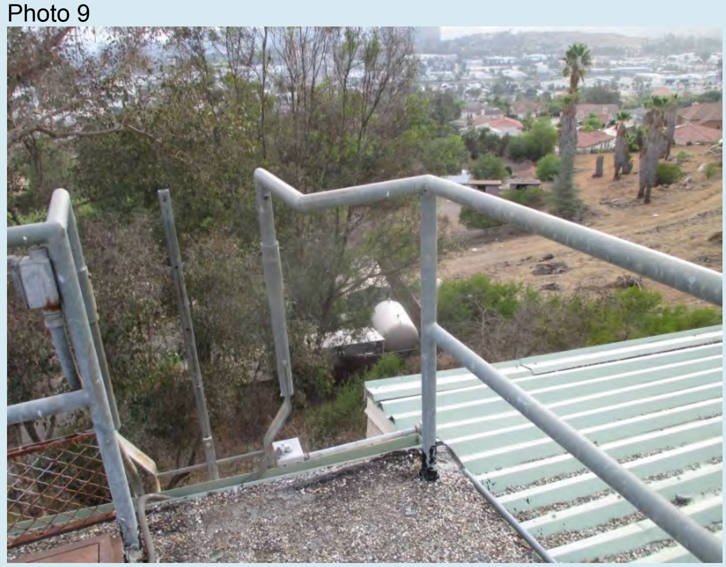
Deodar Reservoir Civil/Site and Corrosion Observations		Photo
Ladders	Stainless steel external ladder located on NE side of reservoir entrance. Transition from ladder to guardrail is secure; gap between safety climb device and guardrail is narrow (Photo 8). Interior ladder was observed to be in good condition.	Photo 8 
Handrails and Guardrails	Guardrails were found to be in good condition (Photo 9).	Photo 9 

Table 11-1. Civil/Site and Corrosion Observations

Deodar Reservoir Civil/Site and Corrosion Observations		Photo
Signage	Signage on site is includes only on the entrance and exit gate. Signage was observed to be in good condition (Photo 10).	Photo 10 

11.2.1 Conclusions

Based on visual inspections of the civil/site and corrosion observations at Deodar Reservoir at the time of the condition assessment, the following conclusions are made:

- No site security fence. There are two typical swing steel gates with lock and signage located at both the east and west side of the entrance.
- North end of reservoir consists of deep slope from end of pavement to adjacent landscape.
- Signage on site is includes only on the entrance and exit gate.
- Hinges and screws on roof hatch were found to be corroded.
- Spot corrosion was observed on internal overflow structure.
- No confined space signage installed at the roof hatch location.

11.3 Structural Observations and Conclusions

11.3.1 As-Built Drawings Review

Based on cursory review of the as-built drawings prepared by James M Montgomery Consulting Engineers, Inc., dated October 1976, the Deodar Reservoir is a partially buried, circular shaped, prestressed concrete reservoir with a 6-inch thick reinforced concrete floor and an 8-inch thick reinforced concrete core wall with an exterior gunite layer providing cover for the circumferential prestressed reinforcing. The prestressed concrete wall extends to an elevation 31 feet above the wall footing. An 8-inch thick reinforced masonry wall sits on top of the prestressed concrete wall and varies in height based on the slope of the roof. The roof consists of a wood rafter and girder system supported by a reinforced concrete column at the center of the reservoir and the reinforced masonry wall at the exterior. The reservoir is approximately 86 feet in diameter with a maximum water depth of approximately 30 feet at overflow.

The original reservoir design included seismic cable system at the wall to foundation connection, but further analysis will be required to determine if the reservoir is in compliance with the current seismic code.

Table 11-2. Exterior Observations

Deodar Reservoir Structural Exterior Observations		Photo
Roof	General condition of reservoir roof. Deterioration of exposed wood at center vent (Photo 1).	Photo 1 
Roof	Typical condition of roof deck. Some corroded fasteners. Evidence of damage due to thermal movement including missing and replaced fasteners. Missing foam insulation (Photo 2).	Photo 2 
Roof	Typical condition of roof deck near center vent. Isolated damage to due to workers walking on deck at these locations (Photo 3).	Photo 3 

Table 11-2. Exterior Observations

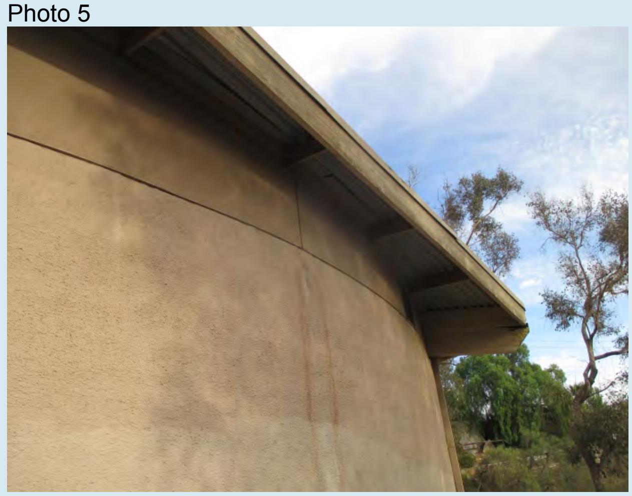
Deodar Reservoir Structural Exterior Observations		Photo
Roof Drains	Up close inspection at the roof drains not possible at the time of inspection. Roof drains appear to be functioning properly, but design is similar to Pechstein Reservoir and similar issues likely occurring at this reservoir. See Section 4.3.2, Photo 3 for additional information (Photo 4).	Photo 4 
Roof/Wall	Typical condition of wood framing at top of masonry wall. Active moisture with visible wood deterioration. Staining of concrete wall below joint in masonry wall stucco finish (Photo 5).	Photo 5 
Roof	Typical condition of valley girder. Deterioration and delamination of glu-lam beam with active moisture, algae growth, and staining visible (Photo 6).	Photo 6 

Table 11-2. Exterior Observations

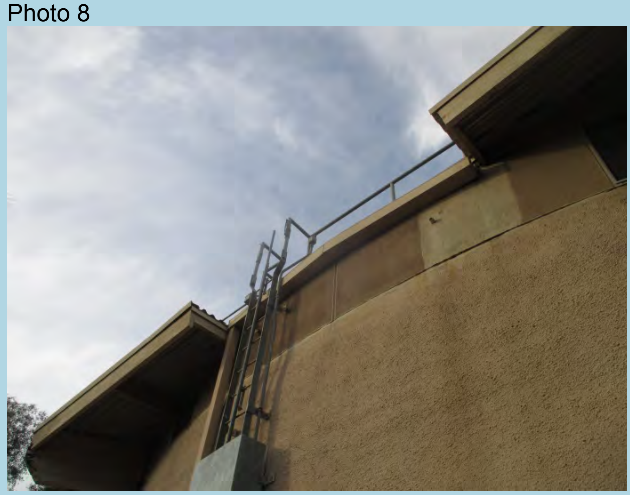
Deodar Reservoir Structural Exterior Observations		Photo
Roof	Typical condition of ridge girder. Deterioration and delamination of glu-lam beam with no active moisture visible (Photo 7).	Photo 7 
Wall	Corrosion of masonry wall stucco trim at roof blockout for access ladder with staining of concrete wall below (Photo 8).	Photo 8 

Table 11-3. Interior Observations

Deodar Reservoir Structural Interior Observations		Photo
Roof/Wall	Typical condition of interior roof framing and support. Progressively more severe moisture accumulation (condensation) and corrosion of framing connections moving from ridge to valley. Light entering reservoir at locations of missing foam insulation (Photo 1).	Photo 1 

11.3.2 Conclusions

Based on inspection of the visible portions of Deodar Reservoir at the time of the condition assessment, the following conclusions are made:

- The observed condition of the exposed wood at the center vent is expected of the age of its construction and does not appear to be affecting its function. All exposed wood should be recoated and, if necessary, replaced to ensure functionality of the vent structure.
- Damage due to thermal movement is typical of this type of roof deck. The deck connections to all supports should be routinely inspected and, if necessary, replaced, for safety and to ensure its functionality.
- Roof deck loading should be limited until all missing deck attachments are secured.
- All missing foam insulation should be replaced to restore weather-tightness of the reservoir.
- Current design of the roof drains allows accumulation of debris and moisture at the low points, leading to deterioration of the roof system at these locations. Leaking of the gutters is also contributing to the active moisture and damage observed at the valley girders. The roof drains should be redesigned to prevent these conditions.
- The major contributor to the deterioration of the roof framing is the lack of ventilation inside the reservoir. This lack of ventilation allows condensation to form on the roof components. The condensation accumulates as it travels from ridge to valley, causing progressively more severe damage with accumulation. The observed deterioration caused by this condition includes; deterioration of the rafters and valley girders with active moisture, algae growth, and staining (Section 11.4.2, Photos 5 – 6), corrosion and failure of wood connections at the interior of the

reservoir (Section 11.4.3, Photo 1, and as described by VID staff). Without improvements to the ventilation, the observed deterioration will continue.

- The corrosion of the stucco trim on the masonry wall is not currently a structural concern but should be repaired to maintain long-term serviceability of the metal.
- The staining of the exterior wall surface is not a structural concern and will be resolved with ventilation improvements.

11.4 Recommendations

The following recommendations address the deficiencies noted during the field inspections. Section 11.4.1 and 11.4.2 include recommendations pertaining to minor maintenance, repair work and ongoing monitoring. Section 11.4.3 lists potentially larger scale improvements and recommendations for further investigation. An overall condition rating and prioritization of reservoir improvements is included in Section 12. Section 12 also contains proposed recommendation phasing, recommendations for additional assessment and a budgetary level opinion of cost summary for inclusion in the District's Capital Improvement Plan.

11.4.1 Civil/Site and Corrosion Recommendations

The following are civil site and corrosion improvement recommendations to be considered for the Deodar Reservoir site:

- Install new chain link security fence surrounding reservoir boundary.
- Appropriate signage shall be included with the installation of VID surrounding security fence.
- Replace corroded hinges and screws on roof hatch.
- Remove corrosion and repair coating on interior overflow structure.
- Install confined space signage on roof hatch. Use black text on signage to prevent fading.

11.4.2 Structural Recommendations

The following are structural improvement recommendations to be considered for the Deodar Reservoir site:

- Replace all missing foam insulation to restore the weather-tightness of the reservoir.
- Reconfigure the roof drains to prevent accumulation of debris and moisture at the low points and deterioration of the roof system at these locations.
- Provide ventilation improvements for the reservoir to prevent accumulation of condensation and deterioration of the roof framing and its connections.
- Repair the corrosion of the stucco trim on the masonry wall to maintain long-term serviceability of the metal.
- Clean all staining of the exterior wall surface.

- Monitor all corrosion of the interior metal components

11.4.3 Recommendations for Further Investigation

The following are potentially larger scale improvements and recommendations for further investigation for the Deodar Reservoir site:

- Repair (clean and recoat) and, if necessary, replace all deteriorated exposed wood.
- Replace all damaged or missing roof deck connections and their supports. Provide routine inspection for safety. Limit roof deck loading until all missing roof deck supports are replaced and deck attachments secured.
- Perform a detailed condition assessment of the reservoir interior.
- Full reservoir roof replacement following the results of the detailed condition assessment of the reservoir interior.
- Perform a seismic evaluation of the reservoir to determine if it is in compliance with the current seismic code.
- Investigate the stability of the erosion on the northwest side of the reservoir entrance.

12 Overall Results Summary

The HDR standardized Condition Assessment Ratings System (CARS) was utilized to help guide the inspection team while conducting the reservoir inspections. CARS promotes consistency from site to site to facilitate proper prioritization of the reservoirs civil site, corrosion and structural aspects.

The criteria specified in the CARS are grouped into four categories as follows:

10. Structural
11. Site (non-reservoir)
12. Aesthetic (reservoir only)
13. Safety/Security

The CARS criteria can be adapted as necessary, depending on the type of structure and the level of assessment to be conducted.

Each criterion was scored on a scale or listed as Not Applicable. The scoring criteria are displayed in Table 12-1.

Table 12-1. Scoring Criteria

Score	Description	Phasing
0	No action required	--
1	Minor (7+ years)	Long-Term
3	Moderate (2-6 years)	Mid-Term
5	Immediate (0-2 years)	Near-Term
N/A	Not Applicable	--

The reservoir site, civil, and structural observation descriptions, item numbers, and the corresponding criteria classes are listed in Table 12-2 through Table 12-10. Appendix A details the outcomes of the site observation results for each reservoir.

12.1 Site Survey Results

The reservoir site inspections focused on two key areas: civil/site and corrosion conditions and general reservoir structural conditions.

Preliminary observations and ratings were recorded in the field during the site inspections. The inspection team then reviewed photographs taken in the field and other existing data to compare each criterion across all reservoir sites to establish the final scores for each reservoir provided in this report. The priority rankings calculated for each of the ten reservoirs were based on the cumulative scores across all criteria. The criteria applied and corresponding scores for each reservoir site inspection is included in Appendix A.

12.1.1 Civil/Site Survey Results

Table 12-2 includes the civil/site related items and their corresponding criteria classes that were inspected in the field. Table 12-3 includes a summary of the total observation rankings of the civil/site components for each of the ten reservoirs. Scores were assigned using the scoring criteria and descriptions displayed in Table 12-2.

Table 12-2. Reservoir Civil/Site Observations

Civil/Site Observations		
Item Num.	**Description**	**Criteria Category**
S1	Fence	2. Site (non-reservoir) 4. Safety/Security
S2	Gate(s)	2. Site (non-reservoir) 4. Safety/Security
S3	Identification Signage	2. Site (non-reservoir)
S4	Security	4. Safety/Security
S5	Access Road	2. Site (non-reservoir)
S6	Trees & Vegetation	2. Site (non-reservoir) 4. Safety/Security
S7	Drainage	2. Site (non-reservoir)
S8	Stability / Geotechnical	2. Site (non-reservoir) 4. Safety/Security
S9	Site Piping & Appurtenances	2. Site (non-reservoir)

Table 12-3. Civil/Site Observations Summary

Civil/Site Observations		Reservoir Scores									
Item Num.	Description	Lupine Hills	A	Pechstein	HB	C	E1	San Luis Rey	H	MD	Deodar
S1	Fence	0	5	0	5	N/A	0	3	0	5	5
S2	Gate(s)	0	3	0	0	N/A	0	0	0	0	0
S3	Identification Signage	0	5	0	5	0	5	5	0	0	0
S4	Security	0	3	0	3	5	0	0	0	0	0
S5	Access Road	1	0	1	1	1	0	0	5	0	3
S6	Trees & Vegetation	3	5	0	0	0	5	0	5	5	0
S7	Drainage	0	0	3	0	0	0	N/A	0	0	0
S8	Stability / Geotechnical	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	5	5
S9	Site Piping and Appurtenances	N/A	5	N/A	1	N/A	N/A	N/A	N/A	N/A	N/A
	Total Scores	4	26	4	15	6	10	8	10	15	13

Table 12-4 ranks all ten reservoirs from highest to lowest scores. When weighing all key areas inspected equally, A Reservoir ranked first with 26 total points and Pechstein and Lupine Hills Reservoirs tied in last place with a total of 4 points each. Reservoirs that contained at least one item number which scored a 5 were considered for near term, 0-2 years. Eight of the ten reservoirs contain at least one component requiring the recommendation be phased near term (0-2 years) and two of ten requiring the recommendation be phased midterm (2-6 years). Table 12-13 in Section 12.2 includes all recommendations.

Table 12-4. Civil/Site Summary Rankings

Tank Name	Civil/Site Total	Rank (1-10)	Phasing
A Reservoir	26	1	Near-Term
HB Reservoir	15	2	Near-Term
MD Reservoir	15	3	Near-Term
Deodar Reservoir	13	4	Near-Term
H Reservoir	10	5	Near-Term
E1 Reservoir	10	6	Near-Term
C Reservoir	6	7	Near-Term
San Luis Rey Reservoir	8	8	Near-Term
Pechstein Reservoir	4	9	Mid-Term
Lupine Hills Reservoir	4	10	Mid-Term

12.1.2 Civil/Corrosion Results

Table 12-5 includes the civil/corrosion related items and their corresponding criteria classes that were inspected in the field. Table 12-6 includes a summary of the total observations rankings of the civil/corrosion components for each of the ten reservoirs. Scores were assigned using the scoring criteria and descriptions displayed in Table 12-1.

Table 12-5. Reservoir Civil/Corrosion Observations

Reservoir Civil/Corrosion Observations		
Item Num.	**Description**	**Criteria Class**
C1	Roof Hatch	3. Aesthetic (reservoir only) 4. Safety/Security
C2	Safety Signage	4. Safety/Security
C3	Vent Condition	3. Aesthetic (reservoir only)
C4	Ladders & Stairs	4. Safety/Security
C5	Handrails/Guardrails	4. Safety/Security
C6	Overflow Pipe	3. Aesthetic (reservoir only)

Table 12-6. Civil/Corrosion Observations Summary

Reservoir Civil/Corrosion Observations		Reservoir Scores									
Item Num.	Description	Lupine Hills	A	Pechstein	HB	C	E1	San Luis Rey	H	MD	Deodar
C1	Roof Hatch	3	5	3	3	1	3	3	3	3	5
C2	Safety Signage	0	1	1	1	1	1	N/A	1	1	1
C3	Vent Condition	3	0	5	3	3	3	1	0	0	0
C4	Ladders & Stairs	1	0	1	5	5	5	0	3	3	1
C5	Handrails/Guardrails	5	0	0	N/A	3	3	N/A	0	3	0
C6	Overflow Pipe	N/A	N/A	N/A	N/A	N/A	N/A	N/A	3	N/A	3
	Total Scores	12	6	10	12	13	15	4	10	10	10

Table 12-7 ranks all ten reservoirs from highest to lowest scores. When weighing all key areas inspected equally, E1 Reservoir ranked first with 15 total points and San Luis Rey Reservoir ranked last place with a total of 4 points. Reservoirs that contained at least one item number which scored a 5 were considered for near term, 0-2 years. Seven of the ten reservoirs contain at least one component requiring the recommendation be phased near term (0-2 years) and three of ten requiring the recommendation be phased midterm (2-6 years). Table 12-13 in Section 12.2 includes all recommendations.

Table 12-7. Civil/Corrosion Summary Rankings

Tank Name	Civil/Corrosion Total	Rank (1-10)	Phasing
E1 Reservoir	15	1	Near-Term
C Reservoir	13	2	Near-Term
HB Reservoir	12	3	Near-Term
Lupine Hills Reservoir	12	4	Near-Term
Pechstein Reservoir	10	5	Near-Term
Deodar Reservoir	10	6	Near-Term
MD Reservoir	10	7	Mid-Term
H Reservoir	10	8	Mid-Term
A Reservoir	6	9	Near-Term
San Luis Rey Reservoir	4	10	Mid-Term

12.1.3 Structural Survey Results

Table 12-8 includes the structural related items and their corresponding criteria classes that were inspected in the field. Table 12-9 includes a summary of the total observations rankings of the structural components for each of the ten reservoirs. Scores were assigned using the scoring criteria and descriptions displayed in Table 12-1.

Table 12-8. Reservoir Structural Observations

Structural Observations		
Item Num.	Description	Criteria Class
ST1	Foundation	1. Structural
ST2	Wall	1. Structural
ST3	Roof	1. Structural
ST4	Interior Structure	1. Structural

Table 12-9. Structural Observations Summary Table

Structural Observations		Reservoir Scores									
Item Num.	Description	Lupine Hills	A	Pechstein	HB	C	E1	San Luis Rey	H	MD	Deodar
ST1	Foundation	N/A	N/A	N/A	3	N/A	N/A	N/A	N/A	N/A	N/A
ST2	Wall	1	1	3	3	1	1	N/A	1	1	3
ST3	Roof	5	5	5	3	3	3	1	3	3	5
ST4	Interior Structure	N/A	3	5	N/A	N/A	3	1	N/A	N/A	N/A
	Total Scores	6	9	13	9	4	7	2	4	4	8

Table 12-10 ranks all ten reservoirs from highest to lowest scores, considering only the two structural categories that were consistently inspected among all the reservoirs: ST2 Wall and ST3 Roof. Pechstein and Deodar Reservoirs ranked first and second with 8 total points each and San Luis Rey Reservoir ranked last with a total of 1 point. However, San Luis Rey Reservoir is an underground structure and the walls were not able to be inspected. Reservoirs that contained at least one item number which scored a 5 were phased for near term, 0-2 years. Five of the ten reservoirs contain at least one component requiring the recommendation be phased near term (0-2 years), five of ten requiring the recommendation be phased midterm (2-6 years), and one reservoir with a recommendation of long-term (7 or more years). Table 12-13 in Section 12.2 includes all recommendations.

Reservoirs with a recommended phasing of near term require further investigation and a detailed condition assessment of the reservoir's interior to determine whether complete roof replacement and or interior supports are necessary. Through field inspection observations and notes it was determined that the structural deficiencies for each reservoir should take priority over the civil/site and civil/corrosion deficiencies.

Table 12-10. Structural Summary Rankings

Tank Name	Structural Total	Rank (1-10)	Phasing
Pechstein Reservoir	8	1	Near-Term
Deodar Reservoir	8	2	Near-Term
A Reservoir	6	3	Near-Term
Lupine Hills Reservoir	6	4	Near-Term
HB Reservoir	6	5	Mid-Term
E1 Reservoir	4	6	Mid-Term
MD Reservoir	4	7	Mid-Term
H Reservoir	4	8	Mid-Term
C Reservoir	4	9	Mid-Term
San Luis Rey Reservoir	1	10	Long-Term

12.1.4 Overall Ranking Results

Table 12-11 displays the overall scoring results for site, civil/corrosion and structural/corrosion for each of the ten reservoirs. Combining the total values without any weighting of the criteria yielded A Reservoir and HB Reservoir as the highest ranking reservoirs with recommended phasing of near-term and mid-term, respectively. However, it is apparent that structural/corrosion related defect repairs are more imperative since they could affect the safety and reliability of the reservoirs and should be weighted more heavily in the priority assessment.

Table 12-11. Total Ranking Results – Non-weighted

Tank Name	Site Total	Civil/ Corrosion Total	Structural/ Corrosion Total	Overall Total	Rank	Phasing
A Reservoir	26	6	6	38	1	Near-Term
HB Reservoir	15	12	6	33	2	Mid-Term
MD Reservoir	18	10	4	32	3	Mid-Term
Deodar Reservoir	13	10	8	31	4	Near-Term
C Reservoir	6	13	4	23	5	Mid-Term
Pechstein Reservoir	4	10	8	22	6	Near-Term
H Reservoir	10	10	4	24	7	Mid-Term
Lupine Hills Reservoir	4	12	6	22	8	Near-Term
San Luis Rey Reservoir	8	4	1	13	9	Long-Term
E1 Reservoir	10	15	4	29	10	Mid term

Table 12-12 displays the prioritization of the projects following the weighting of the criteria. The site and civil/corrosion and structural components were first normalized to a 100 point total scale. Site and civil/corrosion were then weighted at 20 percent each and structural/corrosion was weighted at 60 percent. Weighting the structural/corrosion criteria at a higher value allowed for a more accurate prioritization of the projects to address safety and reliability concerns first.

Table 12-12. Overall Ranking Results - Weighted

Tank Name	Site Total	Civil/ Corrosion Total	Structural/ Corrosion Total	Overall Total	Rank	Phasing
Deodar Reservoir	10.00	13.33	60.00	83.33	1	Near-Term
Pechstein Reservoir	3.08	13.33	60.00	76.41	2	Near-Term
A Reservoir	20.00	8.00	45.00	73.00	3	Near-Term
HB Reservoir	11.54	16.00	45.00	72.54	4	Near-Term
Lupine Hills Reservoir	3.08	16.00	45.00	64.08	5	Near-Term
E1 Reservoir	7.69	20.00	30.00	57.69	6	Mid-Term
MD Reservoir	13.85	13.33	30.00	57.18	7	Mid-Term
C Reservoir	4.62	17.33	30.00	51.95	8	Mid-Term
H Reservoir	7.69	13.33	30.00	51.03	9	Mid-Term
San Luis Rey Reservoir	6.15	5.33	7.50	18.99	10	Long-Term

Phasing was assigned based on the site inspection scores, if a reservoir contained at least one component scoring a 5, requiring the recommendation be completed near term (0-2 years), it was assigned a phasing of near term (0-2 years). With the weighted criteria, Deodor and Pechstein Reservoirs rank first and second with near-term phasing and San Luis Rey Reservoir ranks last with long-term phasing. It should be noted that, although HB Reservoir's scores did not contain a score of 5, HB was assigned a phasing of near term based on its overall total of 72.54 and ranking of fourth place. Lupine Hills Reservoir's roof was assigned a score of 5, due to poor roof draining and concrete cracking, meriting near term phasing, although its overall total of 64.08 earned a ranking of fifth place.

12.2 Reservoir Priority Recommendations

Field observation and office analysis results of all ten reservoirs determined that operation and maintenance repairs and general upkeep of the reservoirs have been conducted by VID staff since the time of construction resulting in reservoirs in overall very good working condition. All reservoirs had minimal civil site, corrosion and structural issues.

For a majority of the ten reservoirs, there are high priority recommendations that should be addressed near term (0-2 years) regardless of when the full condition assessments are conducted. Some items can be addressed by VID staff; others would require the services of a contractor. A description of immediate, moderate, minor or no action required, and repair timelines of 0-2 years, 2-6 years, 7+ years or N/A were assigned to each reservoir components recommendation. The full list of recommendations are provided in Table 12-13 including the near term priority recommendations. The detailed list of recommendations is included in Appendix A.

Table 12-13 - Reservoir Recommendations

Reservoir	Component	Recommendation	Phasing
Lupine Hills Reservoir	Handrails/Guardrails	Replace guardrails with bolt-down style, use stainless steel anchor bolts.	Near-Term (0-2 years)
	Roof System	Caulk gap between roof and ringwall, fill cracks in roof with concrete sealer.	Near-Term (0-2 years)
	Trees & Vegetation	Remove all vegetation in close proximity to the reservoir and fence.	Mid-Term (2-6 years)
	Roof Hatch	Replace corroded screws on the hatches and doors with stainless hardware.	Mid-Term (2-6 years)
	Vent Condition	Remove rust from the vents conduit plate and repaint conduit plate.	Mid-Term (2-6 years)
	Access Road	Paved access road was in good condition. Monitor signs of minor cracking surrounding the reservoir.	Minor (7+ years)
	Ladders & Stairs	Repaint the anti-climb device on the exterior ladder.	Minor (7+ years)
	Wall	Clean all staining of the exterior wall surface.	Minor (7+ years)
	Further Investigation	Perform a detailed condition assessment of the reservoir interior.	Near-Term (0-2 years)
	Further Investigation	Perform a seismic evaluation of the reservoir.	Near-Term (0-2 years)

Table 12-13 - Reservoir Recommendations

Reservoir	Component	Recommendation	Phasing
A Reservoir	Fence/Gate	Replace existing VID fence with new fence surrounding entire perimeter of reservoir.	Near-Term (0-2 years)
	Signage	Replace faded aging signage. Black text signage is recommended as red text fades in due time.	Near-Term (0-2 years)
	Trees & Vegetation	Continue maintenance of surrounding trees and vegetation.	Near-Term (0-2 years)
	Site Piping & Appurtenances	Replace corroded pipe sections on exterior pipe riser.	Near-Term (0-2 years)
	Roof Hatch	Remove rust and repaint lock cover on the roof hatch.	Near-Term (0-2 years)
	Roof	Modify the roof slope, as required, to prevent ponding and provide proper drainage.	Near-Term (0-2 years)
	Security	Neighboring fence is short and does not provide adequate security, new fence install to resolve issue.	Mid-Term (2-6 years)
	Interior Structure	Repair the spalling in the concrete beam above the overflow.	Mid-Term (2-6 years))
	Safety Signage	Install confined space signage on roof hatch.	Minor (7+ years)
	Wall	Clean all staining of the exterior wall surface.	Minor (7+ years)
	Further Investigation	Perform a detailed condition assessment of the reservoir interior and roof system.	Near-Term (0-2 years)
	Further Investigation	Perform a seismic evaluation of the reservoir.	Near-Term (0-2 years)

Table 12-13 - Reservoir Recommendations

Reservoir	Component	Recommendation	Phasing
Pechstein Reservoir	Vent Condition	Areas of rusting on roof vent should be removed. Secure loose mesh surrounding roof vent. Exposed wood at the center vent should be recoated.	Near-Term (0-2 years)
	Hatches/Doors	Remove rust spots and touch up paint on side access door interior.	Near-Term (0-2 years)
	Roof System	Replace rotted intermediate and valley roof beams, install ridge vents, replace corroded joist hangers, close roofing gaps, and replace corroded sheet metal screws.	Near-Term (0-2 years)
	Interior Structure	Repair (clean and recoat) and, if necessary, replace all deteriorated exposed wood.	Near-Term (0-2 years)
	Drainage	Monitor and cleaned roof gutters frequently to prevent additional vegetation growth.	Mid-Term (Continuous)
	Roof Hatch	Remove rust spots on hatches and touch up chipped off paint from access door interior.	Mid-Term (2-6 years)
	Wall	Clean all staining of the exterior wall surface. Regularly monitor the cracking of the stucco finish.	Mid-Term (2-6 years)
	Access Road	Monitor continuous cracks along pavement along the access road.	Minor (Continuous)
	Safety Signage	Install confined space signage on roof hatch.	Minor (7+ years)
	Ladders & Stairs	Remove minor rusting on ladder. Ladder in overall good condition.	Minor (7+ years)
	Further Investigation	Perform a detailed condition assessment of the reservoir interior, roof system, ventilation, and roof drains.	Near-Term (0-2 years)
	Further Investigation	Perform a seismic evaluation of the reservoir.	Near-Term (0-2 years)

Table 12-13 - Reservoir Recommendations

Reservoir	Component	Recommendation	Phasing
HB Reservoir	Fence/Gate	Install security fence surrounding reservoir. Security fence installation will also resolve reservoir accessibility issues.	Near-Term (0-2 years)
	Signage	Include 'No Trespassing' signage associated with installation of security gate.	Near-Term (0-2 years)
	Ladders & Stairs	Exterior ladder at the time of inspections is scheduled to be replaced. Interior ladder has corroded brackets which need to be replaced.	Near-Term (0-2 years)
	Security	Installation of full boundary chain link fence to prevent unauthorized entry and resolve potential security issues.	Mid-Term
	Roof Hatch	Repair the exposed corroded reinforcing in the concrete at the roof hatch opening.	Mid-Term (2-6 years)
	Vent Condition	Remove minor rust build up on interior of roof vent.	Mid-Term (2-6 years)
	Foundation	Seal all cracking and scaling of the foundation concrete.	Mid-Term (2-6 years)
	Wall	Seal leaks in liner, monitor prestress wire.	Mid-Term (2-6 years)
	Roof	Modify the roof slope, as required, to prevent ponding and provide proper drainage.	Mid-Term (2-6 years)
	Access Road	Monitor continuous cracks along pavement along the access road.	Minor (Continuous)
	Site Piping & Appurtenances	Remove surface rust and repaint corroded areas on exterior piping in enclosure.	Minor (7+ years)
	Safety Signage	Install confined space signage on roof hatch.	Minor (7+ years)
	Further Investigation	Perform a detailed condition assessment of the reservoir interior and leaking liner.	Near-Term (0-2 years)
	Further Investigation	Perform a seismic evaluation of the reservoir.	Near-Term (0-2 years)

Table 12-13 - Reservoir Recommendations

Reservoir	Component	Recommendation	Phasing
C Reservoir	Ladders & Stairs	The fixed exterior ladders are not OSHA compliant and require further modification.	Near-Term (0-2 years)
	Vent Condition	Replace roof vents mesh covering.	Mid-Term (2-6 years)
	Handrails/Guardrails	Install toe boards on guardrail system.	Mid-Term (2-6 years)
	Roof	Modify the roof slope, as required, to prevent ponding and provide proper drainage.	Mid-Term (2-6 years)
	Roof Hatch	Remove minor corrosion on interior of aluminum hatch on reservoir roof.	Minor (7+ years)
	Safety Signage	Install confined space signage on roof hatch.	Minor (7+ years)
	Wall	Clean all staining of the exterior wall surface and seal all cracking in the exterior wall.	Minor (7+ years)
	Further Investigation	Perform a seismic evaluation of the reservoir.	Mid-Term (2-6 years)
E1 Reservoir	Identification Signage	Replace faded signage. Black text signage is recommended as red text fades in due time.	Near-Term (0-2 years)
	Trees & Vegetation	Remove 2 large trees located on east side of fence and all vegetation within five feet from fence to prevent accessibility interference.	Near-Term (0-2 years)
	Ladders & Stairs	Exterior ladder's first step rung should be placed such that it is compliant with OSHA fixed ladder requirements.	Near-Term (0-2 years)
	Roof Hatch	Replace corroded screwed on lock cover of roof hatch.	Mid-Term (2-6 years)
	Vent Condition	Caulk bottom of roof vent.	Mid-Term (2-6 years)
	Handrails/Guardrails	Install toe boards on guardrail system.	Mid-Term (2-6 years)
	Roof	Modify the roof slope, as required, to prevent ponding and provide proper drainage.	Mid-Term (2-6 years)
	Interior Structure	Remove roof beam corrosion.	Mid-Term (2-6 years)
	Safety Signage	Install confined space signage on roof hatch.	Minor (7+ years)
	Wall	Clean all staining of the exterior wall surface. Seal all cracking in the exterior wall to prevent long-term damage to the concrete and embedded steel reinforcing due to moisture infiltration.	Minor (7+ years)
	Further Investigation	Perform a seismic evaluation of the reservoir.	Mid-Term (2-6 years)

Table 12-13 - Reservoir Recommendations

Reservoir	Component	Recommendation	Phasing
San Luis Rey Reservoir	Identification Signage	Replace faded VID signage. Black text signage is recommended as red text fades in due time.	Near-Term (0-2 years)
	Fence	Consider installation of VID security fence surrounding perimeter of reservoir.	Mid-Term (2-6 years)
	Roof Hatch	Replace corroded hardware on both hatches including prop bar, hinge screws, and lock cover screws. Seal all cracking in the concrete around the access hatch.	Mid-Term (2-6 years)
	Vent Condition	Some rust bleed and cracking of mortar; Vent cap has been recoated.	Minor (7+ years)
	Roof	Monitor concrete cracking observed at roof.	Minor (Continuous)
	Interior Structure	Clean all corrosion of the interior metal components.	Minor (7+ years)
	Further Investigation	Perform a detailed condition assessment of the reservoir interior.	Mid-Term (2-6 years)
	Further Investigation	Perform a seismic evaluation of the reservoir.	Mid-Term (2-6 years)
H Reservoir	Access Road	Repair uneven pavement and pavement cracking in surrounding access road.	Near-Term (0-2 years)
	Trees & Vegetation	Remove all vegetation in close proximity to the reservoir, fence, and other structures. Remove all shrubbery and weeds intertwined into links of fence.	Near-Term (0-2 years)
	Roof Hatch	Replace corroded conduit fittings on roof and inside access hatch.	Mid-Term (2-6 years)
	Ladders & Stairs	Insure stairway on NE side of reservoir is OSHA compliant, consider installation of inner handrail.	Mid-Term (2-6 years)
	Overflow Pipe	Remove corrosion and repair coating on interior overflow structure.	Mid-Term (2-6 years)
	Roof	Repair the roof, as required, to prevent ponding and provide proper drainage.	Mid-Term (2-6 years)
	Safety Signage	Install confined space signage on roof hatch.	Minor (7+ years)
	Wall	Monitor shotcrete cracking.	Minor (7+ years)
	Further Investigation	Perform a detailed condition assessment of the reservoir interior.	Mid-Term (2-6 years)
	Further Investigation	Perform a seismic evaluation of the reservoir.	Mid-Term (2-6 years)

Table 12-13 - Reservoir Recommendations

Reservoir	Component	Recommendation	Phasing
MD Reservoir	Fence/Gate	Repair fence damage where excessive plant growth intruded perimeter fence.	Near-Term (0-2 years)
	Trees/Vegetation	Remove all vegetation in close proximity to the reservoir, fence and other structures.	Near-Term (0-2 years)
	Stability / Geotechnical	Investigate the stability of the erosion on the west side of the reservoir.	Near-Term (0-2 years)
	Access Road	Investigate the stability of the erosion on the west side of the reservoir.	Mid-Term (2-6 years)
	Roof Hatch	Remove rust on roof hatch and repaint lock cover, replace corroded screws with stainless hardware.	Mid-Term (2-6 years)
	Ladders & Stairs	Remove rust and recoat corroded areas on interior ladder platform and supports.	Mid-Term (2-6 years)
	Handrails/Guardrails	Add ladder extensions above roof level.	Mid-Term (2-6 years)
	Roof	Repair the roof, as required, to prevent ponding and provide proper drainage.	Mid-Term (2-6 years)
	Safety Signage	Install confined space signage on roof hatch.	Minor (7+ years)
	Wall	Clean all staining of the exterior wall surface and seal all cracking in the exterior wall.	Minor (7+ years)
	Further Investigation	Perform a seismic evaluation of the reservoir and evaluate site erosion.	Mid-Term (2-6 years)
Deodar Reservoir	Fence/Gate	Install new chain link security fence surrounding reservoir boundary.	Near-Term (0-2 years)
	Stability/Geotechnical	Investigate the stability of the erosion on the northwest side of the reservoir entrance.	Near-Term (0-2 years)
	Roof Hatch	Replace corroded hinges and screws on roof hatch.	Near-Term (0-2 years)
	Roof	Replace all missing foam insulation to restore the weather-tightness of the reservoir. Reconfigure the roof drains to prevent accumulation of debris and moisture at the low points and deterioration of the roof system at these locations. Provide ventilation improvements for the reservoir to prevent accumulation of condensation and deterioration of the roof framing and its connections.	Near-Term (0-2 years)
	Access Road	Investigate the stability of the erosion on the northwest side of the reservoir entrance.	Mid-Term (2-6 years)

Table 12-13 - Reservoir Recommendations

Reservoir	Component	Recommendation	Phasing
	Overflow Pipe	Remove corrosion and repair coating on interior overflow structure.	Mid-Term (2-6 years)
	Wall	Clean all staining of the exterior wall surface.	Mid-Term (2-6 years)
	Safety Signage	Install confined space signage on roof hatch.	Minor (7+ years)
	Ladders & Stairs	Investigate narrow gap between safety climb device and guardrail.	Minor (7+ years)
	Further Investigation	Perform a detailed condition assessment of the reservoir interior, roof system and site erosion.	Near-Term (0-2 years)
	Further Investigation	Perform a seismic evaluation of the reservoir.	Near-Term (0-2 years)

12.3 CIP Cost Summary

HDR has prepared the construction cost estimate for the ten VID reservoirs using cost factors (i.e. unit costs) and supplemental cost data from similar HDR projects. Costs for individual line items in cost summary tables are in current (February 2017) dollars. Appendix B includes reservoir roof replacement options and costs for reservoirs A, Pechstein, and Deodar and reservoir replacement costs for reservoirs A, Pechstein, H, and Deodar. Unit costs for reservoir replacement are included in Table 12-14. Reservoir replacement and roof replacement is not recommended until further detailed condition assessment of the reservoirs interior is completed.

Costs should be adjusted based on when the actual repairs take place to account for future cost increases. VID will determine schedule based on available funding and the relative rank of the rehabilitation needs compared to all VID reservoirs.

Table 12-14. Unit Costs

Reservoir Replacement - Unit Costs	
Volume (GAL)	Unit Cost ($/GAL)
< 1,000,000	$2.00
1,000,000 - 3,000,000	$1.50
3,000,000 - 6,000,000	$1.25
6,000,000 – 20,000,000	$1.00

The individual cost for each bid item in the total probable cost opinions includes labor, materials, equipment, taxes, and contractor overhead and profit. The total cost incorporates a contingency factor of 10% due to its preliminary level of development and unexpected field conditions at the time of repairs. Due to the variance in quantities that may be observed in the field for this type of rehabilitation project, the contract documents will require unit pricing from the contractor. The contractor should be held to the unit pricing for all change orders for anticipated variances in estimated versus actual quantities. A work allowance determined by VID is recommended to accommodate such variances and avoid any construction delays.

Rehabilitation work outlined in the cost opinion includes the civil/site, corrosion, and structural recommendations outlined in this report. Where further investigation was recommended for seismic analysis for all ten reservoirs and an internal inspection for 7 of the 10 reservoirs, these costs are noted. Full roof replacement and full reservoir replacement costs is provided for four of the ten reservoirs but is only recommended following the results of a detailed condition assessment of the reservoir interior. The further investigation total cost includes the costs for additional assessments that are detailed for each reservoir in Appendix B; contingency markups were not included with these values since they are engineering studies. Table 12-15 displays the summary of the total probable cost opinions for all phases starting with near term, mid-term, and long-term. The total probable cost opinions for each of the ten reservoirs components are detailed in Appendix B.

