Wastewater Master Plan

2022 Wastewater Master Plan

Lake Havasu City, Arizona

Prepared for:

LAKE HAVASU CITY

Prepared by:

Jacobs

November 2022

Executive Summary

Lake Havasu City (City) operates a wastewater system consisting of three wastewater treatment plants (WWTPs), 49 sewer lift stations (public and private), 49 miles of sewer force main, and 350 miles of sewer gravity mains. The City provides wastewater services to approximately 58,000 people.

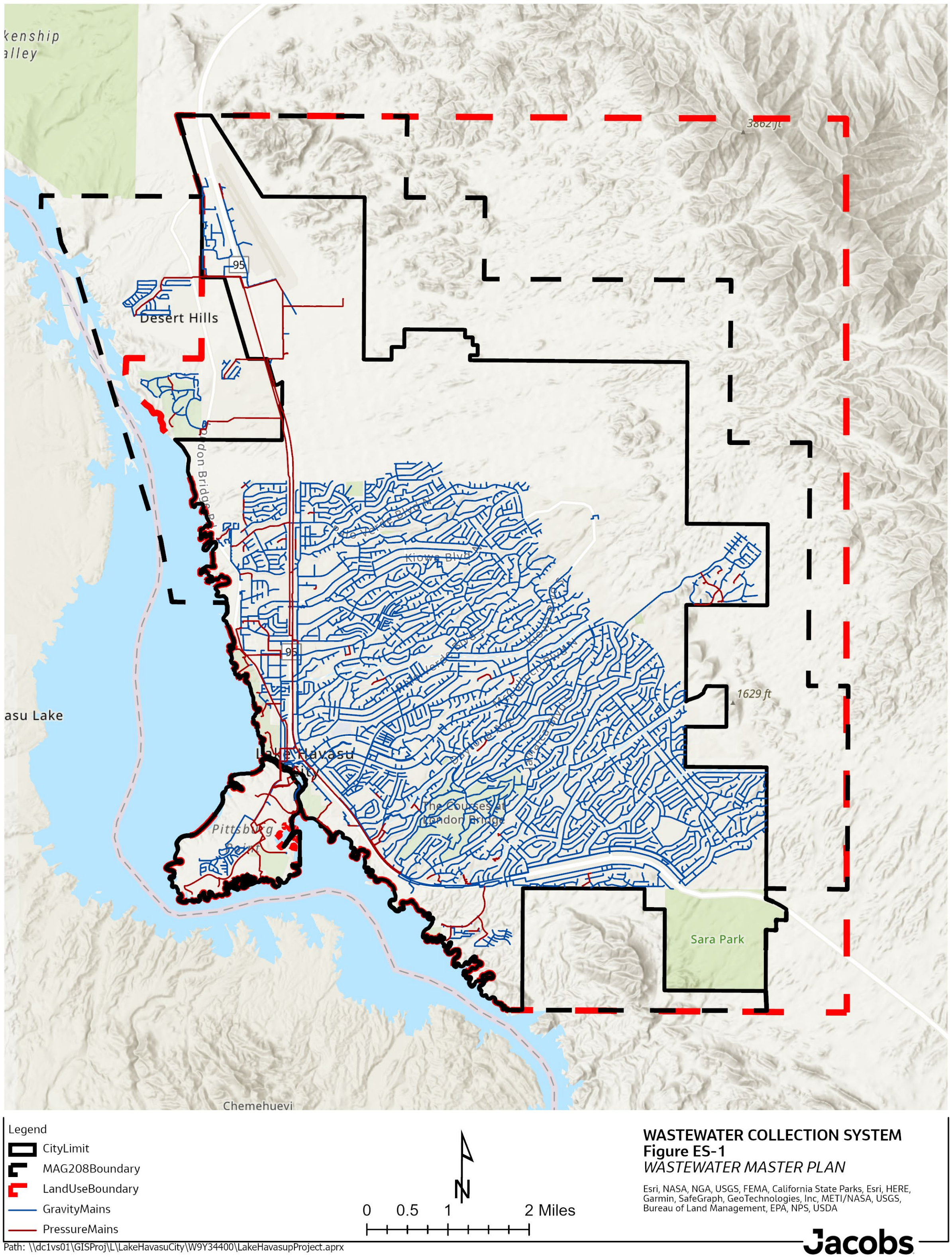
The primary purpose of this Wastewater Master Plan is to update the wastewater generation forecast, evaluate regional pumping and treatment plant infrastructure, identify effluent reuse opportunities and wastewater disposal management options, and develop near-term and identify long-term capital improvement projects and cost estimates. It is organized into the following sections:

- Overview
- Basis of Planning
- Hydraulic Model Update and Verification
- Wastewater System Evaluation
- Treatment Plant Assessment
- Reclaimed/Reuse Evaluations
- Capital Improvement Plan Implementation

Wastewater System Background

The City provides wastewater service to customers both within and outside its City limits. The City is obligated to provide sewer service within the boundary established by the Mohave County 208 Water Quality Management Plan (208 Plan). In addition, the City's 2016 General Plan includes a sphere of influence that extends beyond its 208 Plan boundary in certain potential development areas. Figure ES-1 presents the existing sewer service area, the City boundary, the 208 Plan boundary, and the City's 2016 General Plan's sphere of influence.

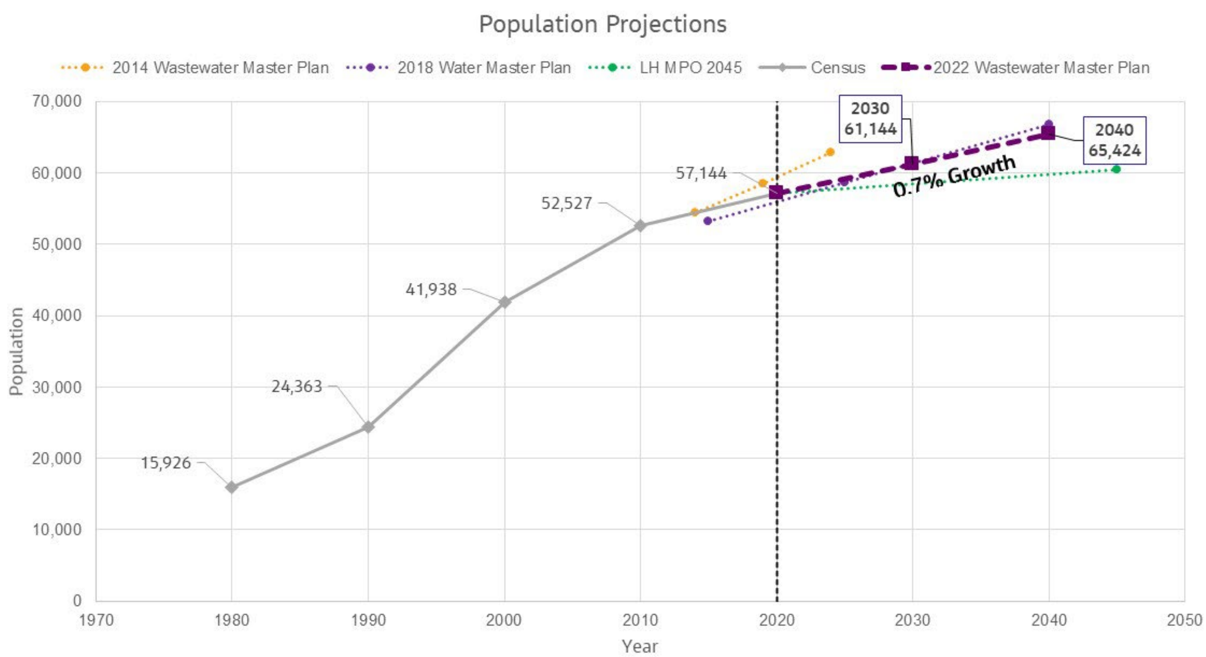
Figure ES-1. Sewer Service Area, City Boundary, 208 Plan Boundary, 2016 General Plan Sphere Of Influence



Wastewater Flow Projections

Figure ES-2 shows population-based projected growth through 2040. Jacobs reviewed population growth with the City Planning and Zoning Division and agreed to use a 0.7 percent growth rate per the Lake Havasu Metropolitan Planning Organization (MPO) Regional Transportation Plan (MPO 2022) to forecast future wastewater flows.

Figure ES-2. Lake Havasu City Population Forecast



By applying a sewer unit generation rate of 70 gallons per capita per day to the population projections consistent with the 2018 Water Master Plan and the Lake Havasu MPO document, wastewater flow projections were determined through 2040 for the basin (MPO 2022). Figure 3-5 correlates the projected population growth to an average annual daily (AAD) flow through 2040. Table ES-1 presents flow projections to the year 2040 using maximum month (MM) to AAD peaking factors and peak daily (PD) to AAD peaking factors.

Table ES-1. Estimated Wastewater Flow Projections with Peaking Factors

Year	Average Annual Flow (mgd)	Average Yearly Increase (%)	Maximum Month Flow (mgd)	Peak Daily Flow (mgd)
2021	4.0	-	4.34	4.97
2030	4.3	0.7	4.64[a]	5.59[b]
2040	4.6	0.7	4.97[a]	5.98[b]

[a] Calculated based on AAD × 1.08

[b] Calculated based on AAD × 1.30

mgd = million gallon(s) per day

Figure ES-3 shows wastewater system flow projections through 2040.

Figure ES-3. Wastewater Flow Projections

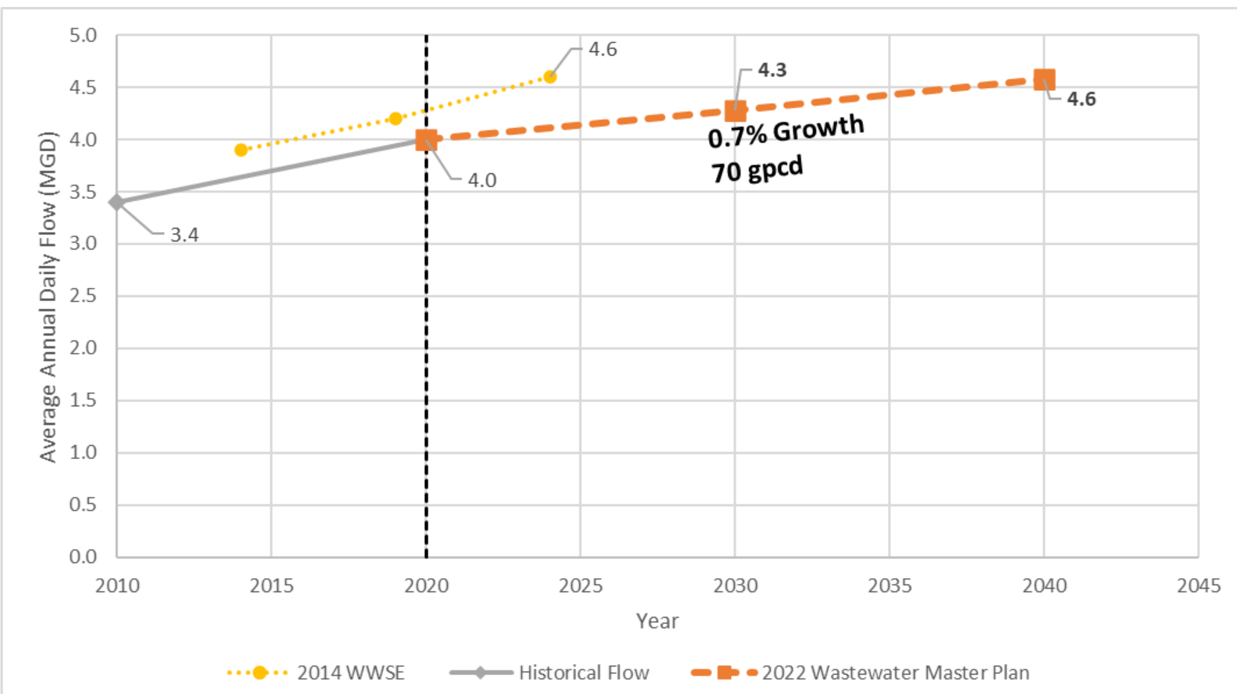
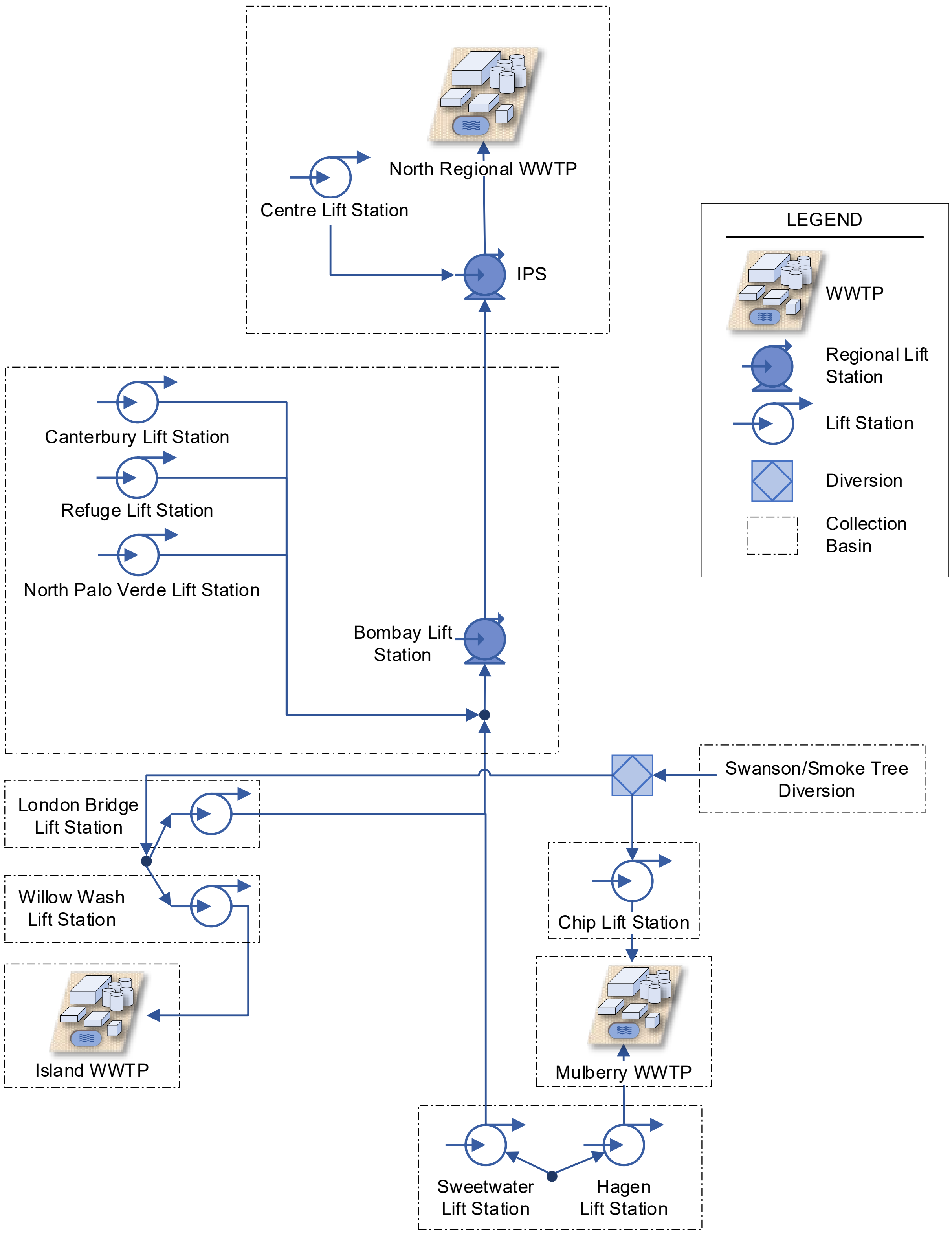


Figure ES-4. Wastewater Collection System



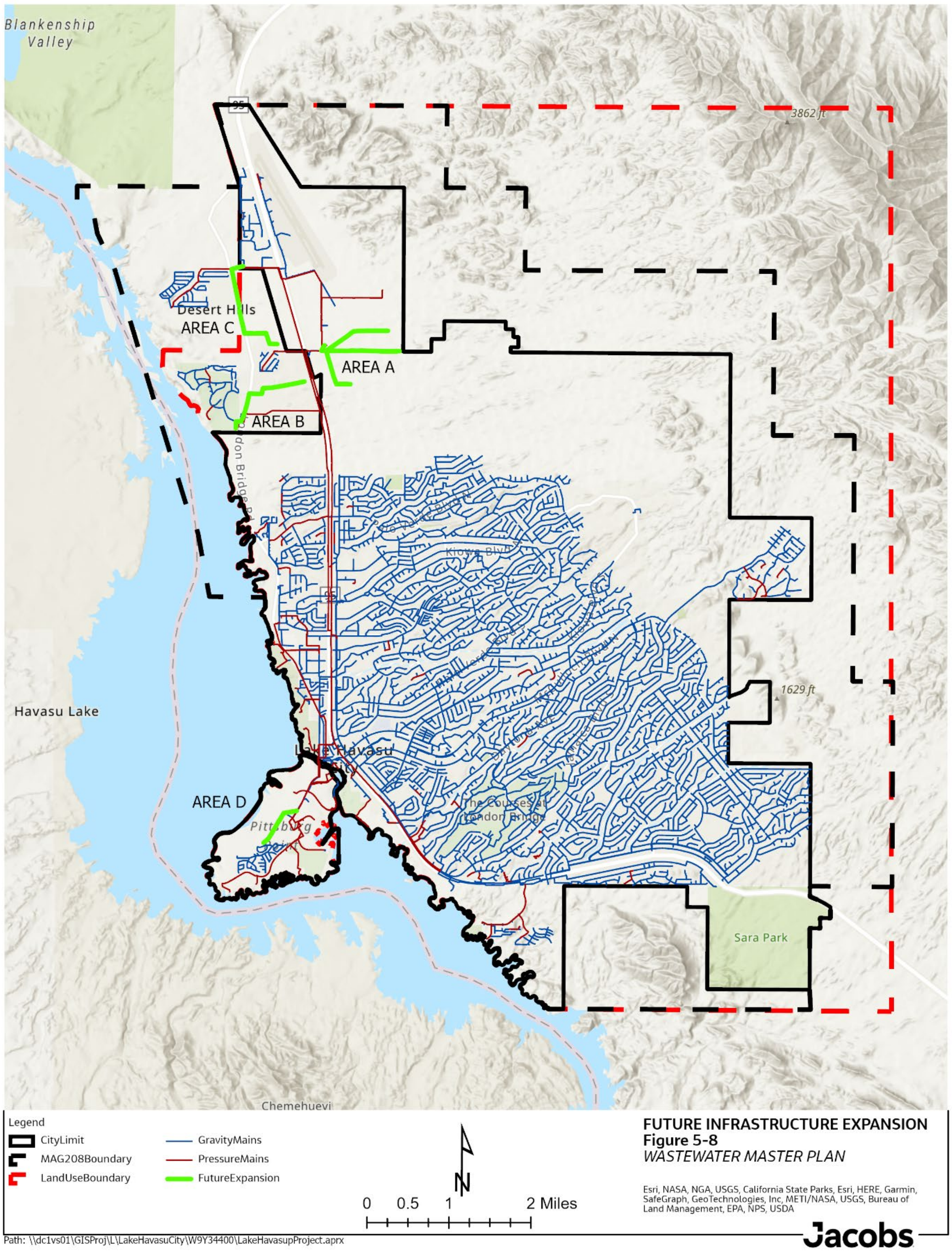
The North Regional WWTP and regional pumping system was originally planned to convey and treat approximately 14 mgd of average daily flow. Based on the current wastewater projections, in 2040, the City will have a total of 4.6 mgd of average daily flow. With such a large discrepancy in flow projections, the regional pumping system was evaluated in detail to identify opportunities to optimize the system. The regional pumping system primarily includes the following lift stations: Sweetwater, London Bridge, Bombay, and the influent pump station (IPS), which conveys flows to the North Regional WWTP, as shown on Figure ES-4.

Future gravity mains were added to the hydraulic model to route the flows from the new growth areas into the existing collection system where required. These sewers were located using ground slope information derived from U.S. Geological Survey elevation contours and are intended to provide the general feasibility of collection system routing alternatives (gravity sewers versus lift stations). The actual location of future sewers will be based on future development design plans that are subject to City approval. Future sewers were added in the following areas as shown on Figure ES-5:

- **Area A** – Gravity sewers east of Highway 95 south and east of the IPS
- **Area B** – Gravity sewers west of Highway 95 and east of the Refuge Development
- **Area C** – Gravity sewers and a lift station west of Highway 95 and southwest of the airport
- **Area D** – Gravity sewers and a new lift station and force main for the Island WWTP, southwest of the Island WWTP

Development activity is occurring in Area A. A detailed assessment of this area was conducted to accommodate the Victoria Farms Development and maximize gravity flows in the area. The findings of the assessment are presented in Appendix F, North Regional Sub-Area Master Plan Technical Memorandum.

Figure ES-5. Future Infrastructure Expansion



Treatment Plant Assessment

Figure ES-6 presents the flow projections for each WWTP. It is observed that Island WWTP service is anticipated to grow only marginally and therefore there is not much difference between the historical versus the projected flows. No growth is anticipated in the Mulberry WWTP service area and the same is reflected in the 2040 projected values for different flow conditions. Growth is anticipated in the North Regional WWTP service area and the same is reflected in the 2040 projected values when compared to the historical data.

The analysis assumed the current wastewater flow split between the three WWTPs would be applicable in 2040. The City has the ability to divert more or less flow to each WWTP. If the flow split ratio changes, then the 2040 projected flows will need to be updated accordingly. It is recommended that the City continue to monitor the different flow conditions to the three WWTPs and accordingly update the flow projections during the next master plan update.

There are no capacity or regulatory requirements for expansion or upgrades at any of the three WWTPs. All recommended upgrades are for optimization, efficiency, or regular maintenance.

Figure ES-6. 2040 Influent Flow Projections for the City's WWTPs (historical data are from 2015–2021)

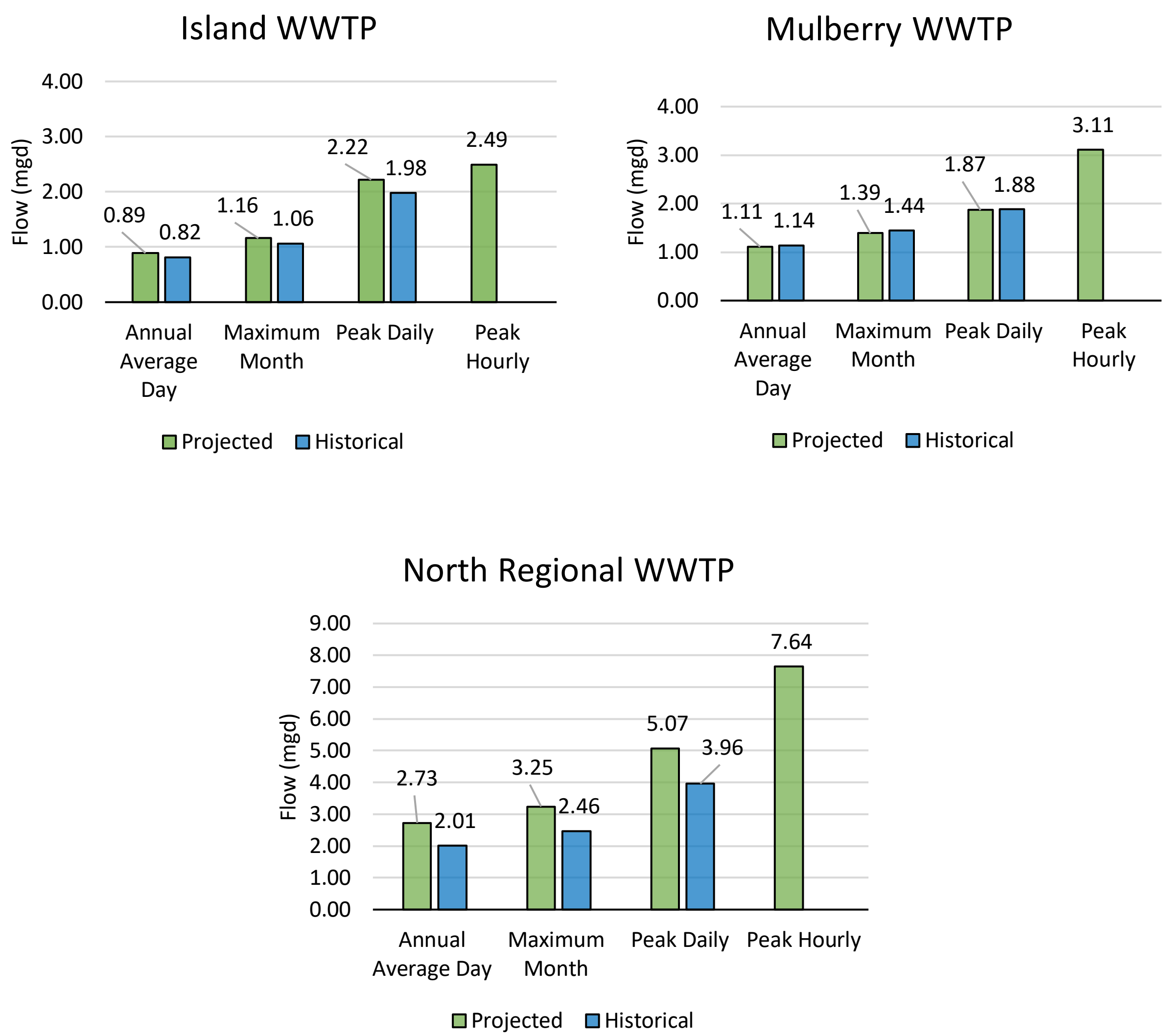


Figure ES-7 shows influent load projections for the City's 3 wastewater treatment plants.

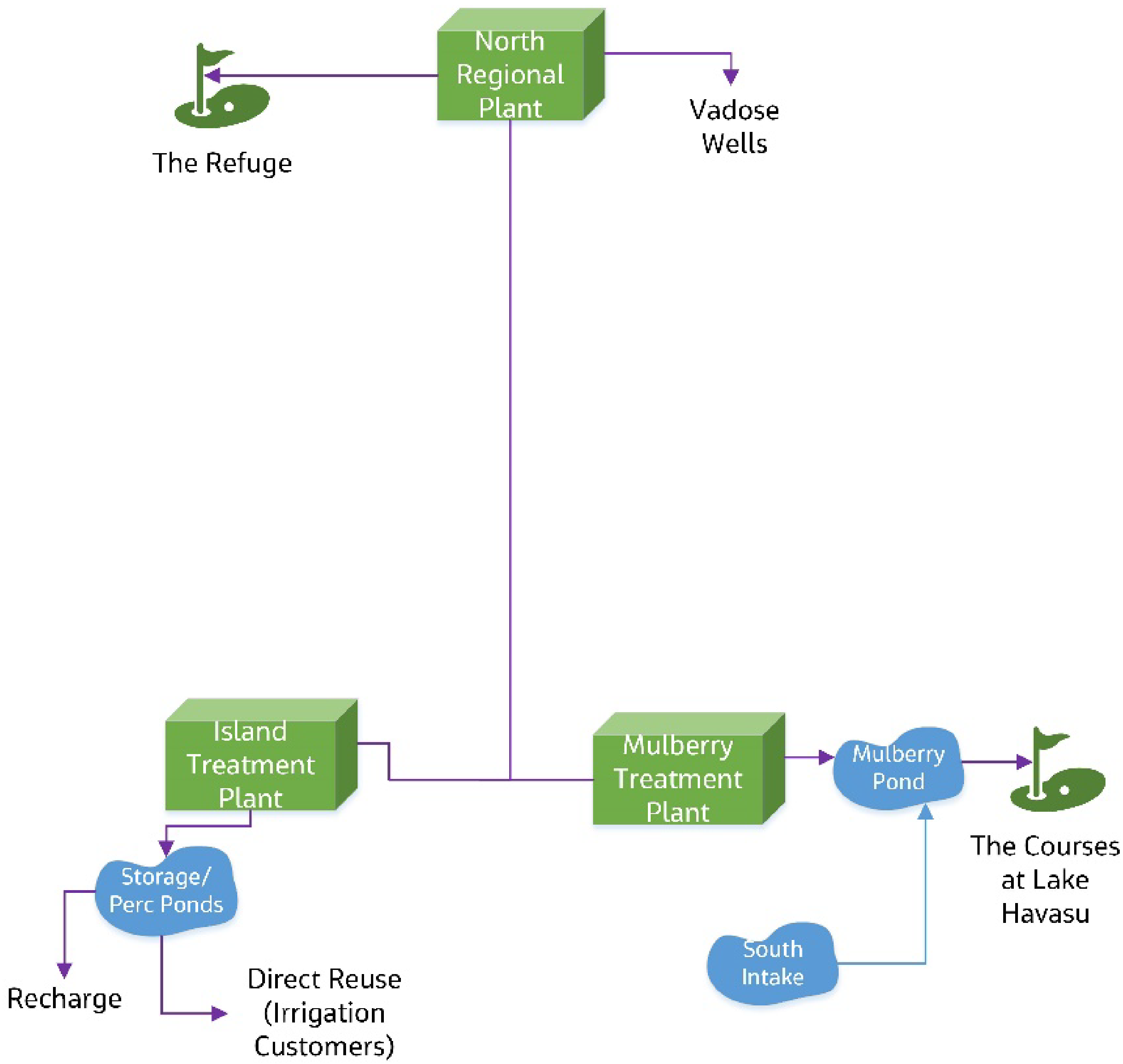
Figure ES-7. 2040 Influent Load Projections for the City's WWTPs

Reclaimed/Reuse System

The City's reclaimed water system is a "closed system" where all effluent must either be directly reused by customers or recharged. A schematic is shown on Figure ES-8, which highlights reuse customers, including two major golf courses and irrigation customers near the Island WWTP. The City may also recharge effluent via vadose zone wells at the North Regional WWTP or the percolation ponds at the Island WWTP. If needed, there is an intake from the Colorado River that may be used to supplement supply to The Lake Havasu Golf Club. The system is flexible in that reclaimed water may be redirected between facilities as summarized as follows:

- Effluent from the North Regional WWTP may be directed to the Island WWTP or the Mulberry WWTP
- Effluent from the Mulberry WWTP may be directed to the Island WWTP
- Effluent from the Island WWTP may be directed to the Mulberry WWTP (rarely operated)

Figure ES-8. Reclaimed System Schematic



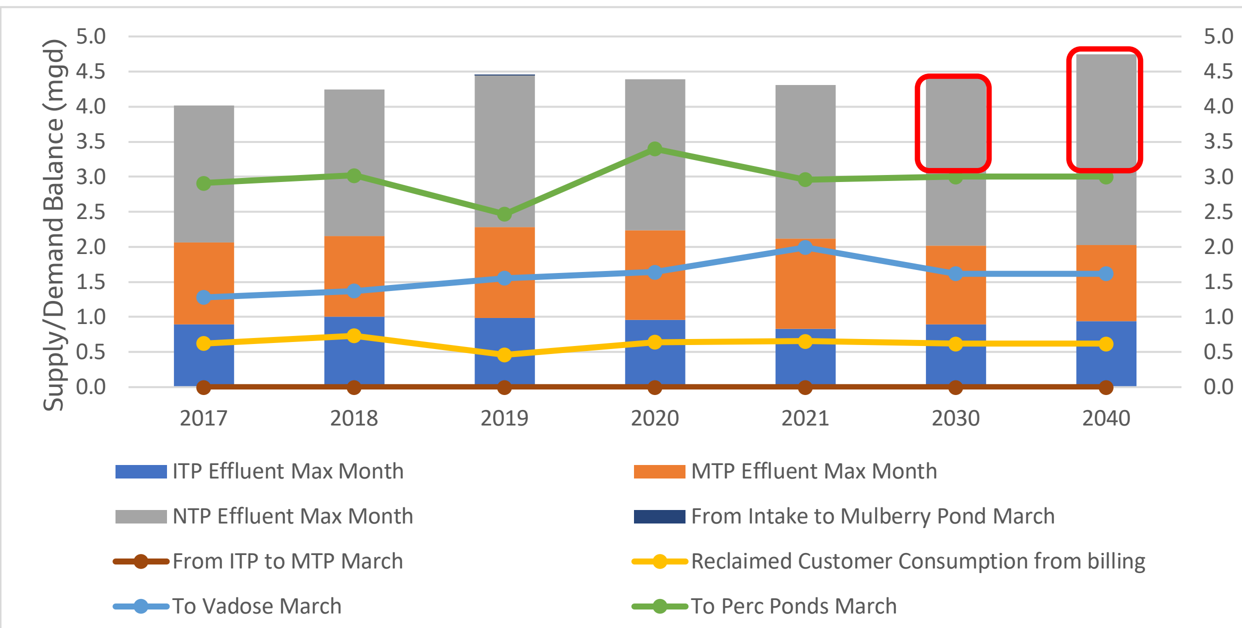
To determine the future needs of the reclaimed water system, Jacobs used the wastewater flow projections to estimate effluent available in the future. The same influent-to-effluent ratios were applied to each plant, where the amount of reclaimed water produced by the plants will be about 4.3 mgd by 2040.

There are very few additional customers that may use reclaimed water for irrigation purposes in the future; Sara Park is a potential future large user, but the investment required to build reclaimed water delivery infrastructure to the south end of the City is cost prohibitive. Because of this constraint, Jacobs assumed future reclaimed customer consumption would equal the 5-year historical average (either annual, in March, or minimum, depending on the scenario). Other assumptions are noted as follows:

- Deliveries to the vadose wells are limited to 1 mgd (current maximum capacity)
- Deliveries to the percolation ponds are equal to the 5-year historical average (either annually or in March depending on the scenario)
- Reclaimed water transferred from Island WWTP to Mulberry WWTP is equal to the 5-year historical average (either annually or in March depending on the scenario)

To evaluate a worst-case scenario, Jacobs also compared the effluent available under MM conditions against minimum reclaimed water customer consumption. The 5-year historical minimum reclaimed water consumed by customers was applied to the 2030 and 2040 projections as shown on Figure ES-9.

Figure ES-9. Maximum Month Reclaimed Water Balance with Minimum Reclaimed Water Customer Consumption Projection



Note: These data represent a hypothetical condition and do not portray historical results that occurred simultaneously.

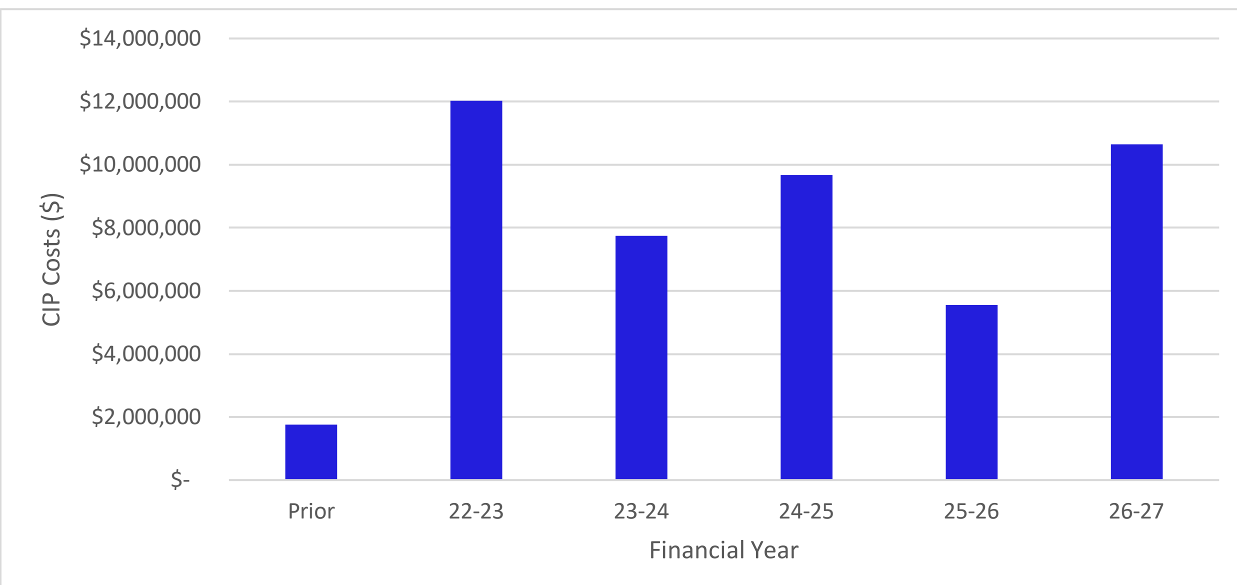
The red rectangles in the figure represent the amount of vadose well capacity that the City would need to add in the future. By 2030, the City would need about 1.5 mgd of additional capacity and nearly 2 mgd of additional capacity by 2040. The primary method for expanded effluent management will be expanded vadose zone wells. In addition to vadose well expansion, the City may consider other alternatives in the future, including:

- Direct potable reuse
- Rehabilitation of the Island WWTP percolation ponds
- Discharge via an effluent outfall pipeline into the Colorado River
- Conversion of existing potable water irrigation customers to reclaimed water use

Capital Improvement Plan

The projects presented in this section are recommended to maintain and improve the wastewater collections, treatment, and reclaimed systems and continue to meet projected growth. Projects are prioritized by capacity, reliability, or rehabilitation improvements to the existing system. The recommended wastewater capital improvement plan (CIP) provides the City's customers with a system that meets the design criteria and can be operated efficiently and reliably. Should projected growth forecast during the planning horizon (2040) not be realized, there may be opportunities to defer or eliminate some projects. Figure ES-10 illustrates the 5-year CIP costs. The Wastewater Master Plan identified new projects included in the wastewater CIP, as well as changes to the costs or schedule of existing projects. These new projects/changes are summarized following the table and chart.

Figure ES-10. 5-year CIP Costs



Collection System

Lift Station Improvements

As part of the regional pumping system optimization evaluation, upgrades were identified at four lift stations. The recommended improvements at each lift station are:

- Sweetwater Lift Station Pump Replacement
- London Bridge Lift Station Pump Replacement
- Bombay Lift Station Pump Replacement and Bar Screen Installation
- Influent Pump Station Pump Replacement and Surge Improvements

Future Expansion Areas

Four areas were identified for future system expansion as described as follows:

- Area "A" includes a backbone deep sewer, trunk sewer extensions, two new local sewer lift stations, and force mains to service future growth in the area. Jacobs recommends redirecting the Canterbury and Refuge lift stations to connect to the backbone deep sewer by extending a force main across Highway 95 and abandoning portions of the common force main per recommendations made in the North Regional Sub-Area Master Plan.
- Area "B" includes providing a backbone gravity sewer to the Refuge lift station.
- Area "C" includes providing a backbone gravity sewer to a new local pump station. The new pump station will convey flows through a new force main to the Centre lift station.
- Area "D" includes providing a new lift station, force main, and backbone gravity sewer to convey flows to the Island WWTP.

