JACOBS

Lake Havasu City Water Master Plan

Final Draft

April 2019

City of Lake Havasu

LAKE HAVASU CITY

Acknowledgements

Jacobs (Legacy CH2M) would like to express its sincere appreciation for the assistance and cooperation provided by the management and staff of the City of Lake Havasu City during the preparation of this Master Plan Update. In particular, the efforts of following individuals are acknowledged and greatly appreciated.

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Acronyms and Abbreviations

acre-feet/yr	ac-ft/yr
American Water Works Association	AWWA
Arizona Department of Administration	ADOA
Arizona Department of Resources	ADWR
Arizona Revised Statues	ARS
Asbestos Cement	AC
Association for Advancement of Cost Engineering	AACE
Average Day Demand	ADD
Belt Filter Press	BFP
Booster Pump Station	BPS
Capital Improvement Plan	CIP
Cement Mortar Lined Steel	CML&C
Chlorine Contact Basin	CCB
Computerized Maintenance Management System	CMMS
Consequence of Failure	COF
Disinfection Byproducts	DBP
Feet Per Second	fps
Gallons Per Day	gpcd
Gallons Per Minutes	gpm
Havasu Riviera	HR
Health & Safety	H&S
High Water Levels	HWLs
Horizontal Collector Well	HCW
Hydraulic Grade Line	HGL
Lake Havasu City	City
Likelihood of Failure	LOF
Max Day Demand	MDD
Metropolitan Planning Organization	MPO
Million Gallons per day	MGD
Peak Hour Demand	PHD
per square inch	psi
Polyvinyl Chloride	PVC
Pressure Reducing Station	PRS
Pressure Reducing Valve	PRV
Programmable Logic Controller	PLC
Pump Station	PS
Total Dissolved Solids	TDS
Trihalomethanes	THM
Ultraviolet	UV
Wastewater Treatment Plant	WWTP
Water Conservation Assessment	WCA
Water Treatment Plant	WTP

Overview

This section introduces the master plan report, purpose, scope of work, and summarizes prior master plans prepared for the City of Lake Havasu City, Arizona.

1.1 Introduction and Purpose

Lake Havasu City (City) is located along the Colorado River and is situated along Lake Havasu in the west central area of the State of Arizona. The City was designed as a master planned community in 1963 by Mr. Robert McCulloch with an emphasis on recreation and residential development. As such, the City experiences a tremendous influx of seasonal and weekend visitors through the year resulting in a large transient population that can impact the water and wastewater systems.

Source: Lake Havasu City Flickr Album

The City operates a water distribution system consisting of 7 major pressure zones, 14 booster pump stations (BPS), 26 reservoirs, 7 system pressure reducing stations, 9 wells, 1 horizontal collector well, and nearly 500 miles of pipe, ranging in diameter from 4 inches to 48 inches. The City serves water to approximately 54,000 people.

The Water Master Plan is one of many documents that are used to plan for future infrastructure needs of the City to ensure a reliable water supply and service to all customers throughout the year. Utility master plans are typically prepared every 5 to 10 years depending on a community's growth and land use, changes in water supply and demand, aging infrastructure, and regulatory and financial requirements. The City has typically followed a 10-year cycle on master plan preparation.

A water master plan was completed in 1992 which laid the foundation for development of the current water supply and water treatment plant. In 2007, the City completed a comprehensive water master plan that looked at entire water supply and distribution system needs to meet projected demands out to 2050. The 2007 Water Master Plan was based on an existing demand of 8.2 million gallons per day (MGD). A buildout population of 96,000 people was also assumed in 2007.

The 2018 Water Master Plan is an update to the 2007 Water Master Plan. The primary purpose is to update the water demand forecast considering current water use trends and conservation, revised population forecast based on a new General Plan, and develop near-term and identify long-term capital improvement projects. A critical element and focus of the 2018 Water Master Plan is to address water supply capacity, reliability, and vulnerability, given the condition of the City's existing North Wellfield. The 2018 Water Master Plan also incorporates recent technical studies prepared for the City including recent BPSs and tank condition assessments, water reuse planning, developer studies, and an updated water system hydraulic model.

1.2 Scope

The 2018 Water Master Plan scope of work focuses on water supply resources and reliability and water distribution system upgrades to meet existing water demands. Moreover, critical for the City is to address redundancy, risk, and consequence of failure for its major water supply, a horizontal collector well (HCW) and long-term sustainability of the North Wellfield to meet the City's future water supply needs. An evaluation of future water treatment requirements in terms of capacity, rehabilitation, and reliability is also included in this master plan.

A summary of the 2018 Water Master Plan scope of work includes the following:

- Review system design criteria
- Develop water demand projections
- Evaluate water supply sources and capacities
- Assess storage and pumping
- Update hydraulic water model
- Summarize condition assessment efforts
- Present detailed 5-year Capital Improvement Plan (CIP) and identify future CIPs through 2040

The 2018 Water Master Plan will serve as a basis for an updated, prioritized CIP and system-wide recommendations for the City over the next 5 to 20 years.

1.3 Recent Master Plans

1.3.1 Water

The 2007 Water Master Plan was a comprehensive review of distribution pumping, and storage, and developed a long-range capital program. The plan addressed the City's water system capacity including BPSs, reservoirs, and supply wells. A hydraulic model (H20Map by Innovyze) of the water distribution system was developed as part of the 2007 Water Master Plan and has subsequently been updated in 2012 and most recently in 2017.

The 2007 Water Master Plan forecasted average annual water demands of 18.3 MGD and 25.5 MGD, and maximum day demands of 32.2 MGD and 44.8 MGD by 2025 and 2040, respectively. At the time, per capita water use was substantially higher than today, approximately 233 gpcd, and the City's population forecast for 2025 was to be 74,000 (Lake Havasu City, 2007). The 2018 Water Master Plan estimates a 2025 population of 58,570, reflective of the recent economic downturn and slower growth in housing. With nearly 15,000 less people by 2025 and much lower per capita water use, the need for a master plan update is timely to evaluate distribution capacity needs in-light of less water being required.

1.3.2 Wastewater

In 2014 the City completed the Wastewater System Expansion Program Oversight Finalization (2014 Wastewater Report) (Carollo, 2014) that updated unit sewer generation rates per capita and revised future flow projections based on the latest population projections. The 2014 Wastewater Report was commissioned to address the performance and capacity of the entire sewer collection system with the completion of the City's septic to sewer program and with several years of operating data.

Projected 2024 wastewater flows were 4.6 MGD based on a unit sewer generation rate of 77 gpdpc. The 2014 Wastewater Report concluded that the wastewater system continues to experience lower sewer flows than designed and has components of the system with excess capacity. Accordingly, no major capital improvements were recommended on the wastewater side.

The City is currently being served by three wastewater treatment plants (WWTPs). The Island WWTP with an ultimate design capacity of 2.5 MGD and the Mulberry WWTP with ultimate design capacity of 2.2 MGD are the older wastewater facilities located in the heart of the City. The third and newest WWTP, the North Regional WWTP, has a current capacity of 3.5 MGD and is located in the far north portions of the City. The location requires series pumping to convey flows to the plant headworks. This facility was designed to be expanded to an ultimate capacity of 14.0 MGD (Water Conservation Plan Lake Havasu City, 2015) based on the design criteria and population forecasts being used at that time.

Sewer collection infrastructure includes: nearly 350 miles of gravity sewer maintained and operated by the Wastewater Division, 25 miles of force main, 49 City owned wastewater BPSs, and 10 BPSs owned by private parties (Carollo, 2014).

Basis of Planning

This section describes and establishes the basis of planning for the water master plan including the water service area, land use information, population data, and a review of the local water source setting. The basis for the City's water demand forecast and seasonal peaking is also presented though an analysis of existing water consumption data and land use.

2.1 Water Service Area Description

2.1.1 Service Boundary

The adopted City 2016 General Plan documents the City's water service area, which defines the geographic boundary for the future water demand forecast. Referring to **Figure 2-1**, developable land within the water service areas is primarily within the City municipal boundary, and the remaining area is within Mohave County. The 2016 General Plan area boundary extends well beyond the water service area and City boundary. However, the population forecast through 2040 and associated projected water demand is assumed to occur within the water service boundary for the 2018 Master Plan Update.

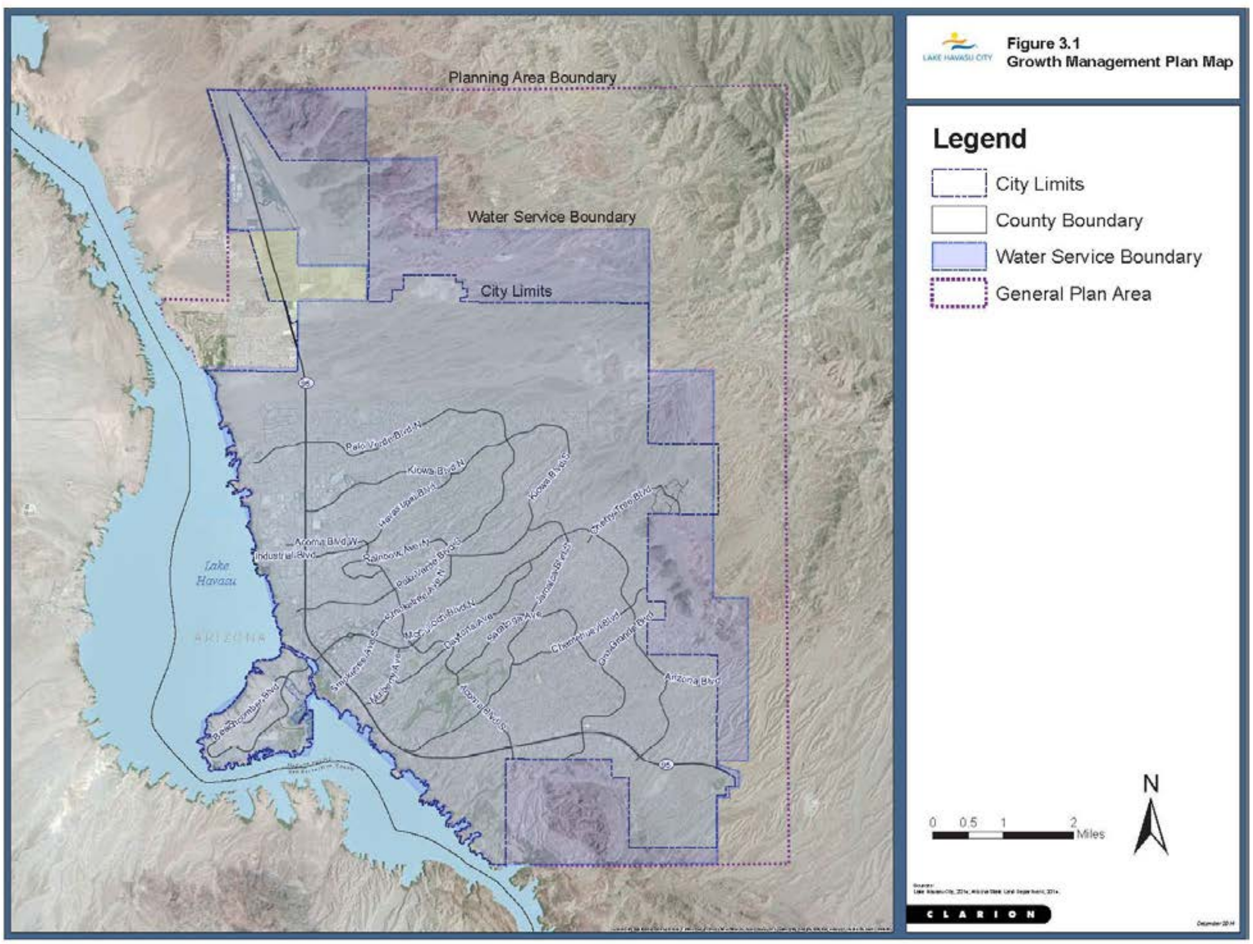


Figure 2-1. Map of the Lake Havasu City Service Area

Source: 2016 General Plan (Clarion 2016)

2.1.2 Setting

Lake Havasu City is situated along the eastern bank of the Colorado River and Lake Havasu, which is formed by Parker Dam. The City lies at the western foothills of the Mohave Mountains in the west central area of Arizona and is located approximately 200 miles from the Phoenix Metropolitan Area. The area is highly undulating with hills and washes. Due to the low precipitation in the region, the area is sparsely vegetated and has typical native desert terrain.

2.1.3 Climate

The region experiences over 300 days of sunshine with low humidity. The City experiences extreme heat in the summer, and summer temperatures range from 78°F to 120°F. In any single year, the City has experienced as many as 145 days over 100°F and 86 days over 110°F along with up to 32 days with overnight lows above 90°F. Winter temperatures range from 37°F to 68°F (Water Conservation Plan Lake Havasu City, 2015).

The region experiences low precipitation volumes. There have been years where less than 2 inches of rain were recorded. Precipitation usually occurs during the winter months, January to March. During the summer monsoon season, July to early September, scattered convective thunderstorms occur. The evapotranspiration rates are very high during the summers, and annual calculated volume losses exceed 7 feet. This results in a deficit in plant water requirements. Since the area lacks significant precipitation rates, native vegetation is sparse. Native vegetation consists of acacia, bursage, brittlebush, cacti, creosote mesquite, ocotillo, paloverde, and annual flowering weeds and grasses (Water Conservation Plan Lake Havasu City, 2015).

2.2 Land Use

2.2.1 General Plan

The 2016 Lake Havasu City General Plan (2016 General Plan) is a long-range plan to guide the future growth of the community. The Arizona Revised Statutes require that each city adopt a comprehensive, long-range General Plan to guide the community’s physical development. The purpose of the General Plan is to:

- Express the community’s vision
- Identify the community’s goals and development priorities
- Serve as a policy guide for local decision-making
- Fulfill legal requirements created by state law

The 2016 General Plan update is a statement of policy and an expression of the community’s vision for the future. The plan is a tool to help guide and shape the planning area’s physical development (Clarion, 2015).

Regardless of whether or not the City’s population remains above 50,000 people, simply exceeding this population threshold triggered a specific set of requirements under the Revised Statutes and Growing Smarter Act. The 2016 General Plan meets the requirements of Growing Smarter as well as the General Plan requirements outlined in Arizona Revised Statutes (ARS) 9- 461.05.

2.2.2 Existing Land Use

Existing management plans for Lake Havasu City establish land use categories within the City's Water Service Area. The land use is divided into the following categories: rural residential, low density, medium density, high density, resort, business/government, commercial, school, irrigation, and industrial. The major land uses within the City are residential, commercial/industrial, and recreation/resort and undeveloped lands.

The City is comprised of several different character areas: the originally platted residential neighborhoods; the tourism-based area along much of the Shoreline and on the Island; and the urban core, which consists of Downtown Lake Havasu and other commercial/employment areas that serve both tourists and local residents (Clarion, 2015). Residential areas are located throughout the City, and the commercial and industrial areas are concentrated in narrow strips that parallel the main traffic routes: Highway 95, Lake Havasu Avenue, North Kiowa Boulevard, and McCulloch Boulevard. The recreation and resort areas include the Lake shoreline, Island, and golf course facilities in the southwestern portion of the City. A majority of the undeveloped area is located in the northern and eastern parts of the City. The northern portion is separated by an unincorporated area that is served by EPCOR Utilities. The northern area lends itself to future growth based on its terrain, while the eastern area is characterized by steeper topography that may constrain its maximum development potential.

The federal and state-owned lands bordering the City also provide development constraints. Large portions of the island and the City are publicly owned. Although the City has conducted initial planning for certain areas, such as Body Beach and the Island, and these plans are reflected in the Future Land Use Plan for the City, they remain undeveloped because development approval is not within the City’s authority.

2.2.3 Buildout

Much of the vacant land within the City’s planning area is publicly owned. Therefore, most growth will occur through incremental infill and redevelopment. Primary opportunity areas for non-residential and mixed-use infill and redevelopment include Downtown Lake Havasu City, portions of the Highway 95 Corridor as it passes through the City, and the Bridgewater Channel area. Opportunities for residential infill exist throughout the originally platted area; however, many of the remaining lots have limited potential due to their size, ownership, physical characteristics, or location. (Clarion, 2015)

Tables 2-1 and **Table 2-2** below from the 2016 General Plan summarize the residential and non-residential maximum build-out potential by land use type within the City’s water service area boundary.

Table 2-1. Expanded Water Service Area: Residential Build-Out

Land Use	Average Density (du/ac)	Total Acres	Buildable Acres	Potential Dwelling Units
Rural Residential	1.25	5,673	3,971	4,964
Low Density Residential	3	15,908	11,136	33,407
Medium Density Residential	7	70	49	343
High Density Residential	15	1,182	946	13,416
Resort Residential	7	89	62	437
Resort Related Island*	-	656	-	-
Resort Related Mainland*	-	142	-	-
Total		23,641	16,050	52,130

Source: Page 43, Table 4.4 of LHC General Plan

Table 2-2. Expanded Water Service Area: Non-Residential Build-Out

Land Use	Average Density (FAR)	Total Acres	Buildable Acres	Gross Floor Area (square feet)
Neighborhood Commercial	0.22	67	47	452,447
Commercial/Mixed-Use (Nodal)	0.22	250	175	1,677,060
Commercial/Mixed Use	0.22	2,105	1,474	14,120,845
Employment	0.20	2,080	1,248	10,872,576
Resort	0.22	86	51	492,519
Resort Related	0.22	676	474	4,538,038
Resort Related Island*	-	656	-	-
Resort Related Mainland*	-	142	-	-
Total		6,575	3,837	32,105,279

Source: Page 43, Table 4.5 of LHC General Plan

2.2.4 Population / Population Projections

Population estimates between censuses for the City are completed by the State of Arizona Department of Administration (ADOA). The ADOA methodology for population estimates is much improved over past efforts by other state agencies, yet it does not account for seasonal population variations. Winter visitors in Lake Havasu City spend between 4 to 6 months in the warmer climate only to leave for the summer. During this period, the City's population grows significantly as many people will occupy second homes or stay in recreations vehicles. It was estimated in the 2015 Water Conservation Plan that this population increase may be as high as 50 percent or more at times. (Water Conservation Plan Lake Havasu City, 2015)

The references used in the development of the population projections are as follows:

- Arizona Department of Administration - https://population.az.gov/population-projections
- Lake Havasu City General Plan (2016)

Lake Havasu City's population has increased from 41,938 in 2000 to nearly 54,000 residents in 2016. According to the Arizona Office of Employment and Population Statistics, the City is projected to add around 14,000 additional residents by 2040. Population projections for Lake Havasu City indicate a slow, but steady increase of residents in Lake Havasu and Mohave County over the next 25 years.

The following section presents the population-based methodology to estimate water use for 2025, 2040, and Build-out. It can be challenging to predict future water use for the City with its seasonal population increase and influx of weekend visitors through the year. The City's current population and US Census data was interpolated, coupled with population data estimated in the City's Metropolitan Planning Organization (MPO) document for the City's water service area to generate population estimates.

The following data sources also served as the basis for the process of establishing the existing baseline water demand and the population-based future demand:

- City's water distribution hydraulic model (Atkins, 2017)
- 2015 Water Conservation Plan (City, 2015)
- Lake Havasu City MPO's 2040 Long Range Transportation Plan (2015)
- 2014 Wastewater Report (Carollo, 2014)
- US census data (2010), and
- The Lake Havasu City Reclaimed Water Management Study (2014)

These documents, in addition to City's input, were used for the population-based approach through the year 2040 and Build-out. Household projections from wastewater flow projections in the wastewater system expansion program and the MPO's Long-Range Transportation Plan were also reviewed to determine the growth potential in single-family and multi-family service connections.

Figure 2-2 shows population-based projected growth through 2040. The wastewater plan population forecast, assumed a growth rate of 1.5 percent from 2014 through 2025. Using the future population reported in the City's MPO document, the population growth was estimated to be 0.7 percent. from 2014 through 2040. With this growth rate, roughly 500 people are added each year between 2014 and 2025 and between 2025 and 2040, respectively. 5,400 and 8,100 people are added between 2014 and 2025 and between 2025 and 2040, respectively. **In summary, the growth between 2014 and year 2025**

would add approximately 2,350 new homes or 214 homes per year. Assuming 2.3 people per home, the growth between year 2025 and year 2040 would add approximately 3,500 new homes or 230 homes per year, which appears to be conservative based on recent trends in the City.

Figure 2-3 presents the overall City future population growth based on the build-out population. The City predicted "build-out" population for the City in the future would be 96,000 as stated in the City's 2016 General Plan. It was assumed that growth rate of 0.7 percent would be a conservative estimate. Based on 0.7% population growth, a long-range population forecast is shown from 2040 to the assumed 96,000 build-out population. The build out population is about a 30,000 increase in population from 2040. Based on an average increase of 500 people per year, it would be equivalent to another 60 years to build-out (year 2100).

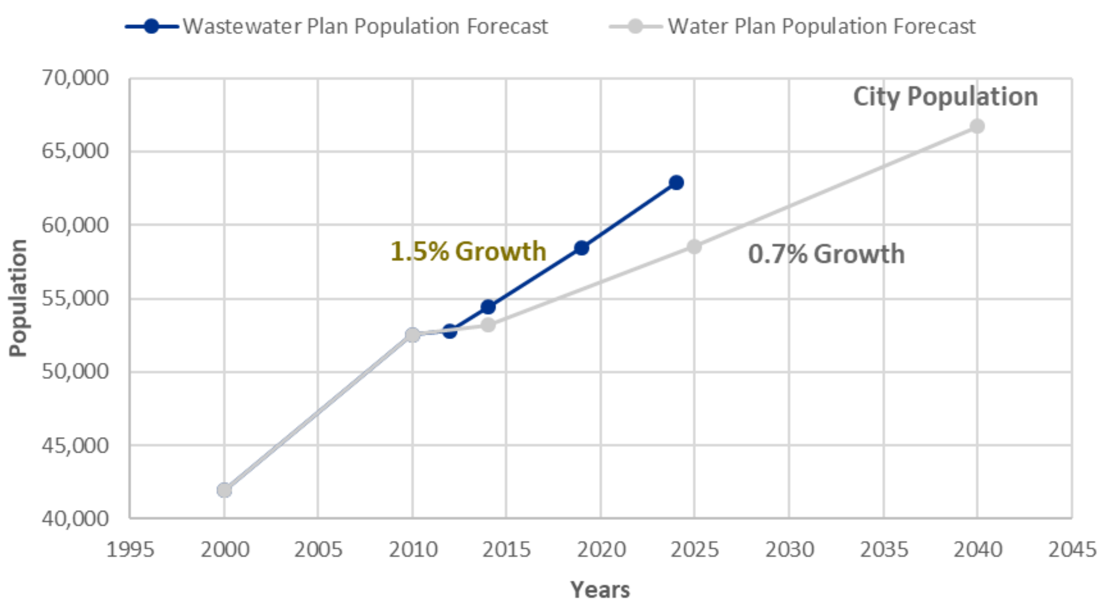


Figure 2-2. Master Plan Population Projections (2040)

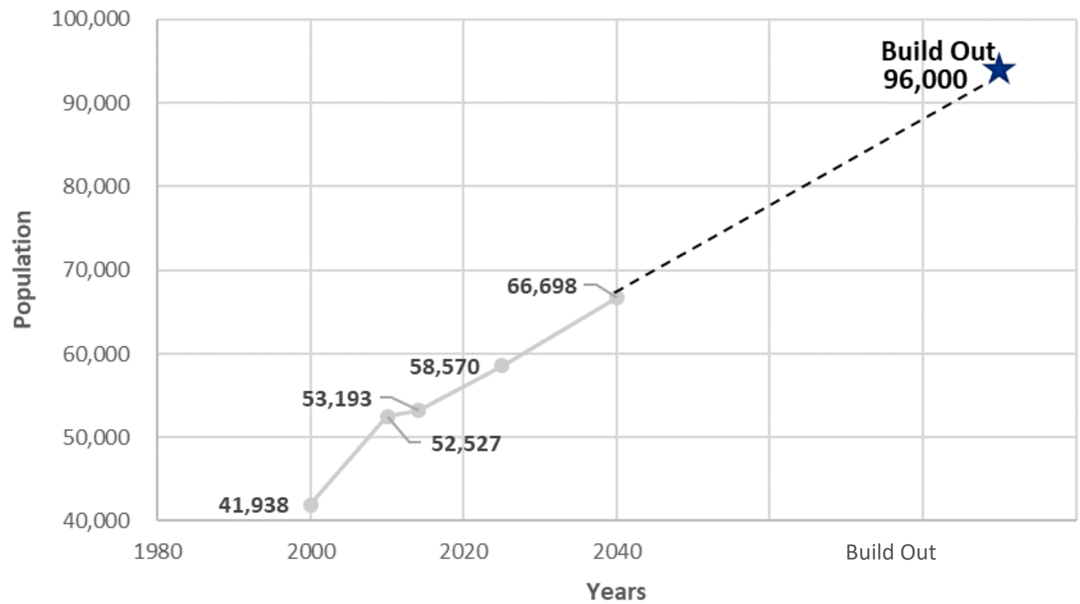


Figure 2-3. BUILDOUT Population Projections

2.3 Water Resources Overview

2.3.1 Water Supply

The principal water source for Lake Havasu City is contracted, 4th priority Colorado River water entitlements that total 28,581.7 acre-feet/year (ac-ft/yr). This amount is a combination of contracts from the Bureau of reclamation and with the Mohave County Water Authority, which provide volumes of 19,180 ac-ft/yr and 9,389 ac-ft/yr respectively. Additionally, 2,139 ac-ft of 4th priority Colorado River water was secured in 2009 by the City through another MCWA subcontract (Water Conservation Plan Lake Havasu City, 2015). Furthermore, a 12.7 ac-ft 4th priority allocation was transferred to the City in 2012 from a developer planning to construct a small marina on Lake Havasu. This water is exclusively reserved to compensate for lake surface evaporation due to the enlargement of the lake's surface area.

2.3.2 Groundwater

Lake Havasu City primarily diverts water from a 25 MGD capacity, sixteen-foot inside diameter, HCW. This well has the capability of producing up to 32 MGD over short, high demand periods. Nine conventional production wells located in two well fields on the northwest side of the City (7 wells) and on Pittsburg Island (2 wells) are kept in reserve for emergency use. Section 4 includes a detailed capacity evaluation and assessment of the City's well supply.

All wells penetrate the Colorado River Aquifer, which is hydrologically connected to the Colorado River/Lake Havasu. The City has private water wells for landscape irrigation and provide untreated water to a golf course and to a cemetery lawn, as well as City Hall. Raw lake water is seasonally withdrawn through a surface water intake (called the South Intake) to supplement effluent demands, stored at the City's Mulberry (WWTP). The mix of effluent and lake water is then pumped to the 36-hole London Bridge Golf Course for irrigation. The South Intake has typically provided between 1.0 and- 1.5 MGD of raw source water during the summer months for golf course irrigation.

2.3.3 Water Reuse

The City's practice of reusing treated wastewater (effluent) for landscape and turf irrigation has reduced the demand on the City's Colorado River water supply. This has resulted in the City lowering annual allocation requests. Approximately 2,020 ac-ft of effluent was sold in 2014 to irrigation customers, about one half of the City's total annual generated effluent. Effluent will play a larger water management role in the future as the City moves to convert public potable water irrigation systems to effluent.

Approximately 50 percent of the treated effluent from the City's three WWTPs (Island, Mulberry, and North Regional) is reused, and the remaining balance is either recharged in percolation or discharged to evaporation ponds. Virtually 100 percent of the treated effluent from the Mulberry WWTP is reused for golf course irrigation. The North Regional WWTP reuse is sent to the Refuge Golf Course and some of this effluent is injected into the subsurface through vadose zone wells for storage, any excess effluent is conveyed to the Island WWTP percolation ponds. Historically, the combined annual volume of effluent is between 1,600 and 2,000 ac-ft.

2.4 Design Criteria

This section summarizes the recommended water system design criteria for the master plan. **Table 2-3** below provides a summary of supply, distribution and reliability criteria for the Master Plan. The following subsections include a discussion on the major criteria.

Table 2-3. Potable Water Design Criteria

Description	Criteria
Water Supply • HCW • North Well Field	Max Day Demand + 10% (water loss)
Water Supply Reliability • HCW • North Well Field	Provide redundancy in the event of loss of the HCW supply. Range of back-up water supply from additional wells should range from: • Max: 100% Max Month Supply • Min: 100% Avg Day Supply
WTP Supply/Production	Max Day Demand +10% (water loss)
Peaking Factors	
Minimum Day/Average Day Ratio	0.7
Maximum Day/Average Day Ratio	1.5
Maximum Month/Average Day Ratio	1.2
Peak Hour/Average Day Ratio	2.5

Table 2-3. Potable Water Design Criteria (Continued)

Storage Criteria	Sum of the following:
Operational	20% of Maximum Day Demand
Fire	Based on largest Fire Zone (see Fire criteria below)
Emergency	100% of Average Day Demand
Transmission/Distribution Pipeline Criteria	
Maximum Velocity – Max Day Demand	
Pipe < 36"	5 fps
Pipe ≥ 36"	6 fps
Maximum Velocity – Peak Hour Demand	7 fps
Pressures Criteria	
Maximum Velocity – Fire Flow	15 fps
Maximum Headloss – Peak Hour Demand	10 ft/1,000 ft
Minimum Residual Pressure – Fire Flow	20 psi
Minimum Residual Pressure – Peak Hour Demand	40 psi
Minimum Static Pressure	50 psi
Minimum Desired Static Pressure (New Development)	≥ 60 psi
Booster Pump Station Criteria	
Without Storage	Capacity equal to larger of Peak Hour or Max Day Demand + Fire Flow
With Adequate Storage	Capacity equal to Max Day Demand (MDD)
Firm Capacity	Capacity with single pump (or largest pump) out of service
Pressure Zone Supply Reliability	Pressure Zones with three (3) or more BPS supply Average Day Demands (ADD) with one station out of service
Booster Station	Minimum of three (3) equally sized pumps. Back-up generator supplies sufficient power for firm capacity
Fire Demand Criteria	Based on International Fire Code. Fire flow credit is allotted for sprinklered buildings. A minimum fire flow of 1,500 gpm is required if sprinkler systems are not installed.

2.4.1 Water Supply

Referring to **Table 2-3**, water supply facilities including wells and the WTP must be sized to meet the maximum day demand for the year and is consistent with AWWA supply guidelines. A water loss allowance is also included as part of the design criteria. The City has made a high priority to provide redundancy should the City lose its major water supply source, the HCW.

2.4.2 Pipelines

Water system piping serves three basic purposes:

- To transfer water from the source of production to storage.
- To distribute water from the source or storage to the consumer.
- To provide a conduit to supply firefighting water.