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Table 12-15. Summary of Probable Cost Opinions for Recommended Improvements

Rank	Reservoir Name	Near-Term (0-2 years)	Mid-Term (2-6 years)	Minor (7+ years)	Total Direct Costs (Near-Term, Mid-Term and Minor)	Total Indirect Costs (Fees &Contingency)	Total Probable Cost Options (Rounded)	Further Investigation Assessments Near-Term (0-2 years)	Further Investigation Assessments Mid-Term (2-6 years)
1	Deodar Reservoir	$42,000	$9,814	$150	$51,964	$25,807	$78,000	$57,000	
2	Pechstein Reservoir	$45,850	$1,072	$150	$47,072	$23,446	$71,000	$81,000	
3	A Reservoir	$26,508	$5,100	$2,649	$34,257	$17,033	$52,000	$31,000	
4	HB Reservoir	$28,200	$28,342	$550	$57,092	$28,353	$86,000	$61,000	
5	Lupine Hills Reservoir	$27,392	$2,850	$981	$31,222	$15,540	$47,000	$61,000	
6	C Reservoir	$12,850	$6,188	$4,827	$23,866	$11,892	$36,000		$10,000
7	E1 Reservoir	$10,450	$9,488	$2,427	$22,366	$11,052	$34,000		$10,000
8	MD Reservoir	$4,300	$8,858	$1,693	$14,851	$7,348	$23,000		$16,000
9	H Reservoir	$9,000	$27,318	$150	$36,468	$18,128	$55,000		$61,000
10	San Luis Rey Reservoir	$5,150	$803	$1,000	$6,953	$3,404	$11,000		$61,000

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Appendix A. VID Reservoirs – Site Visit and Office Analysis Results

TABLE A-1

Vista Irrigation District
Triage Site Visit Results &
Office Analysis Results

Lupine Hills Reservoir

Address:	**2450 Lupine Hills Drive**		
	Vista, CA 92081		
Inspection Date:	**11/14/2016**		
HDR Team:	Frost, Heraypur, Yarn		

Evaluation Criteria:

(1)	Structural
(2)	Site (non-reservoir)
(3)	Aesthetic (reservoir only)
(4)	Safety/Security

Scoring Criteria:

0	No action required
1	Minor (7+ years)
3	Moderate (2-6 years)
5	Immediate (0-2 years)
N/A	Not Applicable

Lupine Hills Civil/Site Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
S1	Fence	2,4	Chain link fence observed to be in good condition	0
S2	Gate(s)	2,4	Chain link gate observed to be in good condition	0
S3	Identification Signage	2	Signage includes VID and no trespassing signs located along the fence	0
S4	Security	4	Fence at adequate height and front gate contained lock	0
S5	Access Road	2	Paved access road was in good condition with only signs of minor cracking surrounding the reservoir	1
S6	Trees & Vegetation	2,4	Minimal tree growth on the surrounding fence	3
S7	Drainage	2	Roof drains spill runoff into gutter around perimeter of the access road with good slope for runoff	0
S8	Stability / Geotechnical	2,4	N/A	N/A
S9	Site Piping & Appurtenances	2	N/A	N/A
			Total Civil/Site Score	**4**

Lupine Hills Reservoir Civil Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
C1	Roof Hatch	3,4	Steel screws and the intrusion alarm switch were found to be corroded	3
C2	Safety Signage	4	Confined space signage on site	0
C3	Vent Condition	3	Contained rust, chipped paint, and corroded conduit brackets	3
C4	Ladders & Stairs	4	Chipped paint and rust on anti-climb device in exterior ladder	1
C5	Handrails/Guardrails	4	Broken and corroded guardrails should be replaced with bolt-down style guardrails using stainless steel anchor bolts	5
C6	Overflow Pipe	3	N/A	N/A
			Total Score	**12**

Structural Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
ST1	Foundation	1	N/A	N/A
ST2	Wall	1	Cracking in gunite finish; Staining of exterior wall surface	1
ST3	Roof	1	Roof/wall joint; Poor roof drainage; Concrete cracking	5
ST4	Interior Structure	1	No Interior Inspection	N/A
			Total Score	**6**
			Grand Total Score	**22**

TABLE A-2

Vista Irrigation District
Triage Site Visit Results &
Office Analysis Results

A Reservoir

Address:	**770 Virginia Place**	**Evaluation Criteria:**		**Scoring Criteria:**	
	San Marcos, CA 92078	(1)	Structural	0	No action required
Inspection Date:	**11/14/2016**	(2)	Site (non-reservoir)	1	Minor (7+ years)
HDR Team:	Frost, Heraypur, Yarn	(3)	Aesthetic (reservoir only)	3	Moderate (2-6 years)
		(4)	Safety/Security	5	Immediate (0-2 years)
				N/A	Not Applicable

A Reservoir Civil/Site Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
S1	Fence	2,4	Fence in back area of reservoir is damaged due to the neighboring homes property	5
S2	Gate(s)	2,4	Full access of the surrounding fence at reservoir requires entry and re-entry through two separate fencing areas	3
S3	Identification Signage	2	Signage observed to be in faded and in poor condition	5
S4	Security	4	Neighboring fence is short and does not provide adequate security, Front gate contained lock	3
S5	Access Road	2	Adequate space for field crews and multiple vehicles	0
S6	Trees & Vegetation	2,4	Typical trees and vegetation from adjacent properties, minimal tree growth	5
S7	Drainage	2	Roof drains spill runoff into gravel slope that leads down to the street below	0
S8	Stability / Geotechnical	2,4	N/A	N/A
S9	Site Piping & Appurtenances	2	Corrosion and localized metal loss observed on exterior pipe riser	5
			Total Civil/Site Score	**26**

Reservoir Civil Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
C1	Roof Hatch	3,4	Rust and chipped paint on lock cover of roof hatch	5
C2	Safety Signage	4	No confined space signage installed on roof hatch or overflow hatches	1
C3	Vent Condition	3	Vents were observed to be in good condition	0
C4	Ladders & Stairs	4	Typical surface rusting observed on exterior ladder, overall good condition	0
C5	Handrails/Guardrails	4	Aluminum guardrails located at roof hatch and float box areas were in good condition	0
C6	Overflow Pipe	3	N/A	N/A
			Total Score	**6**

Structural Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
ST1	Foundation	1	N/A	N/A
ST2	Wall	1	Wall cracking; Staining of exterior wall surface	1
ST3	Roof	1	Poor roof drainage; Roof deflection under load	5
ST4	Interior Structure	1	Concrete spalling above overflow; Roof beam corrosion	3
			Total Score	**9**
			Grand Total Score	**41**

TABLE A-3

Vista Irrigation District
Triage Site Visit Results &
Office Analysis Results

Pechstein Reservoir

Address:	**3700 Bluebird Canyon Road**	**Evaluation Criteria:**		**Scoring Criteria:**	
	Vista, CA 92084	(1)	Structural	0	No action required
Inspection Date:	**11/15/2016**	(2)	Site (non-reservoir)	1	Minor (7+ years)
HDR Team:	Frost, Heraypur, Yarn	(3)	Aesthetic (reservoir only)	3	Moderate (2-6 years)
		(4)	Safety/Security	5	Immediate (0-2 years)
				N/A	Not Applicable

Pechstein Reservoir Civil/Site Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
S1	Fence	2,4	Chain link fence with barbed wire on top observed to be in good condition	0
S2	Gate(s)	2,4	Chain link gate observed to be in good condition	0
S3	Identification Signage	2	Site signage includes signs located on the entrance gate and along the surrounding fence	0
S4	Security	4	Fence at adequate height and front gate contained lock	0
S5	Access Road	2	Continuous cracks along pavement surrounding reservoir is not currently of concern but should be periodically monitored	1
S6	Trees & Vegetation	2,4	Typical trees and vegetation outside of the site fence	0
S7	Drainage	2	Roof gutters allow runoff into surrounding channel, minimal ponding observed. Typical accumulation of debris at roof drains	3
S8	Stability / Geotechnical	2,4	N/A	N/A
S9	Site Piping & Appurtenances	2	N/A	N/A
			Total Civil/Site Score	**4**

Reservoir Civil Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
C1	Roof Hatch	3,4	Rust spots on hatches and chipped off paint from access door interior	3
C2	Safety Signage	4	No confined space signage installed at roof hatch location	1
C3	Vent Condition	3	The roof vent had areas of rusting and loose mesh; Exposed wood at the center vent should be recoated	5
C4	Ladders & Stairs	4	Ladder contained minor rusting but in overall good condition; Stairs localized corrosion was observed on the interior door surface	1
C5	Handrails/Guardrails	4	Galvanized guardrails at reservoir entrance stairway and roof access ladder were found to be in good condition	0
C6	Overflow Pipe	3	N/A	N/A
			Total Score	**10**

Structural Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
ST1	Foundation	1	N/A	N/A
ST2	Wall	1	Staining of exterior wall surface; Cracking of stucco finish	3
ST3	Roof	1	Deteriorated wood; Damaged or missing roof deck connections and supports; Missing foam insulation; Poor roof drainage configuration	5
ST4	Interior Structure	1	Corrosion and failure of wood connections at the interior of the reservoir	5
			Total Score	**13**
			Grand Total Score	**27**

TABLE A-4

Vista Irrigation District
Triage Site Visit Results &
Office Analysis Results

HB Reservoir

Address:	**3791 Buena Creek Road**	(1)	Structural	0	No action required
	Vista, CA 92084	(2)	Site (non-reservoir)	1	Minor (7+ years)
Inspection Date:	**11/15/2016**	(3)	Aesthetic (reservoir only)	3	Moderate (2-6 years)
HDR Team:	Frost, Heraypur, Yarn	(4)	Safety/Security	5	Immediate (0-2 years)
				N/A	Not Applicable

Evaluation Criteria: (1)–(4) above; Scoring Criteria: 0, 1, 3, 5, N/A above.

HB Reservoir Civil/Site Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
S1	Fence	2,4	No VID fence surrounding the perimeter of the reservoir	5
S2	Gate(s)	2,4	Single swing gate observed to be in good condition	0
S3	Identification Signage	2	Signs located on the entrance gate; No signage located along the perimeter of the reservoir	5
S4	Security	4	No security fence surrounding the perimeter of the reservoir; Front gate contains lock	3
S5	Access Road	2	Access road pavement is in good condition with minor cracking observed	1
S6	Trees & Vegetation	2,4	Surrounding trees and vegetation out of vicinity of site operation and maintenance	0
S7	Drainage	2	Roof drains located throughout perimeter of reservoir; Good slope for runoff into drain	0
S8	Stability / Geotechnical	2,4	N/A	N/A
S9	Site Piping & Appurtenances	2	External pipe riser and blow-off in enclosure near site gate have surface rusting and chipped paint	1
			Total Civil/Site Score	**15**

Reservoir Civil Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
C1	Roof Hatch	3,4	Exposed corroded reinforcing at roof access hatch	3
C2	Safety Signage	4	No confined space signage installed on roof hatch	1
C3	Vent Condition	3	Minor rusting developed near vent mesh, corroded nut and washer on vent base plate	3
C4	Ladders & Stairs	4	Exterior ladder at the time of inspections is scheduled to be replaced Interior ladder has corroded brackets which need to be replaced	5
C5	Handrails/Guardrails	4	Handrail and guardrail at the time of inspections is scheduled to be replaced	N/A
C6	Overflow Pipe	3	N/A	N/A
			Total Score	**12**

Structural Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
ST1	Foundation	1	Concrete cracking and scaling	3
ST2	Wall	1	Cracking of shotcrete finish	3
ST3	Roof	1	Concrete cracking; Poor roof drainage; Exposed reinforcing	3
ST4	Interior Structure	1	No Interior Inspection	N/A
			Total Score	**9**
			Grand Total Score	**36**

TABLE A-5

Vista Irrigation District
Triage Site Visit Results &
Office Analysis Results

C Reservoir

Address:	**1301 Summit Terrace**	**Evaluation Criteria:**		**Scoring Criteria:**	
	Vista, CA 92083	(1)	Structural	0	No action required
Inspection Date:	**11/15/2016**	(2)	Site (non-reservoir)	1	Minor (7+ years)
HDR Team:	Frost, Heraypur, Yarn	(3)	Aesthetic (reservoir only)	3	Moderate (2-6 years)
		(4)	Safety/Security	5	Immediate (0-2 years)
				N/A	Not Applicable

C Reservoir Civil/Site Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
S1	Fence	2,4	VID fence does not surround entire perimeter of reservoir; Partial wood fence separates adjacent properties	5
S2	Gate(s)	2,4	No VID main site access gate	5
S3	Identification Signage	2	Site signage includes VID and Tresspassing signs	0
S4	Security	4	No security gate at the entrance could result in unauthorized entry	5
S5	Access Road	2	Paved access road provides adequate space. Contains areas of uneven surface and minor cracks that are not of concern but should be monitored	1
S6	Trees & Vegetation	2,4	Typical trees and vegetation on site; Minor shrubbery around perimeter of reservoir	0
S7	Drainage	2	Paved gutter around perimeter of reservoir observed to be clear and good condition	0
S8	Stability / Geotechnical	2,4	N/A	N/A
S9	Site Piping & Appurtenances	2	N/A	N/A
			Total Civil/Site Score	**16**

Reservoir Civil Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
C1	Roof Hatch	3,4	Minor corrosion on interior of aluminum hatch on reservoir roof	1
C2	Safety Signage	4	No confined space signage installed on roof hatch or overflow hatches	1
C3	Vent Condition	3	Minor surface rusting on roof vent cap, Bug screen is loose and torn at one corner	3
C4	Ladders & Stairs	4	Exterior wood ladder leading to guardrail does not comply with OSHA Standards.	5
C5	Handrails/Guardrails	4	Toe board is missing, Aluminum guardrail observed to be in overall good condition	3
C6	Overflow Pipe	3	N/A	N/A
			Total Score	**13**

Structural Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
ST1	Foundation	1	N/A	N/A
ST2	Wall	1	Wall cracking; Staining of exterior wall surface	1
ST3	Roof	1	Poor roof drainage; Roof deflection under load	3
ST4	Interior Structure	1	No Interior Inspection	N/A
			Total Score	**4**
			Grand Total Score	**33**

TABLE A-6

Vista Irrigation District
Triage Site Visit Results &
Office Analysis Results

E1 Reservoir

Address:	**1122 Cabrillo Circle**	(1)	Structural	0	No action required
	Vista, CA 92084	(2)	Site (non-reservoir)	1	Minor (7+ years)
Inspection Date:	**11/15/2016**	(3)	Aesthetic (reservoir only)	3	Moderate (2-6 years)
HDR Team:	Frost, Heraypur, Yarn	(4)	Safety/Security	5	Immediate (0-2 years)
				N/A	Not Applicable

Evaluation Criteria: (1)–(4) above; Scoring Criteria: 0, 1, 3, 5, N/A above.

E1 Reservoir Civil/Site Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
S1	Fence	2,4	VID chain link fence surrounding entire perimeter of reservoir observed to be in good condition	0
S2	Gate(s)	2,4	Chain link gate observed to be in good condition	0
S3	Identification Signage	2	Signage observed to be aging with fading text	5
S4	Security	4	Fence at adequate height and front gate contained lock	0
S5	Access Road	2	Paved access road and a set of stairs lead to the reservoir entrance	0
S6	Trees & Vegetation	2,4	Tree growth over a majority of the fence along the west side; two large trees located within five feet of reservoir Imposing accessibility interference	5
S7	Drainage	2	Runoff drains to surrounding dirt area, Good slope down to access road	0
S8	Stability / Geotechnical	2,4	N/A	N/A
S9	Site Piping & Appurtenances	2	N/A	N/A
			Total Civil/Site Score	**10**

Reservoir Civil Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
C1	Roof Hatch	3,4	Surface rusting observed on lock cover of roof hatch; Corroded screws on latch mechanism and Minor staining on interior of aluminum hatch	3
C2	Safety Signage	4	No confined space signage installed on roof hatch or overflow hatches	1
C3	Vent Condition	3	Cracks in caulking at vent bottom; Center vent observed to be in overall good condition	3
C4	Ladders & Stairs	4	Exterior ladder has minor surface rusting and first step bracket is too high	5
C5	Handrails/Guardrails	4	Guardrail is located on the reservoir roof at the hatch; Missing toe boards on roof	3
C6	Overflow Pipe	3	N/A	N/A
			Total Score	**15**

Structural Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
ST1	Foundation	1	N/A	N/A
ST2	Wall	1	Wall cracking; Staining of exterior wall surface	1
ST3	Roof	1	Poor roof drainage; Roof deflection under load	3
ST4	Interior Structure	1	Roof beam corrosion	3
			Total Score	**7**
			Grand Total Score	**32**

TABLE A-7

Vista Irrigation District
Triage Site Visit Results &
Office Analysis Results

San Luis Rey Reservoir

Address:	**1700 Anza Avenue**
	Vista, CA 92084
Inspection Date:	**11/16/2016**
HDR Team:	Frost, Heraypur, Yarn

Evaluation Criteria:

(1)	Structural
(2)	Site (non-reservoir)
(3)	Aesthetic (reservoir only)
(4)	Safety/Security

Scoring Criteria:

0	No action required
1	Minor (7+ years)
3	Moderate (2-6 years)
5	Immediate (0-2 years)
N/A	Not Applicable

San Luis Rey Civil/Site Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
S1	Fence	2,4	Security fence partially surrounding perimeter of reservoir	3
S2	Gate(s)	2,4	Typical chain link gate located at entrance in good condition	0
S3	Identification Signage	2	Signage observed to be aging with fading text	5
S4	Security	4	Front gate at adequate height and contained lock	0
S5	Access Road	2	Areas of minor cracking along pavement but was observed to be in overall good condition	0
S6	Trees & Vegetation	2,4	Shrubbery located on top of reservoir appears to be well maintained	0
S7	Drainage	2	N/A	N/A
S8	Stability / Geotechnical	2,4	N/A	N/A
S9	Site Piping & Appurtenances	2	N/A	N/A
			Total Civil/Site Score	**8**

Reservoir Civil Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
C1	Roof Hatch	3,4	Significant corrosion was observed on entry hatch locks and hinge hardware; Corrosion of the abandoned anchors in the concrete should be addressed	3
C2	Safety Signage	4	N/A	N/A
C3	Vent Condition	3	Some rust bleed and cracking of mortar; Vent cap has been recoated	1
C4	Ladders & Stairs	4	Interior stainless steel ladder found to be in good condition	0
C5	Handrails/Guardrails	4	N/A	N/A
C6	Overflow Pipe	3	N/A	N/A
			Total Score	**4**

Structural Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
ST1	Foundation	1	N/A	N/A
ST2	Wall	1	N/A	N/A
ST3	Roof	1	Concrete cracking	1
ST4	Interior Structure	1	Corroded anchors	1
			Total Score	**2**
			Grand Total Score	**14**

TABLE A-8

Vista Irrigation District
Triage Site Visit Results &
Office Analysis Results

H Reservoir

Address:	**2082 Pleasant Heights Drive**	**Evaluation Criteria:**		**Scoring Criteria:**	
	Vista, CA 92084	(1)	Structural	0	No action required
Inspection Date:	**11/16/2016**	(2)	Site (non-reservoir)	1	Minor (7+ years)
HDR Team:	Frost, Heraypur, Yarn	(3)	Aesthetic (reservoir only)	3	Moderate (2-6 years)
		(4)	Safety/Security	5	Immediate (0-2 years)
				N/A	Not Applicable

H Reservoir Civil/Site Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
S1	Fence	2,4	Site perimeter is enclosed by a chain link fence observed to be in good condition	0
S2	Gate(s)	2,4	Typical rolling steel gate observed to be in good condition	0
S3	Identification Signage	2	Signage on site includes entrance, no trespassing and caution signs	0
S4	Security	4	Fence at adequate height and front gate contained lock	0
S5	Access Road	2	Access road surrounding reservoir has cases of uneven pavement and cracking	5
S6	Trees & Vegetation	2,4	Fence contains overgrown shrubbery in certain areas; Tree growth over fence all along NE side	5
S7	Drainage	2	Concrete lined ditch surrounding reservoir boundary observed to be in good condition	0
S8	Stability / Geotechnical	2,4	N/A	N/A
S9	Site Piping & Appurtenances	2	N/A	N/A
			Total Civil/Site Score	**10**

Reservoir Civil Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
C1	Roof Hatch	3,4	Hatches interior conduit brackets were found to be corroded	3
C2	Safety Signage	4	No confined space signage installed on roof hatches	1
C3	Vent Condition	3	Center vent assembly was observed to be in good condition	0
C4	Ladders & Stairs	4	Stainless steel interior ladder located in pump station observed to be in good condition; Stairway on NE side of reservoir entrance is missing inner handrail	3
C5	Handrails/Guardrails	4	Three guardrails located on roof observed to be in good condition	0
C6	Overflow Pipe	3	Spot corrosion observed on overflow pipe	3
			Total Score	**10**

Structural Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
ST1	Foundation	1	N/A	N/A
ST2	Wall	1	Shotcrete cracking requires monitoring	1
ST3	Roof	1	Cracking on the roof deck should be addressed to prevent long-term damage	3
ST4	Interior Structure	1	No Interior Inspection	N/A
			Total Score	**4**
			Grand Total Score	**24**

TABLE A-9

Vista Irrigation District
Triage Site Visit Results &
Office Analysis Results

MD Reservoir

Address:	**2093 Rockhoff Road**	
	Escondido, CA 92026	
Inspection Date:	**11/16/2016**	
HDR Team:	Frost, Heraypur, Yarn	

Evaluation Criteria:

(1)	Structural
(2)	Site (non-reservoir)
(3)	Aesthetic (reservoir only)
(4)	Safety/Security

Scoring Criteria:

0	No action required
1	Minor (7+ years)
3	Moderate (2-6 years)
5	Immediate (0-2 years)
N/A	Not Applicable

MD Reservoir Civil/Site Observations				
Item No.	**Description**	**Criteria Class**	**Inspection Notes**	**Score (0-5)**
S1	Fence	2,4	Reservoir is bounded by chain link fence; overgrown shrub and bushes conflict with fence	5
S2	Gate(s)	2,4	Typical swing steel gate with barbed wire observed to be in good condition	0
S3	Identification Signage	2	Site signage includes signs located on the entrance gate and surrounding reservoir	0
S4	Security	4	Fence at adequate height and front gate contained lock	0
S5	Access Road	2	Paved access road leads up to entrance and gravel surrounding reservoir, Erosion on access road west side of reservoir	3
S6	Trees & Vegetation	2,4	Excessive plant growth in multiple areas encroaches perimeter fence which caused damage	5
S7	Drainage	2	Concrete lined ditch NW of entrance contains minor sediments but was in overall good condition	0
S8	Stability / Geotechnical	2,4	Major case of erosion in O&M gravel road NE of entrance causing hazardous slope	5
S9	Site Piping & Appurtenances	2	N/A	N/A
			Total Civil/Site Score	**18**

Reservoir Civil Observations				
Item No.	**Description**	**Criteria Class**	**Inspection Notes**	**Score (0-5)**
C1	Roof Hatch	3,4	Roof hatch was observed to have surface rusting on the lock cover; Steel screws were found corroded	3
C2	Safety Signage	4	No confined space signage installed on roof hatches	1
C3	Vent Condition	3	Center vent was found to be in good condition	0
C4	Ladders & Stairs	4	Stainless steel external ladder observed to be in good condition; Spot corrosion and pitting on interior platform and ladder	3
C5	Handrails/Guardrails	4	Top of ladder does not have handrail which is a safety hazard; Roof guardrails were found in good condition	3
C6	Overflow Pipe	3	N/A	N/A
			Total Score	**10**

Structural Observations				
Item No.	**Description**	**Criteria Class**	**Inspection Notes**	**Score (0-5)**
ST1	Foundation	1	N/A	N/A
ST2	Wall	1	Wall cracking; Staining of exterior wall surface	1
ST3	Roof	1	Poor roof drainage; Roof deflection under load	3
ST4	Interior Structure	1	No Interior Inspection	N/A
			Total Score	4
			Grand Total Score	**32**

TABLE A-10

Vista Irrigation District
Triage Site Visit Results &
Office Analysis Results

Deodar Reservoir

Address:	**947 Deodar Road**	**Evaluation Criteria:**		**Scoring Criteria:**	
	San Marcos, CA 92069	(1)	Structural	0	No action required
Inspection Date:	**11/16/2016**	(2)	Site (non-reservoir)	1	Minor (7+ years)
HDR Team:	Frost, Heraypur, Yarn	(3)	Aesthetic (reservoir only)	3	Moderate (2-6 years)
		(4)	Safety/Security	5	Immediate (0-2 years)
				N/A	Not Applicable

Deodar Civil/Site Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
S1	Fence	2,4	No VID site enclosure fence	5
S2	Gate(s)	2,4	Two typical swing steel gates observed to be in good condition	0
S3	Identification Signage	2	Signage on site is located on the entrance and exit gate; Signage was observed to be in good condition	0
S4	Security	4	Fence at adequate height and front gate contained lock	0
S5	Access Road	2	Paved access road barely accessible for one crew vehicle; Erosion on access road west side of reservoir. Paved road observed to be in good condition	3
S6	Trees & Vegetation	2,4	Trees and vegetation growth surrounding reservoir within distance does not conflict with operations	0
S7	Drainage	2	Roof drains surrounding reservoir contain good slope for runoff on North side	0
S8	Stability / Geotechnical	2,4	North end of reservoir consists of deep slope from end of pavement to adjacent landscape	5
S9	Site Piping & Appurtenances	2	N/A	N/A
			Total Civil/Site Score	**13**

Reservoir Civil Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
C1	Roof Hatch	3,4	Roof hatch found to have corroded screws and hinge	5
C2	Safety Signage	4	No confined space signage installed on roof hatch	1
C3	Vent Condition	3	Center vent was found to be in good condition	0
C4	Ladders & Stairs	4	Transition from ladder to guardrail is secure; gap between safety climb device and guardrail is narrow; Interior stainless ladder and guardrail were found to be in good condition	1
C5	Handrails/Guardrails	4	Guardrails were found to be in good condition	0
C6	Overflow Pipe	3	Spot corrosion observed on overflow pipe	3
			Total Score	**10**

Structural Observations

Item No.	Description	Criteria Class	Inspection Notes	Score (0-5)
ST1	Foundation	1	N/A	N/A
ST2	Wall	1	Staining of exterior wall surface; Corrosion of stucco trim	3
ST3	Roof	1	Deteriorated wood; Damaged or missing roof deck connections and supports; Missing foam insulation; Poor roof drainage configuration	5
ST4	Interior Structure	1	No Interior Inspection	N/A
			Total Score	**8**
			Grand Total Score	**31**

Appendix B. VID Reservoirs – Probable Cost Opinions

Vista Irrigation District Lupine Hills Reservoir Probable Cost Opinion						
No.	**Description**	**Unit**	**Quantity**	**Unit Price**	**Subtotal**	**Total**
1	Reservoir Roof Improvements					$24,892
	*Modify the roof slope, as required, to prevent ponding and provide proper drainage	SF	1,500	$9.68	$14,521.24	
	Seal all cracking on the roof deck and hatch curb	LF	768	$6.05	$4,646.80	
	Replace the joint material at the roof/wall interface and seal the joint	LF	430	$13.31	$5,723.79	
2	Reservoir Ventilation Improvements					$450
	Remove rust from the vents conduit plate and repaint conduit plate	LS	1	$200	$200	
	Replace conduit brackets	EA	10	$25	$250	
3	Reservoir Exterior Improvements					$3,881
	Replace corroded screws on the hatches and doors with stainless hardware	LS	1	$400	$400	
	Repaint the anti-climb device on the exterior ladder	LS	1	$200	$200	
	Replace guardrails with bolt-down style guardrails using stainless steel anchor bolts	LF	50	$50	$2,500	
	Clean all staining of the exterior wall surface	SF	645	$1.21	$780.52	
4	Site Improvements					$2,000
	Remove all vegetation in close proximity to the reservoir and fence. Ensure roof gutters are periodically cleaned	LS	1	$2,000	$2,000	
				Total Direct Costs		$31,222
			Mobilization/Demobilization	3.00%		$937
			Contractor's Contingency	5.00%		$1,561
			Construction Fee	7.00%		$2,186
			General Conditions	10.00%		$3,122
			Tax, Insurance & Bond	11.00%		$3,434
				Total Indirect Costs		$11,240
				Probable Construction Cost (Rounded)		$43,000
			Owner's Contingency	10.00%		$4,300
				Total Probable Construction Costs (Rounded)		**$48,000**

*Does not include interior roof repair which will be evaluated under Additional Assessments.

Vista Irrigation District - Additional Assessments Lupine Hills Reservoir Probable Cost Opinion - Recommendations for Further Investigation						
No.	**Description**	**Unit**	**Quantity**	**Unit Price**	**Subtotal**	**Total**
Additional Assessments						$61,000
1	Perform a detailed condition assessment of the reservoir interior (Assumes a raft (float) inspection, dry inspection, roof inspection and preparation of a report)	LS	1	$41,000	$41,000	
2	Perform a seismic evaluation of the reservoir (includes predesign report, roof evaluation, and ventilation)	LS	1	$20,000	$20,000	
				Total Direct Costs (Rounded)		**$61,000**

Vista Irrigation District
A Reservoir Probable Cost Opinion

No.	Description	Unit	Quantity	Unit Price	Subtotal	Total
1	Reservoir Roof Improvements					$5,808
	Repair soft spots and termite damage on the existing roof system	Pending Detailed Interior Inspection				
	*Modify the roof slope, as required, to prevent ponding and provide proper drainage	SF	600	$9.68	$5,808.50	
2	Reservoir Interior Improvements					$5,100
	Repair the spalling in the concrete beam above the overflow	LS	1	$1,500	$1,500	
	Clean and coat corroding sill bolts on ring walls	EA	80	$30	$2,400	
	Clean and coat corroding roof beams	EA	4	$300	$1,200	
3	Reservoir Exterior Improvements					$4,599
	Remove surface rust and touch up paint on exterior pipe riser	LS	1	$200	$200	
	Replace corroded pipe spool	EA	1	$1,000	$1,000	
	Replace "NO TRESPASSING" sign	EA	1	$150	$150	
	Install confined space signage on roof hatch	EA	1	$150	$150	
	Remove rust and repaint lock cover on the roof hatch	EA	1	$100	$100	
	Remove surface rust and recoat steel support beam in float box	LS	1	$500	$500	
	Clean all staining of the exterior wall surface	SF	315	$1.21	$381.18	
	Seal all cracking in the exterior wall	LF	350	$6.05	$2,117.68	
4	Site Improvements					$15,750
	Replace partial chain link fence and gate with full chain link fence surrounding reservoir property	LS	1	$15,750	$15,750	
5	Landscaping					$3,000
	Remove all vegetation in close proximity to the reservoir, other structures and piping	LS	1	$3,000	$3,000	
	Total Direct Costs					**$34,257**
	Mobilization/Demobilization			3.00%		$1,028
	Contractor's Contingency			5.00%		$1,713
	Construction Fee			7.00%		$2,398
	General Conditions			10.00%		$3,426
	Tax, Insurance & Bond			11.00%		$3,768
	Total Indirect Costs					**$12,333**
	Probable Construction Cost (Rounded)					$47,000
	Owner's Contingency			10.00%		$4,700
	Total Probable Construction Costs (Rounded)					**$52,000**

*Does not include interior roof repair which will be evaluated under Additional Assessments.

Vista Irrigation District - Additional Assessments
A Reservoir Probable Cost Opinion - Recommendations for Further Investigation

No.	Description	Unit	Quantity	Unit Price	Subtotal
Additional Assessments					
1	Perform a detailed condition assessment of the reservoir interior (Includes dry inspection, roof inspection and preparation of a report)	LS	1	$21,000	$21,000
2	Perform a seismic evaluation of the reservoir (Includes predesign report, roof evaluation, and ventilation)	LS	1	$10,000	$10,000
	Total Direct Costs (Rounded)				**$31,000**

Vista Irrigation District - Roof Replacement Options
A Reservoir Probable Cost Opinion - Recommendations for Further Investigation

No.	Description	Unit	Quantity	Unit Price	Total Direct Costs	Mobilization/ Demobilization	Contractor's Contingency	Construction Fee	General Conditions	Tax, Insurance & Bond	Total Indirect Costs	Probable Construction Cost (Rounded)	Owner's Contingency	Total Probable Construction Costs (Rounded)
						3.00%	5.00%	7.00%	10.00%	11.00%			10.00%	
Roof Replacement Options														
1	Reservoir Roof Replacement - Aluminum Roof	LS	1	$236,000	$236,000	$7,080	$11,800	$16,520	$23,600	$25,960	$84,960	$321,000	$32,100	$354,000
2	Reservoir Roof Replacement - Aluminum Roof, Column Supported Dome	LS	1	$330,000	$330,000	$9,900	$16,500	$23,100	$33,000	$36,300	$118,800	$449,000	$44,900	$494,000
3	Reservoir Roof Replacement - Flat Slab	LS	1	$550,000	$550,000	$16,500	$27,500	$38,500	$55,000	$60,500	$198,000	$748,000	$74,800	$823,000
4	Reservoir Roof Replacement - Free Spanning Dome (Optimal dimensions: 28' SWD, 2' Freeboard, 68' Inner Diamete	LS	1	$1,225,000	$1,225,000	$36,750	$61,250	$85,750	$122,500	$134,750	$441,000	$1,666,000	$166,600	$1,833,000
5	Reservoir Roof Replacement - Column Supported Flat Slab Roof (Optimal dimensions: 28' SWD, 2' Freeboard, 68' Inner Diamete	LS	1	$1,275,000	$1,275,000	$38,250	$63,750	$89,250	$127,500	$140,250	$459,000	$1,734,000	$173,400	$1,908,000

NOTE - Reservoir replacement is not recommended until further detailed condition assessment of the reservoir interior is completed.

Vista Irrigation District - Reservoir Replacement
A Reservoir Probable Cost Opinion

No.	Description	Unit	Quantity	Unit Price	Subtotal
1	**Reservoir Replacement** - Based on total capacity and includes: planning, engineering design, environmental, legal, construction, limited site work, piping upgrades, valve replacements, re-painting, and coating, construction management and contract administration. Reservoir unit price includes a 25 percent factor for costs associated with demolition and removal.	GAL	800,000	$2.00	$1,600,000
			Construction Contingency	**30%**	$480,000
			Total Direct Costs (Rounded)		**$2,080,000**

Vista Irrigation District
Pechstein Reservoir Probable Cost Opinion

No.	Description	Unit	Quantity	Unit Price	Subtotal	Total
1	Reservoir Roof Improvements					$45,500
	Clean and coat deteriorated exposed wood	LS	1	$10,000	$10,000	
	Replace all missing foam insulation	LS	1	$3,500	$3,500	
	Replace missing or corroded roof deck connectors	LS	1	$2,000	$2,000	
	Reconfigure the roof drains	LS	1	$30,000	$30,000	
2	Reservoir Ventilation Improvements					$150
	Secure mesh over roof vent and remove any areas of rusting	LS	1	$150	$150	
3	Reservoir Exterior Improvements					$822
	Remove rust spots on hatches and touch up chipped off paint from access door interior	LS	1	$200	$200	
	Clean all staining of the exterior wall surface	SF	390	$1.21	$472	
	Install confined space signage on roof hatch	EA	1	$150	$150	
4	Site Improvements					$600
	Algae treatment on channel under roof gutters	LS	1	$600	$600	
					Total Direct Costs	$47,072
				Mobilization/Demobilization	3.00%	$1,412
				Contractor's Contingency	5.00%	$2,354
				Construction Fee	7.00%	$3,295
				General Conditions	10.00%	$4,707
				Tax, Insurance & Bond	11.00%	$5,178
					Total Indirect Costs	$16,946
					Probable Construction Cost (Rounded)	$65,000
				Owner's Contingency	10.00%	$6,500
					Total Probable Construction Costs (Rounded)	**$72,000**

Vista Irrigation District - Additional Assessments
Pechstein Reservoir Probable Cost Opinion - Recommendations for Further Investigation

No.	Description	Unit	Quantity	Unit Price	Subtotal
Additional Assessments					
1	Perform a detailed condition assessment of the reservoir interior (Assumes a raft (float) inspection, dry inspection, roof inspection and preparation of a report)	LS	1	$41,000	$41,000
2	Perform a seismic evaluation of the reservoir (Includes predesign report, roof evaluation, and ventilation)	LS	1	$40,000	$40,000
				Total Direct Costs (Rounded)	**$81,000**

Vista Irrigation District - Roof Replacement Options
Pechstein Reservoir Probable Cost Opinion - Recommendations for Further Investigation

No.	Description	Unit	Quantity	Unit Price	Total Direct Costs	Mobilization/ Demobilization	Contractor's Contingency	Construction Fee	General Conditions	Tax, Insurance & Bond	Total Indirect Costs	Probable Construction Cost (Rounded)	Owner's Contingency	Total Probable Construction Costs (Rounded)
						3.00%	5.00%	7.00%	10.00%	11.00%			10.00%	
Roof Replacement Options														
1	Reservoir Roof Replacement - Aluminum Roof, Clear Span Dome	LS	$1	$2,470,000	$2,470,000	$74,100	$123,500	$172,900	$247,000	$271,700	$889,200	$3,360,000	$336,000	$3,696,000
2	Reservoir Roof Replacement - Aluminum Roof, Column Supported Dome	LS	$1	$2,870,000	$2,870,000	$86,100	$143,500	$200,900	$287,000	$315,700	$1,033,200	$3,904,000	$390,400	$4,295,000
3	Reservoir Roof Replacement - Flat Slab	LS	$1	$3,500,000	$3,500,000	$105,000	$175,000	$245,000	$350,000	$385,000	$1,260,000	$4,760,000	$476,000	$5,236,000
4	Reservoir Roof Replacement - Column Supported Flat Slab Roof (Optimal dimensions: 38' SWD, 2' Freeboard, 300' Inner Diamete	LS	$1	$9,300,000	$9,300,000	$279,000	$465,000	$651,000	$930,000	$1,023,000	$3,348,000	$12,648,000	$1,264,800	$13,913,000

NOTE - Reservoir replacement is not recommended until further detailed condition assessment of the reservoir interior is completed.

Vista Irrigation District
Pechstein Reservoir Probable Cost Opinion - Reservoir Replacement

No.	Description	Unit	Quantity	Unit Price	Subtotal
1	**Reservoir Replacement -** Based on total capacity and includes: planning, engineering design, environmental, legal, construction, limited site work, piping upgrades, valve replacements, re-painting, and coating, construction management and contract administration. Reservoir unit price includes a 25 percent factor for costs associated with demolition and removal.	GAL	20,000,000	$1.00	$20,000,000
			Construction Contingency	30%	$6,000,000
				Total Direct Costs (Rounded)	$26,000,000

Vista Irrigation District
C Reservoir Probable Cost Opinion

No.	Description	Unit	Quantity	Unit Price	Subtotal	Total
1	Reservoir Roof Improvements					$5,808
	*Modify the roof slope, as required, to prevent ponding and provide proper drainage	SF	600	$9.68	$5,808.50	
2	Reservoir Ventilation Improvements					$200
	Replace roof vents mesh covering	LS	1	$200	$200	
3	Reservoir Interior Improvements					$2,400
	Clean and coat corroding sill bolts on ring walls	EA	80	$30	$2,400	
4	Reservoir Exterior Improvements					$7,457
	Install toe boards on guardrail system	LF	60	$3	$180	
	Clean all staining of the exterior wall surface	SF	282	$1.21	$341.25	
	Seal all cracking in the exterior wall	LF	320	$6.05	$1,936.17	
	Modify fixed exterior ladder to make OSHA compliant	LS	1	$5,000	$5,000	
5	Site Improvements					$6,500
	Install main access gate with intrusion alarms	LS	1	$6,050	$6,050	
	Install VID signage on new access gate and security fence	EA	2	$150	$300	
	Install confined space signage on roof hatch	EA	1	$150	$150	
6	Landscaping					$1,500
	Remove all vegetation within five feet from fence	LS	1	$1,500	$1,500	
					Total Direct Costs	$23,866
	Mobilization/Demobilization			3.00%		$716
	Contractor's Contingency			5.00%		$1,193
	Construction Fee			7.00%		$1,671
	General Conditions			10.00%		$2,387
	Tax, Insurance & Bond			11.00%		$2,625
					Total Indirect Costs	$8,592
					Probable Construction Cost (Rounded)	$33,000
	Owner's Contingency			10.00%		$3,300
					Total Probable Construction Costs (Rounded)	**$37,000**

*Does not include interior roof repair which will be evaluated under Additional Assessments

Vista Irrigation District - Additional Assessments
C Reservoir Probable Cost Opinion - Recommendations for Further Investigation

No.	Description	Unit	Quantity	Unit Price	Subtotal	Total
Additional Assessments						$10,000
1	Perform a seismic evaluation of the reservoir (Includes predesign report, roof evaluation, and ventilation)	LS	1	$10,000	$10,000	
					Total Direct Costs (Rounded)	**$10,000**

Vista Irrigation District
HB Reservoir Probable Cost Opinion

No.	Description	Unit	Quantity	Unit Price	Subtotal	Total
1	Reservoir Roof Improvements					$27,562
	*Modify the roof slope, as required, to prevent ponding and provide proper drainage	SF	400	$9.68	$3,872.33	
	Repair the exposed reinforcing at the access hatch	LS	1	$500	$500	
	Seal leaks in liner, monitor prestress wire		Pending Detailed Interior Inspection			
	Seal all cracking on the roof concrete	SF	1,250	$18.15	$22,689.44	
	Repair the exposed reinforcing at the center vent curb	LS	1	$500	$500	
2	Reservoir Ventilation Improvements					$100
	Remove minor rust build up on interior of roof vent	LS	1	$100	$100	
3	Reservoir Interior Improvements					$800
	Replace corroded ladder brackets on interior ladder	LS	1	$800	$800	
4	Reservoir Exterior Improvements					$1,081
	Remove surface rust and repaint corroded areas on exterior piping in enclosure	LS	1	$400	$400	
	Seal all cracking and scaling of the foundation concrete	SF	38	$18.15	$680.68	
5	Site Improvements					$25,550
	Install new chain link security fence surrounding reservoir boundary	LS	1	$25,100	$25,100	
	Provide "NO TRESSPASSING" signage along new chain link fence	LS	2	$150	$300	
	Install confined space signage on roof hatch	EA	1	$150	$150	
6	Landscaping					$2,000
	Remove all vegetation in close proximity to the reservoir entrance gate, other structures and piping	LS	1	$2,000	$2,000	
				Total Direct Costs		$57,092
			Mobilization/Demobilization	3.00%		$1,713
			Contractor's Contingency	5.00%		$2,855
			Construction Fee	7.00%		$3,996
			General Conditions	10.00%		$5,709
			Tax, Insurance & Bond	11.00%		$6,280
				Total Indirect Costs		$20,553
				Probable Construction Cost (Rounded)		$78,000
			Owner's Contingency	10.00%		$7,800
				Total Probable Construction Costs (Rounded)		**$86,000**

*Does not include interior roof repair which will be evaluated under Additional Assessments

Vista Irrigation District - Additional Assessments
HB Reservoir Probable Cost Opinion - Recommendations for Further Investigation

No.	Description	Unit	Quantity	Unit Price	Subtotal	Total
Additional Assessments						$61,000
1	Perform a detailed condition assessment of the reservoir interior (Assumes a raft (float) inspection, dry inspection, roof inspection and preparation of a report)	LS	1	$41,000	$41,000	
2	Perform a seismic evaluation of the reservoir (Includes predesign report, roof evaluation, and ventilation)	LS	1	$20,000	$20,000	
				Total Direct Costs (Rounded)		**$61,000**

Vista Irrigation District
E1 Reservoir Probable Cost Opinion

No.	Description	Unit	Quantity	Unit Price	Subtotal	Total
1	Reservoir Roof Improvements					$5,808
	*Modify the roof slope, as required, to prevent ponding and provide proper drainage	SF	600	$9.68	$5,808.50	
2	Reservoir Ventilation Improvements					$100
	Caulk bottom of roof vent	LS	1	$100	$100	
3	Reservoir Interior Improvements					$3,300
	Clean and coat corroding roof beams	EA	4	$300	$1,200	
	Clean and coat corroding sill bolts on ring walls	EA	70	$30	$2,100	
4	Reservoir Exterior Improvements					$7,857
	Add bracket on exterior ladder for easier roof access	LS	1	$300	$300	
	Install toe boards on guardrail system	LF	60	$3	$180	
	Clean all staining of the exterior wall surface	SF	282	$1.21	$341.25	
	Seal all cracking in the exterior wall	LF	320	$6.05	$1,936.17	
	Modify exterior ladder's first step rung such that its compliant with OSHA fixed ladder requirements	LS	1	$5,000	$5,000	
	Replace corroded screwed on lock cover of roof hatch	LS	1	$100	$100	
5	Site Improvements					$300
	Replace faded VID signage with black text signage	LS	1	$150	$150	
	Install confined space signage on roof hatch	EA	1	$150	$150	
6	Landscaping					$5,000
	Remove 2 large trees located on east side of fence and all vegetation within five feet from fence	LS	1	$5,000	$5,000	
					Total Direct Costs	$22,366
			Mobilization/Demobilization	3.00%		$671
			Contractor's Contingency	5.00%		$1,118
			Construction Fee	7.00%		$1,566
			General Conditions	10.00%		$2,237
			Tax, Insurance & Bond	11.00%		$2,460
					Total Indirect Costs	$8,052
					Probable Construction Cost (Rounded)	$30,000
			Owner's Contingency	10.00%		$3,000
					Total Probable Construction Costs (Rounded)	**$33,000**

*Does not include interior roof repair which will be evaluated under Additional Assessments.

Vista Irrigation District - Additional Assessments
E1 Reservoir Probable Cost Opinion - Recommendations for Further Investigation

No.	Description	Unit	Quantity	Unit Price	Subtotal	Total
Additional Assessments						$10,000
1	Perform a seismic evaluation of the reservoir (Includes predesign report, roof evaluation, and ventilation)	LS	1	$10,000	$10,000	
					Total Direct Costs (Rounded)	**$10,000**

Vista Irrigation District
San Luis Rey Reservoir Probable Cost Opinion

No.	Description	Unit	Quantity	Unit Price	Subtotal	Total
1	Reservoir Interior Improvements					$1,500
	Replace interior metal components	LS	1	$1,000	$1,000	
	Repair or remove abandoned anchors in concrete at access hatch	LS	1	$500	$500	
2	Reservoir Exterior Improvements					$303
	Seal all cracking in the concrete around the access hatch	LF	50	$6.05	$302.53	
3	Site Improvements					$1,150
	Replace faded signage	LS	1	$150	$150	
	Clear sediment and debris from the site structures	LS	1	$1,000	$1,000	
4	Landscaping					$4,000
	Remove all vegetation in close proximity to the reservoir fence and signage	LS	1	$4,000	$4,000	
	Total Direct Costs					$6,953
	Mobilization/Demobilization			3.00%		$209
	Contractor's Contingency			5.00%		$348
	Construction Fee			7.00%		$487
	General Conditions			10.00%		$695
	Tax, Insurance & Bond			11.00%		$765
	Total Indirect Costs					$2,504
	Probable Construction Cost (Rounded)					$9,000
	Owner's Contingency			10.00%		$900
	Total Probable Construction Costs (Rounded)					**$10,000**

Vista Irrigation District - Additional Assessments
San Luis Rey Reservoir Probable Cost Opinion - Recommendations for Further Investigation

No.	Description	Unit	Quantity	Unit Price	Subtotal	Total
Additional Assessments						$61,000
1	Perform a detailed condition assessment of the reservoir interior (Assumes a raft (float) inspection, dry inspection,roof inspection and preparation of a report)	LS	1	$41,000	$41,000	
2	Perform a seismic evaluation of the reservoir (Includes predesign report, roof evaluation, and ventilation)	LS	1	$20,000	$20,000	
	Total Direct Costs (Rounded)					**$61,000**

Vista Irrigation District
H Reservoir Probable Cost Opinion

No.	Description	Unit	Quantity	Unit Price	Subtotal	Total
1	Reservoir Roof Improvements					$19,918
	Replace corroded conduit brackets on roof; replace corroded conduit fittings inside hatch	EA	50	$15	$750	
	*Modify the roof slope, as required, to prevent ponding and provide proper drainage	SF	1,500	$9.68	$14,521.24	
	Seal all cracking on the roof deck and hatch curb	LF	768	$6.05	$4,646.80	
2	Reservoir Interior Improvements					$4,000
	Remove rust and touch up coating on overflow pipe structure	LS	1	$4,000	$4,000	
3	Reservoir Exterior Improvements					$3,400
	Install missing inner handrail on exterior stairs	LS	1	$3,000	$3,000	
	Remove rust and touch up paint on pipe flange and bolts on exterior piping	LS	1	$400	$400	
4	Site Improvements					$5,650
	Clear sediment and debris from the site drainage channel	LS	1	$1,000	$1,000	
	Spot repair cracking in the asphalt concrete access roadway along the perimeter of the reservoir using flowable asphalt	LS	1	$4,500	$4,500	
	Install confined space signage on roof hatch	EA	1	$150	$150	
5	Landscaping					$3,500
	Remove all vegetation in close proximity to the reservoir, fence, and other structures	LS	1	$2,000	$2,000	
	Remove all shrubbery and weeds intertwined into links of fence	LS	1	$1,500	$1,500	
	Total Direct Costs					$36,468
	Mobilization/Demobilization			3.00%		$1,094
	Contractor's Contingency			5.00%		$1,823
	Construction Fee			7.00%		$2,553
	General Conditions			10.00%		$3,647
	Tax, Insurance & Bond			11.00%		$4,011
	Total Indirect Costs					$13,128
	Probable Construction Cost (Rounded)					$50,000
	Owner's Contingency			10.00%		$5,000
	Total Probable Construction Costs (Rounded)					**$55,000**

*Does not include interior roof repair which will be evaluated under Additional Assessments.