Pipeline Rehabilitation and Gravity Replacement

The City maintains an annual fund for miscellaneous pipeline rehabilitation and replacement. This fund allows the City to be proactive in maintaining the collection system.

Wastewater Treatment Plants

Island WWTP

The following improvements to the Island WWTP are recommended to ensure that the WWTP is functioning properly and is always in compliance with its Arizona Department of Environmental Quality (ADEQ) permit.

- Primary treatment improvements, including a new headworks building that includes screens, grit removal, hydraulic capacity upgrades, odor control, and electrical improvements. The existing headworks will reach the end of its useful life in the next 5 years and will need to be rebuilt.
- The new flow equalization basin (FEB) is being negotiated by others and is already included in the City's CIP.

- Installation of submerged diffusers on the floor of the aeration basin instead of the rotating bridge. This will help to protect the concrete walls of the aeration basin.
- Repairing and upgrading the traveling bridge filter to ensure reliable filtration.
- Complete replacement of the ultraviolet (UV) system once it reaches the end of its useful life.
- Repairing and rehabilitating Effluent Pond B and converting it into a percolation pond.

Mulberry WWTP

The following improvements to the Mulberry WWTP are recommended to ensure that the WWTP is functioning properly and is always in compliance with its ADEQ permit.

- The concrete structure of the aeration basins has developed cracks and needs to be repaired. Additionally, the City should evaluate installing aeration diffusers at the bottom of the basin to provide the necessary air for treatment. This will help protect the concrete structure.
- Complete replacement of the UV system once it reaches the end of its useful life.
- Upgrade the effluent pump systems and effluent pond.

North Regional WWTP

The following improvements to the North Regional WWTP are recommended to ensure that the WWTP is functioning properly and is always in compliance with its ADEQ permit.

- Installation of a grit removal system to reduce wear and abrasion of downstream mechanical equipment.
- The FEB has not been cleaned in years and has built up grit and other solid material. It is recommended that the City clean out the FEB to fully use the equalization capacity.
- Complete replacement of the UV system once it reaches the end of its useful life.

Reclaimed/Reuse Water System

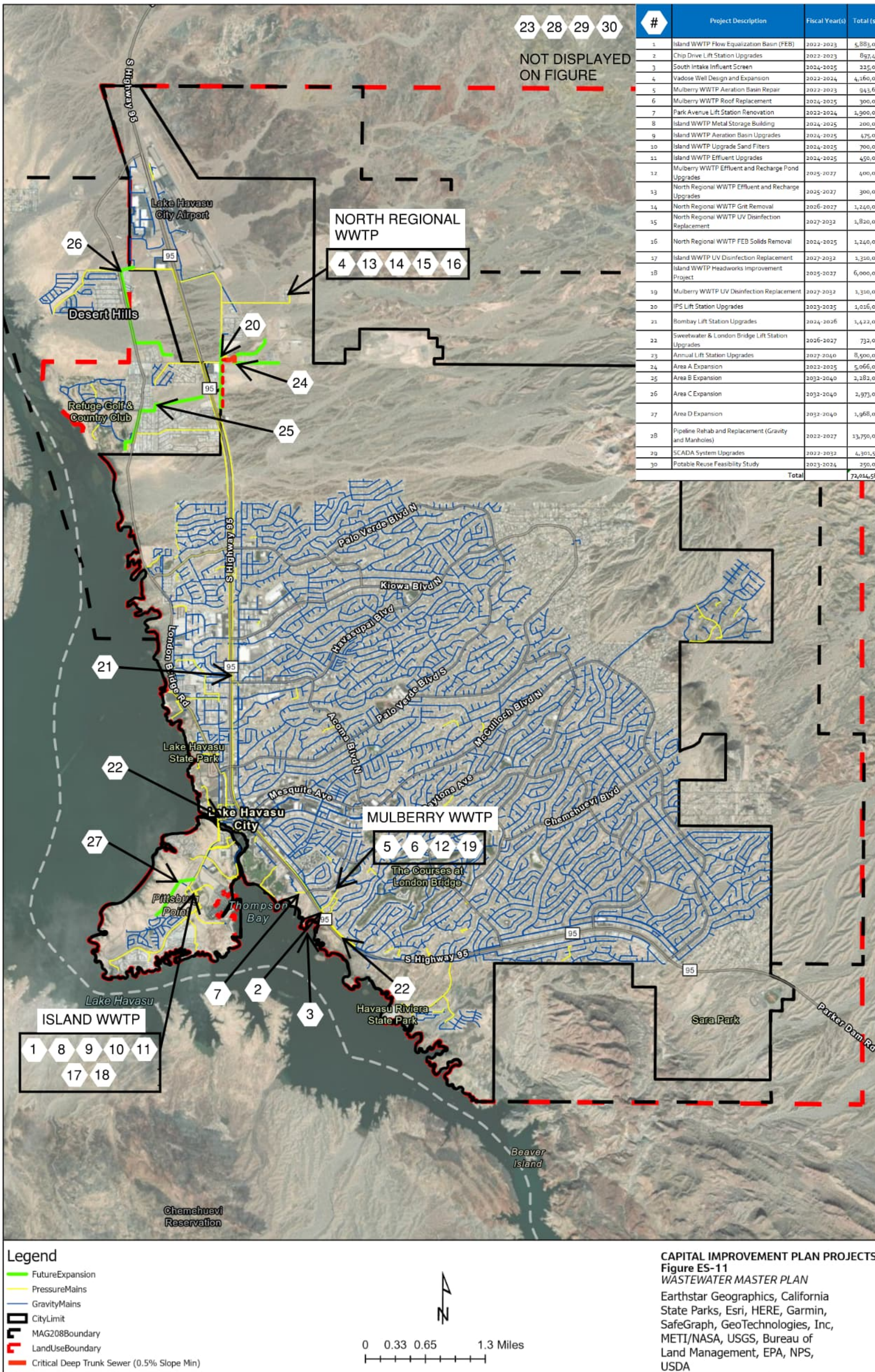
The following two projects are recommended to ensure successful future reclaimed water management:

- Add vadose wells. This project would include approximately 2 mgd of capacity to be confirmed as part of the Vadose Study. Design and construction would occur beginning in fiscal year (FY) 2021-2022, and conclude in FY 2024–2025.
- Undertake a reuse feasibility study to determine the applicability of direct potable reuse or other options available to the City. The study would commence in FY 2023–2024.

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

Figure ES-11 presents the planned CIP projects across the City.

Figure ES-11. Planned Wastewater CIP Projects



#	Project Description	Fiscal Year(s)	Total ($)
1	Island WWTP Flow Equalization Basin (FEB)	2022-2023	5,883,000
2	Chip Drive Lift Station Upgrades	2022-2023	897,480
3	South Intake Influent Screen	2024-2025	225,000
4	Vadose Well Design and Expansion	2022-2024	4,160,000
5	Mulberry WWTP Aeration Basin Repair	2022-2023	943,600
6	Mulberry WWTP Roof Replacement	2024-2025	300,000
7	Park Avenue Lift Station Renovation	2022-2024	1,900,000
8	Island WWTP Metal Storage Building	2024-2025	200,000
9	Island WWTP Aeration Basin Upgrades	2024-2025	475,000
10	Island WWTP Upgrade Sand Filters	2024-2025	700,000
11	Island WWTP Effluent Upgrades	2024-2025	450,000
12	Mulberry WWTP Effluent and Recharge Pond Upgrades	2025-2027	400,000
13	North Regional WWTP Effluent and Recharge Upgrades	2025-2027	300,000
14	North Regional WWTP Grit Removal	2026-2027	1,240,000
15	North Regional WWTP UV Disinfection Replacement	2027-2032	1,820,000
16	North Regional WWTP FEB Solids Removal	2024-2025	1,240,000
17	Island WWTP UV Disinfection Replacement	2027-2032	1,310,000
18	Island WWTP Headworks Improvement Project	2025-2027	6,000,000
19	Mulberry WWTP UV Disinfection Replacement	2027-2032	1,310,000
20	IPS Lift Station Upgrades	2023-2025	1,016,000
21	Bombay Lift Station Upgrades	2024-2026	1,422,000
22	Sweetwater & London Bridge Lift Station Upgrades	2026-2027	732,000
23	Annual Lift Station Upgrades	2027-2040	8,500,000
24	Area A Expansion	2022-2025	5,066,000
25	Area B Expansion	2032-2040	2,282,000
26	Area C Expansion	2032-2040	2,973,000
27	Area D Expansion	2032-2040	1,968,000
28	Pipeline Rehab and Replacement (Gravity and Manholes)	2022-2027	13,750,000
29	SCADA System Upgrades	2022-2032	4,301,500
30	Potable Reuse Feasibility Study	2023-2024	250,000
	Total		72,014,580

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Appendixes

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Acronyms and Abbreviations

Acronym	Definition
AAD	average annual daily
ADEQ	Arizona Department of Environmental Quality
AL	alert limit
APP	Aquifer Protection Permit
BFP	belt filter press
BOD	biochemical oxygen demand
BOD_5	5-day biochemical oxygen demand
$cBOD_5$	5-day carbonaceous (nitrification inhibited) biochemical oxygen demand
CIP	capital improvement plan
City	Lake Havasu City
COD	chemical oxygen demand
d/D	ratio of maximum depth of flow to pipe diameter
DO	dissolved oxygen
FEB	flow equalization basin
ft/sec	feet per second
FY	fiscal year
GIS	geographic information system
gpcd	gallon(s) per capita per day
gpd	gallon(s) per day
gpm	gallon(s) per minute
H_2S	hydrogen sulfide

Acronym	Definition
hp	horsepower
I/I	inflow and infiltration
IPS	influent pump station
lb/d	pound(s) per day
MBR	membrane bioreactor
MCFCD	Mohave County Flood Control District
mg	milligram(s)
mgd	million gallon(s) per day
mg/L	milligram(s) per liter
mL	milliliter(s)
MM	maximum month
MOP	Manual of Practice
MPO	Metropolitan Planning Organization
N/A	not available
PD	peak daily
RAS	return activated sludge
RDII	rainfall-derived inflow and infiltration
rpm	revolution(s) per minute
SCADA	supervisory control and data acquisition
SPA	state point analysis
TDH	total dynamic head
TSS	total suspended solids
UV	ultraviolet

Acronym	Definition
VFD	variable frequency drive
WAS	waste activated sludge
WEF	Water Environment Federation
WTP	water treatment plant
WWSE	Wastewater System Expansion
WWTP	wastewater treatment plant

1. Overview

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 affect the water and wastewater systems.

The City operates a wastewater system consisting of 3 wastewater treatment plants (WWTPs), 49 sewer lift stations (public and private), 49 miles of sewer force main, and 350 miles of sewer gravity mains. The City provides wastewater services to approximately 58,000 people.

The Wastewater Master Plan is one of many documents that are used to plan for future infrastructure needs of the City to ensure reliable wastewater collection, treatment, and disposal services for 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 wastewater generation, aging infrastructure, and regulatory and financial requirements.

After completing several wastewater master planning studies in the late 1990s and early 2000s, the City embarked on its $348 million Wastewater System Expansion (WWSE) Program in 2002 converting existing septic systems to a new sewer collection system and reducing nitrate loading to the Colorado River. The award-winning construction project was completed in 2011, 2 years ahead of schedule, and under budget (estimated at $463 million). Under the WWSE, the City eliminated more than 20,000 septic systems and constructed a new treatment plant. In 2014, the City completed the Wastewater System Expansion Program Oversight Finalization Report (Lake Havasu City 2014), which summarized the recently completed wastewater expansion system, documented sewer generation rates, and laid out a sewer system capital improvement program for the City.

The primary purpose of this Wastewater Master Plan is to update the wastewater generation forecast, evaluate regional pumping and treatment plant infrastructure, identify effluent reuse opportunities and wastewater disposal management options, and develop near-term and identify long-term capital improvement projects and cost estimates.

1.2 SCOPE

The 2022 Wastewater Master Plan scope of work focuses on updating wastewater generation projections, updating the existing sewer system hydraulic model, optimizing the regional conveyance and pumping infrastructure, evaluating the existing treatment plant infrastructure, identifying effluent reuse options, and developing a capital improvement program.

The 2022 Wastewater Master Plan scope of work includes the following:

- Flow Projections
- Model Update
- Collection System Modeling and Optimization
- Facility Evaluations and Recommendations
- Reclaimed/Reuse Evaluations
- Capital Improvement Plan Implementation

1.3 RECENT MASTER PLANS

This section summarizes the latest City master plans for wastewater and water systems.

1.3.1 WASTEWATER MASTER PLANS

The 2014 WWSE 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 and flow data. The report also updated unit sewer generation rates per capita and revised future flow projections based on the latest population projections.

Projected 2024 wastewater flows were 4.6 million gallons per day (mgd) based on a unit sewer generation rate of 77 gallons per day per capita. The 2014 WWSE Report concluded that the wastewater system continued to experience lower than anticipated sewer flows than it was designed for, and major components of the system have excess capacity. Accordingly, no significant conveyance capital improvements were recommended for the wastewater system. The 2014 WWSE Report scope of work did not include a detailed evaluation of the treatment plants.

The City is served by three WWTPs. The last master plan update for the WWTPs was prepared by AMEC Earth and Environmental in July 2009. The Island WWTP with a design capacity of 2.5 mgd and the Mulberry WWTP with a design capacity of 2.2 mgd are the older wastewater facilities located in the heart of the City, as noted by the previous master plan update. The third and newest WWTP, the North Regional WWTP, has a capacity of 3.5 mgd and is located in the far north portion of the City. (Note: all flow capacities are downstream of influent flow equalization). The location requires series pumping to convey flows to the plant headworks a distance of approximately 4.3 miles from the Bombay Lift Station to the North Regional WWTP. 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 the time of the WWSE Program.

1.3.2 WATER MASTER PLAN

The City completed a comprehensive Water Master Plan Update in 2018 (2018 WMPU)(Lake Havasu City 2018). The 2018 WMPU focused on water supply resources and reliability and water distribution system upgrades to meet existing and projected water demands. In addition, the 2018 WMPU addressed system redundancy, risk, and consequences of failure for its major water supply, a horizontal collector well, and the long-term sustainability of the North Wellfield to meet the City's future water supply needs. The 2018 WMPU also provided a 5- and 10-year capital improvement plan.

2. Basis of Planning

This section describes and establishes the basis of planning for this Wastewater Master Plan, including the wastewater service area, land use and major development information, and population data.

2.1 WASTEWATER SERVICE AREA DESCRIPTION

The City provides wastewater service to customers both within and outside its City limits. The City is obligated to provide sewer service within the boundary established by the Mohave County 208 Water Quality Management Plan (208 Plan). In addition, the City's 2016 General Plan includes a sphere of influence that extends beyond its 208 Plan boundary in certain potential development areas. Figure 2-1 presents the existing sewer service area, the City boundary, the 208 Plan boundary, and the City's 2016 General Plan's sphere of influence for reference and each of the boundaries are described in the following sections.

2.1.1 WASTEWATER SERVICE AREA

The City's existing wastewater service area is approximately 27 square miles and generally serves the majority of the City limits and portions of Desert Hills and Crystal Beach (Figure 2-1 shows this area in the form of gravity mains and pressure mains). The wastewater system is maintained and operated by the City's Wastewater Division and includes nearly 350 miles of gravity sewer mains, 25 miles of force mains, 49 wastewater pump stations, 3 wastewater treatment plants, and an effluent disposal system.

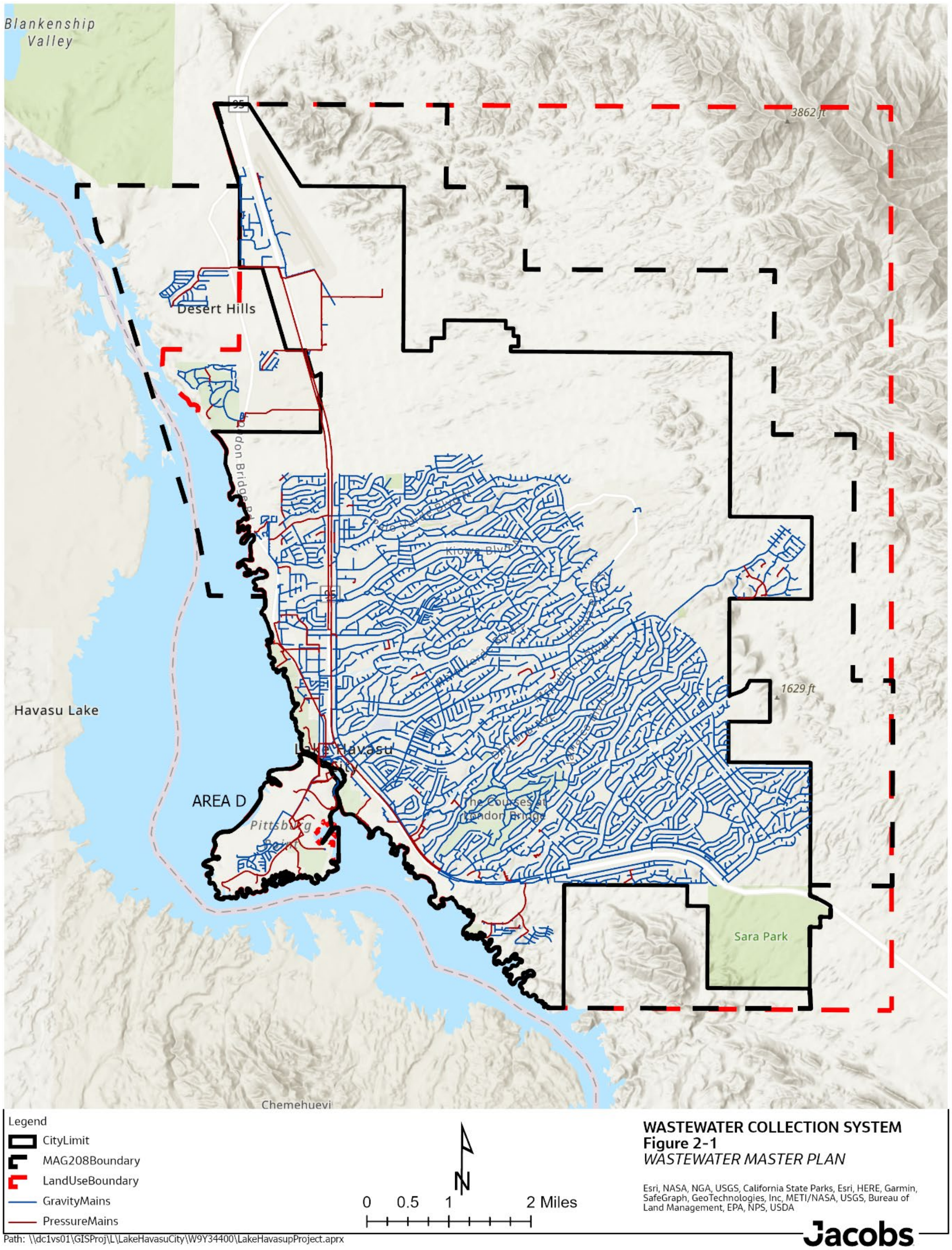
2.1.2 CITY BOUNDARY

The City is located in Mohave County and encompasses approximately 46 square miles. The City was incorporated in 1978 and provides a number of services to its residents, including water and wastewater collection, treatment, and disposal. A small portion of the City wastewater served area is served water by a private water utility (EPCOR).

2.1.3 208 BOUNDARY

The 208 Plan was developed in accordance with the Clean Water Act requirement to develop a regional water quality plan. One purpose of this plan is to encourage areawide planning of wastewater conveyance and treatment facilities so that they are sized appropriately for the communities or groups of communities they serve. The City is a Designated Management Agency, meaning it is the agent designated by the Governor of Arizona to implement recommendations of the 208 Plan. The City's wastewater system planning includes considerations for additional future flows that are outside the currently sewered areas but within the 208 Plan boundary. The existing 208 Plan boundary encompasses approximately 71 square miles.

Figure 2-1. Lake Havasu City Sewer Service Area



2.2 LAND USE

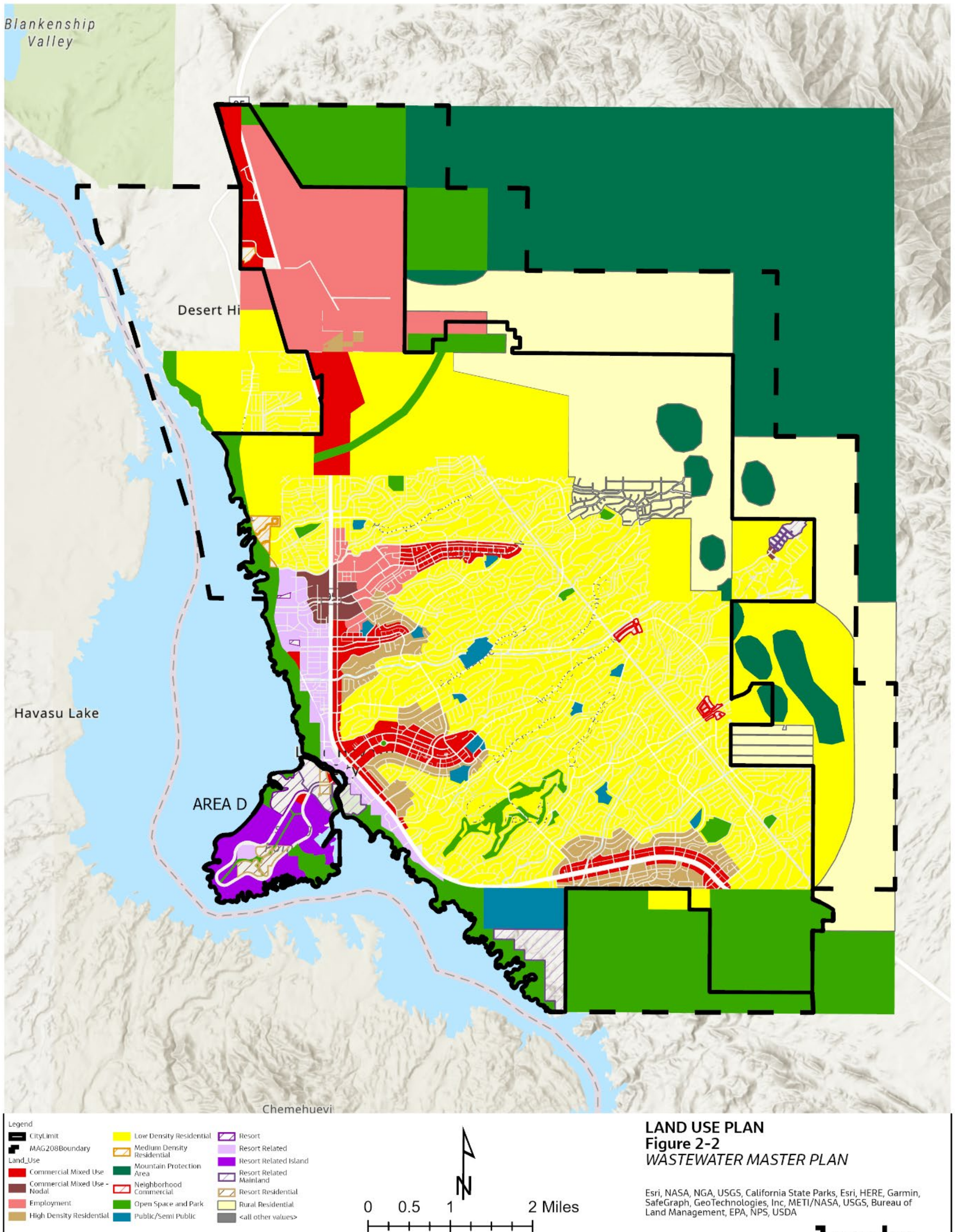
2.2.1 GENERAL PLAN

The 2016 Lake Havasu City General Plan (2016 General Plan) is a long-range plan for guiding 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 is a statement of policy and an expression of the community's vision for the future. The plan is a tool for helping to guide and shape the planning area's physical development (Lake Havasu City 2016). For this Wastewater Master Plan, as with the 2018 WMPU, the 2016 General Plan was used as a reference to help shape projected wastewater generation (Lake Havasu City 2016). The current land use designation from the 2016 General Plan is shown on Figure 2-2.

Figure 2-2. Lake Havasu City General Plan Land Use



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2.2.2 POPULATION PROJECTIONS

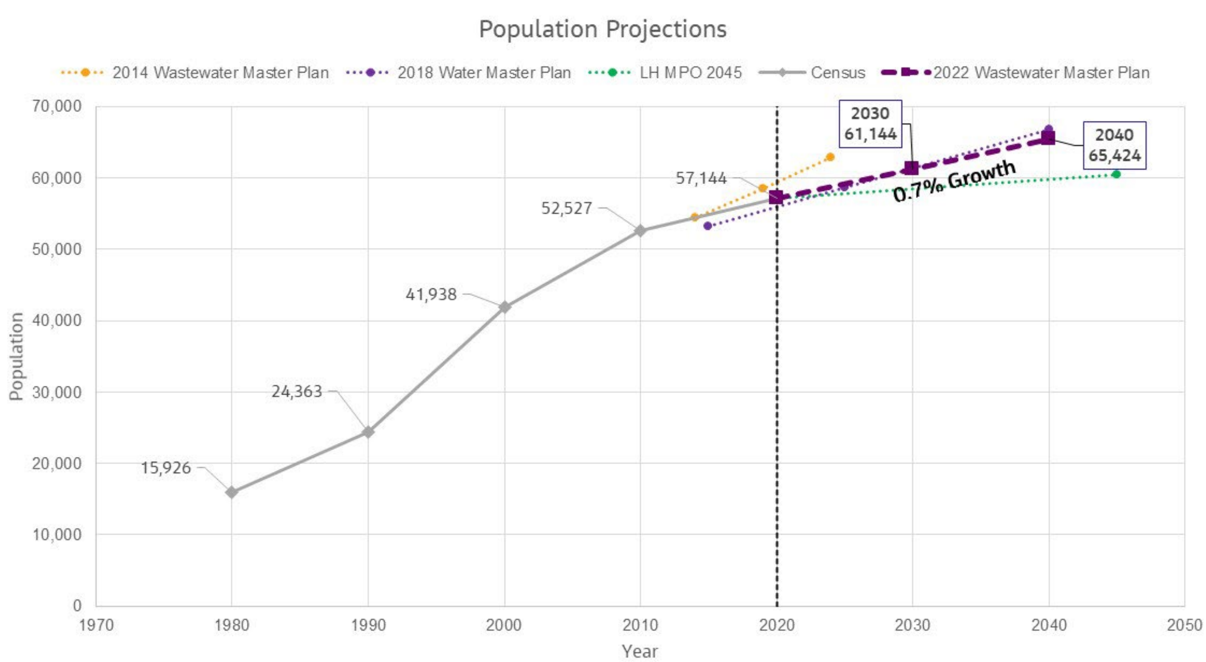
Population estimates were updated based on the recent forecasts performed as part of the 2018 WMPU (Lake Havasu City 2018) and recently completed census data (USCB 2020). The sources used in the development of the population projections are as follows:

- Arizona Commerce Authority
- United States Census Bureau
- Lake Havasu City General Plan (2016)

According to the United States Census Bureau, the City's population has increased from 41,938 in 2000 to 52,547 in 2010 and 57,144 in 2020. According to the Arizona Commerce Authority, the City is projected to add around 14,000 additional residents by 2040 (Arizona Commerce Authority n.d.). Population projections for Lake Havasu City indicate a slow but steady increase of residents in the City and Mohave County over the next 25 years (Arizona Commerce Authority n.d.).

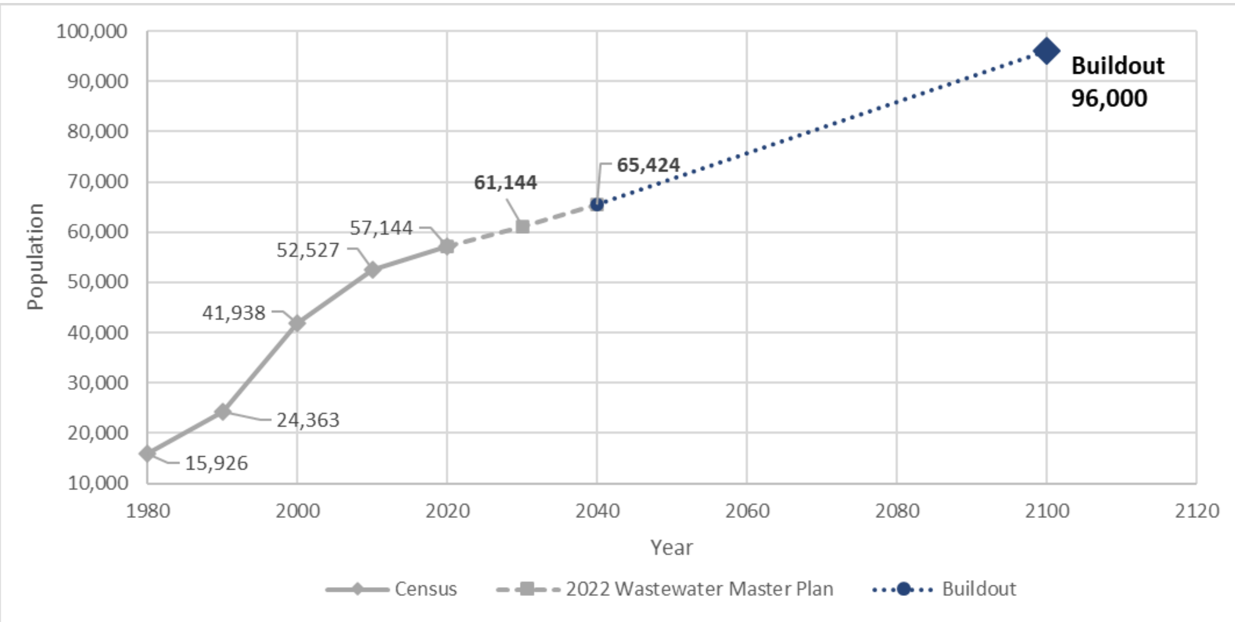
Figure 2-3 shows population-based projected growth through 2040. The 2014 WWSE population forecast assumed a growth rate of 1.5 percent from 2014 through 2025. Using the more recent population projections reported in the Lake Havasu Metropolitan Planning Organization (MPO) Regional Transportation Plan (MPO 2022), the population growth was estimated to be 0.7 percent from 2014 through 2040. Jacobs reviewed this growth rate with the City Planning and Zoning Division and agreed to use 0.7 percent to forecast future wastewater flows.

Figure 2-3. Lake Havasu City Population Forecast



The predicted build-out population for the City in the future would be 96,000 as stated in the 2016 General Plan. (Build-out population is determined by the City's water allocation, and is discussed further in the 2018 Water Master Plan). The plan assumed that a growth rate of 0.7 percent would be a conservative estimate. Figure 2-4 presents the overall City future population growth based on the build-out population. Based on 0.7 percent 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-person increase from 2040. Based on an average increase of 500 people per year, it would be equivalent to another 60 years to reach the build-out population (year 2100).

Figure 2-4. Lake Havasu City Build-out Population Forecast



2.3 DESIGN CRITERIA

This section summarizes the recommended wastewater system design criteria for the Wastewater Master Plan based on review of previous master plans, industry standards, and input from City wastewater staff. Table 2-1 provides a summary of treatment plant, pump station, force main, and gravity main design criteria.

Table 2-1. Design Criteria

Item	Recommended Criteria
Lift Station Criteria	
Minimum Number of Pumps	Two
Minimum Pump Capacity	Each pump must be able to operate at the peak wet weather design flow and head.

Table 2-1. Design Criteria

Item	Recommended Criteria
Standby Capacity	The lift station must be capable of operating at peak wet weather flow with any one pump out of service.
Emergency Power	Required when pump station average day flow is ≥ 10,000 gpd. Emergency generation must be capable of operating all pumps at the same time. Emergency power may be required depending on site conditions based on City discretion.
Emergency Storage Capacity	1 hour at the peak wet weather flow. Usable volume of wet well from pump to top of the inlet pipe.
Force Main Criteria	
Minimum Pipe Diameter	4-inch, for all public force mains. Private force mains with grinder pumps may use smaller diameters.
Minimum Velocity	3 ft/sec for lengths < 2,000 feet 3.5 ft/sec for lengths 2,000 < 5,000 feet 4 ft/sec for lengths > 5,000 feet
Maximum Velocity	7 ft/sec
Hazen Williams "'C"' Factor	130, unless otherwise approved by the City
Gravity Main Criteria	
Minimum Pipe Diameter	≥ 8-inch, except first 400 feet of a dead-end line may be 6-inch if the line is not planned for extension.
Minimum Velocity	2 ft/sec at peak dry weather flows.
Manning's Roughness Coefficient	0.013
Maximum Peak d/D Ratio for Existing Sewers	0.75 for peak wet weather flow.
Maximum Peak d/D Design Criteria For New Sewers	0.75 for peak wet weather flow.
Treatment Plant Criteria	
Planning for Expanded Treatment Capacity	Based on the AL limits for the monthly average flows defined in the respective APP. Once the AL limit has been reached, the City is required to submit a permit amendment to either expand the treatment facility or submit a report detailing the reasons if an expansion is not necessary.
Hydraulic Capacity	Physical unit processes upstream of flow equalization are based on the peak hourly flow. Physical unit processes downstream of flow equalization are based upon the peak equalized flow rate.

Table 2-1. Design Criteria

Item	Recommended Criteria
Treatment Capacity	Biological treatment process capacity (secondary treatment) are based on the maximum month organic and nitrogen loading

APP = Aquifer Protection Permit
AL = Alert Limit
d/D = ratio of maximum depth of flow to pipe diameter
ft/sec = feet per second
gpd = gallon(s) per day

2.3.1 LIFT STATIONS

Table 2-1 includes lift station or pump station criteria. A more detailed discussion of the optimization of each lift station or pump station is discussed in Section 5.2. Lift stations are to be designed for peak wet weather conditions of the build-out population, with considerations given to phasing if deemed appropriate by the City. A redundant pump must be included, equal in size to the largest pump in the facility. Odor control may be required depending on site conditions based on City discretion.

2.3.2 FORCE MAINS

Table 2-1 includes the criteria for force mains. Minimum velocity criteria is implemented to prevent excess hydrogen sulfide gas generation in long force mains.

2.3.3 GRAVITY MAINS

Table 2-1 includes the criteria for gravity mains. Minimum velocity criteria is implemented to keep the gravity mains cleansed of static wastewater. Siphon mains are not allowed by the City.

2.3.4 TREATMENT PLANTS

Table 2-1 includes the criteria for treatment plants. A more detailed assessment of each treatment plant is discussed in Section 6. This Includes influent flows and loading, effluent criteria, a wastewater staffing plan, and a biosolids management plan.

3. Wastewater Flow Estimates

This section describes the existing wastewater flows and outlines future wastewater flow projections.

3.1 HISTORICAL WASTEWATER FLOWS

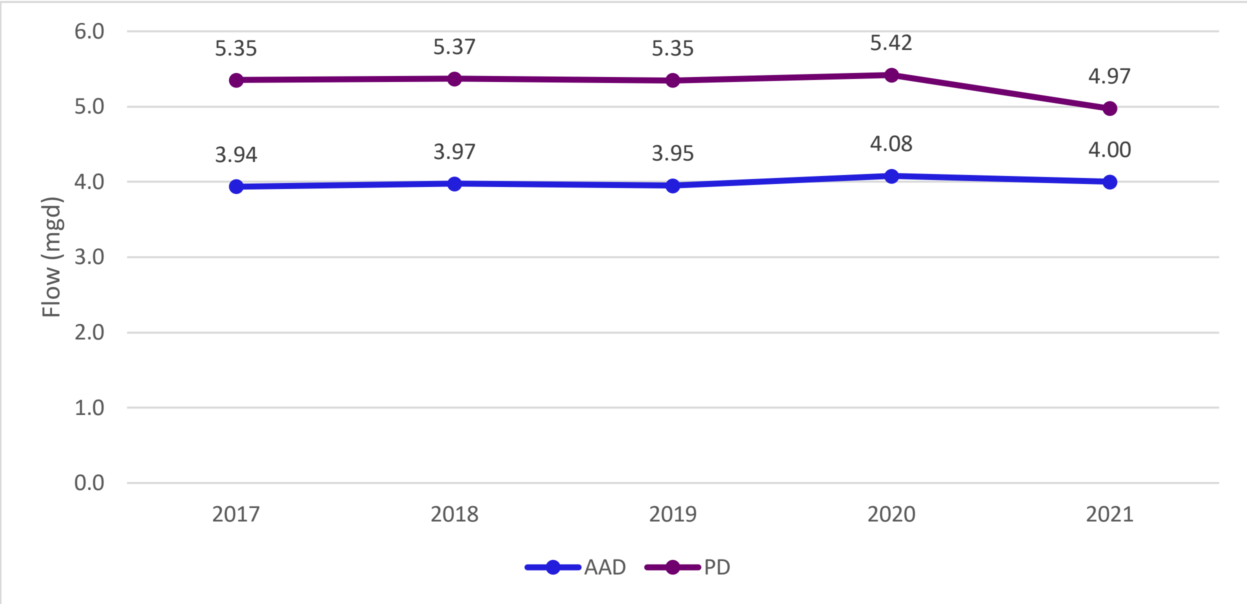
The City records daily wastewater flows to each of the WWTPs. There are influent flowmeters located at the Island WWTP, Mulberry WWTP, and the North Regional WWTP influent pump station. The WWTP flows from 2017 through 2021 were reviewed to determine average annual daily flows and peak daily flows, which are summarized in Table 3-1. Since 2017, the City's total average annual daily (AAD) wastewater flow was between 3.94 and 4.00 mgd with total peak average daily (PDs) flows of approximately 5.4 mgd, as illustrated on Figure 3-1.

Table 3-1. Historical Wastewater Flows

Year	Island WWTP (mgd)		Mulberry WWTP (mgd)		North Regional WWTP (mgd)		Total AAD (mgd)
	AAD	PD	AAD	PD	AAD	PD	
2017	0.77	1.98	1.21	1.62	1.96	2.65	3.94
2018	0.91	1.06	1.02	1.66	2.05	3.96	3.97
2019	0.78	1.04	1.27	1.86	1.90	3.19	3.95
2020	0.87	1.01	1.14	1.72	2.07	2.67	4.08
2021	0.75	0.87	1.21	1.88	2.05	2.91	4.00

The City's current AAD flow is approximately 4.0 mgd, the majority of which is treated at the North Regional WWTP.

Figure 3-1. Lake Havasu City Historical Flow (2017–2021)



3.1.1 UNIT WASTEWATER LOADS

Unit wastewater loads that express wastewater generation on a per acre basis were developed in the 2014 WWSE (Lake Havasu City 2014). Table 3-2 presents the wastewater unit loads.