Transmission and distribution mains are sized for the greater of the following two demand conditions:

- Maximum day demand plus fire flow, or
- Peak hour demand.

Pipeline sizing criteria are established in order to minimize system head loss, optimize pumping energy requirements, reduce scouring of pipeline interior protective coatings, and minimize wear on in-line valves. This is especially important for large Cement Mortar-lined and Coated Steel or Ductile Iron transmission mains. Pipeline velocities are limited to 7 feet per second (fps) for all operating conditions, except maximum day plus fire, in which case velocities may not exceed 15 fps. Typically, transmission mains are sized under peak hour conditions, while distribution mains are sized for maximum day plus fire flow. Looping is desired, where applicable, to maintain water quality and reliability. In special circumstances, piping facilities may operate outside maximum ranges if minimum residual pressures are met and conditional approval is obtained from the City.

2.4.3 Peaking Factors

The demand peaking factors shown in **Table 2-3** are based on an analysis of current and historical City peak flows. The current criteria for minimum day, maximum day, and maximum month peaking factors of 0.7, 1.5 and 1.2, respectively, were validated based on a review of 2014 and 2015 data. The peak hour factor of 2.5 has been used in past City Master Plans and is consistent with industry standards for the size of water service area.

2.4.4 Fire Flow

Fire flow analysis is conducted to ensure adequate protection is provided during fire emergencies. In addition to supplying adequate flows, a minimum residual pressure of 20 pounds per square inch (psi) is required to maintain the integrity of the distribution system. Therefore, the City's water infrastructure will be evaluated to determine whether a minimum pressure of 20 psi will be maintained in a maximum day demand plus fire flow condition.

The Fire Marshal confirmed the City had adopted the International Fire Code for determining the fire flow requirements for new development. The Code also allows for a reduction in fire flow based on fire sprinklers being installed.

Fire storage would be calculated based on the largest fire flow and duration required in a pressure zone based on American Water Works Association (AWWA) M42 criteria. In most cases for the City, the largest fire flow will be commercial/industrial type uses based on large building square footages. Fire storage can typically be shared between reservoirs in the same pressure zone. For closed zones, fire storage is located in the nearest reservoir supplying the BPS.

2.4.5 Booster Pump Stations

Table 2-3 includes industry standard sizing criteria for BPS facilities, which is to provide a firm maximum day capacity based on available storage within a pressure zone with a single pump out of service. Peak hour and fire flow demands in excess of MDD are typically met from water stored in the reservoirs in that zone. Standby pumping units with capacity equal to the largest unit in a BPS and emergency backup power are required for each station. Closed BPSs must be able to deliver larger of peak hour or max day plus fire flow demand with the required standby capacity, including backup power.

2.4.6 Pressures

The following pressure criteria are recommended to assess the adequacy of the water transmission/distribution system under the two demand conditions:

- Peak Hour Demand: Pressures should be greater than 50 pounds psi. Pressures higher than 80 psi require an individual house pressure regulating on each service line per the City Building Code. Criteria are established to account for distribution system and backflow prevention facility head loss in order to achieve a minimum service pressure of 40 psi.
- Maximum Day Demand plus Fire-Flow Condition: A minimum of 20 psi at the point of maximum fire draft.

Minimum residual Pressure criteria for fire flow and peak hour demand is as follows:

- Fire Flow ≥ 20 psi
- Peak Hour Demand ≥ 40 psi

Minimum static pressure and desired static pressure criteria is as follows:

- Static pressure ≥ 50 psi
- Desired Static Pressure New Development ≥ 60 psi

2.4.7 Storage Facilities

This section includes a review of the 2007 storage criteria and presents the recommended 2019 storage criteria for the distribution system.

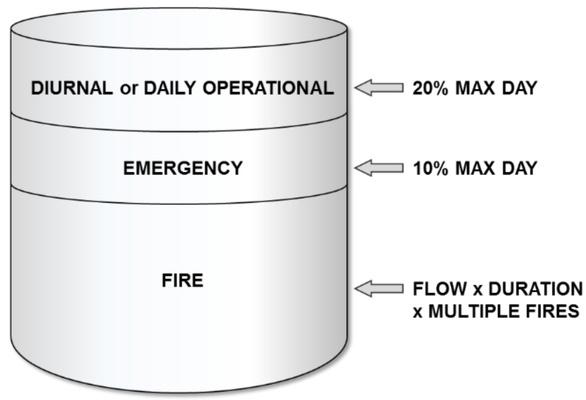
Storage Criteria

As shown in the graphic, the 2007 storage criteria had a heavy emphasis on fire storage resulting in:

- Over 75% of tank capacity was for fire storage
- Fire storage was driving future tank sizes
- Created water quality challenges for operators

Based on a review of AWWA Manual M42, storage should be sized based on three storage components:

- Operational
- Fire
- Emergency



2007 Master Plan Storage Criteria

Fire Storage

The 2007 Master Plan was found to be conservative based an insurance application standard that assumed multiple fires occurring simultaneously in a pressure zone based on service population. It is recommended that fire storage be provided for the largest fire flow and duration in each pressure zone, respectively. Fire storage is assumed to be shared amongst multiple tanks in a pressure zone.

Operational Storage

For the 2019 storage criteria, no change is recommended in operational storage based on a review of operational parameters and reservoir level pump settings.

Emergency Storage

Emergency storage is needed in the distribution system for:

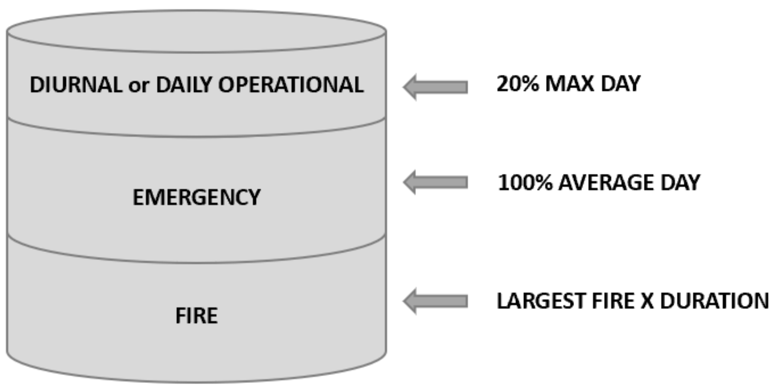
- Localized outage in the zone distribution system being served
- Lower pressure zone BPS outage
- Other tanks in the pressure zone are out of service
- Forebay storage for a BPS

Emergency storage is unique to every system and pressure zone, various industry standards ranges include:

- 100% Max Day Demand
- 100% Average Day Demand
- 50% Average Day Demand

Through discussions with City the engineering and operation staff, it was agreed that one average day demand would be sufficient in the distribution system under the 2019 storage criteria.

The recommended 2019 storage criteria is shown on the following graphic and includes operational, and the revised emergency and fire storage components.



2019 Water Master Plan Storage Criteria

The City does consider on a site by site basis based on the service zone characteristics whether it would be beneficial to split the required storage volume identified for a particular site into two tanks from an operational standpoint. In this manner, the City would have the flexibility to be able to take one tank out of service at a time for maintenance activities. This would also afford the option for operating only one tank during low demand in order to more closely manage system water quality.

2.4.8 Pressure Regulating Stations

The City owns and operates seven pressure reducing stations (PRSs) in the distribution system as shown in **Table 2-4**. Design criteria typically includes a main valve (sized for downstream zone fire flow) and smaller bypass valve (sized for average or peak demands). Recently, the City has equipped new water BPSs with a pressure reducing valve (PRV) to allow flow to be bypassed from the higher zone to the lower zone. This should be evaluated on a case by case basis for each new BPS project.

Table 2-4. Pressure Regulating Station Summary

LHC ID	Name	Zone	Valve Diameter (inch)		Setting (psi)		Elevation (ft)
			Primary	Secondary	Primary	Secondary	
1	North Havasu East	Zone 2- Isolated Zone 1 (Airport, Mall, Home Depot)	8	2	128	75	730
2	North Havasu West	Zone 2- Isolated Zone 1 (Airport, Mall, Home Depot)	10	4	128	78	730
3	C-Booster	Zone 1 - Island	10	6	148	82	460
4	Well 2	Zone 1 - Island	10	4	140	90	480
5	McCulloch	Zone 1 - Island	4	-	140	90	480
6	Vagabond	Zone 4 - Zone 3	6	-	156	90	1,040
7	Cherrytree	Zone 6 – Zone 5	8	2	150	57	1,435

Water Demand Development

Section 3 presents an evaluation of the City's existing water use and establishes a baseline existing demand to evaluate existing system capacity. Water use by end use and per capita is discussed in addition to seasonal variations and peaking factors. Water demands by pressure zone are summarized and a future demand forecast is presented for 2040 and Build-out.

3.1 Existing Baseline Demand

The Master Plan develops an existing demand or baseline scenario for the purposes of evaluating existing water system capacity and identifying potential water system deficiencies. Critical to this analysis is establishment of a "baseline" or existing demand condition. Water use per capita in the City has reduced the past few years with continued education efforts, water conservation measures, and water audits. In addition, the nature of the City's population results in variability from year to year and seasonally due to the high volume of winter visitors and frequent vacationers throughout the year

Shown above are several of the factors influencing water use and conservation

Key sources of information provided by the City served as the basis of the potable water use analysis to develop the existing baseline demand:

- Customer billing (sales) records for the past three years (2015-2017)
- Water production (supply) records for years 2013-2017
- City's updated water distribution hydraulic model Innovyze H2OMap v.10 *(Existing Water Distribution Model Update (Model Update) August 2017*

The customer billing records were reviewed for the past three calendar years and found to be very consistent in total City water consumption. Based on the City water sales data analyzed, the average annual daily water demand or "sales" for 2016 was approximately 9.5 MGD. The Atkins Hydraulic Model Update was about 14% lower (approximately 8.2 MGD) and believed to be missing a portion of the irrigation demands. As part of the resubmittal of Technical Memorandum #2, submitted August 23, 2018, irrigation meter demands were provided by the City (including a water meter shapefile) and added to the hydraulic model. Accordingly, t**he 2018 Master Plan is based on a water demand of 9.5 MGD for existing water use in the City**. This lower average compares to a City average annual water demand in 2010 of approximately 10.7 MGD. The trend of reduced potable water consumption is consistent with other similar sized Arizona cities. **Figure 3-1** illustrates this trend back to 2008 and is shown on a monthly basis (City of Lake Havasu, 2017).

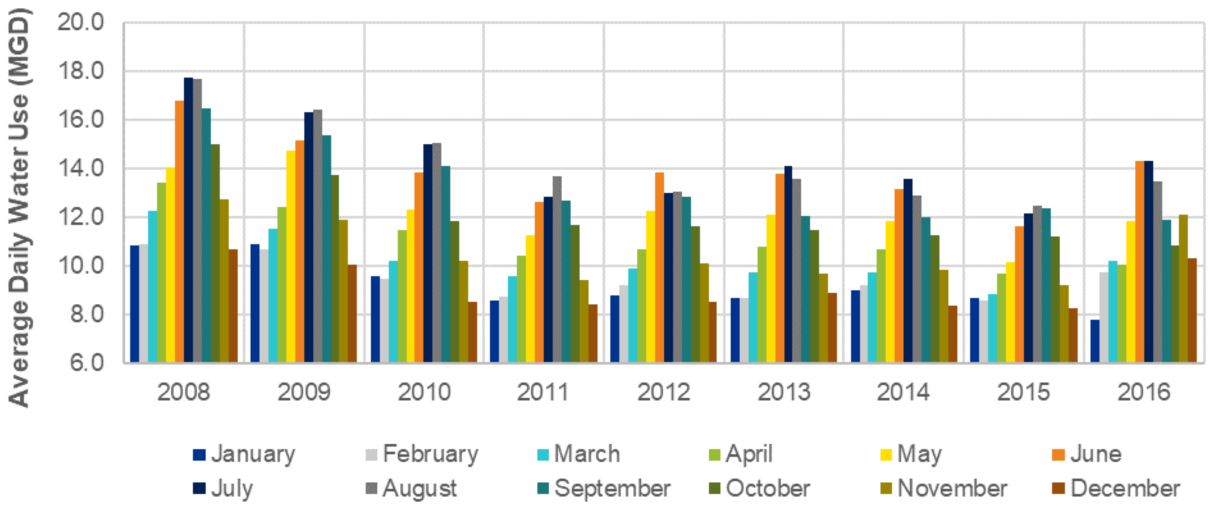


Figure 3-1. 2008-2016 CITY MONTHLY WATER SUPPLY

Water Use by Land Use

As part of the water use analysis, water demands by major land use categories were reviewed and evaluated. **Figure 3-2** illustrates the percentage of annual average water use by meter land use categories for 2016, including irrigation. Of note, more than 60% of the total City water demand was single family residential use. Multifamily water use, including apartments and condominiums, constitutes to about 10% of total annual water demand, resulting in over 70% of the City water use being attributed to residential. Irrigation demand accounted for 15% of the City water use, and commercial water use in the City is about 8% of the total and typically includes businesses, offices and retail commercial, namely shopping centers and neighborhood stores. Irrigation use includes water meters serving both residential and commercial uses. Residences typically use an irrigation meter for large outdoor irrigation, swimming pool automatic refills, and swamp coolers.

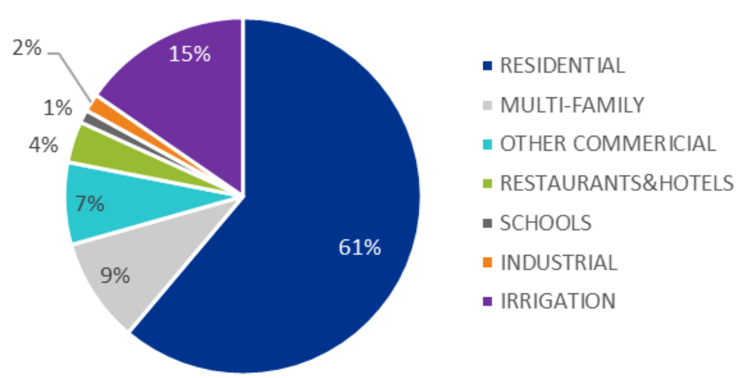


Figure 3-2. Water Use by Land Use (2016)

Figure 3-3 has been included to illustrate the breakdown of irrigation use throughout the City. Nearly 50% percent of irrigation is for residential use which includes both multi-family and single-family consumption.

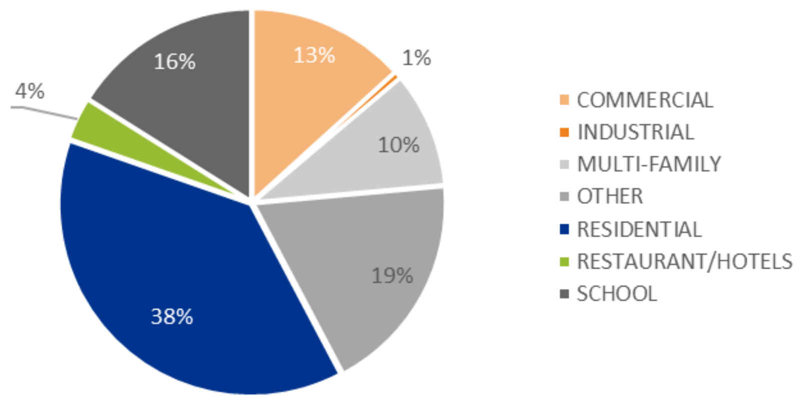


Figure 3-3. Irrigation Water Use by Land Use (2016)

Monthly water use by major land use type was also reviewed to observe demand patterns through the course of the year. **Table 3-1** summarizes monthly average day demand for single family residential, multi-family, and non-residential categories (including schools, commercial, and industrial uses). As previously noted for 2016, about 70% of the annual water is consumed by residential and multifamily uses. On a monthly basis, this distribution remains consistent between residential and commercial through the course of the year. **Table 3-1** summarizes the average annual daily water demand for 2016 is approximately 9.5 MGD and includes a comparison of monthly peaking factors relative to average, as described in the following section.

Table 3-1. 2016 Summary of Monthly Average Day, Monthly Total Water Use and Peak Monthly Factor

Month	Residential (MGD)	Multifamily (MGD)	Non-residential (MGD)	Monthly Total (MGD)	Peak Monthly Factor (2016)	Peak Monthly Factor (MP, 007)
January	4.4	0.8	2.0	7.1	0.75	0.75
February	4.2	0.8	1.6	6.6	0.70	0.76
March	5.0	0.9	2.2	8.0	0.85	0.70
April	5.0	0.9	2.3	8.2	0.86	0.82
May	5.0	0.9	2.5	8.3	0.88	0.84
June	6.3	0.9	2.8	10.0	1.06	1.01
July	6.7	0.9	3.3	10.9	1.14	1.41
August	6.9	0.9	3.4	11.3	1.19	1.33
September	7.1	0.9	3.5	11.5	1.21	1.32
October	6.6	1.0	3.8	11.3	1.19	1.12
November	5.4	0.8	2.9	9.2	0.96	0.95
December	5.8	0.9	2.7	9.5	1.00	0.99
Annual Average	5.8	0.9	2.8	9.5	0.98	1.00

*Average demand reflects water sales and does not include non-revenue water

Table 3-2 includes a month summary between non-irrigation and irrigation demands for the baseline year.

Table 3-2. 2016 Summary of Monthly Average Day, Non-Irrigation and Irrigation Demands

Month	Non-Irrigation (MGD)	Irrigation (MGD)	Monthly Total (MGD)
January	6.3	0.9	7.1
February	6.0	0.6	6.6
March	7.1	1.0	8.0
April	7.1	1.1	8.2
May	7.1	1.2	8.3
June	8.5	1.5	10.0
July	9.1	1.8	10.9
August	9.4	1.9	11.3
September	9.5	2.0	11.5
October	9.1	2.2	11.3
November	7.5	1.6	9.2
December	8.0	1.4	9.5
Annual Average	8.0	1.5	9.5

Seasonal Variation

In general, peaking factors are used to estimate water increases over average day demands for various conditions and are used to evaluate available capacity in the water system. Moreover, a peaking factor is the multiplier that translates ADD to MDD or to peak hour demand (PHD). These factors are used in the hydraulic model to represent a condition when the system is most stressed or to substantiate diurnal patterns (both maximums and minimums) for extended period simulations. Monthly peak factors calculated from the 2016 water-meter data, as shown in **Table 3-1**, are indicative of the required average water supply during the peak summer months. For example, the maximum month peaking factor was 1.21 for September 2016 with an average daily demand of 11.5 MGD. Within the peak month, there will be days above the monthly average which is referred to as the maximum day demand. In comparison, the maximum month factor was 1.41 for July in 2007, repeating a trend in recent water conservation practices and demand management. This lower maximum month factor and corresponding maximum day factor are used in estimating future water supply requirements, BPS capacity and storage tank capacity needs.

Figure 3-4 presents monthly average daily water demand for 2016. The maximum month water demand was 11.5 MGD, whereas, the minimum demand was 6.6 MGD. The annual average water demand for the City in 2016 was 9.5 MGD. As expected, during the summer months (i.e., from June through October) higher monthly water use compared to winter months was observed.

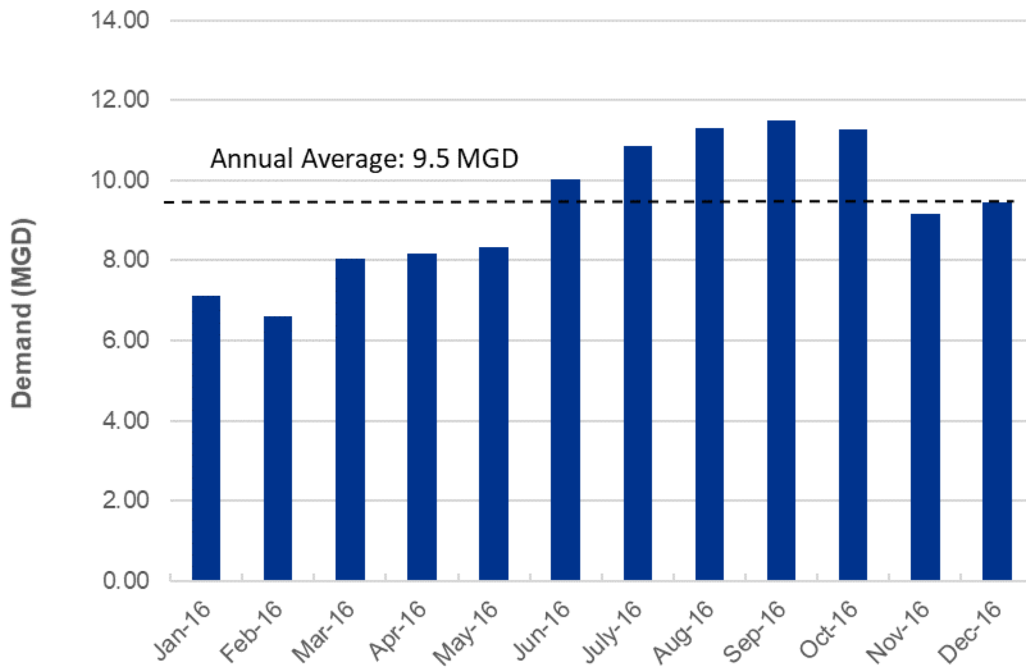


Figure 3-4. Monthly Average Water Demand During 2016

Water Use per Capita

An average per capita consumption is difficult to establish for the City due to the transient nature and high variability throughout the year. Based on the assumed baseline water demand of 9.5 MGD in 2016, and an estimated population of 54,089, the average per capita water demand is approximately 175 gallons per day (gpcd). However, a slightly higher per capita of 185 gpcd was used to project the future water demand and provide a level of conservativeness due to the annual variability of population.

Between 2000 and 2014, the City of Lake Havasu's unit water per capita demand has steadily decreased from 260 gpcd to approximately 186 gpcd (Water Conservation Plan, 2015). Although the City may see a slight future decline in per capita use, 185 gpcd appears reasonable for use in future demand projections based on the recent water use trend.

3.1.1 Existing Demand by Pressure Zone

The City's water distribution system consists of several major pressure zones to serve the varying topography. **Figure 3-5** illustrates spatially within the City the major pressure zones served by a BPS and in most case storage tanks (Zone 1 through Zone 6) as presented in the H2OMap hydraulic model's pipe network (Atkins, 2017).

It can be seen from **Figure 3-5** that the pressure zone areas for 1, 2, and 3 are the largest zones servicing the highly developed central and western parts of the City. Also shown to the right inset on **Figure 3-5**, is the small portion of water distribution system in the north served by Zone 2.

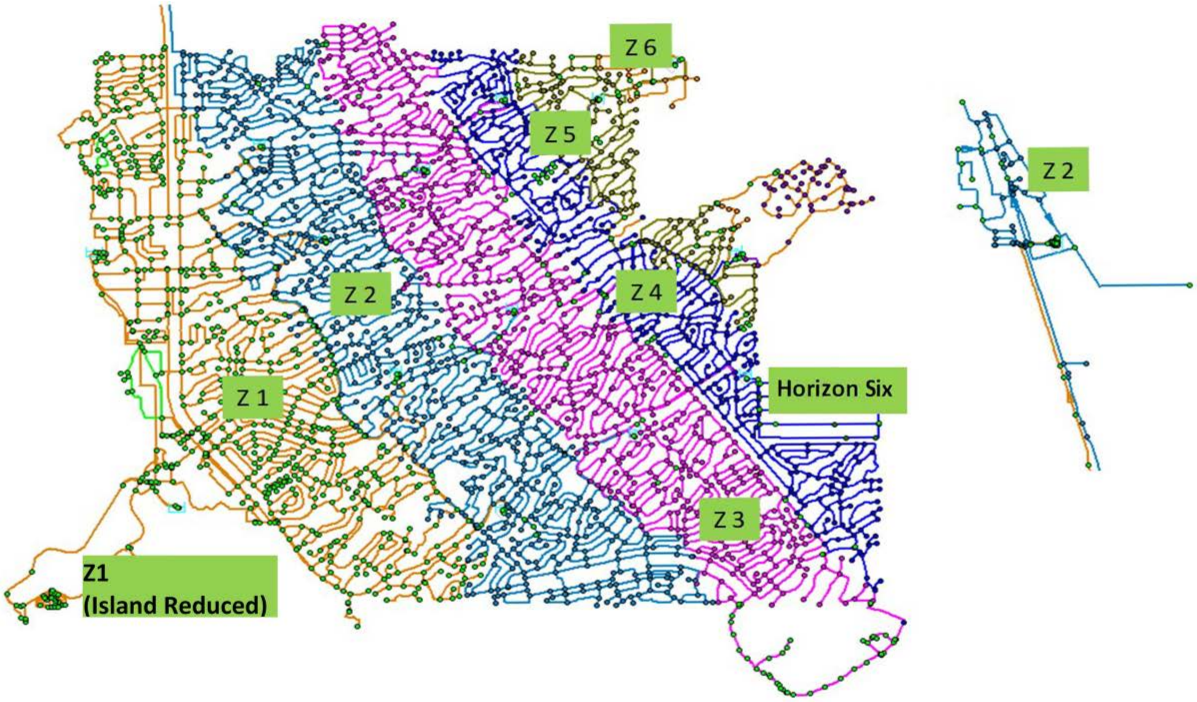


Figure 3-5. Lake Havasu City Pipe Network by Pressure Zones (2016)

Table 3-3 presents a summary of ADD and MDD by the major pressure zones for 2016. The existing maximum day demand is assumed 1.5 times higher than existing average demand based on a review of daily water supply records at the City's Water Treatment Plant (WTP). Total water demand in Zones 1, 2, and 3 contribute to 80% of total water use in the City. The remaining 20 percent is distributed among Zones 4 through 6 including Horizon Six.

Table 3-3. Average Daily Water Demand and Maximum Demand by Pressure Zone (2016)

Pressure Zones	Existing Total (gpm)	Existing Total (MGD)	Existing Maximum Demand (gpm)	Existing Maximum Demand (MGD)	%Existing Total
1[1]	2,055[1]	3.0	3,083	4.4	31%
2	2,077	3.0	3,115	4.5	31%
3	1,289	1.9	1,933	2.8	20%
4	581	0.8	871	1.3	9%
5	298	0.4	447	0.6	5%
6	253	0.4	380	0.5	4%
Horizon Six	41	0.1	62	0.1	1%
Total	6,594	9.5	9,892	14.2	100%

1. Zone 1 includes a small pressure reduced zone serving the Island area.

Also, of note are the largest potable water uses (non-irrigation) in the City and their location by street address. This survey is completed to investigate if a small percent of the customers has a major impact on the system capacity. Summarized below in **Table 3-4** are the Top 10 potable users for Year 2016 in the City which make up approximately 6 percent of the total City demand. Note this top 10 list may vary from year to year.

Table 3-4. Year 2016 - Top 10 Potable Water Users by Location (non-irrigation)

User Name	Street Address	Rate Class	Meter Number	Total (MGD)
Havasu Regional Medical Center	1840 Willow Ave	Hospital	89027528 E10051783	0.15
12. Sam's BeachComber Resort	601 Beachcomber Blvd	RV Parks	12063872 E10051781	0.07
Los Lagos Homeowners Association	0 Los Lagos II	Condos	E09010455	0.06
Sunset Mobile Home Park	1510 Sunset Dr	Multi-Family	MF13013566 MF13013600 MR13908775	0.05
London Bridge Resort	1477 Queens Bay	Hotel/Motel	96998825	0.04
Windsor Beach Gated Community	375 London Bridge Rd	Condos	M0304000001	0.04
Hampton Inn	245 London Bridge Rd	Hotel/Motel	M251615	0.04
Sterilite Corporation	2201 College Dr	Industrial	MF13004130	0.06
			M6304000612 15589042	0.03
North Regional Waste Water Treatment Plant	7001 Whelan Dr	Professional	52688591 6254360 8103055 8103056 11514822	0.03
Queens Condos	777 Harrah Way	Condos	12102758	0.03
			Total	**0.53**

3.1.2 Peaking Factors

A potable water system must be able to supply water at rates that fluctuate over a wide range and meet minimum residual pressure requirements. Water demands most important to the planning, design, and operation of a water system include annual average day, maximum day, peak hour, and fire flow. ADD is the total annual water use divided by the number of days in the year. The ADD is used as the baseline for projecting MDD and PHD and typically for estimating operating costs and expected revenues. The MDD is the maximum quantity of water used on any day of the year, is used to size BPS and storage reservoir facilities, as noted in Section 2. PHD is the maximum rate of water used during any one hour of the year and typically occurs during the maximum day and the early morning hours. PHD flow rates often impose the most severe hydraulic condition and result in lowest residual distribution system pressures. PHD's are usually met through a combination of system supply, typically from pumping and storage facilities.

Figure 3-6 presents monthly peak factors estimated from the monthly total demand over the annual average demand for 2016. As expected, higher than average peak factors than ADD were observed in summer months. Maximum monthly peak factor in September 2016 is 21% higher than the average monthly demand and the minimum monthly peak factor in February 2016 is 30% lower than the annual average. Higher water use in summer is attributable to increased water use for landscape watering, swimming pools, swamp coolers and weekend activities. In the winter, there may be more transient population, but less outdoor water demand with swimming pool refills and swamp coolers.

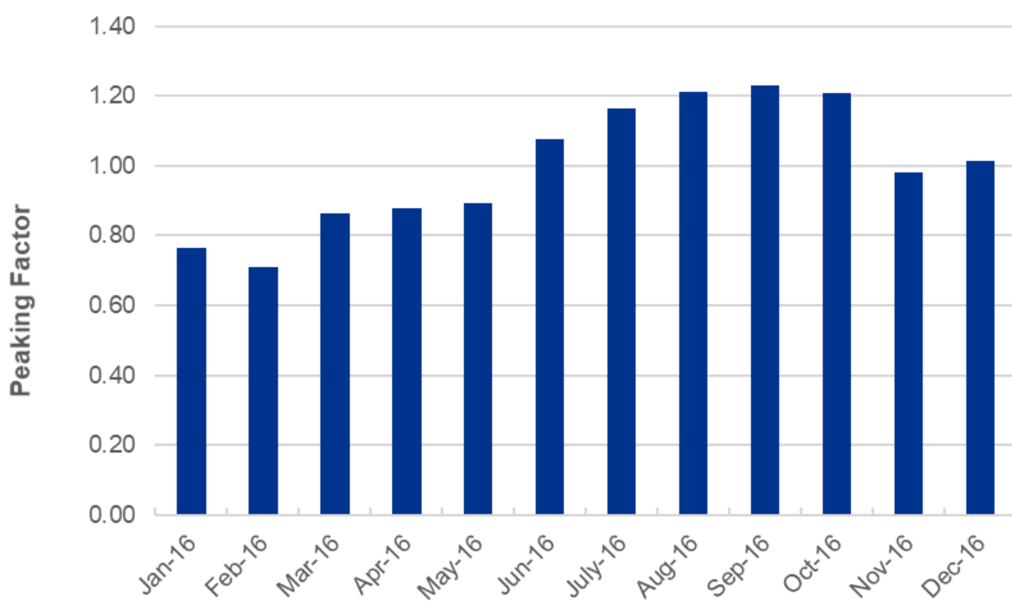


Figure 3-6. Monthly Peak Factor for 2016

A MDD peaking factor of 1.50 appears to be adequately conservative based on trends over the past 5 to 10 years and a review of WTP supply records. **Table 3-3** also summarizes the existing MDD by pressure zone, which will be used to evaluate each zone's water supply requirements.