Vista Irrigation District - Additional Assessments
H Reservoir Probable Cost Opinion - Recommendations for Further Investigation

No.	Description	Unit	Quantity	Unit Price	Subtotal	Total
Additional Assessments						$61,000
1	Perform a detailed condition assessment of the reservoir interior (Assumes a raft (float) inspection, dry inspection, , roof inspection and preparation of a report)	LS	1	$41,000	$41,000	
2	Perform a seismic evaluation of the reservoir (Includes predesign report, roof evaluation, and ventilation)	LS	1	$20,000	$20,000	
	Total Direct Costs (Rounded)					**$61,000**

Vista Irrigation District
MD Reservoir Probable Cost Opinion

No.	Description	Unit	Quantity	Unit Price	Subtotal	Total
1	Reservoir Roof Improvements					$5,808
	*Modify the roof slope, as required, to prevent ponding and provide proper drainage	SF	600	$9.68	$5,808.50	
2	Reservoir Interior Improvements					$1,750
	Remove rust and recoat corroded areas on interior ladder platform and supports	LS	1	$400	$400	
	Clean and coat corroding sill bolts on ring walls	EA	45	$30	$1,350	
3	Reservoir Exterior Improvements					$2,843
	Remove rust on roof hatch and repaint lock cover, replace corroded screws with stainless hardware	LS	1	$200	$200	
	Add ladder extensions above roof level	LS	1	$800	$800	
	Clean all staining of the exterior wall surface	SF	175	$1.21	$211.77	
	Seal all cracking in the exterior wall	LF	220	$6.05	$1,331.11	
	Remove corrosion and pitting on interior platform and ladder	LS	1	$300	$300	
4	Site Improvements					$1,450
	Clear sediment and debris from the site drainage channel	LS	1	$700	$700	
	Repair fence where excessive plan growth created damage	LS	1	$600	$600	
	Install confined space signage on roof hatch	EA	1	$150	$150	
5	Landscaping					$3,000
	Remove all vegetation in close proximity to the reservoir, fence and other structures	LS	1	$3,000	$3,000	
				Total Direct Costs		$14,851
			Mobilization/Demobilization	3.00%		$446
			Contractor's Contingency	5.00%		$743
			Construction Fee	7.00%		$1,040
			General Conditions	10.00%		$1,485
			Tax, Insurance & Bond	11.00%		$1,634
				Total Indirect Costs		$5,348
				Probable Construction Cost (Rounded)		$20,000
			Owner's Contingency	10.00%		$2,000
				Total Probable Construction Costs (Rounded)		**$22,000**

*Does not include interior roof repair which will be evaluated under Additional Assessments.

Vista Irrigation District - Additional Assessments
MD Reservoir Probable Cost Opinion - Recommendations for Further Investigation

No.	Description	Unit	Quantity	Unit Price	Subtotal	Total
Additional Assessments						$16,000
1	Perform a seismic evaluation of the reservoir (Includes predesign report, roof evaluation, and ventilation)	LS	1	$10,000	$10,000	
2	Geotechnical: Investigate the stability of the erosion on the west side of the reservoir	LS	1	$6,000	$6,000	
				Total Direct Costs (Rounded)		**$16,000**

Vista Irrigation District
Deodar Reservoir Probable Cost Opinion

No.	Description	Unit	Quantity	Unit Price	Subtotal	Total
1	Reservoir Roof Improvements					$22,750
	Clean and coat deteriorated exposed wood	LS	1	$5,000	$5,000	
	Replace all missing foam insulation to restore the weather-tightness of the reservoir	LS	1	$1,750	$1,750	
	Replace missing or corroded roof deck connectors	LS	1	$1,000	$1,000	
	Reconfigure the roof drains to prevent accumulation of debris and moisture at the low points	LS	1	$15,000	$15,000	
2	Reservoir Ventilation Improvements					
	Provide ventilation improvements for the reservoir to prevent accumulation of condensation and deterioration of the roof framing and its connections	LS	Pending Detailed Interior Inspection			
3	Reservoir Interior Improvements					$1,000
	Replace all missing foam insulation to restore the weather-tightness of the reservoir	SF	Pending Detailed Interior Inspection			
	Clean all corrosion of the interior metal components	LS	1	$1,000	$1,000	
4	Reservoir Exterior Improvements					$9,214
	Replace corroded screws and hinges with stainless hardware on hatch	LS	1	$400	$400	
	Replace corroded screws on conduit with stainless	LS	1	$200	$200	
	Remove rust and touch up coating on overflow pipe structure	LS	1	$4,000	$4,000	
	Repair the corrosion of the stucco trim on the masonry wall to maintain long-term serviceability of the metal	SF	150	$30	$4,500	
	Clean all staining of the exterior wall surface	SF	95	$1.21	$114	
5	Site Improvements					$14,000
	Install new chain link security fence surrounding reservoir boundary	LS	1	$13,550	$13,550	
	Install VID signage on new security fence	LS	2	$150	$300	
	Install confined space signage on roof hatch	EA	1	$150	$150	
6	Landscaping					$5,000
	Clear all vegetation within five feet from new fence installation	LS	1	$3,000	$3,000	
	Remove all vegetation in close proximity to the reservoir and access road	LS	1	$2,000	$2,000	
					Total Direct Costs	$51,964
				Mobilization/Demobilization	3.00%	$1,559
				Contractor's Contingency	5.00%	$2,598
				Construction Fee	7.00%	$3,638
				General Conditions	10.00%	$5,196
				Tax, Insurance & Bond	11.00%	$5,716
					Total Indirect Costs	$18,707
					Probable Construction Cost (Rounded)	$71,000
				Owner's Contingency	10.00%	$7,100
					Total Probable Construction Costs (Rounded)	**$79,000**

Vista Irrigation District - Additional Assessments
Deodar Reservoir Probable Cost Opinion - Recommendations for Further Investigation

No.	Description	Unit	Quantity	Unit Price	Subtotal
Additional Assessments					
1	Perform a detailed condition assessment of the reservoir interior (Includes dry inspection and roof inspection)	LS	1	$41,000	$41,000
2	Perform a seismic evaluation of the reservoir (Includes predesign report, roof evaluation, and ventilation)	LS	1	$10,000	$10,000
3	Geotechnical: Investigate the stability of the erosion on the northwest side of the reservoir entrance	LS	1	$6,000	$6,000
				Total Direct Costs (Rounded)	**$57,000**

Vista Irrigation District - Roof Replacement Options
Deodar Reservoir Probable Cost Opinion - Recommendations for Further Investigation

No.	Description	Unit	Quantity	Unit Price	Total Direct Costs	Mobilization/ Demobilization	Contractor's Contingency	Construction Fee	General Conditions	Tax, Insurance & Bond	Total Indirect Costs	Probable Construction Cost (Rounded)	Owner's Contingency	Total Probable Construction Costs (Rounded)
						3.00%	5.00%	7.00%	10.00%	11.00%			10.00%	
Roof Replacement Options														
1	Reservoir Roof Replacement - Aluminum Roof, Clear Span Dome	LS	1	$210,000	$210,000	$6,300	$10,500	$14,700	$21,000	$23,100	$75,600	$286,000	$28,600	$315,000
2	Reservoir Roof Replacement - Aluminum Roof, Column Supported Dome	LS	1	$260,000	$260,000	$7,800	$13,000	$18,200	$26,000	$28,600	$93,600	$354,000	$35,400	$390,000
3	Reservoir Roof Replacement - Flat Sla	LS	1	$500,000	$500,000	$15,000	$25,000	$35,000	$50,000	$55,000	$180,000	$680,000	$68,000	$748,000

NOTE - Reservoir replacement is not recommended until further detailed condition assessment of the reservoir interior is completed.

Vista Irrigation District
Deodar Reservoir Probable Cost Opinion - Reservoir Replacement

No.	Description	Unit	Quantity	Unit Price	Subtotal
1	**Reservoir Replacement -** Based on total capacity and includes: planning, engineering design, environmental, legal, construction, limited site work, piping upgrades, valve replacements, re-painting, and coating, construction management and contract administration. Reservoir unit price includes a 25 percent factor for costs associated with demolition and removal.	GAL	1,300,000	$1.50	$1,950,000
	Construction Contingency			**30%**	$585,000
	Total Direct Costs (Rounded)				**$2,535,000**

Appendix C. Exhibits

Exhibit A. Pipe Diameter and Material Wall Map

Exhibit B. Pipe Age Wall Map

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Vista Irrigation District Distribution System Pipe Materials and Diameter

VID Potable Water Master Plan
October 2016

Vista Irrigation District Distribution System Pipe Install Year

VID Potable Water Master Plan
October 2016

Appendix D. Hydraulic Model Software Selection Technical Memorandum

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Hydraulic Model Software Selection Technical Memorandum

2016 Water Master Plan Update

Vista Irrigation District

Vista, CA

January 11, 2017

1.0 Introduction

Vista Irrigation District (District) is in the process of updating its 2000 Potable Water Master Plan and hydraulic model. The hydraulic model was developed using the Innovyze H2ONET software platform. As part of the 2016 Master Plan Update, the District is considering upgrading the hydraulic model software platform to increase its usefulness to the District's staff and consultants. This Technical Memorandum (TM) presents the capabilities and constraints of the current modeling software and identifies the cost and benefits for upgrading the modeling software.

1.1 Current Hydraulic Model

The District's current potable water hydraulic model was converted to H2ONET V3 during the District's 2000 Master Plan effort. Pipeline data (alignment, length, diameter and roughness coefficient), node data (average day demands and elevations), reservoir dimensional data and valve data (location, type and size) were included in the model. Table 1 lists the facilities currently included in the District's H2ONET model.

Table 1. District's H2ONET Hydraulic Model Components

Facility Type	Count	Note
Pipe Segments	3,010	Represents 342 miles of pipe
Valves	74	Includes 17 combination valves that are modeled using a pressure reducing valve and a pressure sustaining valve operating in parallel
Junctions	2,261	1,513 of which are demand junctions
Tanks	10	
Supply Points/ Interconnections (Reservoirs)	15	
Pumps	8	Includes duty and standby pumps located at 3 pump stations

The District operates 17 combination valves that have the ability to switch between pressure reducing and pressure sustaining modes by throttling the flow to achieve the desired minimum pressure settings upstream and maximum pressure downstream. Since valves in H2ONET are either reducing or sustaining and not both, two separate valves were modeled in parallel to represent the combination regulator. Logic controls are used to determine which valve should be active depending on the pressures upstream and downstream of the valve.

Because some of the pressure zones are large and are fed at numerous points, there are a number of combination valves that operate in parallel to serve the same pressure zone. With numerous combination valves in the system, extended period simulations of the existing water system are performed using a 10 minute time step. Usually a time step of one hour is sufficient, but a shorter time step is required to minimize modeling instabilities when switching between the reducing and sustaining valve functionality. The rules used to operate the valves in the model vary with each scenario in order to make all of the combination valves in the model stable and the simulation to converge. Therefore, the rules must be confirmed and possibly modified when new scenarios are run, which can be time consuming. The approach to modeling combination valves is an important criterion for selecting a new hydraulic model platform. This topic is discussed further in Section 2.

1.2 Model Updates

The H2ONET model has been well maintained, but is not fully up to date with the District's GIS. District staff are considering converting this model to an all-pipe model that can be integrated with the District's GIS system and better representing operation of the combination valves. This TM explores the advantages and disadvantages of conversion to various alternative software platforms.

1.3 Overview of Alternative Software Platforms

Seven (7) alternative software platforms were considered for the District's potable water hydraulic model: EPANET (free software developed by US EPA), four (4) products by Innovyze: H2ONET (District's current software), H2OMAP Water, InfoWater, InfoWorks WS, and two (2) products by Bentley Systems: WaterCAD and WaterGEMS. The manufacturer's overviews of these platforms are provided below and a comparison of the cost and key features are included in Section 3.

EPANET is public domain software that may be freely copied and distributed. It is a Windows 95/98/NT/XP program. EPANET performs extended period simulation of the water movement and quality behavior within pressurized pipe networks. EPANET's Windows user interface provides a visual network editor that simplifies the process of building piping network models and editing their properties and data. EPANET provides an integrated computer environment for editing input data. Various data reporting and visualization tools are used to assist in interpreting the results of a network analysis.

H2ONET by Innovyze. Based on AutoCAD graphics, H2ONET Analyzer optimizes on-line data integration and bi-directional information exchange for complete network model creation and maintenance, eliminating time-consuming translations and ensuring data integrity and reliability. Using the same calculation engine as H_2OMap Water and InfoWater, H2ONET performs fast, reliable, and comprehensive hydraulic and dynamic water quality modeling, energy management, real-time simulation and control, fire flow analysis, and with automated on-line SCADA interface.

H2OMap Water by Innovyze. Built using advanced Object-Oriented Geospatial Component model, H2OMAP Water provides the most powerful and practical GIS platform for water utility solutions. As a stand-alone GIS-based program, H2OMAP Water combines spatial analysis tools and mapping functions with sophisticated and accurate network modeling for complete infrastructure (asset) management and business planning. It performs fast, reliable and comprehensive hydraulic and dynamic water quality modeling, energy management (with true variable speed pumping), real-time simulation and control with on-line SCADA interface, complete fire flow analysis, and unidirectional flushing.

InfoWater by Innovyze is a fully GIS integrated water distribution modeling and management software application. Built atop ArcGIS™ using the latest Microsoft .NET and ESRI ArcObjects component technologies, InfoWater seamlessly integrates advanced water network modeling and optimization functionality with the latest generation of ArcGIS. InfoWater capitalizes on the intelligence and versatility of the geodatabase architecture to deliver unparalleled levels of geospatial analysis, infrastructure management and business planning. Its unique interoperable geospatial framework enables world-record performance, scalability, reliability, functionality and flexibility - all within the ArcGIS environment.

InfoWorks WS by Innovyze uses an enhanced version of the WesNet engine, world renowned for its speed with large networks and ability to cope with 'difficult' networks. A full range of simulation capabilities is included as standard with InfoWorks WS, including water quality and sediment modeling, fire flow modeling, critical link analysis, unidirectional flushing, demand area and leakage analysis, energy use and cost calculations, auto calibration of networks and carbon footprint analysis. In addition, User Programmable Control (UPC) allows the user to change the state of control elements based on the status of sensors, in order to optimize operating regimes within a network.

WaterCAD by Bentley is a subset of WaterGEMS. WaterCAD helps you improve design productivity, with:

- Streamlined model building: Leverage and import virtually any external data format to jumpstart the model accurately, easily allocate water demands, and automate terrain extraction and node allocation.
- Organized assessment of alternatives: Assess and compare an unlimited number of physical, design, water demand, network topology, and operational scenarios.
- CAD interoperability: Model in a familiar platform, leveraging CAD tools and shortcuts when using WaterCAD from within MicroStation or AutoCAD. You can also choose to use WaterCAD as a stand-alone application, for additional flexibility.

WaterGEMS by Bentley runs your model from within ArcGIS and for optimization modules (calibration, design, pump scheduling, pipe assessment, SCADA integration, and network simplification). WaterGEMS provides numerous software tools for:

- Intelligent planning for system reliability: The capability of the water network to adequately serve its customers must be evaluated whenever system growth is anticipated. With WaterGEMS, effectively identify potential problem areas, accommodate service area growth, and plan capital improvements.
- Optimized operations for system efficiency: Realistically modeling the operation of complex water systems can be difficult. With WaterGEMS, model pump accurately,

optimize pumping strategies, and plan shutdowns and routine operations to minimize disruption.

- Reliable asset renewal decision support for system sustainability: WaterGEMS tools such as Pipe Renewal Planner analyzes and compares a wide range of variables to prioritize renewal decisions.

2.0 Modeling Combination Valves

The District's potable water distribution system serves 12 pressure zones with hydraulic grades cascading from a high of 984 ft above sea level (a.s.l.) at the eastern end of the District, to a low of 484 ft a.s.l. at the western end of the District. The hydraulic grade line (HGL) in these pressure zones are maintained using combination valves that have both pressure reducing and pressure sustaining features. This type of valve can make operations challenging, particularly if multiple combination valves are implemented in parallel, but are necessary to control flow rates from higher to lower zones to prevent "robbing" supply from the upper zone. Representing combination valve operations in the hydraulic model also presents significant challenges.

2.1 Combination Valve Operations

Combination valves work as a combination of a pressure sustaining valve (PSV) and a pressure reducing valve (PRV). Combination valves strive to maintain both the minimum upstream pressure above the prescribed upstream setting (PSV function) and the maximum downstream pressure below the prescribed downstream setting (PRV function). During typical combination valve operation, one of the two valves is always fully open and the other valve controls the flow. Since the transition between PSV operation and PRV operation is not instantaneous, there can be transitional periods where both valves are operating in tandem to adjust the opening.

In a system with numerous combination valves that connect the same pressure zones, as is the case in District's system, the operation of combination valves can become unstable during transitional periods. If these parallel combination valves have similar or overlapping settings, transition periods of the valves might coincide. In this situation the operation of valves might become unstable. In the physical world, built-in controls with time delays are used to stabilize combination valve operation. In a hydraulic model, rules based controls need to be used to achieve model stability.

2.2 Modeling Representation of Combination Valves

In most hydraulic models, combination valves are typically represented as a set of one PSV and one PRV placed in series with a short pipe in-between (see Figure 1). Some modeling platforms, such as InfoWorks WS, encapsulate this implementation into a single valve.

Valve settings are used to make sure that the valve operates as expected and that the model is stable. If multiple combination valves operate in parallel, appropriate valve settings are needed to ensure the model stability by avoiding overlapping transition periods. Finally, if the settings of parallel combination valves are overlapping in the system, i.e. the settings of different valves are close enough to allow the switch between PSV and PRV operation at the same time step, rule-based controls are needed to maintain model stability. Alternatively, the model calculation time step could to be reduced to avoid simultaneous combination valve transition.

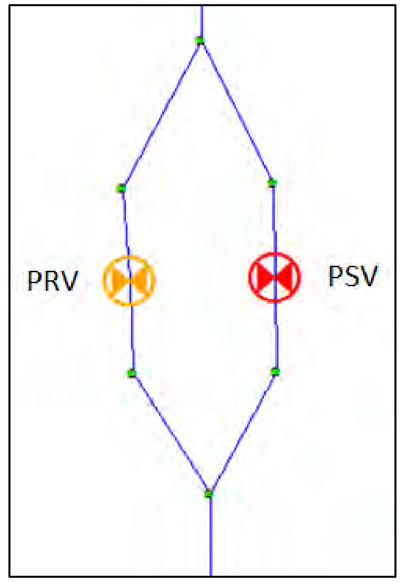

Figure 1 - Combination Valve Modeled with Valves in Series

The District's current H2ONET model uses a non-standard implementation of the combination valves with PRV and PSV placed in parallel for each combination valve (see Figure 2). This approach requires rule-based controls to operate a single combination valve.

With the addition of parallel combination valves with overlapping settings, these rule-based controls become increasingly more complex leading to a model that is less stable. In order to manage simultaneous transition of multiple combination valves in the same time step, the model needs to iterate between different PRV and PSV settings on all valves with the same time step. Since rule-based controls are executed only once during a time step, they cannot guarantee the setting convergence.

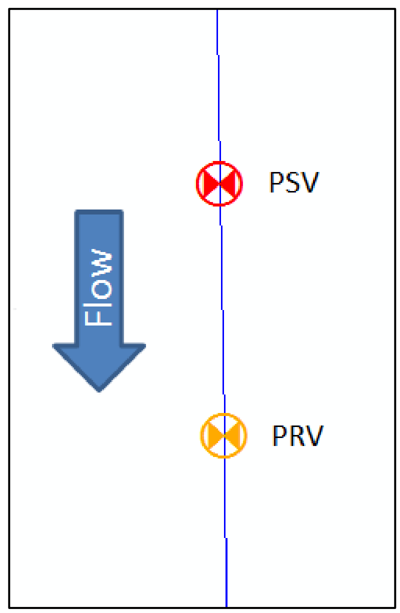

Figure 2 - Combination Valve Modeled with Valves in Parallel

HDR recommends that combination valves in the District's model be converted to the standard implementation with PSV and PRV in series, as shown in Figure 1. Valve settings should be adjusted to avoid simultaneous transition. This should bring the majority of parallel combination valves within convergence. The remaining valves might still need rule-based controls but this should be used only if no other option is available.

HDR has conducted preliminary tests of standard combination valve implementation in the InfoWater platform, which uses the same software engine as the H2ONET platform. The testing has demonstrated that at least some rule-based controls can be removed from the model, making the model more stable. While working for Wallingford Software (now Innovyze), HDR's National Hydraulic modeling expert, Sasa Tomic, wrote the specifications for the combination valves in InfoWorks WS. As such we have confidence that we understand how to model these combination valves in the alternative software platforms.

3.0 Alternative Software Comparison

Seven (7) hydraulic modeling software platforms were selected for comparison. Those major water distribution modeling products include free-ware EPANET by U.S. Environmental Protection Agency (EPA); four (4) different products by Innovyze (H2OMAP Water, InfoWater, InfoWorks WS and the District's current platform, H2ONET) and two (2) products by Bentley Systems (WaterCAD and WaterGEMS). Of the seven (7) examined products, all but InfoWorks WS use some version of EPANET engine as the hydraulic solver. Thus, they should all perform similarly in terms of combination valve modeling.

Based on communication with the District, HDR has identified the following criteria to be used for model comparison: integration with GIS, key modeling features, including those specifically requested by District staff, and modeling software costs. Note that because the District currently uses an Innovyze product, the Innovyze representative has stated that a discount is available when upgrading to the H20Map or InfoWater products, but not for the InfoWorks product.

Comparison results for key modeling features are presented in Table 2. Product features that are highlighted red have below-average implementation and are less favorable for the District. Product features that are highlighted blue have above-average implementation and better fulfill the District's needs.

3.1 Model Integration with GIS

The district has identified GIS integration as one of the determining factors in the software selection. GIS functionality examined includes the ability to:

- create and update models from GIS data
- operate the model inside of ESRI ArcGIS (GUI Integration)
- share model results with GIS database and ArcGIS
- convert selected modeling software to a ArcGIS-integrated platform in the future
- maintain one-to-one (1:1) relationship between GIS and model databases

3.1.1 Model Build and Update from GIS Data

Only two products do not offer full ability to build and update models from GIS: EPANET and WaterCAD. Both WaterCAD and EPANET provide shapefile data import but cannot exchange data with ArcGIS geodatabases. WaterCAD GIS-integration functionality is better than EPANET with model-build wizards and the ability to create automated data import procedures. However, the need to export data from GIS to shapefile before it can be imported into WaterCAD is considered to be a major deficiency.

3.1.2 Graphical User Interface (GUI) Integration with ArcGIS

Only two products can run inside ArcGIS platform: InfoWater and WaterGEMS. WaterGEMS can also run inside Autodesk AutoCAD and Bentley Systems MicroStation.

Table 2. Water Distribution Modeling Software Comparison

Evaluation Criteria	US EPA	Innovyze				Bentley Systems	
	EPANET	H_2ONET	H_2OMap Water	InfoWater	InfoWorks WS	WaterCAD	Water GEMS
GIS INTEGRATION							
Model Build from GIS	Limited	Limited	Full	Full	Full	Limited	Full
ArcGIS GUI Integration	None	None	None	Full	None	None	Full
Share Result with GIS	Very Limited	Limited	Limited	Full	Full	Limited	Full
Upgrade to GIS	Full	Full	Full	Full	Limited	Full	Full
1:1 Correlation	Full	Full	Full	Full	Full	Full	Full
DISTRICT REQUESTED FEATURES							
Combination Valve Modeling	Standard	Standard	Standard	Standard	Integrated	Standard	Standard
Risk Assessment	None	None	Limited	Limited	Full	Limited	Limited
SCADA	None	None	SCADAWatch	SCADAWatch	InfoWorks Live	SCADA Connect	SCADA Connect
Automated Demand Adjustment	None	None	SCADAWatch	SCADAWatch	InfoWorks Live	SCADA Connect	SCADA Connect
ADDITIONAL FEATURES							
Automatic Conversion from H2ONET	Yes	N/A	Yes	Yes	No	No	No
Ease of Use	Average	Average	Easy	Easy	Complex	Easy	Easy
Model Management	None	Scenario with inheritance and Data sets with data duplication	Scenario with inheritance and Data sets with data duplication	Scenario with inheritance and Data sets with data duplication	Scenario/Run, Alternative and Versioning support	Scenario and Alternative support	Scenario and Alternative support
Automated Network Zoning	None	None	None	InfoWater PZM Separate product	Included	Included	Included
Intermediate Time Steps	Included	Calculated but not reported	Calculated but not reported	Calculated but not reported	Included	Included	Included
COST							
Initial Cost (up to 5,000 links)	$0	$0	$3,000* (4000 links)	$3,000* (4000 links)	$15,000	$10,000	$15,000
Annual Maintenance Costs for 5,000 links	$0 (unsupported)	$1,000	$1,000	$1,000	$2,500	$2,400	$3,600
Add Ons				Suite $1,000 Executive Suite $2,000			
Cost of Conversion from H2ONET	Minimal	N/A	Minimal	Minimal	Moderate	Moderate	Moderate
Automated Network Zoning	None	None	None	InfoWater PZM Included in Executive Suite	Included	Included	Included
SCADA	None	None	SCADAWatch $23,000, plus $15,000 training			None	SCADA Connect Included
Basic Training Cost	Not available	1 day training - $1,600/person & $800 for each additional person (at their facility - Pasadena, CA) OR custom, on-site training for 2-days, up to 8 people for $10K				2 day training - $495/person (at their facility - Waterbury, CT)	

* Upgrade Cost = $2,000 Upgrade Fee + $1,000 software cost difference

3.1.3 Share Results with GIS

InfoWater and WaterGEMS can run inside of ArcGIS platform and offer full integration of model results with ArcGIS. InfoWorks WS can import GIS files as background data which allows the user to cross-query GIS and model data and results. Other products can display some GIS data as background files with limited ability to cross-query the model and GIS.

3.1.4 Upgrade to GIS Based Software

InfoWater can convert H2OMAP and H2ONET Water files and WaterGEMS can covert WaterCAD files. EPANET files can be imported in both WaterGEMS and InfoWater. Only InfoWorks WS files cannot be directly imported in a GIS-enabled product.

3.1.5 One to One (1:1) Integration

All products are capable of running all-main models with 1:1 correlation to GIS database. WaterCAD and WaterGEMS offer additional features for GIS ID management that allow multiple IDs to be shared for the same element, so the model can be developed using different data sources.

3.1.6 Conclusion

InfoWater and WaterGEMS stand out as the products with the most GIS integration, closely followed by InfoWorks WS. H2OMAP Water and WaterCAD offer the basic GIS integration without the ability to work with GIS and model data at the same time.

3.2 Additional Modeling Features

The District has identified several key modeling features to be examined in modeling software comparison:

- Combination valve modeling
- Pipe criticality analysis for the risk assessment
- SCADA integration
- Automated demand adjustment

Several additional features were selected by HDR as they were deemed beneficial to the District:

- Automatic Conversion from H2ONET and Ease of Use
- Model data management
- Automated Network Zoning tool
- Intermediate time step management

3.2.1 Combination Valve Modeling

All products allow modeling combination valves as a pressure sustaining valve (PSV) and pressure reducing valve (PRV) in series with a short pipe in-between, as discussed in Section 2. InfoWorks WS encapsulates this implementation into a single object that simplifies the valve representation and offers some flexibility in maintaining model stability. This does not however guarantee that the District's 17 combination valves will be seamlessly integrated into the InfoWorks WS model without additional controls needed to attain model stability.

3.2.2 Pipe Criticality Analysis for Risk Assessment

While all products, with the exception of EPANET and H2OMAP Water, offer some type of valve or pipe criticality assessment, only InfoWorks WS offers a complete pipe criticality assessment tool. However, unless the District plans to run pipe criticality analysis regularly, it might not be necessary to convert the model to InfoWorks WS only for this feature.

3.2.3 SCADA Integration and Automated Demand Adjustment

All products, with the exception of EPANET and H2ONET, have companion software that provides SCADA integration including boundary condition setting, initial state setting, demand adjustment, automated model runs and other real-time modeling features. In addition to the need to acquire additional software to enable this functionality, significant effort needs to be invested in the establishment and maintenance of SCADA links and the development of real-time modeling processes. SCADAWatch is the add-on program for H2OMAP Water, InfoWater and InfoWorks. InfoWorks Live is another add-on program that integrates SCADA with InfoWorks WS, but is quite complex and generally used only by very large utilities. SCADAConnect is the add-on program for the Bentley Systems products, WaterCAD and WaterGEMS.

3.2.4 Automatic Conversion from H2ONET and Ease of Use

While all products are relatively easy-to-use, EPANET, H2ONET and H2OMAP Water have somewhat outdated interfaces. InfoWorks WS is a powerful tool and with the additional power comes extra complexity that detracts from the ease of use. Because of the District's familiarly with H2ONET and ArcGIS, it is expected that the District would find H2OMAP Water and InfoWater somewhat easier to use than a new product.

3.2.5 Model Data Management

Only EPANET does not offer the ability to manage multiple scenarios and alternatives in a single model database. InfoWorks WS, WaterCAD and WaterGEMS offer these features without data duplication which can prevent some modeling errors.

3.2.6 Automated Network Zoning

Because of District's complex network with multiple pressure zones, having a tool that can easily delineate hydraulically separated zones might be an important feature for model analysis. All products except EPANET have such tool. InfoWorks WS, WaterCAD and WaterGEMS include the pressure-zone delineation tool in the core product while InfoWater, H2ONET and H2OMAP Water require that such tool be purchased separately. HDR will use available automated network zoning tools during the course of the 2016 Water Master Plan Update. Once the model update is completed, the District staff would only need the automated network zoning tool if they wanted to experiment with further adjusting the system's pressure zone boundaries.

3.2.7 Intermediate Time Steps

Because of the issues with combination valves, it might be advantageous for the District to see the results at every calculated time step, and not only at the scheduled time steps. While all products will insert a time step whenever it is needed to activate a control, InfoWater, H2ONET and H2OMAP Water do not report the results for such time steps.

3.3 Modeling Software Cost

Software cost is often one of the key decision components when selecting a new modeling platform. Software costs include the initial product cost, cost of the annual software maintenance and technical support and the training cost. Cost comparison results for the examined products are presented in Table 2, Water Distribution Modeling Software Package Comparisons. With exception of the free-ware EPANET, software products with similar functionality have similar prices if purchasing a new product. However, because the District currently uses an Innovyze product, a discount is available when upgrading to the H20MAP or InfoWater products. This discount is not available for the Innovyze InfoWorks WS product.

The District's is current on the annual maintenance of its H2ONET license through January 14, 2017, which makes the District eligible for upgrade advantages. There is a one time upgrade fee of $2,000 to go from H2ONET to H2OMAP Water (stand alone) or InfoWater (GIS platform). The District receives full credit for their existing license to apply to the upgrade, which was $7,000 for the 4,000 link H2ONET model. H2OMAP Water and InfoWater Basic cost $8,000, so VID would pay the $1,000 difference in price between H2OMAP/InfoWater and H2ONET. Annual maintenance costs will increase from $800 to $1,000 per year. As the cost to upgrade to either H2OMAP Water or InfoWater is the same, Innovyze recommends the InfoWater platform which has more features that meet the District's needs.

InfoWater offers upgrades from the Basic to Suite and Executive Suite for an additional one time cost of $1,000 and $2,000, respectively and a corresponding increase in annual maintenance fees of $500 and $1,000, respectively. The Suite Extension includes Calibrator, Demand Allocator, Designer, Protector, Scheduler, Skeletonizer, Valve Criticality Modeling, WQ Calibrator features and the Executive Suite Extension includes those features as well as NetVIEW, Leakage Detection Manager, Pressure Zone Manager (PZM), Sensor Location Manager, Sustainability features. These InfoWater upgrades can be purchased anytime at cost, without an additional upgrade fee.

While InfoWorks WS does provide additional features that may be useful to the District, it is significantly more costly. The District must buy a full license at a cost of $15,000 (no upgrade credit provided) and the maintenance support cost is $2,500 per year.

WaterCAD costs $8,000 to $10,000 with the respective annual maintenance support between $1,000 and $2,400. WaterGEMS cost $15,000 with a corresponding annual maintenance cost of $2,500 and $3,600. These products include additional features that may or may not prove useful to the District.

Additional soft costs include the labor requirements to convert the current model to the new modeling platform. The conversion from H2ONET to InfoWater is less labor intensive than the other software products, as it shares the Innovyze platform. Since the District has already invested in Innovyze products and is familiar with their interface, InfoWater stands out as a reasonably priced, user friendly product with the features that will be useful to the District.

4.0 Recommendations

InfoWater provides the key features requested by the District at the lower cost due to Innovyze upgrade pricing, and has a user interface similar to what the District is used to, eliminating the need for extensive training. Features that are not available in InfoWater, such as detailed pipe critically assessment and intermediate time-step reporting, are not deemed of sufficient value to justify the cost of the conversion to a different product, particularly because the District is pursuing condition risk assessment separately from the capacity analysis.

At a significantly higher cost, InfoWorks potentially provides a one step solution to modeling the combination valve, although some additional controls may be necessary to stabilize the District's model due to all the combination valves that operate in parallel. Conversion from H2ONET to InfoWorks would also be more time intensive, as model conversion would need to be conducted one scenario at a time. Also, the District would not be able to easily integrate the model with GIS interface in the future if InfoWorks WS is selected.

Ideally, the combination valve modeling issue can be resolved using the lower cost InfoWater platform. HDR is in the process of reconfiguring all combination valves to a PSV and a PRV in series to test the stability of the model in the Innovyze calculation engine shared by H2ONET, H2OMAP Water, and InfoWater. For testing, the InfoWater platform was selected for data conversion from H2ONET because it is a direct conversion.

The reconfiguration of modeled combination valves is time consuming. To date, HDR has conducted the process for a couple of pressure zones, with a model re-run after each combination valve is reconfigured from an in-parallel to an in-series configuration. This approach helps identify the valves that have overlapping settings. The settings for such valves are then adjusted to maintain model stability. To date, we have had success in eliminating rule based controls, but conversion of the entire model is needed to finalize the results. Rule-based controls will be considered only if the model is not able to satisfactory match the system control valve settings.

Since the need for the rule-based controls for the combination valves may be manifested in all products, we recommend proceeding with the full model conversion to InfoWater. Based on our efforts to date, we believe that conversion of the combination valves in InfoWater will be attainable with minimal rule-based controls. If the combination valve adjustments are fully implemented in InfoWater and the model cannot be stabilized, we will reevaluate whether the issue of the combination valves needs to be included in the District's final software selection. As we noted earlier, there is also no guarantee that a conversion to InfoWorks will eliminate all rule-based controls, so completing the conversion in InfoWater at this time seems to be the most cost efficient approach.

Appendix E. Hydraulic Model Geographic Information System Integration Technical Memorandum

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Hydraulic Model GIS Integration Technical Memorandum

2017 Water Master Plan Update

Vista Irrigation District

Vista, CA

April 3, 2017

1.0 Introduction

Vista Irrigation District (District) is in the process of updating its 2000 Potable Water Master Plan and hydraulic models. The District currently maintains three hydraulic models representing the primary Vista service area and the two smaller Boot and Bennett service areas. The hydraulic models were developed using the Innovyze H2ONET software platform and currently represent the District's distribution systems as skeletonized models. The District has decided to convert the H2ONET models to the Innovyze InfoWater modeling platform and combine them into a single, District-wide model. As part of the 2017 Master Plan Update, the combined hydraulic model will be updated with the latest geographic information system (GIS) information from the District's geodatabase (gdb). As part of this process, the District is considering establishing a one-to-one model pipe-to-GIS pipe relationship. This Technical Memorandum (TM) presents the costs and benefits of establishing such a relationship between the hydraulic model and the District's gdb.

2.0 Establishing a Model-to-GIS Relationship

Updating the model with the gdb to establish a one-to-one relationship with the gdb can be done using the InfoWater Import Manager. The District's current sewer collection system gdb is very detailed and has excellent topology, so establishing a one-to-one relationship between the gdb and the model is relatively straightforward. However, InfoWater license prices are set by model pipe count. The most affordable InfoWater license limits model scenarios to 4,000 pipes. This base license can be used to run the District's current distribution system models, even if the Boot and Bennet models are imported to the primary model. The H2ONET models were imported directly to InfoWater so the pipe counts remain consistent for the two software platforms. Pipe totals for the three H2ONET models consist of 3,309 pipes total including:

- Main system: 3,100 pipes
- Boot: 24 pipes
- Bennet: 185 pipes

On the other hand, the District's current gdb has a pipe count of approximately 18,215 pipes for VID's primary distribution system (not including laterals and the flume). The pipe count in the current hydraulic models reflects shorter, contiguous pipes in the gdb as one combined longer pipe if the pipes share the same material and diameter. This allows the current model to correctly represent the hydraulics of the distribution system with fewer pipes. Additionally, laterals, the flume, and relatively short dead-end pipes are not included in the model. If all gdb pipes were included in the hydraulic model, VID would be required to purchase an unlimited pipe count InfoWater license in order to run the model, which would result in an $8,500 cost increase compared with the 4,000 pipe license as shown in Table 1 below.

Table 1. Costs for Upgrading to InfoWater from H2ONET License

InfoWater License No. of Links	Upgrade Fee	Difference in Software Cost	Difference in Maintenance Cost	Total
4,000	$2,000	$0	$0	$2,000
Unlimited	$2,000	$8,000	$500	$10,500
Cost Difference	$0	$8,000	$500	$8,500

In addition to the license cost benefit of maintaining a model with fewer pipes, there are other practical advantages to maintaining a <4,000 pipe model:

- Fewer pipes means less computation time and reduced chance for errors.
- Also, limiting the number of pipes in the model reduces the chance of incorrect information being loaded into the model and in general makes the model easier to manage.

Alternatively, the advantages of maintaining a one-to-one gdb-to-model relationship, and upgrading the InfoWater license to accommodate an unlimited no. of links, include:

- Easily locating assets in the model and comparing the assets with the current gdb
- Updating the model directly from the gdb in order to keep the model current

InfoWater modeling platform can provide similar functionality for models with or without a one-to-one GIS-to-model relationship. Because InfoWater runs within the ArcMap GIS platform, the gdb and the model can be displayed simultaneously and the gdb can compared with the model spatially. In order to investigate or update a particular asset in the model, the user would simply query the asset in the gdb and zoom to the asset in ArcMap. Then the modeled representation of the asset can be selected spatially and the model information can be compared or updated based on the gdb. New distribution system infrastructure can be imported directly into the model from the gdb using the InfoWater Import Manager. However, when trying to maintain a 4,000 pipe count model for licensing purposes, short, contiguous pipes may need to be combined as longer pipes, and laterals and short length dead-end pipes may need to be excluded in order to keep pipe count down.

3.0 Costs, Benefits, and Recommendation

In conclusion, maintaining a one-to-one gdb-to-model relationship has the advantage of locating assets within the model and updating the model with new gdb information relatively easily. However, the license cost of running an all pipe model is higher, and InfoWater provides the advantage of being able to compare the gdb and the model within the ArcMap platform. Additionally, models with fewer pipes are able to correctly represent the hydraulics of a distribution system, are quicker to run, and are easier to manage. A summary of the costs and benefits to one-to-one GIS-to-model relationship is as follows:

Costs:

- More expensive license
- Takes model longer to run
- More chances for errors in the model
- Harder to manage
- InfoWater provides similar functionality for a <4,000 pipe model

Benefits:

- VID geodatabase ideal for importing to the model
- Update of GIS infrastructure information in the model using InfoWater Import Manager
- Quick look up of GIS infrastructure information in the model

The final recommendation is that the District maintain a skeletonized hydraulic model and forgo the one-to-one model-to-GIS relationship. A skeletonized model would be more cost effective, less prone to error, easier to manage, and have similar functionality as a model with a one-to-one model-to-GIS relationship.

Appendix F. Hydraulic Model Validation Technical Memorandum

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Hydraulic Model Development & Validation Technical Memorandum

2017 Water Master Plan Update

Vista Irrigation District

Vista, CA

April 3, 2017

1.0 Introduction

This technical memorandum (TM) describes the water system computer model development, including verification with field data that was performed as part of the Vista Irrigation District (District) 2017 Water Master Plan Update (2017 Master Plan).

2.0 Existing System Model Development

The computational hydraulic model of the District's distribution system has passed through three main phases in its historical development. In its original phase, the hydraulic model was constructed by the District using Cybernet. This original model included information relevant to the distribution system at the time, including pipeline data (alignment, length, diameter, and roughness coefficient), node data (average day demands and elevations), reservoir dimensional data, and valve data (location, type, and size).

In the second phase of model development, as part of the 2000 mast plan effort, the model was converted from Cybernet to H2ONET Version 3 by Innovyze. As part of this process, the model was also updated to represent the District's distribution system at the time. In addition, the model was verified using a combination of field and SCADA data collected over a 24 our period from November 9-10, 1999. As part of the model update, special attention was given to the modeling of pressure reducing/sustaining combination regulators (combination regulators), which were not offered as a standard control valve option in H2ONET Version 3. Modeling of the combination regulators is discussed below.

In the third phase of model development is the 2017 Master Plan. Between the 2000 master plan effort and the 2017 Master Plan, the H2ONET version of the hydraulic model was maintained and updated by the District. A summary of the steps taken to update the model for in the 2017 Master Plan is included below.

Model Conversion to InfoWater

As part of the 2017 Master Plan, the District's existing H2ONET model was converted to InfoWater Version 12.2 by Innovyze. InfoWater includes features that were more desirable to District staff than H2ONET, including the ability to run the modeling software in ArcMap. Prior to converting the model to InfoWater, a comparison of seven of the leading water distribution system modeling software packages was developed for the District's review, and it was determined that InfoWater was the best fit for the District's needs. H2ONET and InfoWater are both distributed by Innovyze, and InfoWater includes the ability to automatically import H2ONET models, so conversion between the two software packages required minimum effort. A summary of the model software selection process is included in Appendix C-1, Hydraulic Model Software Selection TM.

As part of the model conversion to InfoWater, the two smaller H2ONET models of the Boot and Bennett systems were also imported into the new InfoWater model.

Model to GIS Relationship

As part of the 2017 update, the District's hydraulic model was updated based on the latest geospatial information system (GIS) information in the District's geodatabase (gdb). As part of this process, the District explored the practicality of establishing a one-to-one model-to-GIS relationship where the modeled facilities could be linked to the gdb facilities via a unique identification number. A cost, benefit analysis was preformed and it was determined that the costs of establishing a one-to-one relationship with the GIS would outweigh the benefits. The existing hydraulic model is skeletonized and represents the distribution system with fewer primary distribution pipes than are included in the gdb. Increasing the number of pipes in the model could make the model unnecessarily complex leading to increased errors, longer run times, and resulting in a model that is more difficult to manage. Additionally, matching the model pipes with the GIS would require an upgraded license due to the large number of pipes in the gdb.

Maintaining a relationship between the model and the District's GIS information is a priority, even if the relationship is not a one-to-one facility relationship with the gdb. The selection of InfoWater as the software for the model conversion allows for a visual comparison of the relationship between the gdb and the model in ArcMap. In ArcMap, the gdb facilities can be overlaid on the modeled facilities allowing for a quick comparison.

A summary of the cost, benefit analysis of establishing a one-to-one relationship between the model and the District's GIS is included in Appendix C-2 of the 2017 Master Plan, Hydraulic Model GIS Integration TM.

Modeling Combination Regulators

In addition to converting the District's H2ONET model to InfoWater, the pressure reducing/sustaining combination regulators in the model were updated as part of the 2017 Master Plan.

At the time of the 2000 master plan effort, the District had 17 combination regulators, which have the ability to switch between pressure reducing and pressure sustaining modes by throttling the flow to achieve the desired pressure settings upstream and/or downstream. Since valves in H2ONET are either pressure reducing valves (PRVs) or pressure sustaining valves (PSVs) and not both, a PRV and a PSV were modeled in parallel to represent the combination regulator. Logic controls were used to open one and close the other, or vice versa, and then switch if necessary, depending on pressures upstream and downstream of the valve.