Table 3-2. Unit Wastewater Load Criteria

Land Use	Wastewater Unit Load (gpad)[a]
Commercial	472
Commercial (Nodal)	279
Employment	447
Rural Residential	235
Low Density Residential	379
Medium Density Residential	669
High Density Residential	660
Mountain Protection Area	-
Open Space and Park	0
Public/Semi- Public	-
Resort	286
Resort Related	743
Resort Related Island	749

Table 3-2. Unit Wastewater Load Criteria

Land Use	Wastewater Unit Load (gpad)[a]
Resort Related Mainland	177
Resort Residential	3,161

[a] Unit rates from Wastewater System Expansion Program Oversight Finalization Report (Lake Havasu City 2014).

gpad = gallon(s) per acre per day

The unit loads presented in Table 3-2 were verified by comparing additional growth estimates provided in the 2014 WWSE Report with recent treatment plant flow data (Lake Havasu City 2014). Observed flow data from the WWTPs were in line with the growth projections noted in the 2014 WWSE Report using the aforementioned unit loads (Lake Havasu City 2014).

3.1.2 PEAKING FACTORS

Peaking factors were estimated using historical flow data collected from the City's WWTPs from 2017 through 2021. These factors are used in sizing critical components of the WWTPs, pump station and force main sizing, as well as the gravity collection system. Table 3-3 provides the AAD, maximum month (MM) average daily flow, and PD for the entire system for years 2017 through 2021.

Table 3-3. Peaking Factor Development

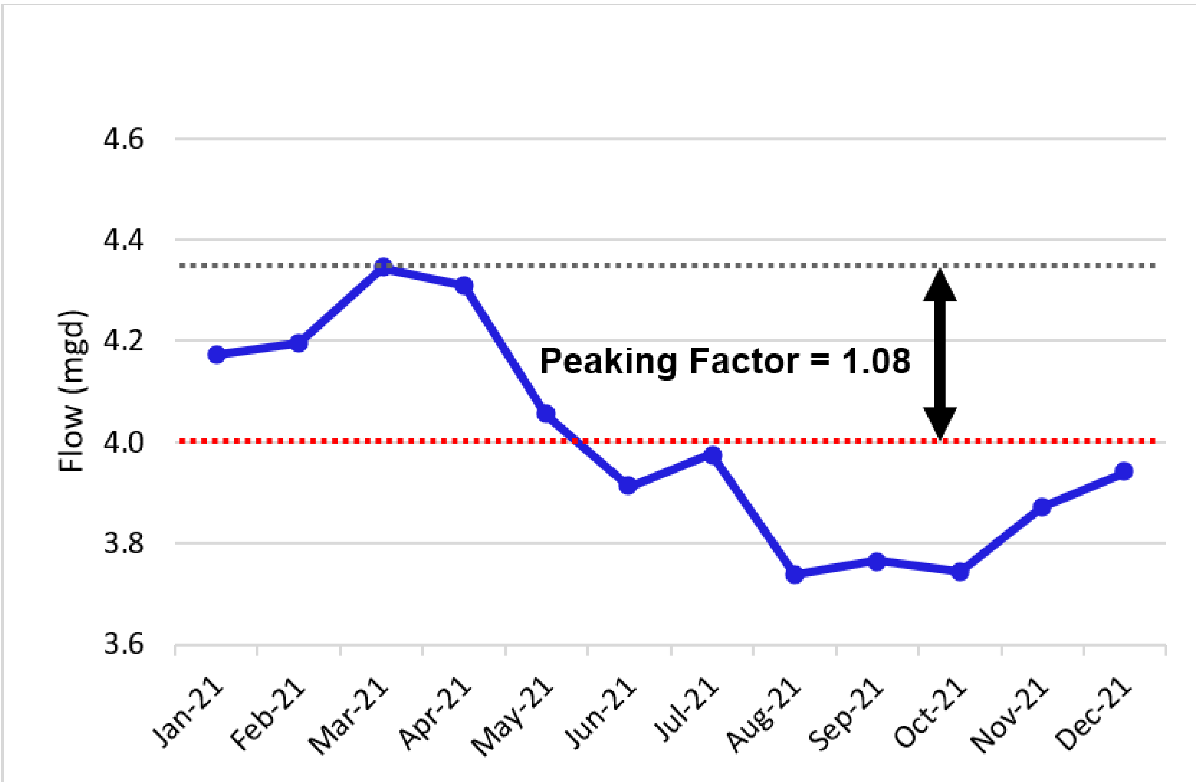
Year	Total (mgd)		
	AAD	MM	PD
2017	3.94	4.23[a]	5.35
2018	3.97	4.31[a]	5.37
2019	3.95	4.28[b]	5.35
2020	4.08	4.26[a]	5.42
2021	4.00	4.35[a]	4.97

[a] Maximum month = March of recorded year

[b] Maximum month = February of recorded year

Total average monthly influent flow for 2021 is shown on Figure 3-2. The AAD was 4.0 mgd with a MM of 4.35 mgd, resulting in a 1.08 peaking factor from MM to AAD. In the 2014 WWSE Report, the MM to AAD peaking factor ranged from 1.1 to 1.2 for the City's WWTPs (Lake Havasu City 2014).

Figure 3-2. Total Monthly Influent Flow – 2021



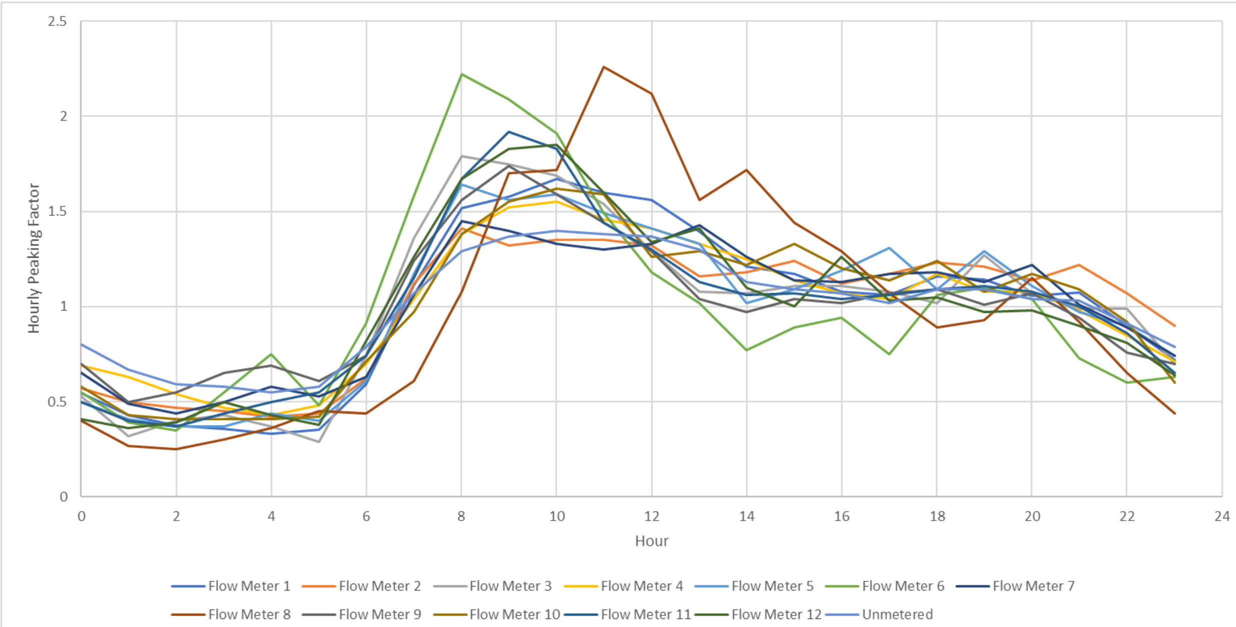
PD to AAD peaking factors ranged from 1.5 to 1.8 in the 2014 WWSE Report, which was developed based on basin-specific diurnal patterns (Lake Havasu City 2014). Current PD to AAD peaking factors ranged from 1.24 to 2.58 and are detailed in Table 3-4. Peaking factors will be applied in the flow projections to estimate future MM and PD flow conditions.

Table 3-4. Peak Day Factors by Treatment Plant

Year	Island	Mulberry	North	Overall System
2017	2.58	1.34	1.36	1.36
2018	2.16	1.63	1.94	1.35
2019	1.92	1.46	1.68	1.35
2020	1.76	1.51	1.29	1.33
2021	1.78	1.56	1.42	1.24

Basin-specific diurnal patterns (hourly peaking factors) were developed as part of the calibration effort in the 2014 WWSE. The calibrated hourly peaking factor values from the 2014 WWSE ranged from 0.25 to 2.26 and are presented on Figure 3-3 (Lake Havasu City 2014). Because model calibration was not part of the scope of this effort, the hourly peaking factors were not modified in the collection system model.

Figure 3-3. Hourly Peaking Factors



3.1.3 INFLOW AND INFILTRATION

Inflow and infiltration (I/I) rates were estimated by reviewing WWTP flow records the day before and the day of large-scale rainfall events. Given the arid desert climate, these rainfall events are rare and may just occur in the late summer monsoon season, or winter months. Mohave County Flood Control District (MCFCD) online data was used to identify rainfall events from 2017 to 2021. MCFCD rain gauges for the Lake Havasu City area are shown on Figure 3-4. Online data was reviewed for five of the MCFCD rain gauges within the Lake Havasu City area: Crystal Beach, Desert Hills, North Lake Havasu, Lake Havasu City, and Horizon 6. MCFCD identified 8 significant rainfall events during the allocated time period (greater than 0.6 inch). Table 3-5 documents the rain gauge, date of recording, rain increment (in inches), total WWTP flow the day prior, total WWTP flow the day of, and the delta between the 2 days. Duplicate rain increment readings at different rain gauges were removed from the table.

Figure 3-4. Mohave County Flood Control District Rain Gauge Locations

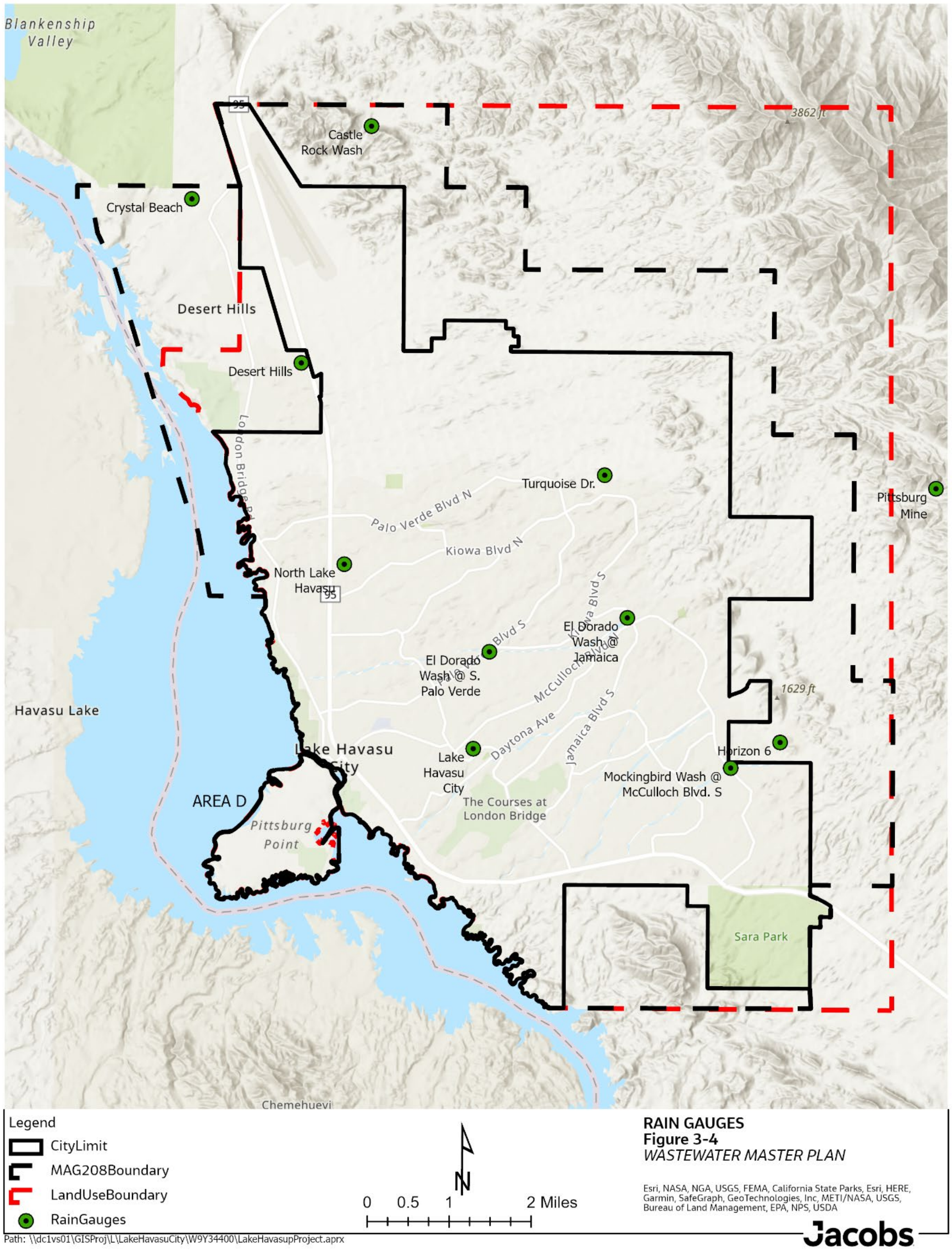


Table 3-5. Significant Rainfall Event Measurements

Rain Gauge Site	Date of Recording	Rain Increment (in)	WWTP Flow Day Prior (mgd)	WWTP Flow Day of (mgd)	Delta
Crystal Beach	1/1/2017	1.16	N/A	3.75	-
	10/13/2018	0.64	3.69	4.18	0.49
	2/14/2019	0.88	4.16	4.77	0.61
	11/28/2019	0.96	4.12	4.74	0.62
	3/12/2020	1.12	4.38	4.95	0.57
	3/18/2020	1.12	4.23	5.42	1.19
Desert Hills	9/9/2017	0.88	3.49	4.10	0.61
North Lake Havasu	10/13/2018	0.68	3.69	4.18	0.49
	3/12/2019	0.80	3.91	5.14	1.23
Lake Havasu City	1/15/2019	0.80	4.39	4.46	0.07

N/A = not available

The rainfall-derived I/I (RDII) ranged from 0.07 to 1.23, which equates to 1.75 percent to 30.75 percent of the AAD. With a typical I/I of 40 to 50 percent of the AAD, this issue appears relatively minor for the City. It is possible that variations in flows could be attributed to factors beyond wet weather (that is, seasonal generation). It is recommended that the City implement a program to monitor and measure flows for dry and wet weather periods to verify system flows. Routine maintenance of the collection system, including, but not limited to, utility access hole inspections, lift station evaluations, and pipe cleaning, is recommended to protect the system against I/I. An I/I Correction Plan may be warranted, which typically calls for upstream flow monitoring, pipeline inspections, and smoke testing, if there is higher I/I in the future.

3.2 FLOW PROJECTIONS

By applying a sewer unit generation rate of 70 gallons per capita per day (gpcd) to the population projections presented in the 2018 Water Master Plan and the Lake Havasu MPO document, wastewater flow projections were determined through 2040 for the basin (MPO 2022). In accordance with the MPO document and the 2018 Water Master Plan, the population growth was estimated to be 0.7 percent from 2014 through 2040. Population estimates and estimated wastewater flows are provided in Table 3-6. With this growth rate, roughly 400 people are added each year between 2020 and 2030, and 428 people are added each year between 2030 and 2040. Figure 3-5 correlates the projected population growth discussed in Section 2.2.2 to an AAD flow through 2040. Table 3-7 presents flow projections to the year 2040 using MM to AAD peaking factors and PD to ADD peaking factors.

Table 3-6. Estimated Future Wastewater Projections and Population

Year	Population	Population Change	Estimated Per Capital Flow (gpcd)	Flow (mgd)
2020	57,144	--	70	4.0
2030	61,144	+4,000	70	4.3
2040	65,424	+4,280	70	4.6

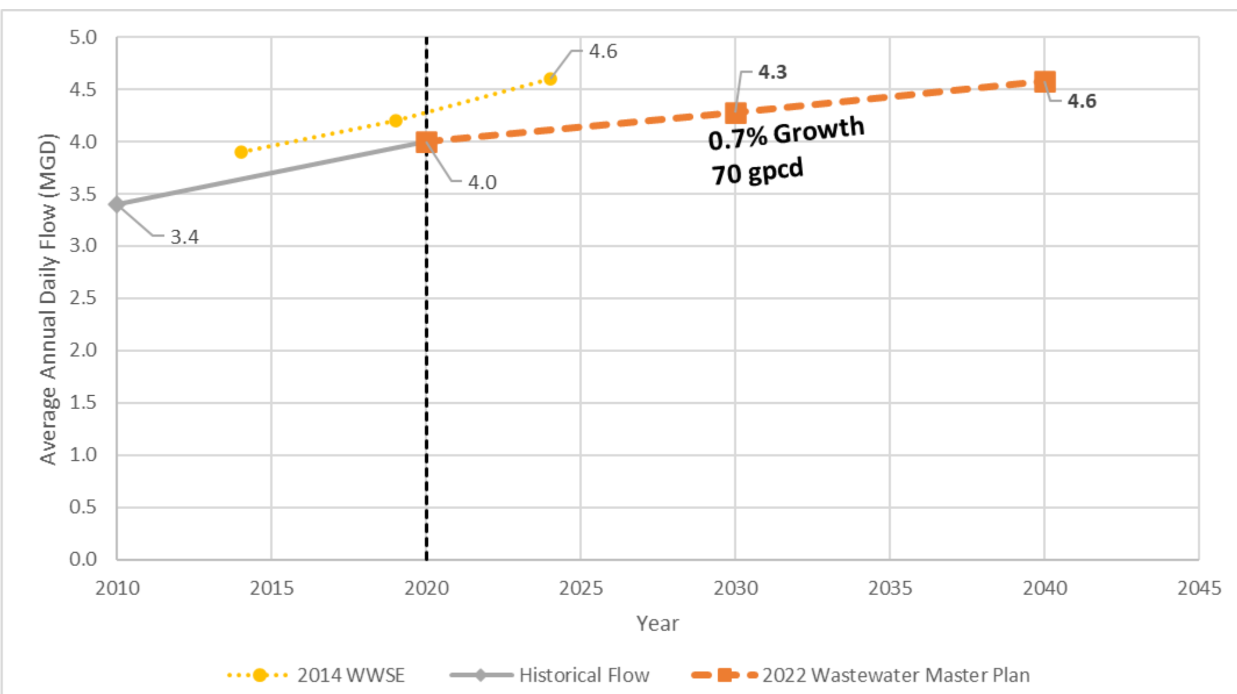
Table 3-7. Estimated Wastewater Flow Projections with Peaking Factors

Year	Average Annual Flow (mgd)	Average Yearly Increase (%)	Maximum Month Flow (mgd)	Peak Daily Flow (mgd)
2021	4.0	--	4.34	4.97
2030	4.3	0.7	4.64[a]	5.59[b]
2040	4.6	0.7	4.97[a]	5.98[b]

[a] Calculated based on AAD × 1.08

[b] Calculated based on AAD × 1.30

Figure 3-5. Wastewater Flow Projections



4. Hydraulic Model Update and Verification

This section describes updates to the hydraulic model developed for the City's wastewater collection system, which dates back to 2002. The last major update to the model was the 2014 WWSE, which included an extensive recalibration of the model. The focus of the current model updates is to include any new infrastructure constructed since 2014 along with adjustments to major lift stations and force mains to provide an accurate representation between the model and system performance.

4.1 BACKGROUND

As part of the WWSE program, the City developed a hydraulic model of the collection system. The first hydraulic model was developed using HydroWorks software between 2002 and 2005. The model was converted to InfoWorks CS (previously MWHSoft) in 2005 and updated to include areas captured in the WWSE. An additional update was completed in 2009 to reflect additional WWSE areas constructed from the previous update in 2005. As part of the 2014 WWSE, the model was updated to accomplish several tasks, including recalibration based on 2013 flow conditions, evaluation of system capacity under varying flow and operational conditions, evaluation of system capacity based on planned growth, and identification of capital improvements within the next 5 to 10 years. As part of the current master plan update, the model is being converted from InfoWorks CS to InfoWorks ICM because Innovyze discontinued support for InfoWorks CS. Similar to the 2014 WWSE, goals of the model update include:

- Validation of the model for 2021 flow conditions
- Evaluation of existing system capacity under multiple flow and operational conditions
- Evaluation of future system capacity based on planned growth and development under multiple flow and operational conditions
- Optimization of wastewater flows to each WWTP and maximizing reuse
- Identification of capital improvements that address system deficiencies (2030 and 2040 time horizons)

4.2 MODEL UPDATE

This section documents model updates since the 2014 WWSE, including geographic information system (GIS) updates to the gravity sewer network and as-built review of major lift stations and force mains. Additionally, flow loadings for existing conditions (2021) and future conditions (2030 and 2040) were updated in the model.

4.2.1 GIS UPDATES

The City provided an update to its GIS database for infrastructure that was constructed through 2021. Although the spatial attributes were provided, no field attributes were provided with the GIS information for recently constructed infrastructure. Based on a review of developments constructed between 2013 and 2021, new developments were predominantly sewer extensions from new subdivisions located along the extremities of the system. Additional adjustments to flow loadings were considered in the collection system evaluation, but the recent sewer extensions were not included in the model and were assumed to have available capacity. It is recommended that the recent sewer extensions be added to the model in the future once attribute information is added to the relevant GIS layers.

4.2.2 AS-BUILT UPDATES

As-built drawings were used to assign accurate piping, dimensions, valve, and gate configurations to the City's major lift stations for analysis in the model. Configurations were modified for the following stations:

- London Bridge
- Willow Wash
- Sweetwater
- Hagen
- Bombay
- Influent pump station (IPS)

As part of the as-built updates, vertical controls were assigned to force mains to identify vertical bends and pipe elevations along the force main. Prior to this update, these lift stations were included as point-to-point connections between the pump discharge and the downstream gravity sewer connection point. Vertical alignment updates were completed for the following force mains:

- London Bridge
- Sweetwater
- Bombay
- IPS

Additionally, the Tarpon downstream gravity sewer was added to the model using as-built drawings. Tarpon was completed in 2010 and runs from the intersection of Saratoga Avenue and Acoma Boulevard to just upstream of the Mulberry WWTP.

4.2.3 FLOW LOADING UPDATE

As discussed in Section 3.2 and shown in Figure 3-5, the 2014 WWSE projection at year 2024 is actually closer to the current wastewater master plan projection in 2040 because of slower than anticipated growth from 2015 to 2020 and lower wastewater flows due to water conservation. Figure 4-1 highlights areas of the system that have seen growth since the 2014 WWSE. Additional flow loadings were added to the existing model in these areas to achieve the current AAD (4.0 mgd).

Figure 4-2 shows the City's known planned developments that are likely to be built before 2040. This planned growth absorption corresponds to population estimates for 2040 with an estimated AAD of 4.6 mgd.

Figure 4-1. Growth Areas Since Previous Master Plan

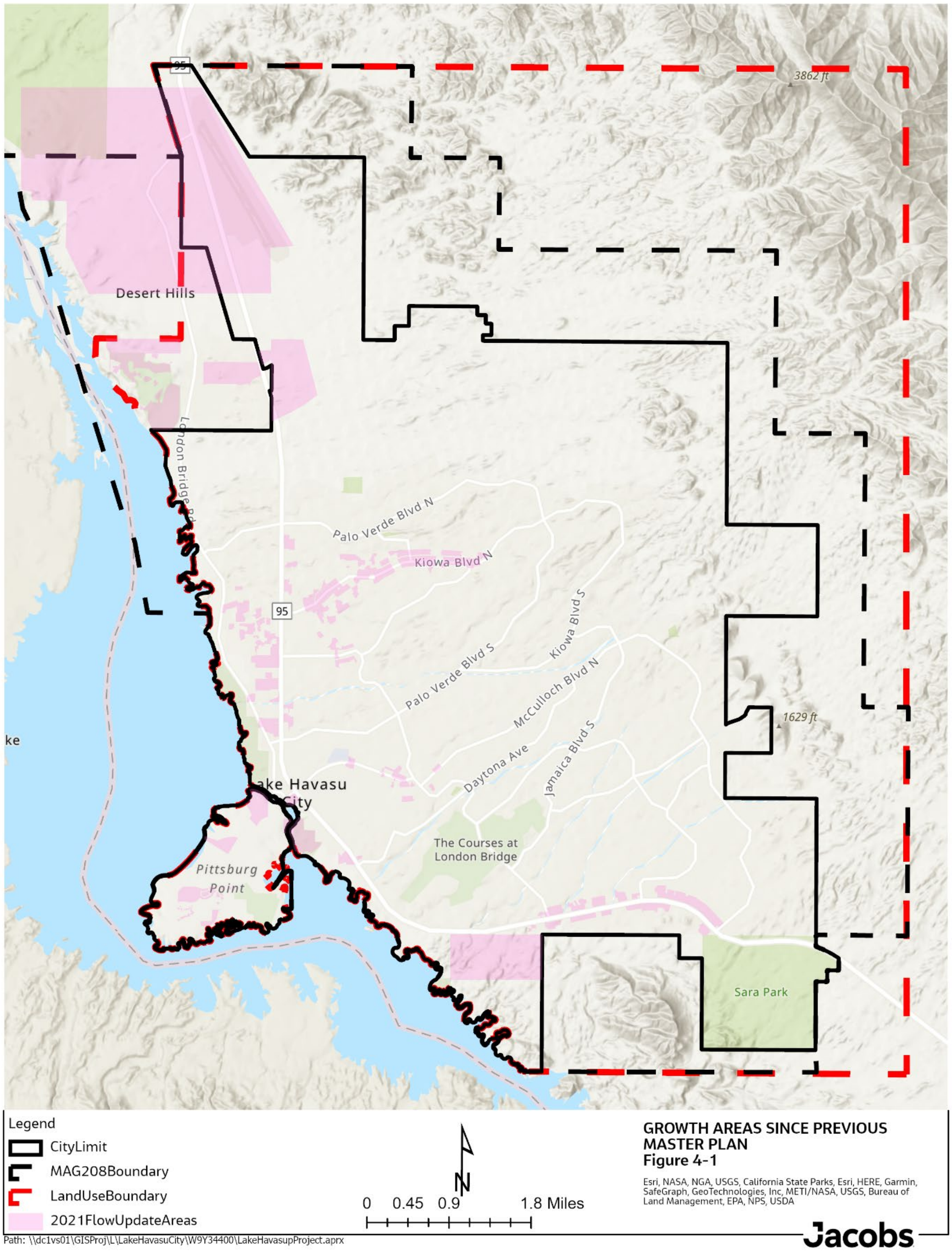
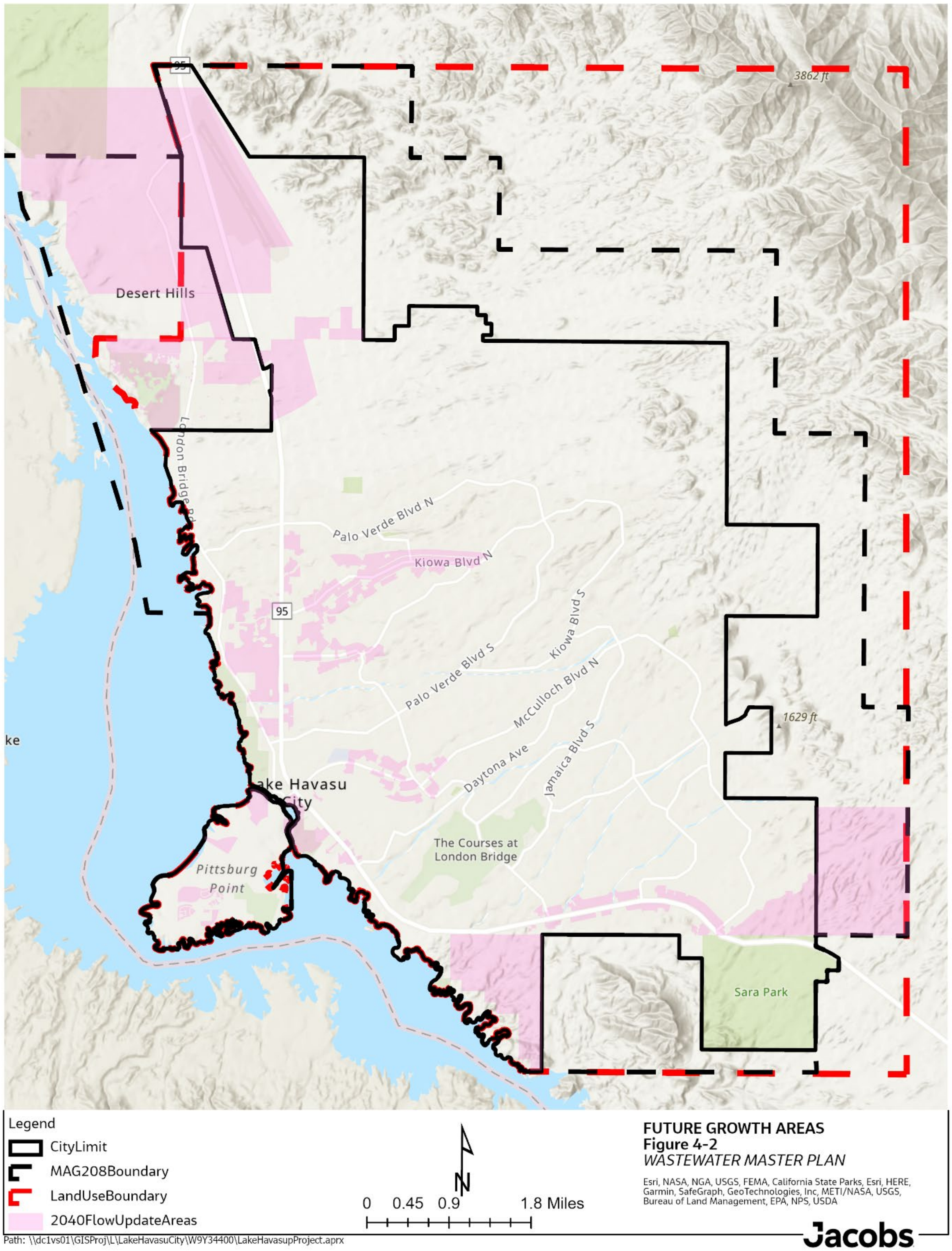


Figure 4-2. Future Growth Areas



4.3 MODEL VALIDATION

A model validation exercise was performed to confirm the collection system model was functioning as expected. Table 4-1 summarizes the model validation results by comparing observed SCADA flow data at major lift stations to the flow outputs generated in the model. March 2020 SCADA data was used in the validation as this was a period of high flow rates at the treatment plants and was representative of Max Day conditions.

Overall, there is a good correlation between the observed SCADA flow data and model generated data. Discrepancies in the peak instantaneous flows at London Bridge, Willow Wash, and Bombay were observed. This is due to 2-pump operation at these lift stations for short periods of time, but the collection system model was not able to replicate this brief change in operation.

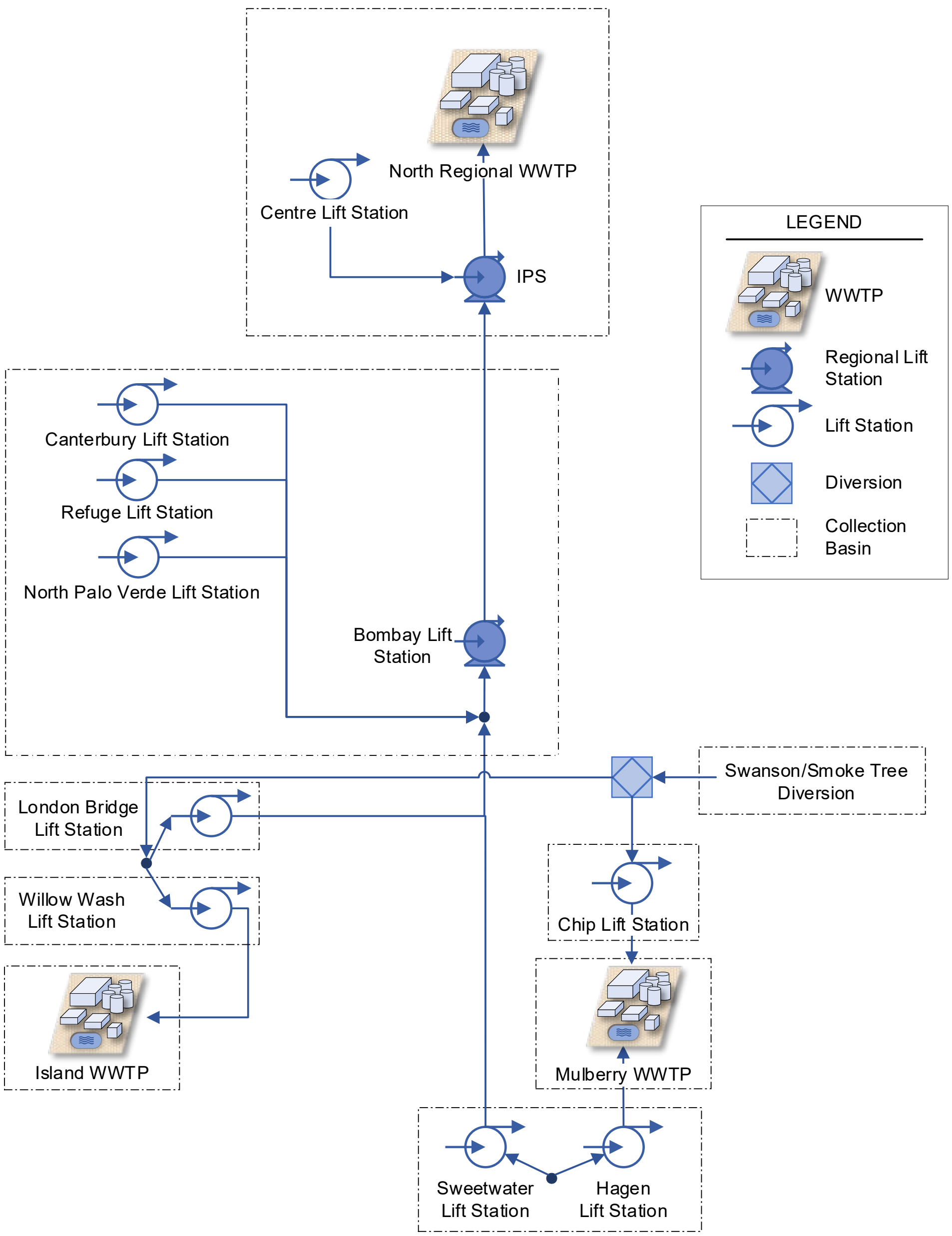
Table 4-1. Model Validation

	March 2020 SCADA Data (3/14 - 3/22)		Model Validation (2021 Max Day)		Difference	
Lift Station	Peak Instantaneous Flow (MGD)	Daily Flow (MGD)	Peak Instantaneous Flow (MGD)	Daily Flow (MGD)	Peak Instantaneous Flow (MGD)	Daily Flow (MGD)
IPS	8.9	2.2	9.0	2.8	0.1	0.6
Bombay	8.1	2.5	6.7	2.8	-1.4	0.3
London Bridge	6.5	1.0	3.3	1.2	-3.2	0.2
Willow Wash	3.4	0.9	0.6	0.6	-2.8	-0.3
Sweetwater	2.5	1.0	3.2	0.7	0.7	-0.3
Hagen	2.1	0.7	1.6	0.4	-0.5	-0.3

5. Wastewater System Evaluation

This section provides a description of the system evaluation performed as part of this Wastewater Master Plan. The system evaluation includes: a capacity analysis of the wastewater system, an optimization assessment of the regional pumping system, and identification of future infrastructure expansion. For reference, Figure 5-1 presents a schematic of the wastewater system highlighting the wastewater treatment plants, and major lift stations, and flow diversions.

Figure 5-1. Wastewater System Schematic



5.1 CAPACITY ANALYSIS

The City's sewer collection system was designed to promote flexibility to convey flows as needed to the three WWTPs. With this flexibility in mind, the collection system was evaluated under the "worst case" flow condition to analyze the existing collection system's capacity. The worst case was identified as the 2040 flow conditions for maximum daily flows and operating the system to maximize flows to the North Regional WWTP. The operational assumptions for this scenario included the Hagen Lift Station not operating, the Swanson/Smoke Tree Diversion closed to the Mulberry WWTP, and the Willow Wash Lift Station not operating. The worst-case model simulation identified no significant capacity deficiencies.

5.2 REGIONAL PUMPING SYSTEM OPTIMIZATION ASSESSMENT

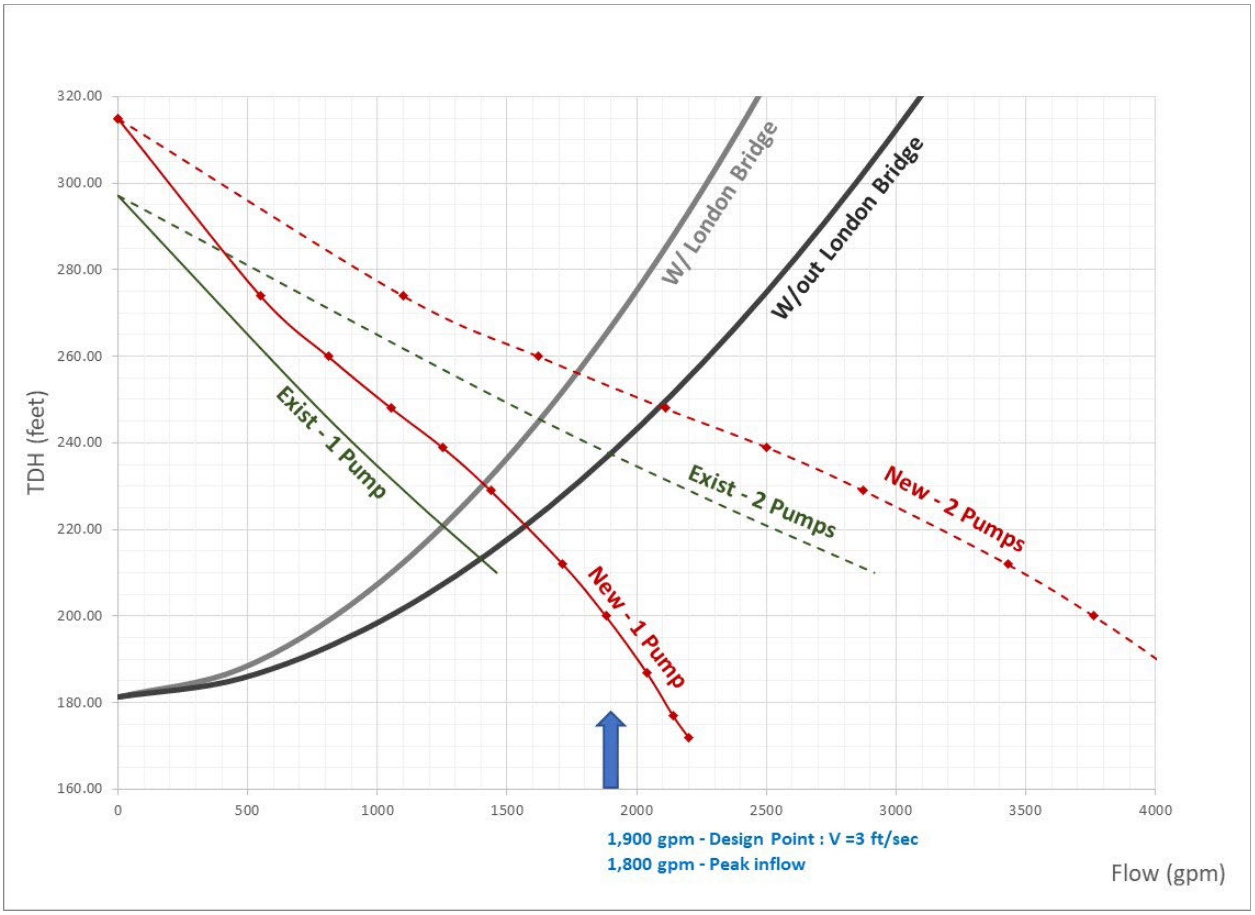
The North Regional WWTP and regional pumping system was originally planned to convey and treat approximately 14 mgd of average daily flow. Based on the current wastewater projections, in 2040, the City will have a total of 4.6 mgd of average daily flow. With such a large discrepancy in flow projections, the regional pumping system was evaluated in detail to identify opportunities to optimize the system. The regional pumping system primarily includes the following lift stations: Sweetwater, London Bridge, Bombay, and IPS, which conveys flows to the North Regional WWTP.