3.1.3 Non-Revenue Water

Water supply data from the FIT 1 Sparling Meter (the master water meter at the WTP) from 2007 through 2017 was reviewed to compare with the annual water sales data. **Figure 3-7** presents water supply and water use data for 2016 in MGD. The reported monthly water supply normally exceeds the water sales through the course of the year and can vary seasonally. The difference between water supply and water sales is referred to as unaccounted-for water. Un-accounted for water is largest during the high demand summer months, while the unaccounted-for water for the City is minimal in the November through January timeframe. The importance of unaccounted-for water is further discussed in the next section.

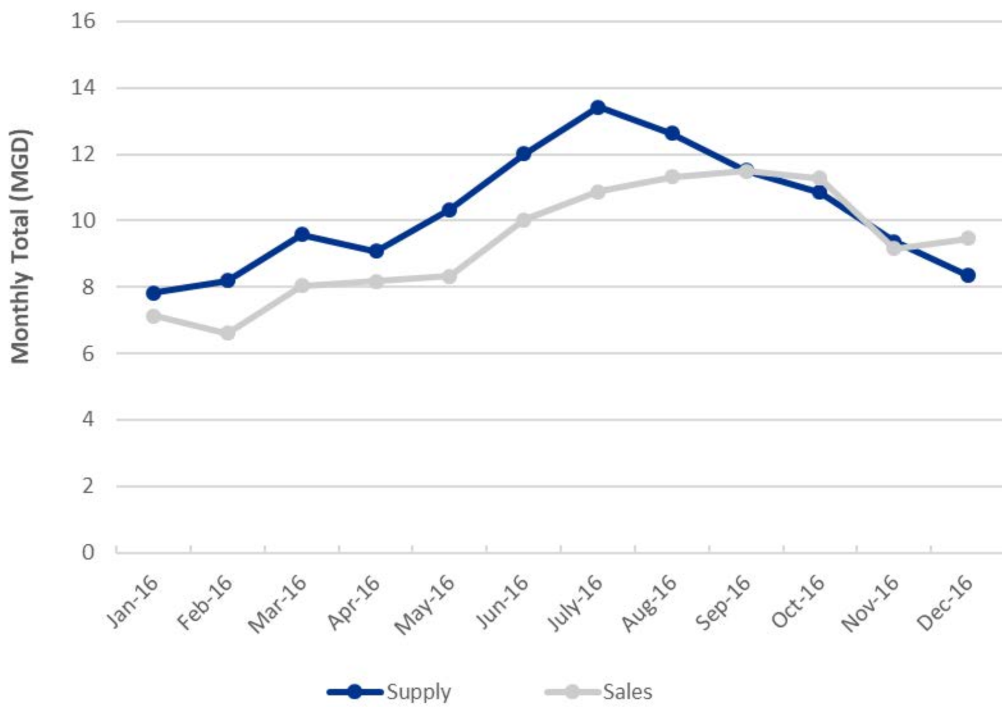


Figure 3-7. Monthly Water Supply and Water Use in MGD for 2016

Unaccounted for water, also called non-revenue water, is the difference between the amount of water supplied at the City WTP and the amount of water distributed to the City's customers. Non-revenue water is lost from the distribution system through a variety of ways, both authorized and unauthorized, including water for firefighting, pipe flushing, hydrant testing, leakage from pipelines, pipe breaks, individual water meter errors, and theft.

Water losses also include "real losses" and unaccounted for water loss. Real water losses are physical water losses from the water distribution system. Generally, water loss occurs in two ways: (i) apparent losses: the water is either not measured correctly via the metering and billing process, (hence the water is not really lost, it simply was not correctly accounted for), or (ii) real losses: the water is leaked out of the system somewhere between the WTP to the customer's meter or service (Lake Havasu City Water Audit Report, 2015).

Figure 3-7 compares the supply production and sales, and the unaccounted water is represented in the gap between the two plotted lines. Included in the unaccounted-for water are system losses (due to leaks, water main breaks, unauthorized consumption, reservoir overflows, or inaccurate meters) and water used in system operations. Water lost during conveyance as well as unaccounted-for water should be considered when projecting total water demand and supply requirements. One observed monthly water loss anomaly is when water meters are read on a staggered four-week cycle. The overall annual loss in the City appears to be reasonable allowance, however a few months though appear to be skewed and may be a result of the meter reading cycle. **Figure 3-8** shows monthly unaccounted-for water for 2016. Annual average unaccounted for water was estimated as 10 percent. Total volume of water lost in 2016 was 350 MG or an average of 0.96 MGD. Most Arizona cities range from 6 percent to a maximum of 10 percent water loss.

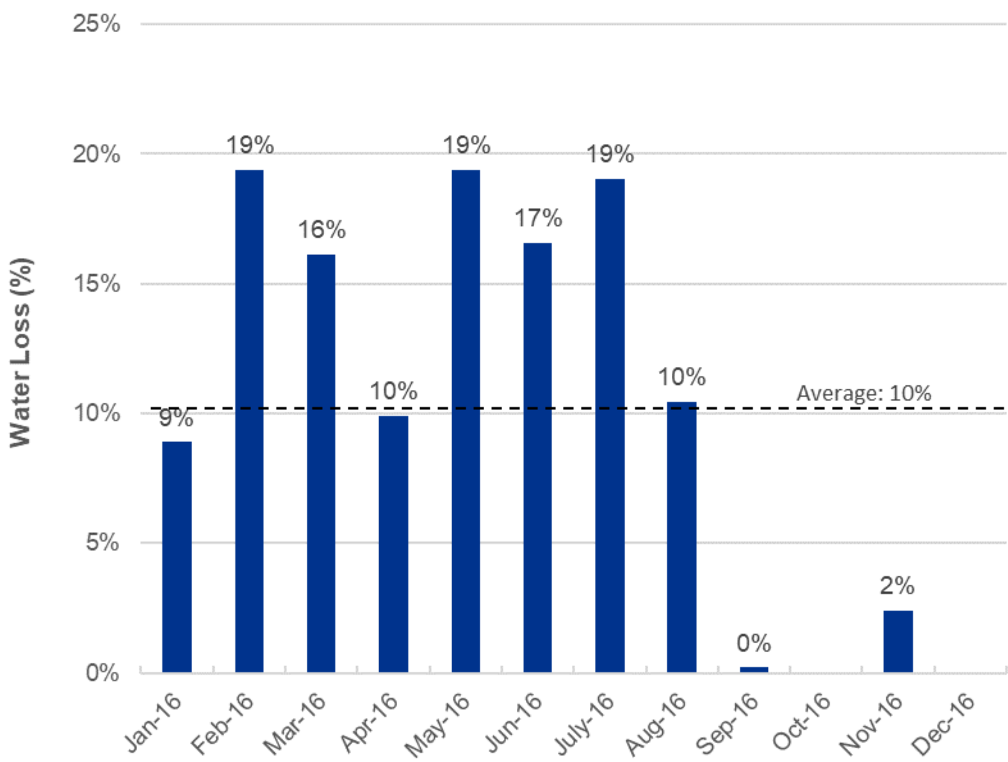


Figure 3-8. Monthly Percentage of Unaccounted-for Water for 2016

For the Master Plan Update, an additional allowance of 10 percent will be included on the water demand forecast to account for both unaccounted water and provide a contingency for planning purposes, in analyzing the water distribution system and developing long term water supply needs.

To reduce water losses, the City should continue to implement the following actions:

- Continue to promptly repair identified water system leaks,
- Monitor water consumption versus production so that potential water loss can be identified,
- Calibrate water meters periodically,
- Assess and replace less accurate water meters (oversized water meters can contribute to inaccurate readings), and

- The sizing, specifications, and performance of the Sparling master water meter at the WTP was reviewed and appears to be accurate under lower baseline flows at the WTP. The meter should be continually calibrated in accordance with manufacturer recommendations.

3.2 Future Water Demand Forecast

Utilizing the population projections from Section 2.2.4, the recommended per capita water use of 185 gpd was used to develop the water demand projections for 2025 and 2040. As shown in **Table 3-5**, the water demand or "sales" is expected to increase to 10.8 MGD and 12.3 MGD, in 2025 and 2040 respectively. As "sales" data, this does not include an additional 10 percent for unaccounted water in the distribution system. The predicted build out sales demand for the City is estimated to be 17.8 MGD annually. The potable water demand forecast conservatively does not consider any new increases in recycled water demand from the conversion of potable irrigation meters. The City could potentially convert 0.3 to 0.5 MGD of potable water irrigation to recycled water in the future. However, cost/benefit issues may impact the ability to realize this demand. **Figure 3-9** presents water demand based on the build-out population.

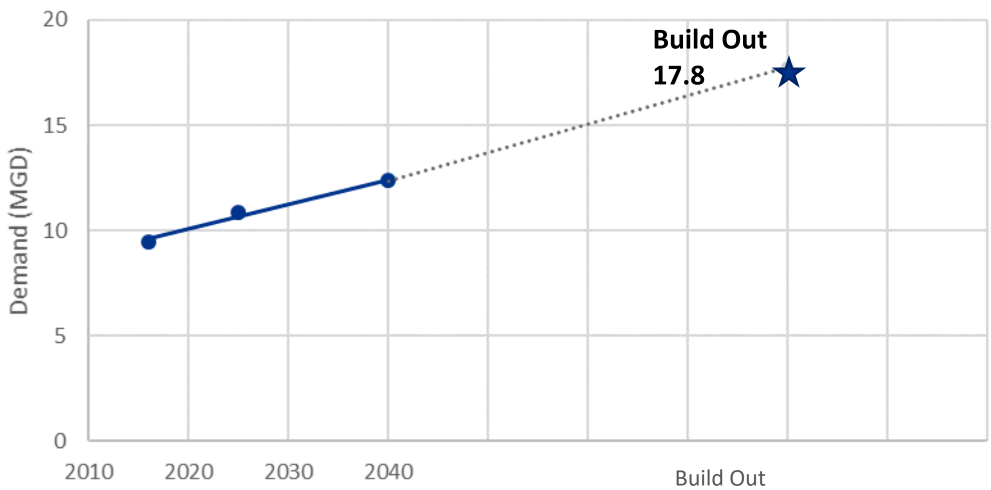


Figure 3-9. Master Plan Update Water Demand Forecast

Table 3-5. Annual Total Population, Water Demand and Per Capita Water Demand

Year	Population	Demand (MGD)	Demand per person (gpcd)
2016	54,089	9.5	175
2025	58,570	10.8	185
2040	66,698	12.3	185
Build-out*	96,000	17.8	185

*Based on assumed 2016 population

Table 3-6 presents the total ADD by pressure zone for 2040 which was estimated based on an allocation the future water demand to the vacant lands within the water service area. The system boundary for the water distribution system for the Lake Havasu City in the future is assumed to remain at the current service area.

Table 3-6. Future Average and Maximum Day Water Demand by Pressure Zone (2040)

Pressure Zones	Future ADD (gpm)	Future ADD (MGD)	Maximum Demand (gpm)	Maximum Demand (MGD)
1	2,671	3.8	4,006	5.8
2	2,698	3.9	4,048	5.8
3	1,675	2.4	2,512	3.6
4	755	1.1	1,132	1.6
5	387	0.6	581	0.8
6	329	0.5	494	0.7
Horizon Six	54	0.1	81	0.1
Total	8,569	12.3	12,853	18.5

Near Term Development Forecast

The City has seen a recent resurgence in development projects and construction activity, including the re-start of Foothills Estates, approvals for the Havasu Riviera Project (aka Havasu 280), Sara Park expansion, and a new hotel project (Holiday Inn Express). The City continues to see infill development, both commercial and residential, on vacant lots through-out the City. **Table 3-7** below summarizes the major near-term projects and their respective water use based on recent water studies and/or City design criteria. Several of these projects likely will develop over a 10 to 15-year period depending on the economy. The 2025 water demand forecast (**Figure 3-9**) increases average annual water use by 700,000 gpd. It is anticipated that most of this increase by 2025 will be from the projects shown below which represent an increase of 790,000 gpd. The 1,800 proposed residential units at an assumed growth rate of 200 units per year would result in about a 10-year build plan.

Table 3-7. Near-Term Water Demands

Development	Pressure Zone	Proposed Units	Average Annual Demand (gpd)	Max Day or Peak Hour Demand (gpm)
Havasu Riviera Master Plan Development		294		
Golf Course[2]	1	3	74,111	83
Botanical Gardens		100		
Business/ Government		4		
Havasu Riviera Arizona State Park[1]	1	18	43,362	48
Marina		18		
Havasu Riviera Resort Community[1]		860		
Residential	1	850	361,280	401
Resort		10		
Sara Park[3]	3	-	278,655	710
Foothills Estate[4]	6 and 7	583	261,059	290
Campbell[5]	4	102	26,520	28
BlueWater[5]	5	97	25,030	28
Total	-	1,767	791,362	

[1]Water Master Plan for the Havasu Riviera Project (ARQ Engineering, LLC, 2017)
[2]If project is developed by the general contractor, the golf course will be served by recycled water, and no potable demands are assumed.
[3]Sara Park Memo (CH2M, 2017)
[4]Foothills Estates Master Plan (ARQ Engineering, LLC, 2017)
[5]Bluewater Development Water System Analysis (CH2M, 2017)

Water Demand Summary

In summary the proposed water demand forecast for 2040 will serve as a basis for the Master Plan Update and will be used to determine the required water supply and backbone infrastructure to support the future growth. Key findings are noted below:

1. The existing water demand in the City has reduced slightly from 2010, resulting in average annual water sales of 9.5 MGD.
2. Existing water supply and storage requirements will be based on the existing sales demand of 9.5 MGD plus 10% to included unaccounted for water in the system or 10.5 MGD.
3. A population-based methodology was used to forecast future water demands for 2025, 2040, and build-out.
4. Overall per capita water use for projecting future water use is about 185 gpdpc.
5. The 2040 population forecast is approximately 66,700 people for the Water Service Area, resulting in a demand of 12.3 MGD. When considering unaccounted for water this will require an average annual water supply of 13.5 MGD or 15,120 afy.
6. The build-out population forecast is approximately 96,000 for the City, resulting in a demand of 17.8 MGD. This will require an average annual water supply of 19.6 MGD or 22,000 afy, which is well below the City's Colorado River allocation.
7. Over the next 7 to 10 years the City should see an increase of about 700,000 gpd of increased water use associated with several planned development projects, included in **Table 3-7**.

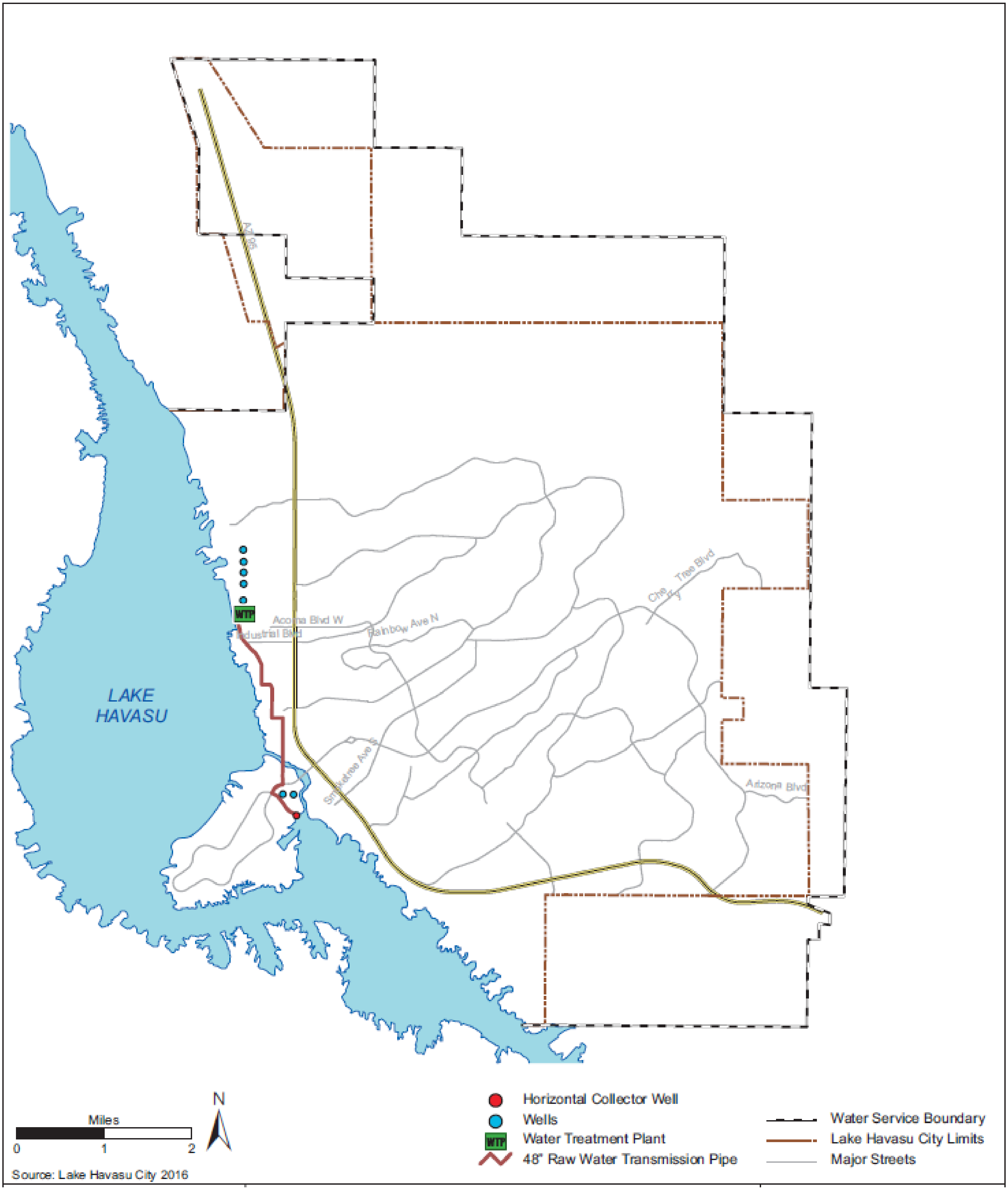
Water Supply

4.1 Colorado River Allocation

The water source for Lake Havasu City is derived from entitlements to the Colorado River and its hydraulically connected Colorado River Aquifer. The City's groundwater facilities have historically consisted of conventional vertical groundwater wells near the Colorado River that serve as riverbank filtration devices. The wells draw Colorado River water through the subsurface sediments and into the wells. In 2001, the City put a high capacity Horizontal Collector Well (HCW) into service which provided 94 percent of the City's groundwater flows in the year 2012 (Wilson, 2013).

4.2 Existing Groundwater Well Supply

The City provided information on their existing wells for use in this Master Planning effort in email correspondence dated February 17, 2016 (Morris, 2016). The information included nine conventional wells and one HCW. Two of the conventional wells and the HCW are considered Central Wellfield wells as they are located on Pittsburg Island near the west central part of the City. **Figure 4-1** illustrates the general location of the HCW, North Wellfield and WTP.



Lake Havasu City Location Map

Figure 4-1

Seven of the conventional wells are considered North Wellfield wells and are located in the northern part of the City, just north of the existing WTP. Historically, the City also operated five wells in their South Wellfield near Rotary Community Park, however, these were abandoned in 2013 (Wilson, 2013). The City's Central Wellfield is currently operated for raw water supply. The North Wellfield wells, although generally equipped with pumps and piping, are not used due to reported heavy sand and silt production (Wilson, 2013), and mechanical issues with two of the well's pumps and motors (Morris, 2016).

The City's well collection system piping consists of a single 48-inch pipeline that conveys raw water north from the Central Wellfield and the HCW to the WTP. A second set of collection piping exists to convey water from the North Wellfield south to the WTP. The 48-inch pipeline runs beneath the channel that separates Pittsburg Island from the mainland and has been reported to have been installed using a horizontal drilling technique. This pipeline is reported to be very deep beneath the channel such that any required repairs to the deep portions of the pipeline would likely not be feasible (Morris, 2017). This fact highlights a vulnerable point in the City's water supply system. Because the City now relies on the HCW and the Central Wellfield for essentially all of their raw water supply, an interruption in service of the 48-inch pipeline would cut off the City's raw water supply until such time repairs or replacement of the 48-inch pipeline could be made.

Conventional wells used by Lake Havasu were researched from the City's existing well records, supplemented by searching the Arizona Department of Water Resources (ADWR) Well Registry for the "55" registered wells, and the ADWR Imaged Records for the "35" wells documents. Briefly, the "55" wells in the State registry are those non-exempt wells registered with ADWR after about June 1980. The "35" wells are those wells listed with the State from the late 1960's through about June 1980. The "35" wells are generally those that were abandoned or taken out of use before the "55" registry was created. All non-exempt wells in the State are required to be registered.

The City wells identified for this work are listed in **Table 4-1**. The well capacities listed are as recorded in the ADWR Well Registry.

Table 4-1. Wells Reviewed in LHC for this Master Plan

	Date Constructed	Age (Years)	Depth[1] (feet)	Diameter (inch)	Capacity (gpm)	Initial SC[2] (gpm/ft)
North Wellfield Existing						
Well 18	3/7/1986	32	450	30/20	1,700	10
Well 15	3/7/1977	41	550	16	1,300	unknown
Well 14	8/20/1975	42	509	20/16	1,100	13
Well 13	4/4/1975	43	511	20/16	1,100	11
Well 10	1/31/1975	43	550	20/16	1,000	11
Well 12	11/15/1974	43	405	12/10	700	8
Well 11	9/20/1974	43	440	12/10	700	11
North Wellfield Abandoned or Inactive						
Well 8[3]	6/27/1965	na	155	12	475	6
Well 3	1/12/1987	31	160	20	500	6
Central Wellfield Existing						
Well 2	4/12/1979	39	163	20	2,200	115
Well 9	4/21/1990	28	175	18	2,900	139
HCW	4/16/2000	18	97	192	17,400	484
Central Wellfield Abandoned						
Well 9[3]	1/10/1969	49	200	20	1,850	22
South Wellfield Abandoned						
Well 16[3]	7/27/1977	na	500	20/16	1,302	unknown
Well 21[3]	2/3/1992	na	137	18	310	5
Well 17[3]	6/26/1984	na	160	20	700	5
Well 4[3]	4/22/1977	na	493	17	1,613	unknown
Well 6[3]	1/4/1965	na	130	12	unknown	unknown

[1] From Ground Surface to Inside Bottom of Finished Well
[2] SC = Specific Capacity
[3] Abandoned
na – Not Applicable

4.3 Well Supply Evaluation

Specific Capacity values, as listed in **Table 4-1**, represent the well's pumping rate divided by the drop-in water level in the well due to pumping (Todd, 1980). This drop is typically referred to as drawdown. Specific capacity values are not constants for any given well. Specific capacity values are a function of pumping rate and pumping duration, in addition to the loss of a well's capacity due to plugging. Specific Capacity can be calculated using **Equation 1** as follows.

Equation 1: Specific Capacity

$$SC = \frac{Q}{\Delta s}$$

Variables

SC = Specific Capacity (gpm/ft)
Q = Pumping rate (gpm)
Δs = Drawdown, change in well water level due to pumping (feet)

Wells in **Table 4-1**, where locations are known, are shown on **Figures 4-1**, and **4-2**. Accurate locations of the abandoned wells are not known; therefore, a location map of the South Wellfield is not included. The South wellfield was previously in the vicinity of Rotary Park in the South part of the City.

Aerial Image © 2017 Google Earth. Annotation © 2017 CH2M HILL

Figure 4-2. Lake Havasu North Wellfield

Aerial Image © 2017 Google Earth. Annotation © 2017 CH2M HILL

Figure 4-3. Central Wellfield

4.3.1 Horizontal Collector Well

Horizontal Collector Well

The City HCW consists of a central concrete caisson with fourteen (14) stainless steel lateral screens projected horizontally near the base of the caisson. The collector caisson has an inside diameter of 16 feet and 2 ½ foot thick caisson walls. The internal depth of the caisson is 98 feet from the top of the caisson to the caisson floor.

Eleven lower lateral screens were installed at 3 feet above the floor of the caisson and three upper lateral screens were installed at 5.5 feet above the floor of the caisson. Each lateral is equipped with 30 feet to 210 feet of 12-inch diameter wire-wound stainless-steel well screen. The total lateral length installed is 1,900 feet. The laterals were installed through stainless steel port assemblies, which were grouted into the wall of the caisson. The laterals were each equipped with non-rising stem resilient seat gate valves. The gate valves operators are submerged in the caisson; valve stem risers were not installed to enable valve operation inside the HCW building.

The HCW was tested following its construction by two well pumping tests in March 2000. The first was a variable rate pumping test which pumped the well at four different rates for two hours each. The four pumping rates were 6.3 MGD, 12.5 MGD, 18.7 MGD, and, 25.0 MGD. The second test was a constant rate pumping test that ended on April 16, 2000, where the HCW was pumped at a constant rate of 25.0 MGD for a period of 30 days.

The geologic logs for the HCW and the test hole constructed for it indicate the hydrogeological conditions are similar to the area of existing Wells 2 and 9. That is sand, gravel, and cobbles in at least the upper 100 feet of the sediments.

Horizontal Collector Well Observations

In order to evaluate the HCW performance, the original pumping test data from the year 2000 was re-evaluated. It should be noted that during the original 30-day constant rate pumping test, the pumping water level in the HCW did not go steady state but was slowly dropping throughout the pumping duration. On the last day of the original 30-day constant rate pumping test, the drawdown in the HCW was 35.97 feet and was at still declining by about 0.06 ft/day due to the pumping.

The HCW was rated by the HCW design engineer at 25 MGD with an equilibrium drawdown of 40.4 feet (Layne, 2000). The year 2000 testing originally predicted an equilibrium specific capacity of 430 gpm/ft at a 25 MGD pumping rate. These values can be considered the HCW baseline performance.

Available pumping rates and water level records at the HCW were obtained from the City from January 1, 2015 through November 16, 2017. Records before that time were not available on the SCADA system computer due to a system upgrade at the end of year 2014. The pumping rates and water level records were reviewed and compared to values that were predicted from the original pumping test in order to assess the current hydraulic capacity of the HCW.

To perform this comparison, first the drawdown in the HCW was calculated from the water levels provided by the City. The water levels provided were the height of the pumping water level above the caisson bottom. Drawdown “s” which is typically the parameter of interest to hydrogeologists, can be calculated using **Equation 2** as follows:

Equation 2: Drawdown

$$s = D - h - SWL$$

<u>Variables</u>

s = Caisson Drawdown
D = Depth of Caisson
h = Height of Pumping Water Level above the Caisson Floor
SWL = Static Water Level in the Caisson Measured from the Top of the Caisson

A plot of the drawdown values observed in the HCW is presented in **Figure 4-5**.

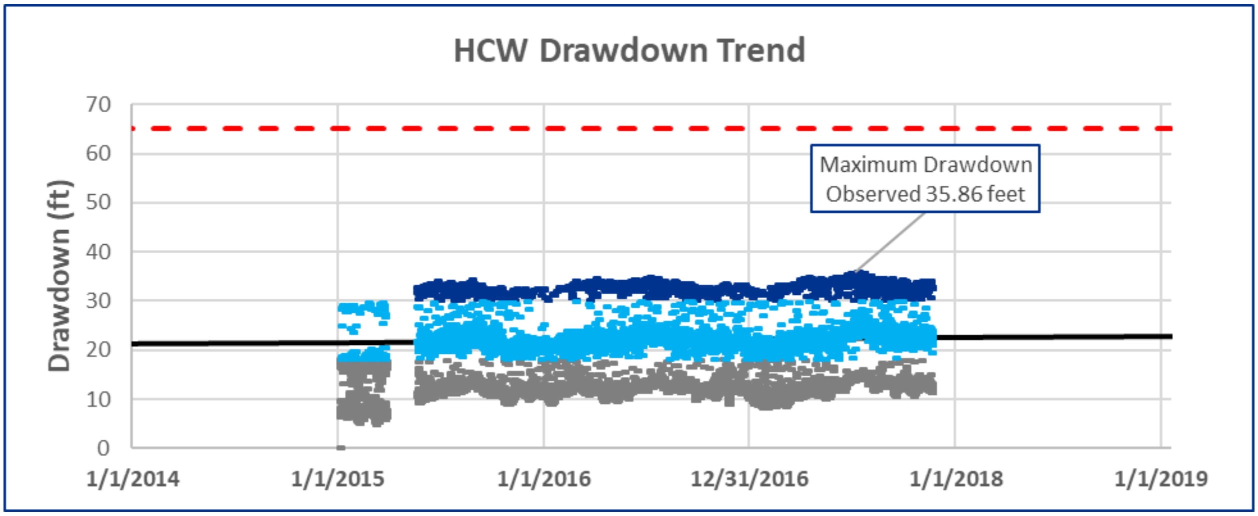


Figure 4-5. Observed HCW Drawdown Data

Drawdowns are shown in three different colors in **Figure 4-5**. Interpretation of the water level data indicates that the HCW is operated in three modes. These three modes of operation are as follows:

1. Operating one HCW pump at 8,500 gpm delivering water to the WTP. Drawdowns associated with this mode are shown as light blue.
2. Operating two pumps at 17,000 gpm. This mode of operation is shown in darker blue.
3. Not operating any of the HCW's three pumps. This mode of operation is shown in dark grey.

Operating the HCW in Mode 2 results in the highest drawdowns and Mode 3 results in the HCW water levels beginning to recover and exhibits the lowest drawdowns. Mode 2 and Mode 3 operations only last for several hours at a time. It is seen that the HCW drawdowns do not get to zero (static) during the short periods of time the HCW is not pumping. The HCW actual static water level is very close to the Lake Havasu water level as the island where the HCW is located consists of unconsolidated sediments surrounded by water. The saturation level of the sediments is about the lake level, and that is the actual static water level across the whole island. During the initial pumping tests on the HCW as performed by Layne in the year 2000, the static water level in the HCW immediately prior to the start of the multiple-rate test was 445.74 feet msl. This was 1.11 feet below Lake Havasu which was at 446.85 feet msl. Given the top of the caisson is at an elevation of 452.27 feet, which puts the static water level at about 6.5 feet below the top of the caisson. The water levels in the HCW would take a considerable amount of time to fully return to a static condition once all pumping ceased.