For the 2017 Master Plan, the model was updated to represent the combination regulators as a PSV and a PRV in series (up-gradient to down-gradient) with no logic controls. Trial model runs indicated that this configuration results in expected combination valve operation. In order to stabilize the InfoWater computational engine, small diameter check valves were included at each combination regulator running from the lower head pressure zone to the higher head pressure zone. The head of the respective pressure zones does not allow flow through these check valves, but the check valves maintain hydraulic continuity in the case that the model valves are closed; a situation that would normally result in disconnected nodes in the model, a critical error. All combination regulators in the converted InfoWater model were updated to this configuration. The result is a model that can be run and updated without the need for manually input control logic.

When reviewing options of modeling software for the model conversion, the ability of the software to properly represent combination regulators was a primary concern. A review of the ability of InfoWater to represent combination regulators is provided in Appendix C-1 of the 2017 Master Plan, Hydraulic Model Software Selection TM.

Operations

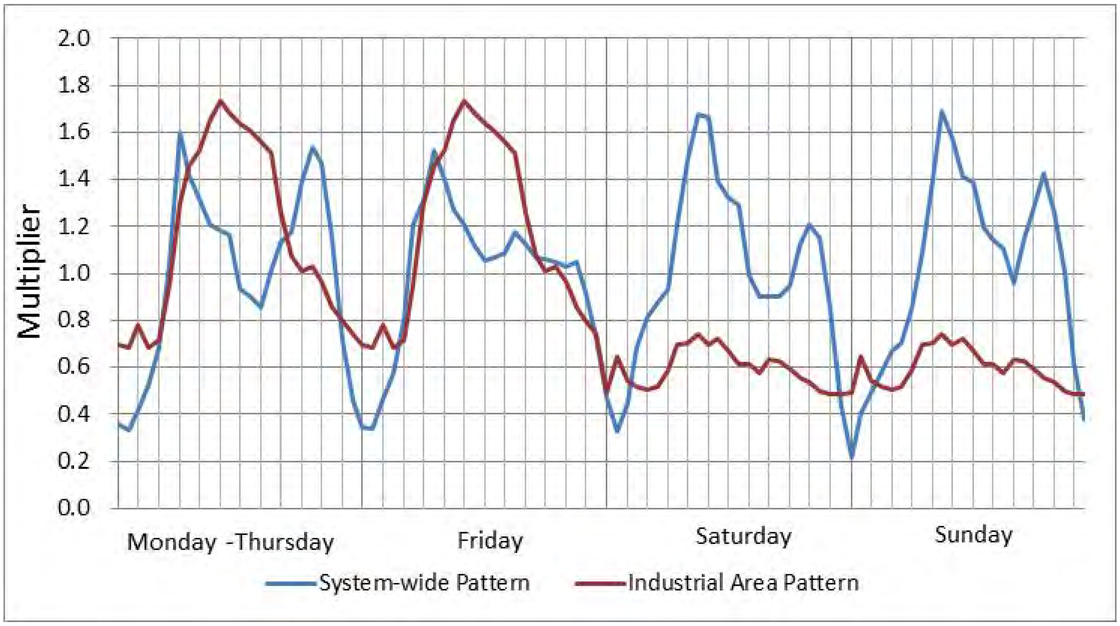
Operations information in the hydraulic model was updated based on information provided by District Operations staff including facility settings, SCADA data, and conceptual information about how the distribution system is operated.

As discussed in the previous section, the operational control information in the hydraulic model was updated as part of the combination regulator update. Regulator settings were updated in the model based on set points provided by Operations staff. Settings were included for manually adjusted regulators and regulators controlled by the SCADA system. Regulator settings included in the model are listed in Chapter 6 of the 2017 Master Plan.

SCADA data were used to calculate hourly system demands. A water balance spreadsheet was set up for the primary service area. The spreadsheet included boundary conditions, including inflows from the flume and SDCWA connections, and reservoir levels. The spreadsheet was populated with SCADA data, and hourly service area demand values were calculated based on total inflow and temporal changes in reservoir volume. The results were hourly demand values that could be used to develop a diurnal demand pattern for the system. The resulting diurnal pattern is shown in Figure 2-1.

In addition to the system-wide demand pattern, a separate industrial demand pattern was used for areas with industrial land uses. Analysis of wastewater generation patterns done as part of the City of Vista Sewer Master Plan indicate that industrial areas in the southern part of the primary service area experience water use patterns that are markedly different than the system-wide pattern. The industrial area sewer generation curve was normalized and used as the diurnal demand pattern for these industrial areas. The resulting demand pattern is shown in Figure 2-1.

Figure 2-1. Modeled Diurnal Patterns



System-wide Pattern Source: Calculated using Vista service area water balance based on SCADA data. Industrial Area Pattern Source: Based on City of Vista industrial sewer generation diurnal pattern.

In addition to facility settings and data, District Operations staff provided information on typical system operations for various situations.

Demands

Model demands were developed based on calendar year 2014 billing and supply information as discussed in Chapter 5 of the 2017 Master Plan. Billing data were provided as bimonthly water use volumes. Billing accounts were linked to a meter GIS layer from the District's gdb which provided the spatial location of each meter. Water supply data were also provided by the District. The water supply data sets included daily supply volumes from each of the SDCWA connections and total production from the Escondido Vista Water Treatment Plant.

Average day model demands were developed and allocated to the hydraulic model using a multistep process. First, the bimonthly water use volumes were totaled for the 2014 calendar year resulting in the total annual demand values for each meter, including the Boot and Bennett service areas. Second, total annual water supply to the District was calculated by adding the monthly District supply values for the 2014 calendar year. The supply information provided by the District accounts for transfers into and out of the District's service areas. Third, water loss was calculated for 2014 at 4% based on a comparison of the supply total and billing records total as discussed in Chapter 3. Fourth, the billing record demand values were increased by 4% to account for water loss and the resulting values were divided by 365 to calculate average day demand. Fourth, the resulting demand values for each meter were linked with the meter GIS layer and spatially allocated to the nearest model demand node using ArcMap. The final result is a model average day demand set based on 2014 billing data and adjusted for system water loss. The resulting average day demands included in the model are shown in Table 2-1.

Table 2-1. Modeled Average Day Demands per Service Area

Service Area	Average Day Demand (gpd)	Average Day Demand (gpm)
Vista	16,705,440	11,601
Boot	102,240	71
Bennett	492,480	342
Total	17,300,160	12,014

Source: 2014 billing data adjusted for 4% water loss spatially allocated by meter location

In addition to average day demands, the model demand sets were developed for maximum day, minimum day, and peak hour demands. The peaking factors for maximum day and minimum day were calculated based on water supply information as discussed in Chapter 3 of the 2017 Master Plan. Peaking factor and model scenario demand values are displayed in Table 2-2. Because the distribution of water use across the District varies throughout the year due to seasonal activities like irrigation, winter month billing records were used for minimum day demand calculations and summer month billing records were used for maximum day demand calculations. The resulting meter values were then adjusted based on the respective planning peaking factors and allocated to the model. The peak hour peaking factor was calculated based on a combination of the system-wide diurnal demand pattern derived from SCADA data and the maximum day peaking factor.

Table 2-2. Model Demands per Scenario

Model Scenario		Average Day	Maximum Day	Peak Hour	Minimum Day
Peaking Factor		1	2	3	0.5
Service Area Demands (gpm)	Vista	11,601	23,202	34,803	5,801
	Boot	71	142	213	36
	Bennett	342	684	1,026	171
	Total	12,014	24,028	36,042	6,007

3.0 Hydraulic Model Validation

Hydraulic model validation consisted of two main stages including macro level verification and micro level calibration. Macro level verification consisted of adjusting the model for demand distribution, diurnal patterns, water loss, and system operations. The goal of macro level verification is to demonstrate that the model represents system demands and behavior during extended period simulation in a qualitative comparison with SCADA data. Micro level calibration consisted of adjusting the model for pipe roughness factors and system response to hydrant tests. The goal of micro level calibration is for model results to replicate hydrant test field data for static and residual hydrant pressures in a quantitative comparison.

Model Verification

Model verification was conducted based on both summer and winter demand conditions. The summer model verification was based on detailed system operations data for August 2016 and comparing this data with model analysis results. The winter model verification was based on system operations data for January 2016. For each of these scenarios, hourly reservoir levels, aqueduct turnout flows to the District, and Vista Flume flows will be recorded during the calibration period in the District's SCADA system. Based on the hourly flow entering the distribution system at the aqueduct turnouts and Vista Flume and the fluctuating reservoir levels, a mass balance spreadsheet was developed to calculate the a 24 hour water demand curve for week day and weekend conditions. The resulting demand curves illustrate total system demands during the verification period.

In addition to water supply and tank level data, model verification will rely on available system pressure SCADA data, corresponding combination and altitude valve operations, pump SCADA data, and additional valve and system setting information provided by operations staff. Seasonal system operations settings were incorporated into the model for each of the validation scenarios.

For each of the verification scenarios, model verification was determined based on a qualitative comparison of tank level and pressure SCADA data with extended period model output. The goal was to demonstrate that the model represents the magnitude and diurnal patterns of system pressures and tank levels.

Tank levels from the extended period model simulation results were compared with available SCADA data in a qualitative comparative analysis. The model results comparisons for tank levels are displayed in Figure 3-1 though Figure 3-19. The model results indicate a good fit with the tank level SCADA data for each of the model verification scenarios.

Model Calibration

Micro level model calibration was based primarily on fire flow test data, corresponding SCADA data, and current system operations information. Fire flow testing was conducted over a two day period in April 2017 subsequent to the model verification per the VID Fire Flow Testing Plan. The field hydrant test report is included as Appendix D of the 2017 Master Plan. The testing consisted of 21 hydrant tests located throughout the District's pressure zones recording hydrant flows and hydrant static and residual pressures. Each test included a flow hydrant and two residual hydrants. System and pressure zone boundary conditions were recorded via SCADA when available and valve setting and operation were noted by field personnel during each hydrant test. Model calibration consisted of checking model pressures with the field data. The goal was to have model pressures match to within ten percent of field data. The goal is to meet the criteria for each of the hydrant tests, but because errors can occur during data collection, if the criteria are met for 90 percent of the hydrant tests was considered acceptable. In addition to the hydrant tests, pressure loggers were installed in the system per the VID Fire Flow Testing Plan. Data from the pressure loggers and corresponding SCADA data, including tank levels and system pressures, will also be used to characterize boundary conditions and system behavior at the time of the hydrant tests.

Model calibration results are displayed in Table 3-1. All of the model results matched the hydrant tests to within ten percent with regard to static and residual pressures. In the case of test 7 and test 20, model results indicated that particular valves may be closed in each of the testing areas. These valves are noted in Table 3-1. Based on the model results comparison with the field hydrant tests, the model was assumed calibrated. The combination of model verification at the macro level and hydrant test model calibration at the micro level demonstrates that the model represents the real world system for both extended period simulation modeling and fire flow modeling.

Figure 3-1. San Luis Rey Reservoir Summer Validation Model Output vs. SCADA Data

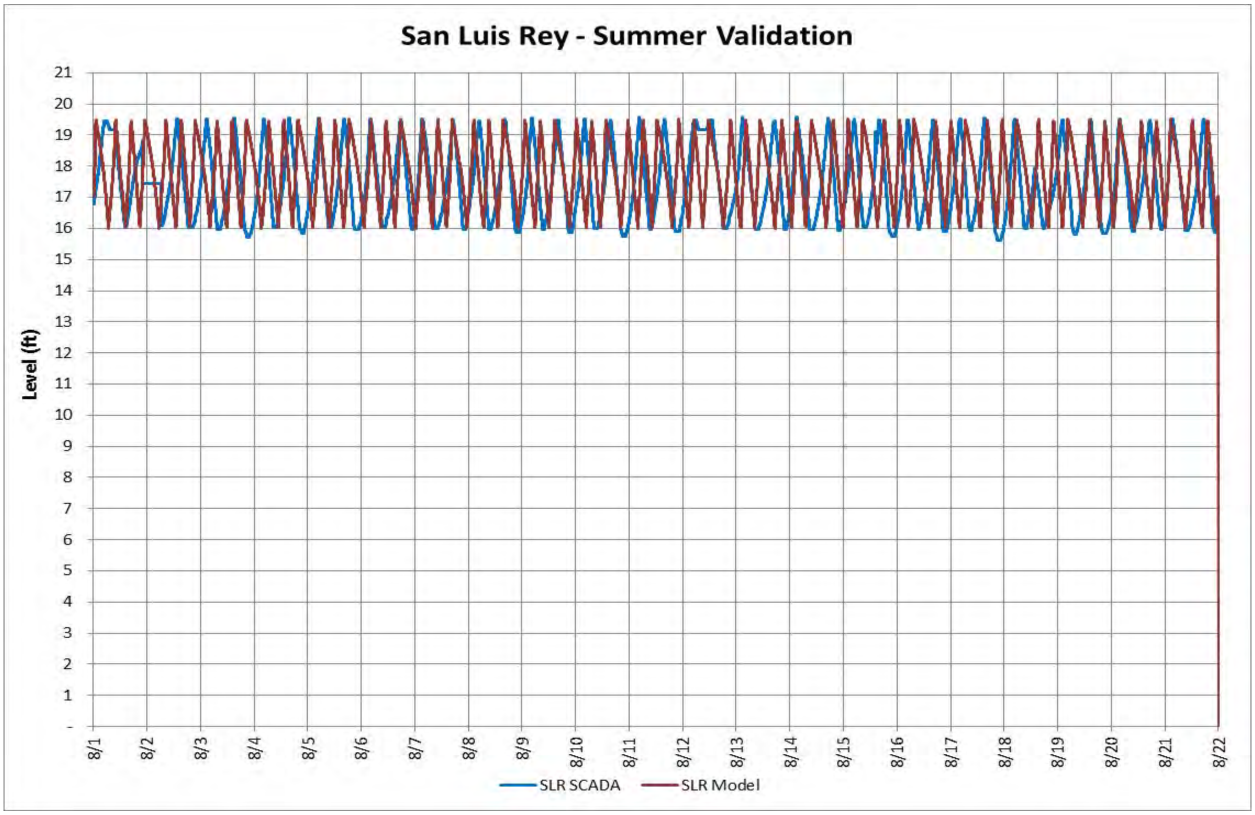


Figure 3-2. E-1 Reservoir Summer Validation Model Output vs. SCADA Data

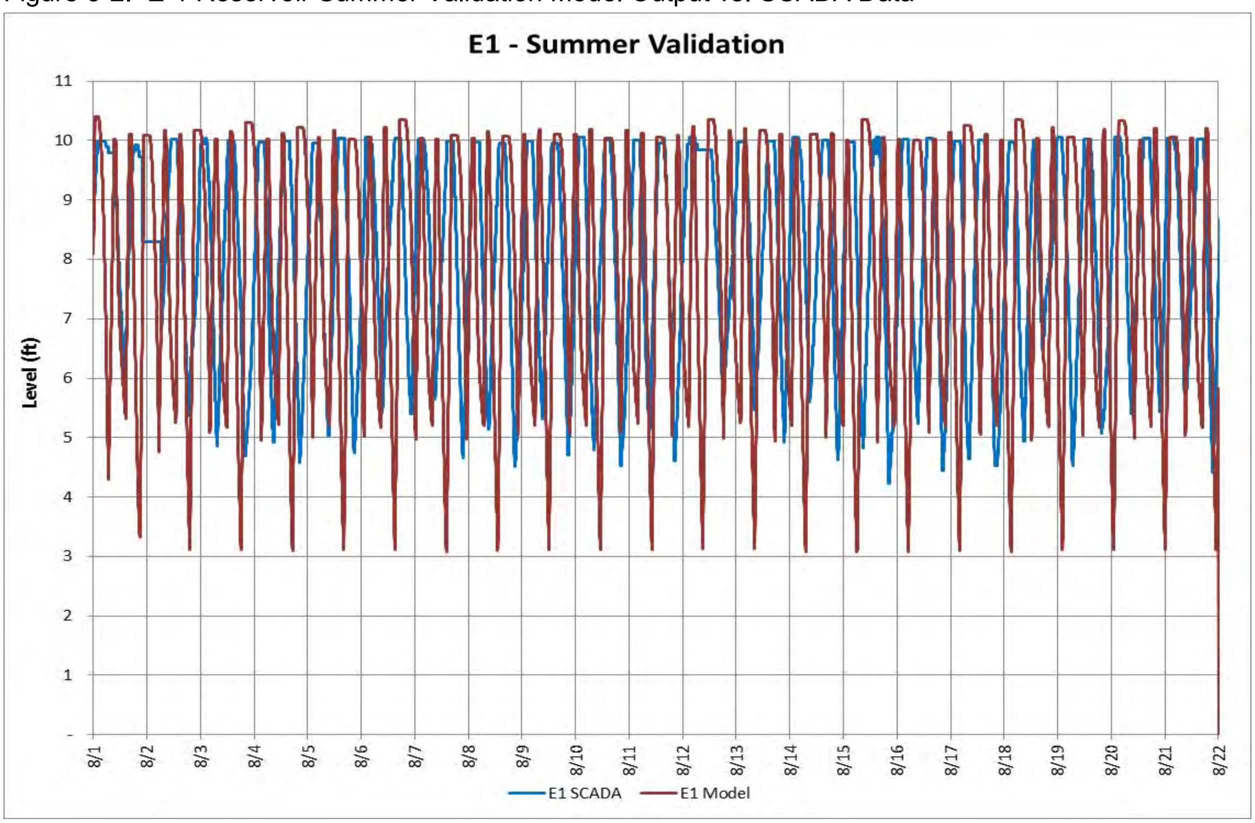


Figure 3-3. Lupine Hills Reservoir Summer Validation Model Output vs. SCADA Data

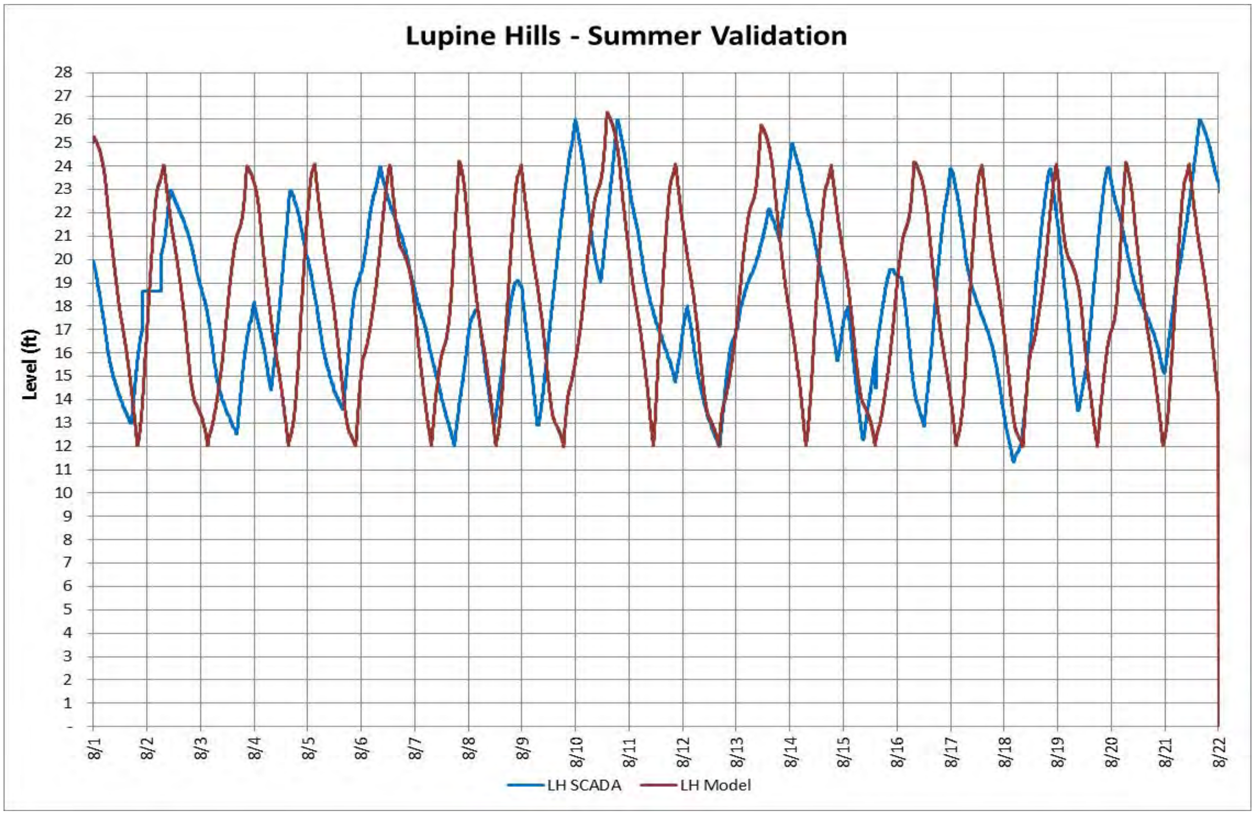


Figure 3-4. A Reservoir Summer Validation Model Output vs. SCADA Data

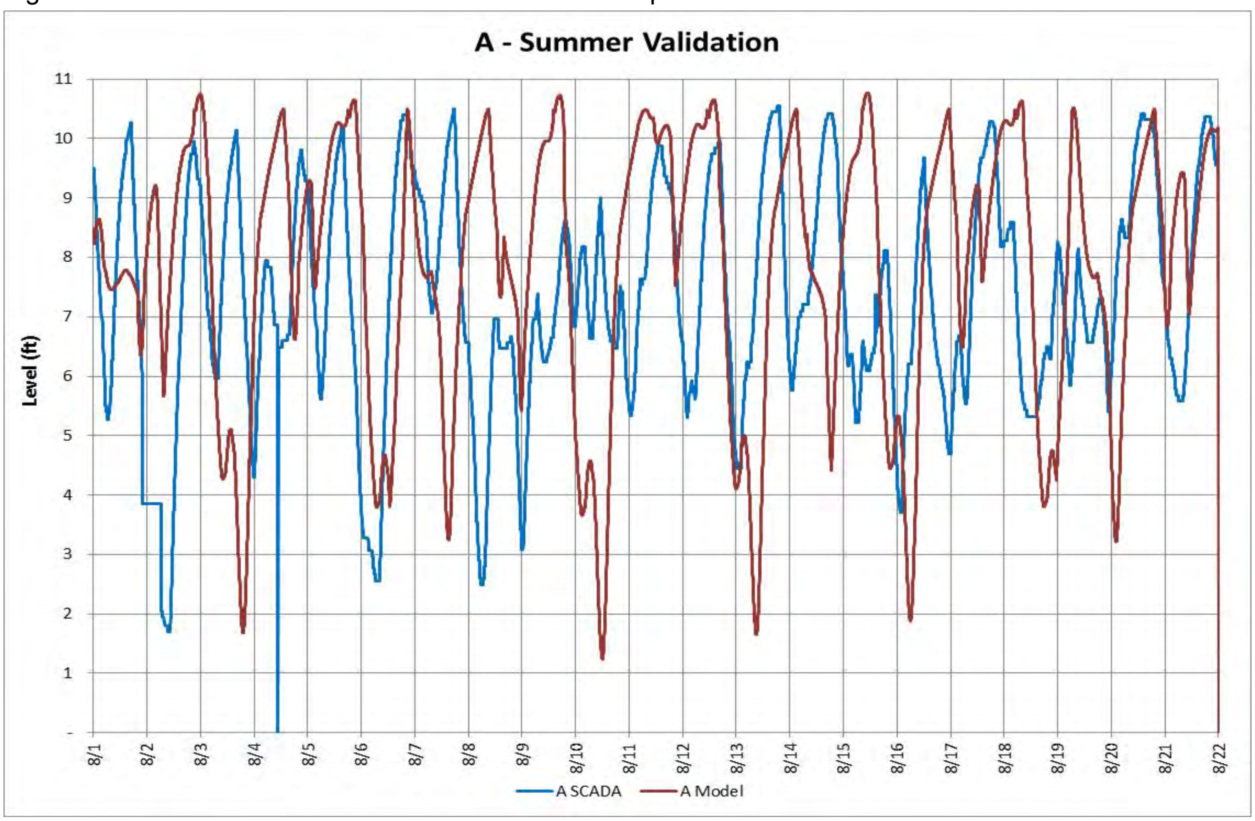


Figure 3-5. E Reservoir Summer Validation Model Output vs. SCADA Data

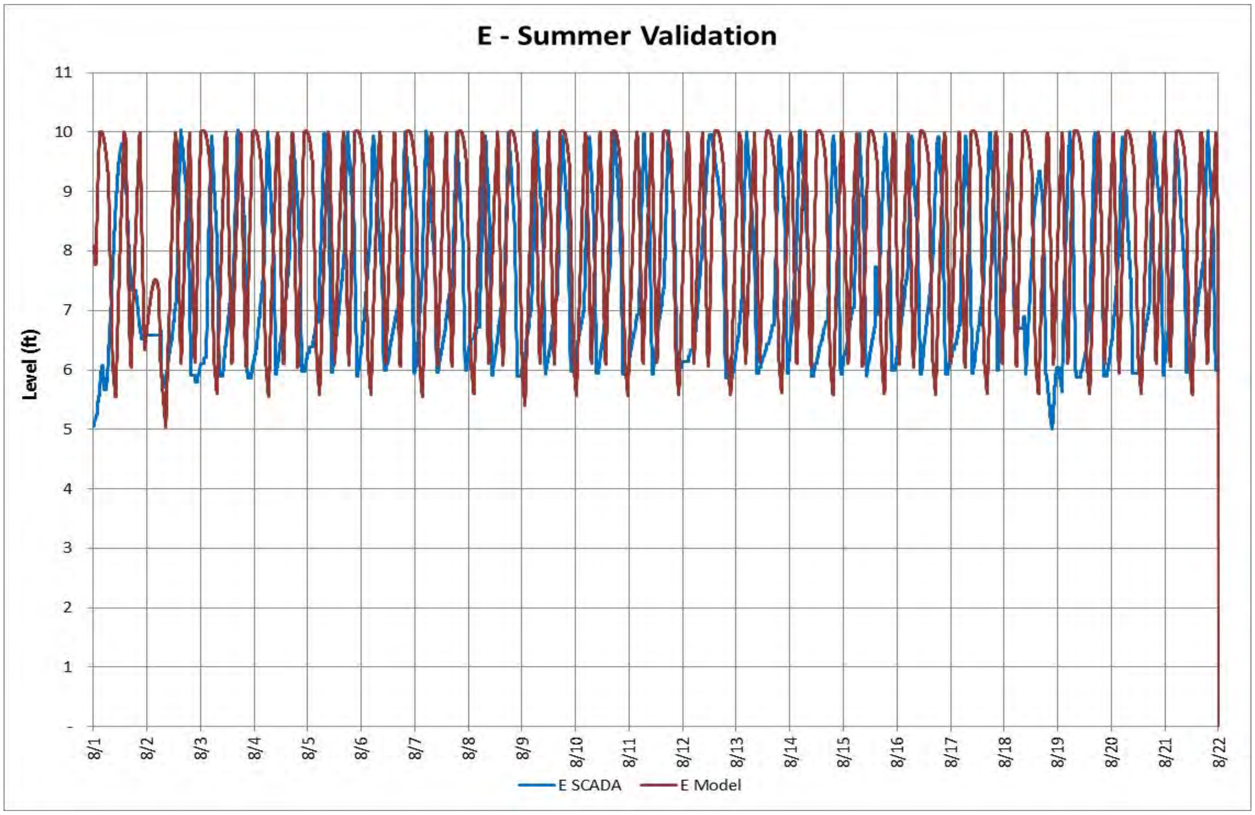


Figure 3-6. C Reservoir Summer Validation Model Output vs. SCADA Data

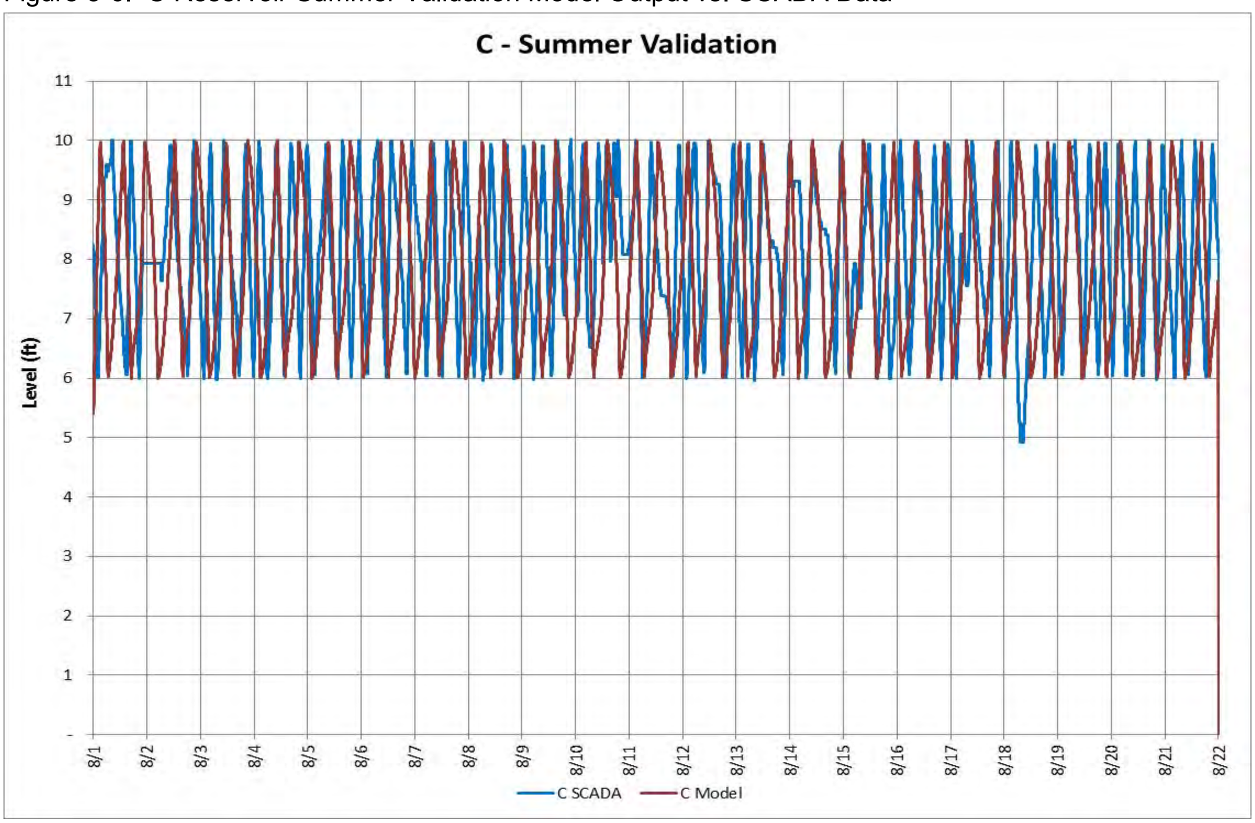


Figure 3-7. H Reservoir Summer Validation Model Output vs. SCADA Data

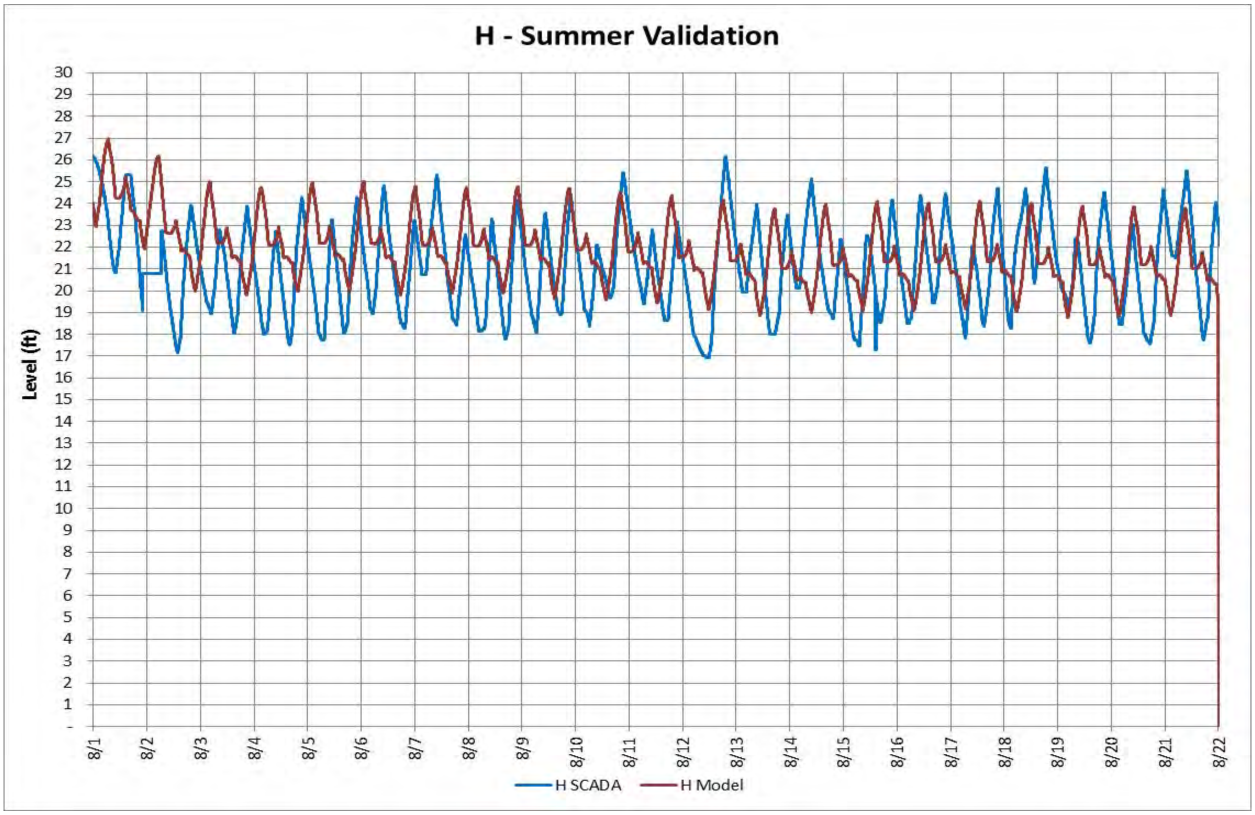


Figure 3-8. Pechstein Reservoir Summer Validation Model Output vs. SCADA Data

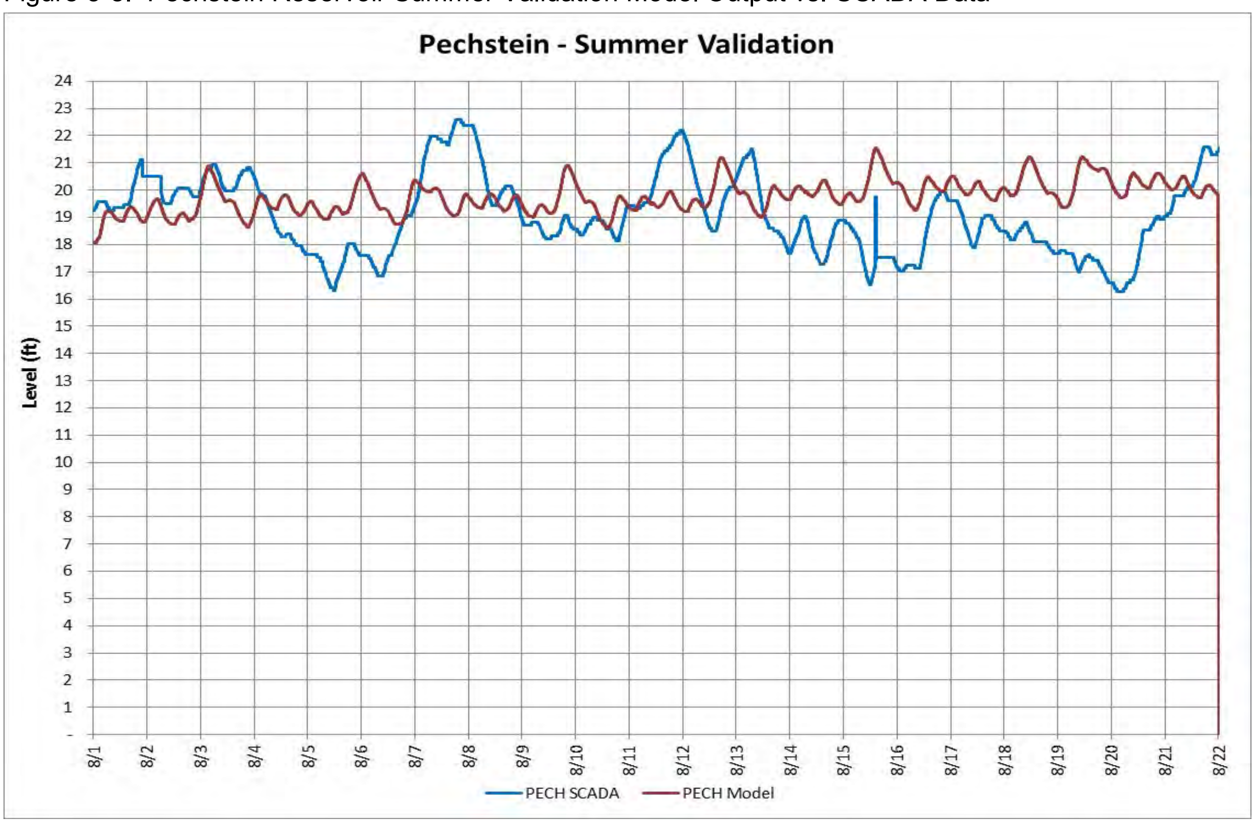


Figure 3-9. HB Reservoir Summer Validation Model Output vs. SCADA Data

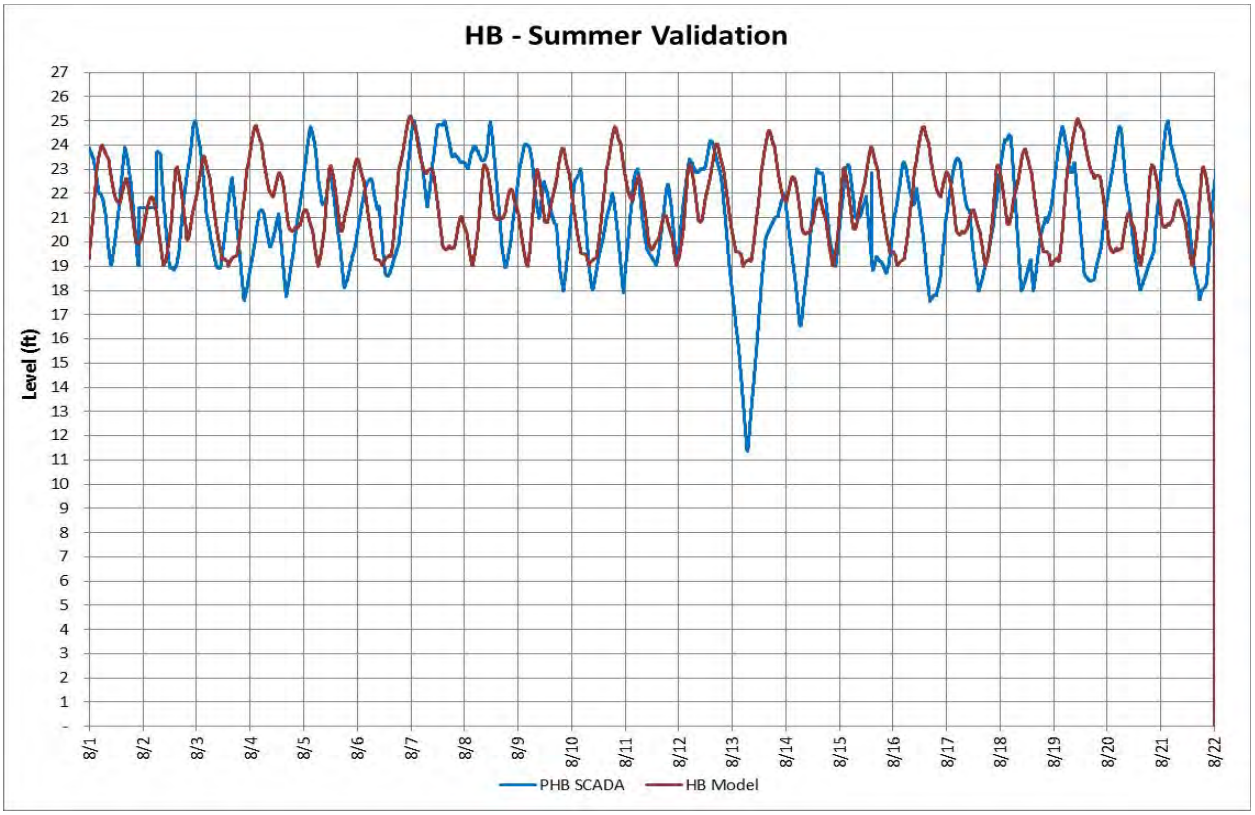


Figure 3-10. San Luis Rey Reservoir Winter Validation Model Output vs. SCADA Data

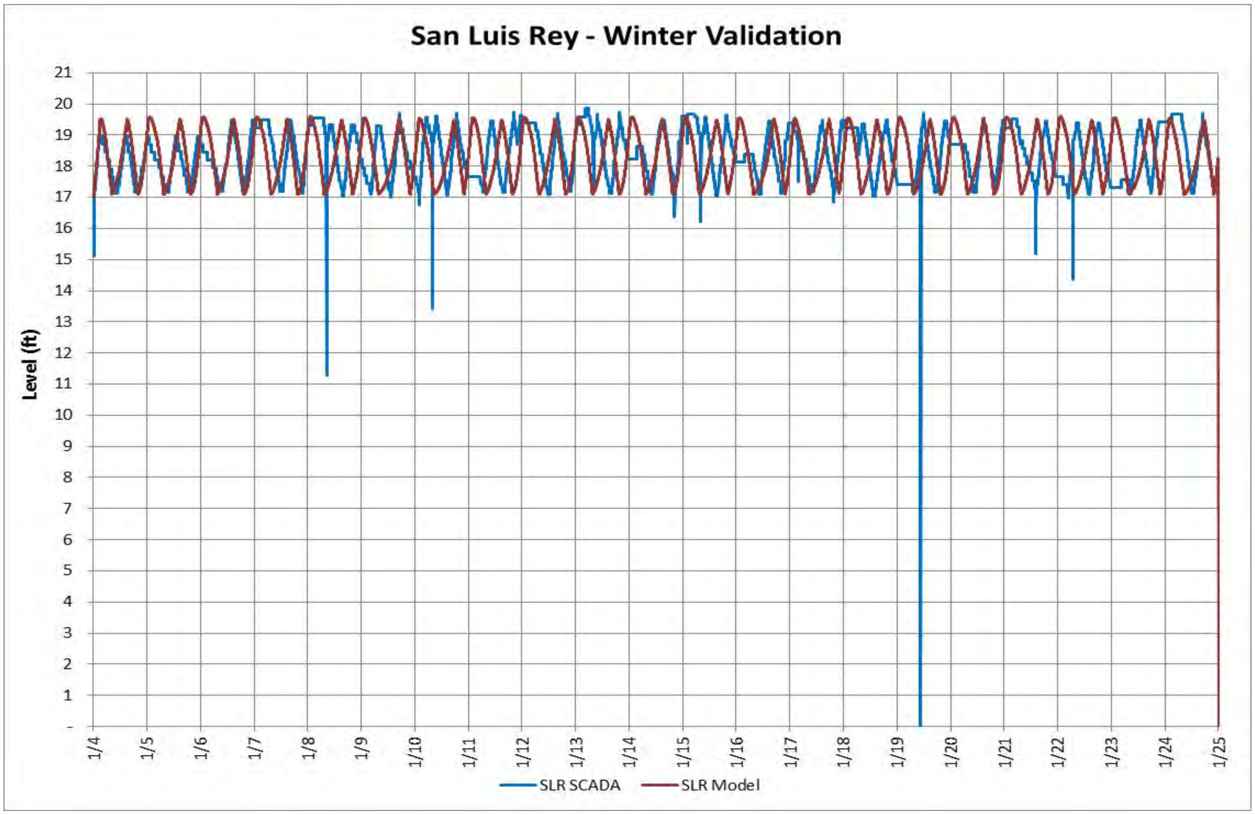


Figure 3-11. E1 Reservoir Winter Validation Model Output vs. SCADA Data

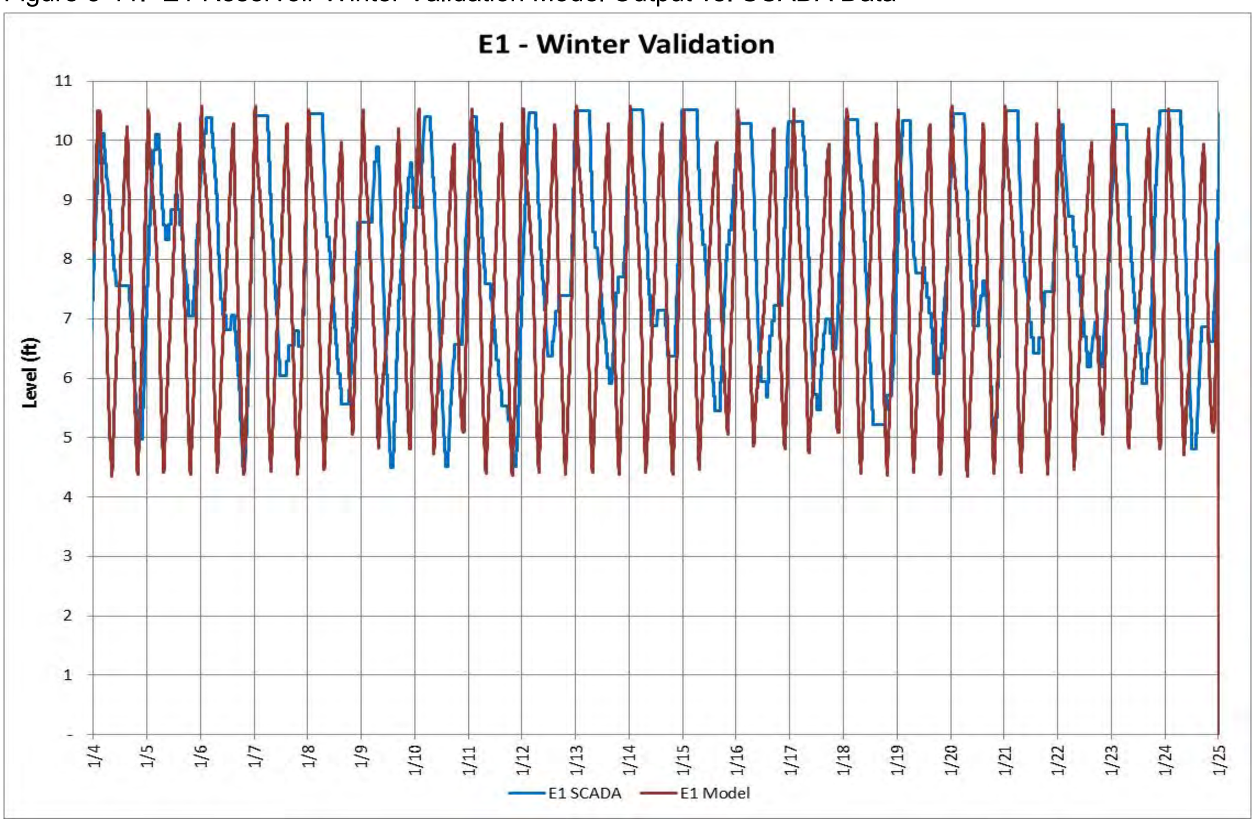


Figure 3-12. Lupine Hills Reservoir Winter Validation Model Output vs. SCADA Data