For each lift station, system performance curves were developed and the existing pump operations were evaluated. The following sections provide a summary of each lift station's current operations and optimization recommendations.

5.2.1 SWEETWATER LIFT STATION

The Sweetwater Lift Station pumps flows from the south of the City to the Bombay Lift Station. The Sweetwater Lift Station force main is approximately 11,000 feet of 16-inch-diameter C-900 PVC pipe that joins with the London Bridge force main near Highway 95 and Mesquite Avenue. The combined force main is 24 inches in diameter and extends approximately 11,500 feet and connects to the Bombay Lift Station. Figure 5-2 presents the system performance curve for the Sweetwater Lift Station.

Figure 5-2. Sweetwater Lift Station System Performance Curve



gpm = gallon(s) per minute
TDH = total dynamic head

Based on an evaluation using the hydraulic model and the system performance curve, the existing pumps were designed to operate with two pumps running at a time to meet the peak inflow and maintain cleansing velocities in the force main(s). Hydraulic model results identified that at no time were two pumps operating together and indicated that existing single-pump operation was occurring at the far right of the pump curve. Operating pumps at the far right of their pump curves results in shortened life cycles and is not recommended as a long-term strategy.

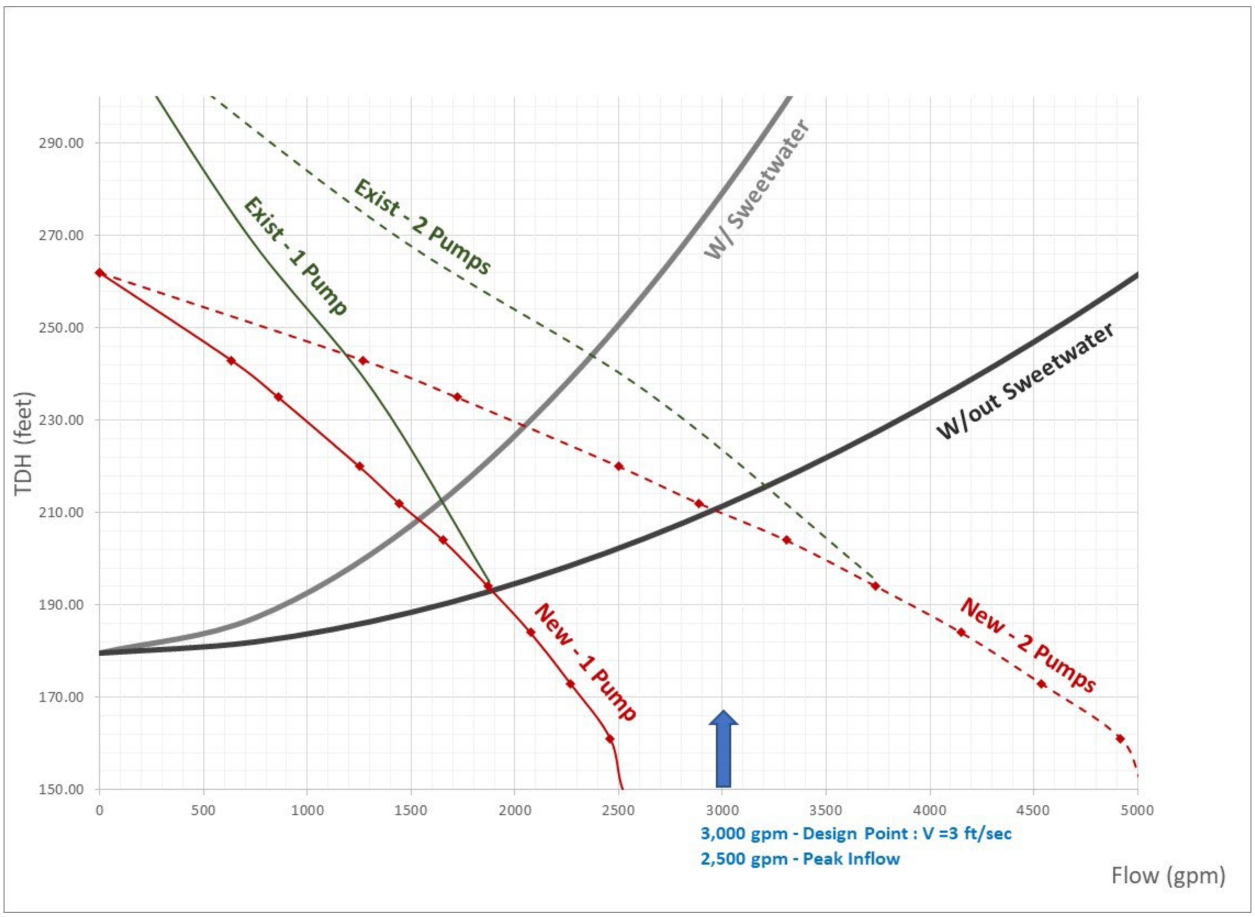
As part of the optimization evaluation, evaluation of different pump design strategies was conducted using the system performance curve and pump selection tools for submersible solids handling pumps. The evaluation identified that a slightly larger pump may have significant benefits for the system, including maintaining pump operations within the manufacturer's operating range under single- and dual-pump operation, and increasing force main velocities for single-pump operations. The new pump identified in the system performance curve is a Fairbanks Nijhuis 4-inch 5434 MV operating with a 16.00-inch impeller at 1,785 revolutions per minute (rpm) with 150-horsepower (hp) motors.

It is recommended that the pump replacement be further evaluated in a predesign report that evaluates impacts on the station from the larger pumps as well as additional operational upgrades, if identified. Because the pumps are operating, the recommended pump replacements are a low priority. If the City begins to experience frequent pump failures at the station, the recommended pump replacement and station upgrades should become a higher priority.

5.2.2 LONDON BRIDGE LIFT STATION

The London Bridge Lift Station pumps flows from a large portion of the center of the City to the Bombay Lift Station. The London Bridge Lift Station force main is approximately 2,000 feet of 20 inches in diameter ductile iron pipe that joins with the Sweetwater force main near Highway 95 and Mesquite Avenue. The combined force main is 24 inches in diameter and extends approximately 11,500 feet and connects to the Bombay Lift Station. Figure 5-3 presents the system performance curve for the London Bridge Lift Station.

Figure 5-3. London Bridge Lift Station System Performance Curve



Based on an evaluation of the hydraulic model and the system performance curve, the existing pumps were designed to operate with two pumps running at one time to meet the peak inflow and maintain cleansing velocities in the force main(s). Hydraulic model results identified that at no time were two pumps operating together and indicated that the existing single-pump operation was occurring at the far right of the pump curve. Operating pumps at the far right of their curves results in shortened life cycles and is not recommended as a long-term strategy.

As part of the optimization evaluation, an evaluation of different pump design strategies was conducted using the system performance curve and pump selection tools for submersible solids handling pumps. The evaluation identified that a slightly different pump may have significant benefits for system by maintaining pump operations within the manufacturer's operating range under single- and dual-pump operation. The new pump identified in the system performance curve is Fairbanks Nijhuis 5-inch 5436 W MT WD operating with a 14.98-inch impeller at 1,750 rpm with 150-hp motors.

It is recommended that the pump replacement be further evaluated in a predesign report that evaluates impacts on the station from the different pumps as well as additional operational upgrades, if identified. Because the pumps are operating, the recommended replacement is a low priority. If the City begins to experience routine pump failures at the station, the recommended pump replacement and station upgrades should become a higher priority.

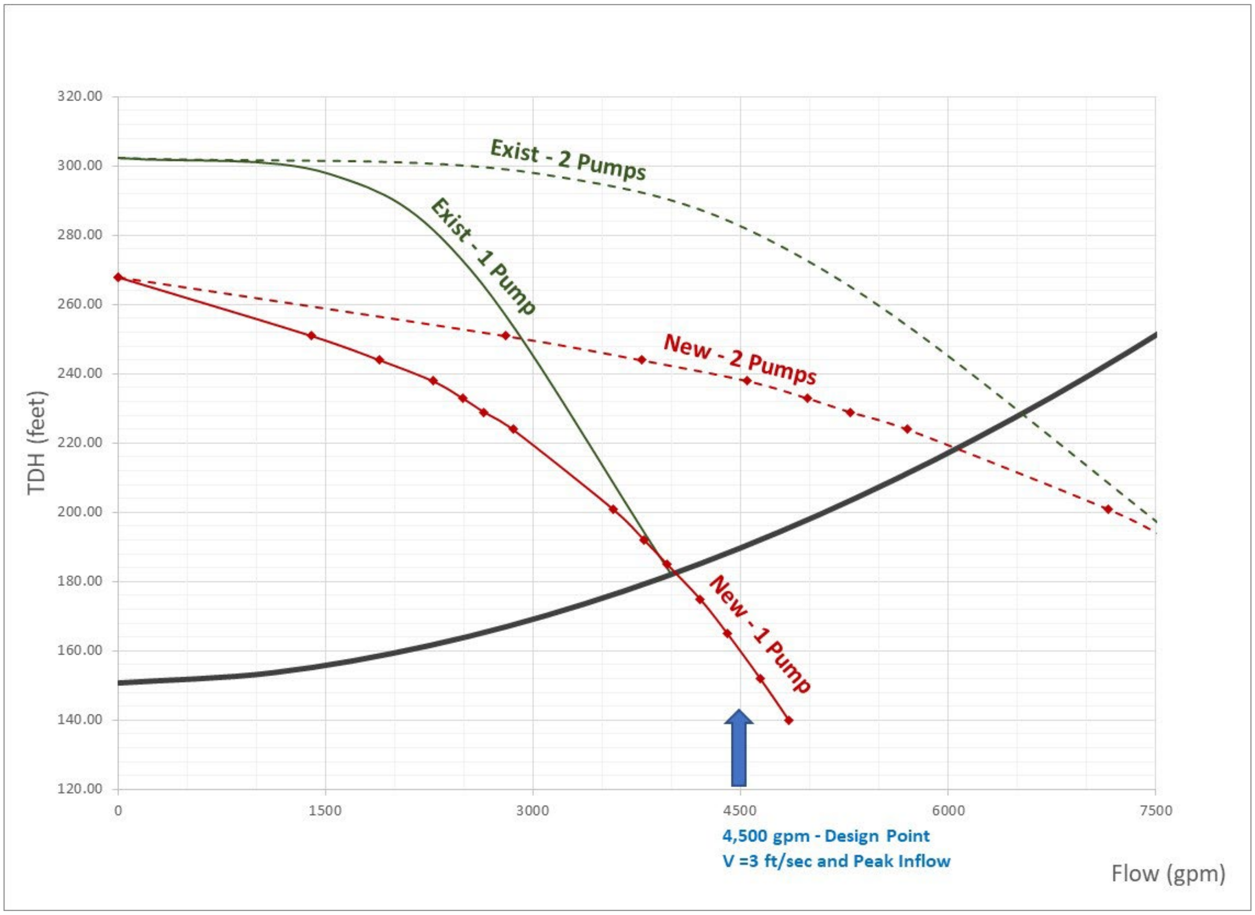
5.2.3 BOMBAY LIFT STATION

The Bombay Lift Station pumps flows from the Sweetwater and London Bridge Lift Stations and the northern portions of the City to the IPS. The Bombay Lift Station force main is 24-inches in diameter and extends for approximately 15,000 feet. Figure 5-4 presents the system performance curve for the Bombay Lift Station.

Based on an evaluation using the hydraulic model and the system performance curve, the existing pumps were designed to operate with two pumps running at one time to meet a much larger anticipated peak inflow. Hydraulic model results identified that at no time were two pumps operating together and indicated that existing single-pump operation was occurring at the far right of the pump curve. Operating pumps at the far right of their curves results in shortened life cycles and is not recommended as a long-term strategy.

As part of the optimization evaluation, evaluation of different pump design strategies was conducted using the system performance curve and pump selection tools for submersible solids handling pumps. The evaluation identified that a smaller pump may have significant benefits for the system by maintaining pump operations within the manufacturer's operating range under single- and dual-pump operation. The new pump identified in the system performance curve is a Fairbanks Nijhuis 6-inch 5436 MV operating with a 15.82-inch impeller at 1,785 rpm with 250-hp motors. The selected pump will operate slightly below the design point and will require occasional two-pump operation. This operational strategy is preferred over the use of a larger pump.

Figure 5-4. Bombay Lift Station System Performance Curve



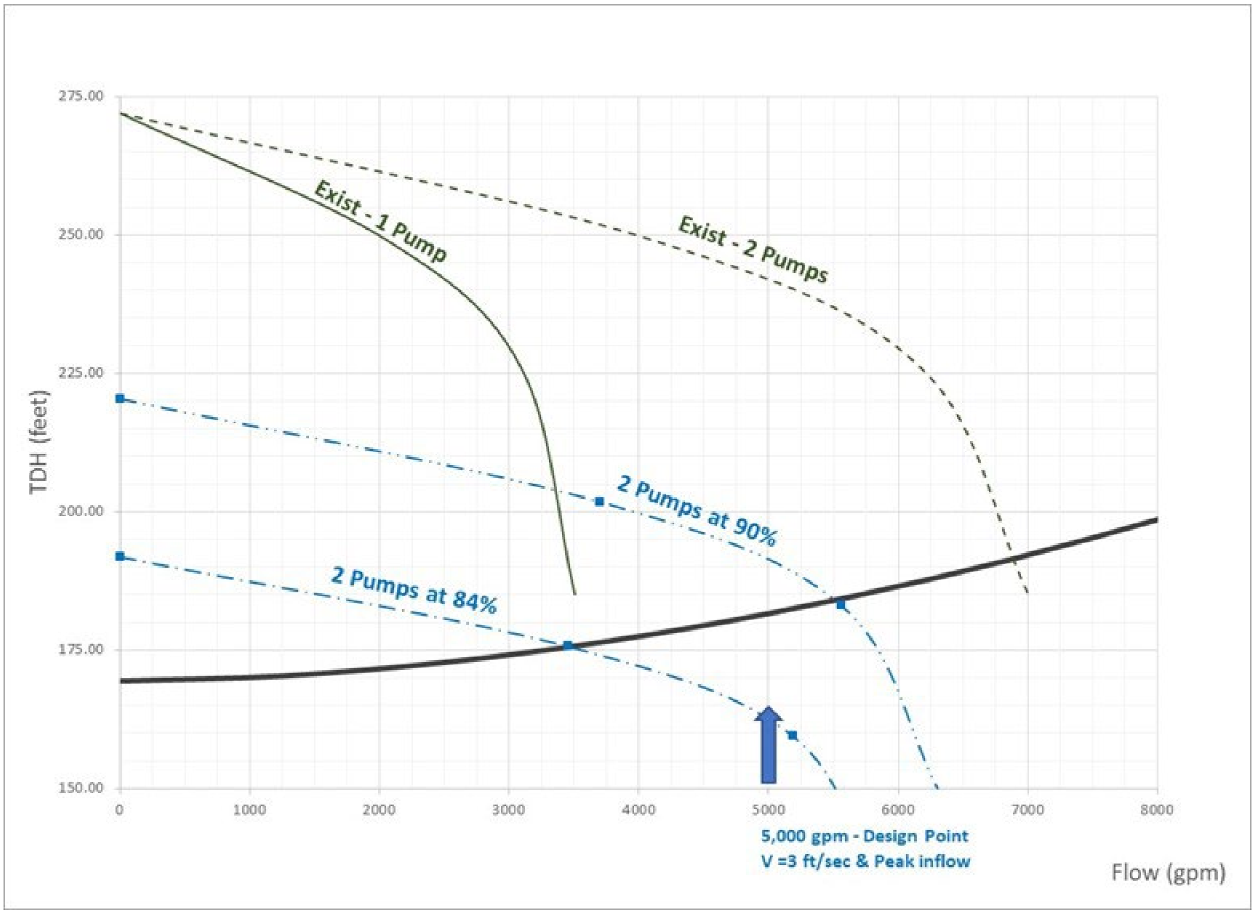
It is recommended that the pump replacement be further evaluated in a predesign report that evaluates impacts on the station from smaller pumps as well as additional operational upgrades, if identified. City operations staff have identified significant challenges with "ragging" at the Bombay Lift Station as shown on Figure 5-5. To mitigate the ragging, City staff clean the wet well weekly and every 6 weeks by draining the wet well to the bottom and removing all material. The ragging issue presents a major risk for pump failure and sanitary sewer overflows. The predesign report should include an evaluation of installing a bar screen or trash rake system at the station to protect pump operations to provide screening for the pump station. In addition, the site should be enclosed by fencing or block wall. Because of the existing pump operation and challenges with ragging, the recommended improvements at the station are high priority.

Figure 5-5. Rag Ball Being Removed from the Bombay Pump Station

5.2.4 INFLUENT PUMP STATION

The IPS pumps flows from the Bombay Lift Station to the North Regional WWTP. The IPS force main is 24 inches in diameter and extends approximately 7,000 feet. Based on site reconnaissance and discussions with operations staff, significant transient or "surge" pressures are persistent in the force main and have caused pipe breaks in the ductile iron pipeline near the lift station. The City is adding surge mitigation measures at the station, including implementing variable frequency drives (VFDs) and installing a bladder tank. Figure 5-6 presents the system performance curve for the IPS Lift Station with the existing pumps operating with VFDs.

Figure 5-6. IPS Lift Station System Performance Curve



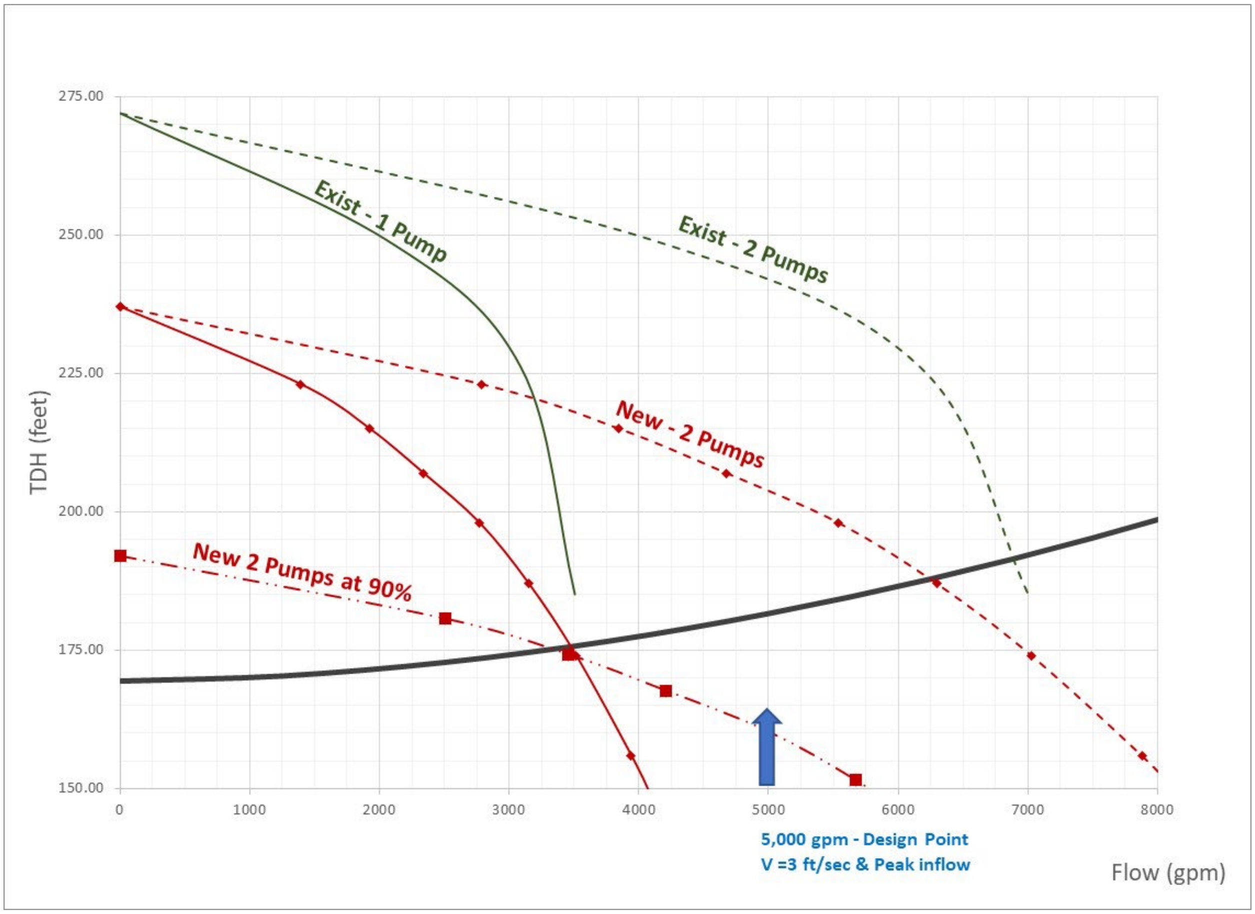
Based on an evaluation of the hydraulic model and the system performance curve, the existing pumps were designed to operate with two pumps running at one time to meet a much larger anticipated peak inflow. Hydraulic model results identified that for only a short period were two pumps operating together and indicated that existing single-pump operation was occurring at the far right of the pump curve. Operating pumps at the far right of their curves results in shortened life cycles and is not recommended as a long-term strategy.

Evaluating the existing pumps with the use of VFDs would allow for two pumps to operate between 3,500 gpm at 84 percent speed to 5,500 gpm at 90 percent speed within the pumps' normal operating ranges. This would cover the estimated peak inflow and allow for more continuous operations with force main velocities ranging from approximately 2.5 to 3.5 ft/sec.

As part of the optimization evaluation, Jacobs evaluated different pump design strategies using the system performance curve and pump selection tools for submersible solids handling pumps. The evaluation identified that a smaller pump may have significant benefits because it could cover similar operational ranges with two pumps and allow for single-pump operation as well. The new pump identified in the system performance curve is a Fairbanks Nijhuis 6-inch 5436 L operating with a

15.02-inch impeller at 1,785 rpm with 250-hp motors. Figure 5-7 presents the system performance curve for the IPS Lift Station with the existing pumps operating with VFDs.

Figure 5-7. IPS Lift Station System Performance Curve with New Pumps



The proposed pump operations would include single-pump operation at approximately 3,500 gpm to handle low flows. When the level is triggered for the second pump, the VFDs would ramp down to 90 percent speed for both pumps and cover an operational range between 3,500 and 6,000 gpm.

It is recommended that the pump replacement be further evaluated in a predesign report that includes the surge improvements. Because of the existing pump operation and surge issues, the recommended improvements at the station are high priority.

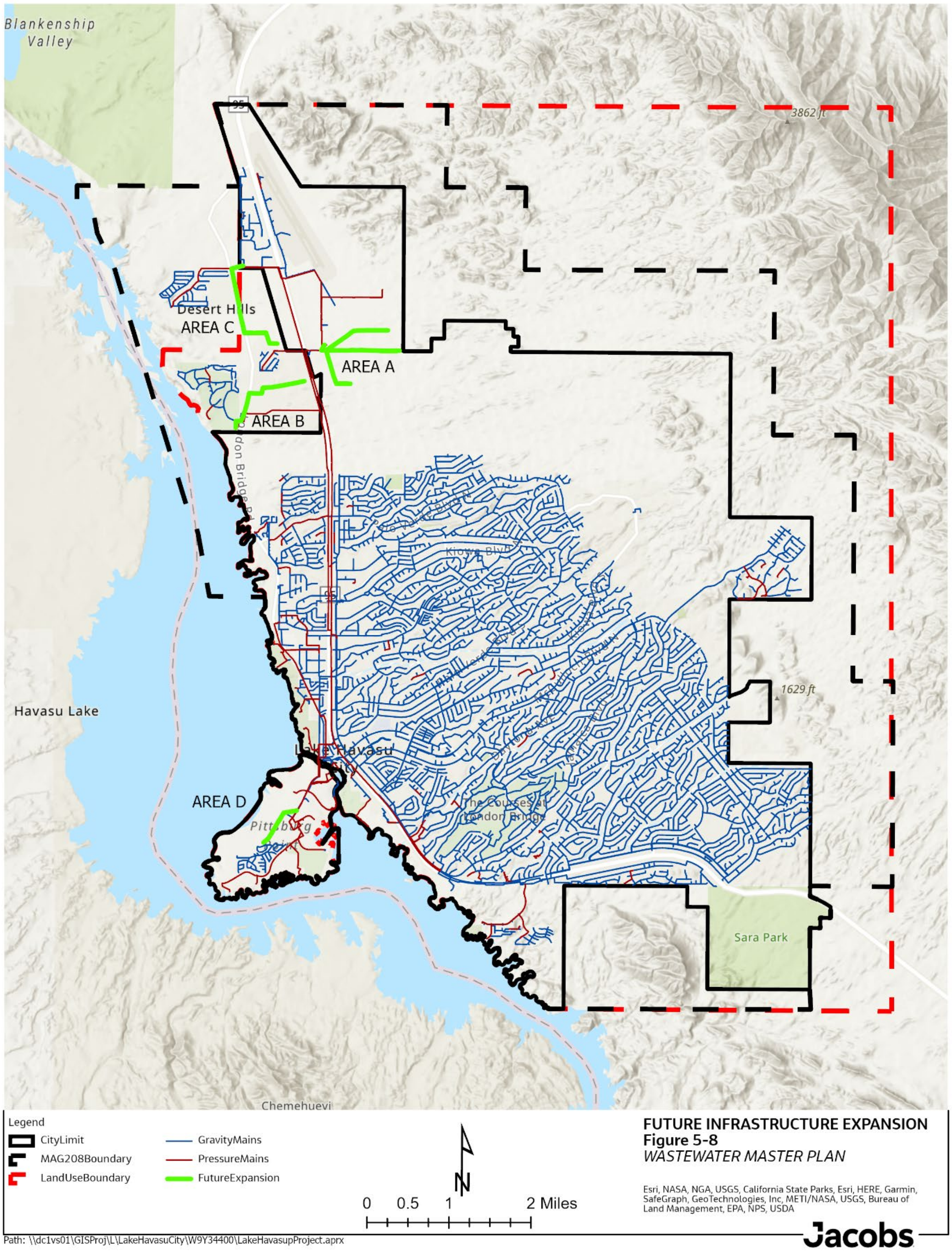
5.3 FUTURE INFRASTRUCTURE EXPANSION

Future gravity mains were added to the hydraulic model to route the flows from the new growth areas into the existing collection system where required. These sewers were located using ground slope information derived from U.S. Geological Survey elevation contours and are intended to provide the general feasibility of collection system routing alternatives (gravity sewers versus lift stations). The actual location of future sewers will be based on future development design plans that are subject to City approval. Future sewers were added in the following areas as shown on Figure 5-8:

- **Area A** – Gravity sewers east of Highway 95 south and east of the IPS
- **Area B** – Gravity sewers west of Highway 95 and east of the Refuge Development
- **Area C** – Gravity sewers and a lift station west of Highway 95 and southwest of the airport
- **Area D** – Gravity sewers and a new lift station and force main for the Island WWTP, southwest of the Island WWTP

Development activity is occurring in Area A. A detailed assessment of this area was conducted to accommodate the Victoria Farms Development and maximize gravity flows in the area. The findings of the assessment are presented in as Appendix F, North Regional Sub-Area Master Plan Technical Memorandum.

Figure 5-8. Future Infrastructure Expansion



6. Treatment Plant Assessment

This section provides an assessment of the current operations and capacity of each of the City's three WWTPs, as well as plans for future plant staffing and biosolids management.

6.1 INFLUENT FLOWS AND LOADS ANALYSIS AND PROJECTIONS

This section presents the analysis of historical influent loads for all three WWTPs. Peaking factors for the MM condition were analyzed for the historical years (2015 to 2021) and were used to develop the load projections. The influent load projections were developed assuming the same wastewater flow split used currently between the three plants will be used for 2040 conditions. The anticipated loads may change if the flow split between the WWTPs is modified.

The capacity of physical processes at the WWTPs, such as headworks or disinfection, are rated based upon hydraulic capacity. This assessment assumes peak hourly flow as the design criteria for physical processes. Biological processes, such as secondary treatment, are rated based upon loading capacity (carbon or nutrients). This assessment assumes MM loads as the design criteria for biological processes.

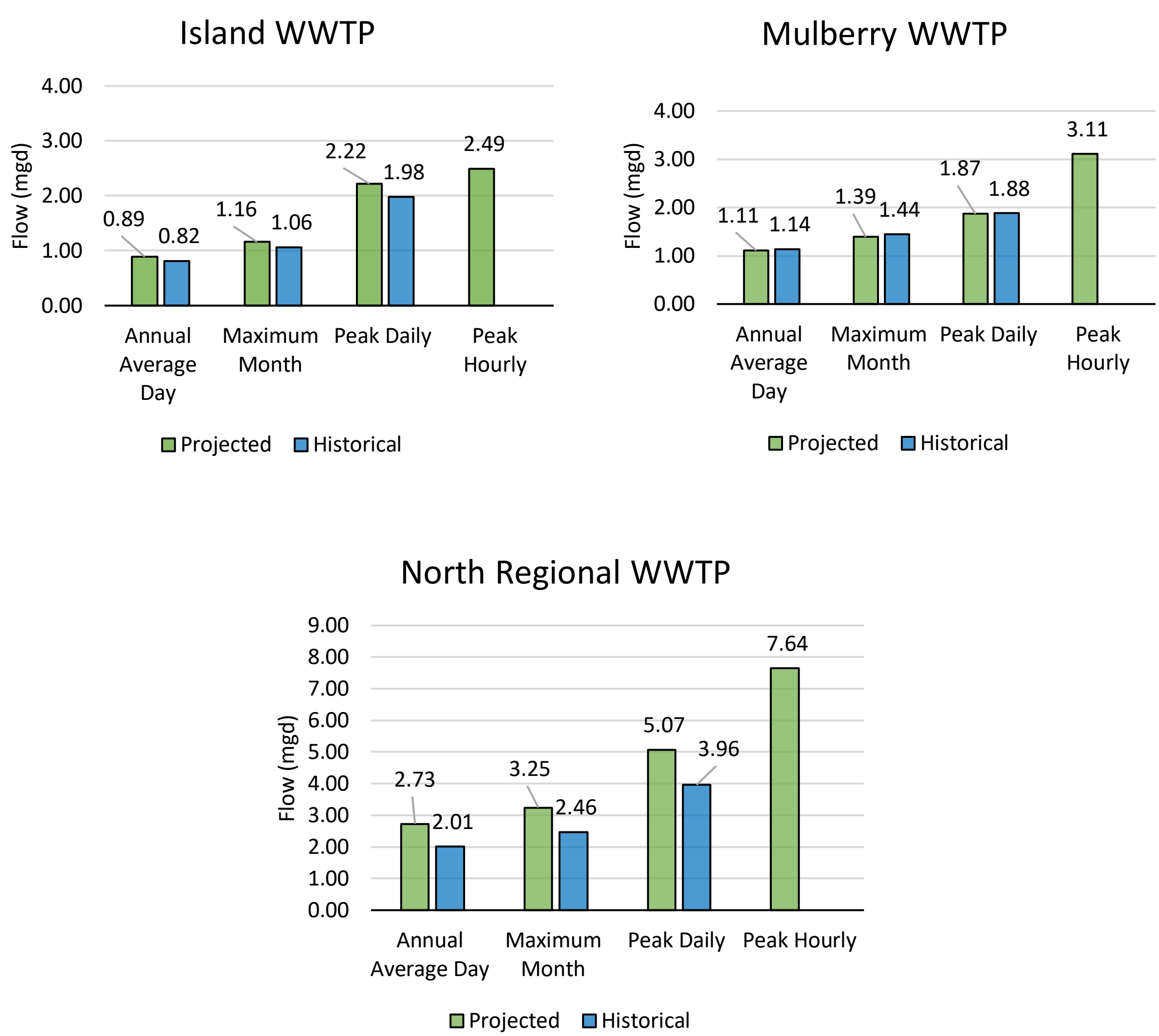
6.1.1 PROJECTED FLOWS

The 2040 projected wastewater flows for the entire service region was developed based on the analysis conducted on the historical wastewater flows presented in Section 3 and the historical peaking factors for the MM and peak daily conditions. The 2040 annual average day was multiplied by the 95th percentile peaking factors to calculate the maximum month and peak daily conditions to estimate the MM flow and PD flow for 2040 for the three WWTPs. The peak hourly flows were determined by multiplying a peaking factor of 2.8 with the 2040 annual average day (Metcalf & Eddy, AECOM 2014). Detailed information about how the flow projections were developed is in Appendix A.

Figure 6-1 presents the flow projections for each WWTP. It is observed that Island WWTP service is anticipated to grow only marginally and therefore there is not much difference between the historical versus the projected flows. No growth is anticipated in the Mulberry WWTP service area and the same is reflected in the 2040 projected values for different flow conditions. Growth is anticipated in the North Regional WWTP service area and the same is reflected in the 2040 projected values when compared to the historical data.

The analysis assumed the current wastewater flow split between the three WWTPs would be applicable in 2040. The City has the ability to divert more or less flow to each WWTP. If the flow split ratio changes, then the 2040 projected flows will need to be updated accordingly. It is recommended that the City continue to monitor the different flow conditions to the three WWTPs and accordingly update the flow projections during the next master plan update.

Figure 6-1. 2040 Influent Flow Projections for the City's WWTPs (Historical Data are from 2015–2021)

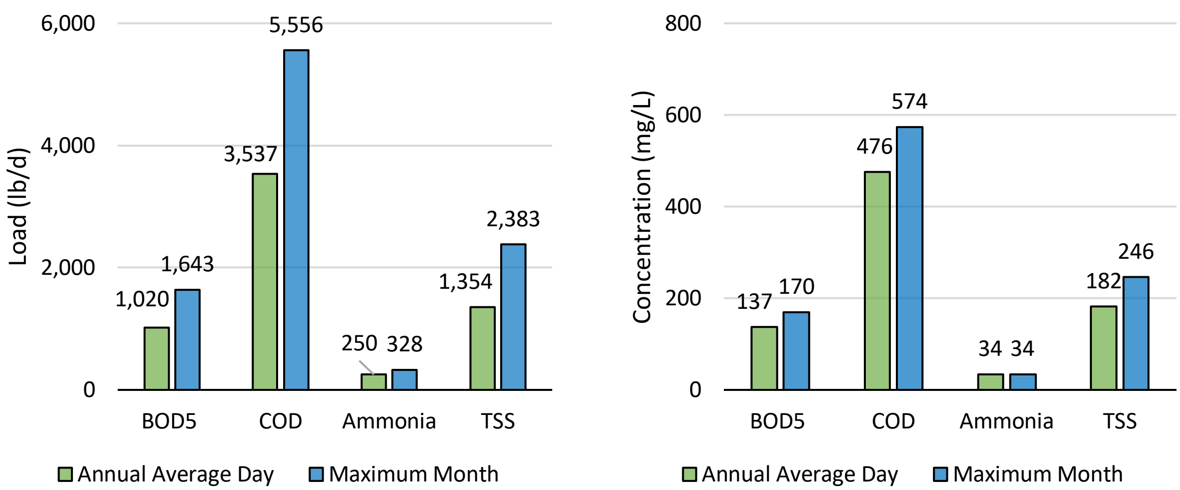


6.1.2 PROJECTED LOADS TO ISLAND WWTP

Projected loads for the Island WWTP were calculated based on the historical loads and peaking factors. A detailed description of load projections can be found in Appendix B. The 2040 projected loads for the average annual day and MM conditions are presented on Figure 6-2. The 2009 Master Plan Update by AMEC Earth and Environmental noted that the flows and 5-day biochemical oxygen demand (BOD_5) concentrations to the Island WWTP had decreased since the startup of the North Regional WWTP. BOD_5 is used to measure the amount of organic material present in the wastewater. This decrease may have resulted from the diversion of high-strength wastewater generated in the main parts of the City to other WWTPs. Island WWTP now serves the resort and beach communities at Lake Havasu, which typically generates wastewater with lower organic material. The chemical oxygen demand (COD)/BOD_5 ratio is much higher than the typical values of 1.8 to 2.2 (according to the Water Environment

Federation [WEF] Manual of Practice [MOP] Number 8), indicating that some of the BOD_5 may be consumed in the sewer collection systems because of low velocities and higher detention times. A certain amount of organic material is needed for the proper functioning of the WWTP, especially the secondary treatment process. Secondary treatment uses microorganisms to treat and remove wastewater constituents such as nitrogen and phosphorus. The organic material or BOD_5 serves as a food source for the microorganisms. The remaining constituents fall within the acceptable ranges (WEF MOP 8).