The average rate of HCW pumping for the three-year period was 9,720 gpm. This value was determined in order to calculate a HCW drawdown and specific capacity to compare with the original year 2000 HCW pumping test.

During the year 2000 HCW pumping tests, drawdowns were measured over time. The constant rate test was conducted at 17,400 gpm, which is equal to the HCW's rated capacity. The results of that test were used to calculate what drawdown would have been expected to occur if the year 2000 constant rate test was conducted at a pumping rate of 9,720 gpm, the average HCW pumping rate for the three-year comparison period. This calculation was first compared to the water levels over the past three years and the results are presented in **Figure 4-6**.

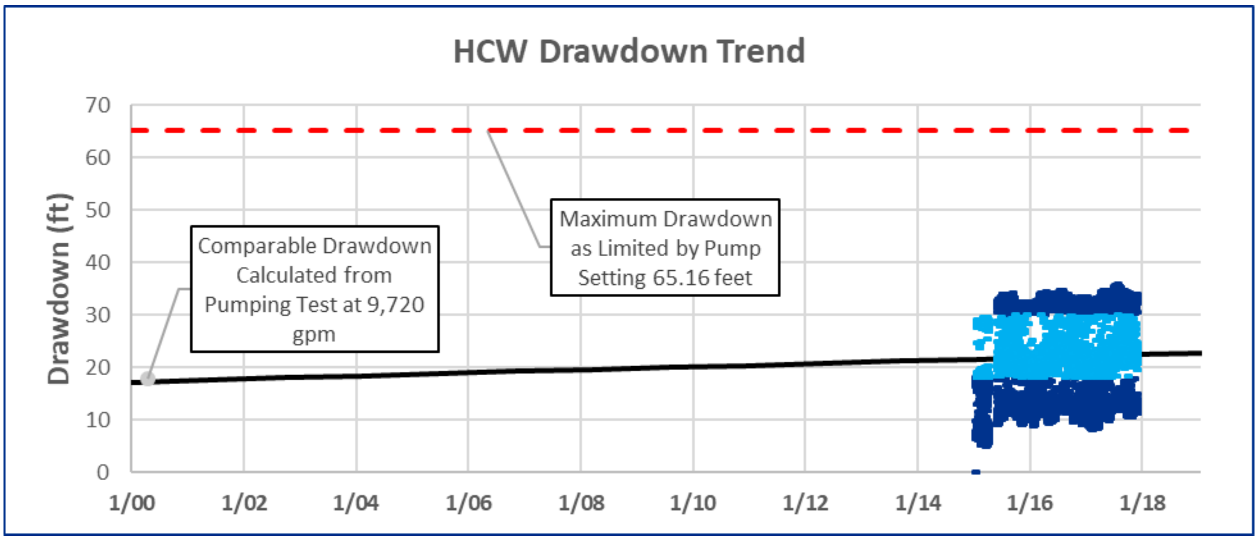


Figure 4-6. Observed HCW Drawdown Data Compared to the Year 2000 Pumping Test Results

Figure 4-6 indicates that the drawdown in the HCW has continuously increased over time when it should be operating at a steady state mode by now. This observation would indicate that the HCW may be losing capacity. It is not uncommon for an HCW to lose capacity over the time period of almost 18 years, and there are steps presented in Section 4.4 that should be taken in an attempt to restore that capacity.

Both vertical wells and HCW capacity is typically tracked by the well's specific capacity. It was stated earlier that specific capacity values are a function of pumping rate and pumping duration, in addition to the loss of a wells capacity due to plugging. Specific capacity is a simple way to review a well's pumping performance as pumping rates from a given well are typically within a reasonably narrow band of rates, and pumping duration many times is as continuous as possible. Under these conditions, and if well plugging is a predominant factor in well performance, specific capacity trends are very helpful in analyzing well capacity issues.

Specific capacity values were calculated for the HCW immediately following its construction and testing, and over the past three years using the pumping rates and water level records obtained from the City. The values are presented in **Table 4-2**.

Table 4-2. Specific Capacity Values Observed and Calculated for the HCW

Measurement / Calculation	Date	Specific Capacity (gpm/ft)	Description
1	4/16/2000	486	Measured at the end of the 30-day, year 2000 pumping test run at 17,400 gpm
2	4/16/2000	430	Calculated at the end of the year 2017 at a pumping rate of 17,400 gpm, assuming the HCW water levels reached equilibrium at a drawdown of 40.4 feet, as presented by Layne, (Layne, 2000), based on the year 2000 test
3	4/16/2000	523	Calculated at end of the year 2017 at a pumping rate of 9,720 gpm, assuming the HCW water levels reached equilibrium, based on the year 2000 test
4	11/16/2017	431	Observed at the end of year 2015-2017 data from the City rates and water level records at 9,720 gpm
5	11/16/2017	322	Calculated year 2015-2017 data from the City pumping rates and water level records at 17,400 gpm

The values in **Table 4-2** indicate that at the end of the 30-day, year 2000 pumping test on the HCW, the specific capacity was observed to be 486 gpm/ft and was estimated by Layne to be 430 gpm/ft after the HCW reached equilibrium, or steady state conditions. Current operations are pumping the HCW at an average rate of 9,720 gpm, which should then result with an observed HCW specific capacity of 523 gpm/ft.

However, the 2015 through 2017 water level and pumping rate data provided for the HCW indicate that the specific capacity is 431 gpm/ft at an average pumping rate of 9,720 gpm. If the HCW were pumping at the full capacity of 17,400 gpm, the specific capacity may currently be as low as 322 gpm/ft. These values represent the current HCW specific capacity with the 322 gpm/ft value being the most representative of how the HCW is operating today.

The specific capacity decrease is seen to be about 25 percent at the 17,400 gpm pumping rates and about 18% at the 9,720 gpm pumping rates. Correspondingly, the ultimate capacity of the HCW would then also have decreased about 25 percent, or from 25 MGD to about 19 MGD.

Currently the City operates the HCW by semi-continuous pumping at an average 9,720 gpm which includes periodic increases to about 17,400 gpm every few days for several hours only. The HCW is sustaining the current operations, although as seen in **Figure 4-5**, the maximum drawdown has been observed within 29 feet of the maximum for a short period of time.

During the year 2018, the City had an inspection of the HCW performed. The inspection was performed by Building Crafts, Inc. and included diving the HCW, inspecting the laterals, and measuring the flow contribution from each lateral. The results of the inspection analyses reported that although the HCW is currently operated for short durations at its rated capacity of 25 mgd, this capacity may be exceeding the acceptable mechanical capacity of the HCW. Operation for prolonged periods of time at this capacity may exceed the mechanical capacity of the laterals thereby reducing the long-term useful life of the HCW. (Building Crafts, 2019)

In February 2019, the City reported higher than usual turbidity production from the HCW. Reportedly, the turbidity was observed about one week following the start of a restroom renovation, which is located over two of the HCW laterals. Divers were sent into the HCW for an inspection on April 5, 2019 and found the two laterals closest to the restroom full of sand, with a pile of sand accumulated on the HCW floor. The laterals were valved off which was expected to further reduce the HCW capacity in the range of 8 percent. (Clark, 2019) This capacity can be restored by cleaning the HCW, but at the current time the HCW capacity can be estimated at about 17.5 mgd. The HCW Historic capacity is presented in Table 4-3.

Table 4-3. Historic Capacity of the HCW

Condition	Date	Capacity (MGD)	Description
1	4/16/2000	25.0	Following HCW Construction
2	2018	19.0	As estimated following the 2018 HCW Inspection
3	2019	17.5	Following Second Inspection and Valving off Two Laterals

Summary of Second HCW Investigation

The City has been looking for a location for a second HCW since 2009 (Wilson, 2013). In 2010, the City contracted with Ranney Corporation to conduct a Phase 1 paper study to identify locations to site a new HCW to supply the City with +/- 25 MGD of water supply. The Phase 1 study identified the Rotary Park Beach area as the preferred location (Ranney, 2010). A Phase 2 investigation was conducted in 2011 which included the construction of three test borings in that area. The findings of the Phase 2 investigation concluded that the central and southeast portions of the Rotary Park Beach area were not conducive for high capacity water wells (Ranney, 2011).

The City later identified six other potential locations for a second HCW (Wilson, 2013). These locations were based on knowledge of the area, and hydrogeologic investigations were conducted. These locations are listed below.

- Site Six – On island side of channel
- Nautical Golf Course -- On island side of channel
- Crazy Horse Campground South – On island side of channel
- Crazy Horse Campground North – On island side of channel
- Site 4 – Windsor State Park – On City side of channel
- Near WTP – On City side of channel

In January 2016, the City requested proposals to construct two exploratory boreholes near the WTP (Lake Havasu City, 2016). Discussions with the City indicate the two locations explored encountered a hard-conglomerate layer at relatively shallow depths that would preclude setting a future HCW caisson using the standard clamshell excavation technique. Drilling was terminated at that point and the crews were demobilized. Although the locations would not support an HCW, they may be candidate sites for vertical wells.

Negotiations to obtain permission to explore other locations have been reported by the City to be slow and difficult. Additionally, four of the identified potential locations are on the Pittsburg Point island, and although another HCW on the island would be useful, it does not help with potential vulnerabilities of the single 48-inch pipeline running under the channel to the WTP.

4.3.2 North Wellfield

Well details of the construction and capacity of the North Wellfield wells were reviewed in order to develop an assessment of potential future use of the wellfield. The well details that were used in this assessment are listed in **Tables 4-4** and **4-5**.

Table 4-4. North Wellfield Existing Wells

Existing			Casing			Perforations	
Well No.	*Age (years)*	*Material*	*Dia. (inch)*	*Interval (feet)*	*Wall (inch)*	*Interval (feet)*	*Cut (inch)*
18	32	Mild Steel	30	0 to 20	Unknown	None	None
		Mild Steel	20	0 to 150	0.250	None	None
		Mild Steel	16	150 to 450	0.250	150 to 450	1/4 by 2 ¾
15	41	Mild Steel	16	0 to 550	Unknown	Unknown	Unknown
14	42	Mild Steel	20	0 to 84	0.250	None	None
		Mild Steel	16	77 to 509	0.250	89 to 497	1/4 x 1 ½
13	43	Mild Steel	20	0 to 157	0.250	None	None
		Mild Steel	16	139 to 511	0.250	145 to 505	1/4 x 1 ½
10	43	Mild Steel	24	0 to 20	0.312	None	None
		Mild Steel	20	0 to 150	0.134	65 to 130	1/4 x 2 ½
		Mild Steel	16	140 to 550	0.250	150 to 550	1/4 x 1 ½
12	43	Mild Steel	12	0 to 148	0.25	60 to 132	1/4 x 1 ½
			10		0.25	132 to 405	3/16 x 3
11	43	Mild Steel	12	0 to 150	0.25	60 to 136	1/4 x 1 ½
			10	140 to 440	0.25	239 to 440	3/16 x 3

Table 4-5. North Wellfield Wells Abandoned or Inactive

Taken Out of Use		Casing				Perforations	
Well No.	*Age (years)*	*Material*	*Dia. (in)*	*Interval (feet)*	*Wall (inch)*	*Interval (feet)*	*Cut (inch)*
8[1]	na	Mild Steel	12	0 to 110	0.25	20 to 110	0.25
3	31	Mild Steel	20	0 to 160	0.312	60 to 160	1/8 X 2 3/8

[1]Abandoned

North Wellfield Observations

The North Wellfield existing wells are between 32 and 43 years old and are constructed of steel pipe with a wall thickness that ranges between 0.134-inch and 0.312-inch. The casings are also all perforated with hole openings ranging from 3/16-inch to 1/4-inch. The useful life of mild steel well casings is affected by many environmental factors, some of them being different soils and waters in contact with the casings, and the relationship between the physical, chemical, and biological components of the well environments. It has been stated by the well casing manufacturer, Roscoe Moss Company, that in general, the useful life of water wells in Arizona typically ranges from 40 to 50 years. (Roscoe Moss, undated). An economic analysis done in that Case Study used the economic life of carbon (mild) steel water well casing as 25 years.

The existing wells are mostly a telescoped construction with a larger upper casing first setting and then a smaller casing set through the upper casing to the lower depths. This type of well construction likely indicates two separate layers of aquifer sediments. The geologic logs reviewed for these wells indicates that the sediments are in general approximately 150 feet of sand, gravel, and boulders overlying approximately 400 feet of similar but finer, more clayey materials. The geological log observations match the well casing telescoped design. Well specific capacities for the existing wells in the Central Wellfield are seen to be considerably higher than the wells in the North Wellfield.

It should be noted that the inactive wells in the North Wellfield appear to be constructed into only the upper sand, gravel, and boulder aquifer layer. The wells that are inactive also appear to have lower pumping capacities than the wells located more north in the wellfield. It is unknown if the lower pumping capacities are the result of aquifer formation variation in the south part of the wellfield, limited available drawdown, or a combination of the two. It is notable in **Table 4-1** that the original specific capacity values as calculated for the initial well testing on record are lower in the shallower, inactive, and abandoned wells.

The perforations in the pipe used for well screen is a typical mechanically perforated size however, it is likely too large for the surrounding aquifer sediments especially in the lower portions of the wells. Due to this and the deteriorating condition of the wells, the water produced is of higher turbidity and historically caused problems at the water treatment plant. Current day well construction rarely uses well slot screens greater than 0.125-inch and then only in fractured rock or very coarse, cobble like sediments.

North Wellfield Drawdown Interference

Drawdown created in an aquifer from a pumping well takes the shape of a cone with aquifer drawdown created even at considerable distance from the pumping well. Other wells located within the cone of depression will see this drawdown as a lowering of the water level in the well. This imposed drawdown is known as well interference.

The North Wellfield existing wells are arranged along a north-south alignment, with the well spacing ranging between 390 feet to 740 feet apart. The close proximity of the North Wellfield wells may preclude operating some of the wells simultaneously because of well interference from adjacent wells. Well interference was estimated from the well details presented earlier by estimating an aquifer transmissivity from the calculated specific capacity values (Driscoll, 1986) and then by using the non-equilibrium equation (Driscoll, 1986) to calculate aquifer drawdowns from each pumping well on the two wells closest to it. It must be noted that this calculation is based on specific capacities calculated from data in the ADWR Well Registry and is therefore only an approximation. To provide an example of the drawdown interference magnitudes, the drawdowns were calculated assuming only one well was pumping for each calculation. The calculated interference values are presented in **Table 4-6**.

Table 4-6. North Wellfield Well Interference Drawdowns

Pumping Well (No.)	Adjacent Wells					
	Closest Well (No.)	Distance (feet)	Drawdown (feet)	2nd Closest (No.)	Distance (feet)	Drawdown (feet)
18	15	387	45.5	14	866	29.8
15	18	387	34.8	14	479	29.1
14	15	479	24.6	13	648	21.6
13	14	648	21.6	10	739	21.7
10	12	150	42.0	13	739	19.7
12	10	150	29.4	11	244	24.8
11	12	244	24.8	10	392	18.5

Notes: 1. Interference Drawdown is Calculated Assuming Only One Adjacent Well is Pumping

The well drawdown available for pumping and interference drawdown is 200 to 260 feet. It is seen from **Table 4-6** that the well interference from multiple wells running could be a substantial percentage of the total available drawdown. If all the wells were pumped simultaneously, several of the wells would dewater and not be able to sustain any pumping.

4.3.3 Island Wells (Central)

Well details of the construction and capacity of the Central Wellfield wells were reviewed in order to develop an assessment of potential future use of the wellfield. The well details that were used in this assessment are listed in **Tables 4-7** and **4-8**.

Table 4-7. Central Wellfield Existing Wells

Existing			Casing			Perforations	Well
Age		*Material*	*Dia. (inch)*	*Interval (feet)*	*Wall (inch)*	*Interval (feet)*	*Cut (inch)*
No.	*(years)*						
2	39	Mild Steel	20	0 to 163	Unknown	Unknown	Unknown
9	28	Mild Steel	Unknown	0 to 60	Unknown	None	None
		SST	18	0 to 175	Unknown	90 to 165	0.04 louver
HCW	18	Conc./SST	192	0 to 97	30	91 to 94	0.010 to 0.125

Table 4-8. Central Wellfield Abandoned Wells

Existing			Casing			Perforations	Well
Age		*Material*	*Dia. (inch)*	*Interval (feet)*	*Wall (inch)*	*Interval (feet)*	*Cut (inch)*
No.	*(years)*						
9	na	Mild Steel	20	0 to 155	0.268	90 to 155	¼ x 2

Central Wellfield Observations

The Central Wellfield existing conventional wells are between 28 and 39 years old. Existing Well 2 is constructed of steel pipe and is believed to be perforated, while Well 9 is constructed of mild and stainless-steel pipe and stainless-steel louver screen. Very little construction information was found for Well 2 other than it is 163 feet deep, the well is 39 years old, the geologic log indicates the aquifer formation is all sands and gravels, and the well is high capacity producing about 2,200 gpm.

Much more information was located in the ADWR 55 registry for Well 9. The upper 60 feet of casing is listed as mild steel, followed by stainless steel pipe and screen down to 175 feet. The geologic log for Well 9 indicates it is completed into sands and gravels that overlie a hard conglomerate.

As discussed in the North Wellfield section, the useful life of well casings in Arizona typically ranges from 40 to 50 years. (Roscoe Moss, undated), and considering mild carbon steel casing, the economic life may be only 25 years.

Well specific capacity values in the Central Wellfield are seen to be 115 gpm/ft and 139 gpm/ft for wells 2 and 9, respectively. The specific capacity of the HCW was 484 gpm/ft at the end of the original pumping test, which is very high, but the unique well construction makes the specific capacity value not directly comparable to the existing vertical wells.

Central Wellfield Drawdown Interference

As discussed in the North Wellfield Sections, drawdown created in the aquifer from a pumping well takes the shape of a cone with aquifer drawdown created even at considerable distance from the pumping well. Other wells located within the cone of depression will see this drawdown as a lowering of the water level in the well. This imposed drawdown is known as well interference.

The Central Wellfield existing wells are arranged in a triangular arrangement, with the well spacing ranging between 700 feet to 1,500 feet apart. The Central Wellfield wells are spaced further apart than the North Wellfield and the interference drawdowns for the conventional Wells 2 and 9 are smaller than seen in the North Wellfield. The interference drawdown between the HCW and the vertical wells 2 and 9 are larger but considering the high rate of pumping from the HCW, the interference drawdown is within reason.

Well interference was estimated as discussed in the North Wellfield Sections and were calculated assuming only one well was pumping for each calculation. The calculated interference values are presented in **Table 4-9**. An actual field test would be required to confirm the calculated values.

Table 4-9. Central Wellfield Well Interference

Pumping Well (No.)	Adjacent Wells					
	Closest Well (No.)	Distance (feet)	Drawdown (feet)	2nd Closest (No.)	Distance (feet)	Drawdown (feet)
2	9	699	6.3	HCW	1,509	3.2
9	2	699	8.3	HCW	1,219	4.4
HCW	9	1,219	26.3	2	1,509	25.5

[1] Interference Drawdown is Calculated Assuming Only One Adjacent Well is Pumping

The well drawdown available for pumping drawdown in the conventional Wells 2 and 9 is 62 to 71 feet. The well drawdown available for pumping drawdown in the HCW is 65 feet as limited by the existing well pump setting depths. The maximum well drawdown in the HCW if the pumps were lowered would be about 83 feet.

4.3.4 South Well Field

The City historically operated five wells in their South Wellfield near Rotary Community Park, however, these were abandoned in 2013 (Wilson, 2013). Four of these wells were used for potable water supply and one, Well 21, was used for irrigation purposes. Information on the South Wellfield wells was obtained from ADWR's well registry and is reported in **Table 4-1**.

Based on the limited information available for the South Wellfield, it would be expected that the hydrogeological conditions in the South Wellfield are similar to the North Wellfield. However, all infrastructure in the South Wellfield has been abandoned. The South Wellfield was also located more than 2 ½ miles from the WTP, which would add considerable cost to any water supply options in that area. For this reason, the South Wellfield area should be lower on the priority list of water supply options for the City.

4.4 Capacity Evaluation

Under normal water demands the City must be able to supply the maximum day demand condition with all supply facilities in service, and the redundant water supplies ready in stand-by mode. Peak hour demands exceeding maximum day demands are typically met by operational storage, while the water sources supply maximum day demand.

The City's current fully operational raw water supply components consist of one HCW, and two vertical wells in the Central Wellfield. The vertical wells (No. 2 and No. 9) can be currently operated at or close to their design capacity, however the HCW has apparently lost capacity over its time in service based on the analysis in Section 4.3.1. The design capacity and current observed sustainable capacity of these water supply facilities are presented in **Table 4-10**.

Table 4-10. Future Water Supply Capacities

Raw Water Supply Component	Design Capacity MGD	Current Observed Sustainable Capacity MGD
HCW	25.0	17.5
Well 2	3.2	3.2
Well 9	4.2	4.2
Total	**32.4**	**24.9**

The City water system when all facilities are in operation can supply both the existing and 2040 maximum day water demand as noted in **Table 4-11**. However, little redundancy exists in the case of a raw water supply component failure during current and future demand scenarios.

Table 4-11. Water Supply Analysis

Demand Scenario	Estimated Max Day Demand		Current Observed Sustainable Capacity	
	MGD	gpm	MGD	gpm
Existing	14.2	9,800	17.5	14,900
2040	18.5	12,900	17.5[1]	14,900[1]

[1] The existing HCW has been seen to be experiencing a continual decline in capacity and it is assumed HCW rehabilitation will be successful in restoring and maintaining HCW sustainable capacity somewhere between current and design capacity to meet the 2040 projected max day demand

Existing water demands can be currently supplied by the existing three fully utilized Central Wellfield water supply components under all demand conditions, if there are no unexpected outages of any of the components during a maximum day scenario. A high priority for the City should be to complete inspection on the HCW and perform the recommended maintenance to increase capacity. The next section describes potential water supply shortages and consequences and response needed to continue to deliver reliable supplies.

Seven wells also exist in the North Wellfield and are generally equipped with pumps and piping. However, the North Wellfield wells are not used due to reported heavy sand and silt production. Mechanical issues also are present at two of the wells regarding their pumps and motors.

In the event of a water supply shortage emergency, the City could likely rely on some of the North Wellfield wells for supply, depending on the water treatment plants ability to treat the sand and silt in the produced water. A North Wellfield well review was conducted by and ranked the wells in priority testing for operational readiness. This ranking is presented in Table 4-12.

Table 4-12. Existing North Wellfield Wells Testing Priority

Well	Testing Priority	Capacity (gpm)	Comments
Well 18	1	1,700	Newest Casing, Highest Capacity
Well 15	2	1,300	2nd Newest Casing, 2nd Highest Capacity, Auxiliary Engine
Well 10	3	1,000	16-inch Casing
Well 11	4	700	Recently Tested and Operable
Well 12	5	700	10-inch Casing
Well 13	6	1,100	Reported Motor Failure
Well 14	7	1,100	Pump and Motor not Installed

It may be possible to use the five highest ranking wells shown in Table 4-12 to supply water in an emergency, however some testing and purging should be done to confirm the wells readiness to operate.

4.5 Supply Reliability and Consequences

The City water system is vulnerability to a water supply loss due to the condition of the North Wellfield to provide reliable water supply during an emergency. The City should plan to develop a reliable back-up supply and provide as a minimum between average day (1.0 x avg) and a maximum month supply (1.2 x avg). During the winter months, an average day supply may meet nearly 100 percent of the demand, whereas during the summer months, an average day supply may only meet 70 percent of the demand requiring a level of mandatory water conservation.

The Master Plan recommends re-investing and conducting the necessary well siting and water quality modeling studies to re-develop the North Well Field supply as a high priory project. In the near term, the City should, at a minimum, purge and test the wells to develop some back-up supply for the next several years until new wells are constructed. These water supply projects and timelines are included in Section 8. The City may desire to continue to explore a second HCW, including capacity testing if right of way and permits are secured. However, the updated CIP reflects capital costs associated with development of the North Wellfield. Therefore, this should only be considered if the North Wellfield cannot be fully developed to provide back-up supply.

Table 4-13. Water Supply Events and Consequences

Event	Frequency	Duration	Existing Response Asset	Consequence
HCW Periodic Maintenance	Once every 10 years	1 to 3 months	City Operates Island Wells No. 2 and No. 9. Water restrictions may be required. Improvements to North Wellfield or 3rd Island Well would provide redundancy. Perform HCW maintenance with one pump in service to increase water supply.	**Moderate to Significant.** Without a reliable North Wellfield, the City could only meet approximately 50 to 60 percent of average demand.
HCW Failure: (lateral screen collapse or pump failure)	Once every 30 years	6 to 9 months	City Operates Island Wells No. 2 and No. 9. Water restrictions would be required. Improvements to North Wellfield of 3rd Island Well would provide redundancy	**Significant.** Without a reliable North Wellfield, the City could only meet approximately 50 to 60 percent of average demand.
HCW Supply Pipeline Failure (under Lake)	Once every 50 years	9 to 12 months	Water restrictions would be required. North Wellfield would have to be operated to provide limited capacity. Improvements to North Wellfield or parallel transmission main would be needed to provide redundancy.	**Significant to Catastrophic.** Loss of all Central Wellfield well supplies, including HCW, until pipeline could be repaired. Long term use of existing North Wellfield may impact water quality at WTP due to high turbidity.

4.6 Well Supply Summary and Findings

The findings of this investigation are summarized as follows:

- Lake Havasu City historically operated three wellfields. The North, Central, and South Wellfields. Currently only the Central Wellfield is used to supply water to the City. The South Wellfield has been abandoned with all infrastructure removed. The North Wellfield is generally intact however the wellfield is no longer used because of excessive silt and sand production. The City no longer maintains an active discharge permit to purge the wells.
- The City currently operates one large HCW in the Central Wellfield. The HCW was put in service in the year 2001 and has an installed capacity of 25 MGD. The HCW is highly depended on by the City for most of its water supply needs.
- The aquifers in all three wellfields appear suitable for future use. The North and South Wellfield areas would be expected to support wells with about 2 MGD (1,400 gpm) capacities. The Central Wellfield area is more productive and currently supports two wells, each with over 3 MGD (2,100 gpm) capacities.
- A single 48-inch collection pipeline conveys water from the Central Wellfield to the water treatment plant. Should this pipeline require maintenance or be taken out of service, the City's water supply would be curtailed, requiring use of the North Wellfield.
- All but one of the City's existing conventional wells are over 32 years old and constructed of mild steel casing and perforated screens. Well 9 in the Central Wellfield is 28 years old and much of the casing and screen is stainless steel and would be expected to have a much longer service life than carbon steel. Mild steel cased wells would only be expected to have an economic life of about 25 years.
- Flow and water level data over the past three years from the HCW in the Central Wellfield indicates the HCW is likely losing capacity. Although the HCW can currently supply the water needs of the City, as water demands increase the HCW may not be able to supply all the water supply needs in the future.
- The City water supply system as it exists today has some redundant capacity, but an interruption in service of its HCW or 48-inch collection pipeline could result in a limited supply of water to its customers.
- The City recently explored two locations near the WTP, and another in the Rotary Park Beach area for a second HCW. They found geologic conditions that would preclude setting an HCW caisson in those areas.

4.7 Well Supply Recommendations

The recommendations include both the development of additional redundant well water supply capacities and the servicing of the HCW. Appendix A includes site photos of the well fields and includes preliminary cost opinions to re-develop the North Wellfield over a period of time, which is further discussed in Section 8. The recommendations are as follows:

- An inspection of the HCW was recently completed. The condition of the central caisson, the isolation valves on each HCW lateral, and the condition of the lateral screens were inspected. This inspection and subsequent developments in the area suggest the HCW should be cleaned and rehabilitated.
- Based on the findings of the inspection, develop a rehabilitation and maintenance program for the HCW, Section 8 includes estimated costs

- As part of the cleaning and rehabilitation and maintenance program, develop a plan to prevent service interruptions and unexpected failures during the HCW downtime.
- Conduct an engineered pumping test on the HCW following cleaning and rehabilitation to evaluate the final hydraulic capacity. Compare the test results with the originally reported capacity and assess any degree of capacity loss and the current sustainable capacity.
- Initiate planning and design for the North Wellfield and identify potential well sites for new wells. The planning should be structured around finding the best three or four locations for new North Wellfield wells and the best single site for a new well in the Central Wellfield. The long-term goal would be to develop 7,000 gpm (10 MGD) of well capacity in the North and Central Wellfields to provide water supply redundancy to the HCW.
- Develop potential pipeline alignments for the North and Central Wellfield new well sites.
- Develop new well specifications to be used to solicit bids for the construction of new wells following site selection and acquisition.

4.8 Water Reuse Summary

The City continues to operate a recycled water system which is an integral part of the City's water supply portfolio and wastewater disposal system. The City primarily serve the local golf courses near the existing WWTPs. These include the Nautical GC (Island WWTP), London Bridge GC (Mulberry WWTP), and the Refuge GC (North Regional WWTP).

Irrigation is the primary use of the recycled water, but this water can also be used for plant water, construction, fire flow, and industrial applications that do not require potable water. The Island and Mulberry WWTPs reclaimed water supplies are fully utilized in the summer months and at times require supplemental water from the City's lake intake, which also helps lower the TDS. The North Regional WWTP has excess capacity and the City has plans to expand the recycled system if cost effective and the customer base can be developed. Water not used for irrigation is injected into the vadose zone through wells at the North Regional Plant or infiltrated into the ground via the percolation basins at the Island WWTP. (Carollo, 2015).

The Water Master Plan conservatively assumes the potential irrigation demands identified to be converted to potable water remains on the potable system. **Table 4-14** below shows the existing recycled water users by WWTP service area and includes future potential users that may be converted from potable water to recycled water (Carollo, 2015). The Nautical GC may be considered for future residential development thereby reducing the recycled water demand, although the City does maintain an agreement to use the land as a spray irrigation field for effluent disposal, if needed.