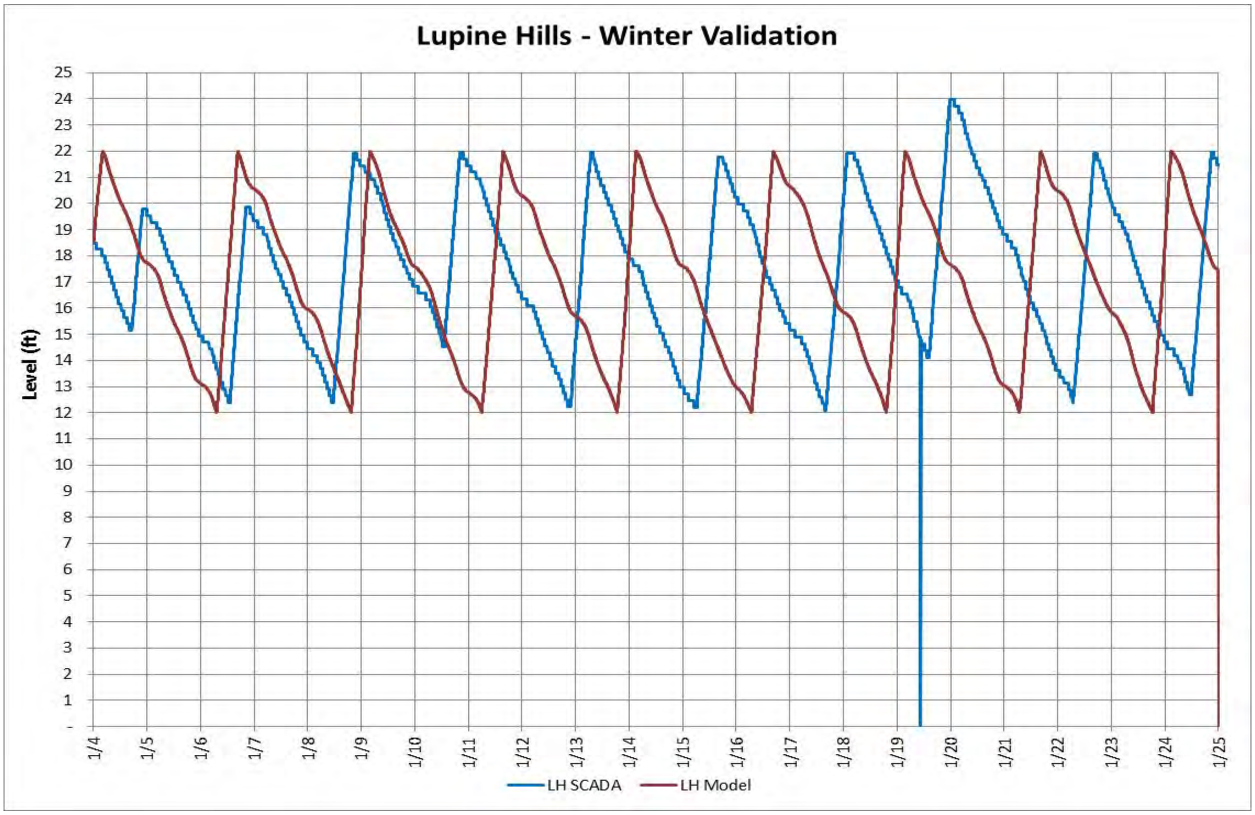


Figure 3-13. A Reservoir Winter Validation Model Output vs. SCADA Data

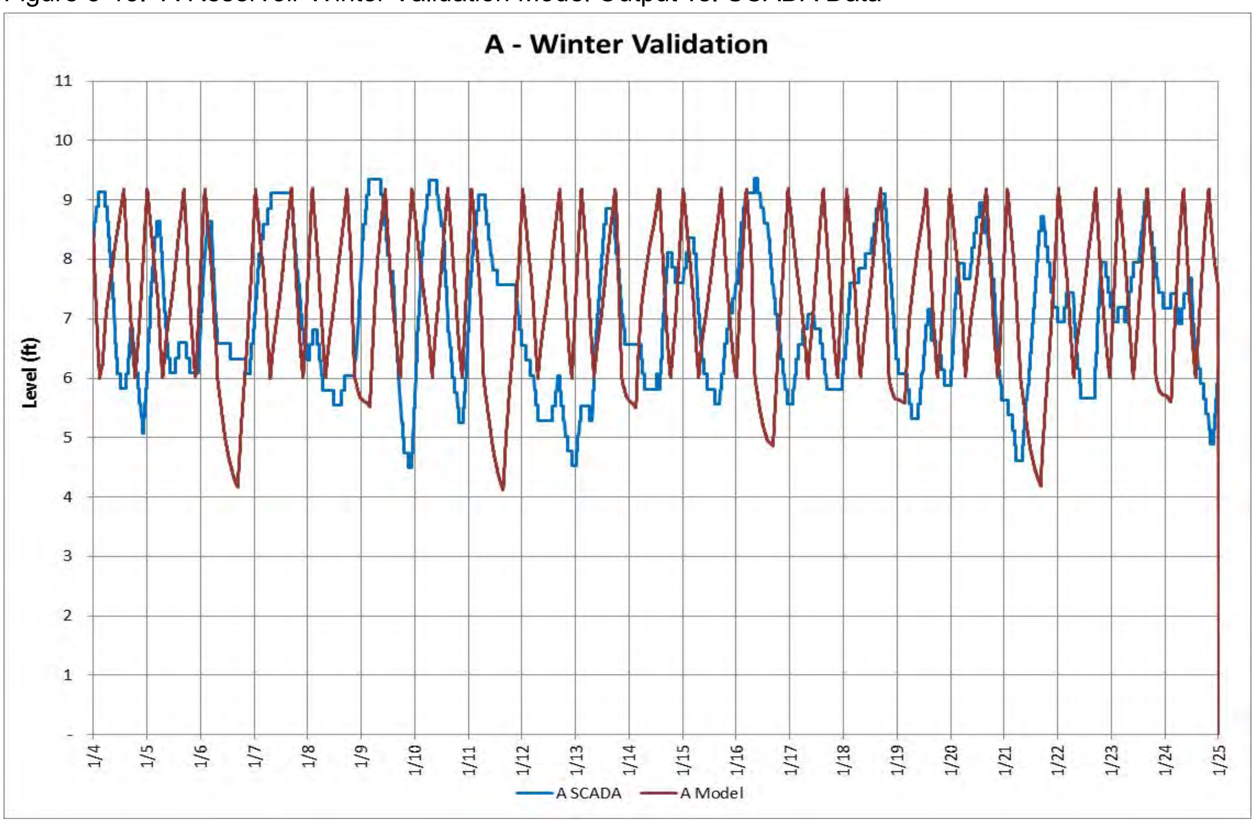


Figure 3-14. E Reservoir Winter Validation Model Output vs. SCADA Data

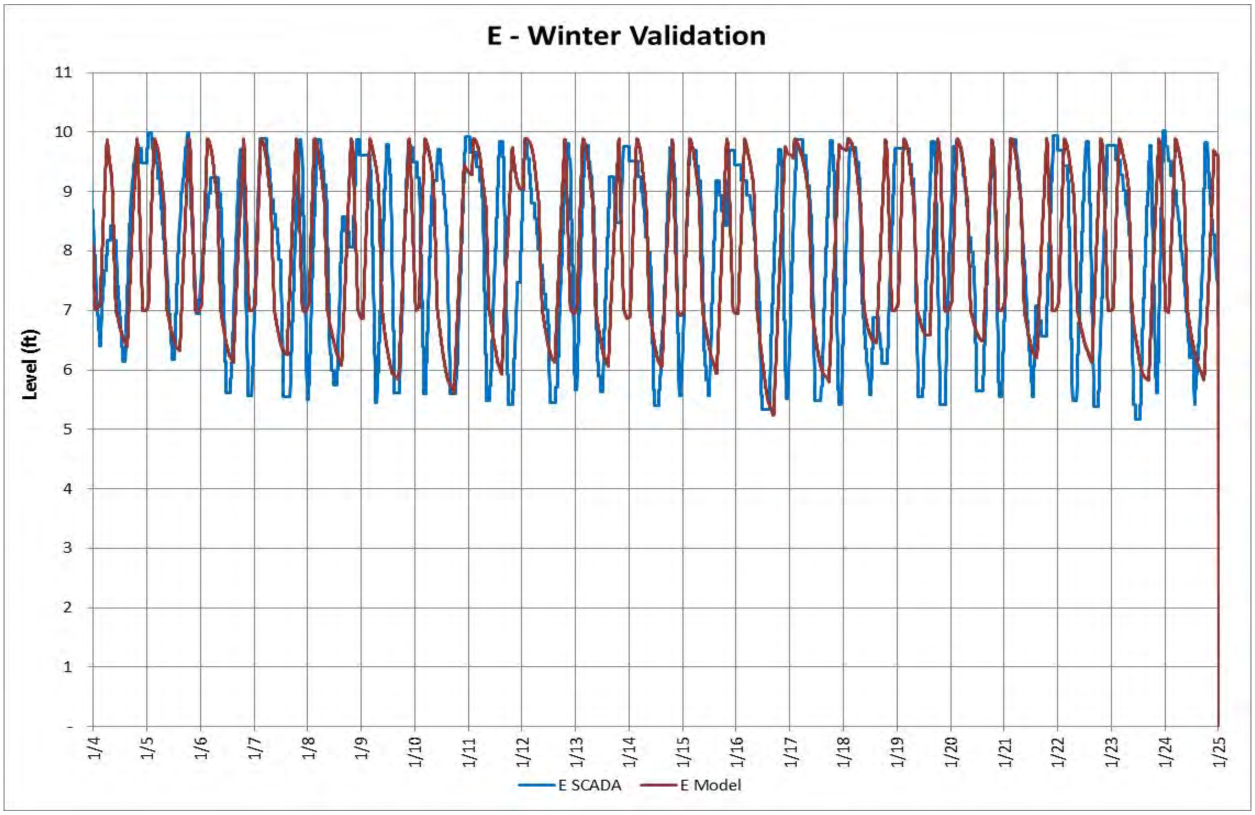


Figure 3-15. C Reservoir Winter Validation Model Output vs. SCADA Data

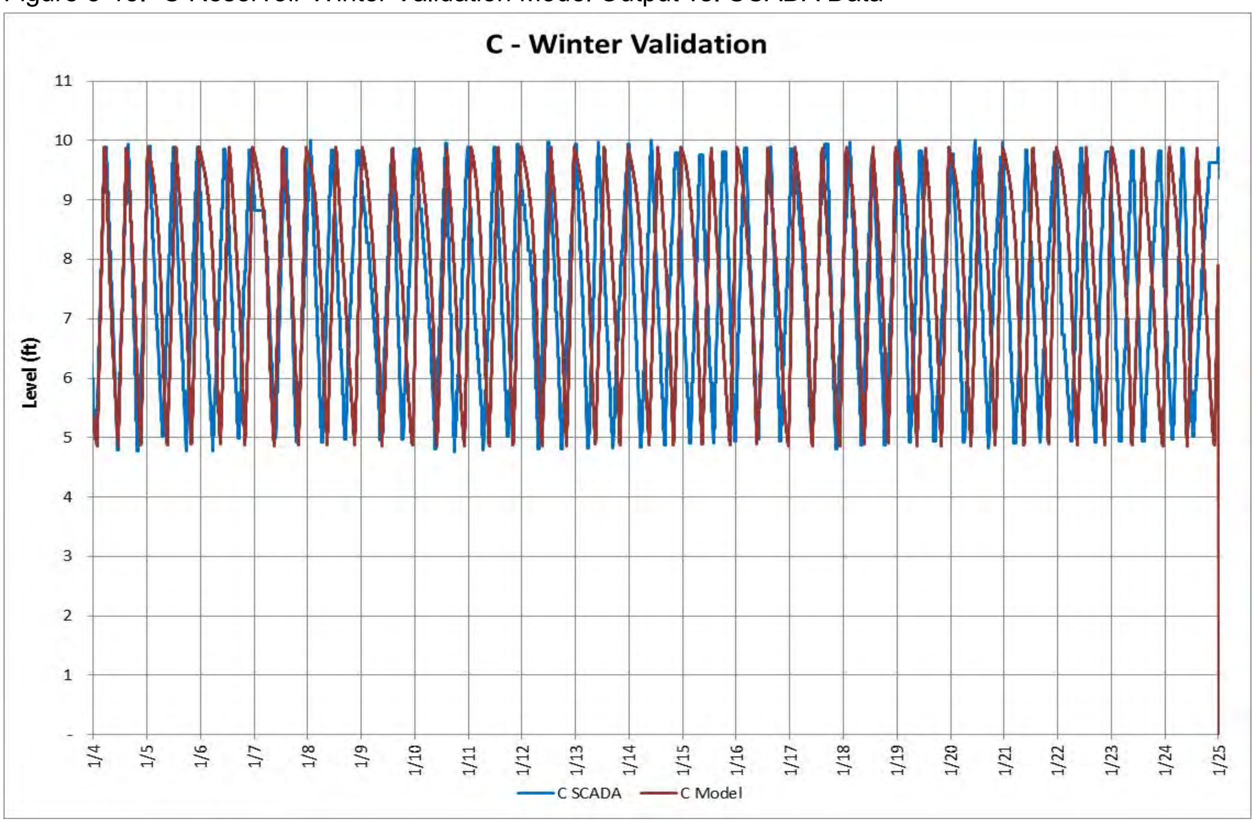


Figure 3-16. H Reservoir Winter Validation Model Output vs. SCADA Data

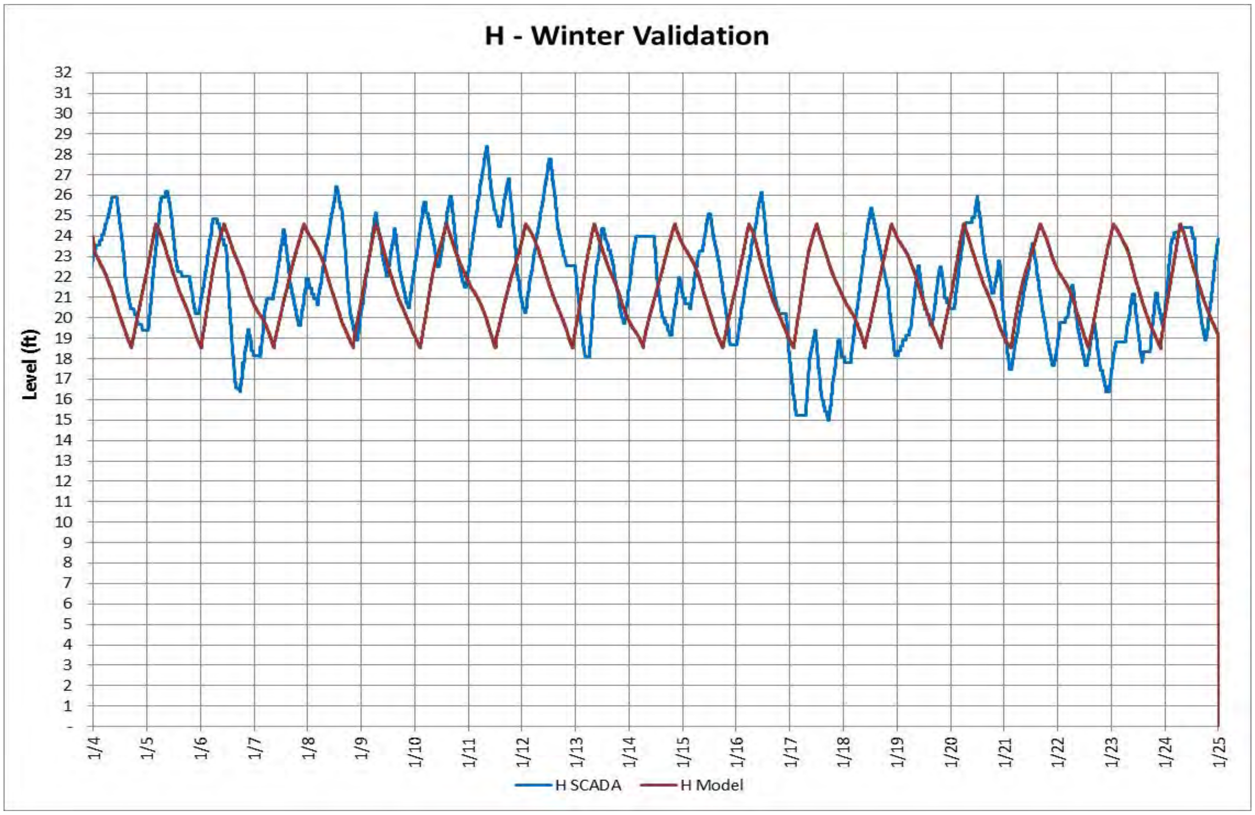


Figure 3-17. Pechstein Reservoir Winter Validation Model Output vs. SCADA Data

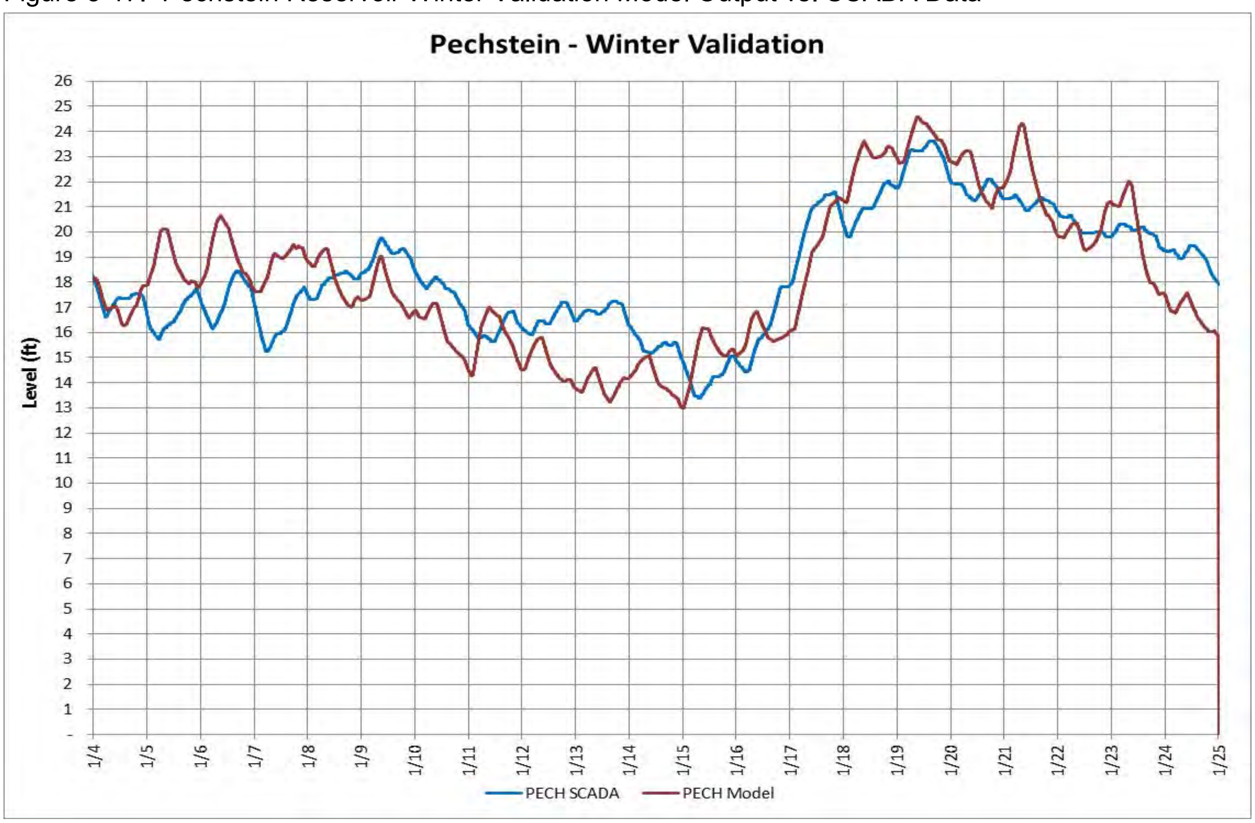


Figure 3-18. HP Reservoir Winter Validation Model Output vs. SCADA Data

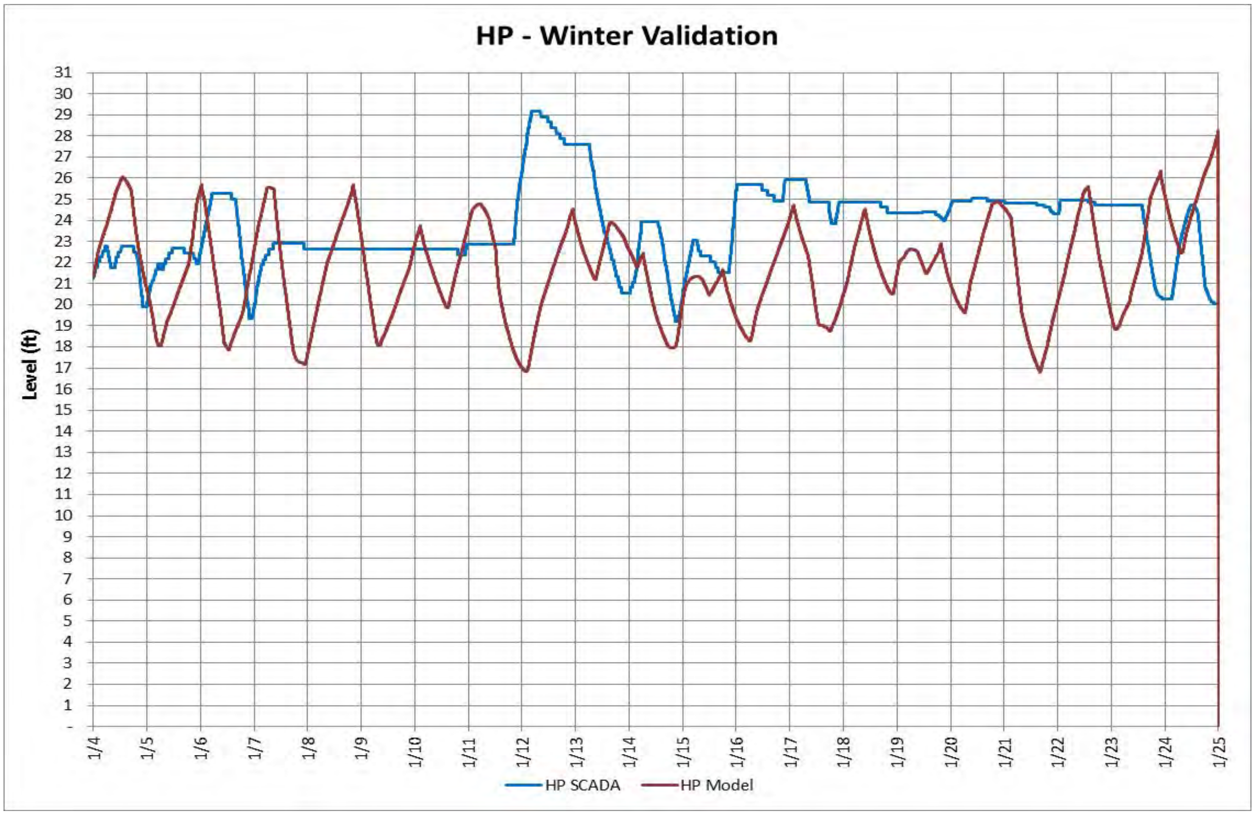


Figure 3-19. HB Reservoir Winter Validation Model Output vs. SCADA Data

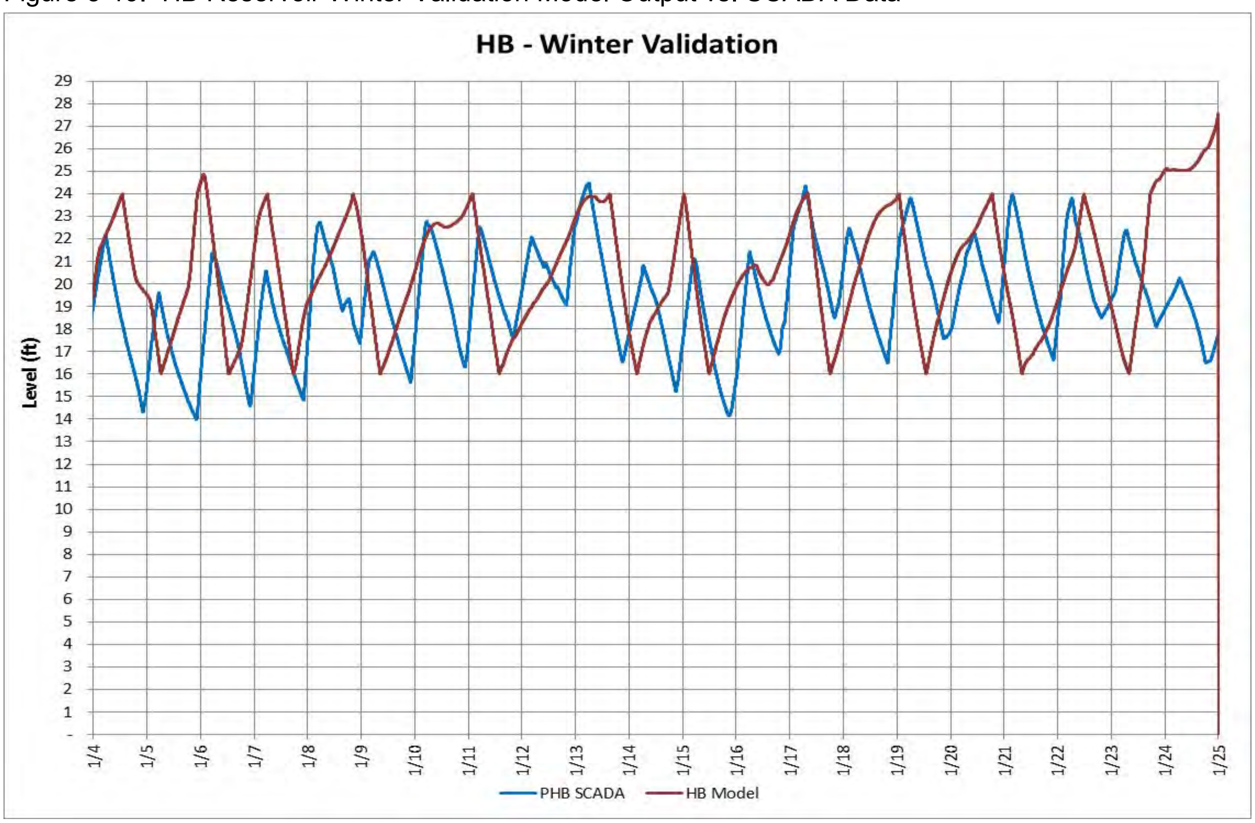


Table 3-1. **Field Hydrant Test Comparison with Model Output**

Zone	Test	Time	Flowing Hydrant		Residual Hydrant 1									Residual Hydrant 2									Note
					Static Pressure [1]			Residual Pressure [2]			Pressure Drop [3]			Static Pressure [1]			Residual Pressure [2]			Pressure Drop [3]			
			H-ID	Flow (gpm)	Field (PSI)	Model (PSI)	Diff [4] (%)	Field (PSI)	Model (PSI)	Diff [4] (%)	Field (PSI)	Model (PSI)	Diff [5] (PSI)	Field (PSI)	Model (PSI)	Diff [4] (%)	Field (PSI)	Model (PSI)	Diff [4] (%)	Field (PSI)	Model (PSI)	Diff [5] (PSI)	
550	1	10:42	13481	850	60	60	0%	56	59	-5%	4	1	3	99	99	0%	99	98	1%	0	1	-1	
550	2	11:15	12559	975	53	54	-2%	50	52	-4%	3	2	1	64	63	2%	62	62	0%	2	1	1	
707	3	12:00	13303	1,100	94	97	-3%	94	94	0%	0	3	-3	131	133	-2%	130	130	0%	1	3	-2	
707	4	12:58	97163	1,150	78	78	0%	71	73	-3%	7	5	2	88	89	-1%	82	84	-2%	6	5	1	
707	5	13:54	13799	900	92	94	-2%	88	89	-1%	4	5	-1	114	116	-2%	112	110	2%	2	6	-4	
837	6	15:18	13502	1,000-1300	143	144	-1%	140	136	3%	3	8	-5	146	145	1%	142	141	1%	4	4	0	
837	7	16:04	13347	1,300	103	107	-4%	100	103	-3%	3	4	-1	100	97	3%	97	96	1%	3	1	2	Valve 5235 may be closed
837	8	16:34	12617	1,500	146	141	3%	145	134	8%	1	7	-6	153	148	3%	151	141	7%	2	7	-5	
637	9	8:01	14327	1,300	80	80	0%	76	75	1%	4	5	-1	73	72	1%	69	69	0%	4	3	1	
637	10	8:23	14399	1,300	112	108	4%	107	100	7%	5	8	-3	118	113	4%	114	103	10%	4	10	-6	
637	11	8:40	11850	1,500	108	103	5%	104	97	7%	4	6	-2	119	117	2%	111	110	1%	8	7	1	
486	12	9:07	11891	1,000	68	68	0%	61	59	3%	7	9	-2	68	68	0%	60	60	0%	8	8	0	
565	13	9:49	14913	1,100	52	52	0%	48	50	-4%	4	2	2	81	78	4%	79	71	10%	2	7	-5	
565	14	10:25	14929	1,600	89	89	0%	88	84	5%	1	5	-4	103	103	0%	102	101	1%	1	2	-1	
668	15	11:34	12878	1,450	124	124	0%	117	122	-4%	7	2	5	127	127	0%	110	101	8%	17	26	-9	
668	16	10:56	12496	1,200	93	95	-2%	78	85	-9%	15	10	5	92	93	-1%	77	82	-6%	15	11	4	
810	17	12:16	12216	1,350	72	71	1%	64	66	-3%	8	5	3	63	62	2%	55	60	-9%	8	2	6	
752	18	13:43	14800	1,650	148	148	0%	146	137	6%	2	11	-9	150	139	7%	146	131	10%	4	8	-4	
752	19	14:05	11642	1,650	164	164	0%	164	157	4%	0	7	-7	121	118	2%	119	115	3%	2	3	-1	
984	20	14:33	11937	1,500	108	105	3%	106	101	5%	2	4	-2	124	119	4%	120	112	7%	4	7	-3	Valve 2651 may be closed
984	21	15:19	14458	1,000	88	88	0%	80	83	-4%	8	5	3	67	64	4%	58	62	-7%	9	2	7	

Note 1: Measured pressure at residual hydrant with flow hydrant closed

Note 2: Measured pressure at residual hydrant with flow hydrant open

Note 3: Difference between static pressure and residual pressure at residual hydrant

Note 4: Percent difference calculated based on field data versus model output as follows: (field-model)/field. Absolute values less than 10% highlighted in green. Absolute values greater than 10% highlighted in yellow.

Note 5: Difference in pressure drop calculated based on field data versus model output as follows: field-model. Absolute values less than 10% highlighted in green. Absolute values greater than 10% highlighted in yellow.

Note 5: Difference in pressure drop calculated based on field data versus model output as follows: field-model. Absolute values less than 10 psi highlighted in green. Absolute values greater than 10 psi highlighted in yellow.

Appendix G. Fire Flow Test Report

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August 17, 2017

HDR
Attn: Jennifer Duffy
8690 Balboa Avenue, Suite 200
San Diego, CA 92123

Subject: Vista Irrigation District Hydrant Testing Results

Ms. Duffy,

Thank you for the opportunity to assist you with the development and calibration of the water system model for Vista Irrigation District (VID). The purpose of this letter is to provide HDR with a summary of the data captured during the hydrant testing.

Field Testing Plan

To collect flow and pressure data for use in water system calibration, the following data was captured; continuous pressure recording at a select location, flow and pressure during hydrant flow testing, and flow and boundary condition recording.

Continuous Pressure Recording

To aide in the hydraulic model calibration hydrant pressure data was collected using a pressure monitor (recorder), on a hydrant cap pressure gauge. Prior to installing the pressure recorder, the hydrant was flushed. At the Field Testing kickoff, detailed system operations were discussed and placement of the recorders was determined. Table 1 summarizes the location of the recorders and the range of pressure that was recorded. Attachment A includes the continuous recorder data.

Table 1. Continuous Pressure Recorders

Recorder	Fire Hydrant	Location	Pressure Recorded (PSI)		
			Min	Avg.	Max
WCC1	11831	1455 W. Vista Way	90.0	93.9	95.7
WCC2	13015	1509 W. Knapp Drive	86.9	88.0	88.7
VID1	14021	1951 Bella Vista Drive	73.5	80.9	86.0
VID2	11214	1931 Alta Vista Drive	87.3	91.4	94.7

Hydrant Flow Tests

Twenty-one hydrant flow tests were conducted to collect model calibration data. During the hydrant flow tests, three hydrants located in close proximity were monitored. During each test, one hydrant is tested for flow and pressure (Flowing Hydrant), and two hydrants are tested for pressure only (Residual Hydrants).

Flowing Hydrant 13799 (Test 5)

The goal of the hydrant testing was to achieve a flow that is as close to a minimum required fire flow as possible without dropping the pressure anywhere in the system by more than 20 psi.

The selected test locations were identified in different geographic regions, for different pipe sizes, to obtain an adequate number of calibration points across the whole model and for various pipe types and sizes. The tests were reorganized chronologically in alphabetic order and the results are presented in Table 2. Attachment B includes the detailed test logs.


Residual Hydrant 13124 (Test 17)

Table 2. Hydrant Flow Test

Test	Time	Flowing Hydrant			Residual Hydrant 1			Residual Hydrant 2		
		H-ID	Flow (gpm)	Flowing (PSI)	H-ID	Static (PSI)	Flowing (PSI)	H-ID	Static (PSI)	Flowing (PSI)
Day 1 – Tuesday August 1, 2017										
1	10:42	13481	850	30	13049	60	56	13483	99	99
2	11:15	12559	975	40	12560	53	50	13881	64	62
3	12:00	13303	1,100	55	13412	94	94	13425	131	130
4	12:58	97163	1,150	50	14313	78	71	14314	88	82
5	13:54	13799	900	35	13978	92	88	14144	114	112
6	15:18	13502	1,000-1,300	Unk.	12735	143	140	14599	146	142
7	16:04	13347	1,300	Unk.	13370	103	100	13635	100	96
8	16:34	12617	1,500	Unk.	14292	146	145	14322	153	151
Day 2 – Wednesday August 2, 2017										
9	8:01	14327	1,300	60	14331	80	76	14739	73	69
10	8:23	14399	1,300	60	14379	112	107	14410	118	114
11	8:40	11850	1,500	80	11611	108	104	11596	119	111
12	9:07	11891	1,000	35	11886	68	61	11881	68	60
13	9:49	14913	1,100	42	12074	52	48	12126	81	79
14	10:25	14929	1,600	80	13176	89	88	97049	103	102
15	11:34	12878	1,450	75	13213	124	117	13243	127	110
16	10:56	12496	1,200	55	12260	93	78	13013	92	77
17	12:16	12216	1,350	65	12210	72	64	13124	63	55
18	13:43	14800	1,650	100	11268	148	146	14892	150	146
19	14:05	11642	1,650	100	11661	164	164	12018	121	119
20	14:33	11937	1,500	80	11933	108	106	11939	124	120
21	15:19	14458	1,000	30	14453	88	80	14463	67	58

Boundary Condition Recording

Boundary conditions were recorded at the time each test is conducted. Boundary conditions were taken via communications with VID personnel located at pressure reducing valves and with SCADA reads at reservoirs, pump stations, and pressure control valves. Attachment C includes field data collected at VID manned pressure reducing valves.

Summary of Observations

The following is a summary of the major observations from the fire flow testing:

- The permanent pressure monitors, or telogs, generally observed typical system pressure patterns, with slightly lower pressures during peak morning periods.
- The fire flow tests showed that the system was robust and able to meet typical minimum fire flows with only limited pressure drops in the system, with the following exceptions:
    - Test 15, 30242 Au Bon Climat Court (668 Pressure Zone) had an approximate 15psi pressure drop at residual hydrant B (Montrachet Street). This is not a concern as the hydrant is located at the end of the pressure zone and had above 100psi of pressure at the residual hydrant.
    - Test 16, Gail Drive at Kevin Drive (668 Pressure Zone) had an approximate 15 psi pressure drop at residual hydrant B (Taylor Street). This is not a concern as the hydrant is located at the end of the pressure zone and had above 75psi of pressure at the residual hydrant.
    - The pressure reducing valves did not open significantly, which suggests that each pressure zone is well looped and supplied from reservoir storage and/or supply feeds.