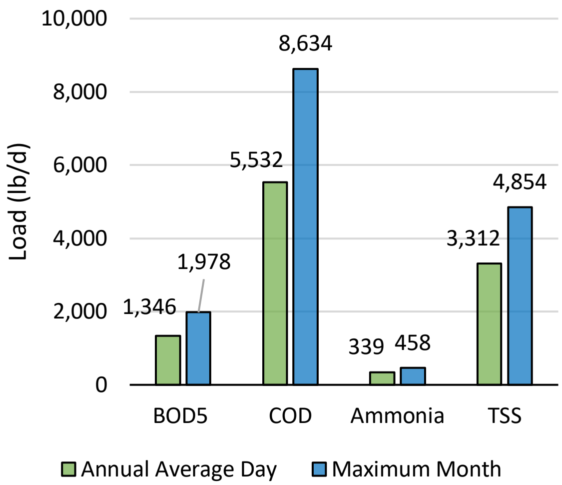
Figure 6-2. Influent Load Projections for Island WWTP in 2040

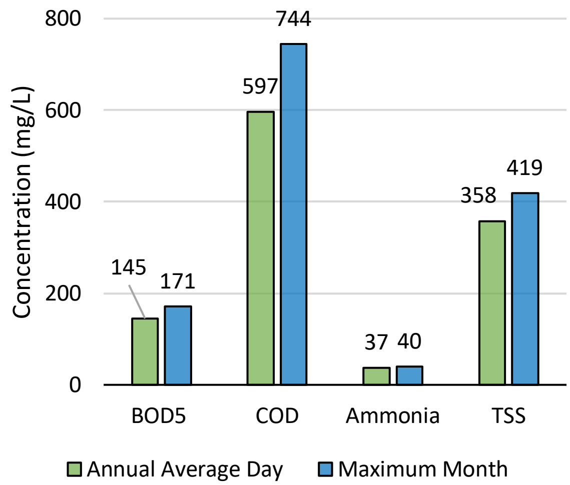


6.1.3 PROJECTED LOADS TO MULBERRY WWTP

Projected loads for the Mulberry WWTP were calculated based on the historical loads. A detailed description of load projections can be found in Appendix B. The 2040 projected loads for the average annual day and MM conditions are presented on Figure 6-3. Mulberry WWTP receives more residential and commercial wastewater flows than Island WWTP. Higher BOD_5 loads are observed in the influent but the concentration is quite similar to Island WWTP's. This indicates that Mulberry WWTP may also be subject to the same flow aging issues as Island WWTP. The COD/ BOD_5 ratio is much higher than the typical values of 1.8 to 2.2 (WEF MOP 8), indicating that some of the BOD_5 may be consumed in the sewer collection systems because of low velocities and high detention times.

Figure 6-3. Influent Load Projections for Mulberry WWTP in 2040

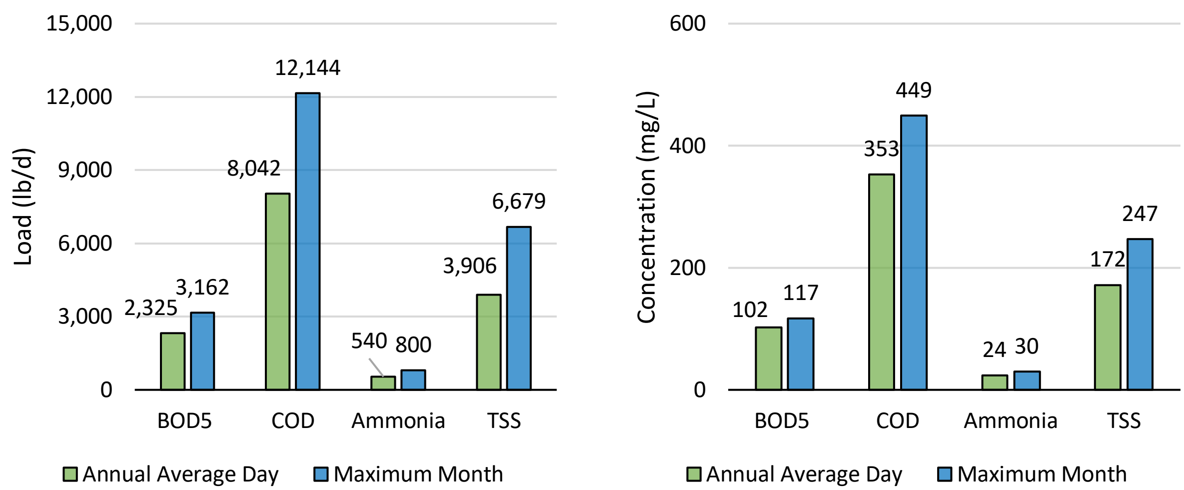




6.1.4 PROJECTED LOADS TO NORTH REGIONAL WWTP

Projected loads for the North Regional WWTP were calculated based on the historical loads. A detailed description of load projections can be found in Appendix B. The 2040 projected loads for the average annual day and MM conditions are presented on Figure 6-4. Very low BOD_5 concentrations are observed in the wastewater going to North Regional WWTP. Wastewater from the City is conveyed to North Regional WWTP over a stretch of 3 to 4 miles of sewer pipe and force mains. Given the low water velocities, it can potentially create conditions suitable for the wastewater to become septic and start off-gassing hydrogen sulfide. The City adds Alkagen to reduce the occurrence of such conditions. However, the chemical added destroys a portion of the BOD_5, which may be needed by the WWTP for its operation.

Figure 6-4. Influent Load Projections for North Regional WWTP in 2040



6.2 EFFLUENT CRITERIA

All of the WWTPs operate under an amendment to the APP issued by the Arizona Department of Environmental Quality (ADEQ). The effluent generated after treatment is permitted only for beneficial reuse under a reclaimed water permit or recharged to groundwater at one or more facilities connected through the Lake Havasu City Recharge System. Because the effluent generated at the treatment plants will be reclaimed via irrigation, via aquifer recharge by the percolation ponds at the Island WWTP, or via recharge by the vadose wells at the North Regional WWTP, Class A+ Reclaimed Water Quality Standards are applicable. The permits prohibit any discharges into the surrounding arroyos or water bodies. All facilities meet the Best Available Demonstrated Control Technology per the Arizona Environmental Quality Act (Arizona Revised Statutes Section 49-243.B.1). Table 6-1 presents the permitted capacity of each facility and the applicable water quality standards.

Table 6-1. Effluent Requirements for the Treatment Facilities

Parameter	Island Treatment Plant	Mulberry Treatment Plant	North Regional Treatment Plant
Permit Number	P-101611	P-101612	P- 105478
Permitted Design Flow, mgd	2.5	2.2	3.5
BOD_5/$cBOD_5$, mg/L	Not reported	Not reported	Not reported
TSS, mg/L	Not reported	Not reported	Not reported
pH	Not reported	Not reported	Not reported
Total Nitrogen 5-Sample Geometric Mean, mg N/L	< 10	< 10	< 10
Nitrate, mg N/L	Not reported	Not reported	Not reported
Nitrite, mg N/L	Not reported	Not reported	Not reported

Table 6-1. Effluent Requirements for the Treatment Facilities

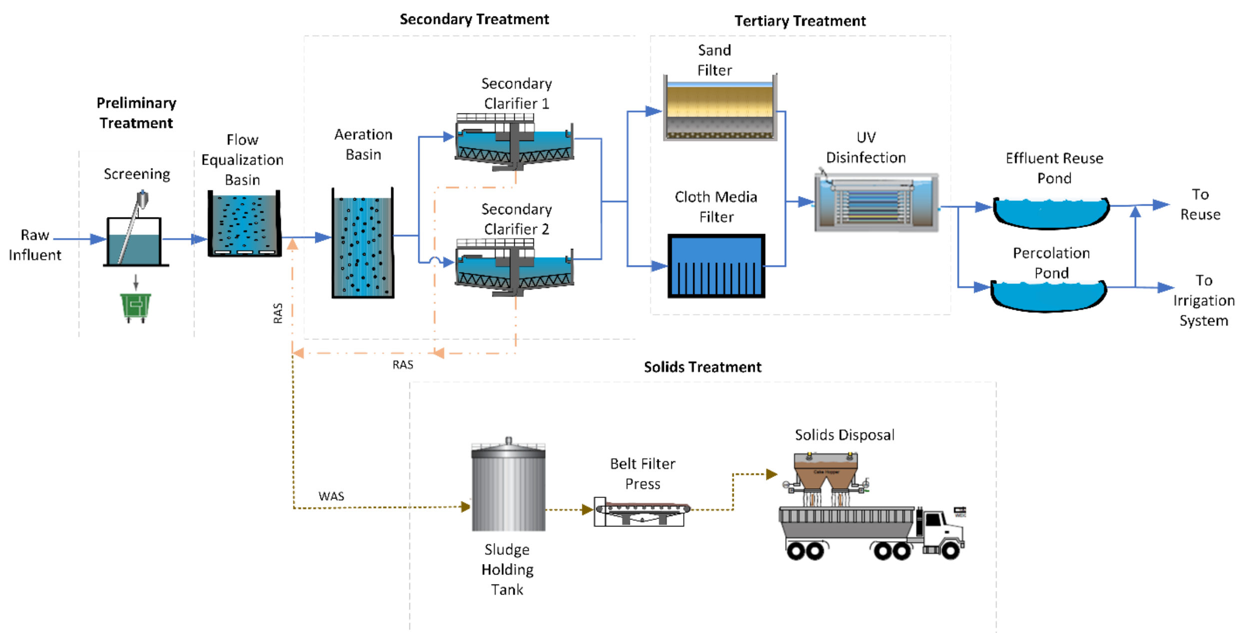
Parameter	Island Treatment Plant	Mulberry Treatment Plant	North Regional Treatment Plant
Nitrate and Nitrogen, mg N/L	Not reported	Not reported	Not reported
Phosphorus, mg/L	Not reported	Not reported	Not reported
Turbidity, NTU	< 2.0 (24-hour mean) < 5.0 (single sample maximum)	< 2.0 (24-hour mean) < 5.0 (single sample maximum)	< 2.0 (24-hour mean) < 5.0 (single sample maximum)
Disinfection, per 100 mL	None detected in 4 of 7 consecutive samples < 23 units per 100 mL (single sample maximum)	None detected in 4 of 7 consecutive samples < 23 units per 100 mL (single sample maximum)	None detected in 4 of 7 consecutive samples < 23 units per 100 mL (single sample maximum)

$cBOD_5$ = 5-day carbonaceous (nitrification inhibited) biochemical oxygen demand
mg = milligram(s)
mg N/L = milligram(s) per liter (measured as nitrogen)
mg/L = milligram(s) per liter
mL = milliliter(s)
NTU = nephelometric turbidity unit
TSS = total suspended solids

6.3 ISLAND WWTP

The Island WWTP is the City's oldest WWTP with a permitted flow of 2.5 mgd. Figure 6-5 shows the general process schematic for the WWTP. Influent wastewater is treated using a combination of screening, flow equalization (under construction), biological treatment with activated sludge processes, tertiary filtration and ultraviolet (UV) disinfection. The effluent produced meets the requirements of Class A+ Reclaimed water set by ADEQ and is either used for irrigating the golf course or discharged into the groundwater through percolation ponds. The solids are generated as a byproduct of the biological treatment and are sent to the sludge holding tank for storage and then dewatered before being disposed of in the City's landfill.

Figure 6-5. Island WWTP General Process Schematic



A detailed evaluation of each Island WWTP main unit process is described in Appendix C. Each section in Appendix C discusses the current unit processes and any operational problems or opportunities for optimization and, provides an analysis of the WWTP's capacity to treat flows and/or loads through the planning period and recommended modifications. The results and recommendations from this analysis for each unit process are summarized in the following sections.

6.3.1 PRELIMINARY TREATMENT

- The screens are estimated to have sufficient capacity to handle the 2040 peak hourly projections.
- The frames of the static screens showed signs of corrosion and will need to be replaced. Wire mesh sections were recently replaced and are in good condition. The screens will reach the end of their useful life in the next 5 years and will need to be replaced completely.
- The Headworks Building needs additional structural repairs to avoid further deterioration. A portion of the structure is supported by external wing walls, and cracking and settlement are observed in the walls as shown on Figure 6-6.
- It is recommended that the City improve ventilation to protect the electrical gear and wiring from further hydrogen sulfide corrosion.
- Jacobs recommends including a grit removal system in future Headworks Building upgrades. Common industry practice is for grit removal to precede secondary treatment in those treatment plants that do not have primary clarification. Removal of grit prevents unnecessary abrasion and wear of mechanical equipment, grit deposition in pipelines and channels, and accumulation of grit in the flow equalization basin (FEB), aeration basins, and sludge holding tanks.

- Recent upgrades have altered the water level in the FEB and affected upstream hydraulics in the Headworks structure, resulting in decreased influent screening capacity. The Headworks Building requires screens with higher elevations to restore the previous capacity.
- Because of the extensive upgrades and repairs needed, it is recommended that the City construct a new Headworks Building, which will include the following:
  - New influent channels
  - New multi-rake bar screens
  - Grit removal
  - Odor control connected to the new FEB odor control system
  - Electrical improvements
- The new facility would also allow the WWTP to fully use the FEB volume available for flow equalization. The City expressed a preference to locate the new Headworks Building immediately to the north of the existing Headworks Building, occupying the location of the current chemical storage structure, which will be demolished. The existing headworks structure will become a storage building. The new facility, with new screens, needs to be online by 2027, when the existing screens will have reached their useful life. Jacobs recommends that the City determine the exact location, components, and configuration of the new Headworks Building during the detailed design phase.

Figure 6-6. Cracks Observed in the South Wall of the Headworks Building

6.3.2 Flow Equalization Basin

The new FEB, when complete, is anticipated to have sufficient capacity to handle the 2040 peak day flows.

6.3.3 SECONDARY TREATMENT

- Our high-level analysis of secondary treatment at the Island WWTP indicates that there is sufficient capacity to treat the 2040 MM flows and loads. It is also recommended that the operators pay close attention to the BOD_5 and ammonia loadings to the Island WWTP, to ensure that there are no disruptions to the nutrient removal processes.
- If the City has concerns about the secondary treatment performance or is looking to expand or modify the secondary treatment, it is recommended that the City carry out a detailed analysis of their secondary treatment using more detailed influent characterization and dynamic simulations.
- State point analysis (SPA) carried out for the secondary clarifiers indicate the firm capacity of the system is 1.6 mgd when Clarifier 1 (larger clarifier) is offline. Flows can be temporarily stored in the FEB or divert wastewater to the North Regional WWTP or Mulberry WWTP.
- The return activated sludge (RAS) pump station is anticipated to have sufficient capacity for the 2040 MM flows and loads.
- The secondary effluent pump station that pumps secondary effluent to the tertiary filters appeared to be in good condition during the site visit, and the operators have reported that the pumps are in good operating condition as well.
- Operations staff suspect that there are air leaks in piping between the Blower Building and the center of the Schreiber activated sludge unit. These potential air leaks should be investigated, located, and repaired, if present.

6.3.4 TERTIARY FILTRATION

- The capacity analysis indicated that the tertiary filtration system has sufficient capacity to handle the 2040 peak hourly flows as the flows to the WWTP are equalized and then sent to the downstream processes.
- Cracks were observed in the walls of the sand filter shown on Figure 6-7 during the site walkthrough. It recommended that the City pay close attention to the structural integrity of the concrete and steel associated with the filters and make necessary repairs.
- Jacobs recommends that the cloth filters that have been inoperable since 2018 be repaired and used along with the sand filters for daily operations. The City has received a quote for the rehabilitation of the cloth disk filters from the manufacturer.

Figure 6-7. Cracks Observed in the Concrete Walls of the Sand Filter

6.3.5 DISINFECTION

- The capacity analysis of the UV disinfection system indicates that the system has sufficient capacity to handle the 2040 peak hourly flows with both channels operating, as it is equalized in the FEB in the near future.
- The capacity of the disinfection system is reduced if one channel is offline, decreasing the overall capacity of the WWTP until both channels are operational.
- The operators have indicated that they use third-party UV lamps instead of the Trojan-recommended lamps because of budgetary constraints. This may result in performance issues such as reduced bulb life, as well as reduced UV dosage to properly disinfect the wastewater, potentially leading to noncompliance with ADEQ's water quality standards for Class A+ reuse. Jacobs recommends using Trojan bulbs for optimal UV system performance.
- The UV system has been in service for almost 18 years. Typical lifespan of the Trojan UV3000Plus is 20 to 25 years with proper maintenance. The City is working with Trojan to replace the lamp sleeve cleaning system, hydraulic controls, and effluent gates.
- Despite these maintenance activities, the City should anticipate having to replace the entire system in the next 10 years.

6.3.6 ODOR CONTROL

- It is recommended that the City extend the odor control to the belt filter press (BFP) room in the Headworks Building to avoid hydrogen sulfide corrosion of mechanical and electrical equipment.
- The odor control unit at the sludge holding tank should either be replaced or connected to the existing unit if there is sufficient capacity.

6.3.7 EFFLUENT SYSTEM

- The liner of Effluent Pond A appeared to be in fair condition but will need to be replaced in the next 5 years.
- Pond B liner showed significant signs of degradation.
- Pond C and Pond D appeared to be in good condition during the site walkthrough. These percolation ponds are regularly cycled and are disked and windrowed four times each per year by WWTP staff to ensure there is sufficient volume and surface area available to enhance percolation.
- Jacobs recommends that the City convert Pond B into a percolation pond to allowed greater flexibility in handling reclaimed water.

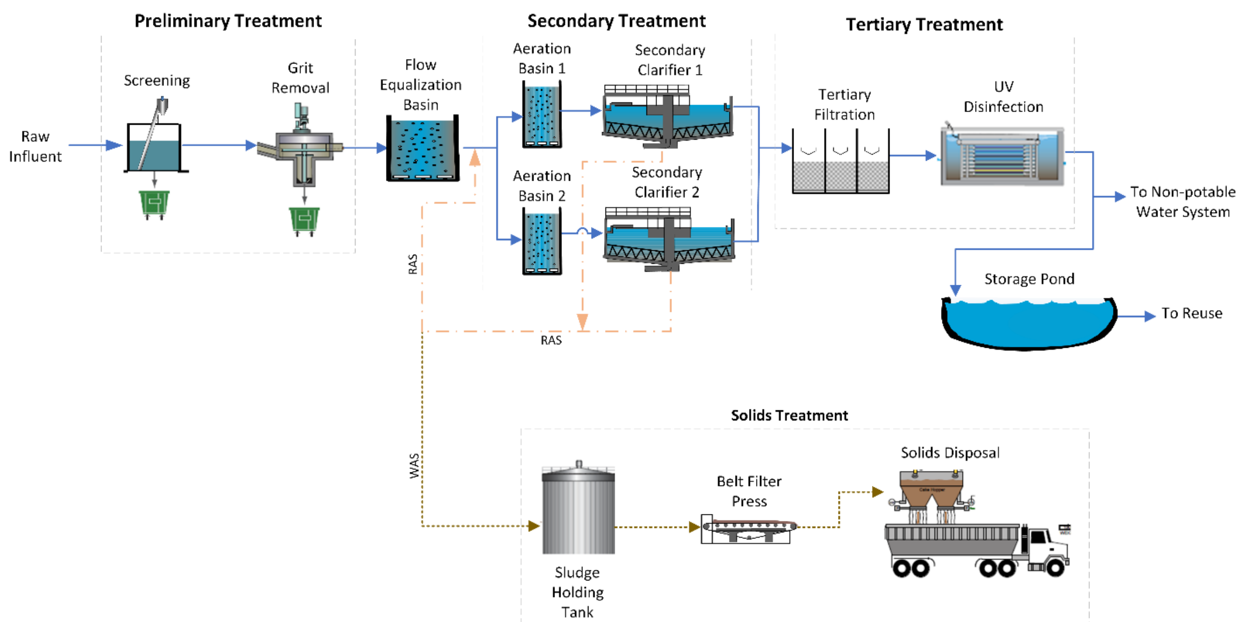
6.3.8 SOLIDS HANDLING

- Overall, the different unit processes of the solids treatment have sufficient capacity to process the 2040 flows and loads.
- A concern identified during the capacity analysis is the cake thickness produced from the BFP equipment. The cake total percent solids is quite low compared to the typical industrial value of 15 to 18 percent from a BFP. This means that every truck load of cake hauled to the landfill has a higher water content than design conditions and results in higher hauling costs. The proposed solutions to this concern have been identified and are discussed in Section 6.7.
- The analysis assumed the same conditions observed historically would still be applicable in 2040. Any changes to the waste activated sludge (WAS) stream resulting from operational changes or addition of new solids stream at Island WWTP will require a reanalysis of the solids treatment.

6.4 MULBERRY WWTP

The Mulberry WWTP is the City's second oldest WWTP with a permitted flow of 2.2 mgd. Figure 6-8 shows the general process schematic for the WWTP. Influent wastewater is treated using a combination of screening, grit removal, flow equalization, biological treatment with activated sludge processes, tertiary filtration, and UV disinfection. The effluent produced meets the requirements of Class A+ Reclaimed water set by ADEQ and is typically used for irrigating golf courses. The solids are generated as a byproduct of the biological treatment and are sent to the sludge holding tank for storage and then dewatered before being disposed of in the City's landfill.

Figure 6-8. Mulberry WWTP General Process Schematic



A detailed evaluation of each Mulberry WWTP main unit process is described in Appendix D. Each section in Appendix D discusses the current unit processes and any operational problems or opportunities for optimization and, provides an analysis of its capacity to treat flows and/or loads through the planning period and recommended modifications. The results and recommendations from the analysis for each unit process are summarized in the following sections.

6.4.1 PRELIMINARY TREATMENT

- The capacity analysis of the preliminary treatment indicated that the systems have sufficient capacity to handle the 2040 peak hourly flows.
- The screens and grit removal were replaced in 2014 and appeared to be in good condition during the site walkthrough.
- It is recommended that the ultrasonic level recorder at the Parshall Flume be calibrated yearly to ensure accurate flow rate data are recorded.

6.4.2 Flow Equalization Basin

- The capacity analysis of the FEB indicated that there is sufficient capacity to handle the 2040 peak day flows.
- The FEB was last cleaned several years ago. It is recommended that the City remove accumulated solids from the FEB to fully use the equalization capacity.
- The influent pumps have sufficient capacity and appeared to be in good condition during the site walkthrough.

6.4.3 SECONDARY TREATMENT

- Our high-level analysis of secondary treatment at the Mulberry WWTP indicates that there is sufficient capacity to treat the 2040 MM flows and loads. It is also recommended that the operators pay close attention to the BOD_5 and ammonia loadings to the Mulberry WWTP, to ensure that there are no disruptions to the nutrient removal processes.
- If the City has concerns about the secondary treatment performance or is looking to expand or modify the secondary treatment, it is recommended that the City carry out a detailed analysis of their secondary treatment using more detailed influent characterization and dynamic simulations.
- SPA carried out for the secondary clarifiers indicated that there is sufficient capacity when both clarifiers are operating. When one clarifier is offline, the other clarifier can handle flows up to 2.2 mgd but is at capacity in terms of solids loading rate and surface overflow rate. Operating in this condition is recommended only for short durations. Flows can be temporarily stored in the FEB or divert wastewater to the North Regional WWTP or Island WWTP.
- During the site visit, cracks were observed in the top wall of the aeration basins as shown on Figure 6-9. The wall of the aeration basins is not completely circular and as a result the weight of the rotating bridge is not evenly distributed. Jacobs recommends that the City repair the concrete in the top wall before cracking accelerates. Additionally, the City is considering piloting submerged fine bubble diffusers with three mixers in one of the aeration basins. If the pilot is successful, the City may consider making the installations permanent and removing the rotating bridge.
- The operators noted that if the secondary clarifiers overflow, the clarified wastewater has the potential to flow offsite into the Daytona Wash, which is connected to the Colorado River. The site is significantly sloped on the southeast side of the Mulberry WWTP near Aeration Basin 2. One such overflow event occurred several years ago that washed away soil and damaged the property fence as shown on Figure 6-10. Jacobs recommends that curbing be designed and installed near the clarifiers to divert any potential overflow to an onsite catchment.
- The RAS pump station has sufficient capacity, but there are no flowmeters on the RAS piping. The operators use the pump run time to get an approximate daily value. Jacobs recommends installing a flowmeter on the RAS discharge piping to allow RAS flow quantification.
- The top of the RAS sump pit walkway is protected by only a chain railing as shown on Figure 6-11. This represents a fall hazard, with the drop almost 20 feet, and it is recommended that the chain be replaced with a more effective and substantial railing.

Figure 6-9. Cracks Observed on the Edge of the Aeration Basin Concrete Wall

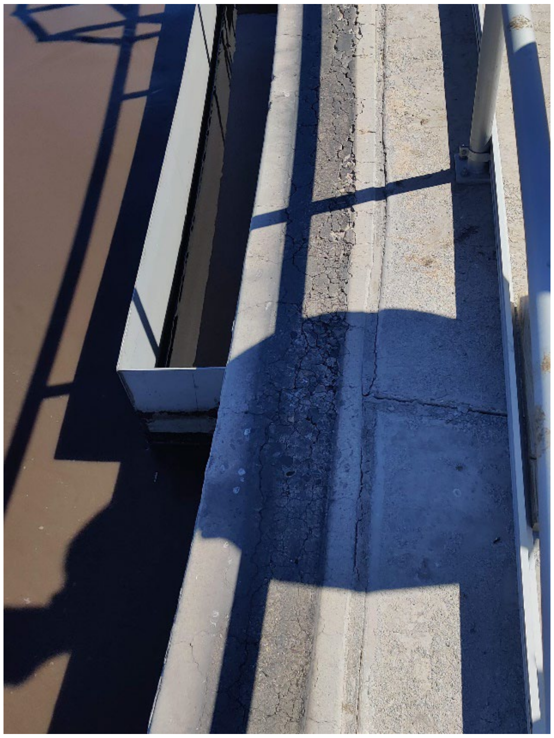


Figure 6-10. (a) Southeast Corner of Mulberry WWTP near Aeration Basin, (b) Impact on the Site from a Clarifier Overflow Event

Figure 6-11. Chain Railing Installed at the Top of the RAS Sump Pit; RAS Pumps are Located to the East

6.4.4 TERTIARY TREATMENT

- The capacity analysis of the tertiary filtration system indicated that this system has sufficient capacity to handle the 2040 peak hourly flows as the flows coming into the WWTP are equalized and then sent to the downstream processes.
- The filters were upgraded in 2020 and appeared to be in good condition during the site walkthrough.

6.4.5 DISINFECTION

- The capacity analysis of the UV disinfection system indicates that the system has sufficient capacity to handle the 2040 peak hourly flows with both channels operating, as it is equalized in the FEB.
- The capacity of the disinfection system is reduced if one channel is offline, decreasing the overall capacity of the WWTP until both channels are operational.
- The operators have indicated that they use third-party UV lamps instead of the Trojan-recommended lamps because of budgetary constraints. This may result in performance issues such as reduced bulb life, as well as reduced UV dosage to properly disinfect the wastewater, potentially leading to noncompliance with ADEQ's water quality standards for Class A+ reuse. Jacobs recommends using Trojan bulbs for optimal UV system performance.
- The UV system has been in service for almost 18 years. Typical lifespan of the Trojan UV3000Plus is 20 to 25 years with proper maintenance.

- Jacobs recommends that the UV disinfection system be connected to the SCADA system to provide greater system and operator control.
- Despite these maintenance activities, the City should anticipate having to replace the entire system in the next 10 years.

6.4.6 ODOR CONTROL

The odor control unit has reached the end of its useful life and needs to be replaced. Jacobs recommends installing a biological odor control unit in place of the wet scrubbers. The biological filter would eliminate the need for the City to procure chemicals, resulting in significant savings. The unit is also relatively easier to operate and is more energy efficient.

6.4.7 EFFLUENT SYSTEM

The lined pond at the Mulberry WWTP that is used to store the treated effluent and associated pump stations appeared to be in good condition during the site walkthrough.

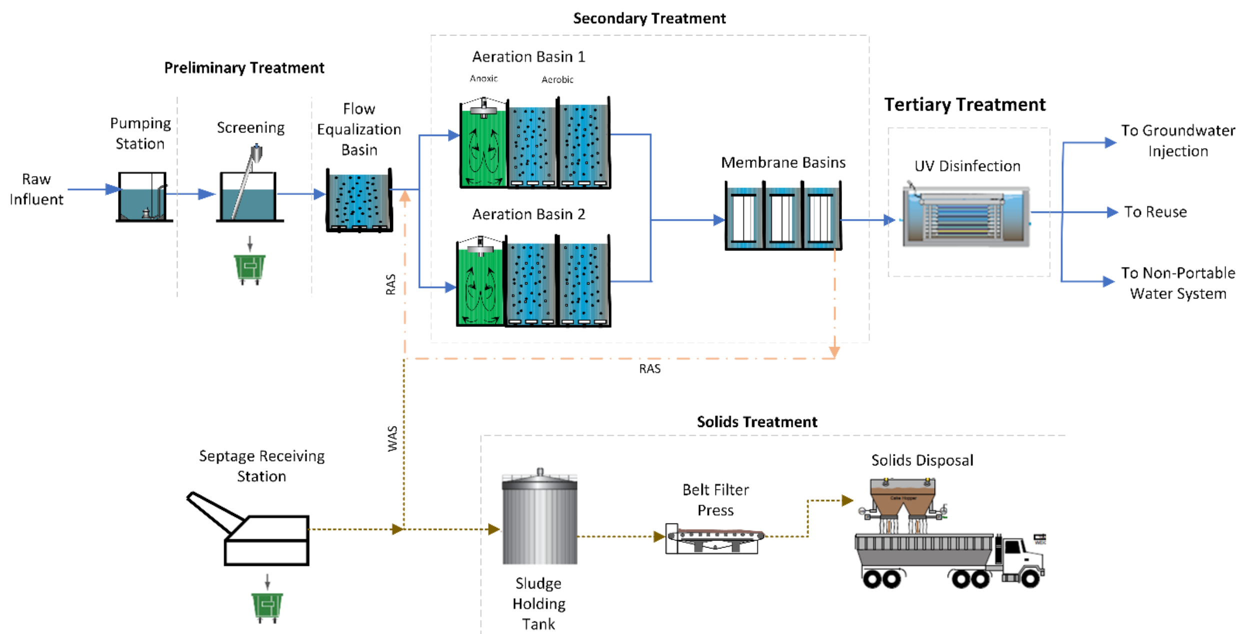
6.4.8 SOLIDS HANDLING

- The sludge holding tank capacity under current conditions cannot store sludge for more than 3 days on average. The capacity is further reduced for the 2040 sludge projections. Jacobs recommends that the City monitor the capacity of the sludge holding tank for the next several years and then decide based on accumulated data how to increase the capacity of the tank.
- The remaining unit processes of the solids treatment have sufficient capacity to deal with the 2040 flows and loads.
- A concern identified during the capacity analysis is the cake thickness produced from the BFP equipment. The cake total percent solids is quite low compared to the typical industrial value of 15 to 18 percent from a BFP. This means that every truck load of cake hauled to the landfill has a higher water content than design conditions and results in higher hauling costs. The proposed solutions to this concern have been identified and are discussed in Section 6.7.
- The analysis assumed the same conditions observed historically would still be applicable in 2040. Any changes to the WAS stream resulting from operational changes or addition of new solids stream at the Mulberry WWTP will require a reanalysis of the solids treatment.

6.5 NORTH REGIONAL WWTP

The North Regional WWTP is the City's newest WWTP, constructed in 2007 with a permitted flow of 3.5 mgd. Figure 6-12 shows the general process schematic for the WWTP. Influent wastewater is treated using a combination of screening, flow equalization, biological treatment with activated sludge processes, membrane bioreactor (MBR), and UV disinfection. The effluent produced meets the requirements of Class A+ Reclaimed water set by ADEQ and is used to recharge the groundwater through vadose wells. The solids are generated as a byproduct of the biological treatment and are sent to the sludge holding tank for storage and then dewatered before being disposed of in the City's landfill.

Figure 6-12. North Regional WWTP General Process Schematic



A detailed evaluation of each of the North Regional WWTP's main unit processes is described in Appendix E. Each section in Appendix E discusses the current unit processes and any operational problems or opportunities for optimization and, provides an analysis of its capacity to treat flows and/or loads through the planning period and recommended modifications. The results and recommendations from this analysis for each unit process are summarized in the following sections.

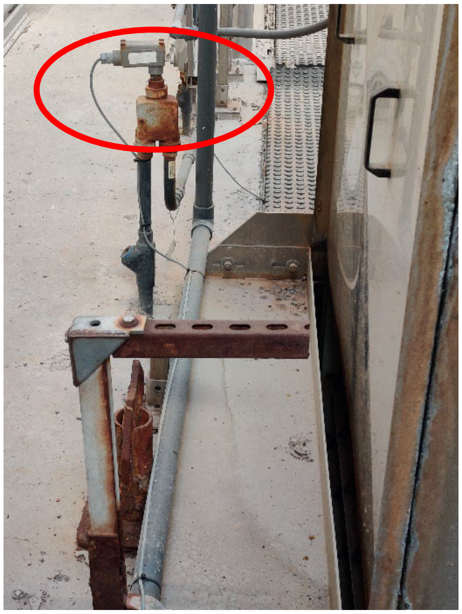
6.5.1 PRELIMINARY TREATMENT

- The capacity analysis of the screening system indicated that this system does not have sufficient capacity to handle the 2040 peak hourly flows. However, if current projections match actual growth, any potential expansion does not need to occur until close to 2040. The screens were recently replaced in 2020 and have a typical lifespan of 15 to 20 years with proper maintenance. Therefore, Jacobs recommends that the City monitor the hourly flows to the North Regional WWTP and based on accumulated data decide on the need for expansion of the screening system.
- The screening room is poorly ventilated and several components in the room showed severe signs of corrosion. The fine screens were installed only several years ago, but the steel enclosure of the screens already shows signs of H_2S corrosion, as shown on Figure 6-13. Plant operations staff developed a ventilation layout for the screening area, which they had installed approximately a year ago, which has helped to lessen H_2S corrosion concerns.
- Severe corrosion was also observed in electrical conduits and in the fire sprinkler system as shown on Figure 6-14. Jacobs recommends that a thorough inspection be performed of the electrical systems and piping networks.

- The grating over the influent channels in the Headworks Building is in poor condition because of historical H_2S corrosion and could potentially pose a safety hazard for the operators. It is recommended to replace the grating as soon as possible.
- Jacobs recommends including a grit removal system in a future Headworks Building upgrade. Common industry practice is for grit removal to precede secondary treatment in those treatment plants that do not have primary clarification. Removal of grit prevents unnecessary abrasion and wear of mechanical equipment, grit deposition in pipelines and channels, and accumulation of grit in the FEB, aeration basins, and sludge holding tanks. The City and Jacobs have set a preliminary date for implementation of the new grit removal facility for fiscal year (FY) 2026–2027.

Figure 6-13. Screen Enclosure Showing Signs of Corrosion
Note newer ventilation piping along wall

Figure 6-14. Typical H_2S Corrosion of the Electrical System and Fire Sprinklers



6.5.2 FLOW EQUALIZATION BASIN

Similar to the preliminary treatment, the FEB at the North Regional WWTP has insufficient capacity to handle one day of the 2040 peak day flows, based on the capacity analysis described in Appendix E.

- It is recommended that the City carefully monitor the peak flows to the North Regional WWTP and water levels in the FEB, and then decide, based on accumulated data, whether another FEB is needed. Space is available next to the existing tank to install the new FEB if warranted.
- Solids accumulated at the bottom of the FEB have not been removed since startup of the North Regional WWTP in 2006. This decreases the useful volume available for wastewater storage and equalization. It is recommended that the City remove the solids from the FEB to fully use the equalization capacity.
- Jacobs recommends installation of a bypass line around the FEB. Having a permanent bypass line would facilitate FEB cleaning and other maintenance activities and provide operational flexibility.
- The FEB pumps are reported to be operating well and have sufficient capacity to handle the 2040 flows. However, the lack of grit removal and accumulation of solids in the FEB may tend to wear out the pumps faster than their typical lifespan. It is recommended that operators pay close attention to the condition of the pumps and undertake necessary maintenance activities to keep the pumps operational.

6.5.3 SECONDARY TREATMENT

- The capacity analysis indicated that the aeration basins and the MBR system have sufficient capacity to treat the 2040 flows and loads.
- It is recommended that the City further automate the airflow system at the North Regional WWTP by including automatic valves on the header pipe going to the two aeration basins, and additional dissolved oxygen (DO) probes in the basins. This will allow for greater control over the air being delivered to the aeration basins and potentially save energy costs.
- The ultrafiltration membranes have reached the end of their useful life and are being replaced one membrane train at a time.
- The recovery clean process is performed only once every 1.5 years instead of once every 4 to 6 months, as recommended. The operators have indicated that the Island WWTP and the Mulberry WWTP are unable to manage the flows diverted from the North Regional WWTP during the cleaning cycle, which typically lasts for a couple of days. The lack of regular recovery cleaning will lead to reduced membrane life and the need for more frequent replacements.
- If the City has concerns about the secondary treatment performance or is looking to expand or modify the secondary treatment, it is recommended that the City carry out a detailed analysis of their secondary treatment using more detailed influent characterization and dynamic simulations.

6.5.4 DISINFECTION

- The capacity analysis of the UV disinfection system indicates that the system has sufficient capacity to handle the 2040 peak hourly flows but lacks redundancy because there is only one UV channel.
- The operators have indicated that they use third-party UV lamps instead of the Trojan-recommended lamps because of budgetary constraints. This may result in performance issues such as reduced bulb life, as well as reduced UV dosage to properly disinfect the wastewater, potentially leading to noncompliance with ADEQ's water quality standards for Class A+ reuse. Jacobs recommends using Trojan bulbs for optimal UV system performance.
- The UV system has been in service for almost 16 years. Typical lifespan of the Trojan UV3000Plus is 20 to 25 years with proper maintenance.
- Jacobs recommends that the UV disinfection system be connected to the SCADA system to provide greater system and operator control.
- The City should anticipate having to replace the entire system in the next 10 years.
- During the next upgrade to the UV disinfection system, additional channels should be constructed to ensure that the system has sufficient redundancy.

6.5.5 ODOR CONTROL

The odor control unit has reached the end of its useful life, maintenance and chemical costs are rapidly increasing, and the unit needs to be replaced in the near future. Jacobs recommends installing a biological odor control unit in place of the wet scrubbers. The biological filter would eliminate the need for the City to procure chemicals, resulting in significant savings in chemical costs. A biological unit would also be relatively easier to operate and would be more energy efficient.

6.5.6 SOLIDS HANDLING

- Overall, the different unit processes of the solids treatment have sufficient capacity to deal with the 2040 flows and loads.
- A concern identified during the capacity analysis is the cake thickness produced from the BFP equipment. The cake total percent solids is quite low compared to the typical industrial value of 15 to 18 percent from a BFP. This means that every truck load of cake hauled to the landfill has a higher water content than design conditions and results in higher hauling costs. The proposed solutions to this concern have been identified and are discussed in Section 6.7.
- Similar to the screening room, severe H_2S corrosion was observed in the BFP dewatering room in of the Headworks Building. During the site visit, severe corrosion was observed in the overhead pipes, pipe fittings, flanges, BFP overhead supports, and electrical conduits as shown on Figure 6-15. It is recommended that the City immediately improve the ventilation in the room, and subsequently replace corroded piping and electrical fixtures.
- The analysis assumed the same conditions observed historically would still be applicable in 2040. Any changes to the WAS stream resulting from operational changes or the addition of a new solids stream at the North Regional WWTP will require a reanalysis of the solids treatment.

Figure 6-15. Hydrogen Sulfide Corrosion is Observed on the Fire Sprinkler Piping and BFP Overhead Supports

6.6 WASTEWATER STAFFING PLAN

In addition to staffing observations made over multiple visits to each WWTP, Jacobs interviewed Mr. Thilak Fernando, Wastewater Superintendent, and Mr. Keith Lueken, Utilities Supervisor, for their input on staffing needs, concerns, and strategy.