Table 4-14. Existing and Future Reclaimed Water Use

Customer	Acreage	Status	Average Annual Water Use (gpm)	Maximum Daily Water Use (gpm)
Island Plant Service Area				
Island Ball Fields	3.5	Existing	14	27
Islander Resort2	0.5	Existing	2	4
Marina	1.2	Existing	5	10
Nautical Estates	1	Existing	4	8
Nautical Resort	1	Existing	4	8
Nautical Resort Golf Course	50	Existing	200	386
Aquatic Center	0.4	Future	2	3
Grand Island Park	2.86	Future	11	22
London Bridge Beach	2.31	Future	9	18
Rotary Park	13.55	Future	54	105
		10% Loss	30	59
		Subtotal	**335**	**650**
Mulberry Plant Service Area				
London Bridge Golf Course	190	Existing	760	1,466
Cypress Ball Fields	2.67	Future	11	21
ASU Lake Havasu City Campus	6.58	Future	26	51
Jack Hardie Park	0.56	Future	2	4
		10% Loss	80	154
		Subtotal	**879**	**1,696**
North Regional Plant Service Area				
Refuge Golf Course	55	Existing	220	424
Nautilus Elementary School	2.93	Future	12	23
SR 95 Landscaping		Future	9	17
		10% Loss	24	46
		Subtotal	**265**	**510**
		Existing + Future Demand	**1,479**	**2,856**
Future Customers Currently on Wells				
Bridgewater Links Golf Course	30	Future	120	231
Lake Havasu Memorial Gardens Cemetery	5	Future	20	39
		10% Loss	14	27
		Subtotal	**154**	**297**

[1] *Source: Originally presented as Table 1.5 in the Lake Havasu City Wastewater System Improvements Technical Memorandum No. 1*

Section 4.0 Additional References

Building Crafts, Inc., March 2019, “Collector Well Inspection Report, City of Lake Havasu City, Arizona”

Mark Clark, April 2019, City of Lake Havasu, email correspondence dated April 5, 2019, to Greg Froslie, City of Lake Havasu. “Preliminary Info from Exploratory Dives at Collector”.

Water Treatment Plant Assessment

Section 5 provides a summary of current operations of the City's WTP and a condition assessment of the facility based on a site investigation conducted in the spring of 2018. Upgrades and rehabilitation projects are recommended to meet new term and future water supply needs for the City.

WTP Photos from site visit on 2/22/18

5.1 Overview of Water Treatment Plant Process and Current Operations

The City's WTP was constructed in 2004 and designed with a rated capacity of 26 MGD. Currently, the WTP operates at a reduced capacity to meet City demands. The average production in 2017 was reported as 12.5 MGD to satisfy the City's potable water needs. The WTP is primarily supplied by the HCW, with a capacity of approximately 25 MGD, located south of the WTP that pumps groundwater from the local aquifer. Wells 2 and 9 on the island can also provide water supply, as needed, since the groundwater is in close proximity to Lake Havasu, it was originally categorized by the State as under the influence of surface water but was later recategorized as groundwater (not under the influence). The WTP is designed primarily to remove manganese (Secondary MCL = 0.050 mg/L) and arsenic (Primary MCL = 0.010 mg/L). The treatment process includes aeration, ferric chloride addition, biological sand filtration, ultraviolet disinfection, and chlorine gas addition. Filter backwash from the biological sand filters is collected in wastewater holding tanks before being treated by a thickener clarifier and belt filter press. The decant from the wastewater holding tanks is recycled within the WTP and the solids generated are hauled offsite for landfill disposal. A process flow schematic of the City's WTP is presented in **Figure 5-1**.

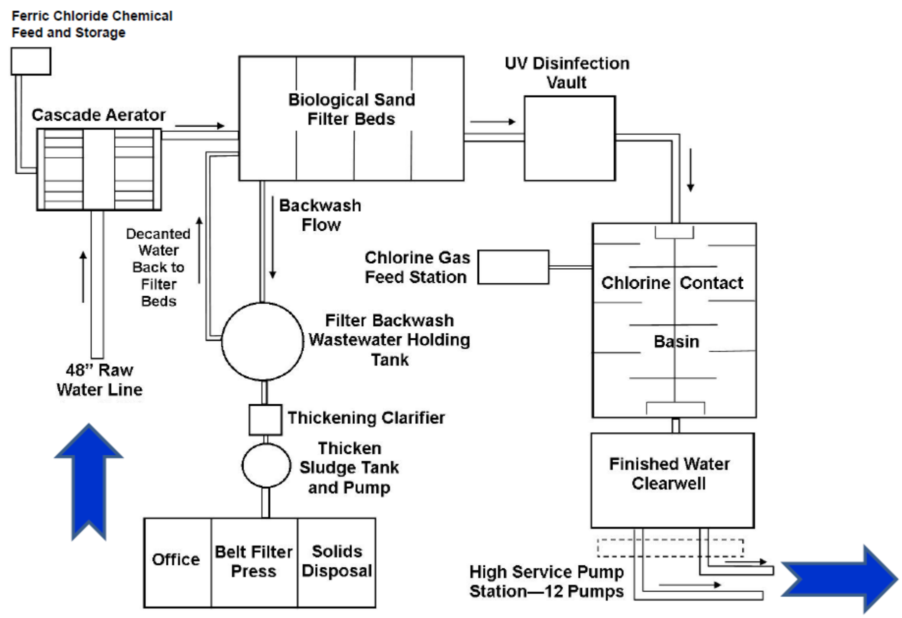


Figure 5-1. Lake Havasu City Water Treatment Plant Process Flow Schematic

Discussions with City staff indicate that the WTP is in compliance with all State and Federal drinking water regulations and meets the City's water quality targets. A review of the City's 2016 Consumer Confidence Report shows that the WTP produces finished water with manganese levels below the detection limit and arsenic levels between non-detect and 0.009 mg/L, which is below the federal MCL.

5.2 Water Treatment Plant

During the site visit, CH2M (now Jacobs) completed a walkthrough of the entire plant to observe and discuss each step of the treatment process. Generally, the WTP is in good condition because the City operating staff has a good maintenance program plan in place and the WTP site is clean and well maintained; however, due to the age of the plant, several equipment pieces appear to be reaching the end of their useful life. These equipment pieces will likely require replacement that can be completed as part of a 20-year scheduled maintenance. The subsequent sections summarize CH2M's observations for each major plant component. Appendix B includes site photos from the visit.

5.2.1 Horizontal Collector Well (HCW) Pumps

Section 4 includes a detailed explanation of the HCW. Although the site visit did not cover the collector well or well pumps, the City is working on completing a condition assessment in early 2019 year to determine if the pumps should be refurbished or replaced as part of a scheduled 20-year maintenance.

5.2.2 Raw Water Transmission Line

A 48-inch Raw water transmission pipe delivers groundwater from the HCW to the City's WTP. Portions of the pipeline are directly under the Lake. To provide redundancy, the City is considering improving the wellfield north of the WTP. A second source of water supply would enhance reliability should the collector well be out of service. Conveyance reliability could be added should the City consider installing a parallel 24-inch from the collector well pumps to the WTP, however, as noted in Section 4, a more cost-effective option would appear to be re-operating the North Wellfield.

5.2.3 Raw Water Flow Meter

Within the WTP limits, the 48-inch raw water pipeline is equipped with a 36-inch flow meter housed in a vault. The original flow meter was an insertion meter but required frequent calibration (every 3-4 days) and was replaced with a Sparling propeller flowmeter that is calibrated by the manufacturer once a year. The City owns two-meter heads that are rotated during calibration. Under low flow conditions (8 MGD), the velocity in the 36-inch flow meter is approximately 1.7 fps, which is a low velocity. The flowmeter manufacturer (Sparling) has confirmed that this is within the acceptable range of the flowmeter. It is recommended that the City perform continue to perform scheduled calibration on this flowmeter as specified by Sparling.

The City also maintains an 18-inch bypass pipe within the plant that was installed after initial construction. The bypass pipe is completely buried, not equipped with any flow measuring devices, and has historically been utilized when maintenance is being performed on the raw water flow meter. Installing a flow meter on the 18-inch bypass line or on the future 24-inch bypass line would allow the City to have a more accurate flow meter if this line was used regularly. A 24-inch flow meter can accurately read flows up to 12 MGD and an 18-inch flow meter could read flows up to approximately 9 MGD.

5.2.4 Cascade Aerator

The raw water pipeline conveys water to a cascade aerator that facilitates oxygen transfer from the atmosphere into the raw water. The target dissolved oxygen level maintained in the water through this process is 8 mg/L. The oxidized environment created by the aerator is needed to facilitate microbial growth in the downstream biological filters. The target pH after aeration of 7.0 to 8.0 is maintained and no pH adjustment (via caustic) is required. To mitigate plant and algae growth on the steps of the aerator, the City installed mesh shade structures over each aerator train.

Dust from the nearby concrete plant covers the exposed pipes within the aeration facility, and likely is also blown into the water. The City should consider a more permanent structure around the aerator that would still provide gas transfer at the top but prevent dust from the neighboring facility as part of the 20-year scheduled maintenance.

5.2.5 Biological Filters

The biological filtration process (MANGAZUR®) consists of four gravity filters that use Infilco Degrmont's Biolite™ media to support microbial growth required to remove manganese and enable high-rate filtration. Ferric chloride is dosed in-line upstream of the filters at approximately 4 mg/L to coagulate arsenic. The filters were designed to operate at a loading rate of 3.75 gpm/ft^2 at the rated capacity of 26 MGD. Generally, the filters operate well and effectively remove manganese to non-detect levels and suspended solids to provide a filter effluent below 0.03 NTU as well as coagulated arsenic. Note that many filtration plants operate at higher loading rates (5 to 6 gpm/ft^2) and the City may consider investigating operating the filters at higher loading rates if and when the WTP is ever expanded.

The filters were originally installed with retractable screens to provide shade and dust protection; however, these were removed due to challenges associated with maintenance. The City operating staff expressed interest in adding an enclosure or building around the filters to serve this same function. This addition should also be considered as part of a scheduled 20-year maintenance.

5.2.6 Backwash System

The backwash system, comprising of the backwash pump and blowers, is used to backwash the filters with unchlorinated, UV disinfected water. The backwash regime includes several steps: (1) air scour only, (2) combined air scour and back flushing, (3) back flushing only, and (4) filter to waste. Since start-up, there have been no operating issues associated with the backwash system. The operating staff occasionally observed turbidity spikes in the filter effluent when the plant flow increases abruptly, and the filters are close to needing a backwash. To address this issue, the City could consider initiating the backwash regime earlier when such flow conditions are observed. Alternatively, the filters could be loaded with additional media; however, this would require further evaluation of the available depth and investigation of the underdrain design. It is recommended that the City consider evaluating this option when the filters are rehabilitated, and the media is replaced once the WTP reaches the 20-year mark. A filter assessment is recommended in the short-term. The filter assessment would include filter measures, media sample analysis, media retention analysis and a filter excavation.

5.2.7 UV Disinfection

The filter effluent is disinfected by a low pressure ultraviolet (UV) light system which consists of two parallel 48-inch UV reactors provided by Wedeco. Each reactor operates at a design dose of 20 to 30 mJ/cm^3 and is installed with 72 lamps fitted across 6 banks. Routine maintenance includes annual cleaning of the lamp sleeves and lamp replacement. The Programmable Logic Controller (PLC) is set up to track the operating time of each lamp and provide lamp replacement reminders to the operating staff upon reaching 14,000 hours of operation (approximately 583 days of online time). Plant staff indicated having issues with optimizing operations of the UV system and has been working with the system supplier (Wedeco) to resolve this. Reprogramming by Wedeco will be completed in 2019.

Based on the operator feedback provided, it appears that the UV lamps are being replaced prematurely based on the rated operating life of 14,000 hours as well as the reduced plant flows. The cause of reduced lamp life could be related to the quality of the power supply or to frequent starts and stops (typically UV lamps should only be subjected to 4 starts/stops per day). It is recommended that the City further investigate this issue to confirm the cause of the premature lamp replacement. In addition, it is recommended that the City explore the prospect of asking Wedeco to reprogram the UV system to automatically toggle operation of each UV bank to maintain a similar online time for all lamps while providing the appropriate dose based on the flow conditions.

Since the State has declared the groundwater supply to not be under the influence of surface water, the City should consider discontinuing the use of the UV facility after discussion with and approval from the State. The potential savings in energy and consumable costs could be significant if this was approved. Additional electrical consumption data would need to be collected to further quantity.

5.2.8 Chlorine Contact Basin/Storage

After UV disinfection, the water flows through the chlorine contact basin (CCB), which is dosed with chlorine to provide disinfection and maintain a residual in the finished water. The CCB has a total volume of 2.5 million gallons and is divided in two equal basins, each of which is baffled to prevent short circuiting. Visual inspection by the operators through access hatches show that the basins are clean, but the City is planning to have the CCB dive inspected.

5.2.9 Gas Chlorine System

The City maintains a gaseous chlorine system that consists of three, one-ton chlorine cylinders housed within secondary containment units called ChlorTainers™ under a shade structure and a small chlorination room in a pre-engineered shed. Currently, the City performs all maintenance on the chlorination system in-house and manually loads each chlorine cylinder. Many water utilities that use gas chlorine, especially around commercial or residential areas are evaluating alternatives for disinfection, if only to minimize their long term need for risk management and hazard communication with the public. It is recommended that the City evaluate chlorine alternatives for the long-term future of the plant, including:

1. Evaluating ways to minimize handling of the one-ton cylinders
2. Replacement of existing shade structure with new chlorine building to reduce risk of chlorine facility
3. Switch from gaseous chlorine to liquid sodium hypochlorite (delivered or on-site generation)

5.2.10 Finished Water Booster Pump Station

The existing finished water BPS, also known as the High Service BPS includes 12 vertical turbine high service pumps. Half the pumps supply the southernmost areas of the City while the other half supply the north and central areas of the City. Each pump has a rated capacity of 3,500 gpm and is equipped with variable frequency drives. Together, the pumps provide nearly twice the duty capacity of the original plant (26 MGD) and more than four times the current average flow. The High-Water Service BPS conveys water directly into Zone 1 where it is boosted throughout the City. Section 7 includes further discussion on its operations and recommendations for future possible upgrades. In addition, the City is planning on replacing the existing ball valves on the discharge piping with cla-type valves based on high repair/maintenance costs and recent input by the vendor indicating that the installed ball valve product line is expected to be discontinued.

5.2.11 Chemical Feed Systems

The WTP was designed and constructed with several chemical storage and feed systems, which are located at various locations in the WTP. **Table 5-1** presents a summary of the original purpose of each chemical system and current status. The City performs regular maintenance activities on the chemical pumps to keep them in good condition. The dosing pumps were original installed without any covers or enclosures for sun protection. The City later installed mesh screens to shade the pumps which require replacement every few years; on occasion the sun shades have been dislodged by under strong wind conditions. The City may consider installing a building or enclosure around the chemical pumps to provide more robust protection from the elements which will help extend equipment life.

Table 5-1. Chemical Storage and Feed System

Chemical	Purpose	Current Status
Caustic	pH adjustment prior to biological filters	Inactive
Ferric Chloride	Coagulation of arsenic prior to biological filters	Active
Potassium Permanganate	Manganese oxidation during startup	Inactive and never required/used
Phosphoric Acid	Nutrient source for biological filters	Inactive and never required/used
Chlorine Gas	Disinfection	Active
Polyphosphate	Sequestering	Inactive and never required/used
Polymer – Backwash Waste Storage	Enhance settling of backwash waste solids	Active
Polymer – Plate Settler	Enhance clarification of backwash waste solids	Active
Polymer – Belt Filter Press	Dewatering aid to increase cake dryness	Active

5.2.12 Backwash Wastewater Holding Tanks, Solids Pump Station and Decant Pump Station

The backwash wastewater holding tanks provide storage and equalization for process wastewater from the biological filters (filter backwash and filter to waste) and thickening clarifier decant. Each of the two wastewater holding tanks are dosed with polymer to promote the settling of solids, which are directed to the thickener clarifier, while the decant is recycled back to the head of the WTP and blended with the raw water. Typical maintenance activities for the wastewater holding tanks include regular rinsing performed through access hatches located on the dome covers that enclose each tank. The City may consider segregating the cleaner process waste flows (e.g. filter to waste) from other flows (e.g. backwash waste, plate settler decant) to reduce the frequency of cleaning required for one of the wastewater holding tanks.

The solids PS, which conveys solids from the wastewater holding tanks to the thickener clarifier, and the decant PS, which convey the wastewater holding tank overflow back to the head of the WTP, are both maintained in-house by the City. The City is currently in the process of rebuilding both the solids and decant pumps, as well as replacing the controllers to the motors as part of scheduled maintenance.

5.2.13 Thickener Clarifier

The solids in the wastewater holding tanks are treated by a single duty thickener clarifier which is dosed with polymer. The thickened solids are collected in a small holding tank and PS that delivers the solids to the belt filter press system. No operational or maintenance issues for these facilities were reported by the operating staff.

5.2.14 Belt Filter Press

The City operates a single duty belt filter press (BFP) to dewater the thickened solids generated by the thickener clarifier. The BFP is housed in a building and is installed to drop dewatered solids into a conveyor belt to transfer solids to a trailer parked in a truck bay adjacent to the belt filter press room.

The dry solids are disposed of at the Municipal Landfill and the filtrate from the BFP is discharged to the sewer. The dewatering process is operated on a batch basis (approximately once a week under current flow conditions). Both the BFP and conveyor systems have performed well with limited operational and maintenance issues.

5.2.15 Backup Power Supply

Discussions with the City operations staff indicate that the WTP has not experienced any extended power outages and that the backup power supply is adequate.

5.2.16 Miscellaneous Observations

In addition to what has already been described, the following observations were made based on the site visit and discussions with the City staff:

- As part of the ongoing plant maintenance, the City will be adding soft starters for all pump and motors to reduce the load during start-up and extend equipment life.
- The majority of the field instrumentation was originally installed without any sun protection. The City later installed mesh screens to shade the instruments which require replacement every few years. It is recommended that the City consider installing enclosures at various locations at the WTP to house adjacent field instrumentation, to better protect this important equipment. This should be completed as part of the 20-year scheduled maintenance and has been budgeted in the CIP included in Section 8.
- Operating staff indicated having limited space in the existing operations building for storage or for conducting meetings. The City may consider adding a building to provide additional space for these functions. Additional building square footage of 800 to 1,000 square feet is assumed and is presented as a lower priority project outside the five-year CIP.

5.3 Water Treatment Plant Capacity

Based on the design criteria presented in Section 2, the WTP is required to supply the maximum day demand for the City water system. Under existing demand conditions as presented in Section 3, the existing maximum day supply is estimated to be 15.7 MGD. Therefore, the WTP is operating a little over 60 percent of design capacity. Normally the WTP sees flows in the range of 8-11 MGD on an average day.

At 2040, the maximum day supply required is projected to increase to 19.3 MGD, which is still below the rated capacity of 26 MGD. Therefore, no capacity expansions are required to the WTP by 2040. The City should continue to monitor demands and maximum day use in the event the City experiences more rapid growth. Industry criteria is to plan for an expansion once a WTP reaches approximately 80 percent of design capacity.

5.4 Water Treatment Augmentation Feasibility Study

To address increasing concerns with disinfection byproducts (DBPs) and salinity, a Water Treatment Augmentation Feasibility Study was performed to evaluate opportunities to modify the existing treatment process to reduce organics and total dissolved solids (TDS) in the potable water supply. Enhancing organics removal at the WTP, will enhance the finished water quality by mitigating the formation of DBPs, namely trihalomethanes (THMs). Improving the TDS of the potable water supply will provide significant aesthetic benefits to the potable water (hardness removal) and could result in a reduction or elimination of point-of-use water softeners within the residential and commercial community. Ultimately, any reduction in the potable water TDS will also result in a TDS reduction in the reclaimed water, thereby increasing its potential for beneficial reuse, which is important to the City as it continues to expand its recycled water program.

The study considered several alternatives which include the addition of treatment processes between the existing biological filtration and ultraviolet disinfection systems. The final shortlisted alternatives included [1] lime softening followed by granular activated carbon (GAC), [2] GAC and [3] reverse osmosis (RO). Both Alternatives 1 and 3 would provide removal of hardness and organics but will require significant residuals handling facilities to manage lime solids and RO concentrate waste generated from each respective Alternative. Conversely, Alternative 2 would only provide organics removal but not require additional residuals handling. A cost analysis showed that Alternatives 1 and 3 are not feasible and cost prohibitive while Alternative 2 is feasible and could be considered to enhance organics removal at the WTP. A conceptual design with more refined cost estimates would need to be prepared, only this planning effort is included in the CIP at this time.

5.5 Summary of Recommendations and Considerations

Based on the site visit, the City's WTP is generally in good condition and the site is well maintained. As noted in this document, the City has several ongoing maintenance activities in progress and could benefit from several other upgrades and improvements to the plant. These ongoing activities and recommended improvements are summarized in **Table 5-2**.

Table 5-2. Recommended WTP Improvements

Category	Description
Ongoing or Planned Maintenance	• Condition assessment of horizontal collector well. • Dive Inspection of chlorine contact basin. • Replacement of high service pump ball valves with cla-type valves. • Addition of soft starters to all pump and motors.
Recommended Improvements	• Installation of new flow meter on existing 18-inch raw water bypass line. • Installation of enclosure around the cascade aerator and biological filter system. • Filter assessment, rehabilitation and media replacement. • Investigate potential to increase media depth of biological filters to mitigate breakthrough. • Reprogramming of UV system to optimize lamp operation. Further investigation of the cause of premature lamp replacement is also recommended. • Upgrade Chlorine Disinfection System to enhance operations and safety through the addition of a new chlorine and minimizing loading requirements for the chlorine cylinders. In addition, the City may consider evaluating the potential of switching from gaseous chlorine to liquid sodium hypochlorite. • Install enclosure for field instrumentation. • Construct new operations building (800-1000 square feet) to provide additional meeting space and storage for City operating staff.

Section 8 presents recommended WTP projects in a 5-year CIP and includes estimates for the capital costs. Operational and maintenance projects are also noted; however, maintenance projects would likely be funded by the City Water Operations budget.

Hydraulic Water Model Development

Section 6 describes the review and update of the water system hydraulic model used to analyze the existing water system and identify system updates.

6.1 Existing Water Model Summary

As part of the 2007 Water Master Plan, the City developed a hydraulic model of the City's water system using the Innovyze H2OMap v.10 software based on GIS database information, aerial survey data and as-built drawings. The model includes pipes, BPSs, reservoirs, pressure reducing, and valves. Demands were allocated into the model utilizing customer billing data. The model was later updated and calibrated in 2010 as part of a CIP update. Due to additional water infrastructure and changes in water demands within the City system, the City recently went through an effort to recalibrate and update the H2OMap hydraulic model and incorporate pump and tank upgrades and pipeline improvements. As part of this update, the hydraulic model demand usage and diurnal patterns were reviewed and revised based on City SCADA data and updated meter demand data and are documented in Appendix C (Atkins, 2017). The model was calibrated for both steady-state and extended period simulation scenarios.

The hydraulic model was reviewed as part of this Water Master Plan Update and demands have been updated and allocated using 2016-meter billing data as a baseline; however, no major infrastructure updates were done, and the model was assumed to be calibrated and representative of the City's current water system.

6.2 Model Review and Updates

6.2.1 GIS Data

The modeled facilities were checked against the City's GIS database and include the geographic network of pipes, nodes, tanks, BPSs, valves and supply sources representing the City's potable water system. The model stores the facilities along with associated data as described below. Pipe information includes pipe diameter, length, material (if known), age of pipe, roughness coefficient, and an associated service pressure zone. The roughness coefficient, known as the Hazen Williams "C" factor, is used to estimate friction losses within the pipe. The "C" factor is assigned based on pipe diameter, material, age of pipe and is commonly adjusted during calibration efforts to better represent the actual system operational results. Node information includes an elevation, associated service pressure zone, and an allocated demand usage and diurnal pattern. Pump information includes a site elevation, number of pumps and their associated pump curves, and a service pressure zone. Pump settings were set based on the analysis and updates as part of the model calibration discussed below. Tank information includes a site elevation, tank diameter, high water level, and a service pressure zone.

6.2.2 Demand Allocation

The City provided water meter billing data for the 2016 baseline year. Irrigation meter billing data was later provided as a separate file and then incorporated into the potable demand totals for the City. The baseline potable water demand used for modeling purposes is 9.5 mgd. Demands were assigned in the model using the H2OMap Demand Allocator module based on meter location and the closest modeled node, resulting in approximately 30,000 meters spatially allocated to existing nodes in the model.

Future demands were allocated by pressure zone, distributing the forecasted demand across nodes within that pressure zone unless specific development demands were identified and located. An additional 2.8 mgd was added to the baseline model scenario for 2040, resulting in a future modeled demand of 12.3 mgd.

6.2.3 Model Calibration

The hydraulic model was calibrated under steady-state conditions prior to this Water Master Plan Update. A steady state model represents a snap shot in time of the existing system and is considered to be a high level, macro calibration as global adjustments are done within the model to approximate static operating conditions. “Macro” level calibration procedures utilize continuous pressure monitoring to obtain data points to simulate system operations over an extended period of time. The data is used to establish boundary conditions for steady state calibration.

While utilizing the calibrated “existing system” model to run initial simulations and evaluate future development scenarios, minor revisions were made to the model to correct zone boundary connections and confirming PRV settings in the higher zones.

Water System Evaluation

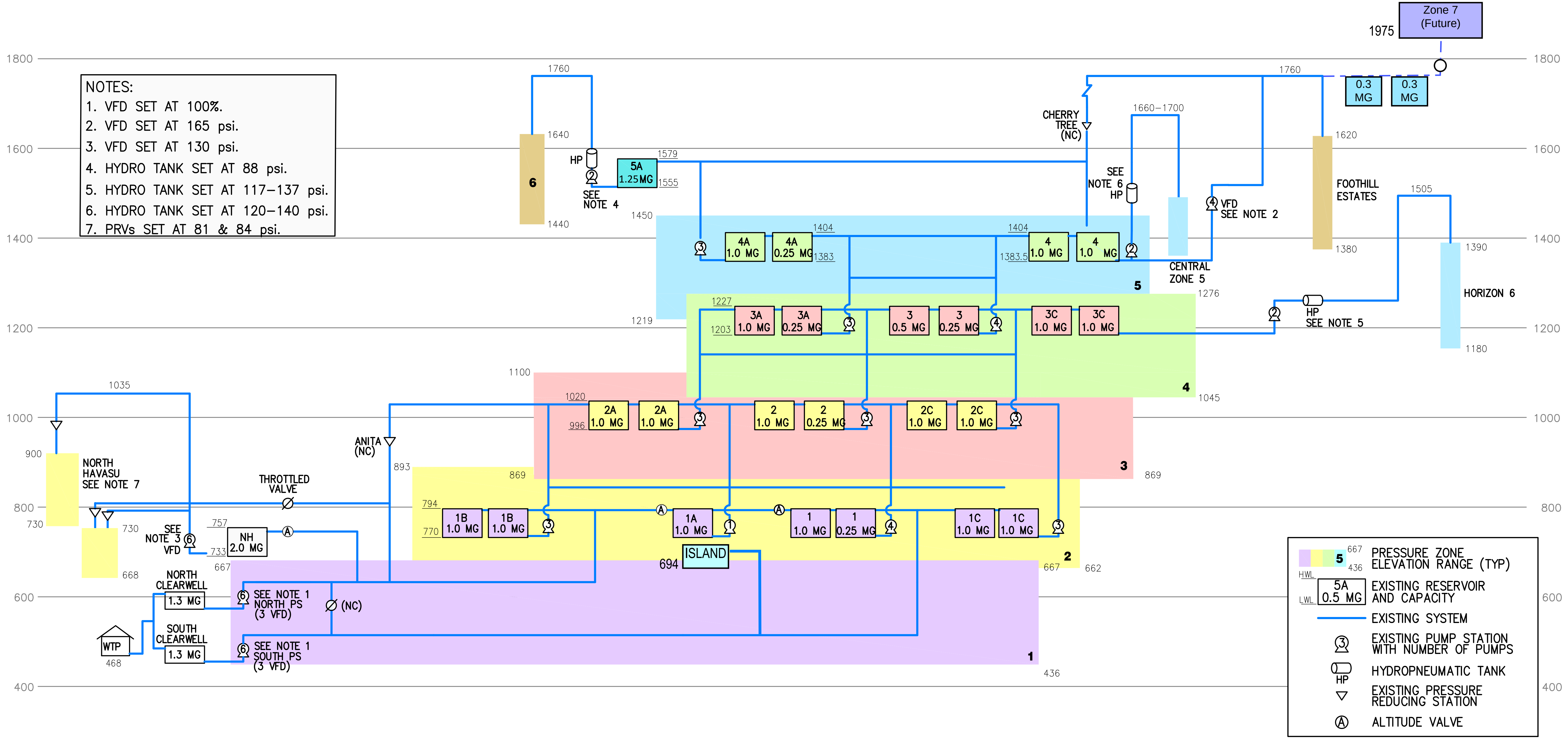
Section 7 presents an overview of the existing water system and its current operations, including a summary of each pressure zone. Existing BPS and storage facilities are evaluated based on the design criteria in **Table 2-3**. The capacity analysis of the existing water distribution system is based on the updated hydraulic model presented in Section 6 and Appendix C, including identification of system deficiencies. Future water system improvements are identified to correct any existing deficiencies and to meet future growth. Cost of the recommended capital facilities and major maintenance items are presented in Section 8.

7.1 Existing Water System

The existing water system consists of over 475 miles of water distribution and transmission pipelines to serve the City. A large percentage of the water systems was constructed between 1960 and 1980 as the City population expanded. Most of the older distribution pipeline was constructed of asbestos cement (AC) and transmission mains were typically cement mortar lined steel (CML&C) pipeline or ductile iron pipeline. Today, most of the new water distribution system is constructed of polyvinyl chloride (PVC) pipe.

All potable drinking water supply originates at the City's WTP and is pumped through the High Service BPS to the north and south via 30-inch and 36-inch diameter transmission pipelines, respectively. The City's water service area topography varies over 1,200 feet, ranging from near 450 feet along Lake Havasu to as high as 1,650 feet in the eastern foothill areas. As a result, the City has constructed a water system consisting of narrow bands of pressures zones established by a series of BPSs and reservoirs and interconnecting transmission mains. In 2018, the City operated 14 water BPSs and 26 distribution system reservoirs, not including the WTP clear well storage.

Figure 7-1 illustrates the City's water system in a hydraulic profile including High Water Levels (HWLs) which establish hydraulic elevations and static pressures, as well as the service elevations between zones. The City has targeted about 200 feet between pressure zones allowing service pressures to typically range from 50 psi to 120 psi. Each pressure zone is unique, and in some cases, high pressures exceeding 150 psi may be necessary to optimally provide water service and not create reduced isolated pressure zones to operate. The following section describes each of the major pressure zones.



POTABLE WATER HYDRAULIC PROFILE
FIGURE 7-1

Source: ATKINS Model Calibration TM, 2013

7.1.1 Pressure Zones

The City's water system is separated into seven major pressure zones which generally run parallel to one another from west to east and have been referred to sequentially starting with Zone 1 originating from the WTP. A number of pressure zones have been planned to extend north and south within the City limits, although the upper zones may only serve isolated areas due to topography constraints. In addition, the pressure zone's service areas decrease in size from the west and central more densely served population areas to the far eastern foothills.