ATTACHMENT A –Continuous Recorder Data

Date	Time	VID 1	VID 2	WCC 1	WCC 2
08/01/2017	09:36	80.1			
08/01/2017	09:37	80.6			
08/01/2017	09:38	80.4			
08/01/2017	09:39	81.6			
08/01/2017	09:40	81.7			
08/01/2017	09:41	81.1			
08/01/2017	09:42	81.0			
08/01/2017	09:43	81.4		94.0	
08/01/2017	09:44	81.4		94.4	
08/01/2017	09:45	80.5		93.6	
08/01/2017	09:46	79.6		94.7	
08/01/2017	09:47	80.7		94.9	
08/01/2017	09:48	79.6		93.3	
08/01/2017	09:49	79.3		94.7	
08/01/2017	09:50	78.7	92.0	94.3	
08/01/2017	09:51	79.9	92.4	93.8	
08/01/2017	09:52	79.6	92.6	93.4	
08/01/2017	09:53	79.9	92.5	92.8	
08/01/2017	09:54	79.8	92.6	93.9	
08/01/2017	09:55	79.0	93.6	93.3	
08/01/2017	09:56	79.5	93.1	93.7	
08/01/2017	09:57	78.9	92.8	93.7	
08/01/2017	09:58	78.9	93.0	92.8	
08/01/2017	09:59	79.5	92.3	93.3	
08/01/2017	10:00	79.2	92.9	94.5	
08/01/2017	10:01	79.5	92.5	93.9	
08/01/2017	10:02	78.8	91.8	93.6	
08/01/2017	10:03	78.4	92.8	93.7	
08/01/2017	10:04	78.5	93.1	94.6	
08/01/2017	10:05	78.0	93.6	93.5	
08/01/2017	10:06	76.8	92.7	94.4	
08/01/2017	10:07	78.5	92.5	93.5	
08/01/2017	10:08	78.3	92.6	94.4	
08/01/2017	10:09	78.0	92.4	94.6	
08/01/2017	10:10	77.7	92.3	93.5	
08/01/2017	10:11	77.9	91.9	94.7	
08/01/2017	10:12	77.9	92.9	92.9	
08/01/2017	10:13	78.6	92.5	94.6	
08/01/2017	10:14	77.6	92.3	93.7	
08/01/2017	10:15	77.9	92.5	93.9	
08/01/2017	10:16	77.5	92.2	93.6	
08/01/2017	10:17	77.3	92.8	94.5	
08/01/2017	10:18	77.6	92.1	93.2	
08/01/2017	10:19	77.3	92.6	94.6	
08/01/2017	10:20	77.1	92.7	94.5	
08/01/2017	10:21	77.4	91.9	94.0	

08/01/2017	10:22	77.8	92.7	94.2
08/01/2017	10:23	77.6	92.7	94.1
08/01/2017	10:24	77.2	92.9	93.7
08/01/2017	10:25	77.0	92.1	94.8
08/01/2017	10:26	76.7	92.1	94.0
08/01/2017	10:27	77.2	92.6	93.6
08/01/2017	10:28	77.1	91.5	93.6
08/01/2017	10:29	76.6	92.0	91.6
08/01/2017	10:30	77.4	92.0	93.2
08/01/2017	10:31	75.7	92.2	94.6
08/01/2017	10:32	76.6	93.6	93.5
08/01/2017	10:33	77.2	93.0	93.5
08/01/2017	10:34	76.9	91.8	93.2
08/01/2017	10:35	77.2	92.6	93.2
08/01/2017	10:36	76.4	92.0	94.4
08/01/2017	10:37	77.9	92.3	93.1
08/01/2017	10:38	76.6	91.9	94.0
08/01/2017	10:39	77.1	92.5	93.0
08/01/2017	10:40	77.3	91.6	94.7
08/01/2017	10:41	77.6	91.6	93.6
08/01/2017	10:42	76.9	92.3	93.7
08/01/2017	10:43	77.8	92.8	94.0
08/01/2017	10:44	77.0	91.3	94.1
08/01/2017	10:45	78.2	91.9	94.2
08/01/2017	10:46	78.5	91.8	93.8
08/01/2017	10:47	78.2	92.6	94.1
08/01/2017	10:48	78.4	92.5	93.0
08/01/2017	10:49	78.6	91.6	93.8
08/01/2017	10:50	78.4	92.1	94.3
08/01/2017	10:51	79.1	91.8	94.2
08/01/2017	10:52	78.9	91.9	94.6
08/01/2017	10:53	78.6	92.2	92.9
08/01/2017	10:54	79.8	92.1	93.7
08/01/2017	10:55	79.5	91.2	93.7
08/01/2017	10:56	79.3	92.4	93.9
08/01/2017	10:57	79.5	91.9	93.2
08/01/2017	10:58	80.0	91.5	94.0
08/01/2017	10:59	79.5	90.9	94.7
08/01/2017	11:00	79.6	91.0	93.8
08/01/2017	11:01	79.4	91.0	94.5
08/01/2017	11:02	80.0	91.5	94.1
08/01/2017	11:03	80.2	91.9	93.0
08/01/2017	11:04	80.0	91.5	94.4
08/01/2017	11:05	80.3	91.1	93.2
08/01/2017	11:06	80.2	91.3	94.7
08/01/2017	11:07	80.1	90.9	93.9
08/01/2017	11:08	80.2	91.2	94.0

08/01/2017	11:09	80.0	90.7	93.9
08/01/2017	11:10	80.2	90.6	93.3
08/01/2017	11:11	80.4	90.6	94.6
08/01/2017	11:12	80.3	90.8	94.1
08/01/2017	11:13	78.8	90.9	92.8
08/01/2017	11:14	81.1	91.0	93.6
08/01/2017	11:15	80.7	89.7	94.4
08/01/2017	11:16	81.2	90.7	95.3
08/01/2017	11:17	81.3	90.8	93.3
08/01/2017	11:18	81.0	90.0	95.2
08/01/2017	11:19	81.2	89.8	94.1
08/01/2017	11:20	81.5	90.6	95.2
08/01/2017	11:21	82.0	90.0	93.9
08/01/2017	11:22	81.8	90.2	93.9
08/01/2017	11:23	81.6	89.8	94.5
08/01/2017	11:24	81.9	89.9	93.7
08/01/2017	11:25	81.8	89.6	94.0
08/01/2017	11:26	82.0	90.0	92.6
08/01/2017	11:27	81.6	89.3	92.7
08/01/2017	11:28	81.5	90.3	94.3
08/01/2017	11:29	81.4	90.1	94.7
08/01/2017	11:30	81.6	89.6	92.9
08/01/2017	11:31	81.7	90.1	94.0
08/01/2017	11:32	81.8	90.0	94.3
08/01/2017	11:33	81.8	90.4	95.4
08/01/2017	11:34	81.8	90.1	93.7
08/01/2017	11:35	82.2	90.0	94.7
08/01/2017	11:36	81.7	90.7	94.2
08/01/2017	11:37	81.7	90.3	94.7
08/01/2017	11:38	81.4	90.3	93.6
08/01/2017	11:39	81.6	90.4	94.1
08/01/2017	11:40	82.0	90.7	94.1
08/01/2017	11:41	81.6	90.4	94.9
08/01/2017	11:42	81.3	90.8	94.4
08/01/2017	11:43	81.6	91.3	94.7
08/01/2017	11:44	81.6	90.1	94.3
08/01/2017	11:45	81.5	90.3	93.7
08/01/2017	11:46	81.7	90.0	94.1
08/01/2017	11:47	81.9	90.7	94.4
08/01/2017	11:48	81.2	90.3	93.0
08/01/2017	11:49	81.5	92.0	94.3
08/01/2017	11:50	81.1	90.5	93.5
08/01/2017	11:51	81.5	90.7	95.1
08/01/2017	11:52	81.7	90.6	94.5
08/01/2017	11:53	81.4	90.1	95.0
08/01/2017	11:54	81.4	90.7	94.1
08/01/2017	11:55	81.2	90.4	93.9

08/01/2017	11:56	81.4	90.6	94.3
08/01/2017	11:57	80.9	91.4	93.9
08/01/2017	11:58	81.1	90.6	94.4
08/01/2017	11:59	79.3	89.7	94.5
08/01/2017	12:00	81.8	90.8	94.1
08/01/2017	12:01	80.9	90.9	94.7
08/01/2017	12:02	81.0	90.6	94.4
08/01/2017	12:03	81.2	90.5	93.5
08/01/2017	12:04	81.7	90.5	93.5
08/01/2017	12:05	80.9	89.8	94.1
08/01/2017	12:06	81.8	90.8	92.7
08/01/2017	12:07	80.8	88.8	93.8
08/01/2017	12:08	81.1	90.0	94.7
08/01/2017	12:09	81.1	90.0	93.7
08/01/2017	12:10	81.1	90.8	93.1
08/01/2017	12:11	81.8	90.4	93.6
08/01/2017	12:12	81.4	90.3	93.3
08/01/2017	12:13	81.1	90.0	93.7
08/01/2017	12:14	81.4	91.0	94.7
08/01/2017	12:15	81.1	90.6	93.9
08/01/2017	12:16	81.1	90.5	93.7
08/01/2017	12:17	80.3	90.4	92.8
08/01/2017	12:18	81.5	89.5	93.4
08/01/2017	12:19	81.5	91.4	93.4
08/01/2017	12:20	81.6	90.6	94.3
08/01/2017	12:21	81.4	89.6	94.2
08/01/2017	12:22	81.3	89.8	94.0
08/01/2017	12:23	81.3	89.8	94.1
08/01/2017	12:24	81.5	90.2	94.9
08/01/2017	12:25	80.7	89.5	93.9
08/01/2017	12:26	81.8	90.3	94.7
08/01/2017	12:27	81.9	90.3	94.0
08/01/2017	12:28	81.6	90.3	93.9
08/01/2017	12:29	81.5	89.7	94.0
08/01/2017	12:30	81.6	90.3	93.7
08/01/2017	12:31	81.4	89.1	93.2
08/01/2017	12:32	81.3	89.7	94.2
08/01/2017	12:33	82.1	89.2	94.6
08/01/2017	12:34	81.5	89.0	93.2
08/01/2017	12:35	80.5	89.8	93.4
08/01/2017	12:36	80.9	90.0	92.9
08/01/2017	12:37	81.2	90.2	94.5
08/01/2017	12:38	81.7	89.1	93.2
08/01/2017	12:39	81.7	89.7	93.9
08/01/2017	12:40	81.9	89.4	93.3
08/01/2017	12:41	82.4	89.2	94.0
08/01/2017	12:42	81.4	89.6	94.3

08/01/2017	12:43	82.3	89.6	94.2
08/01/2017	12:44	82.2	89.8	95.2
08/01/2017	12:45	81.7	89.7	94.0
08/01/2017	12:46	81.9	89.7	93.2
08/01/2017	12:47	81.8	89.5	93.6
08/01/2017	12:48	82.3	88.9	94.6
08/01/2017	12:49	82.0	89.2	94.3
08/01/2017	12:50	82.0	89.3	94.9
08/01/2017	12:51	81.6	89.1	93.9
08/01/2017	12:52	81.9	89.2	93.5
08/01/2017	12:53	81.8	89.4	93.9
08/01/2017	12:54	81.7	89.4	94.2
08/01/2017	12:55	82.4	88.3	94.0
08/01/2017	12:56	81.7	90.1	93.1
08/01/2017	12:57	81.9	89.1	93.5
08/01/2017	12:58	82.2	88.5	93.7
08/01/2017	12:59	81.3	88.8	92.5
08/01/2017	13:00	81.9	88.5	94.3
08/01/2017	13:01	81.5	89.5	94.0
08/01/2017	13:02	82.1	88.8	94.3
08/01/2017	13:03	81.7	89.3	94.3
08/01/2017	13:04	81.9	89.3	94.0
08/01/2017	13:05	81.4	89.3	93.9
08/01/2017	13:06	81.8	88.8	93.3
08/01/2017	13:07	81.7	88.1	94.7
08/01/2017	13:08	81.6	88.3	94.4
08/01/2017	13:09	81.6	88.4	94.2
08/01/2017	13:10	82.5	88.6	94.3
08/01/2017	13:11	81.6	88.8	93.9
08/01/2017	13:12	81.8	88.7	93.1
08/01/2017	13:13	82.7	88.5	93.6
08/01/2017	13:14	82.5	88.5	94.3
08/01/2017	13:15	81.8	89.2	93.4
08/01/2017	13:16	81.9	88.0	93.4
08/01/2017	13:17	81.4	88.3	93.6
08/01/2017	13:18	82.7	89.1	94.8
08/01/2017	13:19	81.9	89.6	92.6
08/01/2017	13:20	82.0	88.9	95.1
08/01/2017	13:21	81.8	89.1	94.6
08/01/2017	13:22	82.2	88.7	95.1
08/01/2017	13:23	82.1	89.1	94.8
08/01/2017	13:24	82.9	88.8	93.8
08/01/2017	13:25	80.9	88.6	93.7
08/01/2017	13:26	79.6	89.4	94.6
08/01/2017	13:27	81.7	89.4	95.0
08/01/2017	13:28	81.5	88.8	93.9
08/01/2017	13:29	81.3	89.4	93.0

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08/01/2017	13:31	81.4	88.9	93.5
08/01/2017	13:32	81.3	89.2	94.6
08/01/2017	13:33	81.8	89.2	94.0
08/01/2017	13:34	81.9	88.8	95.0
08/01/2017	13:35	82.7	88.7	94.0
08/01/2017	13:36	82.0	89.6	94.0
08/01/2017	13:37	81.7	90.2	94.9
08/01/2017	13:38	81.5	89.4	94.0
08/01/2017	13:39	81.7	89.3	94.0
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08/01/2017	13:41	81.9	89.4	93.8
08/01/2017	13:42	81.9	89.6	95.0
08/01/2017	13:43	81.9	89.2	94.3
08/01/2017	13:44	82.2	89.5	94.5
08/01/2017	13:45	82.1	89.4	94.4
08/01/2017	13:46	82.2	89.0	93.9
08/01/2017	13:47	82.4	89.8	93.7
08/01/2017	13:48	81.8	89.2	94.3
08/01/2017	13:49	82.6	89.3	94.8
08/01/2017	13:50	82.2	89.2	94.6
08/01/2017	13:51	82.3	89.5	94.1
08/01/2017	13:52	82.2	89.4	93.4
08/01/2017	13:53	81.7	89.1	93.9
08/01/2017	13:54	82.1	89.0	94.3
08/01/2017	13:55	82.1	89.1	93.7
08/01/2017	13:56	81.9	89.0	94.0
08/01/2017	13:57	82.0	89.0	94.4
08/01/2017	13:58	81.5	88.7	94.0
08/01/2017	13:59	81.7	88.8	94.4
08/01/2017	14:00	82.2	89.0	94.4
08/01/2017	14:01	82.2	88.8	94.1
08/01/2017	14:02	82.1	88.6	94.5
08/01/2017	14:03	81.0	89.2	94.2
08/01/2017	14:04	81.5	88.6	94.7
08/01/2017	14:05	82.3	88.7	94.1
08/01/2017	14:06	81.2	89.1	95.1
08/01/2017	14:07	81.1	88.8	94.6
08/01/2017	14:08	81.6	89.2	94.0
08/01/2017	14:09	81.5	89.3	93.8
08/01/2017	14:10	81.2	88.5	94.7
08/01/2017	14:11	82.6	88.9	93.9
08/01/2017	14:12	81.6	89.8	94.1
08/01/2017	14:13	81.9	89.2	93.9
08/01/2017	14:14	82.2	88.4	94.8
08/01/2017	14:15	82.2	89.3	93.4
08/01/2017	14:16	82.5	89.2	95.0

08/01/2017	14:17	82.5	89.4	93.3
08/01/2017	14:18	82.3	88.9	95.4
08/01/2017	14:19	82.6	89.0	94.5
08/01/2017	14:20	82.9	88.7	94.6
08/01/2017	14:21	82.7	89.5	94.4
08/01/2017	14:22	82.1	89.1	94.8
08/01/2017	14:23	82.2	88.5	93.8
08/01/2017	14:24	82.7	89.7	95.0
08/01/2017	14:25	82.6	90.1	94.6
08/01/2017	14:26	82.5	89.2	94.7
08/01/2017	14:27	82.7	88.6	94.4
08/01/2017	14:28	82.4	88.9	93.7
08/01/2017	14:29	82.3	90.1	92.5
08/01/2017	14:30	82.7	89.6	95.4
08/01/2017	14:31	82.9	89.1	94.6
08/01/2017	14:32	82.7	89.6	94.7
08/01/2017	14:33	83.0	88.6	93.6
08/01/2017	14:34	82.8	89.1	94.6
08/01/2017	14:35	82.9	89.3	94.3
08/01/2017	14:36	83.0	89.4	94.9
08/01/2017	14:37	83.6	90.2	94.7
08/01/2017	14:38	82.9	89.7	93.2
08/01/2017	14:39	83.2	90.1	93.7
08/01/2017	14:40	83.8	89.9	94.6
08/01/2017	14:41	83.5	90.4	94.4
08/01/2017	14:42	83.5	90.0	94.0
08/01/2017	14:43	83.6	89.8	94.0
08/01/2017	14:44	83.1	89.5	93.8
08/01/2017	14:45	83.2	89.5	94.3
08/01/2017	14:46	83.8	89.5	94.3
08/01/2017	14:47	83.1	90.5	94.8
08/01/2017	14:48	83.6	90.5	94.1
08/01/2017	14:49	83.8	90.6	93.2
08/01/2017	14:50	83.8	90.8	94.6
08/01/2017	14:51	83.5	90.8	93.5
08/01/2017	14:52	83.7	90.8	94.3
08/01/2017	14:53	83.6	90.6	93.7
08/01/2017	14:54	84.3	90.7	95.3
08/01/2017	14:55	83.6	91.0	94.7
08/01/2017	14:56	84.2	90.7	93.8
08/01/2017	14:57	84.2	90.6	94.6
08/01/2017	14:58	84.2	91.8	93.8
08/01/2017	14:59	84.7	90.4	93.9
08/01/2017	15:00	83.7	90.9	92.1
08/01/2017	15:01	83.8	91.4	94.5
08/01/2017	15:02	84.2	91.8	93.7
08/01/2017	15:03	85.0	91.9	93.8

08/01/2017	15:04	75.9	91.9	93.4
08/01/2017	15:05	80.8	91.0	94.8
08/01/2017	15:06	81.3	90.9	94.6
08/01/2017	15:07	84.0	91.8	93.6
08/01/2017	15:08	84.2	91.9	93.9
08/01/2017	15:09	84.1	91.1	93.3
08/01/2017	15:10	84.7	91.7	94.4
08/01/2017	15:11	83.6	91.6	93.8
08/01/2017	15:12	82.3	91.9	94.3
08/01/2017	15:13	83.8	92.2	94.1
08/01/2017	15:14	84.0	91.8	94.7
08/01/2017	15:15	83.4	92.0	94.0
08/01/2017	15:16	84.4	92.1	94.0
08/01/2017	15:17	83.4	91.7	94.4
08/01/2017	15:18	83.3	91.4	94.4
08/01/2017	15:19	83.0	92.7	94.6
08/01/2017	15:20	82.2	92.6	94.6
08/01/2017	15:21	83.4	92.7	93.9
08/01/2017	15:22	84.2	92.6	94.8
08/01/2017	15:23	84.7	91.8	93.4
08/01/2017	15:24	84.1	92.6	94.7
08/01/2017	15:25	83.6	92.4	94.4
08/01/2017	15:26	83.3	92.7	94.5
08/01/2017	15:27	83.4	92.3	94.3
08/01/2017	15:28	83.4	91.9	94.3
08/01/2017	15:29	84.9	92.3	93.9
08/01/2017	15:30	83.5	92.7	95.0
08/01/2017	15:31	83.7	92.7	94.6
08/01/2017	15:32	83.8	92.6	93.7
08/01/2017	15:33	83.4	92.4	93.7
08/01/2017	15:34	83.9	92.2	94.1
08/01/2017	15:35	83.0	92.1	94.0
08/01/2017	15:36	83.3	92.4	93.5
08/01/2017	15:37	82.9	92.3	94.1
08/01/2017	15:38	82.9	92.5	93.9
08/01/2017	15:39	83.0	92.8	94.5
08/01/2017	15:40	83.1	92.1	94.4
08/01/2017	15:41	83.2	91.8	94.7
08/01/2017	15:42	83.4	92.3	94.1
08/01/2017	15:43	81.8	92.7	94.6
08/01/2017	15:44	82.4	92.1	93.6
08/01/2017	15:45	82.4	92.3	94.2
08/01/2017	15:46	82.1	93.8	94.3
08/01/2017	15:47	83.4	92.4	94.1
08/01/2017	15:48	82.8	92.5	95.0
08/01/2017	15:49	83.1	92.8	93.7
08/01/2017	15:50	82.4	91.8	94.5

08/01/2017	15:51	83.3	92.7	94.0
08/01/2017	15:52	83.0	92.5	94.1
08/01/2017	15:53	82.9	92.3	94.6
08/01/2017	15:54	82.5	91.5	94.4
08/01/2017	15:55	82.8	92.4	94.6
08/01/2017	15:56	83.4	91.7	94.0
08/01/2017	15:57	83.1	92.0	94.3
08/01/2017	15:58	83.2	91.9	94.0
08/01/2017	15:59	83.1	91.4	92.9
08/01/2017	16:00	83.2	92.1	94.3
08/01/2017	16:01	83.7	91.1	94.3
08/01/2017	16:02	82.6	91.5	94.7
08/01/2017	16:03	82.8	91.4	94.1
08/01/2017	16:04	82.4	91.4	94.4
08/01/2017	16:05	81.5	91.4	94.1
08/01/2017	16:06	82.3	92.0	92.3
08/01/2017	16:07	82.4	91.7	93.3
08/01/2017	16:08	82.7	91.8	94.0
08/01/2017	16:09	82.6	91.2	94.3
08/01/2017	16:10	82.8	91.3	94.1
08/01/2017	16:11	82.7	91.1	93.5
08/01/2017	16:12	82.7	91.7	94.0
08/01/2017	16:13	83.2	91.5	93.9
08/01/2017	16:14	82.3	91.1	93.0
08/01/2017	16:15	83.4	91.6	94.1
08/01/2017	16:16	82.9	91.0	94.2
08/01/2017	16:17	82.5	91.3	93.9
08/01/2017	16:18	82.5	90.9	94.6
08/01/2017	16:19	82.4	91.1	93.9
08/01/2017	16:20	82.2	91.8	93.2
08/01/2017	16:21	82.6	91.3	94.0
08/01/2017	16:22	82.4	91.4	94.7
08/01/2017	16:23	83.2	91.2	93.8
08/01/2017	16:24	83.5	90.1	93.8
08/01/2017	16:25	82.8	91.2	93.4
08/01/2017	16:26	83.3	91.0	94.3
08/01/2017	16:27	82.8	90.3	93.9
08/01/2017	16:28	81.8	91.2	93.9
08/01/2017	16:29	82.5	90.4	92.8
08/01/2017	16:30	82.4	90.3	94.0
08/01/2017	16:31	82.0	91.5	93.9
08/01/2017	16:32	81.8	90.4	94.4
08/01/2017	16:33	81.9	90.8	93.7
08/01/2017	16:34	82.6	90.2	94.1
08/01/2017	16:35	82.2	90.5	94.4
08/01/2017	16:36	81.7	90.4	94.4
08/01/2017	16:37	81.4	90.7	93.4

08/01/2017	16:38	81.6	90.4	93.9
08/01/2017	16:39	82.8	89.9	93.7
08/01/2017	16:40	82.6	89.5	93.3
08/01/2017	16:41	83.5	91.1	93.9
08/01/2017	16:42	82.7	90.2	94.3
08/01/2017	16:43	83.1	90.4	93.2
08/01/2017	16:44	82.9	90.7	92.9
08/01/2017	16:45	83.1	90.7	94.6
08/01/2017	16:46	83.5	91.0	94.6
08/01/2017	16:47	82.9	91.1	93.6
08/01/2017	16:48	83.1	91.1	94.1
08/01/2017	16:49	82.9	90.5	94.8
08/01/2017	16:50	82.9	91.3	93.0
08/01/2017	16:51	83.1	90.7	93.5
08/01/2017	16:52	82.6	91.9	94.1
08/01/2017	16:53	83.0	91.6	94.7
08/01/2017	16:54	83.0	90.7	93.7
08/01/2017	16:55	83.1	91.3	94.0
08/01/2017	16:56	83.1	90.7	93.9
08/01/2017	16:57	83.2	91.6	93.9
08/01/2017	16:58	83.5	91.2	94.1
08/01/2017	16:59	83.2	91.1	95.1
08/01/2017	17:00	83.5	90.8	93.9
08/01/2017	17:01	83.2	90.4	94.9
08/01/2017	17:02	83.2	90.4	93.8
08/01/2017	17:03	82.8	90.8	94.8
08/01/2017	17:04	83.3	90.8	94.3
08/01/2017	17:05	83.2	90.5	94.8
08/01/2017	17:06	83.6	90.8	94.7
08/01/2017	17:07	83.9	90.2	94.5
08/01/2017	17:08	83.9	90.8	93.6
08/01/2017	17:09	83.5	90.0	94.7
08/01/2017	17:10	83.4	90.7	93.8
08/01/2017	17:11	82.3	91.2	94.0
08/01/2017	17:12	81.7	90.1	93.3
08/01/2017	17:13	81.8	89.3	93.9
08/01/2017	17:14	82.1	89.2	94.2
08/01/2017	17:15	81.3	90.1	93.9
08/01/2017	17:16	82.0	89.7	93.6
08/01/2017	17:17	81.8	89.7	93.6
08/01/2017	17:18	82.0	89.9	94.3
08/01/2017	17:19	81.5	89.7	94.3
08/01/2017	17:20	81.0	90.3	94.4
08/01/2017	17:21	81.4	89.7	93.6
08/01/2017	17:22	81.0	89.6	93.5
08/01/2017	17:23	82.0	89.8	94.2
08/01/2017	17:24	80.7	89.8	94.4

08/01/2017	17:25	81.3	89.5	94.0
08/01/2017	17:26	81.4	90.2	93.6
08/01/2017	17:27	81.9	91.0	94.2
08/01/2017	17:28	81.4	90.3	94.0
08/01/2017	17:29	81.6	89.8	95.1
08/01/2017	17:30	81.5	90.3	94.4
08/01/2017	17:31	81.3	90.3	94.1
08/01/2017	17:32	81.8	90.1	95.1
08/01/2017	17:33	82.2	89.8	93.3
08/01/2017	17:34	82.2	89.4	94.7
08/01/2017	17:35	81.6	90.0	94.0
08/01/2017	17:36	81.7	89.6	94.4
08/01/2017	17:37	82.7	89.4	93.6
08/01/2017	17:38	81.7	90.3	93.7
08/01/2017	17:39	81.8	89.6	94.4
08/01/2017	17:40	81.9	89.6	93.5
08/01/2017	17:41	82.8	89.5	93.9
08/01/2017	17:42	82.3	90.1	93.7
08/01/2017	17:43	82.4	90.6	94.0
08/01/2017	17:44	81.9	89.7	94.7
08/01/2017	17:45	82.3	90.1	94.0
08/01/2017	17:46	81.7	89.8	94.5
08/01/2017	17:47	82.4	89.9	94.8
08/01/2017	17:48	82.4	90.2	94.2
08/01/2017	17:49	81.5	89.8	94.2
08/01/2017	17:50	81.7	89.9	94.0
08/01/2017	17:51	82.5	89.9	94.0
08/01/2017	17:52	81.3	90.4	94.8
08/01/2017	17:53	81.1	90.4	94.7
08/01/2017	17:54	82.1	90.1	94.1
08/01/2017	17:55	81.3	90.4	94.0
08/01/2017	17:56	81.1	90.2	94.9
08/01/2017	17:57	81.7	90.1	94.2
08/01/2017	17:58	80.9	89.5	93.9
08/01/2017	17:59	81.1	90.8	94.0
08/01/2017	18:00	80.6	90.3	93.7
08/01/2017	18:01	80.9	89.6	93.7
08/01/2017	18:02	80.7	90.6	94.1
08/01/2017	18:03	80.9	90.9	94.3
08/01/2017	18:04	80.9	90.0	94.4
08/01/2017	18:05	80.9	90.5	93.9
08/01/2017	18:06	80.1	90.8	93.8
08/01/2017	18:07	80.3	89.8	93.6
08/01/2017	18:08	81.1	89.9	93.3
08/01/2017	18:09	80.7	90.5	94.1
08/01/2017	18:10	81.3	89.9	94.3
08/01/2017	18:11	81.3	90.7	94.7

08/01/2017	18:12	80.9	90.3	93.9
08/01/2017	18:13	80.6	90.7	93.9
08/01/2017	18:14	80.6	90.5	94.5
08/01/2017	18:15	80.7	90.1	94.2
08/01/2017	18:16	80.7	90.7	94.2
08/01/2017	18:17	80.8	90.8	94.4
08/01/2017	18:18	80.3	90.3	93.3
08/01/2017	18:19	80.3	90.7	94.1
08/01/2017	18:20	80.7	90.7	94.0
08/01/2017	18:21	79.7	90.9	93.6
08/01/2017	18:22	79.7	90.1	93.9
08/01/2017	18:23	81.0	90.1	93.3
08/01/2017	18:24	80.7	90.9	94.7
08/01/2017	18:25	80.4	90.1	94.7
08/01/2017	18:26	80.4	90.4	94.0
08/01/2017	18:27	80.1	91.5	93.3
08/01/2017	18:28	80.8	90.9	94.5
08/01/2017	18:29	80.4	90.1	93.2
08/01/2017	18:30	80.4	90.7	94.1
08/01/2017	18:31	80.9	90.1	94.1
08/01/2017	18:32	80.3	91.2	93.7
08/01/2017	18:33	80.6	90.3	94.2
08/01/2017	18:34	80.0	90.3	94.1
08/01/2017	18:35	80.4	89.9	94.1
08/01/2017	18:36	80.5	91.5	93.6
08/01/2017	18:37	80.3	90.3	93.5
08/01/2017	18:38	80.3	91.3	93.9
08/01/2017	18:39	80.0	91.0	94.0
08/01/2017	18:40	80.7	90.6	94.3
08/01/2017	18:41	79.7	90.6	94.3
08/01/2017	18:42	80.8	89.8	94.0
08/01/2017	18:43	80.0	90.2	94.5
08/01/2017	18:44	80.3	90.9	94.3
08/01/2017	18:45	81.2	90.6	94.9
08/01/2017	18:46	80.8	90.4	94.7
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08/01/2017	18:48	81.1	91.2	94.3
08/01/2017	18:49	81.6	90.1	93.5
08/01/2017	18:50	80.8	91.6	94.4
08/01/2017	18:51	80.8	90.6	94.7
08/01/2017	18:52	81.0	90.8	94.6
08/01/2017	18:53	81.1	90.9	94.0
08/01/2017	18:54	80.1	91.2	94.7
08/01/2017	18:55	80.6	91.5	93.7
08/01/2017	18:56	80.5	90.9	93.9
08/01/2017	18:57	80.3	92.1	94.4
08/01/2017	18:58	80.4	90.9	94.3

08/01/2017	18:59	79.8	91.7	94.4
08/01/2017	19:00	79.3	91.0	94.0
08/01/2017	19:01	80.0	91.7	94.3
08/01/2017	19:02	79.9	91.2	93.7
08/01/2017	19:03	79.3	91.3	94.6
08/01/2017	19:04	78.7	91.9	93.0
08/01/2017	19:05	79.2	90.8	94.7
08/01/2017	19:06	79.0	91.6	94.6
08/01/2017	19:07	80.4	91.0	93.9
08/01/2017	19:08	80.0	91.2	94.4
08/01/2017	19:09	79.7	91.3	94.5
08/01/2017	19:10	80.1	91.5	94.1
08/01/2017	19:11	80.4	91.0	94.7
08/01/2017	19:12	79.3	91.5	94.6
08/01/2017	19:13	79.8	90.9	94.6
08/01/2017	19:14	79.8	92.1	93.9
08/01/2017	19:15	80.8	90.8	94.3
08/01/2017	19:16	78.7	91.6	95.1
08/01/2017	19:17	80.1	91.5	94.2
08/01/2017	19:18	79.6	91.9	93.9
08/01/2017	19:19	79.8	90.8	94.4
08/01/2017	19:20	79.6	90.7	94.3
08/01/2017	19:21	81.0	91.6	92.9
08/01/2017	19:22	81.7	92.0	95.0
08/01/2017	19:23	81.8	91.6	94.4
08/01/2017	19:24	82.3	91.8	94.5
08/01/2017	19:25	81.9	91.9	93.3
08/01/2017	19:26	81.0	90.7	94.2
08/01/2017	19:27	80.7	92.3	93.5
08/01/2017	19:28	80.9	92.2	93.9
08/01/2017	19:29	79.5	90.4	94.2
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08/01/2017	19:31	80.8	91.8	93.0
08/01/2017	19:32	80.2	90.9	94.4
08/01/2017	19:33	80.0	91.4	94.4
08/01/2017	19:34	80.5	91.5	92.3
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08/01/2017	19:37	81.3	91.5	94.0
08/01/2017	19:38	81.4	92.2	94.0
08/01/2017	19:39	81.7	91.9	94.4
08/01/2017	19:40	80.3	91.7	94.4
08/01/2017	19:41	81.1	91.3	93.9
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08/01/2017	19:43	81.3	92.8	93.8
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08/01/2017	19:51	81.3	92.0	94.0
08/01/2017	19:52	81.0	91.5	94.3
08/01/2017	19:53	81.6	91.8	93.3
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08/01/2017	20:00	80.3	92.0	94.7
08/01/2017	20:01	79.3	90.7	94.4
08/01/2017	20:02	80.4	91.6	94.0
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08/01/2017	20:04	78.6	91.6	93.9
08/01/2017	20:05	79.1	91.1	94.3
08/01/2017	20:06	78.9	91.3	93.5
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08/01/2017	20:09	81.2	91.2	94.3
08/01/2017	20:10	79.3	91.7	93.9
08/01/2017	20:11	80.8	91.2	93.6
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08/01/2017	20:13	78.8	92.4	95.1
08/01/2017	20:14	79.8	91.5	93.9
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08/01/2017	20:18	79.7	91.9	94.4
08/01/2017	20:19	80.0	92.0	94.9
08/01/2017	20:20	80.8	92.4	93.9
08/01/2017	20:21	80.8	92.3	94.7
08/01/2017	20:22	80.9	93.2	94.3
08/01/2017	20:23	81.2	91.5	94.3
08/01/2017	20:24	81.7	92.4	94.7
08/01/2017	20:25	80.4	91.9	94.6
08/01/2017	20:26	80.1	92.4	94.2
08/01/2017	20:27	81.2	92.6	94.2
08/01/2017	20:28	80.9	91.9	94.0
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08/01/2017	20:31	79.2	91.5	94.2
08/01/2017	20:32	81.4	92.0	94.0

08/01/2017	20:33	79.9	91.4	94.0
08/01/2017	20:34	79.4	90.8	93.7
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08/01/2017	20:39	79.9	90.9	94.1
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08/01/2017	20:42	79.3	91.0	94.5
08/01/2017	20:43	79.6	92.3	94.9
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08/01/2017	20:47	82.4	90.6	94.0
08/01/2017	20:48	83.1	91.4	94.4
08/01/2017	20:49	82.7	91.8	94.4
08/01/2017	20:50	82.7	91.5	93.5
08/01/2017	20:51	83.0	90.8	94.1
08/01/2017	20:52	82.0	90.9	94.5
08/01/2017	20:53	81.9	91.4	93.6
08/01/2017	20:54	82.8	90.9	93.9
08/01/2017	20:55	82.1	91.8	94.2
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08/01/2017	20:57	81.4	91.6	94.1
08/01/2017	20:58	81.7	91.9	94.3
08/01/2017	20:59	81.9	91.8	94.2
08/01/2017	21:00	80.9	90.1	94.3
08/01/2017	21:01	80.4	91.9	93.2
08/01/2017	21:02	79.4	90.6	93.6
08/01/2017	21:03	79.9	92.3	92.7
08/01/2017	21:04	79.0	91.9	93.6
08/01/2017	21:05	78.9	91.8	93.6
08/01/2017	21:06	79.2	91.6	92.8
08/01/2017	21:07	80.1	91.3	92.8
08/01/2017	21:08	80.3	91.7	94.2
08/01/2017	21:09	80.2	90.4	93.2
08/01/2017	21:10	81.1	91.4	93.3
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08/01/2017	21:12	79.9	90.8	93.0
08/01/2017	21:13	81.8	90.8	93.1
08/01/2017	21:14	80.9	91.7	93.3
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08/01/2017	21:16	81.8	91.8	93.9
08/01/2017	21:17	80.8	91.3	93.9
08/01/2017	21:18	80.0	90.8	94.0
08/01/2017	21:19	80.4	92.3	93.7

08/01/2017	21:20	81.1	91.9	93.2
08/01/2017	21:21	79.6	91.8	93.6
08/01/2017	21:22	81.1	91.3	93.3
08/01/2017	21:23	82.0	91.7	93.3
08/01/2017	21:24	80.1	90.5	94.2
08/01/2017	21:25	80.6	91.6	93.3
08/01/2017	21:26	80.1	91.3	92.8
08/01/2017	21:27	80.8	92.5	92.7
08/01/2017	21:28	82.8	91.7	93.1
08/01/2017	21:29	82.0	91.5	92.6
08/01/2017	21:30	81.6	91.7	93.6
08/01/2017	21:31	81.4	91.9	93.1
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08/01/2017	21:34	81.3	91.6	93.5
08/01/2017	21:35	82.2	91.4	92.5
08/01/2017	21:36	81.8	91.3	93.8
08/01/2017	21:37	82.1	91.2	93.3
08/01/2017	21:38	81.0	91.6	93.0
08/01/2017	21:39	81.8	92.0	93.4
08/01/2017	21:40	81.8	91.5	94.1
08/01/2017	21:41	84.0	92.2	93.3
08/01/2017	21:42	83.3	91.7	92.8
08/01/2017	21:43	83.4	91.9	92.7
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08/01/2017	21:45	83.8	92.3	93.6
08/01/2017	21:46	83.9	91.8	94.0
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08/01/2017	21:48	83.9	92.3	92.9
08/01/2017	21:49	84.3	91.8	93.7
08/01/2017	21:50	83.8	92.3	93.9
08/01/2017	21:51	84.1	93.3	93.5
08/01/2017	21:52	84.2	93.1	93.6
08/01/2017	21:53	84.4	92.4	92.0
08/01/2017	21:54	84.6	92.6	94.1
08/01/2017	21:55	83.7	93.0	93.0
08/01/2017	21:56	84.0	92.7	93.6
08/01/2017	21:57	83.7	92.7	93.6
08/01/2017	21:58	83.7	92.4	93.0
08/01/2017	21:59	83.7	92.8	90.9
08/01/2017	22:00	82.8	93.1	92.7
08/01/2017	22:01	83.4	92.4	91.6
08/01/2017	22:02	83.7	91.5	92.9
08/01/2017	22:03	82.6	93.0	92.2
08/01/2017	22:04	83.1	93.2	93.1
08/01/2017	22:05	83.0	92.4	92.9
08/01/2017	22:06	83.0	92.4	93.6

08/01/2017	22:07	83.6	92.7	93.0
08/01/2017	22:08	83.3	92.9	91.8
08/01/2017	22:09	83.4	93.1	92.5
08/01/2017	22:10	83.1	92.7	91.6
08/01/2017	22:11	83.7	92.2	92.0
08/01/2017	22:12	83.4	92.8	92.6
08/01/2017	22:13	82.7	92.7	93.3
08/01/2017	22:14	83.2	92.7	91.6
08/01/2017	22:15	83.6	92.5	92.1
08/01/2017	22:16	83.1	93.4	92.6
08/01/2017	22:17	83.3	92.4	92.6
08/01/2017	22:18	83.5	92.6	92.9
08/01/2017	22:19	83.2	92.1	93.2
08/01/2017	22:20	82.4	92.4	92.0
08/01/2017	22:21	83.2	92.9	93.6
08/01/2017	22:22	83.1	93.0	93.0
08/01/2017	22:23	83.1	92.4	93.3
08/01/2017	22:24	83.1	92.8	93.0
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08/01/2017	22:26	84.6	92.9	93.3
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08/01/2017	22:28	84.0	92.8	92.5
08/01/2017	22:29	84.2	92.5	93.3
08/01/2017	22:30	83.5	93.6	93.5
08/01/2017	22:31	83.4	92.3	92.9
08/01/2017	22:32	82.4	93.0	92.4
08/01/2017	22:33	83.8	92.7	93.5
08/01/2017	22:34	84.1	92.9	92.9
08/01/2017	22:35	84.0	92.7	92.7
08/01/2017	22:36	84.2	92.0	93.5
08/01/2017	22:37	83.9	93.1	92.5
08/01/2017	22:38	83.1	93.0	93.3
08/01/2017	22:39	83.5	92.7	92.8
08/01/2017	22:40	82.8	92.4	94.4
08/01/2017	22:41	83.5	92.8	93.3
08/01/2017	22:42	83.8	92.7	93.7
08/01/2017	22:43	83.4	92.4	93.6
08/01/2017	22:44	83.4	92.7	93.6
08/01/2017	22:45	83.3	92.4	94.4
08/01/2017	22:46	83.1	92.7	93.8
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08/01/2017	22:48	83.6	92.8	93.3
08/01/2017	22:49	83.5	93.1	93.8
08/01/2017	22:50	83.3	92.8	94.0
08/01/2017	22:51	83.9	92.9	93.4
08/01/2017	22:52	83.4	92.6	94.0
08/01/2017	22:53	83.8	92.8	93.7

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08/01/2017	22:55	84.0	93.3	94.1
08/01/2017	22:56	82.5	93.0	95.1
08/01/2017	22:57	84.5	92.8	93.9
08/01/2017	22:58	81.3	93.1	94.1
08/01/2017	22:59	82.3	92.8	90.9
08/01/2017	23:00	82.8	93.0	94.0
08/01/2017	23:01	81.4	93.2	93.0
08/01/2017	23:02	82.2	93.0	93.0
08/01/2017	23:03	83.2	92.8	93.9
08/01/2017	23:04	83.5	93.2	93.5
08/01/2017	23:05	82.4	92.9	93.3
08/01/2017	23:06	82.9	92.7	92.6
08/01/2017	23:07	83.5	92.6	92.8
08/01/2017	23:08	82.7	92.8	92.8
08/01/2017	23:09	82.8	93.1	93.1
08/01/2017	23:10	82.4	92.7	93.1
08/01/2017	23:11	84.6	92.8	93.3
08/01/2017	23:12	83.1	93.5	93.3
08/01/2017	23:13	82.7	92.4	93.4
08/01/2017	23:14	83.9	92.0	92.6
08/01/2017	23:15	82.8	92.8	93.4
08/01/2017	23:16	83.1	93.2	92.9
08/01/2017	23:17	82.7	93.1	93.7
08/01/2017	23:18	83.1	93.1	93.2
08/01/2017	23:19	82.9	93.1	93.7
08/01/2017	23:20	83.7	92.9	93.0
08/01/2017	23:21	84.0	92.7	93.9
08/01/2017	23:22	83.4	92.8	94.0
08/01/2017	23:23	83.2	93.0	94.2
08/01/2017	23:24	83.8	93.4	93.6
08/01/2017	23:25	83.6	92.6	93.5
08/01/2017	23:26	84.0	93.1	92.8
08/01/2017	23:27	83.3	93.1	93.0
08/01/2017	23:28	83.3	93.0	93.1
08/01/2017	23:29	84.2	93.0	94.3
08/01/2017	23:30	84.0	92.5	93.2
08/01/2017	23:31	84.4	93.6	93.2
08/01/2017	23:32	84.3	93.1	93.3
08/01/2017	23:33	83.9	92.4	93.5
08/01/2017	23:34	83.7	93.1	92.8
08/01/2017	23:35	84.3	93.9	93.3
08/01/2017	23:36	84.6	93.0	93.0
08/01/2017	23:37	84.3	92.8	93.3
08/01/2017	23:38	83.2	93.0	93.7
08/01/2017	23:39	83.6	93.1	92.6
08/01/2017	23:40	85.9	93.0	93.5

08/01/2017	23:41	84.9	93.9	93.5
08/01/2017	23:42	84.3	93.0	93.3
08/01/2017	23:43	84.8	93.2	93.3
08/01/2017	23:44	85.7	93.2	93.9
08/01/2017	23:45	85.3	93.2	94.2
08/01/2017	23:46	85.2	92.7	93.9
08/01/2017	23:47	85.6	92.5	94.5
08/01/2017	23:48	84.8	93.0	94.7
08/01/2017	23:49	84.6	93.1	94.0
08/01/2017	23:50	84.5	92.8	94.9
08/01/2017	23:51	85.5	92.1	94.3
08/01/2017	23:52	84.9	93.2	94.6
08/01/2017	23:53	84.4	93.2	94.1
08/01/2017	23:54	84.6	93.0	93.7
08/01/2017	23:55	84.4	93.3	93.6
08/01/2017	23:56	85.2	93.8	94.1
08/01/2017	23:57	84.4	93.5	93.7
08/01/2017	23:58	85.2	93.6	93.8
08/01/2017	23:59	84.3	93.1	94.0
08/02/2017	00:00	83.0	93.2	93.3
08/02/2017	00:01	83.8	93.9	92.8
08/02/2017	00:02	83.9	93.0	93.3
08/02/2017	00:03	83.7	93.3	94.4
08/02/2017	00:04	84.3	93.0	92.6
08/02/2017	00:05	83.3	93.0	92.6
08/02/2017	00:06	83.3	93.1	93.1
08/02/2017	00:07	82.7	93.5	92.9
08/02/2017	00:08	82.9	93.4	93.4
08/02/2017	00:09	82.3	93.0	93.2
08/02/2017	00:10	82.4	93.4	93.2
08/02/2017	00:11	81.4	93.5	93.5
08/02/2017	00:12	82.6	93.1	93.2
08/02/2017	00:13	82.0	92.9	93.3
08/02/2017	00:14	81.1	92.3	93.0
08/02/2017	00:15	82.2	93.4	93.3
08/02/2017	00:16	82.0	93.3	93.4
08/02/2017	00:17	81.7	93.2	93.5
08/02/2017	00:18	81.6	93.1	92.6
08/02/2017	00:19	82.3	92.1	93.7
08/02/2017	00:20	82.2	93.5	93.7
08/02/2017	00:21	83.1	93.3	93.3
08/02/2017	00:22	82.3	93.3	92.6
08/02/2017	00:23	82.0	93.0	94.0
08/02/2017	00:24	81.9	93.5	93.6
08/02/2017	00:25	82.2	93.1	93.8
08/02/2017	00:26	81.3	93.1	93.7
08/02/2017	00:27	81.9	93.2	93.0

08/02/2017	00:28	81.7	93.6	92.9
08/02/2017	00:29	82.0	93.6	94.7
08/02/2017	00:30	81.5	94.1	92.8
08/02/2017	00:31	81.6	93.1	93.3
08/02/2017	00:32	81.6	93.4	93.3
08/02/2017	00:33	82.2	92.9	91.5
08/02/2017	00:34	81.7	92.8	92.9
08/02/2017	00:35	82.2	93.1	93.7
08/02/2017	00:36	81.8	93.5	93.8
08/02/2017	00:37	83.0	93.4	92.9
08/02/2017	00:38	82.2	93.1	94.8
08/02/2017	00:39	82.5	93.1	92.2
08/02/2017	00:40	82.4	93.5	93.3
08/02/2017	00:41	81.8	92.9	93.3
08/02/2017	00:42	82.4	93.5	93.7
08/02/2017	00:43	82.6	92.8	93.7
08/02/2017	00:44	82.0	93.5	93.5
08/02/2017	00:45	82.8	93.0	93.4
08/02/2017	00:46	81.7	93.6	93.3
08/02/2017	00:47	81.8	93.9	94.1
08/02/2017	00:48	81.3	93.3	93.7
08/02/2017	00:49	81.8	93.3	94.2
08/02/2017	00:50	82.1	93.3	93.2
08/02/2017	00:51	82.0	92.7	91.9
08/02/2017	00:52	82.0	93.0	93.9
08/02/2017	00:53	82.4	93.4	94.1
08/02/2017	00:54	82.8	93.4	93.9
08/02/2017	00:55	81.7	93.3	93.2
08/02/2017	00:56	82.1	93.3	93.5
08/02/2017	00:57	82.6	93.3	94.0
08/02/2017	00:58	82.4	93.8	93.9
08/02/2017	00:59	82.9	93.1	94.0
08/02/2017	01:00	82.2	93.0	93.7
08/02/2017	01:01	81.9	92.7	92.1
08/02/2017	01:02	83.2	92.9	92.2
08/02/2017	01:03	83.3	93.1	93.2
08/02/2017	01:04	82.1	93.2	93.3
08/02/2017	01:05	83.5	93.0	92.2
08/02/2017	01:06	83.3	93.2	92.7
08/02/2017	01:07	83.1	93.7	93.6
08/02/2017	01:08	83.8	93.3	93.2
08/02/2017	01:09	84.5	93.3	93.4
08/02/2017	01:10	84.1	93.4	93.3
08/02/2017	01:11	84.3	93.0	93.2
08/02/2017	01:12	84.3	93.3	93.3
08/02/2017	01:13	84.4	93.0	93.7
08/02/2017	01:14	84.7	93.0	93.2

08/02/2017	01:15	84.7	92.9	92.9
08/02/2017	01:16	84.7	93.1	92.3
08/02/2017	01:17	84.4	93.3	94.0
08/02/2017	01:18	84.3	93.3	93.5
08/02/2017	01:19	84.6	93.0	92.9
08/02/2017	01:20	84.3	93.2	93.4
08/02/2017	01:21	84.9	93.3	92.4
08/02/2017	01:22	84.7	92.1	93.3
08/02/2017	01:23	83.7	93.1	93.5
08/02/2017	01:24	84.5	92.8	93.5
08/02/2017	01:25	84.7	92.9	94.3
08/02/2017	01:26	84.6	93.4	93.3
08/02/2017	01:27	85.0	93.1	91.2
08/02/2017	01:28	84.5	93.7	93.0
08/02/2017	01:29	84.6	93.0	93.7
08/02/2017	01:30	84.4	93.0	93.9
08/02/2017	01:31	85.0	93.4	93.7
08/02/2017	01:32	85.8	93.1	94.9
08/02/2017	01:33	84.9	93.1	93.7
08/02/2017	01:34	85.1	93.1	93.6
08/02/2017	01:35	86.0	93.2	93.5
08/02/2017	01:36	84.8	93.4	93.8
08/02/2017	01:37	85.1	94.0	94.0
08/02/2017	01:38	84.7	93.1	94.7
08/02/2017	01:39	84.5	93.0	95.0
08/02/2017	01:40	85.7	93.3	93.9
08/02/2017	01:41	84.5	93.6	94.3
08/02/2017	01:42	85.2	93.2	93.6
08/02/2017	01:43	85.8	93.4	94.0
08/02/2017	01:44	84.9	93.4	94.7
08/02/2017	01:45	84.7	93.3	92.3
08/02/2017	01:46	85.0	93.6	93.6
08/02/2017	01:47	85.4	93.7	93.8
08/02/2017	01:48	85.0	93.3	94.0
08/02/2017	01:49	84.9	93.3	93.5
08/02/2017	01:50	85.1	93.7	94.0
08/02/2017	01:51	85.3	93.6	94.1
08/02/2017	01:52	84.3	93.9	93.7
08/02/2017	01:53	84.8	93.4	93.8
08/02/2017	01:54	85.1	93.3	94.1
08/02/2017	01:55	84.9	93.0	94.4
08/02/2017	01:56	84.5	93.5	94.6
08/02/2017	01:57	84.0	93.1	94.0
08/02/2017	01:58	85.0	94.0	93.7
08/02/2017	01:59	84.3	93.2	94.3
08/02/2017	02:00	84.4	93.4	93.3
08/02/2017	02:01	84.6	92.3	93.5