The laboratory at the Mulberry WWTP is staffed by three persons. Previously, following Mr. Fernando's transition to Wastewater Superintendent, the staff included only two persons. With the current staffing level, the lab is sufficiently able to handle the workload, meet schedules, and maintain consistent operations and quality. The laboratory workload and equipment do not currently warrant the addition of a fourth individual.

Historically, the Mulberry WWTP and the Island WWTP were staffed by a lead operator, an operator, and a mechanic, with three dedicated staff at each of these two plants. The North Regional WWTP had an additional operator, for a total of four staff members. Several years ago, mechanics were assigned the additional duties of covering lift station maintenance as well. Also, because of system needs, at one point one mechanic transitioned to become a SCADA technician, one mechanic took over the City's Industrial Pretreatment Program, and one mechanic transitioned to become an operator.

The biggest current staffing concern is that the Island WWTP needs a lead operator. City staff are working on filling this key position. Different grades and finer differences between "Steps" in the City's job descriptions are needed to better differentiate utility operator I/II/III positions, skill sets, and salary. Currently the steps in the City's Wastewater Department hiring process jump from 14 to 15 to 21, with too wide of a skill and experience gap between steps 15 and 21. Wastewater management is working with City Human Resources to facilitate the addition of all or part of steps 16 through 20, to better align candidates with positions, and be able to better differentiate and reward wastewater staff for productivity, job performance, and goal achievement.

Wastewater management also has a succession plan in place for senior leadership, with key senior staff being evaluated for potential future upward moves when there are top-level management changes.

6.7 BIOSOLIDS MANAGEMENT PLAN

Sludge or solids are generated as a byproduct of wastewater treatment processes. During the liquids treatment, solids are separated out at various stages. Some of the solids streams such as screenings and grit removed are typically sent to landfills for disposal. The solids streams generated by secondary treatment are further treated before either being used beneficially or disposed of as daily cover in a landfill.

The solids treatment systems at the City's three WWTPs employ similar treatment technologies. The solids produced by the secondary treatment processes are sent to aerobic holding tanks where they are slightly thickened. Minimal treatment of the solids is achieved in these holding tanks. The sludge is then pumped to the dewatering equipment to remove excess water. The dewatered cake or biosolids is hauled to the City's landfill for final disposal. The solids treatment equipment at the three treatment plants have been described in the previous sections.

Because the City is disposing of biosolids in the landfill, the City does not have to follow the federal and state requirements for the beneficial reuse of biosolids (40 *Code of Federal Regulations* Part 503). Instead, the solids must meet the requirements stated by the landfill, which typically include passing the paint filter test and toxicity characteristic leaching procedure (TCLP) analytical testing.

Jacobs contacted the City Manager's office to inquire about the remaining useful life of the City-owned landfill, which is currently operated by Republic Industries. The landfill's life expectancy is 47 years, pending approval of a vertical expansion permit that is expected to add 3.7 million cubic yards of storage space to the landfill, without increasing the landfill's footprint. Based on this information, biosolids disposal at the landfill through 2040 does not pose an issue.

Table 6-2 summarizes the historical and projected cake production at the three treatment facilities. The detailed development of the sludge numbers for the three WWTPs are discussed in Appendices C, D, and E.

Table 6-2. Historical and 2040 Estimated Sludge Production at the Three Treatment Plants

Parameter	Estimated Hours of Operation[a]	Wet Cake Produced, lb/d		Cake Solids, Percent		Dry Cake Produced, lb/d	
		Historical Data (2016–2021)	2040 Flows and Loads	Historical Data (2016–2021)	2040 Flows and Loads	Historical Data (2016–2021)	2040 Flows and Loads
Island WWTP	5 hours per day, 2 days total	27,000	30,800	12	12	3,200	3,700
Mulberry WWTP	5 hours per day, 3 days total	24,500	28,000	11	11	2,700	3,080
North Regional WWTP	6 hours per day, 5 days total	28,230	32,250	14	14	3,950	4,520

[a] Hours of operation for each WWTP were obtained from the operators. Estimated hours of operation for the 2040 conditions are assumed to be the same as the current hours.

lb/d = pound(s) per day

Analysis of the total percent solids of the biosolids indicates that the dewatering equipment is not performing up to its optimal level at the Island WWTP and the Mulberry WWTP. The North Regional WWTP appears to be performing better but is still not up to the typical values of 15 to 18 percent seen in the industry for this type of dewatering equipment. Jacobs recommends the following to improve the solids handling at all three facilities:

- The City should continue with their current solids treatment and disposal methodology, which is a more cost-effective solution than upgrading to allow for the reuse of biosolids.
- Optimize the operations of the dewatering equipment.

- This may include changing the hydraulic/solids feed rates to the BFP. Particularly at the North Regional WWTP, the addition of a lead operator will help optimize BFP operations by freeing up staff to manage the BFP.
- Polymer dosing should also be optimized. This may include changing the rates of polymer addition or changing the polymer type. The City may choose to conduct pilot studies with different polymers before selecting an optimal polymer.

7. Reclaimed/Reuse System Evaluation

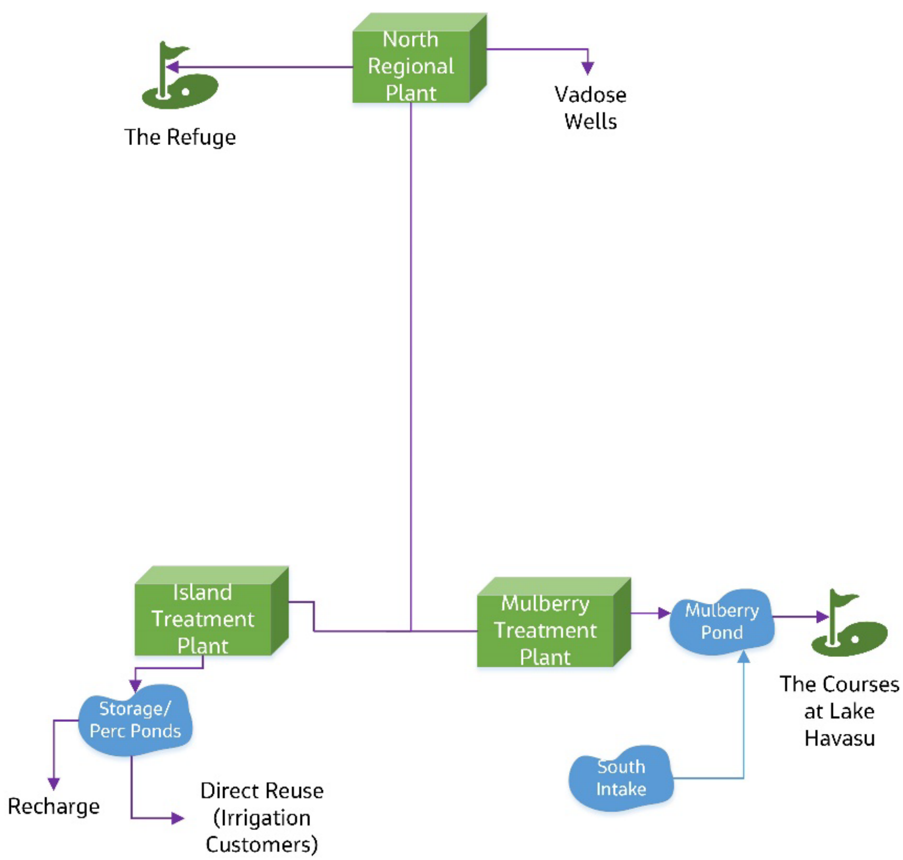
This section provides an overview of the City's reclaimed water system. It includes a summary of the system operation, reclaimed customer consumption trends, and an estimate of future reclaimed water availability, including options for expanding beneficial reuse.

7.1 SYSTEM OVERVIEW

The City's reclaimed water system is a "closed system" where all effluent must either be directly reused by customers or recharged. A schematic is shown on Figure 7-1, which highlights reuse customers, including two major golf courses and irrigation customers near the Island WWTP. The City may also recharge effluent via vadose zone wells at the North Regional WWTP or the percolation ponds at the Island WWTP. If needed, there is an intake from the Colorado River that may be used to supplement supply to The Lake Havasu Golf Club. The system is flexible in that reclaimed water may be redirected between facilities as summarized as follows:

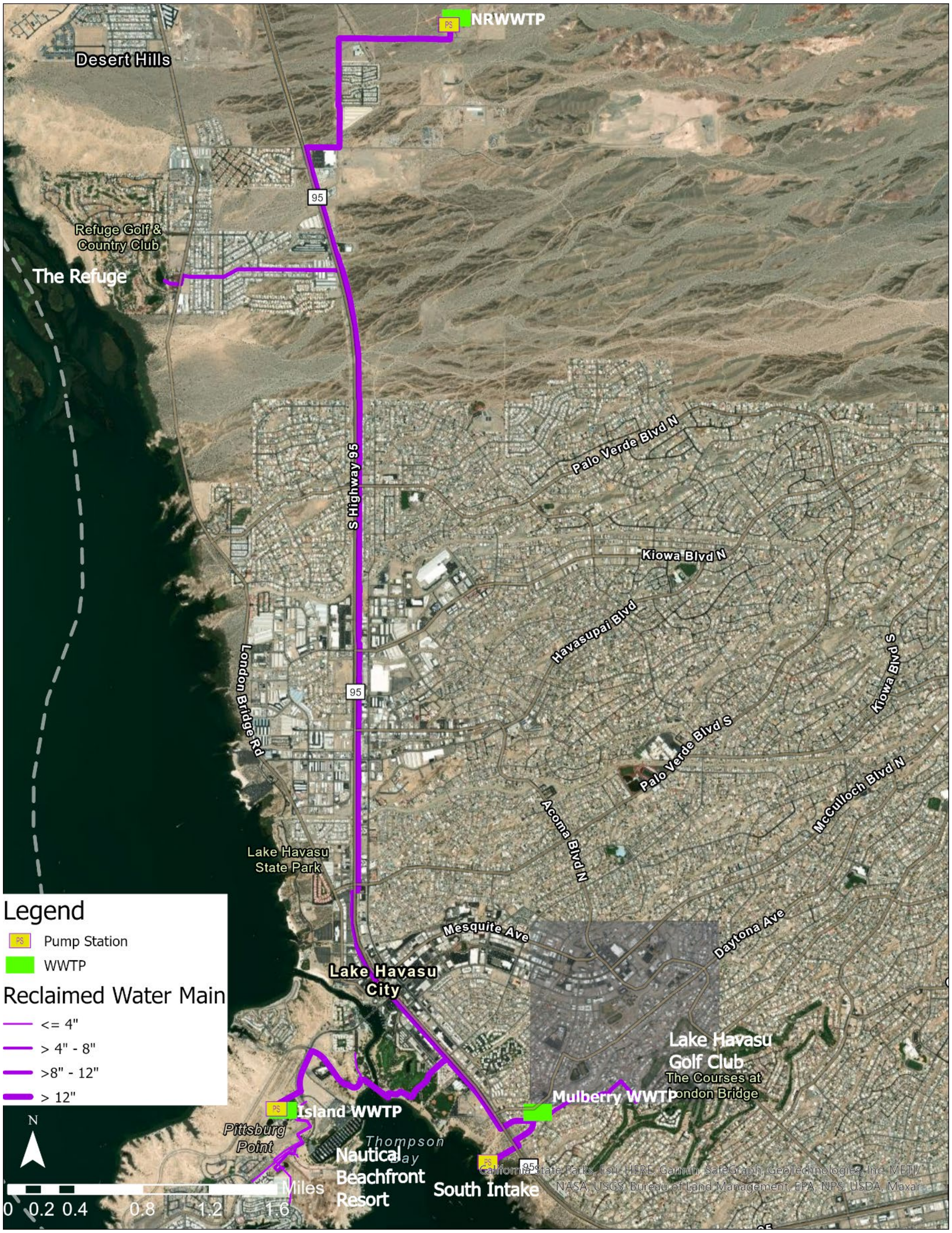
- Effluent from the North Regional WWTP may be directed to the Island WWTP or the Mulberry WWTP
- Effluent from the Mulberry WWTP may be directed to the Island WWTP
- Effluent from the Island WWTP may be directed to the Mulberry WWTP (rarely operated)

Figure 7-1. Reclaimed System Schematic



A map of the reclaimed water distribution system is shown on Figure 7-2. The customers that use the largest volumes of reclaimed water (Lake Havasu Golf Club, The Refuge, and the Nautical Beachfront Resort) are also noted. More detail regarding customer consumption is provided in the next section.

Figure 7-2. Reclaimed Water Distribution System Map



7.2 RECLAIMED CUSTOMER CONSUMPTION

Over the last several years, just under half of the City's reclaimed water has been reused by customers for irrigation. The influent received at the treatment plants is quite consistent during the year, with a peak month that typically occurs in March of each year due largely to the transient population; peak month flows are about 10 percent higher than average. However, customer use of reclaimed effluent varies seasonally with higher volumes used during the peak summer months for irrigation. Figure 7-3 shows the monthly reclaimed customer consumption (displayed as stacked lines such that the difference between each line represents the use of each customer) along with the annual average influent to the wastewater plants.

Figure 7-3. Monthly Reclaimed Customer Consumption

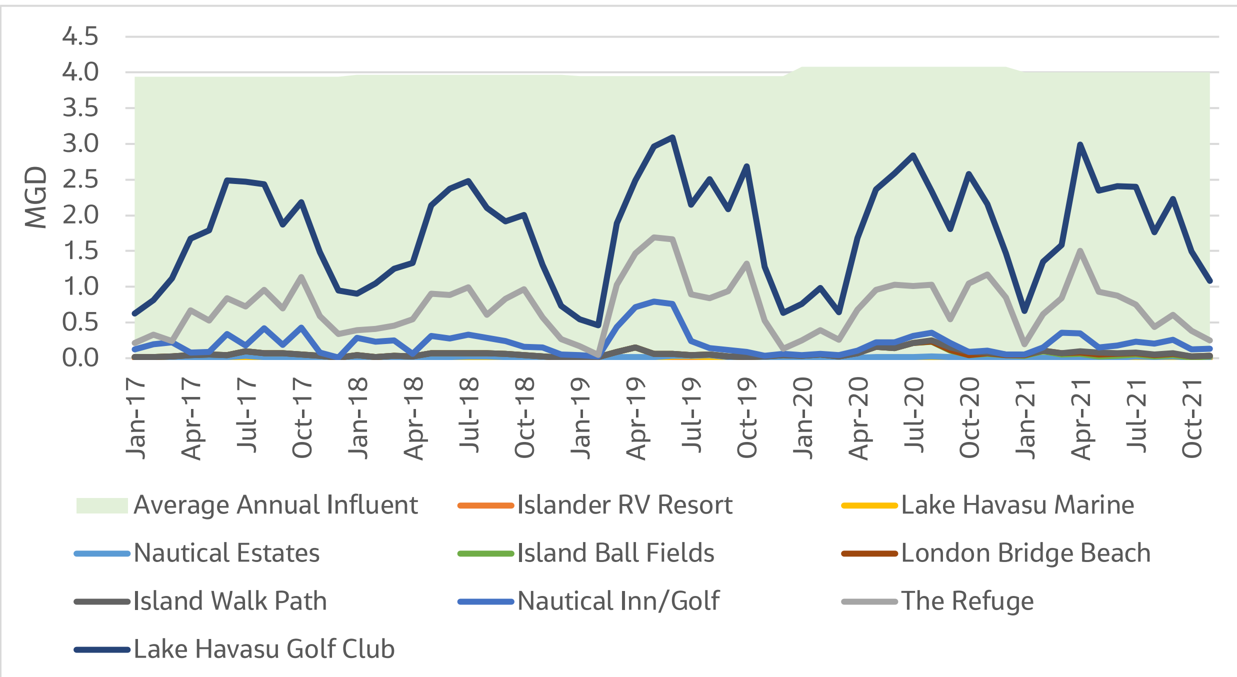


Table 7-1 shows the annual average customer consumption for the last several years. Larger customers are displayed in mgd, whereas smaller customers are displayed in gpd. The Nautical Inn Golf Course has been closed and is no longer used as a spray field for the Island WWTP effluent, although the City does maintain an operations agreement to use the land. The City must replace this loss of demand with increased recharge.

Table 7-1. Annual Average Customer Consumption

Customer	2017	2018	2019	2020	2021a
Million Gallons per Day					
Lake Havasu Golf Club	1.05	0.98	1.00	1.08	1.17
The Refuge	0.41	0.43	0.61	0.61	0.47

Table 7-1. Annual Average Customer Consumption

Customer	2017	2018	2019	2020	2021a
Nautical Inn/Golf	0.15	0.17	0.24	0.04	0.13
Gallons per Day					
Island Ball Fields	28,295	30,279	28,960	83,071	29,178
Nautical Estates	11,820	8,825	10,367	8,911	8,557
Lake Havasu Marine	3,298	3,774	8,372	3,582	3,097
London Bridge Beach	-	-	-	3,367	17,198
Island Walk Path	-	-	-	8,612	9,845
Islander RV Resort	110	1,214	1,011	595	-

[a] Data for December 2021 were not available for the analysis.

Figure 7-4 shows a box and whisker plot of the larger reclaimed water customers, and Figure 7-5 shows a similar plot for the smaller reclaimed water customers. These graphs show each customer's variability or lack thereof from year to year. The minimum and maximum use per year is shown by the whiskers or lines, and the bar represents the middle 50 percent of each customer's use during the year. The median is shown as a horizontal line in the box, and the mean is shown as an "x" in the box.

Figure 7-4. Large Reclaimed Water Customer Box and Whisker Chart

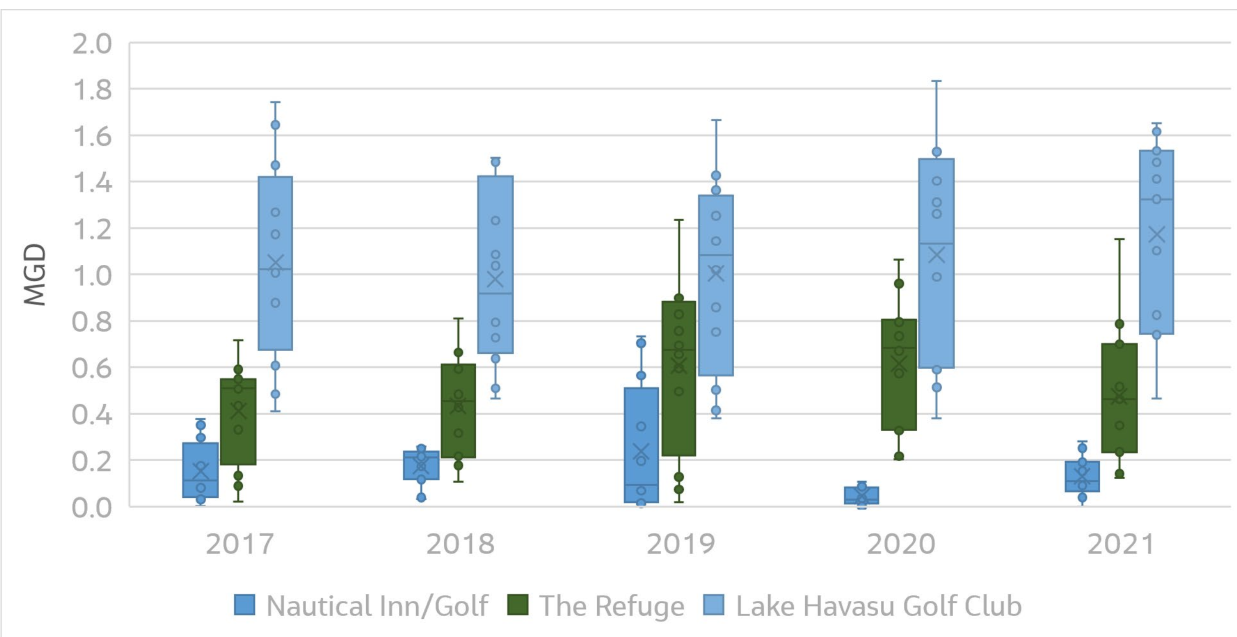
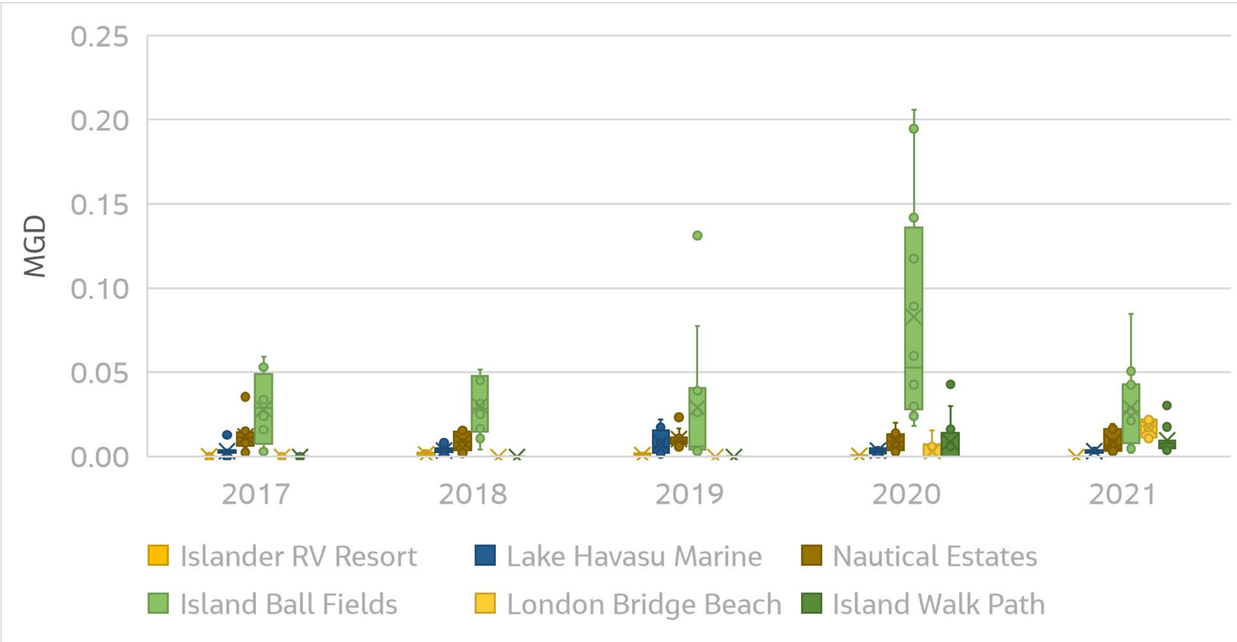


Figure 7-5. Small Reclaimed Water Customer Box and Whisker Chart



7.3 REUSE SYSTEM BALANCE

As described earlier in this section, the City's reclaimed water must either be reused or recharged because there is no outfall to discharge effluent. Jacobs reviewed the City's data to develop an annual and MM (wastewater flow MM) balance of reclaimed water. This process began with estimates of effluent from each treatment plant. Summaries of influent received versus effluent produced at each treatment plant varied within data provided by the City because of the system's ability to transfer flows among the plants. Jacobs also reviewed total influent versus effluent produced across the system, and in 2020 and 2021, the effluent produced exceeded 100 percent of influent received on an annual basis. To avoid issues of estimating too much effluent, Jacobs assumed the following influent-to-effluent ratios at each facility:

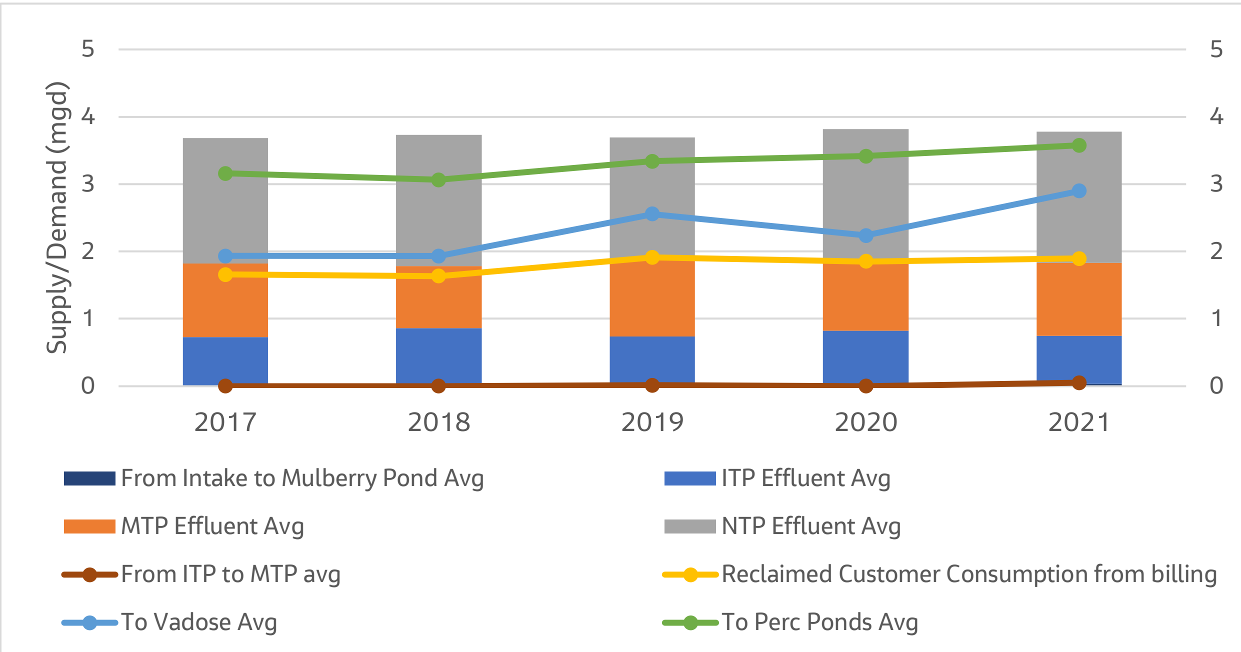
- Mulberry WWTP: 90 percent
- Island WWTP: 95 percent
- North Regional WWTP: 95 percent

The elements included in the reclaimed water balance are summarized as follows:

- **Supplies**
  - Estimated effluent from each WWTP
  - Surface water delivered from the South Intake
- **Demands**
  - Reclaimed customer consumption from the billing system
  - Reclaimed water delivered to the vadose wells
  - Reclaimed water delivered to the Island WWTP percolation ponds
  - Reclaimed water transferred from the Island WWTP to the Mulberry WWTP

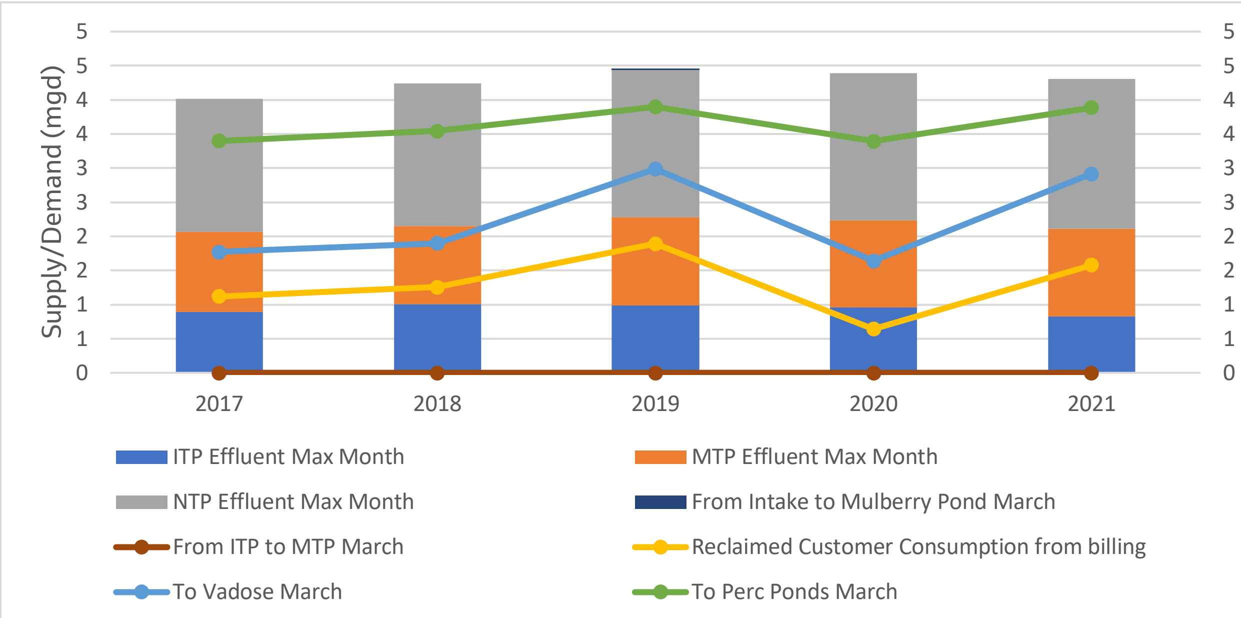
Figure 7-6 shows the annual average reclaimed water balance in recent years. The supplies shown as stacked bars do not fully match the demands shown as stacked lines, but the data are within reason for planning purposes. The discrepancies may be due to flow meter irregularities, offsets of reclaimed water consumption from the billing system from actual dates used to dates when meters were read, or not accounted for in system/pond storage. Overall, just under half of the City's effluent was directly reused by customers with the remainder recharged via the percolation ponds or vadose wells.

Figure 7-6. Annual Average Reclaimed Water Balance



The maximum wastewater flows in the City typically occur in March each year and are about 10 percent higher than the annual average. Jacobs also summarized the MM balance that occurred in March each year paired with reclaimed water deliveries to customers or recharged each March as shown on Figure 7-7. In March, about 30 percent of the City's reclaimed water is reused by customers and the remainder is recharged.

Figure 7-7. Maximum Month Reclaimed Water Balance



7.4 FUTURE RECLAIMED SYSTEM EVALUATION

To determine the future needs of the reclaimed water system, Jacobs used the wastewater flow projections from Section 3 to estimate effluent available in the future. The same influent-to-effluent ratios were applied to each plant as summarized in Section 7.3, where the amount of reclaimed water produced by the plants will be about 4.3 mgd by 2040.

There are very few additional customers that may use reclaimed water for irrigation purposes in the future; Sara Park is a potential future large user, but the investment required to build reclaimed water delivery infrastructure to the south end of the City is cost prohibitive. Because of this constraint, Jacobs assumed future reclaimed customer consumption would equal the 5-year historical average (either annual, in March, or minimum, depending on the scenario). Other assumptions are noted as follows:

- Deliveries to the vadose wells are limited to 1 mgd (current maximum capacity)
- Deliveries to the percolation ponds are equal to the 5-year historical average (either annually or March depending on the scenario)
- Reclaimed water transferred from Island WWTP to Mulberry WWTP are equal to the 5-year historical average (either annually or in March depending on the scenario)

Graphs of the annual average and MM historical data and projections are shown on Figure 7-8 and Figure 7-9.

Figure 7-8. Annual Average Reclaimed Water Balance Projection

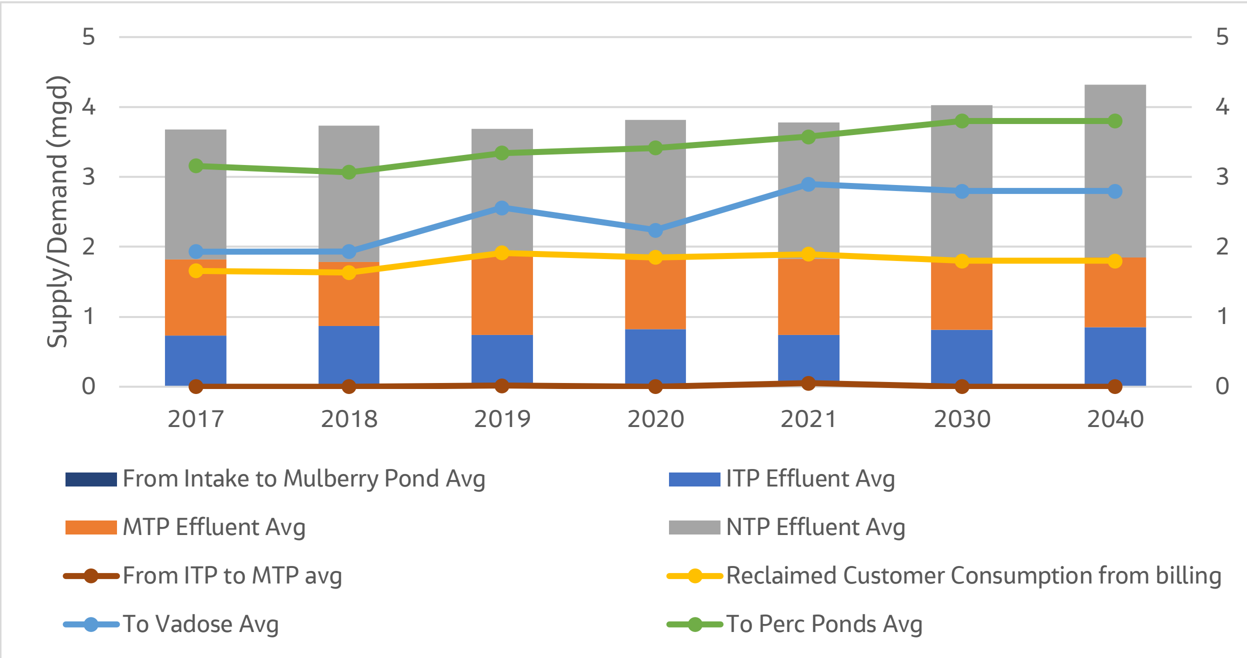
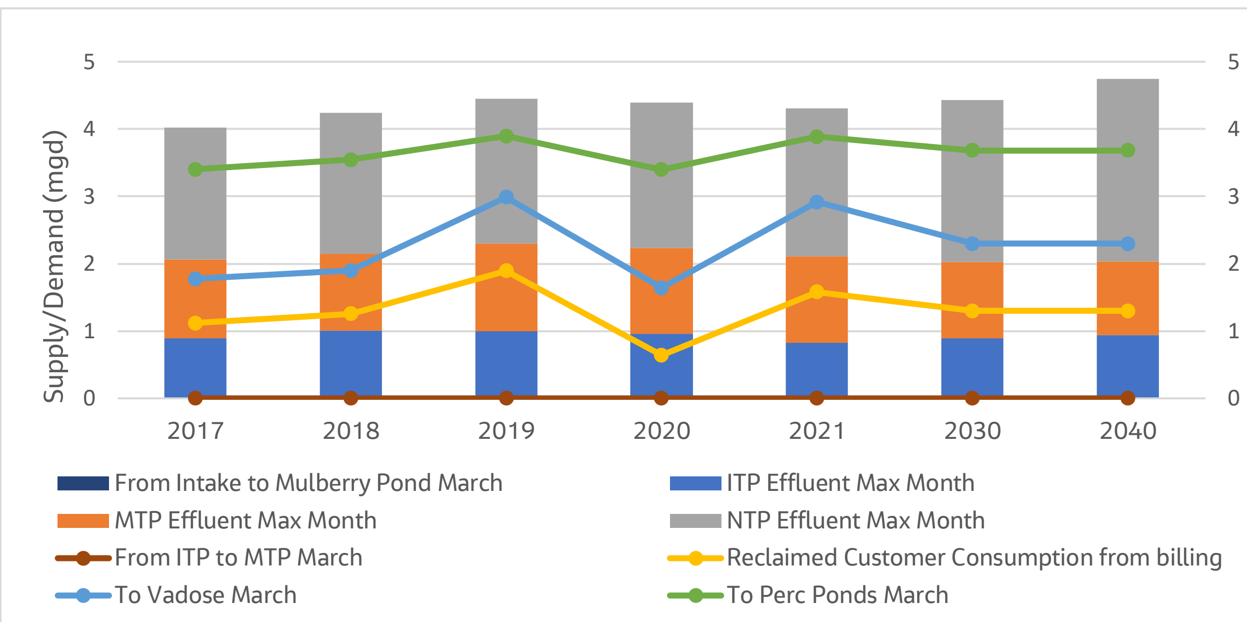
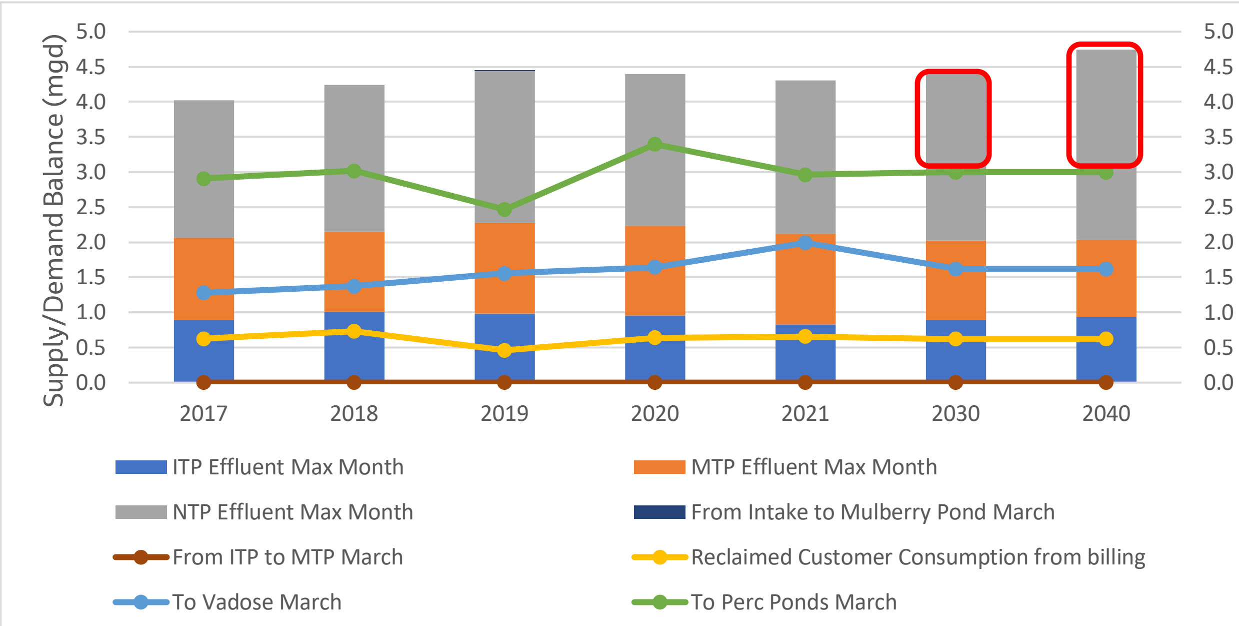


Figure 7-9. Maximum Month Reclaimed Water Balance Projection



To evaluate a worst-case scenario, Jacobs also compared the effluent available under MM conditions against minimum reclaimed water customer consumption. The 5-year historical minimum reclaimed water consumed by customers was applied to the 2030 and 2040 projections as shown on Figure 7-10.