Water is supplied from BPSs in each pressure zone to distribution reservoirs, and in most cases via dedicated transmission mains ("tranny lines"), where reservoirs gravity feed customers in the zone. There are a several closed zones within the distribution system that are served solely by BPSs. The existing pressure zone boundaries are illustrated on **Figure 7-2**. Major components and operations of each of the main pressure zones are described in the following subsections.

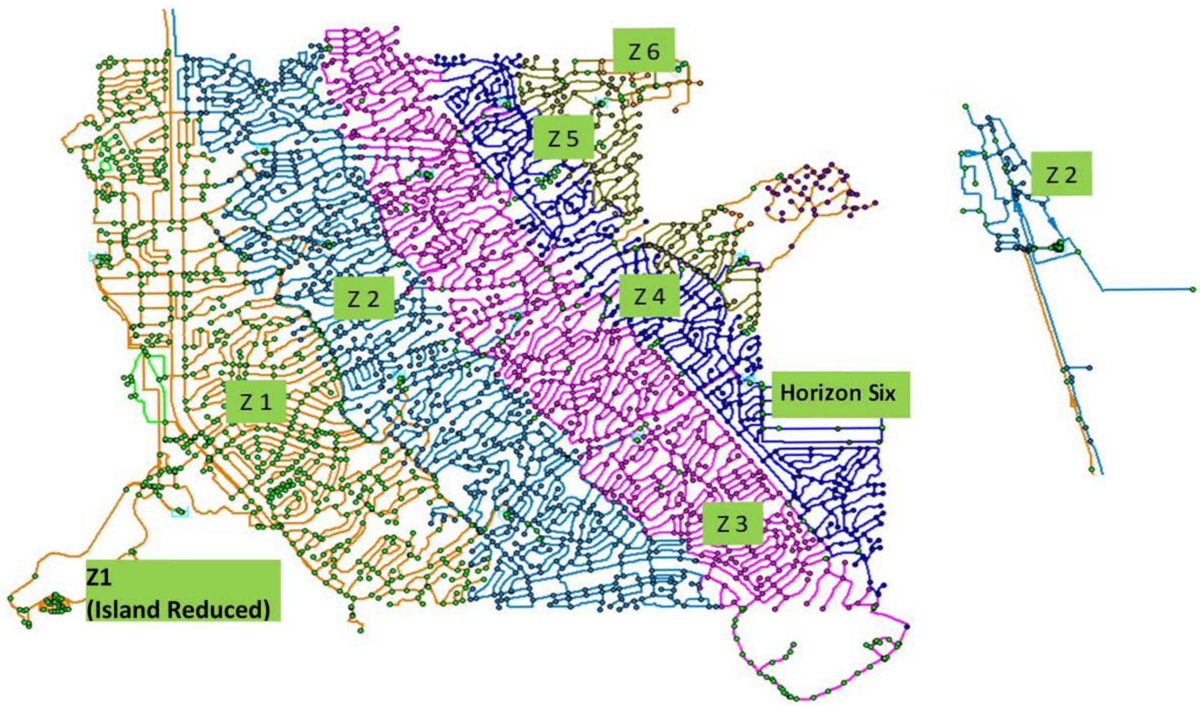
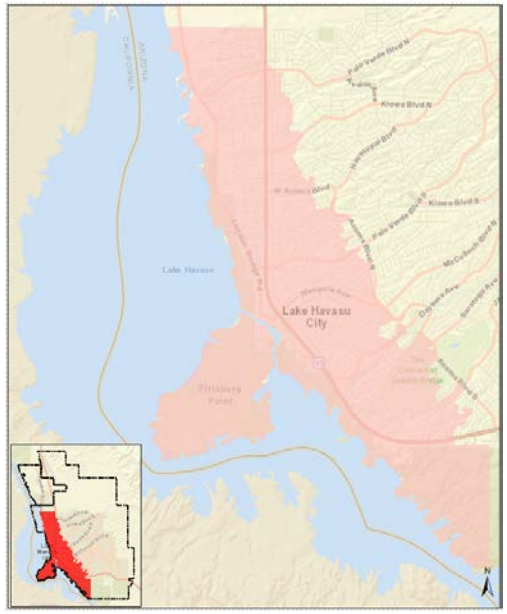


Figure 7-2. Pressure Zone Boundaries

Zone 1

Zone 1 is the first pressure zone established by the High Service BPS at the WTP and supplies all other pressure zones in the water distribution system. The City generally operates a north and south Zone 1 system by splitting the High Service BPS into a north and south delivery, with dedicated pumps respectively. Zone 1 originates at the edge of Lake Havasu and extends east to approximately Acoma Blvd. The zone extends to the north and south to the City limits and primarily includes the State Highway 95 corridor.



Zone 1 (and Reduced Zone 1)

Zone 1 establishes a hydraulic grade line (HGL) of 794 feet, based on the Zone 1 reservoirs HWL and generally serves elevations from 430 feet to 670 feet. Zone 1 is the most populated and includes the central downtown area and all the Lake-front development. The existing average annual demand for Zone 1 is 3.0 mgd. However, during maximum day demands, Zone 1 must supply the entire City-wide maximum day demand of approximately 14.2 mgd (9,900 gpm) to the higher zones.

Zone 1 consists of seven distribution reservoirs and the High Service BPS at the WTP. One set of pumps supplies the north and the other set the south. Zone 1 is characterized by a network of looped water mains ranging in size from 4-inch to 36-inch in diameter. The available Zone 1 storage is 6.3 MG within the seven tanks. Moreover, the City operates two tanks at each of the following sites: 1B, 1, and 1C, and only one tank at Site 1A. All tank sites in Zone 1 include BPSs to serve Zone 2, although Site 1A includes only a single natural gas fired pump that is manually operated.

The High Service BPS was designed and constructed in the early 2000's to serve full build-out of the City, estimated to be 50 mgd of maximum day capacity at the time (note: the 2019 Master Plan updates the 96,000 people build-out maximum day demand to just over 25 mgd). A total of 12 pumps were installed, each at 5 mgd to serve the north and south systems. The City only needs to utilize about 30 percent of the station's capacity to meet maximum day demands. As presented in Section 7.2, the required capacity by 2040 for the High Service Pump Station is approximately 20 MGD. This would require the City to operate a minimum of four pumps plus standby pumps for the north and south, respectively. As pumps are replaced or rehabilitated at the station due to age, the City should explore several options:

- Retire and not replace two to four of the pumping units since the capacity is not needed and reduce overall maintenance costs.
- If pumps need to be replaced, consider the benefit of lower capacity pumps (2 to 2.5 MGD) to work with the larger 5 MGD pumps to potentially improve efficiencies and operational flexibility.

In general, the north pumps at the High Service BPS are operated and controlled by Tank 1B and the south pumps by Tank 1C. A robust transmission system of 24-inch through 36-inch conveys water to the next pressures zone and BPSs.

Zone 1 also includes a small pressure reduced area (Reduced Zone 1) consisting of three PRSs supplying the Island area distribution system. Pressures are reduced about 50 psi from Zone 1 (694 feet HGL), i, thereby creating a small 700-foot Reduced Zone 1, otherwise pressures would exceed 150 psi. Zone 1 also is the sole dedicated feed to the remote North Havasu Tank (HWL = 757 feet), approximately 5 miles north of the WTP, which serves the Walmart commercial area. A single 30-inch and 20-inch transmission main conveys the required water demand and refilling of the tank for this North Havasu system.

Zone 2

Zone 2

Zone 2 is parallel to Zone 1 and extends in a north-east pattern with a zone band width of approximately 1.2 miles, due to the City topography. Zone 2 is served by six tanks with a total storage capacity of 5.3 MG. Two tanks are located at sites 2A, 2, and 2C, respectively. The BPSs at sites, 1B, 1, and 1C establish a hydraulically balanced pressure zone of pumped supply and storage operations within Zone 2, therefore minimizing large pressure swings during peak hour demands. BPS 1A is a stand-by pump supply for Zone 2. The Zone 2 service area is the beginning of more residential portions of the City's pressure zones, although a fair amount of commercial development is still served.

Zone 2 establishes a HGL of 1020 feet, based on the Zone 2 reservoirs HWL and serves elevations ranging from 670 feet to 900 feet. The existing average annual demand for Zone 2 is 3.0 mgd, very similar in size to Zone 1. During maximum day demands, Zone 2 also must supply Zone 3 through Zone 6 maximum day demands of approximately 9.8 mgd (6,800 gpm). Section 7.1.3 includes an evaluation of available BPS capacity within Zone 2.

The Zone 2 transmission mains radiating from each of the three main BPSs range in size from 20-inch to 27-inch in diameter and provide both supply capacity within Zone 2 and to the upper pressure zones as well. Most of the residential areas with Zone 2 are supplied by smaller diameter distribution mains ranging in size from 4-inch to 8-inch.

Zone 2 can also be supplied at times by the North Havasu BPS. The North Havasu BPS must be periodically used to manage water quality in the North Havasu 2.0 MG Tank. The City will operate smaller pumps at the North Havasu BPS to meet daily commercial demands in the Walmart area and excess supply is conveyed back south to Zone 2, nearly 2 miles via a 10-inch pipeline. The North Havasu Zone is further discussed in the next subsection.

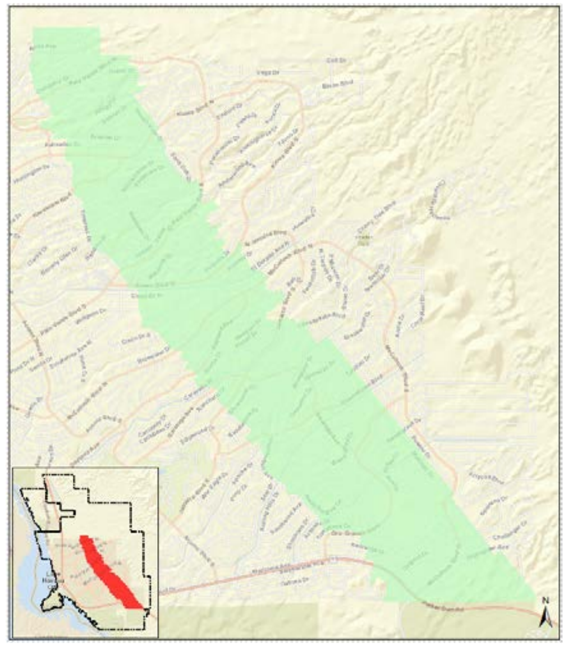
North Havasu Zone 1035 Zone and Reduced 900 Zone

North Havasu Tank (HWL = 757 feet) and North Havasu BPS were constructed to provide water service to the Walmart commercial area, which required a large fire flow (5,000 gpm for 5 hours). The tank is solely filled by Zone 1 through the 20-inch transmission main. The North Havasu BPS delivers pressures at an HGL of 1035 feet, slightly highly higher than Zone 2 (1020 feet) which had resulted in high water services pressures, (over 150 psi) in the Mall area. About five years ago the City created a reduced pressure zone to mitigate the problems of high pressure at the Mall and established a new 900 Zone by constructing two PRSs, near the Airport area. This appears to have solved the problems associated with the high-pressure service failures in buildings and fire sprinkler systems.

The City's major challenge is to maintain stand-by fire pumps for the Mall area with a large reservoir that does not turnover with current domestic demands. Hence, the need arises to recirculate water south back to Zone 2. In the future, the North Havasu storage site will serve as a main water supply to the east as the northern portion of the City develops.

Zone 3

Zone 3 becomes a tighter pressure zone band, approximately 1 mile wide, as the water system extends up steeper topography to the east, while maintaining the desirable service pressure ranges. The Zone 3 system maintains a similar distribution system as Zone 2, with a backbone system of three tanks and three BPSs, respectively. Transmission mains ranging in diameter from 12-inch (in the north zone) to 24-inch, deliver water to the reservoirs. However, there is limited transmission from north to south in the zone. The zone is characterized by looped 8-inch distribution lines that feed a network of smaller pipes. Zone 3 is served by a total of six tanks with two tanks at each site (3A, 3, and 3C). The total available storage is 4.0 MG. BPSs 2A, 2, and 2C provide the pumping capacity in a similar balanced delivery mode as Zone 2.



Zone 3

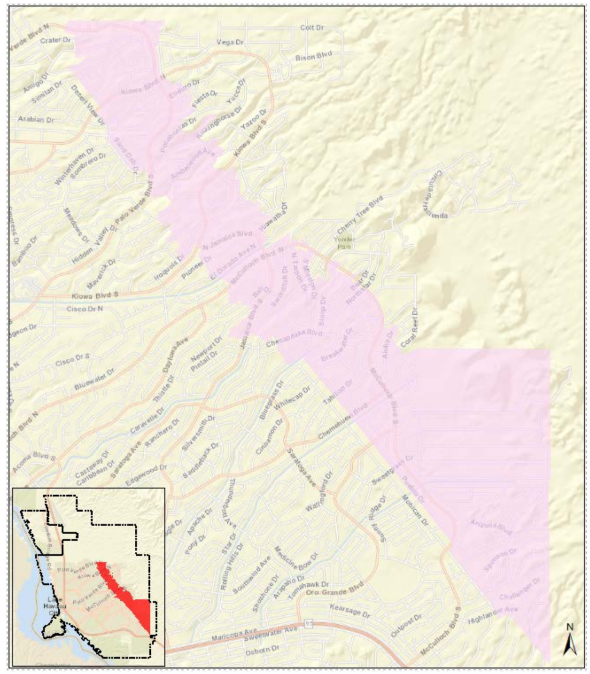
Zone 3 establishes a HGL of 1227 feet, based on the Zone 3 reservoirs HWL and serves elevations ranging from 870 feet to 1100 feet. The Zone 3 service area is predominately residential with some local neighborhood commercial and schools. The existing average annual demand for Zone 3 is 1.9 mgd, about two-thirds the size of Zone 2. During maximum day demands, Zone 3 will be required to supply Zone 4 through Zone 6 maximum day demands, as well, totaling approximately 5.3 mgd (3,700 gpm). Section 7.1.3 includes an evaluation of available BPS capacity within Zone 3.

Reservoir Site 3C serves the far southeast of the City. In addition, the reservoir site serves as forebay storage for the County's Horizon Six water system, which includes a BPS that supplies water to a closed zone to serve a small County area consisting of 240 connections.

The City has recently expanded the Zone 3 water system to serve the new Sara Park water system to the south and created a looped system through the Park by connecting east of State Highway 95 to the Vagabond PRS located at the edge of Zone 4. Sara Park is fed by parallel sources: from Zone 3 by gravity and Zone 4 via the Vagabond PRV. The City is currently in design for a pipeline upgrade across State Highway 95, where the project may include abandoning the older Vagabond PRS and use of the new Sara Park PRS, since they are in series.

Zone 4

Zone 4 is another narrow pressure zone band serving from the far southeast corner of the City up to north of N. Kiowa Road. The western boundary of the zone follows the WAPA Power Transmission Line easement from south to north. The Zone 4 system includes a backbone system of four tanks and two BPSs. Site 4A includes a 1 MG and 0.25 MG tanks. Site 4 has two 1 MG tanks for a total zone capacity of 3.3 MG.


Zone 4

Zone 4 establishes a HGL of 1404 feet, based on the Zone 4 reservoirs HWL and serves elevations ranging from 1045 feet to 1276 feet. The Zone 4 service area is mostly residential. The existing average annual demand for Zone 4 is approximately 0.8 mgd.

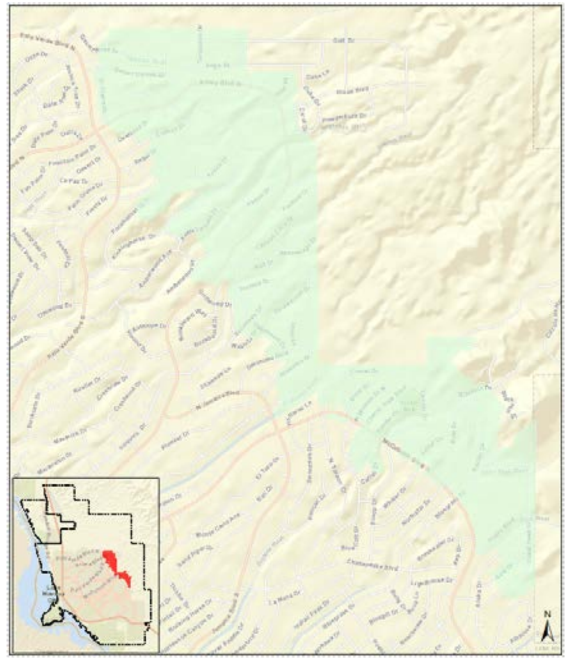
The two BPSs that feed Zone 4 in the City are located at Sites 3 and 3A. A new BPS has been constructed by the County at Site 3C dedicated to serve the Horizon Six water system (discussed below). Section 7.1.3 includes an evaluation of available BPS capacity within Zone 4.

Horizon Six 1500 Zone (County)

The Horizon Six water system, although owned and operated by the County, does take water directly from the City water system at the County's new BPS at Site 3C. Therefore, the County system needs to be taken in to account when evaluating capacity in the City water system. The Horizon Six water system consists primarily of a network of 6-inch water mains between Little Finger Road and Windowrock Road, just east of Lakeside Drive and west of Red Rock Road.

The County has multiple PRSs to reduce pressure in the Horizon Six system, however these appear not to be working based on recent hydraulic studies conducted by the County. The County does have future plans to upgrade the domestic BPS to add fire pumps, as currently the water system has limited fire flow capacity. The existing average annual demand for the Horizon Six water system is approximately 0.06 mgd or 45 gpm.

Zone 5



Zone 5

Zone 5 is established in the far northeast corner of the City where two reservoirs are co-located at Site 5A (0.5 MG and 0.75 MG, respectively), totaling an available storage capacity of 1.3 MG. This north portion of Zone 5 is served solely by BPS 4A. Three pumps are located at BPS 4A for a total pump capacity of 1,150 gpm.

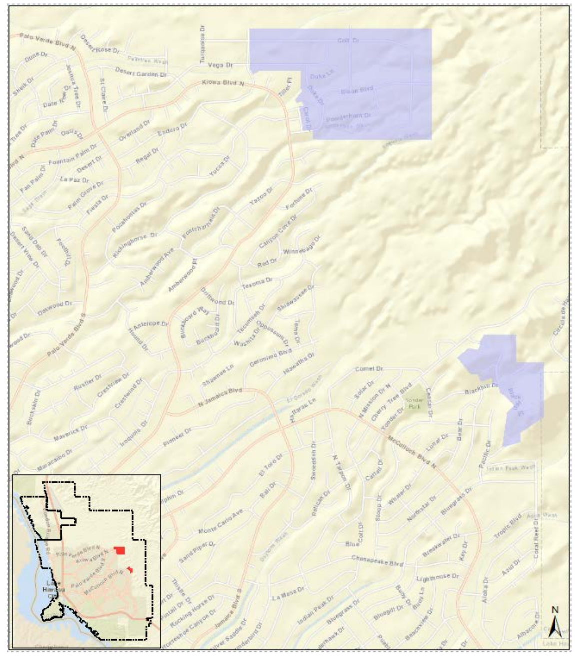
Zone 5 establishes a HGL of 1579 feet, based on the Zone 5 reservoirs HWL and serves elevations ranging from 1220 feet to 1450 feet. The Zone 5 service area includes some of the higher residential areas of the City. Zone 5 extends nearly three miles to the southeast from Tank 5A, making it one of the longer zones not supported by a second reservoir and therefore can be subject to higher pressure swings. In lieu of a second reservoir to the south, the City has installed and operates the Cherry Tree PRS, interconnected with the higher-pressure Foothills water system, to provide additional Zone 5 supply, especially during peak demands. Reservoir siting constraints make it unlikely that a southern Zone 5 tank will be constructed.

A small central section of Zone 5 (30 +/- homes) is served by a separate BPS 4 (Hydro) located at Tank 4 but is not interconnected with Zone 5. The City may want to explore serving this area in the future off the higher-pressure Foothills water system via a PRS once the Foothills system is completed as the current Hydro station has limited fire flow and is in need of upgrade. The existing average annual demand for the entire Zone 5 is approximately 0.4 mgd.

Zone 6

Zone 6 consists of a small closed zone serving the higher elevations around Tank 5A and includes the new water system being constructed as part of the Foothills Estates water systems. These two water systems are not connected and unlikely would be in the future due to topography constraints. The closed Zone 6 water system is supplied by BPS 5A (Hydro).

The City has identified a site for a future Tank 6A to convert the closed system to an open system and simplify operations, but because of the small demands in the zone, the City plans to serve the system from Tank 5A in the near term. In the future, when Tank 6A is constructed, Zone 6 will establish a HGL of 1760 feet and will serve elevations ranging from 1440 feet to 1640 feet. The other part of the Zone 6 system, further to the southeast, only includes the Foothills development as discussed below.



Zone 6

Foothills Estates Zone (1760 and 1975 Zones)

As part of the Master Plan, the Foothills Estates development water system was reviewed to determine an optimum system configuration based on the existing BPS 4 supply, pressures and the availability of tank sites both onsite and offsite. Through a series of workshops with the developer's engineer, the Foothills water system was established to include a new Zone 6 and a closed Zone 7. The latter to serves the proposed residential units in the development that cannot be supplied by the new Zone 6 tanks.

The new Foothills Zone 6 establishes an HGL of 1760 feet, based on the proposed two 336,907-gallon Foothill reservoirs (under construction) and will serve elevations ranging from 1,440 feet to 1,640 feet. All water supply to the expanded Foothills Zone 6 will continue to be from BPS 4, which will shift from a closed zone to an open system with reservoir level control. Four pumps, which have a capacity of 230 gpm each, are installed which results in an available zone pump firm capacity of 690 gpm.

The higher elevations above Zone 6 within Foothills Estates will be supplied be a large pumped closed system due to constraints to site a higher elevation reservoir within the development. The closed zone will operate as a Zone 7 system and could be expanded to the north and east in the future. The latest designed plans for Foothill Estates show that the new closed Zone 7 will operate near HGL of 1975 feet and serve elevations ranging from 1,635 feet to 1,790 feet. The proposed BPS is currently under design and will need to supply both domestic demands and fire flow.

7.1.2 Summary of Pump Station and Reservoir Design Criteria

This section provides a brief summary of the BPS and reservoir criteria presented in Section 2 followed by detailed capacity analysis in the next Section 7.1.3.

Storage Criteria

Water supply facilities are designed to operate at a steady rate over an extended period of time, so storage reservoirs are planned to accommodate fluctuating demands. The factors included in designing reservoir capacity are diurnal demand fluctuations, fire flow, and emergency reserve storage. In some situations, it may be prudent to have additional storage volume to provide additional operational storage. Storage facilities should be designed and operated to meet these conditions, while achieving storage turnover to minimize water quality degradation.

The City has also undertaken a City-wide program to rehabilitate the steel tanks in the water system. Depending on the condition of the tanks, the facilities may be out of service for up to one year during construction. With the large number of tanks in the City system, it is the City' desire to plan for two tanks at a single reservoir site, so one tank can be taken out of service for an extended period of time.

Moreover, storage analysis on a zone by zone basis typically considers full capacity of the tank. In reality, due to overflow elevations and pump level controls, typically less than 100 percent of a tank is available for water storage. Therefore, the "effective storage" for a typical tank may only be about 80 to 85 percent of the total tank volume. The storage tank capacity evaluations take these factors into consideration.

Pump Station Criteria

The City's BPSs boost the water pressure so that service may be provided to users at a higher elevation. This is accomplished by a number of "series" BPSs to move water from Zone 1 to Zone 6. BPSs 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 a service area without a storage reservoir. BPSs supplying a closed system must regulate pressures utilizing multiple pumps, variable speed drives, and/or a hydropneumatic tank.

The City water BPSs as presented in **Table 7-2** are sized based on the following criteria:

- BPSs serving a reservoir system should be designed for maximum day demands.
- When pumping to a closed system, the capacity should equal the larger of either peak hour demand or maximum day plus fire flow demand.
- BPSs should be sized to meet demands with the largest pump out of service (firm capacity).
- When multiple booster stations (minimum of three) supply a zone, average annual water demands should be supplied with the largest BPS out of service.

7.1.3 Capacity Analysis

This section presents a capacity evaluation of the City's storage tanks and BPS per the recommended design criteria.

Existing Water Storage Analysis

Table 7-1 presents an analysis of the storage capacity in the existing water system based on the design criteria and consideration of "effective storage" in each reservoir. Referring to calculations in 7-1 most of the major zones have surplus capacity. Zone 2 may benefit from additional storage in the near term. The existing closed zones including Zone 6 and Foothills Zone 6 are both planned to include future tanks to mitigate current system deficiencies.

Existing Pump Station Analysis

Table 7-2 includes a zone by zone pumping capacity analysis. Each zone is evaluated for zone pumping needs and then compared to the available pumping capacity using the existing pumps serving a zone. Unlike storage which only looks at the zone demand only; the pumping analysis must consider supplying each higher-pressure zone. For example, Zone 3 must be able to supply the Zone 3 demands as well as Zone 4 through Zone 6 demands. Furthermore, Zone 3 is an open system and includes 3 BPSs with a total capacity of 7,790 gpm. Based on the existing demands and pumping criteria, the required pumping capacity is 4,060 gpm resulting in a pumping capacity surplus of 3,720 gpm. All the zones which include tanks have a surplus in pumping capacity, providing the City flexibility to move water from the west to the east. Zone 1 had the largest surplus of pumping capacity at 24,120 gpm, as all the future Zone 1 pumps at the High Service BPS were installed with the construction of the WTP. Zone 6 serving Foothills Estates is a closed zone and should be designed for maximum day demand plus fire flow. This results in a required capacity of 1,290 gpm, and with pumping capacity of only 440 gpm, the system is deficient by 600 gpm. However, this will be corrected when the developer constructs the two planned 336,907-gallon (0.34 MG) tanks.

Table 7-1. Lake Havasu City Existing Storage Analysis

Pressure Zone	Existing Zone ADD (gpm)	Existing Zone ADD (MGD)	Max Day PF	MDD (ADD x PF) (gpm)	MDD (ADD x PF) (MGD)	Number of Tanks	Capacity by Tank (MG)	Operational (0.20 x MDD)	Fire (MG)	Emergency (1.0 x ADD)	Total Required Storage	Available	Surplus / Deficit (MG)
1	2,260	3.25	1.5	3,390	4.88	6 1	1 0.25	1.0	1.20[1]	3.25	5.4	6.3	0.8
2	2,280	3.28	1.5	3,430	4.92	5 1	1 0.25	1.0	1.20[1]	3.28	5.5	5.3	-0.2
3	1,420	2.04	1.5	2,130	3.07	3 1 2	1 0.5 0.25	0.6	1.20[1]	2.04	3.9	4.0	0.1
4	640	0.92	1.5	960	1.38	3 1	1 0.25	0.3	0.54[2]	0.92	1.7	3.3	1.5
5	330	0.48	1.5	490	0.71	1 1	0.5 0.75	0.1	0.18[3]	0.48	0.8	1.3	0.5
6	280	0.40	1.5	420	0.60	-	-	0.1	0.18[3]	0.40	0.7	0.0[4,5]	-0.7

[1]Assumed 5,000 gpm Fire x 4 hours = 1.20 MG

[2]Assumed 3,000 gpm Fire x 3 hours = 0.54 MG

[3]Assumed 1,500 gpm Fire x 2 hours = 0.18 MG

[4]Zone 6 is a closed pressure zone with no existing storage.

[5]Zone 6 (Foothills Estates) is an interim closed pressure zones with two Future 0.34 MG Tanks under construction, which should be operational in 2019 and would mitigate the storage deficiency.

Table 7-2. Lake Havasu City Existing Pumping Capacity Analysis

Zone Serviced	Available zone pump capacity gpm	Pump Station	Number of Pumps	System	Rated Capacity Design gpm	Rated Capacity Design MGD	Firm Capacity gpm	Firm Capacity MGD	Zone AAD	Max Day Demand[1]	Surplus/ Deficit (calculated)
1[2]	35,000	North Bank WTP High Service Pump Station	6	North	3,500	5.0	17,500	25.2	7,254	10,881	24,119
		South Bank WTP High Service Pump Station	6	South	3,500	5.0	17,500	25.2			
2	19,100	Station 1A	1	North	1,000	1.4	1,000	1.4	4,993	7,489	11,611
		Station 1B	4	North	3,300	4.8	9,900	14.3			
		Station 1	4	Central	1,400	2.0	4,200	6.0			
		Station 1C	2	South	2,750	4.0	4,000	5.8			
			1		1,250	1.8					
2	7,170	North Havasu Pump Station	1	North	170	0.2	7,170	10.3	286	5,429	1,741
			2		1,500	2.2					
			3		2,000	2.9					
3	7,785	Station 2A	2	North	1,435	2.1	2,485	3.6	2,709	4,063	3,762
			1		1,050	1.5					
		Station 2	3	Central	1,400	2.0	2,800	2.1			
		Station 2C	2	South	1,750	2.5	2,450	3.5			
			1		700	1.0					
4	5,600	Station 3A	3	North	1,300	1.9	2,600	3.7	1,247	1,870	3,730
		Station 3	4	Central	1,000	1.4	3,000	4.3			
Horizon Six[6]	300	Station 3C	4	South	100	0.1	300	0.4	44	70[7]	230
5	350	Station 4 (Hydro)[3]	2	Central	350	0.5	350	0.5	6	8	342
5	1,150	Station 4A	2	North	650	0.9	1,150	1.7	454	682	468
			1		500	0.7					
6 (Foothills Estates)	690	Station 4 (Foothills Estates)[4]	4	Central	230	0.3	690	1.0	192	1,287	(597)
6	440	Station 5A[5]	2	North	440	0.6	440	0.6	87	1,130	(690)

[1]Max Day Demand (MGD) = 1.5 x AAD
[2]North Havasu BPS assumed sized for MDD + 5,000 gpm Fire Flow
[3]Pump station 4 serves a small closed system of approximately 30 homes
[4]Pump station 4 (Foothill Estates) is an interim closed zone, future open system when Zone 6 tanks are built
[5]Pump station 5A is assumed sized for MDD + 1000 gpm Fire Flow
[6]Served by Mohave County. Pump station was replaced with four 100 gpm pumps. No fire pumps included.
[7]Peaking factor based on 1.6 x average

7.1.4 Existing Distribution System Capacity Analysis

The City water distribution system has no major water system deficiencies when all water facilities are in service, when evaluated against capacity and system design criteria. The City operates a robust and well looped system in which, in some cases, facilities were sized for future demands that have not been realized. In many cases, the areas where a 1,500 gpm fire flow cannot be supplied were likely designed to a lower fire flow standard at the time. The City should continue its annual program to upsize these areas. These projects can be integrated with a more comprehensive look at pipeline replacement prioritization as discussed in Section 7.3.3.