08/02/2017	02:02	84.4	93.5	94.2
08/02/2017	02:03	84.4	93.2	93.3
08/02/2017	02:04	84.1	93.2	94.3
08/02/2017	02:05	83.8	93.3	93.7
08/02/2017	02:06	84.5	93.3	92.8
08/02/2017	02:07	84.0	93.4	92.7
08/02/2017	02:08	84.9	93.6	94.0
08/02/2017	02:09	84.2	93.0	93.6
08/02/2017	02:10	84.3	93.5	92.6
08/02/2017	02:11	84.9	92.2	93.0
08/02/2017	02:12	83.8	93.3	93.6
08/02/2017	02:13	83.7	93.2	92.8
08/02/2017	02:14	83.1	93.4	93.4
08/02/2017	02:15	83.0	93.4	93.9
08/02/2017	02:16	83.3	92.6	93.6
08/02/2017	02:17	83.7	93.4	90.1
08/02/2017	02:18	83.5	93.3	94.0
08/02/2017	02:19	84.1	93.4	93.6
08/02/2017	02:20	83.6	92.7	93.7
08/02/2017	02:21	84.0	93.2	94.4
08/02/2017	02:22	84.3	93.1	93.5
08/02/2017	02:23	83.5	93.9	93.6
08/02/2017	02:24	84.0	93.3	94.1
08/02/2017	02:25	84.2	93.5	94.1
08/02/2017	02:26	83.4	93.4	94.6
08/02/2017	02:27	83.9	93.6	93.7
08/02/2017	02:28	83.4	93.6	94.2
08/02/2017	02:29	83.9	93.5	94.0
08/02/2017	02:30	83.8	93.4	93.7
08/02/2017	02:31	84.4	92.5	91.8
08/02/2017	02:32	83.2	93.6	93.2
08/02/2017	02:33	84.0	93.1	93.0
08/02/2017	02:34	83.5	93.4	92.6
08/02/2017	02:35	83.7	93.2	93.4
08/02/2017	02:36	84.3	93.4	92.5
08/02/2017	02:37	84.4	93.5	92.2
08/02/2017	02:38	83.8	93.0	94.3
08/02/2017	02:39	84.1	93.3	92.9
08/02/2017	02:40	83.8	93.1	93.0
08/02/2017	02:41	83.5	93.0	93.2
08/02/2017	02:42	84.2	93.1	93.5
08/02/2017	02:43	83.9	93.4	91.5
08/02/2017	02:44	83.5	93.2	93.3
08/02/2017	02:45	83.2	92.9	93.3
08/02/2017	02:46	82.9	93.1	93.2
08/02/2017	02:47	83.0	93.1	93.6
08/02/2017	02:48	82.6	94.2	93.8

08/02/2017	02:49	83.0	94.1	94.0
08/02/2017	02:50	83.3	93.3	93.7
08/02/2017	02:51	82.9	93.7	94.2
08/02/2017	02:52	83.1	93.4	93.7
08/02/2017	02:53	82.5	93.3	93.8
08/02/2017	02:54	82.9	93.1	94.3
08/02/2017	02:55	82.8	93.3	94.1
08/02/2017	02:56	82.6	93.3	94.2
08/02/2017	02:57	82.6	93.5	93.8
08/02/2017	02:58	82.4	93.3	94.0
08/02/2017	02:59	82.6	93.7	93.6
08/02/2017	03:00	81.4	93.4	94.4
08/02/2017	03:01	81.8	93.3	93.5
08/02/2017	03:02	80.9	93.3	91.9
08/02/2017	03:03	80.1	93.2	93.7
08/02/2017	03:04	79.8	92.9	93.5
08/02/2017	03:05	80.2	93.3	92.4
08/02/2017	03:06	80.4	93.6	93.5
08/02/2017	03:07	80.0	93.4	93.5
08/02/2017	03:08	80.9	93.3	92.8
08/02/2017	03:09	79.8	93.4	92.4
08/02/2017	03:10	80.1	92.9	93.0
08/02/2017	03:11	80.0	93.5	93.2
08/02/2017	03:12	79.9	93.6	92.6
08/02/2017	03:13	79.7	93.1	92.9
08/02/2017	03:14	80.1	92.9	93.5
08/02/2017	03:15	79.0	93.4	93.6
08/02/2017	03:16	79.3	93.5	92.6
08/02/2017	03:17	80.1	93.3	93.1
08/02/2017	03:18	79.2	93.2	93.8
08/02/2017	03:19	79.6	93.3	93.5
08/02/2017	03:20	81.7	93.5	93.5
08/02/2017	03:21	80.4	92.5	92.6
08/02/2017	03:22	79.5	93.3	93.2
08/02/2017	03:23	80.7	93.4	93.8
08/02/2017	03:24	79.6	93.6	93.6
08/02/2017	03:25	80.3	93.8	93.7
08/02/2017	03:26	80.3	93.6	92.9
08/02/2017	03:27	79.8	93.5	92.9
08/02/2017	03:28	79.4	93.6	94.0
08/02/2017	03:29	79.7	93.6	94.4
08/02/2017	03:30	79.8	93.4	93.6
08/02/2017	03:31	82.7	93.5	93.1
08/02/2017	03:32	82.8	93.1	93.3
08/02/2017	03:33	82.5	93.9	93.2
08/02/2017	03:34	81.9	93.4	93.6
08/02/2017	03:35	82.4	93.6	93.8

08/02/2017	03:36	82.2	94.3	92.3
08/02/2017	03:37	83.1	93.3	93.3
08/02/2017	03:38	82.7	93.0	94.4
08/02/2017	03:39	83.0	93.9	94.0
08/02/2017	03:40	83.2	93.2	93.4
08/02/2017	03:41	83.4	93.3	94.6
08/02/2017	03:42	82.9	93.5	94.3
08/02/2017	03:43	83.3	93.3	93.5
08/02/2017	03:44	84.1	93.2	92.4
08/02/2017	03:45	82.5	93.8	92.2
08/02/2017	03:46	83.7	93.3	93.0
08/02/2017	03:47	83.9	93.5	93.9
08/02/2017	03:48	83.8	93.5	93.3
08/02/2017	03:49	83.8	93.7	92.6
08/02/2017	03:50	82.4	93.0	93.3
08/02/2017	03:51	82.8	93.6	92.6
08/02/2017	03:52	84.0	93.6	92.8
08/02/2017	03:53	83.2	93.6	93.1
08/02/2017	03:54	83.1	94.3	93.2
08/02/2017	03:55	83.8	92.9	94.3
08/02/2017	03:56	83.5	93.3	93.6
08/02/2017	03:57	82.3	93.4	93.3
08/02/2017	03:58	83.0	94.0	92.5
08/02/2017	03:59	82.0	93.8	94.5
08/02/2017	04:00	81.8	93.5	92.9
08/02/2017	04:01	81.0	93.3	93.3
08/02/2017	04:02	79.8	94.7	93.0
08/02/2017	04:03	80.5	94.3	92.6
08/02/2017	04:04	80.2	93.6	92.8
08/02/2017	04:05	79.5	93.3	93.2
08/02/2017	04:06	79.3	93.6	93.0
08/02/2017	04:07	79.6	93.5	92.8
08/02/2017	04:08	80.2	94.5	93.5
08/02/2017	04:09	80.6	93.6	93.1
08/02/2017	04:10	80.7	93.3	93.3
08/02/2017	04:11	80.5	93.6	93.7
08/02/2017	04:12	80.1	92.6	93.7
08/02/2017	04:13	81.2	93.1	93.0
08/02/2017	04:14	79.6	93.6	93.7
08/02/2017	04:15	81.4	93.4	93.9
08/02/2017	04:16	81.1	93.6	93.7
08/02/2017	04:17	80.9	92.8	93.5
08/02/2017	04:18	79.0	93.3	94.1
08/02/2017	04:19	79.9	92.6	93.7
08/02/2017	04:20	79.4	93.2	93.8
08/02/2017	04:21	80.4	93.2	93.5
08/02/2017	04:22	80.7	92.8	93.6

08/02/2017	04:23	81.2	93.5	93.8
08/02/2017	04:24	81.6	93.6	94.3
08/02/2017	04:25	79.7	93.8	93.7
08/02/2017	04:26	81.0	93.5	95.1
08/02/2017	04:27	81.2	93.5	94.4
08/02/2017	04:28	81.4	93.4	94.0
08/02/2017	04:29	79.2	93.8	94.5
08/02/2017	04:30	80.5	93.6	94.0
08/02/2017	04:31	79.9	93.5	93.8
08/02/2017	04:32	80.2	94.1	93.7
08/02/2017	04:33	79.6	93.0	91.9
08/02/2017	04:34	80.5	93.5	94.2
08/02/2017	04:35	81.4	93.1	94.0
08/02/2017	04:36	80.9	93.0	94.0
08/02/2017	04:37	80.0	93.8	94.2
08/02/2017	04:38	80.5	93.8	93.5
08/02/2017	04:39	80.9	93.6	92.3
08/02/2017	04:40	81.2	93.7	93.6
08/02/2017	04:41	81.3	93.0	94.6
08/02/2017	04:42	79.9	92.5	93.9
08/02/2017	04:43	79.9	93.1	93.6
08/02/2017	04:44	80.7	93.3	94.2
08/02/2017	04:45	79.9	92.8	93.6
08/02/2017	04:46	79.9	94.7	93.3
08/02/2017	04:47	80.4	93.3	93.7
08/02/2017	04:48	80.2	92.7	93.7
08/02/2017	04:49	80.8	93.5	93.9
08/02/2017	04:50	80.9	94.4	94.1
08/02/2017	04:51	80.3	93.2	92.9
08/02/2017	04:52	80.1	93.7	93.9
08/02/2017	04:53	80.3	93.2	93.6
08/02/2017	04:54	80.9	93.2	93.7
08/02/2017	04:55	81.0	92.4	93.8
08/02/2017	04:56	79.6	91.9	94.3
08/02/2017	04:57	80.7	92.8	92.9
08/02/2017	04:58	81.1	93.1	93.5
08/02/2017	04:59	80.4	93.6	94.1
08/02/2017	05:00	78.6	92.8	92.5
08/02/2017	05:01	78.2	92.9	92.3
08/02/2017	05:02	80.4	93.6	93.4
08/02/2017	05:03	77.6	92.9	93.4
08/02/2017	05:04	76.8	92.3	93.2
08/02/2017	05:05	78.1	92.3	94.0
08/02/2017	05:06	77.8	93.4	93.6
08/02/2017	05:07	78.0	93.9	94.0
08/02/2017	05:08	78.5	93.2	93.7
08/02/2017	05:09	79.4	94.0	93.5

08/02/2017	05:10	78.0	91.8	93.8
08/02/2017	05:11	80.1	93.1	93.7
08/02/2017	05:12	78.8	92.5	93.7
08/02/2017	05:13	79.6	93.3	92.6
08/02/2017	05:14	78.1	92.7	94.2
08/02/2017	05:15	78.2	93.8	93.7
08/02/2017	05:16	79.2	93.3	93.3
08/02/2017	05:17	79.8	93.7	93.9
08/02/2017	05:18	78.2	92.9	93.9
08/02/2017	05:19	79.0	92.9	94.0
08/02/2017	05:20	77.3	92.2	93.4
08/02/2017	05:21	78.2	92.1	93.7
08/02/2017	05:22	75.8	92.3	93.6
08/02/2017	05:23	76.1	91.9	93.9
08/02/2017	05:24	76.6	92.0	94.3
08/02/2017	05:25	77.0	93.7	94.4
08/02/2017	05:26	76.1	93.1	94.7
08/02/2017	05:27	77.9	93.3	91.6
08/02/2017	05:28	77.0	93.2	94.3
08/02/2017	05:29	76.1	91.9	94.1
08/02/2017	05:30	76.9	92.2	94.8
08/02/2017	05:31	77.1	92.9	94.1
08/02/2017	05:32	79.3	92.6	94.9
08/02/2017	05:33	75.5	92.5	93.9
08/02/2017	05:34	74.8	93.4	93.7
08/02/2017	05:35	76.0	92.9	93.5
08/02/2017	05:36	76.3	92.6	93.8
08/02/2017	05:37	76.7	92.4	93.5
08/02/2017	05:38	75.4	92.7	93.8
08/02/2017	05:39	74.5	91.7	94.0
08/02/2017	05:40	77.8	93.6	94.3
08/02/2017	05:41	77.2	92.5	94.6
08/02/2017	05:42	76.6	92.6	93.5
08/02/2017	05:43	76.2	92.7	92.8
08/02/2017	05:44	80.0	93.2	93.6
08/02/2017	05:45	81.8	93.3	93.2
08/02/2017	05:46	81.1	93.0	92.7
08/02/2017	05:47	80.2	93.5	92.9
08/02/2017	05:48	82.2	92.4	92.9
08/02/2017	05:49	81.1	94.0	93.6
08/02/2017	05:50	80.5	93.5	93.4
08/02/2017	05:51	80.9	92.7	93.5
08/02/2017	05:52	80.4	93.4	91.6
08/02/2017	05:53	82.4	92.7	92.5
08/02/2017	05:54	80.8	93.1	92.8
08/02/2017	05:55	80.9	93.4	93.2
08/02/2017	05:56	81.0	93.2	93.6

08/02/2017	05:57	79.9	92.6	93.5
08/02/2017	05:58	81.1	93.1	94.0
08/02/2017	05:59	80.3	92.5	93.1
08/02/2017	06:00	80.4	93.2	94.4
08/02/2017	06:01	78.3	92.6	92.6
08/02/2017	06:02	78.5	93.3	92.6
08/02/2017	06:03	78.1	92.0	92.6
08/02/2017	06:04	78.3	91.7	93.0
08/02/2017	06:05	79.7	92.2	93.5
08/02/2017	06:06	77.9	93.7	93.3
08/02/2017	06:07	77.7	92.6	93.7
08/02/2017	06:08	77.7	93.4	93.2
08/02/2017	06:09	77.7	91.7	93.8
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08/02/2017	06:11	79.5	93.0	93.8
08/02/2017	06:12	79.0	93.1	93.0
08/02/2017	06:13	78.1	92.7	93.3
08/02/2017	06:14	76.6	93.8	93.7
08/02/2017	06:15	79.8	93.1	93.6
08/02/2017	06:16	79.0	93.1	93.7
08/02/2017	06:17	79.1	93.2	90.0
08/02/2017	06:18	80.5	92.8	93.0
08/02/2017	06:19	79.3	92.7	93.5
08/02/2017	06:20	78.9	92.1	93.7
08/02/2017	06:21	78.7	93.5	93.8
08/02/2017	06:22	79.3	94.5	93.8
08/02/2017	06:23	79.9	93.0	94.1
08/02/2017	06:24	79.2	92.1	94.3
08/02/2017	06:25	79.2	93.8	94.1
08/02/2017	06:26	81.3	91.8	92.7
08/02/2017	06:27	80.7	93.5	93.9
08/02/2017	06:28	80.4	93.7	92.9
08/02/2017	06:29	79.2	93.1	93.9
08/02/2017	06:30	80.0	93.0	93.0
08/02/2017	06:31	77.8	93.2	93.0
08/02/2017	06:32	78.2	91.4	94.5
08/02/2017	06:33	78.5	92.8	94.0
08/02/2017	06:34	78.8	93.1	94.2
08/02/2017	06:35	78.5	92.6	93.3
08/02/2017	06:36	80.6	92.8	93.9
08/02/2017	06:37	81.4	91.9	92.8
08/02/2017	06:38	78.4	93.7	94.1
08/02/2017	06:39	81.0	92.8	93.9
08/02/2017	06:40	80.4	92.8	94.3
08/02/2017	06:41	80.6	93.2	93.8
08/02/2017	06:42	81.0	92.0	94.6
08/02/2017	06:43	81.6	93.3	92.6

08/02/2017	06:44	80.4	92.7	94.1
08/02/2017	06:45	78.8	92.3	94.7
08/02/2017	06:46	80.5	93.3	95.1
08/02/2017	06:47	81.4	93.8	94.0
08/02/2017	06:48	80.4	91.8	94.4
08/02/2017	06:49	80.4	91.9	94.1
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08/02/2017	06:51	81.2	92.4	94.9
08/02/2017	06:52	81.7	93.1	94.0
08/02/2017	06:53	81.8	92.9	94.9
08/02/2017	06:54	80.8	92.4	94.6
08/02/2017	06:55	81.5	93.8	93.7
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08/02/2017	06:57	81.4	93.3	94.5
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08/02/2017	07:13	77.1	92.7	94.0
08/02/2017	07:14	77.3	91.9	93.0
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08/02/2017	07:16	79.8	92.7	93.4
08/02/2017	07:17	79.0	92.9	93.0
08/02/2017	07:18	80.5	92.6	93.5
08/02/2017	07:19	80.6	92.8	93.7
08/02/2017	07:20	78.1	91.8	93.3
08/02/2017	07:21	78.2	90.5	94.0
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08/02/2017	07:23	78.0	92.7	93.3
08/02/2017	07:24	78.0	92.2	94.0
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08/02/2017	07:26	79.5	92.9	93.3
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08/02/2017	07:33	77.9	92.2	94.3
08/02/2017	07:34	77.9	92.1	94.4
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08/02/2017	07:42	79.6	91.9	94.0
08/02/2017	07:43	78.0	92.1	93.5
08/02/2017	07:44	78.5	91.4	93.7
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08/02/2017	07:53	78.6	91.5	94.3
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08/02/2017	08:14	77.4	93.2	94.5
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08/02/2017	08:24	78.2	93.7	94.5
08/02/2017	08:25	76.7	92.0	95.0
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08/02/2017	08:28	78.8	91.9	94.4
08/02/2017	08:29	78.4	91.5	94.1
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08/02/2017	08:48	78.7	92.6	94.4
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08/02/2017	08:53	78.7	91.6	94.2
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08/02/2017	09:01	76.3	91.4	94.1
08/02/2017	09:02	79.7	91.9	93.3
08/02/2017	09:03	78.9	92.4	94.0
08/02/2017	09:04	78.9	93.4	94.7

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08/02/2017	09:06	78.0	92.1	95.1
08/02/2017	09:07	76.3	93.2	94.1
08/02/2017	09:08	76.7	91.3	92.3
08/02/2017	09:09	76.3	92.2	91.8
08/02/2017	09:10	76.3	91.4	90.9
08/02/2017	09:11	77.5	92.6	91.6
08/02/2017	09:12	77.1	91.9	93.6
08/02/2017	09:13	76.8	91.9	93.8
08/02/2017	09:14	77.1	91.6	93.5
08/02/2017	09:15	76.6	92.1	93.7
08/02/2017	09:16	75.9	91.6	92.6
08/02/2017	09:17	74.8	90.8	94.0
08/02/2017	09:18	74.5	90.7	94.3
08/02/2017	09:19	76.2	91.9	95.2
08/02/2017	09:20	76.0	91.3	93.7
08/02/2017	09:21	74.3	90.6	93.9
08/02/2017	09:22	75.7	92.4	93.5
08/02/2017	09:23	75.9	92.2	94.0
08/02/2017	09:24	75.1	91.7	93.6
08/02/2017	09:25	74.7	91.4	94.0
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08/02/2017	09:35	76.1	91.8	94.0
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08/02/2017	09:53	78.0	91.3	94.5
08/02/2017	09:54	76.8	92.2	94.0
08/02/2017	09:55	77.6	90.6	94.1
08/02/2017	09:56	77.6	90.9	93.7
08/02/2017	09:57	77.6	91.5	94.0
08/02/2017	09:58	76.6	91.6	93.9
08/02/2017	09:59	77.8	91.7	94.0
08/02/2017	10:00	77.5	91.0	92.9
08/02/2017	10:01	77.0	90.9	94.2
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08/02/2017	10:03	76.1	91.5	93.3
08/02/2017	10:04	76.6	92.7	93.5
08/02/2017	10:05	74.4	91.3	94.8
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08/02/2017	10:20	77.2	92.3	93.8
08/02/2017	10:21	76.5	89.8	94.7
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08/02/2017	10:24	75.7	91.5	93.6
08/02/2017	10:25	76.3	91.3	94.6
08/02/2017	10:26	76.2	91.2	94.4
08/02/2017	10:27	76.2	92.0	94.1
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08/02/2017	10:32	75.0	91.2	94.6
08/02/2017	10:33	75.8	91.4	94.1
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08/02/2017	10:49	78.3	92.0	93.7
08/02/2017	10:50	78.2	91.3	93.5
08/02/2017	10:51	77.2	91.6	94.6
08/02/2017	10:52	78.8	91.3	94.7
08/02/2017	10:53	78.6	91.2	94.4
08/02/2017	10:54	78.1	91.2	95.5
08/02/2017	10:55	78.6	91.2	94.4
08/02/2017	10:56	77.2	91.6	94.6
08/02/2017	10:57	76.2	92.0	94.7
08/02/2017	10:58	77.1	92.1	93.8
08/02/2017	10:59	75.0	91.7	94.3
08/02/2017	11:00	79.2	90.7	94.6
08/02/2017	11:01	79.2	90.9	94.1
08/02/2017	11:02	79.8	91.3	94.0
08/02/2017	11:03	79.2	91.8	94.7
08/02/2017	11:04	79.6	91.1	94.6
08/02/2017	11:05	79.5	90.3	94.1
08/02/2017	11:06	79.8	90.3	94.7
08/02/2017	11:07	79.4	90.6	93.7
08/02/2017	11:08	79.6	90.9	93.5
08/02/2017	11:09	80.1	90.7	94.3
08/02/2017	11:10	79.2	90.9	93.9
08/02/2017	11:11	79.6	90.9	93.9
08/02/2017	11:12	79.2	90.9	93.1
08/02/2017	11:13	79.2	90.5	94.9
08/02/2017	11:14	79.0	90.6	94.3
08/02/2017	11:15	78.8	90.1	93.9
08/02/2017	11:16	79.2	90.1	94.6
08/02/2017	11:17	79.6	90.0	95.2
08/02/2017	11:18	79.5	90.6	93.6
08/02/2017	11:19	79.1	89.9	94.4
08/02/2017	11:20	79.8	90.8	94.6
08/02/2017	11:21	79.2	90.6	94.2
08/02/2017	11:22	79.4	91.0	94.4
08/02/2017	11:23	79.6	90.5	93.9
08/02/2017	11:24	79.3	90.2	94.1
08/02/2017	11:25	79.5	90.0	93.6

08/02/2017	11:26	80.6	90.1	94.9
08/02/2017	11:27	78.8	90.6	93.6
08/02/2017	11:28	79.5	90.3	94.0
08/02/2017	11:29	79.2	90.6	94.1
08/02/2017	11:30	80.1	89.6	93.9
08/02/2017	11:31	79.7	90.3	94.0
08/02/2017	11:32	79.3	90.2	93.8
08/02/2017	11:33	79.6	90.4	94.6
08/02/2017	11:34	78.5	90.6	94.6
08/02/2017	11:35	79.5	90.2	92.8
08/02/2017	11:36	79.3	90.7	94.4
08/02/2017	11:37	79.7	89.9	94.7
08/02/2017	11:38	79.9	90.6	95.3
08/02/2017	11:39	79.9	90.5	93.8
08/02/2017	11:40	80.2	90.5	94.7
08/02/2017	11:41	80.0	90.6	94.7
08/02/2017	11:42	79.1	90.7	94.6
08/02/2017	11:43	79.7	90.6	94.9
08/02/2017	11:44	79.9	89.7	94.0
08/02/2017	11:45	80.4	90.5	94.0
08/02/2017	11:46	79.8	90.1	93.7
08/02/2017	11:47	79.5	90.5	94.4
08/02/2017	11:48	79.9	90.1	93.5
08/02/2017	11:49	79.8	90.1	94.8
08/02/2017	11:50	79.5	89.8	94.9
08/02/2017	11:51	80.3	90.1	93.5
08/02/2017	11:52	80.5	91.2	94.9
08/02/2017	11:53	80.3	90.5	94.1
08/02/2017	11:54	80.1	90.3	94.9
08/02/2017	11:55	80.7	90.1	94.1
08/02/2017	11:56	78.1	90.1	94.3
08/02/2017	11:57	79.5	89.4	94.0
08/02/2017	11:58	79.6	90.1	94.0
08/02/2017	11:59	80.1	90.2	93.1
08/02/2017	12:00	79.9	90.7	94.0
08/02/2017	12:01	80.0	89.8	94.8
08/02/2017	12:02	79.8	90.1	94.1
08/02/2017	12:03	82.4	90.3	95.6
08/02/2017	12:04	81.2	90.6	93.8
08/02/2017	12:05	80.9	90.3	94.1
08/02/2017	12:06	80.8	89.9	93.7
08/02/2017	12:07	79.1	89.8	95.3
08/02/2017	12:08	80.3	91.2	93.9
08/02/2017	12:09	80.8	90.2	94.1
08/02/2017	12:10	80.0	89.6	94.4
08/02/2017	12:11	79.9	90.2	94.5
08/02/2017	12:12	80.1	90.1	94.1

08/02/2017	12:13	80.9	90.5	93.9
08/02/2017	12:14	80.8	90.1	93.3
08/02/2017	12:15	81.0	90.8	94.2
08/02/2017	12:16	81.1	89.7	93.7
08/02/2017	12:17	81.1	90.4	94.2
08/02/2017	12:18	80.8	90.5	93.7
08/02/2017	12:19	80.8	89.2	94.7
08/02/2017	12:20	81.0	90.3	94.0
08/02/2017	12:21	81.1	90.4	94.1
08/02/2017	12:22	80.2	90.2	95.4
08/02/2017	12:23	81.8	90.6	93.6
08/02/2017	12:24	81.4	89.6	95.1
08/02/2017	12:25	81.8	90.1	95.3
08/02/2017	12:26	80.6	90.3	94.4
08/02/2017	12:27	80.4	91.0	94.3
08/02/2017	12:28	80.4	89.9	94.4
08/02/2017	12:29	81.2	90.3	94.2
08/02/2017	12:30	80.7	91.1	94.3
08/02/2017	12:31	80.8	89.1	94.1
08/02/2017	12:32	80.1	89.7	93.9
08/02/2017	12:33	81.1	89.2	94.9
08/02/2017	12:34	80.9	89.6	94.3
08/02/2017	12:35	80.2	89.7	94.7
08/02/2017	12:36	81.1	89.8	93.1
08/02/2017	12:37	81.0	88.4	94.4
08/02/2017	12:38	80.5	89.8	93.9
08/02/2017	12:39	80.6	89.0	94.0
08/02/2017	12:40	81.0	89.6	94.4
08/02/2017	12:41	80.8	89.1	93.6
08/02/2017	12:42	81.5	89.5	93.8
08/02/2017	12:43	81.1	89.4	94.5
08/02/2017	12:44	80.8	88.8	94.5
08/02/2017	12:45	81.2	88.4	94.1
08/02/2017	12:46	79.8	88.9	95.7
08/02/2017	12:47	81.0	90.1	94.3
08/02/2017	12:48	80.9	89.4	93.8
08/02/2017	12:49	81.5	89.1	94.8
08/02/2017	12:50	80.7	88.8	94.3
08/02/2017	12:51	81.3	88.9	93.9
08/02/2017	12:52	81.4	88.4	93.7
08/02/2017	12:53	81.5	89.0	94.3
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08/02/2017	12:56	81.9	88.9	94.0
08/02/2017	12:57	82.1	89.0	93.7
08/02/2017	12:58	81.1	88.7	94.0
08/02/2017	12:59	81.6	89.6	94.7

08/02/2017	13:00	81.8	89.1	95.3
08/02/2017	13:01	80.9	89.1	93.5
08/02/2017	13:02	82.0	89.1	94.4
08/02/2017	13:03	82.0	89.3	94.1
08/02/2017	13:04	81.8	90.1	94.4
08/02/2017	13:05	81.8	89.3	94.5
08/02/2017	13:06	81.6	89.2	94.6
08/02/2017	13:07	81.5	88.8	94.0
08/02/2017	13:08	81.1	90.0	94.6
08/02/2017	13:09	81.3	89.5	94.7
08/02/2017	13:10	81.9	89.0	94.2
08/02/2017	13:11	81.4	89.2	94.3
08/02/2017	13:12	81.9	89.5	93.2
08/02/2017	13:13	82.3	89.1	93.9
08/02/2017	13:14	82.1	90.6	92.7
08/02/2017	13:15	81.7	88.9	92.8
08/02/2017	13:16	81.5	89.1	94.2
08/02/2017	13:17	81.5	88.8	94.3
08/02/2017	13:18	82.0	88.8	93.5
08/02/2017	13:19	81.3	90.1	93.6
08/02/2017	13:20	80.4	89.2	93.7
08/02/2017	13:21	81.2	89.9	93.9
08/02/2017	13:22	80.9	89.2	93.8
08/02/2017	13:23	81.3	89.5	94.9
08/02/2017	13:24	82.0	88.0	94.1
08/02/2017	13:25	81.5	89.0	93.7
08/02/2017	13:26	81.8	88.8	93.8
08/02/2017	13:27	81.6	89.3	94.0
08/02/2017	13:28	81.9	88.5	94.2
08/02/2017	13:29	81.4	89.7	94.2
08/02/2017	13:30	80.9	88.3	95.3
08/02/2017	13:31	80.6	89.2	93.9
08/02/2017	13:32	80.9	88.7	94.3
08/02/2017	13:33	80.8	88.3	94.3
08/02/2017	13:34	80.9	88.1	94.7
08/02/2017	13:35	81.4	89.3	94.3
08/02/2017	13:36	81.8	88.6	94.5
08/02/2017	13:37	80.7	89.1	94.8
08/02/2017	13:38	80.4	88.9	94.8
08/02/2017	13:39	81.4	88.3	93.6
08/02/2017	13:40	81.0	88.7	93.7
08/02/2017	13:41	80.7	88.6	94.4
08/02/2017	13:42	81.0	88.9	94.0
08/02/2017	13:43	81.3	89.0	94.9
08/02/2017	13:44	80.2	88.6	94.0
08/02/2017	13:45	81.2	88.7	94.4
08/02/2017	13:46	80.6	87.3	94.1

08/02/2017	13:47	80.8	88.4	93.2	
08/02/2017	13:48	80.4	88.4	94.6	
08/02/2017	13:49	80.1	89.8	94.3	
08/02/2017	13:50	81.3	89.5	94.5	
08/02/2017	13:51	80.3	89.6	94.0	
08/02/2017	13:52	80.2	89.1	94.2	
08/02/2017	13:53	81.2	88.9	93.7	
08/02/2017	13:54	80.3	88.8	93.8	
08/02/2017	13:55	80.3	89.8	94.5	
08/02/2017	13:56	80.7	88.5	92.7	
08/02/2017	13:57	81.7	89.2	94.3	
08/02/2017	13:58	80.4	88.4	93.5	
08/02/2017	13:59	80.9	89.2	94.5	
08/02/2017	14:00	78.8	88.7	93.9	
08/02/2017	14:01	82.1	88.0	94.3	
08/02/2017	14:02	81.3	88.4	92.4	
08/02/2017	14:03	81.6	89.7	94.1	
08/02/2017	14:04	81.2	89.1	93.3	
08/02/2017	14:05	81.1	89.2	94.6	
08/02/2017	14:06	81.2	88.7	94.2	
08/02/2017	14:07	82.1	88.6	93.3	
08/02/2017	14:08	80.5	89.7	92.7	
08/02/2017	14:09	80.8	88.8	94.9	
08/02/2017	14:10	80.3	88.6	94.0	
08/02/2017	14:11	81.3	89.3	93.5	
08/02/2017	14:12	81.6	88.7	94.0	
08/02/2017	14:13	81.6	89.7	94.7	
08/02/2017	14:14	81.7	90.1	94.4	
08/02/2017	14:15	82.2	89.4	93.9	
08/02/2017	14:16	81.4	89.5	94.0	
08/02/2017	14:17	82.4	88.7	94.4	
08/02/2017	14:18	81.6	90.4	94.3	
08/02/2017	14:19	81.3	89.0	93.5	
08/02/2017	14:20	81.7	89.5	93.9	
08/02/2017	14:21	80.5	89.7	93.7	
08/02/2017	14:22	81.3	88.1	94.9	
08/02/2017	14:23	81.7	90.7	94.1	
08/02/2017	14:24	81.7	89.4	94.4	
08/02/2017	14:25	81.5	89.8	95.2	
08/02/2017	14:26	81.8	90.4	94.3	
08/02/2017	14:27	81.6	90.0	94.1	
08/02/2017	14:28	81.1	90.0	94.0	
08/02/2017	14:29	82.3	90.4	93.5	
08/02/2017	14:30	81.7	89.7	93.9	
08/02/2017	14:31	82.2	89.7	94.1	
08/02/2017	14:32	82.2	89.9	93.3	
08/02/2017	14:33	82.6	90.1	94.7	88.4

08/02/2017	14:34	82.0	90.0	95.0	88.0
08/02/2017	14:35	81.9	90.3	93.6	87.7
08/02/2017	14:36	82.0	89.9	93.2	87.9
08/02/2017	14:37	82.2	90.2	94.5	87.7
08/02/2017	14:38	82.4	89.4	93.9	87.7
08/02/2017	14:39	82.1	89.6	94.2	87.5
08/02/2017	14:40	82.0	90.4	93.7	87.7
08/02/2017	14:41	81.6	90.1	93.9	87.6
08/02/2017	14:42	82.2	90.2	93.7	87.6
08/02/2017	14:43	82.4	89.3	93.6	87.8
08/02/2017	14:44	82.1	90.2	95.0	87.7
08/02/2017	14:45	82.9	90.3	94.6	87.9
08/02/2017	14:46	82.0	90.9	94.1	88.1
08/02/2017	14:47	82.6	88.8	93.8	88.1
08/02/2017	14:48	83.0	90.8	93.8	88.3
08/02/2017	14:49	82.6	89.6	94.9	88.2
08/02/2017	14:50	82.0	90.3	94.0	88.2
08/02/2017	14:51	82.0	90.8	93.6	88.2
08/02/2017	14:52	82.4	90.1	94.4	88.0
08/02/2017	14:53	82.4	90.5	93.9	88.3
08/02/2017	14:54	81.9	91.0	94.3	88.1
08/02/2017	14:55	81.7	90.5	93.6	88.0
08/02/2017	14:56	82.1	90.9	94.1	87.8
08/02/2017	14:57	81.2	90.7	94.0	87.8
08/02/2017	14:58	81.7	90.3	94.1	87.8
08/02/2017	14:59	82.7	90.8	94.3	88.0
08/02/2017	15:00	81.4	90.5	93.7	88.0
08/02/2017	15:01	80.8	89.9	94.3	87.8
08/02/2017	15:02	81.2	89.2	93.8	88.3
08/02/2017	15:03	81.8	89.7	93.7	88.4
08/02/2017	15:04	81.4	90.0	93.4	88.3
08/02/2017	15:05	80.1	90.2	94.2	88.1
08/02/2017	15:06	79.4	90.3	94.5	88.2
08/02/2017	15:07	78.9	88.9	94.3	87.7
08/02/2017	15:08	78.8	89.6	93.9	87.8
08/02/2017	15:09	79.9	89.3	93.9	87.7
08/02/2017	15:10	78.8	89.7	92.7	87.7
08/02/2017	15:11	79.2	89.7	94.5	87.8
08/02/2017	15:12	80.2	89.6	94.0	87.7
08/02/2017	15:13	80.6	89.5	94.3	87.9
08/02/2017	15:14	77.4	88.8	93.8	87.6
08/02/2017	15:15	80.1	89.1	94.0	88.1
08/02/2017	15:16	79.4	89.3	94.6	87.6
08/02/2017	15:17	78.8	89.9	93.9	88.1
08/02/2017	15:18	79.6	89.6	93.5	87.9
08/02/2017	15:19	79.9	89.1	95.0	87.9
08/02/2017	15:20	79.7	89.5	93.2	88.0

08/02/2017	15:21	78.6	88.5	93.5	88.0	
08/02/2017	15:22	78.7	89.6	94.3	87.8	
08/02/2017	15:23	79.2	89.8	93.7	87.7	
08/02/2017	15:24	78.0	89.5	94.7	87.7	
08/02/2017	15:25	77.8	88.8	94.6	87.6	
08/02/2017	15:26	79.1	89.3	94.3	87.5	
08/02/2017	15:27	78.7	89.0	94.7	87.6	
08/02/2017	15:28	79.8	89.3	94.6	87.8	
08/02/2017	15:29	79.9	89.0	94.0	87.9	
08/02/2017	15:30	78.9	89.7	94.6	87.9	
08/02/2017	15:31	80.1	89.2	94.3	88.0	
08/02/2017	15:32	78.8	89.6	94.1	88.1	
08/02/2017	15:33	79.6	89.7	94.0	87.9	
08/02/2017	15:34	80.7	90.1	93.7	87.8	
08/02/2017	15:35	78.9	89.7	94.1	87.8	
08/02/2017	15:36	80.9	89.2	93.5	87.8	
08/02/2017	15:37	82.5	89.1	94.9	87.6	
08/02/2017	15:38	82.2	89.8	94.0	87.8	
08/02/2017	15:39		89.2	94.6	88.0	
08/02/2017	15:40		89.6	94.0		88.0
08/02/2017	15:41		90.1	94.4		87.7
08/02/2017	15:42		89.6	94.4		88.3
08/02/2017	15:43		89.9	94.0		88.1
08/02/2017	15:44		89.3	93.3		88.2
08/02/2017	15:45		89.3	94.6		88.2
08/02/2017	15:46		89.0	94.4		88.3
08/02/2017	15:47		90.3	94.1		88.2
08/02/2017	15:48		89.8	94.8		87.9
08/02/2017	15:49		90.3	92.8		88.3
08/02/2017	15:50		89.1	94.3		88.6
08/02/2017	15:51		89.9	94.4		88.6
08/02/2017	15:52		89.2	93.2		88.3
08/02/2017	15:53		90.2	94.6		88.4
08/02/2017	15:54		89.3	94.8		88.6
08/02/2017	15:55		89.0	95.0		88.4
08/02/2017	15:56		89.3	94.7		88.5
08/02/2017	15:57		90.0	94.9		88.5
08/02/2017	15:58		89.9	94.7		88.4
08/02/2017	15:59		89.1	93.8		88.4
08/02/2017	16:00		90.3	94.4		88.4
08/02/2017	16:01		90.1	95.0		88.4
08/02/2017	16:02		89.4	94.8		88.3
08/02/2017	16:03		89.6	93.8		88.7
08/02/2017	16:04		90.3	94.2		88.5
08/02/2017	16:05		89.9	94.1		88.5
08/02/2017	16:06		89.7	94.3		88.7
08/02/2017	16:07		89.8	94.7		88.1

08/02/2017	16:08	89.7	94.5	88.5
08/02/2017	16:09	90.2	94.3	88.5
08/02/2017	16:10	89.6	93.9	88.4
08/02/2017	16:11	90.3	94.3	88.3
08/02/2017	16:12	89.6	94.2	88.5
08/02/2017	16:13	89.9	94.1	88.2
08/02/2017	16:14	89.7	94.3	88.5
08/02/2017	16:15	90.1	93.3	88.4
08/02/2017	16:16	89.6	94.4	88.4
08/02/2017	16:17	90.2	93.9	88.4
08/02/2017	16:18	90.1	94.3	88.2
08/02/2017	16:19	90.0	94.6	88.1
08/02/2017	16:20	89.1	93.4	88.6
08/02/2017	16:21	89.7	94.4	88.3
08/02/2017	16:22	89.9	94.2	87.9
08/02/2017	16:23	89.9	93.2	88.5
08/02/2017	16:24	89.8	95.0	88.0
08/02/2017	16:25	89.4	93.5	88.2
08/02/2017	16:26	90.0	94.6	88.4
08/02/2017	16:27	89.9	94.2	87.6
08/02/2017	16:28	90.1	94.5	88.1
08/02/2017	16:29	90.2	94.2	88.2
08/02/2017	16:30	89.7	93.5	88.3
08/02/2017	16:31	89.4	94.0	88.1
08/02/2017	16:32	89.9	94.6	88.1
08/02/2017	16:33	88.9	94.4	88.4
08/02/2017	16:34	89.9	94.6	88.0
08/02/2017	16:35	89.5	92.8	87.8
08/02/2017	16:36	90.2	94.8	87.7
08/02/2017	16:37	90.3	94.6	88.1
08/02/2017	16:38		94.7	88.0
08/02/2017	16:39		95.6	88.1
08/02/2017	16:40		94.9	87.9
08/02/2017	16:41		94.3	88.0
08/02/2017	16:42			87.9
08/02/2017	16:43			88.3
08/02/2017	16:44			88.1
08/02/2017	16:45			87.9
08/02/2017	16:46			87.9
08/02/2017	16:47			87.5
08/02/2017	16:48			87.6
08/02/2017	16:49			87.6
08/02/2017	16:50			86.9
08/02/2017	16:51			87.6
08/02/2017	16:52			87.7
08/02/2017	16:53			87.5
08/02/2017	16:54			88.0

ATTACHMENT B –Hydrant Testing Results

FLOW TEST DATA LOG SHEET
Vista Irrigation District Master Plan

Pressure Zone: 550

Date: 08/01/2017

PRV/PRSs Settings (IN)				PRV/PRSs Settings (OUT)		
Name	Setting	Notes:		Name	Setting	Notes:
T2		SCADA				
T8D1		CC				
CW36		CW				
T7		J.PAT				
T3E		STEVE				
T3E (12")						
T3	126/52	RICH				

Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B		Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B
1	13481	13049	13483		2	12559	12560	13881
Init. Psi		60	99		Init. Psi		53	64
Flow Psi	30	56	99		Flow Psi	40	50	62
Final Psi		60	100		Final Psi		53	64
Q - Time	850GPM	10:42	10:47		Q - Time	975GPM	11:15	11:21
Init. Psi					Init. Psi			
Flow Psi					Flow Psi			
Final Psi					Final Psi			
Q - Time					Q - Time			

Test No.	RES	Start	Stop		Test No.	PS	Flow	Psi
1	Lupine		SCADA					
2	Lupine		SCADA					

Notes:

FLOW TEST DATA LOG SHEET
Vista Irrigation District Master Plan

Test #	PRV Name	In/Out	Initial Position	Flowing Position	Final Position	Estimated Flow	Notes
1	T3	IN	OPEN 1 1/16"	OPEN 1 1/16"	OPEN 1 1/16"		NO CHANGE
			115/48	115/47	115/48		
2	T3	IN	OPEN 1 1/16"	OPEN 1 1/16"	OPEN 1 1/16"		NO CHANGE
			115/48	115/46	115/48		
1	T7	IN	CLOSED	CLOSED	CLOSED		NO CHANGE
			110/48	110/48	110/48		
2	T7	IN	CLOSED	CLOSED	CLOSED		NO CHANGE
			112/48	112/48	112/48		
1	T3E	IN	CLOSED	CLOSED	CLOSED		
			116/57	117/55	117/56		
2	T3E	IN	CLOSED	CLOSED	CLOSED		
			116/57	116/56	116/57		
1	T8D1	IN	OPEN 3/8"	3/8	3/8		
			130/62	130/60	130/62		
2	T8D1	IN	OPEN 5/16	6/16	5/16		
			130/62	130/60	130/62		
1	CW36	IN	CLOSED	CLOSED	CLOSED		
			124/70	124/69	124/70		
2	CW36	IN	CLOSED	CLOSED	CLOSED		
			124/69	124/69	125/69		

FLOW TEST DATA LOG SHEET
Vista Irrigation District Master Plan

Pressure Zone: 707 Date:

PRV/PRSs Settings (IN)				PRV/PRSs Settings (OUT)		
Name	Setting	Notes:		Name	Setting	Notes:
CW3				T3E(6")		
CW		CC		T3A		
BCS				T2		SCADA
A18				T8D1		
BCS20		STEVE		CW36		CW
				T7		J PAT
CSBD#1				T3E		
VID9				T3E (12")		
CWA10				CX28		
				T3 (to 550)		RICH-NEVER OPENED
				T3 (6" to 630)		RICH

Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B		Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B
3	13303	13412	13425		4	97163	14313	14314
Init. Psi		94	131		Init. Psi		78	88
Flow Psi	55	94	130		Flow Psi	50	71	82
Final Psi		94	131		Final Psi		78	88
Q-Time	1100GPM	12:00	12:06		Q - Time	1150GPM	12:58	01:04
5*	13799	13978	14144					
Init. Psi		92	114		Init. Psi			
Flow Psi	35	88	112		Flow Psi			
Final Psi		92	116		Final Psi			
Q - Time	900GPM	01:54	02:00		Q - Time			

*5-changed to hydrant closer to flow hydrant

Test No.	RES	Start	Stop		Test No.	PS	Flow	Psi
3	A-RES							
4	A-RES							
5	A-RES							

Notes: 5A - 2002 REDWOOD CREST 4B - 648 ROLLING HILLS RANCH
5B - ALDERWOOD & WHITE BIRCH

FLOW TEST DATA LOG SHEET
Vista Irrigation District Master Plan

Test #	PRV Name	In/Out	Initial Position	Flowing Position	Final Position	Estimated Flow	Notes
3	T3	OUT	CLOSED	CLOSED	CLOSED		
			114/48	114/48	114/48		
4	T3	OUT	CLOSED	CLOSED	CLOSED		
			115/48	115/48	115/48		
5	T3	OUT	CLOSED	CLOSED	CLOSED		
			116/48	116/48	116/48		
3	T7	OUT	CLOSED	CLOSED	CLOSED		
			112/47	110/47	110/47		
4	T7	OUT	CLOSED	CLOSED	CLOSED		
			112/47	112/47	112/47		
5	T7	OUT	CLOSED	CLOSED	CLOSED		
			113/48	113/48	113/48		
3	BCS 20	IN	CLOSED	CLOSED	CLOSED		
			118/79	118/79	119/79		
4	BCS 20	IN	CLOSED	CLOSED	CLOSED		
			119/80	119/78	120/80		
5	BCS 20	IN	CLOSED	CLOSED	CLOSED		
			120/81	120/79	119/78		
3	CW	IN	CLOSED	CLOSED	CLOSED		
			153/116	153/112	153/116		
4	CW	IN	CLOSED	CLOSED	CLOSED		
			153/116	153/112	153/116		
5	CW	IN	CLOSED	CLOSED	CLOSED		
			153/116	153/112	153/116		
3	CW 36	OUT	CLOSED	CLOSED	CLOSED		
			125/68	125/67	124/68		
4	CW 36	OUT	CLOSED	CLOSED	CLOSED		
			124/68	122/69	123/69		
5	CW36	OUT	CLOSED	CLOSED	CLOSED		
			125/68	125/68	125/68		

FLOW TEST DATA LOG SHEET
Vista Irrigation District Master Plan

Pressure Zone: 837 Date: 08/01/2017

PRV/PRSs Settings (IN)				PRV/PRSs Settings (OUT)		
Name	Setting	Notes:		Name	Setting	Notes:
AB		RICH		CW	0-10"	CC REPLACEMENT
D2				CW3		CW
HL				BCS		
				D3		
VWD 6				"C"RES		
VWD 7				E43		J PAT
VWD 8				BCS 20		STEVE
CWA8				A18		
CWA9				810	CWSED	

Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B		Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B
6*	13502	12735	14599		7	13347	13370	13635
Init. Psi		143	146		Init. Psi		103	100
Flow Psi		140	142		Flow Psi		100	96
Final Psi		139	146		Final Psi		103	100
Q - Time*	1000-1300	03:18	03:22		Q - Time	1300	04:04	04:09
	1350	3:24	3:29					
8	12617	14292	14322					
Init. Psi		146	153		Init. Psi			
Flow Psi		145	151		Flow Psi			
Final Psi		146	153		Final Psi			
Q - Time	1500	04:34	04:40		Q - Time			

Test No.	RES	Start	Stop		Test No.	PS	Flow	Psi
6	PECH1				6 out	PS10		
6	PECH2				7 out	PS10		
7	PECH1				8 out	PS10		
7	PECH2							
8	PECH2							
8	PECH2							

Notes:*start of WCC diffuser

FLOW TEST DATA LOG SHEET
Vista Irrigation District Master Plan

Test #	PRV Name	In/Out	Initial Position	Flowing Position	Final Position	Estimated Flow	Notes
6	AB	IN	3 6/16	3 12/16	3 6/16		
			100/26	98/25	98/26		
7	AB	IN	3 7/16	3 8/16	3 6/16		
			97/26	96/25	97/25		
8	AB	IN	3 5/16	3 6/16	3 3/16		
			97/26	97/26	97/26		
6	E43	OUT	CL 182/75	CL 182/75	CL 182/75		
7	E43	OUT	CL 182/75	CL 182/75	CL 182/75		
8	E43	OUT	CL 184/75	CL 184/75	CL 184/75		
6	BCS20	OUT	CLOSED	CLOSED	CLOSED		
			120/80	116/80	120/80		
7	BCS20	OUT	CLOSED	CLOSED	CLOSED		
			120/80	117/81	121/79		
8	BCS20	OUT	CLOSED	CLOSED	CLOSED		
			122/80	121/80	122/80		
6	CW3	OUT	OPEN				
			108/62	110/60	109/60		
7	CW3	OUT	OPEN				
			110/60	108/60	110/60		
8	CW3	OUT	OPEN				
			110/60	110/60	110/60		
6	CW	OUT	CLOSED	CLOSED	CLOSED		
			158/115	158/115	158/115		
7	CW	OUT	CLOSED	CLOSED	CLOSED		
			158/115	158/115	158/115		
8	CW	OUT	CLOSED	CLOSED	CLOSED		
			158/115	157/115	158/115		

FLOW TEST DATA LOG SHEET
Vista Irrigation District Master Plan

Pressure Zone: 637 | Date: 08/02/2017

PRV/PRSs Settings (IN)				PRV/PRSs Settings (OUT)		
Name	Setting	Notes:		Name	Setting	Notes:
CX28		SCADA		CX27K		BEN
"C"RES		SCADA		CX27		SCADA
				EX22JF		STEVE

Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B		Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B
9	14327	14331	14739		10	14399	14379	14410
Init. Psi		80	73		Init. Psi		112	118
Flow Psi	60	76	69-70		Flow Psi	60	107	114
Final Psi		80	73		Final Psi		112	119
Q - Time	1300	08:01	08:07		Q - Time	1300	08:23	08:28
11	11850	11611	11596					
Init. Psi		108	119		Init. Psi			
Flow Psi	80	104	111-114		Flow Psi			
Final Psi		108	119		Final Psi			
Q - Time	1500	08:40	08:45		Q - Time			

Test No.	RES	Start	Stop		Test No.	PS	Flow	Psi
9	"C"RES							
10	"C"RES							
11	"C"RES							

Notes:

Test #	PRV Name	In/Out	Initial Position	Flowing Position	Final Position	Estimated Flow	Notes
9	CX27K	OUT					
	3"		OPEN-1/2"	3/4"	1/2"	~700-900 GPM	
			150/88	147/87	153/88		
	8"		CLOSED	CLOSED	CLOSED		
			150/88	147/87	153/88		
10	CX27K						
	3"	OUT	1/2"	1/2"	1/2"	~700 GPM	
			152/88	144/88	155/88		
	8"		CLOSED	CLOSED	CLOSED		
			152/88	144/88	155/88		
11	CX27K	OUT					
	3"		1/2"	1/2"	1/2"	~700 GPM	
			152/88	144/88	156/87		
	8"		CLOSED	CLOSED	CLOSED		
			152/88	144/88	156/87		

FLOW TEST DATA LOG SHEET
Vista Irrigation District Master Plan

Pressure Zone: 486 Date: 08/02/2017

PRV/PRSs Settings (IN)				PRV/PRSs Settings (OUT)		
Name	Setting	Notes:		Name	Setting	Notes:
CX27K		BEN				
EX22		SCADA				
EX20K		JASON				
EX22JF		STEVE				
OC#4		NORMALLY CLOSED				

Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B		Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B
12	11891	11886	11881					
Init. Psi		68	68		Init. Psi			
Flow Psi	35	61	63-60		Flow Psi			
Final Psi		68	68		Final Psi			
Q - Time	1000GPM	09:07	09:12		Q - Time			
Init. Psi					Init. Psi			
Flow Psi					Flow Psi			
Final Psi					Final Psi			
Q - Time					Q - Time			

Test No.	RES	Start	Stop		Test No.	PS	Flow	Psi

Notes:

FLOW TEST DATA LOG SHEET

Vista Irrigation District Master Plan

Test #	PRV Name	In/Out	Initial Position	Flowing Position	Final Position	Estimated Flow	Notes
12	CX27K	IN					
	3"		1/2"	3/4"	1/2"	~700 GPM	
			152/88	152/88	156/88		
	8"		CLOSED	CLOSED	CLOSED		
			152/88	152/88	156/88		

FLOW TEST DATA LOG SHEET
Vista Irrigation District Master Plan

Pressure Zone: 565 Date: 08/02/2017

PRV/PRSs Settings (IN)				PRV/PRSs Settings (OUT)		
Name	Setting	Notes:		Name	Setting	Notes:
CX27				EX22		
E43		BEN		EX20K		
E43S		STEVE				
E32						
F		CW				
E42E		CC				
OC#2						
OC#3						

OLIVE AVE & CIETITA LINDA DR

Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B		Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B
13	14913	12074	12126		14	14929	13176	97049
Init. Psi		52	80-81		Init. Psi		89	102-103
Flow Psi	42	48	79		Flow Psi	80	88	102-103
Final Psi		51	81		Final Psi		89	
Q - Time	1100	09:49	09:55		Q - Time	1600	10:25	10:32

Init. Psi					Init. Psi			
Flow Psi					Flow Psi			
Final Psi					Final Psi			
Q - Time					Q - Time			

Test No.	RES	Start	Stop		Test No.	PS	Flow	Psi
13	E-1							
13	SLR							
14	E-1							
14	SLR							

Notes:

FLOW TEST DATA LOG SHEET
Vista Irrigation District Master Plan

Test #	PRV Name	In/Out	Initial Position	Flowing Position	Final Position	Estimated Flow	Notes
13	E43	IN					
	8"		1/16"	1/16"	1/16"		
			199/69	199/69	200/69		
14	E43	IN					
	8"		1/16"	1/16"	1/16"		
			200/68	200/68	200/70		
13	F	IN	CLOSED	CLOSED	CLOSED		
			130/95	130/90	130/92		
14	F	IN	CLOSED	CLOSED	CLOSED		
			130/95	130/89	130/92		
13	E42E	IN	CLOSED	CLOSED	CLOSED		
			91/51	91/49	91/57		
14	E42E	IN	CLOSED	CLOSED	CLOSED		
			91/52	91/48	91/52		

FLOW TEST DATA LOG SHEET

Vista Irrigation District Master Plan

Pressure Zone: 668 Date: 08/02/2017

PRV/PRSs Settings (IN)				PRV/PRSs Settings (OUT)		
Name	Setting	Notes:		Name	Setting	Notes:
F6		STEVE		F		CW
F12E		SCADA		E42E		CC
HN38		BEN				
810/668		SCADA				
VID 11		SCADA				

MONTRACHET ST

Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B		Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B
15*	12878	13213	13243		16	12496	12260	13013
Init. Psi		124	127		Init. Psi		93	92
Flow Psi	75	117	110-111		Flow Psi	55	78	77
Final Psi		124	127		Final Psi		93	92
Q - Time	1450	11:34	11:40		Q - Time	1200	10:56	11:02
*15-threads on original hydrant did not alllow a good seal								
Init. Psi					Init. Psi			
Flow Psi					Flow Psi			
Final Psi					Final Psi			
Q - Time					Q - Time			

Test No.	RES	Start	Stop		Test No.	PS	Flow	Psi

Notes:

FLOW TEST DATA LOG SHEET
Vista Irrigation District Master Plan

Test #	PRV Name	In/Out	Initial Position	Flowing Position	Final Position	Estimated Flow	Notes
15	HN38	IN					
	3"		CLOSED	CLOSED	CLOSED		
			160/89	160/74	159/86		
	8"	IN	CLOSED	1/2"	CLOSED		
			160/89	160/74	159/86		
16	HN38	IN					
	3"		CLOSED	CLOSED	CLOSED		
			158/92	150/84	160/89		
	8"	IN	CLOSED	CLOSED	CLOSED		
			158/92	150/84	160/89		
15	F	OUT	CLOSED	CLOSED	CLOSED		
			130/89	130/91	130/90		
16	F	OUT	CLOSED	CLOSED	CLOSED		
			130/91	130/90	130/91		
15	E42E	OUT	CLOSED	CLOSED	CLOSED		
			91/51	88/51	92/51		
16	E42E	OUT	CLOSED	CLOSED	CLOSED		
			91/51	75/51	91/51		

FLOW TEST DATA LOG SHEET
Vista Irrigation District Master Plan

Pressure Zone: 810

Date: 08/02/2017

PRV/PRSs Settings (IN)				PRV/PRSs Settings (OUT)		
Name	Setting	Notes:		Name	Setting	Notes:
HN14		JASON		E30S		
HN15				E-E		
				F6		STEVE
				F12E		
				810/668		
				HN38		BEN
OC#1						
VWD#9						
CWA11						

Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B		Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B
17	12216	12210	13124	NEXT EAST HYDRANT VISTA GRANDE DRIVE				
Init. Psi		72	63		Init. Psi			
Flow Psi	65	64	55-60		Flow Psi			
Final Psi		72	60		Final Psi			
Q - Time	1350	12:16	12:22		Q - Time			
Init. Psi					Init. Psi			
Flow Psi					Flow Psi			
Final Psi					Final Psi			
Q - Time					Q - Time			

Test No.	RES	Start	Stop		Test No.	PS	Flow	Psi
17	H RES				17	PS9out		

Notes: 17- original hydrant was being used for a road resurfacing project

Test #	PRV Name	In/Out	Initial Position	Flowing Position	Final Position	Estimated Flow	Notes
17	HN38	OUT					
	3"		CLOSED	CLOSED	CLOSED		
			158/92	152/92	158/92		
	8"		CLOSED	CLOSED	CLOSED		
			158/92	152/92	158/92		

FLOW TEST DATA LOG SHEET
Vista Irrigation District Master Plan

Pressure Zone: 752 Date: 08/02/2017

PRV/PRSs Settings (IN)				PRV/PRSs Settings (OUT)		
Name	Setting	Notes:		Name	Setting	Notes:
E-E		SCADA		E32		SCADA
D3		BEN		E43S		STEVE
E305						

Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B		Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B
18	14800	11268	14892		19	11642	11661	12018
Init. Psi		148	150		Init. Psi		164	121
Flow Psi	100	146	146		Flow Psi	100	164	119-120
Final Psi		148	150		Final Psi		164	120-121
Q - Time	1650	01:43	01:49		Q - Time	1650	02:05	02:13

Init. Psi					Init. Psi			
Flow Psi					Flow Psi			
Final Psi					Final Psi			
Q - Time					Q - Time			

Test No.	RES	Start	Stop		Test No.	PS	Flow	Psi
18	E							
19	E							

Notes:

Test #	PRV Name	In/Out	Initial Position	Flowing Position	Final Position	Estimated Flow	Notes
18	D3	IN					
	8"		1/2"	1/2"	1/2"	~700 GPM	
			107/87	106/87	108/88		
19	D3	IN					
	8"		1/2"	1/2"	1/2"	~700 GPM	
			108/88	104/88	106/87		

FLOW TEST DATA LOG SHEET
Vista Irrigation District Master Plan

Pressure Zone: 984
Telog Hydrant: 14466 (T3B)
Telog Name:

Date: 08/02/2017
Time:
Time:

PRV/PRSs Settings (IN)				PRV/PRSs Settings (OUT)		
Name	Setting	Notes:		Name	Setting	Notes:
				D1		SCADA
				AB BEW		SCADA
				HP REL		SCADA
				HPR		SCADA
				HN14		JASON
				HL16		RICH
VWD#5						
CWA3						

Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B		Test No.	Flowing Hydrant	Residual Hyd. A	Residual Hyd. B
20	11937	11933	11939		21	14458	14453	14463
Init. Psi		108	124		Init. Psi		88	67
Flow Psi	80	106	120-124		Flow Psi	30	80	58-62
Final Psi		108	124		Final Psi		88	67
Q - Time	1500	02:33	02:39		Q - Time	1000	03:19	03:22
							03:23	03:26
Init. Psi					Init. Psi			
Flow Psi					Flow Psi			
Final Psi					Final Psi			
Q - Time					Q - Time			

Test No.	RES	Start	Stop		Test No.	PS	Flow	Psi
20	HB				20	PS1in		
20	HP				20	PS9in		
21	HB				20	PS10in		
21	HP				20	PS11out		
					20	PS12in		
					21	PS1in		
					21	PS9in		
					21	PS10in		
					21	PS11out		
					21	PS12in		

Notes: 21A IS IN CORRECT LOCATION. ALL OTHERS-CHANGE PER RANDY'S KML

FLOW TEST DATA LOG SHEET
Vista Irrigation District Master Plan

Test #	PRV Name	In/Out	Initial Position	Flowing Position	Final Position	Estimated Flow	Notes
20	AB	OUT					
	12"		1/2"	1/2"	1/2"	~1000 GPM	
			88/12	86/11	88/12		
21	AB	OUT	1/2"	1/4"	1/2"	~100-1000 GPM	
	12"		88/12	84/8	88/10		
20	HL16	OUT					
			1/4"-0	CLOSED	CLOSED		REBOUND=113
			110/79				
21	HL16	OUT					
			CLOSED	1/16"	1/16"		REBOUND=105-115
			110/79				

Appendix H. 2000 Master Plan Improvements List

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TABLE 7-1
RECOMMENDED EXISTING SYSTEM IMPROVEMENTS

IDENTIFIER	DESCRIPTION	REASON	CONSTRUCTION COST ($)
EX-1	Construct new 8" in Taylor St. between Paseo Del Sol and Santa Fe Av. (690' - 8"); construct new 10" in Santa Fe Av. And W Bobier Dr. (5725' - 10")	Convert 565 zone west of Goodwin Dr between Taylor St and W Bobier Dr. to 668 zone to increase water pressure in area	597,000
EX-2	Construct new 8" in 1) Gopher Canyon Rd. between E. Vista Wy and Fruitland Dr. (530' - 8"), 2) E. Vista Wy between Hutchinson St. and Osborne St. (5740' - 16"), and 3) Fairview Dr. and behind lots (600' - 8"); parallel existing pipe at intersection of Hutchinson St. and E Vista Wy. (90' - 12")	Convert northernmost portion of 837 zone to 668 zone to reduce high system pressures in former 837 zone	1,114,000
EX-3	Construct new 10", 12", 20", and 24" pipe in East Dr. (35' - 12", 4085' - 20", 645' - 24"), Indian Rock Rd., Los Angeles Dr., Smith Dr., and Cabrillo Cir. (60' - 10", 3660' - 24")	Relocate existing line into public right of way and increase capacity based on future demands	2,062,000
EX-4	Construct new 8" in Kings Rd. between Kings Wy. and Warmlands Av. (385' - 12"); Warmlands Av. between Kings Rd. and Hummingbird Ln. (745' - 8"); Hummingbird Ln. (520' - 8"); Monte Mar Rd. east of Warmlands Av. (375' - 8"); Odell Cir (295' - 8"); Alessandro Ln. west of Vereda Barranca (855' - 8"); behind lots between Alessandro Ln. and Friendly Dr. (280' - 8"); Friendly Dr. (1445' - 8"); and Edgehill Rd. west of HL regulator (1940' - 8")	Extend 976 zone to Edgehill Rd., Friendly Dr., and Alessandro Ln., to consolidate 3 pressure zones in area (976,810,752) and move services to higher pressure zones and relocate services from abandoned lines; Facilitate supply from VID11	534,000
EX-5	Construct new 16" in La Mirada Dr. between Sycamore Av. and Pointsettia Av. (3860' - 16") and 20" in Sycamore Av. between La Mirada Dr. and BCS regulator (7675' - 20")	Connect VID 9 directly to 837 zone to facilitate distribution with aqueduct connection in higher zone	2,386,000
EX-6	Construct new 24" in Blue Bird Canyon Rd. between Pechstein Reservoir and AB Regulator	Provide second feed out of Pechstein to 837 zone to facilitate distribution of local water and increase system reliability	2,609,000
EX-7	Add parallel pipe in Osborne St. from the 9" line west of E Vista Wy. to F Regulator (3990' 8"); Hutchinson St. between Osborne St. and Barsby St. (1310' - 8"); Goodwin Dr. from Barsby St. to approx. 250' south of Rancho Corte (1045' - 8")	Increase pipeline capacity to reduce headloss	482,000
EX-8	Add parallel pipe in E Vista Wy. between Corvalla Dr. and approx. 200' south of Warmlands Av. (545' - 8")	Increase pipeline capacity to reduce headloss	41,000
EX-9	Add parallel pipe in Taylor St. between Kevin Dr. and W Taylor St. (3390' - 8"); add parallel pipe in Rivera St. from Taylor St. approx. 315' north (355' - 8")	Increase pipeline capacity to reduce headloss	285,000

TABLE 7-1
RECOMMENDED EXISTING SYSTEM IMPROVEMENTS
(continued)

IDENTIFIER	DESCRIPTION	REASON	CONSTRUCTION COST ($)
EX-10	Add parallel pipe in Arcadia Av. Between Cale Jules and E Vista Wy. (1025' - 12"); add parallel pipe in intersection of Arcadia Av. And E Vista Wy. (140' - 16")	Increase pipeline capacity to reduce headloss	142,000
EX-11	Add parallel pipe in Kings Rd. between Vista Tierra Del Cielo and Kings Wy. (1525' - 10"); add parallel pipe in Warmlands Av. between Kings Rd. And Queens Wy. (605' - 8")	Increase pipeline capacity to reduce headloss	191,000
EX-12	Add parallel pipe connecting F6 Regulator with 810 zone in E Vista Wy. (340' - 16")	Increase pipeline capacity to reduce headloss	60,000
EX-13	Add parallel pipe in Bandini Pl. between Rancho Vista Rd. and 200' south of Lyon Cir. (1130' - 10")	Increase pipeline capacity to reduce headloss	107,000
EX-14	Add parallel pipe in Santa Fe between Alta Calle and E43 Regulator (225' - 16")	Increase pipeline capacity to reduce headloss	40,000
EX-15	This improvement is not needed		
EX-16	Add parallel pipe in the east-west portion of Anna Ln. (200' - 8")	Increase pipeline capacity to reduce headloss	15,000
EX-17	Add parallel pipe between CW Regulator and the cul-de-sac in Watson Wy. (115' - 8")	Increase pipeline capacity to reduce headloss	9,000
EX-18	This improvement is not needed		
EX-19	New pressure regulating station from 837 => 810 Zone	Separate the 837 zone from the 810 zone to facilitate supply from VID11; Increase system redundancy	119,000
EX-20	This improvement is not needed		
EX-21	Replace existing 6" and 8" pipes in S. Santa Fe Av. with 12" between Cypress Dr. and York Dr., and parallel existing 10" pipes from York Dr. to Buena Creek Rd. (7200' - 12")	Added to provide additional transmission capacity to and from the 717 Zone, which is necessary in lieu of A Reservoir	815,000
EX-22	Construct new 16" pipe in Buena Creek Rd. between S. Santa Fe Av. and Robelini Dr. (1660' - 16")	Added to provide additional transmission capacity to and from the 717 Zone, which is necessary in lieu of A Reservoir	295,000
EX-23	New pressure regulating station from 837 => 717 Zone at VID 9	Provide supply to 717 Zone at VID9 to replace flows from the abandoned VID8 turnout	119,000
		OPINION OF TOTAL PROBABLE CONSTRUCTION COSTS =	**$12,022,000**

Note: Construction costs are order of magnitude estimates and include:
1) engineering, administration, land acquisition, legal, permitting (20%)
2) construction management (10%)
3) contingency (20%)

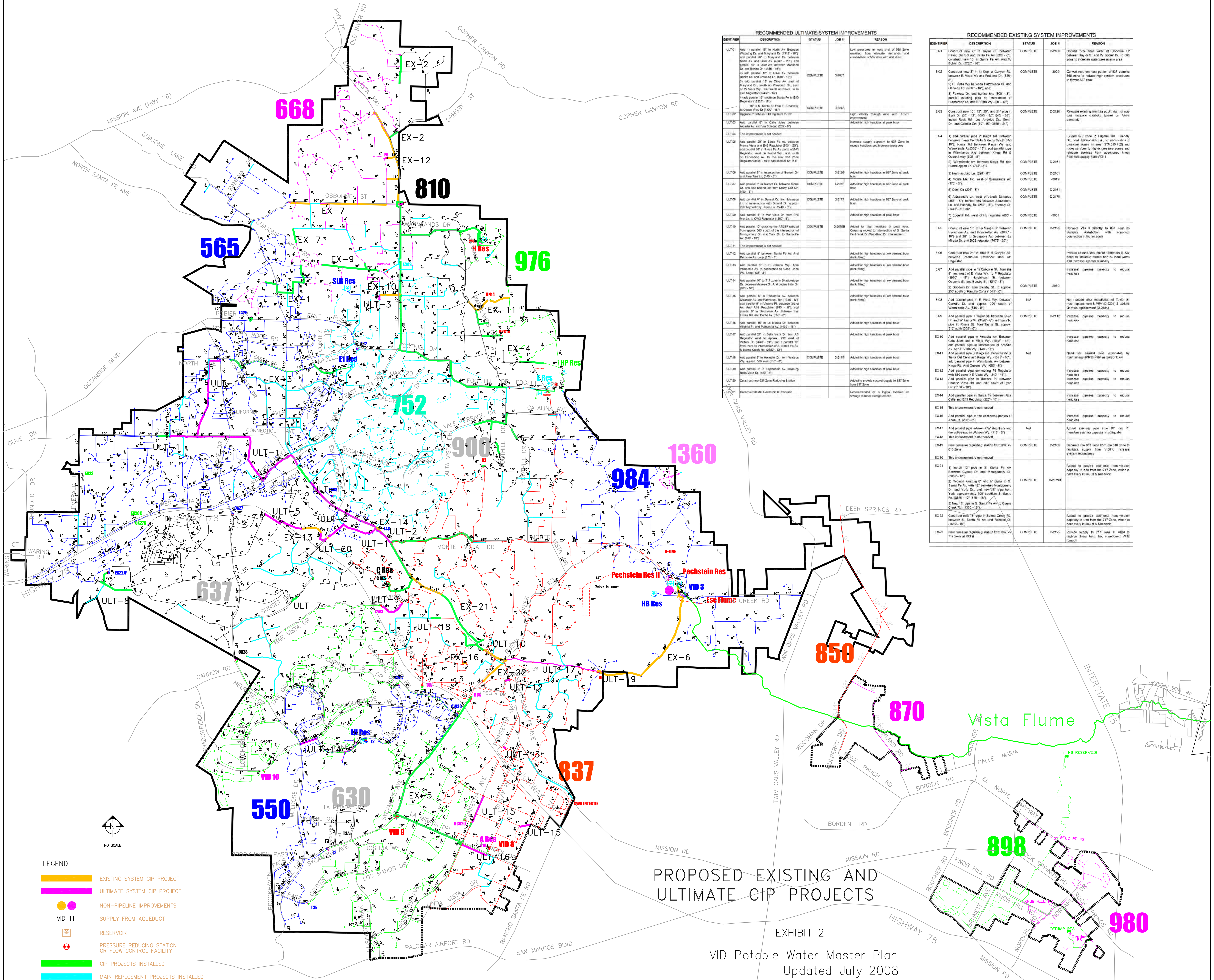
TABLE 8-1
RECOMMENDED ULTIMATE SYSTEM IMPROVEMENTS

IDENTIFIER	DESCRIPTION	REASON	CONSTRUCTION COST ($)
ULT-01	Add parallel 16" in North Av. Between Waxwing Dr. and Maryland Dr. (1315' - 16"); add parallel 20" in Maryland Dr. between North Av. and Olive Av. (4080' - 20"); add parallel 16" in Olive Av. Between Maryland Dr. and Bonita Dr. (1455' - 16"); add parallel 12" in Olive Av. between Bonita Dr. and Brookins Ln. (610' - 12"); add parallel 16" in Olive Av. east of Maryland Dr., south on Plymouth Dr., east on W Vista Wy., and south on Santa Fe to E43 Regulator (13435' - 16")	Low pressures in west end of 565 Zone resulting from ultimate demands and combination of 565 Zone with 486 Zone	3,849,000
ULT-02	Upgrade 8" valve in E43 regulator to 10"	High velocity through valve with ULT-01 improvement	119,000
ULT-03	Add parallel 8" in Calle Jules between Arcadia Av. and Via Soledad (255' - 8")	Added for high headloss at peak hour	19,000
ULT-04	This improvement is not needed		
ULT-05	Add parallel 20" in Santa Fe Av. between Monte Vista and E43 Regulator (800' - 20"); add parallel 16" in Santa Fe Av. north of E43 Regulator, west on Postal Wy., and south on Escondido Av. to the new 637 Zone Regulator (3155' - 16"); add parallel 12" in Escondido Av. south of the new 637 Zone Regulator to Lado de Loma Dr. (480' - 12"); add parallel 10" in Lado de Loma Dr. from Escondido Av. to 200' south of Lyon Cir. (665' - 10"); add parallel 8" in Bandini Pl. from Rancho Vista Rd. crossing Hwy 78 to Hacienda Dr. and terminating at Hacienda Dr. and Matagual Dr. (2290' - 8")	Increase supply capacity to 637 Zone to reduce headloss and increase pressures	1,029,000
ULT-06	Add parallel 8" in intersection of Sunset Dr. and Pine Tree Ln. (140' - 8")	Added for high headloss in 637 Zone at peak hour	11,000
ULT-07	Add parallel 8" in Sunset Dr. between Sierra Ct. and pipe behind lots from Crazy Colt Cir. (490' - 8")	Added for high headloss in 637 Zone at peak hour	37,000
ULT-08	Add parallel 8" in Sunset Dr. from Marazon Ln. to intersection with Sunset Dr. approx. 230' beyond Sky Haven Ln. (2740' - 8")	Added for high headloss in 637 Zone at peak hour	208,000
ULT-09	Add parallel 8" in Mar Vista Dr. from Phil Mar Ln. to CW3 Regulator (1360' - 8")	Added for high headloss at peak hour	103,000

TABLE 8-1
RECOMMENDED ULTIMATE SYSTEM IMPROVEMENTS
(continued)

IDENTIFIER	DESCRIPTION	REASON	CONSTRUCTION COST ($)
ULT-10	Add parallel 10" crossing the AT&SF railroad from approx 560' south of the intersection of Montgomery Dr. and York Dr. to Santa Fe Av. (180' - 10")	Added for high headloss at peak hour	17,000
ULT-11	This improvement is not needed		
ULT-12	Add parallel 8" between Santa Fe Av. And Primrose Av. Loop (275' - 8")	Added for high headloss at low demand hour (tank filling)	21,000
ULT-13	Add parallel 8" in El Sereno Wy. from Poinsettia Av. to connection to Casa Linda Wy. Loop (135' - 8")	Added for high headloss at low demand hour (tank filling)	10,000
ULT-14	Add parallel 16" to 717 zone in Shadowridge Dr. between Melrose Dr. And Lupine Hills Dr. (985' - 16")	Added for high headloss at low demand hour (tank filling)	175,000
ULT-15	Add parallel 8" in Poinsettia Av. between Oleander Av. and Palmcrest Ter. (1735' - 8"); add parallel 8" in Virginia Pl. between Grand Av. And A18 Regulator (745' - 8"); add parallel 8" in Descanso Av. Between Las Flores Rd. and Ponte Av. (650' - 8")	Added for high headloss at low demand hour (tank filling)	181,000
ULT-16	Add parallel 16" in La Mirada Dr. between Virginia Pl. and Poinsettia Av. (1430' - 16")	Added for high headloss at peak hour	253,000
ULT-17	Add parallel 24" in Bella Vista Dr. from AB Regulator west to approx. 730' east of Victory Dr. (2640' - 24"), and a parallel 12" from there to intersection of S. Santa Fe Av. & Buena Creek Rd. (2180' - 12")	Added for high headloss at peak hour	951,000
ULT-18	Add parallel 8" in Hannalei Dr. from Watson Wy. approx. 500' east (515' - 8")	Added for high headloss at peak hour	39,000
ULT-19	Add parallel 8" in Esplendido Av. crossing Bella Vista Dr. (120' - 8")	Added for high headloss at peak hour	9,000
ULT-20	Construct new 637 Zone Reducing Station	Added to provide second supply to 637 Zone from 837 Zone	119,000
ULT-21	Construct 20 MG Pechstein II Reservoir	Recommended as a logical location for storage to meet storage criteria	19,008,000
OPINION OF TOTAL PROBABLE CONSTRUCTION COSTS			**$26,158,000**

Note: Construction costs are order of magnitude estimates and include:
1) engineering, administration, land acquisition, legal, permitting (20%)
2) construction management (10%)
3) contingency (20%)



RECOMMENDED ULTIMATE SYSTEM IMPROVEMENTS

IDENTIFIER	DESCRIPTION	STATUS	JOB #	REASON
ULT-01	Add 1) parallel 16" in North Av. Between Waxwing Dr. and Maryland Dr. (1315' - 16"); add parallel 20" in Maryland Dr. between North Av. and Olive Av. (4080' – 20"); add parallel 16" in Olive Av. Between Maryland Dr. and Bonita Dr. (1455' - 16"); 2) add parallel 12" in Olive Av. between Bonita Dr. and Brookins Ln. (610' - 12"); 3) add parallel 16" in Olive Av. east of Maryland Dr., south on Plymouth Dr., east on W Vista Wy., and south on Santa Fe to E43 Regulator (13435' - 16") 4) add parallel 16" south on Santa Fe to E43 Regulator (12335' - 16"): - 18" in S. Santa Fe from E. Broadway to Ocean View Dr (1100' - 18")	COMPLETE COMPLETE	D-2087 D-2242	Low pressures in west end of 565 Zone resulting from ultimate demands and combination of 565 Zone with 486 Zone
ULT-02	Upgrade 8" valve in E43 regulator to 10"			High velocity through valve with ULT-01 improvement
ULT-03	Add parallel 8" in Calle Jules between Arcadia Av. and Via Soledad (255' - 8")			Added for high headloss at peak hour
ULT-04	This improvement is not needed			
ULT-05	Add parallel 20" in Santa Fe Av. between Monte Vista and E43 Regulator (800' - 20"); add parallel 16" in Santa Fe Av. north of E43 Regulator, west on Postal Wy., and south on Escondido Av. to the new 637 Zone Regulator (3155' - 16"); add parallel 12" in E			Increase supply capacity to 637 Zone to reduce headloss and increase pressures
ULT-06	Add parallel 8" in intersection of Sunset Dr. and Pine Tree Ln. (140' - 8")	COMPLETE	D-2126	Added for high headloss in 637 Zone at peak hour
ULT-07	Add parallel 8" in Sunset Dr. between Sierra Ct. and pipe behind lots from Crazy Colt Cir. (490' - 8")	COMPLETE	I-2928	Added for high headloss in 637 Zone at peak hour
ULT-08	Add parallel 8" in Sunset Dr. from Marazon Ln. to intersection with Sunset Dr. approx. 230' beyond Sky Haven Ln. (2740' - 8")	COMPLETE	D-2173	Added for high headloss in 637 Zone at peak hour
ULT-09	Add parallel 8" in Mar Vista Dr. from Phil Mar Ln. to CW3 Regulator (1360' - 8")			Added for high headloss at peak hour
ULT-10	Add parallel 10" crossing the AT&SF railroad from approx 560' south of the intersection of Montgomery Dr. and York Dr. to Santa Fe Av. (180' - 10")	COMPLETE	D-2079B	Added for high headloss at peak hour. Crossing moved to intersection of S. Santa Fe & York Dr./Woodland Dr. intersection.
ULT-11	This improvement is not needed			
ULT-12	Add parallel 8" between Santa Fe Av. And Primrose Av. Loop (275' - 8")			Added for high headloss at low demand hour (tank filling)
ULT-13	Add parallel 8" in El Sereno Wy. from Poinsettia Av. to connection to Casa Linda Wy. Loop (135' – 8")			Added for high headloss at low demand hour (tank filling)
ULT-14	Add parallel 16" to 717 zone in Shadowridge Dr. between Melrose Dr. And Lupine Hills Dr. (985' - 16")			Added for high headloss at low demand hour (tank filling)
ULT-15	Add parallel 8" in Poinsettia Av. between Oleander Av. and Palmcrest Ter. (1735' - 8"); add parallel 8" in Virginia Pl. between Grand Av. And A18 Regulator (745' – 8"); add parallel 8" in Descanso Av. Between Las Flores Rd. and Ponte Av. (650' - 8")			Added for high headloss at low demand hour (tank filling)
ULT-16	Add parallel 16" in La Mirada Dr. between Virginia Pl. and Poinsettia Av. (1430' - 16")			Added for high headloss at peak hour
ULT-17	Add parallel 24" in Bella Vista Dr. from AB Regulator west to approx. 730' east of Victory Dr. (2640' - 24"), and a parallel 12" from there to intersection of S. Santa Fe Av. & Buena Creek Rd. (2180' - 12")			Added for high headloss at peak hour
ULT-18	Add parallel 8" in Hannalei Dr. from Watson Wy. approx. 500' east (515' - 8")	COMPLETE	D-2135	Added for high headloss at peak hour
ULT-19	Add parallel 8" in Esplendido Av. crossing Bella Vista Dr. (120' - 8")			Added for high headloss at peak hour
ULT-20	Construct new 637 Zone Reducing Station			Added to provide second supply to 637 Zone from 637 Zone
ULT-21	Construct 20 MG Pechstein II Reservoir			Recommended as a logical location for storage to meet storage criteria

RECOMMENDED EXISTING SYSTEM IMPROVEMENTS

IDENTIFIER	DESCRIPTION	STATUS	JOB #	REASON
EX-1	Construct new 8" in Taylor St. between Paseo Del Sol and Santa Fe Av. (690' - 8"); construct new 10" in Santa Fe Av. And W Bobier Dr. (5725' - 10")	COMPLETE	D-2108	Convert 565 zone west of Goodwin Dr between Taylor St and W Bobier Dr. to 668 zone to increase water pressure in area
EX-2	Construct new 8" in 1) Gopher Canyon Rd. between E. Vista Wy and Fruitland Dr. (530' - 8"), 2) E. Vista Wy between Hutchinson St. and Osborne St. (5740' – 16"), and 3) Fairview Dr. and behind lots (600' - 8"); parallel existing pipe at intersection of Hutchinson St. and E Vista Wy. (90' - 12")	COMPLETE	I-3002	Convert northernmost portion of 837 zone to 668 zone to reduce high system pressures in former 837 zone
EX-3	Construct new 10", 12", 20", and 24" pipe in East Dr. (35' - 12", 4085' - 20", 645' - 24"), Indian Rock Rd., Los Angeles Dr., Smith Dr., and Cabrillo Cir. (60' - 10", 3660' - 24")	COMPLETE	D-2120	Relocate existing line into public right of way and increase capacity based on future demands
EX-4	1) add parallel pipe in Kings Rd between between Tierra Del Ceilo & Kings Wy (1525' - 10"); Kings Rd. between Kings Wy and Warmlands Av (385' - 12"); add parallel pipe in Warmlands Ave between Kings Rd & Queens way (605' - 8") 2) Warmlands Av. between Kings Rd and Hummingbird Ln. (745' - 8"); 3) Hummingbird Ln. (520' - 8") 4) Monte Mar Rd. east of Warmlands Av. (375' - 8"); 5) Odell Cir (295' - 8"); 6) Alessandro Ln. west of Vereda Barranca (855' - 8"), behind lots between Alessandro Ln. and Friendly Dr. (280' - 8"), Friendly Dr. (1445' - 8"); and 7) Edgehill Rd. west of HL regulator (600' - 8")	 COMPLETE COMPLETE COMPLETE COMPLETE COMPLETE COMPLETE	 D-2161 D-2161 I-3019 D-2161 D-2179 I-3051	Extend 976 zone to Edgehill Rd., Friendly Dr., and Alessandro Ln., to consolidate 3 pressure zones in area (976,810,752) and move services to higher pressure zones and relocate services from abandoned lines; Facilitate supply from VID11
EX-5	Construct new 16" in La Mirada Dr. between Sycamore Av. and Poinsettia Av. (3860' - 16") and 20" in Sycamore Av. between La Mirada Dr. and BCS regulator (7675' - 20")	COMPLETE	D-2125	Connect VID 9 directly to 837 zone to facilitate distribution with aqueduct connection in higher zone
EX-6	Construct new 24" in Blue Bird Canyon Rd. between Pechstein Reservoir and AB Regulator			Provide second feed out of Pechstein to 837 zone to facilitate distribution of local water and increase system reliability
EX-7	Add parallel pipe in 1) Osborne St. from the 9" line west of E Vista Wy. to F Regulator (3990' - 8"); Hutchinson St. between Osborne St. and Barsby St. (1310' - 8"), 2) Goodwin Dr. from Barsby St. to approx. 250' south of Rancho Corte (1045' - 8")	COMPLETE	I-2980	Increase pipeline capacity to reduce headloss
EX-8	Add parallel pipe in E. Vista Wy. between Corralla Dr. and approx. 200' south of Warmlands Av. (545' - 8")	N/A		Not needed after installation of Taylor St main replacement & PRV (D-2204) & Larkhill Dr main replacement (D-2184)
EX-9	Add parallel pipe in Taylor St. between Kevin Dr. and W Taylor St. (3390' - 8"); add parallel pipe in Rivera St. from Taylor St. approx. 315' north (355' - 8")	COMPLETE	D-2112	Increase pipeline capacity to reduce headloss
EX-10	Add parallel pipe in Arcadia Av. Between Cale Jules and E Vista Wy. (1025' - 12"); add parallel pipe in intersection of Arcadia Av. And E Vista Wy. (140' - 16")			Increase pipeline capacity to reduce headloss
EX-11	Add parallel pipe in Kings Rd. between Vista Tierra Del Cielo and Kings Wy. (1525' - 10"); add parallel pipe in Warmlands Av. between Kings Rd. And Queens Wy. (605' - 8")	N/A		Need for parallel pipe eliminated by maintaining HPR16 PRV as part of EX-4
EX-12	Add parallel pipe connecting F6 Regulator with 810 zone in E Vista Wy. (340' - 16")			Increase pipeline capacity to reduce headloss
EX-13	Add parallel pipe in Bandini Pl. between Rancho Vista Rd. and 200' south of Lyon Cir. (1130' - 10")			Increase pipeline capacity to reduce headloss
EX-14	Add parallel pipe in Santa Fe between Alta Calle and E43 Regulator (225' - 16")			Increase pipeline capacity to reduce headloss
EX-15	This improvement is not needed			
EX-16	Add parallel pipe in the east-west portion of Anna Ln. (200' - 8")			Increase pipeline capacity to reduce headloss
EX-17	Add parallel pipe between CW Regulator and the cul-de-sac in Watson Wy. (115' - 8")	N/A		Actual existing pipe size 10" not 8", therefore existing capacity is adequate
EX-18	This improvement is not needed			
EX-19	New pressure regulating station from 837 => 810 Zone	COMPLETE	D-2169	Separate the 837 zone from the 810 zone to facilitate supply from VID11; Increase system redundancy
EX-20	This improvement is not needed			
EX-21	1) Install 12" pipe in S. Santa Fe Av. Between Cypres Dr. and Montgomery Dr. (2050' - 12") 2) Replace existing 6" and 8" pipes in S. Santa Fe Av. with 12" between Montgomery Dr. and York Dr., and new 18" pipe from York approximately 500' south in S. Santa Fe. (3125' - 12", 625' - 18") 3) New 18" pipe in S. Santa Fe Av. to Buena Creek Rd. (1350 - 18")	COMPLETE	D-2079B	Added to provide additional transmission capacity to and from the 717 Zone, which is necessary in lieu of A Reservoir
EX-22	Construct new 16" pipe in Buena Creek Rd. between S. Santa Fe Av. and Robelini Dr. (1660' - 16")			Added to provide additional transmission capacity to and from the 717 Zone, which is necessary in lieu of A Reservoir
EX-23	New pressure regulating station from 837 => 717 Zone at VID 9	COMPLETE	D-2125	Provide supply to 717 Zone at VID9 to replace flows from the abandoned VID8 turnout

LEGEND

EXISTING SYSTEM CIP PROJECT
ULTIMATE SYSTEM CIP PROJECT
NON-PIPELINE IMPROVEMENTS
VID 11 SUPPLY FROM AQUEDUCT
RESERVOIR
PRESSURE REDUCING STATION OR FLOW CONTROL FACILITY
CIP PROJECTS INSTALLED
MAIN REPLCEMENT PROJECTS INSTALLED

PROPOSED EXISTING AND ULTIMATE CIP PROJECTS

EXHIBIT 2

VID Potable Water Master Plan
Updated July 2008