Figure 7-10. Maximum Month Reclaimed Water Balance with Minimum Reclaimed Water Customer Consumption Projection



Note: These data represent a hypothetical condition and do not portray historical results that occurred simultaneously.

The red rectangles in the figure represent the amount of vadose well capacity that the City would need to add in the future. By 2030, the City would need about 1.5 mgd of additional capacity and nearly 2 mgd of additional capacity by 2040. In addition to vadose well expansion, the City may consider other alternatives in the future as described in Section 7.5.

7.5 REUSE SYSTEM CONSIDERATIONS

Aside from adding vadose well capacity, the City may also consider rehabilitating the Island WWTP percolation ponds for effluent recharge. Another alternative for effluent reuse is to either implement more robust extraction wells to recover water from the vadose wellfield and deliver it to the water treatment plant (WTP) or implement direct potable reuse at the WTP. This option would require construction of a side-stream advanced reclaimed water treatment facility at the WTP to blend this smaller volume of water with that from the existing treatment processes. The advanced reclaimed water treatment facility would include, at a minimum, microfiltration and ultraviolet light disinfection to provide a multibarrier approach that will result in a high level of log removal of wastewater pathogens. Tasks and analysis would also include an extensive characterization of all sources supplying the wastewater treatment facilities, the need to address requirements for control of trace chemicals of health concern that might be present in the effluent(s), and a pilot study using the proposed treatment train and source water.

From an operational perspective, the City would likely be subject to enhanced monitoring requirements and require additional operator training and possible certification. The potential to operate the system year-round is also limited because excess reclaimed water supplies typically occur only in the winter months. With the investment required and constraints described in this section, this option is not recommended at this time.

Another alternative is to evaluate the construction of an effluent outfall. There is an existing reclaimed water distribution line that delivers effluent to The Refuge golf course, which could potentially be extended to serve as an outfall for excess reclaimed water in the winter. A sketch (yellow line) of the location is shown on Figure 7-11.

Figure 7-11. Sketch of Potential Effluent Outfall

This option affords the City more flexibility, but also requires that the City obtain an Arizona Pollutant Discharge Elimination System (AZPDES) permit from ADEQ. Specifically, AZPDES permit form 2A/2S is needed, and the process may require a 208 amendment. The process takes several months, and compliance reporting may include whole effluent toxicity testing.

An outfall would provide redundancy for the vadose wells and the percolation ponds. The City may even be able to decommission the Island WWTP percolation ponds, where the land may be repurposed to public or private use. Although it would also be beneficial to renegotiate the City's Colorado River Diversion Right contract with the United States Bureau of Reclamation to a Consumptive Use contract to receive return flow credit (for effluent returned via a future outfall or the Island WWTP percolation ponds), the renegotiation process may be onerous and unsuccessful.

Lastly, the City may consider implementing an ordinance tied to a drought management plan or other documentation related to water resource management during drought conditions that requires the use of reclaimed water for irrigation. This could be applied to current water customers that irrigate from the potable distribution system or wells that are tallied against the City's potable allocation. A summary of potential customers is shown in Table 7-2.

Table 7-2. Consumption of Potable Water Irrigation Customers to Consider for Reclaimed Irrigation

Customer	Annual Average Use gpd)	Maximum Month Use (gpd)	Notes
Bridgewater Links Golf Course near London Bridge Resort	266,600	353,700	June 2020–May 2021 use with maximum occurring in July 2020
Rotary Park	106,300	211,000	Fiscal Year 2019–2020 with maximum occurring in August 2019
Cypress Park	24,000	54,400	Fiscal Year 2019–2020 with maximum occurring in August 2019
Jack Hardie Park (Upper)	3,700	8,400	Fiscal Year 2019–2020 with maximum occurring in August 2019
Jack Hardie Park (Lower)	2,700	6,600	Fiscal Year 2019–2020 with maximum occurring in August 2019

The golf course and Rotary Park are very close to existing reclaimed water infrastructure, and Cypress Park and Jack Hardie Park have reclaimed water infrastructure nearby that has not yet been placed in service. Converting these customers from potable sources that count against the City's allocation to a reclaimed water supply would provide enough potable water for several hundred homes.

8. Recommended Capital Improvement Plan Overview

This section presents the recommended capital improvement program (CIP) for the City based on the findings of the Wastewater Master Plan. The CIP integrates the previously identified projects with results from the Wastewater Master Plan to provide a detailed 5-year CIP focused on the highest collections, treatment, and reclaimed system projects, as well as identifying major improvements required through the planning period of 2040.

8.1 OVERVIEW

The recommended improvement projects are organized by project type and include:

- Wastewater Collections System
- WWTP Upgrades
- Sewer Pump Stations: Rehabilitation Projects

Proposed phasing for project implementation is noted on Figure 8-1.

8.2 COST METHODOLOGY/ASSUMPTIONS

Unit construction costs were developed using Association for the Advancement of Cost Engineering guidelines for a Class 5 estimate and from recent construction projects within the City for similar projects and similar unit costs on prior work within the City. All costs are presented in 2022 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 percent)
- Environmental, legal, construction management, contract administration (15 percent)
- Contingency (25 percent)

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

- Pipelines ($ per diameter per inch)
  - Gravity sewers - $18
  - Deep gravity sewers - $25
  - Force mains - $35
- Pump Stations
  - New local lift station - $250,000 (100 to 300 gpm capacity)
  - Pump replacements - $400 per pump hp

- Screening facilities - $500,000
- VFDs - $50,000

- WWTP Upgrades
  - Most of the CIP project costs for the WWTP upgrades were provided by the City.
  - The remaining CIP project costs such as UV disinfection system replacement and solids removal in FEBs are based on Jacob's previous experiences at WWTPs of similar size.

8.3 CAPITAL IMPROVEMENT PLAN

The projects presented in this section are recommended to maintain and improve the wastewater collections, treatment, and reclaimed systems and continue to meet projected growth. Projects 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 projected growth forecast during the planning horizon (2040) not be realized, there may be opportunities to defer or eliminate some projects.

Table 8-1 presents the proposed capital improvement plan with capacity, reliability, and rehabilitation projects identified on an annual basis. Figure 8-1 illustrates the 5-year CIP costs.

The Wastewater Master Plan identified new projects included in the wastewater CIP, as well as changes to the costs or schedule of existing projects. These new projects/changes are discussed in the following sections.

Table 8-1. Fiscal Year 2023–2040 Proposed Capital Improvement Plan

Project Number	Project Description	Prior ($)	22-23 ($)	23-24 ($)	24-25 ($)	25-26 ($)	26-27 ($)	27-32 ($)	32-40 ($)	FY 2023–2040 Total ($)	FY 2023–2040 Total with Prior ($)
107007	Island WWTP Flow Equalization Basin (FEB)	1,490,000	4,393,000	-	-	-	-	-	-	4,393,000	5,883,000
107008	Chip Drive Lift Station Upgrades	103,700	793,780	-	-	-	-	-	-	793,780	897,480
107012	South Intake Influent Screen	-	-	-	225,000	-	-	-	-	225,000	225,000
TBD	Vadose Well Design and Expansion	100,000	1,860,000	1,100,000	1,100,000	-	-	-	-	4,060,000	4,160,000
TBD	Mulberry WWTP Aeration Basin Repair	43,600	900,000	-	-	-	-	-	-	900,000	943,600
TBD	Mulberry WWTP Roof Replacement	-	-	-	300,000	-	-	-	-	300,000	300,000
TBD	Park Avenue Lift Station Renovation	-	250,000	1,650,000	-	-	-	-	-	1,900,000	1,900,000
TBD	Island WWTP Metal Storage Building	-	-	-	200,000	-	-	-	-	200,000	200,000
TBD	Island WWTP Aeration Basin Upgrades	-	-	-	475,000	-	-	-	-	475,000	475,000
TBD	Island WWTP Upgrade Sand Filters	-	-	-	700,000	-	-	-	-	700,000	700,000

Table 8-1. Fiscal Year 2023–2040 Proposed Capital Improvement Plan

Project Number	Project Description	Prior ($)	22-23 ($)	23-24 ($)	24-25 ($)	25-26 ($)	26-27 ($)	27-32 ($)	32-40 ($)	FY 2023–2040 Total ($)	FY 2023–2040 Total with Prior ($)
TBD	Island WWTP Effluent Upgrades	-	-	-	450,000	-	-	-	-	450,000	450,000
TBD	Mulberry WWTP Effluent and Recharge Pond Upgrades	-	-	-	-	100,000	300,000	-	-	400,000	400,000
TBD	North Regional WWTP Effluent and Recharge Upgrades	-	-	-	-	150,000	150,000	-	-	300,000	300,000
TBD	North Regional WWTP Grit Removal						1,240,000			1,240,000	1,240,000
TBD	North Regional WWTP UV Disinfection Replacement	-	-	-	-	-	-	1,820,000	-	1,820,000	1,820,000
TBD	North Regional WWTP FEB Solids Removal	-	-	-	1,240,000	-	-	-	-	1,240,000	1,240,000
TBD	Island WWTP UV Disinfection Replacement	-	-	-	-	-	-	1,310,000	-	1,310,000	1,310,000
TBD	Island WWTP Headworks Improvement Project	-	-	-	-	1,000,000	5,000,000	-	-	6,000,000	6,000,000

Table 8-1. Fiscal Year 2023–2040 Proposed Capital Improvement Plan

Project Number	Project Description	Prior ($)	22-23 ($)	23-24 ($)	24-25 ($)	25-26 ($)	26-27 ($)	27-32 ($)	32-40 ($)	FY 2023–2040 Total ($)	FY 2023–2040 Total with Prior ($)
TBD	Mulberry WWTP UV Disinfection Replacement	-	-	-	-	-	-	1,310,000	-	1,310,000	1,310,000
TBD	IPS Lift Station Upgrades	-	-	508,000	508,000	-	-	-	-	1,016,000	1,016,000
TBD	Bombay Lift Station Upgrades	-	-	-	355,500	1,066,500	-	-	-	1,422,000	1,422,000
TBD	Sweetwater & London Bridge Lift Station Upgrades	-	-	-	-	-	732,000	-	-	732,000	732,000
TBD	Annual Lift Station Upgrades							4,250,000	4,250,000	8,500,000	8,500,000
TBD	Area A Expansion	30,000	610,000	1,170,000	774,000	-	-	2,482,000	-	5,036,000	5,066,000
TBD	Area B Expansion	-	-	-	-	-	-	-	2,282,000	2,282,000	2,282,000
TBD	Area C Expansion	-	-	-	-	-	-	-	2,973,000	2,973,000	2,973,000
TBD	Area D Expansion	-	-	-	-	-	-	-	1,968,000	1,968,000	1,968,000
TBD	Pipeline Rehab and Replacement (Gravity and Manholes)	-	2,750,000	2,750,000	2,750,000	2,750,000	2,750,000	-	-	13,750,000	13,750,000
TBD	SCADA System Upgrades[a]		465,500	323,500	587,500	480,000	476,000	1,969,000	-	4,301,500	4,301,500

Table 8-1. Fiscal Year 2023–2040 Proposed Capital Improvement Plan

Project Number	Project Description	Prior ($)	22-23 ($)	23-24 ($)	24-25 ($)	25-26 ($)	26-27 ($)	27-32 ($)	32-40 ($)	FY 2023–2040 Total ($)	FY 2023–2040 Total with Prior ($)
TBD	Potable Reuse Feasibility Study	-	-	250,000	-	-	-	-	-	250,000	250,000
	Total	**1,767,300**	**12,022,280**	**7,751,500**	**9,665,000**	**5,546,500**	**10,648,000**	**13,141,000**	**$11,473,000**	**70,247,280**	**72,014,580**

[a] See the 2022 SCADA Master Plan for detail of individual projects by fiscal year. Annual fiscal year budgets include SCADA costs for capital facility replacement components, hardware improvements, and various software upgrades. The City may determine that some costs may be included in annual operations and maintenance budgets.

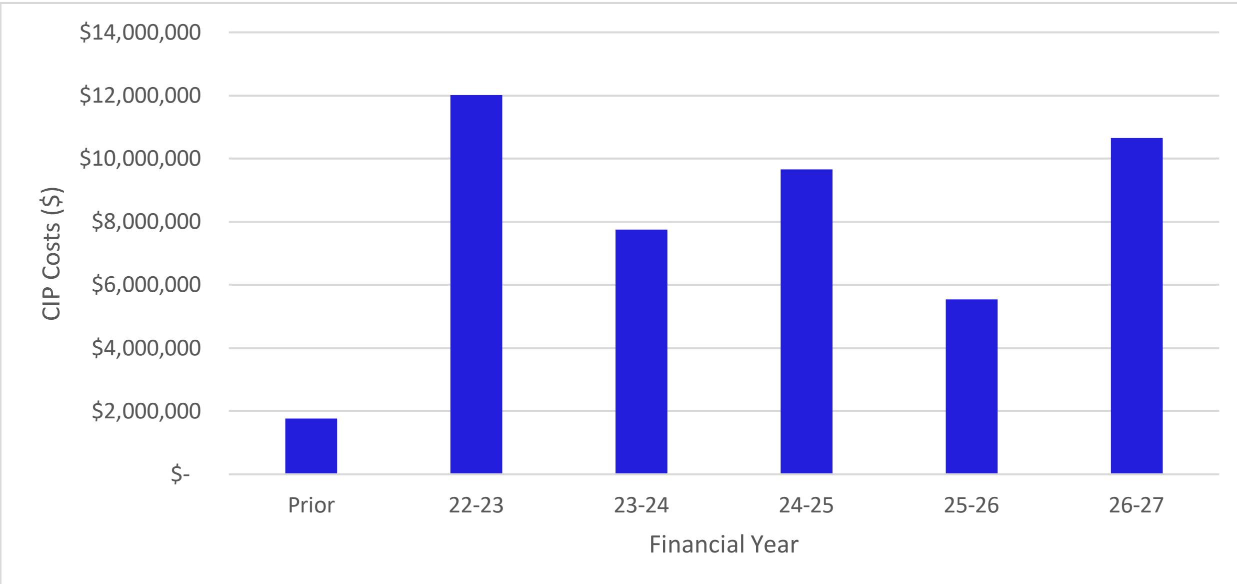
Notes:

All costs in 2022 dollars with no escalation.

Shading indicates items changed or added from previous CIP.

TBD = to be determined

Figure 8-1. 5-year CIP Costs



8.3.1 COLLECTIONS SYSTEM

Lift Station Improvements

As part of the regional pumping system optimization evaluation, upgrades were identified at four lift stations. The following are recommended improvements at each lift station:

- Sweetwater Lift Station Pump Replacement
- London Bridge Lift Station Pump Replacement
- Bombay Lift Station Pump Replacement and Bar Screen Installation
- Influent Pump Station Pump Replacement and Surge Improvements

Future Expansion Areas

The following four areas were identified for future system expansion:

- Area "A" includes a backbone deep sewer, trunk sewer extensions, a new sewer lift station, and force mains to service future growth in the area. Additionally recommend redirecting the Canterbury and Refuge lift stations to connect to the backbone deep sewer by extending a force main across Highway 95 and abandoning portions of the common force main per recommendations made in the North Regional Sub-Area Master Plan.
- Area "B" includes providing a backbone gravity sewer to the Refuge lift station.
- Area "C" includes providing a backbone gravity sewer to a new local pump station. The new pump station will convey flows through a new force main to the Centre lift station.
- Area "D" includes providing a new lift station, force main, and backbone gravity sewer to convey flows to the Island WWTP.

Pipeline Rehab and Gravity Replacement

The City maintains an annual fund for miscellaneous pipeline rehabilitation and replacement. This fund allows the City to be proactive in maintaining the collection system.

8.3.2 WASTEWATER TREATMENT PLANTS

Island WWTP

The following CIP for the Island WWTP is recommended to ensure that the WWTP is functioning properly and is always in compliance with its ADEQ permit.

- Preliminary treatment improvements, including a new headworks building that includes screens, grit removal, hydraulic capacity upgrades, odor control and electrical improvements. The existing headworks will reach the end of its useful life in the next 5 years and will need to be rebuilt.
- The new FEB is being currently negotiated by others and is already included in the City's CIP.
- Installation of submerged diffusers on the floor of the aeration basin instead of the rotating bridge. This will help to protect the concrete walls of the aeration basin
- Repairing and upgrading the traveling bridge filter to ensure reliable filtration.
- Complete replacement of the UV system once it reaches the end of its useful life.
- Repairing and rehabilitating Effluent Pond B and converting it into a percolation pond.

Mulberry WWTP

The following CIP for the Mulberry WWTP is recommended to ensure that the WWTP is functioning properly and is always in compliance with its ADEQ permit.

- The concrete structure of the aeration basins has developed cracks and needs to be repaired. Additionally, the City should evaluate installing aeration diffusers at the bottom of the basin to provide the necessary air for treatment. This will help protect the concrete structure.
- Complete replacement of the UV system once it reaches the end of its useful life.
- Upgrade the effluent pump systems and effluent pond.

North Regional WWTP

The following CIP for the North Regional WWTP is recommended to ensure that the WWTP is functioning properly and is always in compliance with its ADEQ permit.

- Installation of a grit removal system to reduce wear and abrasion of downstream mechanical equipment.
- The FEB has not been cleaned in years and has built up grit and other solid material. It is recommended that the City clean out the FEB to fully use the equalization capacity.
- Complete replacement of the UV system once it reaches the end of its useful life.

8.3.3 RECLAIMED/REUSE WATER SYSTEM

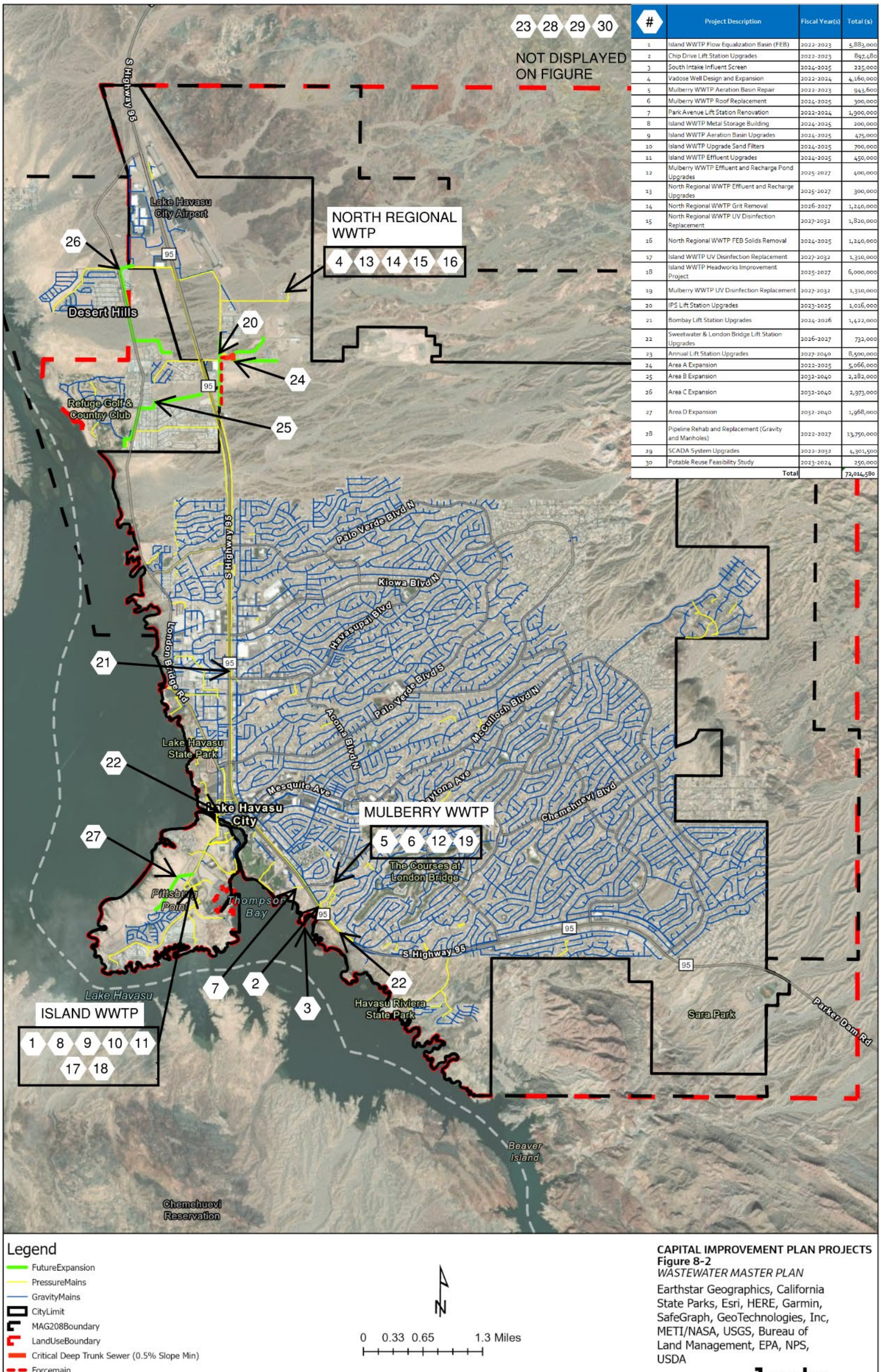
Two projects are recommended to ensure successful future reclaimed water management:

- Recommended additional vadose wells. Project to include approximately 2 mgd of capacity to be confirmed as part of the Vadose Study. Design and construction will occur beginning in FY 2021-2022, and conclude in FY 2024-2025.
- Recommend to undertake a reuse feasibility study to determine the applicability of direct potable reuse or other options available to the City. The study will commence in FY 2023-2024.

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

Figure 8-2 presents the planned CIP projects across the City.

Figure 8-2. Planned Wastewater CIP Projects



#	Project Description	Fiscal Year(s)	Total ($)
1	Island WWTP Flow Equalization Basin (FEB)	2022-2023	5,883,000
2	Chip Drive Lift Station Upgrades	2022-2023	897,480
3	South Intake Influent Screen	2024-2025	225,000
4	Vadose Well Design and Expansion	2022-2024	4,160,000
5	Mulberry WWTP Aeration Basin Repair	2022-2023	943,600
6	Mulberry WWTP Roof Replacement	2024-2025	300,000
7	Park Avenue Lift Station Renovation	2022-2024	1,900,000
8	Island WWTP Metal Storage Building	2024-2025	200,000
9	Island WWTP Aeration Basin Upgrades	2024-2025	475,000
10	Island WWTP Upgrade Sand Filters	2024-2025	700,000
11	Island WWTP Effluent Upgrades	2024-2025	450,000
12	Mulberry WWTP Effluent and Recharge Pond Upgrades	2025-2027	400,000
13	North Regional WWTP Effluent and Recharge Upgrades	2025-2027	300,000
14	North Regional WWTP Grit Removal	2026-2027	1,240,000
15	North Regional WWTP UV Disinfection Replacement	2027-2032	1,820,000
16	North Regional WWTP FEB Solids Removal	2024-2025	1,240,000
17	Island WWTP UV Disinfection Replacement	2027-2032	1,310,000
18	Island WWTP Headworks Improvement Project	2025-2027	6,000,000
19	Mulberry WWTP UV Disinfection Replacement	2027-2032	1,310,000
20	IPS Lift Station Upgrades	2023-2025	1,016,000
21	Bombay Lift Station Upgrades	2024-2026	1,422,000
22	Sweetwater & London Bridge Lift Station Upgrades	2026-2027	732,000
23	Annual Lift Station Upgrades	2027-2040	8,500,000
24	Area A Expansion	2022-2025	5,066,000
25	Area B Expansion	2032-2040	2,282,000
26	Area C Expansion	2032-2040	2,973,000
27	Area D Expansion	2032-2040	1,968,000
28	Pipeline Rehab and Replacement (Gravity and Manholes)	2022-2027	13,750,000
29	SCADA System Upgrades	2022-2032	4,301,500
30	Potable Reuse Feasibility Study	2023-2024	250,000
	Total		72,014,580

9. References

Arizona Commerce authority. n.d. "Population Projections." https://www.azcommerce.com/oeo/population/population-projections/.

Metcalf & Eddy, AECOM. 2014. *Wastewater Engineering Treatment and Resource Recovery*. Fifth Edition.

Lake Havasu City. 2003. Island WWTP Master Plan. Prepared by Burns and McDonnell.

Lake Havasu City. 2003. Mulberry WWTP Master Plan. Prepared by Burns and McDonnell.

Lake Havasu City. 2009. Wastewater Master Plan Update. Prepared by AMEC Earth and Environmental.

Lake Havasu City. 2014. Wastewater System Expansion Program Oversight Finalization. Prepared by Carollo Engineers.

Lake Havasu City. 2016. Lake Havasu City General Plan. Prepared by Clarion Associates. November.

Lake Havasu City. 2018. Water Master Plan Update.

Lake Havasu Metropolitan Planning Organization (MPO). Regional Transportation Plan. February 2022.

United States Census Bureau (USCB). 2020. "QuickFacts, Lake Havasu City, Arizona." https://www.census.gov/quickfacts/fact/table/lakehavasucitycityarizona/PST045221.

Appendix A
Flow Projection Development

Appendix A. Flow Projection Development

To analyze the capacities of the WWTPs, the wastewater flow projections were divided for the individual WWTPs. The historical wastewater flows were analyzed in Section 3 of the Wastewater Master Plan and were used to compute the 2040 annual average flows. Historical peaking factors for the maximum month (MM) and peak daily conditions were computed by dividing the MM flows or peak daily flows by the corresponding annual average day flows, and these are presented in Table A-1.

Table A-1. Historical Peaking Factors for the WWTPs

	Maximum Month/Average			Peak Daily/Average		
Year	Island Treatment Plant	Mulberry Treatment Plant	North Regional Treatment Plant	Island Treatment Plant	Mulberry Treatment Plant	North Regional Treatment Plant
2015	-	1.133	1.158	-	1.704	1.505
2016	-	1.168	1.180	-	1.617	1.449
2017	1.224	1.075	1.054	2.580	1.337	1.356
2018	1.170	1.252	1.073	2.160	1.629	1.936
2019	1.322	1.137	1.193	1.925	1.464	1.676
2020	1.159	1.250	1.095	1.756	1.512	1.291
2021	1.159	1.182	1.129	1.776	1.559	1.421
Average	1.207	1.171	1.126	2.039	1.546	1.519
95th Percentile	1.303	1.251	1.189	2.496	1.681	1.858
Maximum	1.322	1.252	1.193	2.580	1.704	1.936

The 2040 annual average day flows and 95th percentile peaking factors were used to the 2040 projected wastewater flows for the entire service region. The 95th percentile of these peaking factors will be used as a representative measure to project the influent flows. The historical peaking factors at the Island WWTP for 2015 and 2016 were very high and were excluded from the analysis. To compute the peak hourly flows, the average annual flow was multiplied by 2.8 to obtain the peak hourly flows to the individual WWTPs (Metcalf & Eddy, AECOM 2014).

Table A-2 summarizes 2040 projected values for the different flow conditions at the Island WWTP, the Mulberry WWTP, and the North Regional WWTP. It is observed that the Island WWTP service is anticipated to grow only marginally and therefore there is not much difference between the historical versus the projected flows. No growth is anticipated in the Mulberry WWTP service area and the same is reflected in the 2040 projected values for different flow conditions. Growth is anticipated in the North Regional WWTP service area and the same is reflected in the 2040 projected values when compared to the historical data.

The analysis assumed the current wastewater flow split between the three WWTPs would be applicable in 2040. The City has the ability to divert more or less flows to the WWTPs. If the flow split ratio changes, then the 2040 projected flows will need to be updated accordingly.

Table A-2. 2040 Projected Flows for the WWTPs

Treatment Plant	Annual Average Day (mgd)	Maximum Month (mgd)	Peak Daily (mgd)	Peak Hourly (mgd)
Island WWTP	0.89	1.16	2.22	2.49
Mulberry WWTP	1.11	1.39	1.87	3.11
North Regional WWTP	2.73	3.25	5.07	7.64

Appendix B
Load Projection Development

Appendix B. Load Projection Development

Projected Loads to Island WWTP

Analysis of the historical weekly average loads from January 2015 to December 2021 was carried out to determine peak conditions and associated peaking factors. Table B-1 presents a summary of historical 5-day biochemical oxygen demand (BOD_5), chemical oxygen demand (COD), total suspended solids (TSS), and ammonia-N loads.

Table B-1. Historical Influent Loads Observed at Island WWTP

Year	Annual Average Day Load (lb/d)				Maximum Month Load (lb/d)			
	BOD_5	COD	Ammonia-N	TSS	BOD_5	COD	Ammonia-N	TSS
2015	793	2,865	200	632	1,477	4,599	256	1,059
2016	985	3,439	232	1,193	1,426	6,477	347	2,072
2017	711	2,304	183	1,011	1,455	4,427	254	1,642
2018	972	3,824	229	1,523	1,479	5,884	284	2,253
2019	841	3,016	225	1,282	1,339	4,468	311	2,409
2020	996	2,917	242	1,177	1,432	3,904	293	3,111
2021	957	3,472	224	1,484	1,307	4,059	268	1,884
Average	894	3,120	219	1,186	1,416	4,831	288	2,062
95th Percentile	993	3,718	239	1,511	1,478	6,299	336	2,901
Maximum	996	3,824	242	1,523	1,479	6,477	347	3,111

lb/d = pound(s) per day

The historical peaking factors for the maximum month (MM) from January 2015 to December 2021 are presented in Table B-2. These factors were calculated by dividing the MM loads by the corresponding average load for each year. The average of the peaking factors will be used as a representative measure to project the influent loads.

Table B-2. Historical Influent Loads Peaking Factors at Island WWTP

Year	Maximum Month/Average			
	BOD_5	COD	Ammonia	TSS
2015	1.86	1.61	1.28	1.68
2016	1.45	1.88	1.50	1.74
2017	2.05	1.92	1.39	1.62
2018	1.52	1.54	1.24	1.48

Table B-2. Historical Influent Loads Peaking Factors at Island WWTP

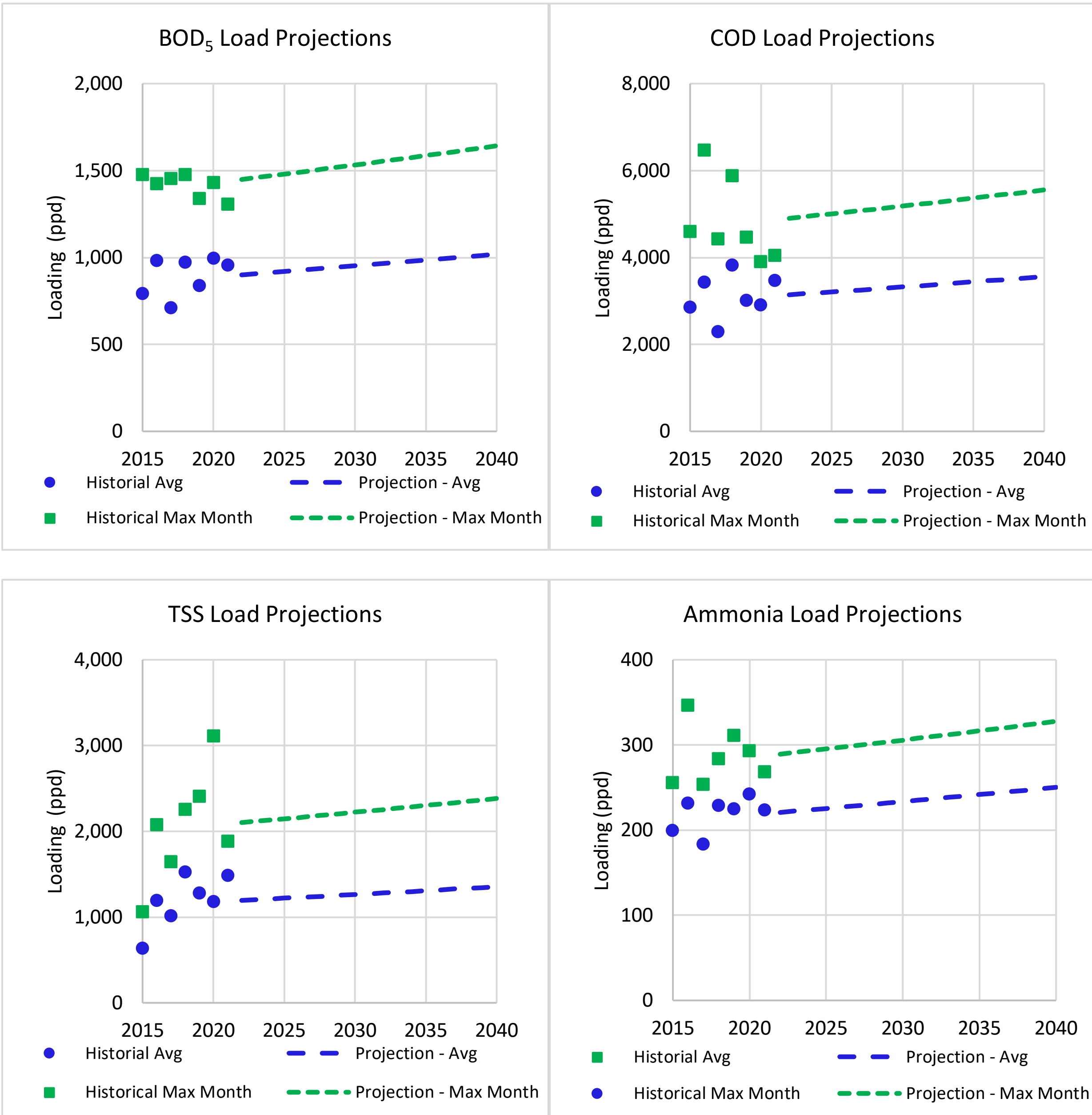
Year	Maximum Month/Average			
	BOD_5	COD	Ammonia	TSS
2019	1.59	1.48	1.38	1.88
2020	1.44	1.34	1.21	2.64
2021	1.37	1.17	1.20	1.27
Average	1.61	1.56	1.31	1.76
95th Percentile	1.99	1.91	1.46	2.41
Maximum	2.05	1.92	1.50	2.64

To develop the influent load projections to the Island WWTP for future conditions, the following methodology was employed:

- The averages from January 2015 to December 2021 were used as a starting point for future projections.
- The loads were escalated by compounding with an annual growth rate of 0.7 percent, which was obtained from the City's 2016 General Plan.
- The average MM peaking factors from 2015 to 2021 were multiplied by the projected average loads to estimate the projected MM loads.
- The influent load projections were developed for the baseline flows scenario, that is, the same wastewater flow split observed currently between the three plants was used for 2040 conditions.

The projected loads for the average annual day and MM are presented on Figure B-1 and in Table B-3. The 2009 Master Plan Update by AMEC Earth and Environmental noted that the flows and BOD_5 concentrations to the Island WWTP had decreased since the startup of the North Regional WWTP. BOD_5 is used to measure the amount of organic material present in the wastewater. This decrease may have resulted from the diversion of high-strength wastewater generated in the main parts of the City to other WWTPs. Island WWTP now serves the resort and beach communities at Lake Havasu, which typically generates wastewater with lower organic material. The COD/ BOD_5 ratio is much higher than the typical values of 1.8 to 2.2 (Water Environment Federation [WEF] Manual of Practice [MOP] 8), indicating that some of the BOD_5 may be consumed in the sewer collection systems because of low velocities and higher detention times. A certain amount of organic material is needed for the proper functioning of the WWTP, especially the secondary treatment process. Secondary treatment uses microorganisms to treat and remove wastewater constituents such as nitrogen and phosphorus. The organic material or BOD_5 serves as a food source for the microorganisms. The remaining constituents fall within the acceptable ranges (WEF MOP 8).

Figure B-1. Influent Load Projections for Island WWTP



ppd = pounds per day

Table B-3. Influent Load Projections for Island WWTP in 2040

Condition	BOD_5	COD	Ammonia	TSS
Annual Average Day (lb/d)	1,020 (137 mg/L)	3,537 (476 mg/L)	250 (34 mg/L)	1,354 (182 mg/L)
Maximum Month (lb/d)	1,643 (170 mg/L)	5,556 (574 mg/L)	328 (34 mg/L)	2,383 (246 mg/L)

Projected Loads to Mulberry WWTP

Analysis of the historical weekly average loads from January 2015 to December 2021 was carried out to determine peak conditions and associated peaking factors. Table B-4 presents a summary of historical BOD_5, COD, TSS, and ammonia loads.

Table B-4. Historical Influent Loads Observed at Mulberry WWTP

Year	Annual Average Day Load (lb/d)				Maximum Month Load (lb/d)			
	BOD_5	COD	Ammonia	TSS	BOD_5	COD	Ammonia	TSS
2015	1,238	4,482	290	2,045	1,576	5,217	398	3,273
2016	1,289	4,605	262	2,653	1,551	5,698	332	3,222
2017	1,296	5,138	286	3,008	1,762	7,120	373	4,086
2018	1,222	4,111	256	2,328	1,674	5,594	308	3,313
2019	1,052	5,369	356	2,854	1,341	6,207	427	3,643
2020	950	4,472	277	3,016	1,431	7,239	356	3,932
2021	1,204	5,975	351	3,296	1,637	6,733	448	3,598
Average	1,179	4,879	297	2,743	1,567	4,879	377	3,581
95th Percentile	1,294	5,793	354	3,212	1,735	5,793	442	4,039
Maximum	1,296	5,975	356	3,296	1,762	5,975	448	4,086

The historical peaking factors for MM from January 2015 to December 2021 are presented in Table B-5. These factors were calculated by dividing the MM loads by the corresponding average load for each year. The 95th percentile of the peaking factors will be used as a representative measure to project the influent loads. The projections were not developed with the average peaking factors because it created trend lines that were below the average yearly historical values for all the constituents (2015 to 2021).

Table B-5. Historical Influent Loads Peaking Factors at Mulberry WWTP

Year	Maximum Month/Average			
	BOD_5	COD	Ammonia	TSS
2015	1.27	1.16	1.37	1.60
2016	1.20	1.24	1.27	1.21
2017	1.36	1.39	1.30	1.36
2018	1.37	1.36	1.20	1.42
2019	1.28	1.16	1.20	1.28
2020	1.51	1.62	1.28	1.30
2021	1.36	1.13	1.28	1.09

Table B-5. Historical Influent Loads Peaking Factors at Mulberry WWTP

Year	Maximum Month/Average			
	BOD_5	COD	Ammonia	TSS
Average	1.34	1.29	1.27	1.32
95th Percentile	1.47	1.55	1.35	1.55
Maximum	1.51	1.62	1.37	1.60

To develop the influent load projections to the Mulberry WWTP for future conditions, the following methodology was employed:

- The averages from January 2015 to December 2021 were used as a starting point for future projections.
- The loads were escalated by compounding with an annual growth rate of 0.7 percent, which was obtained from the City's 2016 General Plan.
- The 95th percentile MM peaking factors from 2015 to 2021 were multiplied by the projected average loads to estimate the projected MM loads.
- The influent load projections were developed for the baseline flows scenario, that is, the same wastewater flow split observed currently between the three WWTPs was used for 2040 conditions.