BPS and reservoir capacity deficiencies are very minor and several will be mitigated with the construction of several new development projects, who are required to build major water facilities. In addition, new reservoirs may be preferred over rehabilitation of an older tank, based on recent cost estimates to rehabilitate existing tanks and findings of several tanks conditions as rehabilitation work was commenced. As an example, Tank 2A (N-2A-06) is reported to be in a condition requiring more rehabilitation than originally anticipated. Since Zone 2 would benefit from increased storage in the future, this reservoir is a candidate for full replacement and accordingly is recommended as a new 1.5 MG Tank replacement project. Moreover, future BPS rehabilitation projects may want to consider replacing pumps with updated capacities based on the water demands estimated in this Master Plan.

The hydraulic model was run at average day, peak hour, select fire flow conditions, and tank filling scenarios under existing steady-state hydraulic conditions. Tanks were assumed at approximately half full and BPSs assumed one or two pumps on depending on the zone demand condition. An example of the robustness of the City's water distribution system, **Figure 7-3 and Figure 7-4** illustrate pipeline velocities from the model and residual pressures at model nodes under peak hour demands.

A summary of the existing system analysis is highlighted as follows:

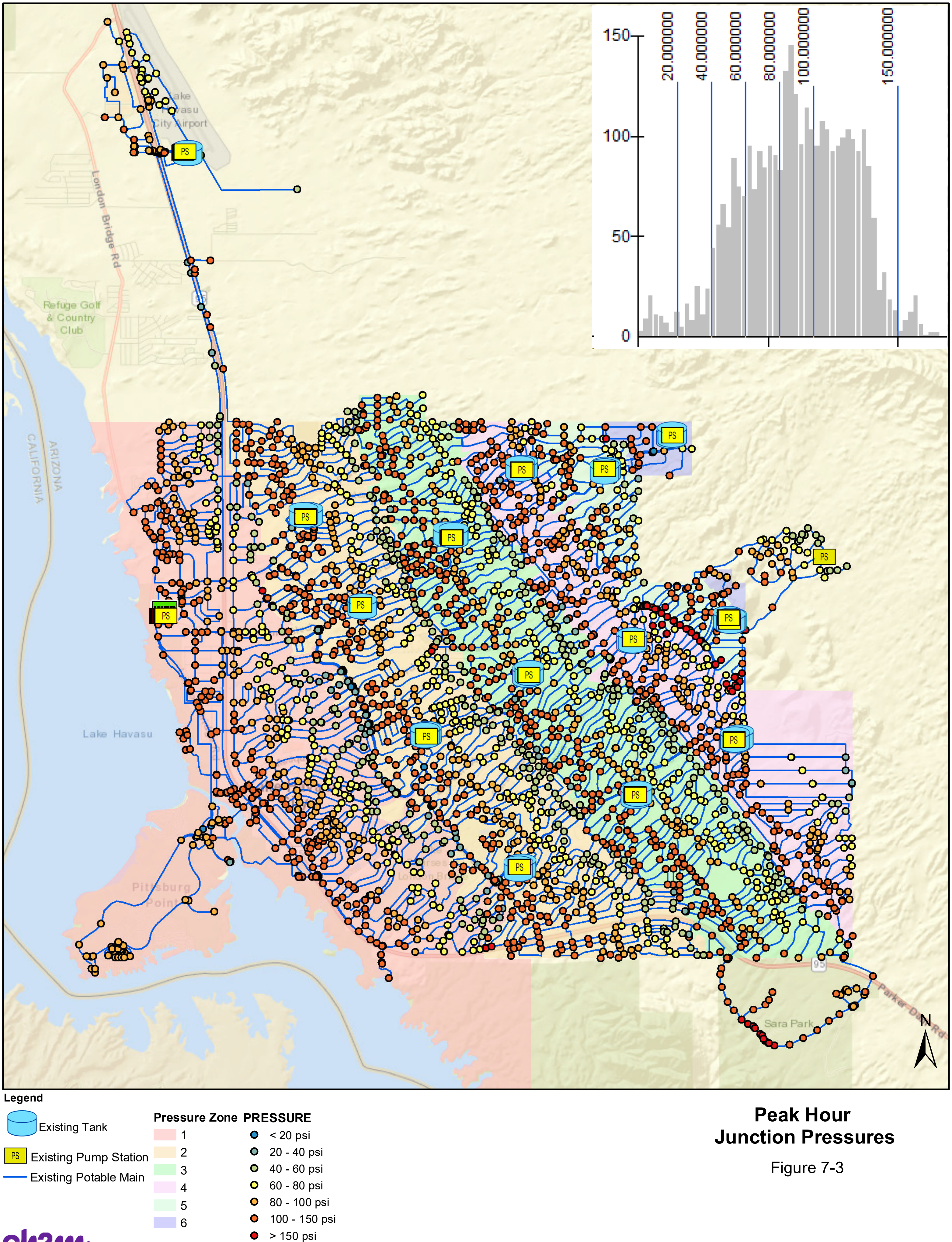
- Under average day demands of approximately 9.5 mgd the hydraulic model results confirm minimal pipeline pressure losses, especially in transmission mains and pressure swings in the overall system with all facilities in service, another indicator of the robustness of the water system.

- Under peak hour demands, pipe velocities (**Figure 7-3**) are well within established criteria, with the majority of pipeline velocities being less than 3 fps and the transmission mains all under 5 fps, indicating no major areas in the system of high-pressure losses. Although there is not a minimum velocity typcailly defined for water systems, since many distribution lines are sized for a much higher fire flow, it is desirable for transmission mains to maintain a minimum velocity of about 2 fps to promote good water circulation.

- **Figure 7-4** illustrates pressures at junction nodes from the hydraulic model under peak hour simulations. Pressure ranges are color coded throughout the system. Referring to Figure 7-4, most of the water system meets the 40 psi minimum pressures. Typical of many water systems, the few exceptions are areas with higher elevation in the zone, and therefore lower than desirable static pressures. This typically occurs right along the pressure zone boundary where the transition from a higher pressure to lower pressure occurs or near reservoir sites.

- A strong indicator of the strength of the water system is the pressure drop relative to static during peak hour conditions. It is desirable to not exceed a 20 to 25 psi drop. For a majority of the City water system, based on the peak hour simulations, the pressure swings are generally 5 to 15 psi in the outer edges of the zones. Only the long narrow zonal areas within Zone 4 and parts of Zone 5 exhibit the higher-pressure swings under peak hour or fire flow demands, but are well within criteria with all facilities in service.

- It is recommended that the City consider several new PRSs between zones to provide added reliability, potentially access available water storage, and assist in managing water quality. Typically, these PRS's should include a main and bypass valve. The smaller bypass valve would be field adjusted to provide minimum flows to promote water circulation. The main valve would be set in a manner to only hope during a high demand downstream such as a fire flow. This would also ensure the PRS is not suppling large flows on a regular basis, which would increase system pumping costs.



Peak Hour Junction Pressures

Figure 7-3

ch2m

April 2019

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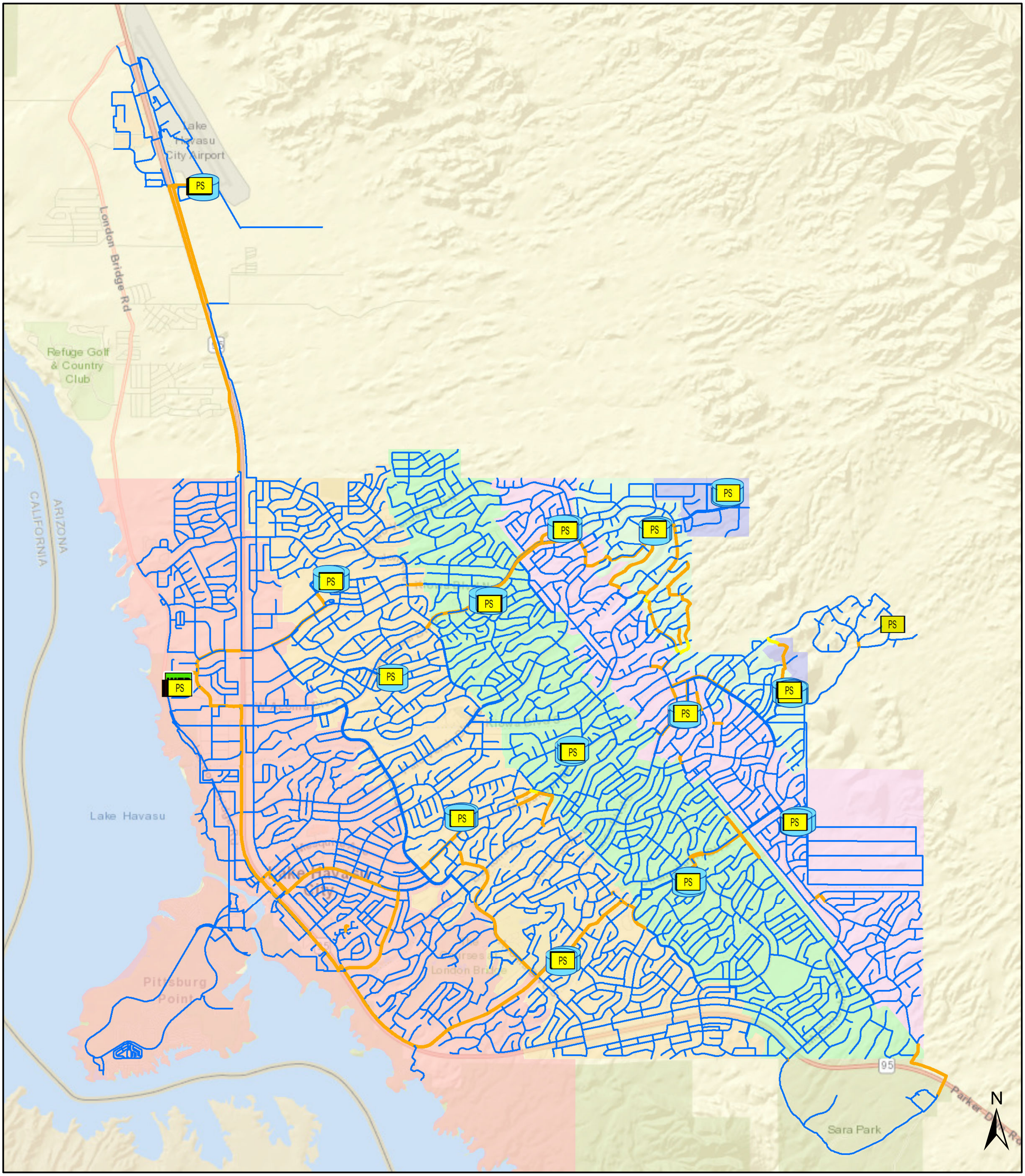

Legend

Existing Tank

Existing Pump Station

Existing Potable Main

Pressure Zone

1

2

3

4

5

6

Pipe

VELOCITY

< 2 fps

2 - 5 fps

> 5 fps

Peak Hour Pipe Velocity

Figure 7-4

ch2m

\\sdofsp01\Projects\LakeHavasuCity\692148WMP\Figures\PeakHourVelocity.mxd

April 2019

- The City does not meet maximum day plus 1,500 gpm fire flow in a number of areas, as these systems were design for lower fire flows at the time of construction, in many cases 500 gpm to 1,000 gpm. The City has recognized this deficiency and is taking steps to replace the smaller diameter namely 4-inch and 6-inch dead-end lines with 8-inch waterlines and adding new fire hydrants, replacing the old hydrant standpipes which had limited capacity.

Section 8 includes the recommended existing water system projects, which also include several small BPS upgrades, tank upgrades, and reliability projects. As discussed in Section 7.3, the City's major focus will continue to be on rehabilitation and/or replacement of BPS and reservoir facilities, a shift to increasing asset management on the distribution system pipelines, and establishment of a prioritized program over the next years to replace aging pipelines. The City has taken major steps with its annual rehabilitation program to replacing a large portion of the small diameter pipelines that have had a high break history.

7.2 2040 System Analysis

The City will continue to experience residential and commercial infill development with the number of vacant parcels available throughout the City. The City's robust water distribution system is able to serve many of these developments, and, in most cases, the availability of fire flow will be the primary determining factor on capacity. For larger planned developments, the City should continue to require Developer's to prepare and submit master water system plans for their development to ensure conformance to the Master Plan and consistency with the City' desired water service pressures and system configurations.

The City's major area of development over the next five to ten years, will largely be the development projects presented in **Table 3-7**. In several cases, the City will see new pressure zones constructed as a result of the projects being development. Developer's required to construct major facilities such as BPSs and reservoirs should work closely with the City to ensure adequate sizing and any oversizing. Developers should also provide sufficient detail in preparation of construction drawings to allow the City to accept the facilities once constructed. In some cases, the City may choose to take a lead role in design and construction, with the Developer providing cost participation depending on the schedule and facility benefit to the City.

The timing of development in the north area of the City is unknown and could occur beyond the planning horizon of this master plan update. However, the City should be prepared for additional development in its planning efforts and so this Master Plan schematically presents a water system that extends water service and supply from the North Havasu Tank and is hydraulically integrated with the City's existing water system pressure zones. The remaining sections summarizes the recommended proposed water system to meet the City's projected population of 67,000 people by 2040.

7.2.1 Future Pressure Zones

In the future, expansion of the City water system and establishment of new pressure zones will likely be driven by new growth. Specifically, the City will see a new water system to serve the Foothills master planned development and projects such as Campbell and Bluewater, all requiring new improvements to pumping and storage. Referring to **Figure 7-1**, the hydraulic profile, the Foothills Estates proposed water system will complete a new Zone 6 system served solely by BPS 4. A new Zone 7 system will be created to serve the higher grounds in Foothills Estates, not serviceable by Zone 6. The Bluewater and

Campbell projects will build-out the remaining developable area of Zone 4, several miles from the Zone 4 tanks, and therefore will require a new Tank 4C and a Zone 5 system.   Due to the size of the projects and topography constraints, the new Zone 5 can be constructed as a small closed pumped system.

To the north, a new series of BPSs and tanks will be necessary to meet planned growth. Since the City has constructed a pressure reduced system off the North Havasu pumped system, one water supply option in the future is to modify the pump head slightly so the station can deliver directly to a future Zone 3 reservoir. The expansion further east would then align with the existing pressure zones of the City and allow potential interconnections as the area develops.

The following section highlights the major water facilities needed to serve the City by 2040 based on an increase in average water demand of nearly 3.0 mgd and maximum day demand increase of 4.8 mgd (3.300 gpm).

7.2.2 Future Storage

Future water demands were estimated for each pressure zone based on the population and demand forecasts presented in Section 3. **Table 7-3** includes a pressure zone by pressure zone analysis of future required water storage capacity. As the City develops north and east, future storage will be needed to stabilize pressures, provide adequate fire storage, and correct any system deficiencies. In the near term, Zone 6 (Foothills Estates) will need two new storage tanks to efficiently provide water service and fire protection as the development is under construction. Furthermore, the Bluewater and Campbell developments will benefit from the addition of a new Tank 4C to stabilize pressures and provide adequate fire flow. In addition, the City will benefit by adding storage and increased pressure and reliability to the south part of Zone 4.

Referring to **Table 7-1** and **Table 7-3**, the core area of the City generally has sufficient water storage. Zone 1 is only slightly deficient but also has access to Clearwell storage at the WTP; therefore, no improvements are recommended. Zone 2 has the largest storage deficit; to correct this deficiency, one option would be to replace the existing 0.25 MG at Site 1 in the future with a 1.25 MG to 1.5 MG reservoir. Alternatively, the City may want to rely on a new PRS between Zone 3 and Zone 2 for added supply redundancy. In the near term, the City does have access to storage in the North Havasu Tank in the far north that can be accessed in an emergency and water can be pumped back to Zone 2 providing added reliability, potentially deferring a Zone 2 Tank improvement. For master planning budgeting purposes, a new Zone 2 Tank (Site 1) was included in the proposed five-year CIP but should be further evaluated based on the aforementioned discussion.

Future development will mostly occur in the northern portion of the City. A series of new tanks have been shown, but locations and sizes will depend on specific development plans. The north area's distance from existing water system will warrant a series of BPSs and tanks to serve the addition of multiple pressure zones. The City's has expressed a desire to match existing pressure zones as the north area develops.

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Table 7-3. Lake Havasu City Future Storage Analysis

Pressure Zone	Existing Zone ADD (gpm)	Existing Zone ADD (MGD)	Max Day PF	MDD (ADD x PF) (gpm)	MDD (ADD x PF) (MGD)	Number of Tanks	Capacity by Tank (MG)	Operational (0.20 x MDD)	Fire (MG)	Emergency (1.0 x ADD)	Total Required Storage	Available	Surplus / Deficit (MG)	Future Tanks
1	2,780	4.00	1.5	4,170	6.00	6 1	1 0.25	1.2	1.20[1]	4.00	6.4	6.3	-0.2	No
2	2,810	4.05	1.5	4,210	6.07	5 1	1 0.25	1.2	1.20[1]	4.05	6.5	5.3	-1.2	Yes
3	1,740	2.51	1.5	2,610	3.76	3 1 2	1 0.5 0.25	0.8	1.20[1]	2.51	4.5	4.0	-0.5	Yes (North)
4	790	1.14	1.5	1,180	1.71	3 1	1 0.25	0.3	0.54[2]	1.14	2.0	3.3	1.2	No
5	400	0.58	1.5	600	0.86	1 1	0.5 0.75	0.2	0.18[3]	0.58	0.9	1.3	0.3	Yes
6	340	0.49	1.5	510	0.73	4	0.25	0.1	0.18[3]	0.49	0.8	1.0	0.2	Yes

[1] Assumed 5,000 gpm Fire x 4 hours = 1.20 MG

[2] Assumed 3,000 gpm Fire x 3 hours = 0.54 MG

[3] Assumed 1,500 gpm Fire x 2 hours = 0.18 MG

7.2.3 Future Pumping

The future water demands were incorporated into a pump capacity analysis for future conditions. **Table 7-4** presents the pumping analysis for 2040 and concludes the major zones (Zones 1 to 4) do not have any BPS capacity issues and continue to have surplus capacity, which provides the City increased flexibility and reliability in meeting demands. The Foothills Estates proposed water system will include two 336,907-gallon tanks, and this Zone 6 will convert from a closed zone to an open system. The result is fire flow will be shifted from the BPS to the tanks, freeing up capacity at the existing BPS. No capacity upgrades are recommended, although the pump capacities should continually be monitored.   A similar situation occurs with the construction of a new Tank 6A thereby shifting the fire flow from BPS 5A to Tank 6A and not requiring any pump capacity upgrades. However, as noted in Section 7.3.2, BPS 5A is in need of immediate upgrade due to it condition and need to provide a minimum 1,000 gpm fire flow demand.

The City does not own and operate a BPS at Tank 3C. As part of the recommendation to construct a new Tank 4C, it is also recommended that a new BPS 4C be constructed concurrently with Tank 4C to allow for improved tank filling and reliability for the City. As the City expands to the north, a series of BPSs and tanks are recommended to support expanded Zones 3 to 6.

In addition to an evaluation of the normal pump operations, an analysis was made as it relates to the pressure zone supply reliability criteria. Pressure zones with three or more BPSs were confirmed to be able to supply the average day demands with one station out of service based on available capacities at the end station. This analysis was performed under average day demands per the design criteria; under peak hour demands this condition may not be able to be met in some portions of the system depending on the demand. In reviewing both the existing and 2040 model simulations, it would appear that the City would benefit by installing several PRS's at the edge of zones to minimize pressure swings during outages. These stations would also provide added reliability and ability to manage water quality by supplying small flows from a higher zone to a lower zone.

As the City continues with its BPS rehabilitation program, summarized in Section 7.3.2, **Table 7-4** can be used to confirm or modify new BPS capacities based on the 2040 maximum day demands in the water system.

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Table 7-4. Lake Havasu City Future Pump Capacity Analysis

Zone Serviced	Available zone pump capacity gpm	Pump Station	Number of Pumps	System	Rated Capacity Design gpm	Rated Capacity Design MGD	Firm Capacity gpm	Firm Capacity MGD	Zone AAD	Max Day Demand[1]	Surplus/ Deficit (calculated)
1[2]	35,000	North Bank WTP High Service Pump Station	6	North	3,500	5.0	17,500	25.2	8,916	13,374	21,626
		South Bank WTP High Service Pump Station	6	South	3,500	5.0	17,500	25.2			
2	19,100	Station 1A	1	North	1,000	1.4	1,000	1.4	6,137	9,206	9,894
		Station 1B	4	North	3,300	4.8	9,900	14.3			
		Station 1	4	Central	1,400	2.0	4,200	6.0			
		Station 1C	2	South	2,750	4.0	4,000	5.8			
			1		1,250	1.8					
2	7,170	North Havasu Pump Station	1	North	170	0.2	7,170	10.3	286	5,429	1,741
			2		1,500	2.2					
			3		2,000	2.9					
3	7,785	Station 2A	2	North	1,435	2.1	2,485	3.6	3,329	4,994	2,741
			1		1,050	1.5					
		Station 2	3	Central	1,400	2.0	2,800	2.1			
		Station 2C	2	South	1,750	2.5	2,450	3.5			
			1		700	1.0					
4	5,600	Station 3A	3	North	1,300	1.9	2,600	3.7	1,542	2,314	3,286
		Station 3	4	Central	1,000	1.4	3,000	4.3			
Horizon Six	300	Station 3C	4	South	100	0.1	300	0.4	44	70[6]	230[7]
5	350	Station 4 (Hydro)[4]	2	Central	350	0.5	350	0.5	6	8	342
5	1,150	Station 4A	2	North	650	0.9	1,150	1.7	201	301	849
			1		500	0.7					
Foothills Estates (Zone 6)	690	Station 4 (Foothills Estates)[4]	4	Central	230	0.3	690	1.0	192	287	403
6	440	Station 5A[5]	2	North	440	0.6	440	0.6	151	1,226	(786)

[1]Max Day Demand (MGD) = 1.5 x AAD
[2]North Havasu BPS assumed sized for MDD + 5,000 gpm Fire Flow
[3]Pump station 4 serves a small closed system of approximately 30 homes
[4]Pump station 4 (Foothill Estates) is an interim closed zone, future open system when Zone 6 tanks are built
[5]Pump station 5A is assumed sized for MDD + 1,000 gpm Fire Flow
[6]Served by Mohave County. Pump station was replaced with four 100 gpm pumps. No fire pumps included.
[7]Peaking factor based on 1.6 x average

7.2.4 2040 Distribution System Analysis

Based on the 2040 model simulations, the following pipeline upgrades to the existing water system are recommended in **Table 7-5** to provide system reliability and meet future capacity needs. It is further assumed the City will continue to annually replace 4-inch and 6-inch mains as part of the fire flow upgrade program. The pipeline projects are in addition to the tank and BPS upgrades discussed in the preceding section. Section 8 presents the proposed CIP and estimate of probable costs.

Table 7-5. Pipeline CIP Projects

No.	CIP Need/Type	Description	Justification
1.	Capacity	State Highway 95 Crossing to serve Sara Park (3,000 feet of 8-inch Replacement)	City has temporary 4-inch crossings highway located inside storm drain. Improve service pressures
2.	Capacity	Acoma Blvd North 12-inch from Green to Jamaica (3,300 ft)	Improve fire flows and reduce peak hour pressure swings
3.	Capacity/Fire Flow	Island waterline: McCulloch Blvd North to Kickapoo 12-inch (10,500 ft)	Support Island development and provide fire flow capacity and reliability
4.	Reliability	WAPA Pipeline Interconnection Chemehuevi and Mohican 16-inch (12,000 ft)	Minimizes pressure swings, improves fire flow, and enhances north south reliability
5.	Capacity	New Transmission Main from BPS 2A to Tank 3A. 16, 24-inch (4,500 ft)	City has increased pumping capacity, this pipeline provides ability to move more water through Tanks, 2A, 3A, 4A, and 5A in the future. Increases reliability overall.
6.	Capacity	New Transmission Main in Palo Verde Blvd. South from BPS 3A, 20-inch (4,300 ft)	Increase Tank 4A and 5A tank fill capacity
7.	Capacity	New Transmission Main in Fiesta Dr and Wash to Tank 4A 12-inch (2,100 ft)	Increase Tank 4A and 5A tank fill capacity
8.	Capacity	New Transmission Main from BPS 4A to Tank 5A 20-inch (4,400 ft)	Increase Tank 5A tank fill capacity
9.	Capacity	New Transmission Main from BPS 5A to proposed tank 6A 12 and 16-inch (1,400 ft)	If Tank 6A site acquired, and tank project moves forward with increased growth.

7.3 Condition Assessment Overview

The City's water distribution system performs well in meeting design criteria, in part due to the parallel nature of its pumping and storage systems to cover the long narrow service band of each pressure zone. One known area of deficiencies are the City's 4-inch water mains limiting fire flows, especially when coupled with the stand pipe fire connections. As part of its annual replacement program, the City should continue to upgrade its system by replacing older 4-inch mains with 8-inch mains. In addition, the City has been installing new fire hydrants to replace stand pipes or to simply improve fire flow coverage.

The City has also implemented Lucity software as its computerized maintenance management system (CMMS), which includes a pipeline data base from the City's GIS. The pipeline data-base includes pipe diameter, pipe material and estimated date of installation. The City is in the process of beginning to track and record pipeline break history in its CMMS, which when complete should greatly assist the City in developing a more refined prioritization replacement plan, one that is not necessarily solely based on age of pipe, but rather considers multiple criteria such as break history and material related to likelihood of failure, and other criteria related to consequence of failure to mitigate risk. One recommended area of update is the pipeline database defaulted all pipelines constructed before 1970 to a 1970 install date, so there is no pipe age shown from the 1960's. Section 7.3.3 summarizes the City's current pipeline database and presents a typical approach to further develop a prioritized replacement plan.

The City has taken a proactive approach to condition assessment for its tanks and BPSs by completing the Water Facility Inventory and Prioritization Report (Atkins 2016), which has been included as part of Appendix D. The City has aggressively moved forward with rehabilitation projects this past year, in response to several steel tank deteriorated conditions and has established a five-year program. The following two sections summarize the condition assessment ranking from the Atkins report, on tanks and BPSs, respectively. Section 8 includes the updated five-year CIP for rehabilitation integrated with Master Plan capacity improvements.

7.3.1 Tanks

The City's water storage tanks were generally constructed between 1965 and 2014. The majority of the tanks are of welded steel construction; two were recently constructed as bolted steel tanks. All welded steel tanks are coated on the exterior and interior to prevent metal deterioration due to corrosion. While generally these tanks are expected to have a useful life of 75 years or more, the interior coatings have a shorter expected useful life of 15 to 20 years. **Table 7-5** summarizes the findings and recommendations from the 2016 Water Facility Inventory and Prioritization Report by tank condition (from poorest to best). The City continues to evaluate the prioritization of the tank conditions on an annual basis.

The City has further prioritized the tank rehabilitation projects over the next five years and has started construction with the program because of several failing tanks. The City recently completed the rehabilitation of Tank N-1B-03 and is under construction with Tank N-1A-05 and N-2A-07. In some cases, the City may need to increase the budget during construction based on additional findings and testing during the rehabilitation process. For example, Tank S-1C-24 is scheduled to be rehabilitated in 2019, however the planned work has required additional non-destructive testing. The Master Plan incorporates this City's planned five-year program but also considers future capacity needs at each of the sites. **Table 7-6** shows the condition ranking of the water storage tanks. The scale ranges from 5 to 1.

A rating of 5 represents tanks in poor condition and a 1 represents tanks in excellent condition (Atkins, 2016).

Table 7-6. Water Storage Tanks Ordered by Condition Ratings

Tank ID	Capacity (MG)	Condition Rating	Tank ID	Capacity (MG)	Condition Rating
N-2A-06[1]	1.00	5	C-2-18	0.25	2.7
N-5A-13[2]	0.50	5	S-1C-24	1.00	2.7
C-2-17[3]	1.00	5	S-2C-27	1.00	2.7
C-4-21	1.00	5	N-1B-03	1.00	2.7
S-1C-25[3]	1.00	5	N-4A-11	0.25	2.6
N-1A-05	1.00	3.4	S-3C-28	1.00	2.5
C-1-15	0.25	3.2	C-3-20	0.50	2.5
N-2A-07	1.00	3.1	N-1B-04	1.00	2.2
N-3A-08	0.25	2.8	C-1-16	1.00	2
N-3A-09	1.00	2.8	S-3C-29[4]	1.00	2
N-4A-10	1.00	2.8	N-HAV-01[5]	2.00	1
S-2C-26	1.00	2.8	N-5A-12[6]	0.75	1
C-3-19	0.25	2.8	C-4-22[6]	1.00	1

[1] Tank not inspected as it is scheduled to be rehabilitated or replaced in 2016.
[2] Tank was inspected by TIC and found to be in very poor condition.
[3] Tank was rehabilitated in 2016.
[4] Tank was inspected by TIC and found to be in good condition with recoating recommended in 6+ years.
[5] Tank was inspected by divers in 2014; the coating was in very good condition with very good adhesion.
[6] Tank is new steel bolted tank constructed in 2014. The walls are glass-infused rather than coated.

7.3.2 Pump Stations

A total of six BPSs were identified in the Water Facility Inventory and Prioritization Report for upgrades or replacements. These projects are summarized below including their estimated condition rating and included in the five-year CIP in Section 8.

Pump Station 1A: Condition Rating 3.3

As noted in the Prioritization Report the City has two options, including removing the BPS. The 2040 storage analysis does confirm the benefit of Tank 1A (1.0 MG) in the system to meet design criteria. However, the City does have flexibility to utilize the North Havasu Tank. Assuming Tank 1A remains, the City should plan to budget a small package BPS at Site 1A to help improve water quality and turnover in this area when operated.

Pump Station 4 & Hydro: Condition Rating 3.1

The City should consider an upgrade of the current BPS to enable the City to provide reliable, long-term service to consumers in this portion of Zone 5, as it is their only sole source of municipal potable water. Replacement of the pumps, piping, foundation, shade structure, and possibly the hydro-pneumatic tank, is recommended. Alternatively, as the Foothills area builds out there may be an option to convert this service area to a small pressure reduced zone off the new Zone 6 Foothills system by constructing a new PRS.

Pump Station 5A: Condition Rating 3.1

This Master Plan recommends the City consider upgrading the existing pumps and hydro-tank due to the deteriorating condition of the existing equipment and the concern the City has expressed with a new Tank 6A and operating as an open system due to low water demands in the zone. This project should be designed to meet the current demands including a 1,000 gpm fire flow while planning for future expansion.

Pump Station 2C: Condition Rating 3.0

A capital project to upgrade and replace the pumps is recommended. Replacement of the BPS would include replacement of the existing pumps with three vertical turbine pumps to meet the buildout firm capacity with backup pumping capability. All new electrical equipment should be specified along with a generator for backup power.

Pump Station 1C: Condition Rating 2.8

A capital project to upgrade and replace the pumps is also recommended for BPS 1C. At the time of this Master Plan, the project is being advertised for design. Replacement of the BPS would include replacement of the existing pumps with three vertical turbine pumps to meet the buildout firm capacity with backup pumping capability. All new electrical equipment and a generator installed for backup power are part of the project.

Pump Station 2A: Condition Rating 2.4

As noted in the Prioritization Report, an upgrade of the existing pumps at Station 2A, is recommended based on the documented condition of the BPS.

7.3.3 Pipelines

The City maintains nearly 475 miles of water distribution pipeline, with construction dating back to the 1960s with the majority being AC Pipe. This section summarizes the pipeline material type, age based on database installation year, and a suggested approach to complete a risk analysis as more pipeline break data is documented in the City database. The GIS data combined with pipe break data can be used to evaluate the current pipe infrastructure to determine which sections would be candidate locations for replacement. As earlier noted, the oldest installation date listed in the GIS attribute data is defaults to 1970, therefore not capturing some of the 1960’s pipelines. Hence, sections of pipes can be much older than recorded in the database. As a first step, the data available was categorized by material type and age. Typical steps to complete a full risk analysis and prioritization program are provided in the following sections.

Figure 7-5 shows the distribution of the installed pipeline material type. The most common pipe material is AC Pipe followed by PVC and ductile iron. The percentage of pipe which is ACP, PVC, and ductile iron is 70%, 16% and 9% respectively.

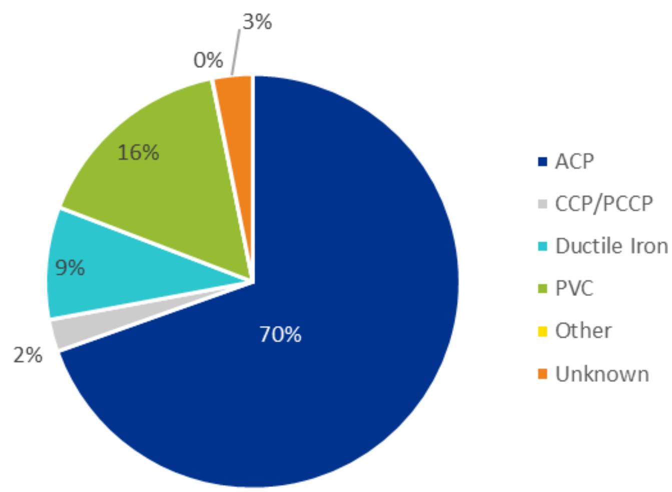


Figure 7-5. Pipeline Material Type

Figure 7-6 shows the percentage of pipe which was installed by decade between 1970 and 2020. According to the data provided by the City, approximately 93% of the current pipeline network was installed between 1970-1979. A significant portion of pipeline was likely constructed before 1970, but defaults in the database to 1970.

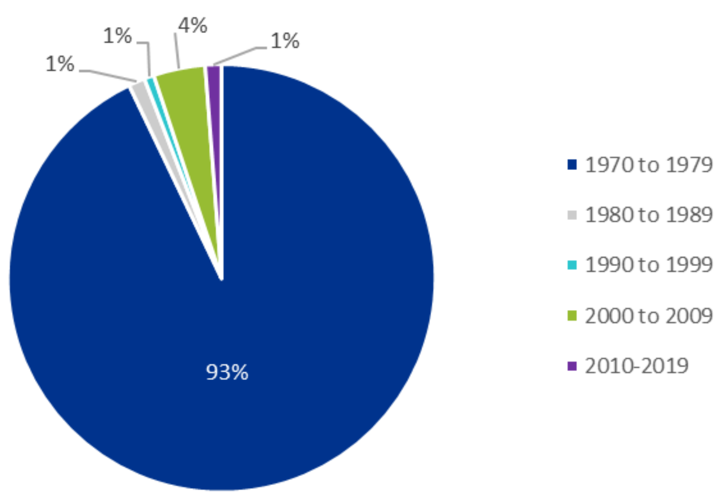


Figure 7-6. Pipeline Installation Year

7.4 Pipeline Risk Analysis Methodology and Approach

A risk analysis can be performed to understand the relative risk each water main has in the City's water distribution system. The primary goal of a risk analysis is to create a uniform and more defensible approach for making repair and replacement decisions. The City's current focus is 4-inch and 6-inch pipelines that are both fire flow deficient and have had reported breaks or leaks. However, a more comprehensive risk analysis process can be used to:

- Prioritize assets,
- Provide insights on where to collect/verify data,
- Guide decision making when there is either a lack of requisite asset information, or when there is sufficient information,
- Inform the condition assessment field crews on prioritized areas for fieldwork,
- Identify operations and maintenance issues/appropriate levels of maintenance and,
- Better capital projects for both short- and long-term CIPs.

Typical analyses use infrastructure asset attribute data from the City's GIS, as well as other GIS data such as roads, land use, and locations of critical customers. Data from the City's hydraulic model can also be incorporated to determine the impact of high pressures on asset failure. Risk scores can be assessed for all City-owned assets as indicated in the City's GIS. The risk scores can be then grouped into risk categories to identify assets that pose the greatest risk of failure. By understanding what is driving the risk of failure, the City can determine the appropriate risk mitigation option, which may be asset repair, replacement, condition assessment, or more frequent maintenance.

While several definitions and theories of risk exist, the Insurance Service Organization (ISO) 31000 definition typically applies to water, wastewater, and storm water infrastructure. Shown below, the calculation of risk is the product of consequence of failure (COF) and likelihood of failure (LOF):

Risk Score = Consequence of Failure × Likelihood of Failure

COF is defined as the impact to service that is a result of an asset failure. For example, the consequence of a water main failure could result in loss of service for residents, resorts, or critical industrial customers. LOF, on the other hand, is the potential for an asset to fail. For example, an old ACP water main may be more likely to fail than a new steel water transmission main.

In the risk framework, consequence and likelihood scores are assessed based on a number of factors. Weighting can be applied to each of the consequence and likelihood factors such that relative importance of each criterion is captured. Therefore, the consequence and likelihood scores used in the risk equation are the sum of the individual criterion's weights and scores.

Consequence or Likelihood = ∑ Category Weight x Category Score

The steps outlined below is a suggested approach to assess the relative risk of the City's water mains:

- Utilize available sources of data, which include:
    - GIS layers available from the City with updated install dates and material
    - Pipe break data added to the City database
    - Hydraulic model

- Develop COF and LOF scoring matrixes
    - Develop customized weighting criteria for each category in the COF and LOF matrixes
    - Matrixes to be developed based on updated data available
- Prepare the GIS data for asset scoring
- Calculate the risk scores
- Analyze risk scores and results
- Refine matrixes and/or weightings from sensitivity analysis and review with City staff

7.4.1 Risk Matrixes

The first step in performing a risk analysis is to develop the COF and LOF matrixes that will be used to score each asset. The COF and LOF matrixes are typically developed through collaboration with the City's staff. Facilitated workshops with engineering and water operations can be held to develop the COF and LOF matrixes, which consist of categories, weights, criteria, and scoring values.

7.4.2 Consequence of Failure (COF)

A COF matrix is typically developed in order to characterize the consequences of an asset failure. The consequence of an asset failure can typically be expressed in terms of four consequence categories:

- Fiscal, Health & Safety (H&S), and Public Confidence
- Proximity to Transportation Corridors
- Loss of Service to Critical Facilities
- Loss of Service to General Population

A numerical score, ranging from 1 to 10, can be assigned to assets for each COF category. A score of 1 is defined as meeting the target service level even if the asset was to fail, whereas a score of 10 indicates that the asset will not maintain the target service level upon failure. Each COF category can then be assigned a weight according to its importance in meeting the City's service goals. The equation for calculating individual asset COF scores is shown below.

$$COF = (W_F * S_F) + (W_{TR} * S_{TR}) + (W_{CF} * S_{CF}) + (W_{GP} * S_{GP})$$

Where

COF = Consequence of failure
W_F *= the weighting (as a percentage) for fiscal, health & safety, and public confidence*
S_F *= the asset's score for fiscal, health & safety, and public confidence*
W_{TR} *= the weighting (as a percentage) for proximity to transportation*
S_{TR} *= the asset's score for proximity to transportation*
W_{CF} *= the weighting (as a percentage) for loss of service to critical facilities*
S_{CF} *= the asset's score for loss of service to critical facilities*
W_{GP} *= the weighting (as a percentage) for loss of service to general population*
S_{GP} *= the asset's score for loss of service to general population*

Table 7-7 below shows a typical COF matrix as a starting point with scoring criteria by each category that can be adjusted by the City, including assigning weights to each category.

Table 7-7. Typical Consequence of Failure Matrix

COF Category	Weight (%)	Criteria and Scoring: 1	3	5	7	10
Fiscal, H&S, and Public Confidence	TBD	Pipes intersecting residential or non-residential land use including resort areas				
Proximity to Transportation	TBD	Pipes intersecting a major roadway or within State Highway 95				
Loss of Service to Critical Facilities	TBD	No critical customers impacted	1 equivalent critical customer within 250 ft	2 equivalent critical customers within 250 ft		3 or more equivalent critical customers within 250 ft
Loss of Service to General Population	TBD	Population density ≥ 0 and ≤ 500	Population density > 500 and ≤ 1,000	Population density > 1,000 and ≤ 2,000	Population density > 2,000 and ≤ 4,000	Population density > 4,000

≥ = greater than or equal to; ≤ = less than or equal to
TBD = To Be Determined

7.4.3 Likelihood of Failure (LOF)

Similar to the COF matrix, a LOF matrix must be developed to characterize the LOF of an asset. The likelihood of an asset failure is typically expressed in terms of four likelihood categories:

- Failure History (Breaks per 100 feet) (City will need to enter data into database)
- Age (Note: City will need to update install dates prior to 1970's)
- Material (City will want to note the older Schedule 40 PVC pipe or CL-100 pipe that was installed)
- Pressure extremes including transients near BPSs

A numerical score, ranging from 1 to 10, can be assigned to assets for each LOF category. A score of 1 represents a negligible chance of failure whereas a score of 10 indicates a high probability of failure. The weighting factor reflects the relative importance for each LOF category. The equation for calculating individual asset LOF scores is shown below.

$$LOF = (W_{FH} * S_{FH}) + (W_A * S_A) + (W_M * S_M) + (W_P * S_P)$$

Where

LOF = likelihood of failure
W_{FH} *= the weighting (as a percentage) for failure history*
S_{FH} *= the asset's score for failure history*
W_A *= the weighting (as a percentage) for age*
S_A *= the asset's score for age*
W_M *= the weighting (as a percentage) for material*
S_M *= the asset's score for material*
W_P *= the weighting (as a percentage) for pressure extremes*
S_P *= the asset's score for pressure extremes*

Table 7-8 below shows a typical LOF matrix as a starting point with scoring criteria by each LOF category that can be adjusted by the City, including assigning weights to each category.

Table 7-8. Likelihood of Failure Matrix

LOF Category	Weight (%)	Criteria	Score
Failure History (Breaks per 100 ft)	TBD	≥ 0 and < 20	1
		≥ 20 and < 40	2
		≥ 40 and < 100	3
		≥ 100 and < 150	6
		≥ 150 and < 200	8
		> 200	10
Age	TBD	≥ 0 and < 10	1
		≥ 10 and < 20	2
		≥ 20 and < 30	5
		≥ 30 and < 40	6
		≥ 40 and < 50	8
		≥ 50 and < 55	9
		> 55	10
Material	TBD	AC	8
		Steel	7
		CI	10
		DI/DIP	1
		PVC	6
		Unknown	5
Pressure Extremes including Transients	TBD	≥ 0 and < 100	1
		≥ 100 and < 200	5
		> 200	8
		< 0	10

< = less than
TBD = To Be Determined

7.4.4 Risk Scoring Analysis

The COF and LOF matrixes are then used to produce a risk score on a scale of 1 to 100 (where 1 is lowest and 100 is the highest) for each City-owned asset. The results can be grouped into priority categories such as: lowest, lower, moderate, higher, and highest. The risk category boundaries are normally determined based on a review of the natural breaks in the risk scores and percentage of assets in each risk category when comparing all of the risk scores. The goal is to distribute the top 20 percent of assets into the highest, higher, and moderate risk categories. The remaining 80 percent was divided among the lowest and lower risk categories. Separating assets into priority categories makes identification and implementation of risk reduction measures more manageable for the City.

COF and LOF drivers for each asset are reviewed, as it is not uncommon for some assets have a high LOF score and a low COF score or vice versa. The ability to visualize the risk analysis results once the analysis is complete can provide a method of strategically reducing risk. The highest LOF scored assets can be targeted for inspection, repair, or replacement. Likewise, the highest COF scored assets can be targeted for routine maintenance.

Recommended Capital Improvement Projects Overview

This section presents the recommended capital improvement program (CIP) for the City based on the findings of the Master Plan integrates the recommendations from the 2016 Water Facility Inventory and Prioritization Report culminating in a detailed 5-year CIP focused on the highest priority water supply and distribution projects, as well as identifying major improvements to meet the 2040 demand conditions.

8.1 Overview

The infrastructure recommendations and associated CIP for the Master Plan includes proposed water supply and water distribution system projects. The recommended improvement projects are organized by project type and include

- Water Resources: Water supply (wells) and water treatment plant upgrades
- Reservoirs (Tanks) and BPSs: Includes both new and rehabilitation projects
- Water Distribution System: Pipelines (capacity and rehabilitation) and PRSs

Proposed phasing for project implementation is noted if recommended in the 5-year CIP. All other projects are included as "future" CIP.

8.2 Cost Methodology/Assumptions

Unit construction costs were developed using AACE International guidelines for a Class 5 estimate and from recent construction projects within the City for similar projects and similar unit costs on prior work with in the City. All costs are presented in 2018 dollars. The CIP project costs include both a construction estimate and a total CIP project budget, with soft costs to reflect the full capitalization inclusive of:

- Planning and engineering design (15%)
- Environmental, legal, construction management, contract administration (15%)
- Contingency (25%)

These estimates are based upon representative available data at the time of this report; however, since project specific conditions are not for every project and since costs of materials and labor fluctuate over time new estimates should be obtained at or near the time of construction of proposed facilities or execution of proposed programs. The estimated unit construction costs, not including soft costs, for various CIP projects are listed below.

- Wells (includes vertical well, pump, motor, electrical, SCADA, and telemetry on a case by case basis)
- Pipelines ($10 per diameter per inch)
- New Reservoirs ($2 per gallon)
- Pump Stations (case by case)

- Pressure Reducing Stations ($75,000 per station)

Table 8-1 includes the unit cost, construction costs and total CIP estimated project budget, which includes the soft costs above.

Table 8-1. LHC CIP Implementation Costs (Final Draft 4/5/2019)

Description	CIP No.	Project Type	Location / Description	Quantity	Units	Base Unit Cost	Construction	Total CIP Cost[3]	FY19/20	FY 20/21	FY 21/22	FY 22/23	FY 23/24	FY 24/25	FY 25/26	FY 26/27	FY 27/28	FY 28/29	FY 29/30
WATER RESOURCE PROJECTS																			
North Well Cost Summary																			
Well Design /Wellhead/ Construction (Site A)	W1	Supply	h of Well 18) and Mon	1	EA	$1,103,500	$1,103,500	$1,710,425	$731,925	$1,103,500									
Well Design /Wellhead/ Construction (Site B)	W2	Supply	Field (Test Site 1 HCW	1	EA	$1,038,500	$1,038,500	$1,609,675			$1,609,675								
Well Design /Wellhead/ Construction (Site C)	W3	Supply	h of Well 15) and Mon	1	EA	$1,103,500	$1,103,500	$1,710,425							$855,213	$855,213			
Well Design /Wellhead/ Construction (Site D)	W4	Supply	th Well Field (at WTP s	1	EA	$1,038,500	$1,038,500	$1,609,675							$804,838	$804,838			
Collection Pipeline	W5	Supply	s A -D (3000', 12"/18"/	1	LS	$238,000	$952,000	$1,475,600		$1,475,600									
Total								**$8,115,800**											
Central Wellfield Cost Summary																			
Second HCW Testing	W6	Supply	orizontal Collector We	1	EA	-	-	$150,000											
Maintenance[1]/Testing/Pump Rehabilitation	W7	Supply	orizontal Collector We	1	EA	-	-	$750,000								$500,000	$250,000		
Well Construction	W8	Supply	al Well and Monitoring	2	EA	$174,000	$348,000	$539,400										$269,700	$269,700
Well Head	W9	Supply	Site E	1	EA	$400,000	$400,000	$620,000										$310,000	$310,000
Collection Pipeline	W10	Supply	Site E (1,900' 16")	1	EA	$652,000	$652,000	$1,010,600										$505,300	$505,300
Total								**$2,920,000**											
Water Treatment Plant																			
Bypass Piping/Metering	T1	Pipeline	WTP	$1	LS	-	$50,000	$77,500				$77,500							
Buildings/Enclosures	T2	Sitework	WTP	1	LS	-	$800,000	$1,240,000				$1,240,000							
Chlorine System	T3	Treatment	WTP	1	LS	-	$400,000	$620,000								$310,000	$310,000		
Filters[1]		Treatment	WTP	1	LS	-	$300,000	$465,000				$465,000							
Total								**$2,402,500**											
WATER STORAGE AND PUMPING PROJECTS																			
Reservoirs																			
Tank 4C (2 x 0.13 MG)[2]	R1	Reservoir	Site 4C	260,000	GAL	$2	$520,000	$806,000			$806,000								
Tank 6A (0.25 MG)	R2	Reservoir	Site 6A	250,000	GAL	$2	$500,000	$775,000							$775,000				
Tank 3D (0.5 MG)[2]	R3	Reservoir	North System	500,000	GAL	$2	$1,000,000	$1,550,000								$1,550,000			
Tank 4D (0.5 MG)[2]	R4	Reservoir	North System	500,000	GAL	$2	$1,000,000	$1,550,000									$1,550,000		
Tank 5D (0.5 MG)[2]	R5	Reservoir	North System	500,000	GAL	$2	$1,000,000	$1,550,000										$1,550,000	
Tank 2C (replace 1.0 MG with 1.5 MG New)	R6	Reservoir	Site 2C	1,500,000	GAL	$2	$3,000,000	$4,650,000						$930,000	$3,720,000				
Tank 3C (replace 1.0 MG with 1.5 MG New, replaces C-3-20 reha	R7	Reservoir	Site 3C	1,500,000	GAL	$2	$3,000,000	$4,650,000			$930,000	$3,720,000							
Tank 2A (replace 1.0 MG with 1.5 MG New, replaces N-2A-07 re	R8	Reservoir	Site 2A	1,500,000	GAL	$2	$3,000,000	$3,450,000		$3,450,000									
Total								**$15,531,000**											
Tank Rehabilitation																			
S-1C-24	R10	Reservoir	South System	1	LS	-	$1,290,323	$2,000,000	$2,000,000										
N-4A-11	R11	Reservoir	North System	1	LS	-	$645,161	$1,000,000		$1,000,000									
Tank Rehabilitation As Needed		Reservoir	System-wide	1	LS	-	-	$10,500,000					$1,500,000	$1,500,000	$1,500,000	$1,500,000	$1,500,000	$1,500,000	$1,500,000
Total								**$13,500,000**											
Pump Stations																			
Station 5A Upgrade (With Fire Flow)	PS1	Pump Station	Site 5A	1	LS	-	$500,000	$775,000			$155,000	$620,000							
Station 3C (3 x 250 gpm)[2]	PS2	Pump Station	Site 4C	1	LS	-	$500,000	$775,000			$775,000								
Station 4 (2 x 25 gpm, 2 x 1,000 gpm)[2]	PS3	Hydro	Site 4	1	LS	-	$400,000	$620,000				$620,000							
North Havasu Upgrades (TBD)	PS4	Pump Station	North System	1	LS	-	$430,000	$666,500							$666,500				
Station 3D (TBD)[2]	PS5	Pump Station	North System	1	LS	-	$750,000	$1,162,500								$1,162,500			
Station 4D (TBD)[2]	PS6	Pump Station	North System	1	LS	-	$750,000	$1,162,500									$1,162,500		
Station 5D (TBD)[2]	PS7	Pump Station	North System	1	LS	-	$750,000	$1,162,500										$1,162,500	
Rehabilitation (As Needed)	PS8	Pump Station	System-wide	1	LS	-	-	$10,500,000					$1,500,000	$1,500,000	$1,500,000	$1,500,000	$1,500,000	$1,500,000	$1,500,000
Total								**$16,824,000**											
WATER DISTRIBUTION PROJECTS																			
Pipeline Capacity / Reliability																			
State Highway 95 Crossing to Sara Park	P1	Pipeline	8 inch mains	3,000	LF	$80	$240,000	$372,000	$372,000										
Acoma Blvd North from Green to Jamaica	P2	Pipeline	12 inch mains	3,300	LF	$120	$396,000	$613,800						$613,800					
Island Water Line: McCulloch Blvd North to Kickapoo	P3	Pipeline	12 inch mains	10,500	LF	$120	$1,260,000	$1,953,000											$1,953,000
WAPA Pipeline Interconnection	P4	Pipeline	16 inch mains	12,000	LF	$120	$1,440,000	$1,440,000	$1,440,000										
New Transmission Main from PS 2A to Tank 3A	P5	Pipeline	16 inch mains	1,000	LF	$160	$160,000	$248,000								$248,000			
New Transmission Main from PS 2A to Tank 3A	P5	Pipeline	24 inch mains	3,500	LF	$240	$840,000	$1,302,000								$1,302,000			
New Transmission Main in Palo Verde Blvd. South from PS 3A	P6	Pipeline	20 inch mains	4,300	LF	$200	$860,000	$1,333,000									$1,333,000		
New Transmission Main in Palo Verde Blvd. South from PS 3A	P6	Pipeline	12 inch mains	2,100	LF	$240	$504,000	$781,200									$781,200		
New Transmission Main from PS 4A to Tank 5A	P8	Pipeline	20 inch mains	4,400	LF	$160	$704,000	$1,091,200										$1,091,200	
New Transmission Main from PS 5A to proposed Tank 6A	P9	Pipeline	12 inch mains	700	LF	$120	$84,000	$130,200											$130,200
New Transmission Main from PS 5A to proposed Tank 6A	P9	Pipeline	16 inch mains	700	LF	$160	$112,000	$173,600											$173,600
Total								**$9,438,000**											
Pipeline Replacements																			
Replacement (AC and PVC Schedule 40 Pipe)		Pipeline	4-12 inch mains	1	LS	-	-	$16,885,000	$1,535,000	$1,535,000	$1,535,000	$1,535,000	$1,535,000	$1,535,000	$1,535,000	$1,535,000	$1,535,000	$1,535,000	$1,535,000
Total								**$16,885,000**											
Pressure Reducing Stations																			
PRV Upgrades		Valves/ Telemetry	Multiple Zones	5	EA	$25,000	$125,000	$193,750		$38,750	$77,500	$77,500							
Pressure Zone Connections / Reliability		Pipeline	Multiple Zones	5	EA	$75,000	$375,000	$581,250				$193,750	$193,750	$193,750					
Total								**$775,000**											
							CIP Total	**$86,391,300**	**$6,078,925**	**$8,602,850**	**$5,888,175**	**$8,548,750**	**$4,728,750**	**$6,272,550**	**$11,356,550**	**$11,267,550**	**$9,921,700**	**$9,423,700**	**$7,876,800**

Notes:

[1]Major Maintenance. No design included in CIP costs

[2]City may participate in the developer project costs

[3]Soft costs were incorporated into the Total CIP Cost. Percent increases are noted below.

Planning and engineering design	15%
Environmental, legal, construction management, contract administration	15%
Contingency	25%
Total Percent Increase from construction cost	55%

8.3 Capital Improvement Plan (CIP)

The CIP presented in this section are recommended to improve the water delivery system and continue to meet future demands. The CIP developed for the City's water system are prioritized by capacity, reliability or rehabilitation improvements to the existing system. The recommended CIP provides the City's customers with a system that meets the design criteria and can be operated efficiently and reliably. Should water demand and water supplies forecasted during the planning horizon (2040) not be realized, there may be opportunities to defer or eliminate some projects.

The proposed Master Plan improvements are illustrated on **Figure 8-1** and are summarized by in **Table 8-1**. A five-year CIP is presented with capacity, reliability and rehabilitation projects identified on an annual basis. Projects to meet 2040 demands or are part of future a City rehabilitation program are shown as future CIP.

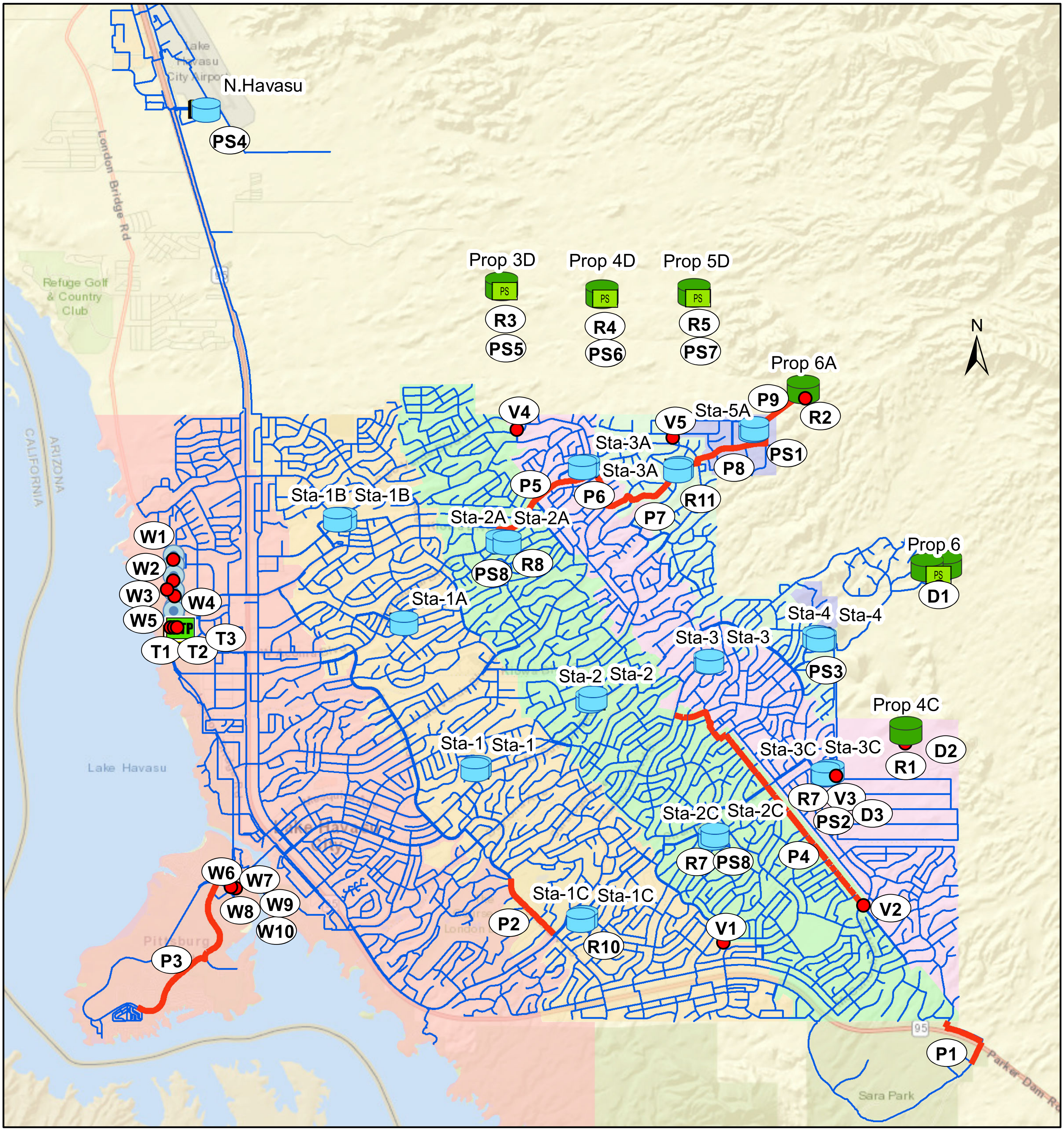
The five-year CIP gives high priority to increasing water supply reliability with the re-development of the North Wellfield to provide important redundancy for the City's water supply. Two well projects are recommended in the first five years and two more wells soon. The City has recently tested and purged the operating wells in the North Wellfield to provide some emergency back-up. The WTP improvements identified in Section 5 are planned for the end of the five-year CIP cycle to correspond with the upcoming 20-year scheduled maintenance work. The WTP continues to perform well, associated with the City's continued excellent maintenance at the Plant

The City's other high priority projects the next five years are tank and pump station rehabilitation projects. These are critical facilities in a water distribution system with multiple pressure zones. The City's next planned are included in the five-year CIP. The smaller tanks at 2C and 3C may be candidates for full replacement and upsizing vs. rehabilitation based on future storage needs in Zone 2 and Zone 3, respectively.

Preliminary pipeline replacement costs have been included to continue the City's ongoing annual program of small diameter pipelines to improve fire flows and also to replace the older Schedule 40 PVC pipe that was installed in many areas of the City. This annual program has recommended to increase over the next five years.

Lastly, a recommended approach is presented to further prioritize pipeline replacement projects in the future, not only the small diameter, but pipelines that may be higher risk to failure and result in more severe impacts to the community. Over 70 percent of the City water distribution system is made up of AC pipeline that was constructed in the 1960's and 1970's. In any cases the pipeline continues to perform well, and in other cases the City has experienced some pipeline failure. The AC pipeline is reaching 50 years of age and with nearly 300 miles of AC pipeline should become a priority to develop a future rehabilitation program that starts to replace some of the pipeline over the next 20 years.

The 5-year and future CIP project budgets can be used to evaluate water revenue requirements and determine the adequacy of existing water rates to fund the proposed capital program and whether rate increases may be warranted in the future.



Legend

Treatment Plant	Pressure Zone	CIP Project Type/No.
Production Well	1	P1 Pipeline
Existing Tank	2	R1 Reservoirs
Future Tank	3	PS1 Pump Stations
Existing Pump Station	4	T1 Water Treatment Plant
Future Pump Station	5	V1 Pressure Reducing Station
Existing Potable Main	6	W1 Water Resource Supply
CIP Project		D1 Developer

Capital Improvement Plan Projects

Figure 8-1

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