The projected loads for the average annual day and MM are presented on Figure B-2 and in Table B-6. The Mulberry WWTP receives more residential and commercial wastewater flows than the Island WWTP. Higher BOD_5 loads are observed in the influent but the concentration is quite similar to the Island WWTP's. This indicates that the Mulberry WWTP may also be subject to the same flow aging issues as the Island WWTP. The COD/ BOD_5 ratio is much higher than the typical values of 1.8 to 2.2 (WEF MOP 8), indicating that some of the BOD_5 may be consumed in the sewer collection systems because of low velocities and high detention times.

Figure B-2. Influent Load Projections for Mulberry WWTP

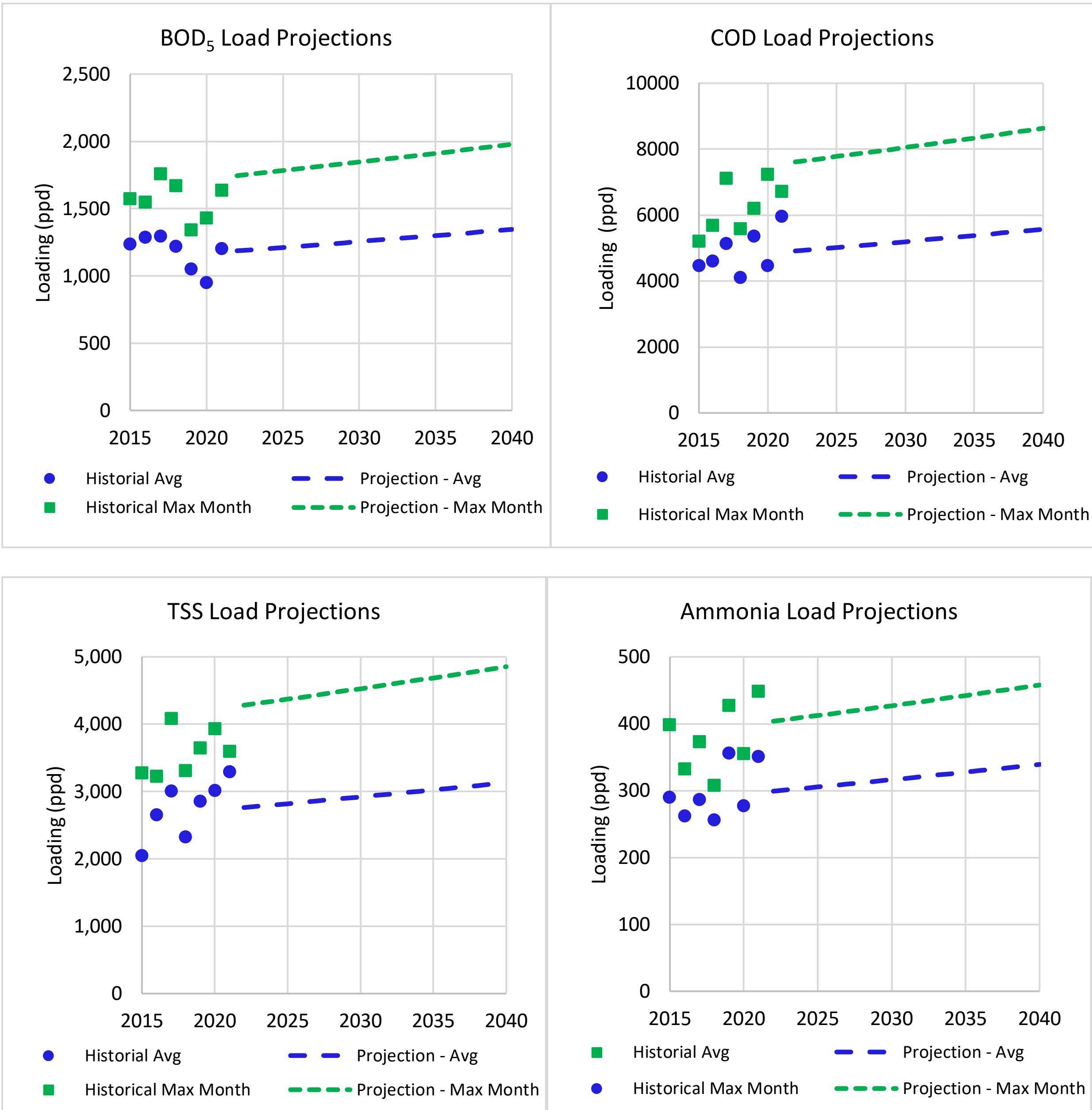


Table B-6. Influent Load Projections for Mulberry WWTP in 2040

Condition	BOD_5	COD	Ammonia	TSS
Annual Average Day (lb/d)	1,346 (145 mg/L)	5,532 (597 mg/L)	339 (37 mg/L)	3,312 (358 mg/L)
Maximum Month (lb/d)	1,978 (171 mg/L)	8,634 (744 mg/L)	458 (40 mg/L)	4,854 (419 mg/L)

Projected Loads to North Regional WWTP

Analysis of the historical weekly average loads from January 2015 to December 2021 was carried out to determine peak conditions and associated peaking factors. Table B-7 presents a summary of historical BOD_5), COD, TSS, and ammonia loads.

Table B-7. Historical Influent Loads Observed at North Regional WWTP

Year	Annual Average Day Load (lb/d)				Maximum Month Load (lb/d)			
	BOD_5	COD	Ammonia	TSS	BOD_5	COD	Ammonia	TSS
2015	1,925	5,031	523	1,618	2,425	7,734	781	2,317
2016	1,962	5,604	418	2,075	2,458	8,106	613	3,751
2017	1,777	6,714	427	3,203	2,255	8,236	582	4,333
2018	2,158	7,047	434	3,161	2,652	8,208	532	4,570
2019	1,975	8,260	508	4,660	2,351	10,076	665	5,632
2020	2,094	7,709	456	4,640	2,919	9,949	570	6,011
2021	2,362	9,289	547	4,591	2,913	13,289	704	6,872
Average	2,036	7,093	473	3,421	2,568	9,371	635	4,784
95th Percentile	2,301	8,981	540	4,654	2,917	12,325	758	6,614
Maximum	2,362	9,289	547	4,660	2,919	13,289	781	6,872

lb/d =

The historical peaking factors for MM from January 2015 to December 2021 are presented in Table B-8. These factors were calculated by dividing the MM loads by the corresponding average load for each year. The 95th percentile of the peaking factors will be used as a representative measure to project the influent loads. The projections were not developed with the average peaking factors because it created trend lines that were below the average yearly historical values for all the constituents (2015 to 2021).

Table B-8. Historical Influent Loads Peaking Factors at North Regional WWTP

Year	Maximum Month/Average			
	BOD_5	COD	Ammonia	TSS
2015	1.26	1.54	1.49	1.43
2016	1.25	1.45	1.47	1.81
2017	1.27	1.23	1.36	1.35
2018	1.23	1.16	1.22	1.45
2019	1.19	1.22	1.31	1.21
2020	1.39	1.29	1.25	1.30

Table B-8. Historical Influent Loads Peaking Factors at North Regional WWTP

Year	Maximum Month/Average			
	BOD_5	COD	Ammonia	TSS
2021	1.23	1.43	1.29	1.50
Average	1.26	1.33	1.34	1.43
95th Percentile	1.36	1.51	1.48	1.71
Maximum	1.39	1.54	1.49	1.81

To develop the influent load projections to the North Regional WWTP for future conditions, the following methodology was employed:

- The averages from January 2015 to December 2021 were used as a starting point for future projections.
- The loads were escalated by compounding with an annual growth rate of 0.7 percent, which was obtained from the City's 2016 General Plan.
- The 95th percentile MM peaking factors from 2015 to 2021 were multiplied by the projected average loads to estimate the projected MM loads.
- The influent load projections were developed for the baseline flows scenario, that is, the same wastewater flow split observed currently to the three WWTPs was used for 2040 conditions.

The projected loads for the average annual day and MM are presented on Figure B-3 and in Table B-9. Very low BOD_5 concentrations are observed in the wastewater going to the North Regional WWTP. Wastewater from the City is conveyed to the North Regional WWTP over a stretch of 3 to 4 miles of sewer pipe and force mains. Given the low water velocities, it can potentially create conditions suitable for the wastewater to become septic and start off-gassing hydrogen sulfide. The City adds Alkagen to reduce the occurrence of such conditions. However, the chemical added destroys a portion of the BOD_5, which is needed by the WWTP for its operation.

Figure B-3. Influent Load Projections for North Regional WWTP

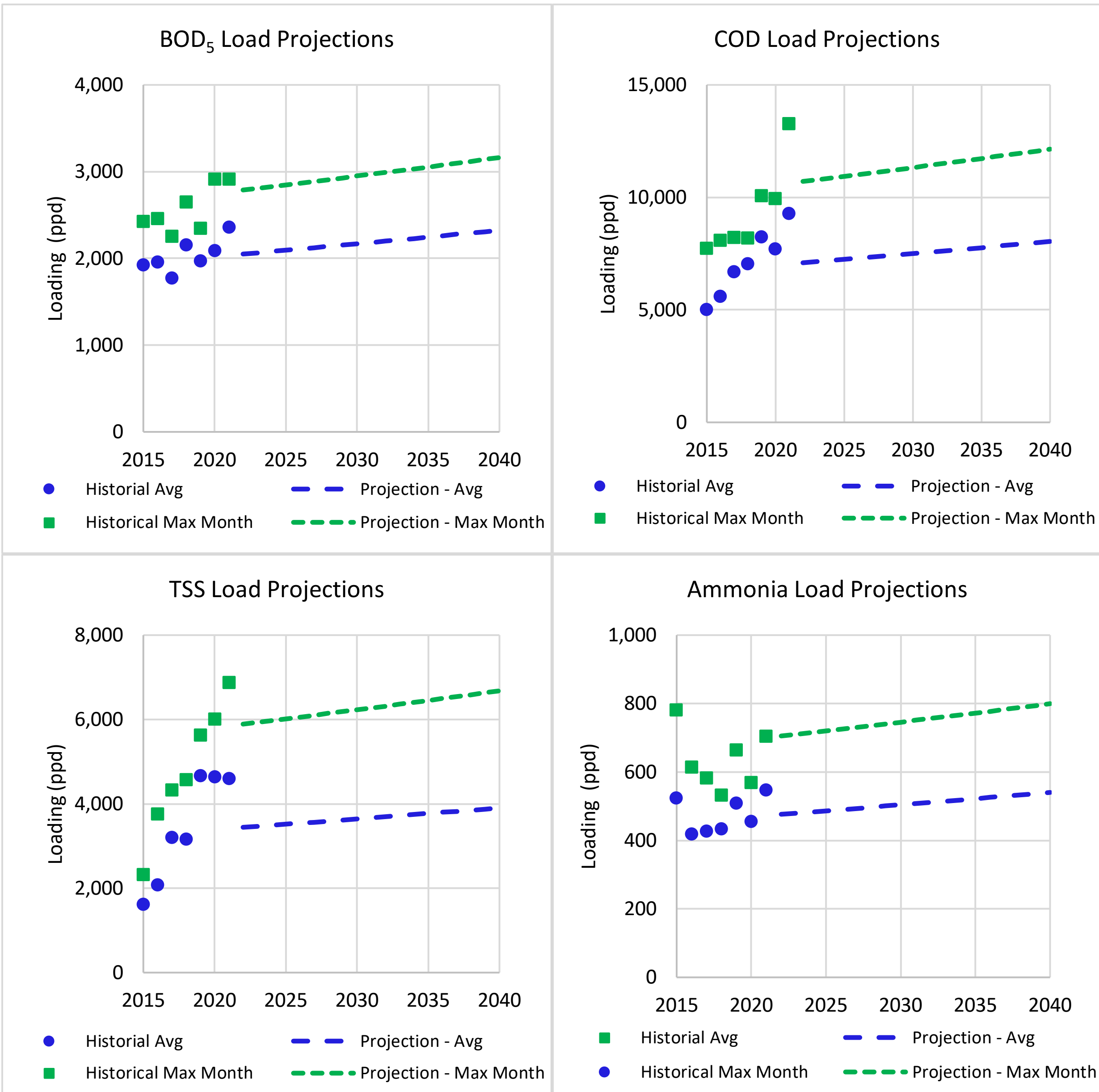


Table B-9. Influent Load Projections for North Regional WWTP in 2040

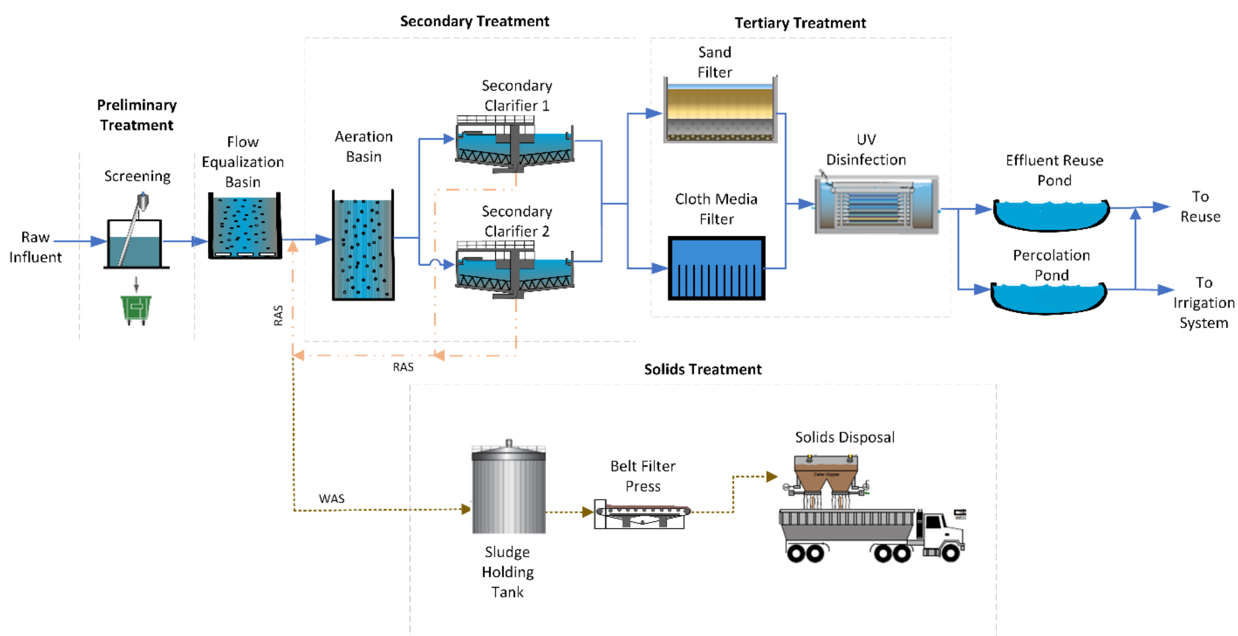
Condition	BOD_5	COD	Ammonia	TSS
Annual Average Day (lb/d)	2,325 (102 mg/L)	8,042 (353 mg/L)	540 (24 mg/L)	3,906 (172 mg/L)
Maximum Month (lb/d)	3,162 (117 mg/L)	12,144 (449 mg/L)	800 (30 mg/L)	6,679 (247 mg/L)

Appendix C
Detailed Capacity Analysis of Island WWTP

Appendix C. Detailed Capacity Analysis of Island WWTP

A detailed evaluation of each Island WWTP main unit process is described in the following sections of the Appendix. Each section discusses the current unit process and any operational problems or opportunities for optimization, provides an analysis of its capacity to treat flows and/or loads through the planning period, and is followed by recommended modifications. The liquids treatment is discussed first, progressing from influent raw sewage to filtered and disinfected effluent, followed by solids handling processes. The general process flow schematic is presented on Figure C-1.

Figure C-1. Island WWTP General Process Flow Schematic



Preliminary Treatment

Description of Existing Facilities

Preliminary wastewater treatment at the Island WWTP is housed in the Headworks Building and consists primarily of static screens to remove rags and large debris. There are five static screens that are operated in a parallel configuration. Four of the static screens are 72 inches wide and the fifth screen is 120 inches wide. The screens have a 0.06-inch wire mesh opening to screen the wastewater. The wire mesh was replaced in 2021. Materials removed by the screens are moved by a conveyor to a compactor and then discharged into a storage hopper for final disposal at the City's landfill.

Capacity Evaluation

The capacity of the screens was evaluated for their ability to treat peak hourly flows at the 2040 conditions. The capacity of the system is based upon the largest screen being out of service. The estimated peak hourly flow to the Island WWTP in 2040 is 2.49 million gallons per day (mgd).

The existing screening system characteristics are summarized in Table C-1. Based on the 2040 peak hourly flow projections for Island WWTP, the screening equipment is estimated to have sufficient capacity.

Table C-1. Screening Design Criteria

Design Criteria	Static Screens
Number of screens	5
Mesh size, inches	0.06
Screen width, inches	Four screens are 72 inches wide One screen is 120 inches wide
Capacity of each screen, mgd	Four screens which are 72 inches wide are rated at 1 mgd One screen which is 120 inches wide is rated at 1.4 mgd
Firm capacity, mgd[a]	4.0
Installed capacity, mgd	5.4

[a] The firm capacity assumes the 120-inch-wide screen is offline.

Recommendations

During the site walkthrough, the frames of the static screens were showing signs of corrosion and will likely need to be replaced. The wire mesh sections were replaced in 2021 and are in good condition. The screens are anticipated to function for another 5 years before they reach their end of useful life and will need to be completely replaced. A portion of the Headworks Building along the southern wall was noted to be experiencing settlement and cracking, reportedly because of the previous saturation of the surrounding soil resulting from a pipe break. A portion of the structure is supported by external wing walls, and cracking and settlement are observed in the walls as shown on Figure C-2. The Headworks Building needs additional structural repairs to avoid further deterioration.

The preliminary treatment at the Island WWTP does not currently include grit removal equipment. Common industry practice is for grit removal to precede secondary treatment in those treatment plants that do not have primary clarification. Removal of grit prevents unnecessary abrasion and wear of mechanical equipment, grit deposition in pipelines and channels, and accumulation of grit in the flow equalization basin (FEB), aeration basins, and sludge holding tanks. Jacobs recommends including a grit removal system in future Headworks Building upgrades.

The electrical room housing the Headworks Building's motor control center (MCC) showed signs of H_2S corrosion. It is recommended that the City improve ventilation to protect the electrical gear and wiring from further corrosion.

The existing screens are installed at a lower elevation, which prevents the City from using the full capacity of the new FEB. If the water level in the FEB is more than the screen elevation, it will back flow into the Headworks Building, causing it to flood.

Because of the extensive repairs and upgrades needed, it is recommended that the City construct a new Headworks Building that includes new screens, grit removal, odor control, and electrical improvements. The new screens would be installed at a higher elevation than the maximum water elevation in the FEB, allowing the WWTP to fully use the FEB volume available for flow equalization.

Figure C-2. Cracks Observed in the South Wall of the Headworks Building

Flow Equalization Basin

Description of the Facility

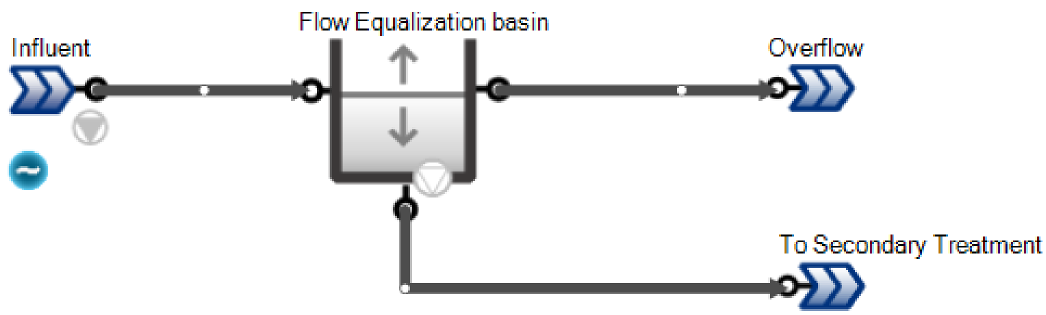
The former FEB was taken offline in 2001 and was later demolished. The City is constructing a new FEB, which will have a capacity of 500,000 gallons.

Capacity Evaluation

The capacity of the FEB was analyzed using Dynamita's SUMO, a commercially available, whole-plant, dynamic simulator. A simplified version of the FEB operation was simulated in Sumo at the 2040 peak day conditions as shown on Figure C-3. The model did not include any other WWTP processes. The following inputs were used in the model:

- The volume of the FEB was set to 500,000 gallons
- Initial volume at start of simulation 100,000 gallons.
- 2040 Peak day influent flows to the Island WWTP were used. Peak day flows typically only occur once in a year, but flows were simulated for 2 peak days to be conservative.
- Flows were limited to 2.5 mgd as this is the maximum flow that the disinfection system can handle.
- Flows to the secondary treatment will be conveyed by gravity.

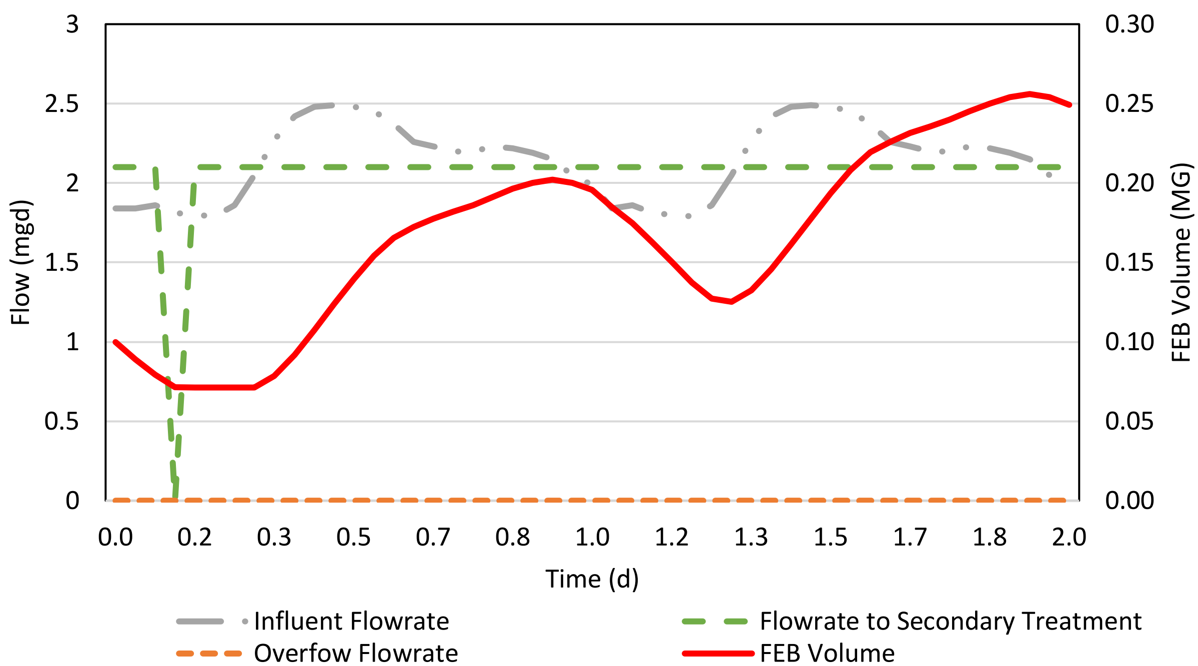
Figure C-3. Sumo Model Configuration for the FEB at Island WWTP



Note: No other process units were simulated.

The goal of the simulation was to observe the impact on the storage volume of the FEB at peak day flows and ensure there is no overflow. The overflow in the model represents any excess flows that cannot be stored. Because the flows to the Island WWTP are the lowest among the three WWTPs, the FEB has plenty of capacity. The modeled flows to secondary treatment were reduced to 2.1 mgd and the FEB was still able to process the 2040 peak day flows with no overflow. Figure C-4 shows the response curve that was generated from the model at 2.1 mgd output from the FEB, but the flows can be increased up to 2.5 mgd if needed.

Figure C-4. Response Curve from the Sumo Model – Flow to Secondary Treatment at 2.1 mgd



Recommendations

The new FEB, when complete, will have sufficient capacity to handle the 2040 peak day flows, based on the analysis presented in this section.

Secondary Treatment

Secondary treatment at the Island WWTP encompasses biological treatment using an activated sludge process and settling out the sludge using a secondary clarification process. The Island WWTP has one aeration basin and two secondary clarifiers for secondary treatment.

Description of Existing Facilities

Aeration Basins

The aeration basin was designed by Schreiber LLC, which is now part of the Parkson Corporation. Aeration Basin 1 was constructed in 1986 as the Schreiber GRD model. The GRD model had two separate zones: aerobic and anoxic. The basin was upgraded to the GRO model in 2003 and includes a full diameter rotating bridge. The GRO model is a continuous sequencing reactor (CSR) system and can achieve nutrient removal in a single basin without the need for separate aerobic/anoxic zones. The system has two distinctive phases, aerobic and anoxic, occurring in a sequence to remove carbon and nitrogen.

In the aerobic phase, air is bubbled through the mixed liquor to allow nitrification to occur. The dissolved oxygen (DO) is typically set between 0.8 and 1.1 milligrams per liter (mg/L). The air is supplied through fine membrane diffusers, which are mounted on a rotating bridge that traverses the aeration basin. The aerobic phase lasts for about 45 minutes to an hour. Then the air is turned off, and the basin turns anoxic, which allows denitrification to occur. The anoxic phase occurs for a time similar to that of the aerobic phase.

Aeration Blowers

The aeration blowers are located in the Blower Building. There are four Kaiser rotary lobe blowers that supply air to the headers mounted on the rotating bridge. Operations staff indicated that they believe there are air leaks in piping outside of the Blower Building.

Secondary Clarification

Secondary clarification is provided by two clarifiers. Clarifier 1 is 93 feet in diameter and Clarifier 2 is 58 feet in diameter. The original wastewater treatment tanks were modified in 1986 to serve as secondary clarifiers. Clarifier 2 underwent further repairs and rehabilitation in 2006. The sidewater depth (SWD) is approximately 12 feet for both clarifiers. The secondary clarifiers were also designed by Schreiber. Under normal operations, Clarifier 1 is typically used, while Clarifier 2 is on standby.

Mixed liquor from the aeration basin enters the clarifier through a center feed well and the velocity of the wastewater is reduced to allow solids to settle out at the bottom of the clarifier. The clear effluent flows over the V-notch weir and then flows to the secondary effluent pump station. The scum collected from the clarifiers is pumped to the sludge holding tank.

Return Activated Sludge Pump Station

The settled sludge at the bottom of the secondary clarifiers is pumped back to the aeration basins using the return activated sludge (RAS) pumps. The RAS pump station has two screw-type pumps provided by Schreiber. The RAS pumps were replaced in 2008.

Secondary Effluent Pump Station

The clarified effluent flows by gravity to the Secondary Effluent Pump Station, where it is pumped to the tertiary treatment system, which includes filtration and ultraviolet (UV) disinfection. There are three Simmons vertical turbine pumps located at the Secondary Effluent Pump Station.

Capacity Evaluation

Aeration Basin

The design criteria for the aeration basin were obtained from the Schreiber Corporation and are included in Attachment C1. The design criteria are summarized in Table C-2.

Table C-2. Aeration Basin Sizing

Design Criteria	Value
Geometry	Circular
Diameter, feet	167
Sidewater depth, feet	17
Volume, million gallons	2.79
Maximum Month Flow, mgd	2.5
Maximum BOD_5 Loading, ppd	6,255 (300 mg/L)
Maximum TSS Loading, ppd	6,255 (300 mg/L)
Maximum Ammonia Loading, ppd	959 (46 mg/L)
Hydraulic Retention Time at Maximum Month Flow, hours	26.74
BOD_5 Loading Rate, lb. BOD_5/1,000 ft^3	16.80
Design MLSS concentration, mg/L	3,600
Design SRT, days	28

BOD_5 = 5-day biochemical oxygen demand
ft^3 = cubic feet
lb = pound(s)
MLSS = mixed liquor suspended solids
ppd = pounds per day
SRT = solids retention time

The overall capacity of the aeration basin is dependent on several factors, including aeration capabilities, overall basin volume, primary effluent mass loading, and the solids retention time or the MLSS concentrations. In this analysis, no process model was developed to provide a detailed estimate of the capacities to treat future flows and loads. Instead, the BOD_5 loading rate and the various influent loads were compared against the design loads to provide a capacity estimate. If the City has concerns about the secondary treatment performance or is looking to expand or modify the secondary treatment, Jacobs recommends that the City conduct a detailed analysis of the secondary treatment using more detailed influent characterization and dynamic simulations.

The capacity of the aeration basin was evaluated using the 2040 maximum month (MM) influent loads as presented in Section 6.1.2. The estimated BOD_5 loading rate is 4.5 lb. BOD_5/1,000 ft^3, which is well below the design criteria, indicating that the aeration basin has sufficient capacity. The BOD_5, TSS, and ammonia loading to the Island WWTP are well below the design criteria. It is important to note that because of the long retention times in the sanitary sewer collection system, the BOD_5 degrades within the sewer pipes resulting in less BOD_5 reaching the Island WWTP. The lack of carbon can severely impact the denitrification process. However, because of the long sludge age in the aeration basin it is possible that some of the slowly biodegradable carbon is consumed during nitrification. It is recommended that the BOD_5 and ammonia loadings be closely monitored to minimize any impacts on the nitrogen removal process.

Aeration Blowers

The existing blower system characteristics are summarized in Table C-3.

Table C-3. Aeration System Sizing

Design Criteria	Value
Number of Units	4
Airflow each Blower, scfm	2,448
Discharge Pressure, psi	8
Motor, hp	150

hp = horsepower
psi = pounds per square inch
scfm = standard cubic feet per minute

The theoretical air demand for carbon and nitrogen removal at the 2040 MM condition was calculated to evaluate the capacity of the existing blowers. The calculation is presented in Table C-4. The parameters used in the calculations were similar to what Schreiber had used to estimate the airflow demand during the aeration basin design. The aeration system at the Island WWTP has sufficient capacity at the 2040 MM conditions because the BOD_5 and ammonia loading are well below the design criteria.

Table C-4. 2040 Air Demand for Maximum Month Flow and Loads

Parameter	Schreiber Design Criteria	At 2040 Maximum Month Condition	Notes
O_2 required for BOD_5, lb O_2/day	9,383	2,465	Assumed 1.5 lb O_2/lb of BOD_5
O_2 required for nitrification, lb O_2/day	4,412	1,509	Assumed 4.6 lb O_2/lb of ammonia
Total AOR, lb O_2/day	13,794	3,974	Credits for denitrification were not considered to generate a conservative value
SOTR, lb O_2/hour	1,107	320	
Airflow, scfm	4,239	1,221	

AOR = actual oxygen requirement
O_2 = oxygen
SOTR = standard oxygen transfer rate

Secondary Clarification

Table C-5 provides a summary of the secondary clarification system at the Island WWTP.

Table C-5. Secondary Clarifier Design Criteria

Parameter	Value
Geometry	Circular
Diameter, feet	Clarifier 1 – 93 Clarifier 2 – 58
Surface Area, ft^2	Clarifier 1 – 6,793 Clarifier 2 – 2,642
Sidewater depth, feet	12.5
SOR, gpd/ ft^2 [a]	368
SVI, mL/g[b]	150

[a] The surface overflow rate is the same for average flow and at peak flow conditions because there is flow equalization.

[b] SVI data was analyzed from 2015 to 2021, and the 95th percentile value was used for the capacity analysis.

ft^2 = square feet
gpd = gallons per day
mL/g = milliliter per gram
SOR = surface overflow rate
SVI = sludge volume index

The capacity of the secondary clarifiers is defined by two criteria:

- Solids loading rate (SLR) on the secondary clarifier: This is controlled by both the MLSS concentration of the system, the solids settleability, and the hydraulic flow through the system. MLSS is controlled by the operating SRT and the solids load to secondary treatment.
- Hydraulic loading rate (HLR) on the secondary clarifier: During wet weather events, flow rates through the secondary clarifiers can be high enough to cause solids not to settle or scour solids from the sludge blanket surface.

The analysis of the SLR on the secondary clarifiers was performed using Jacobs' PClarifier tool. PClarifier uses solid flux theory and state point analysis (SPA) to determine the limiting conditions on the clarifier. SPA is based on solids mass balances around the clarifier. The solids flux curve represents the settling characteristics of a particular MLSS concentration per unit area of the clarifier. The overflow line starts at the origin and the slope of the line is equal to the SOR. The underflow line starts at the calculated SLR on the y-axis and the slope of this line is equal to the RAS flow rate. Where the underflow line intersects with the x-axis, that represents the expected RAS concentration. The intersection of the overflow line and underflow line is defined as the state point. The state point represents the operating point of the clarifier and helps to determine if the clarifier is underloaded, critically loaded or overloaded (Water Environment Federation [WEF] Manual of Practice [MOP] Number 8). For this analysis, the following assumptions were used:

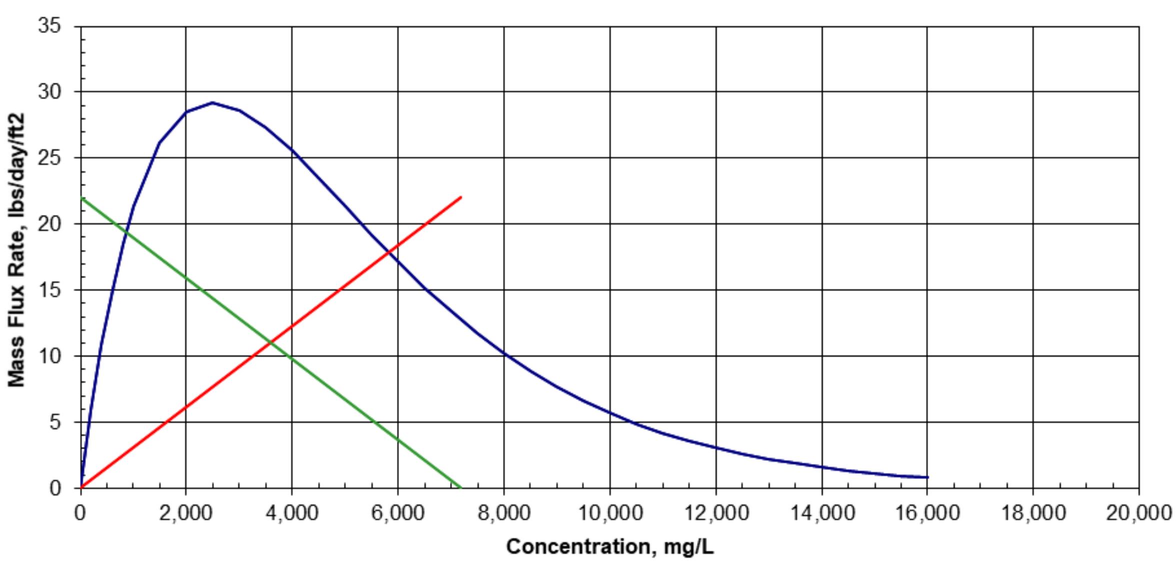
- Design SOR is 368 gpd/ft^2
- MLSS concentration of 3,600 mg/L

- RAS flow was assumed to be equal to the influent flow to the secondary clarifier
- SVI was set to 187 mL/g, which was the 95th percentile value observed in the historical data (2015 to 2021).
- The Daigger 1995 SVI model was used for flux correlation
- Analysis was conducted for two conditions:
  - Only Clarifier 1 is operational
  - Only Clarifier 2 is operational

For the SPA analysis, the maximum allowed SLR is only 80 to 90 percent of the theoretical SLR. This is done to account for inefficiencies associated with the clarifiers. Because the clarifiers at the Island WWTP are of shallow type, the maximum SLR is set to 80 percent. The SPA for the secondary clarifiers for the two conditions is shown on Figure C-5 and Figure C-6. The results from the SPA analysis are presented in Table C-6. When Clarifier 1 is in operation, there is sufficient capacity to treat flows up to 2.5 mgd. However, if Clarifier 1 has to be taken offline for maintenance, Clarifier 2 does not have sufficient capacity to handle flows of 2.5 mgd with SLR being over 100 percent. The flow rate was then reduced to 1.6 mgd, which is estimated to be the maximum allowable flow into the secondary treatment when only Clarifier 2 is operational. The clarifier will be operating at capacity in terms of SLR and SOR. Operating at this condition is only recommended for short durations.

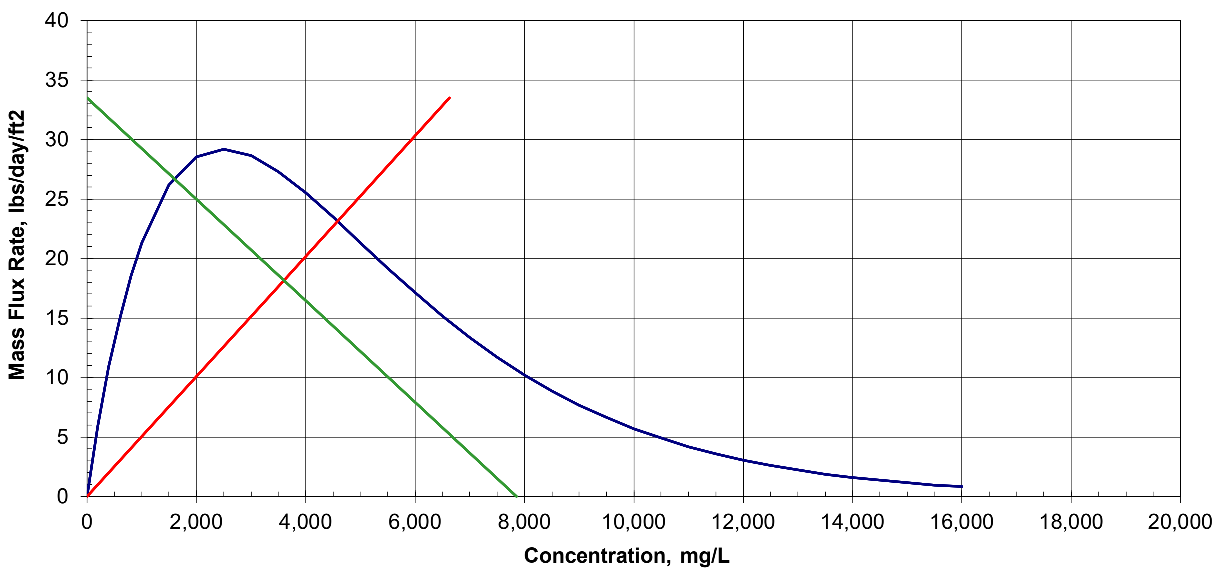
Based on the SPA, the total capacity of the secondary clarification system is estimated to be 2.5 mgd. However, the capacity of this system will be based on the firm capacity and is estimated to be 1.6 mgd. In the event that Clarifier 1 has to be taken offline for maintenance, the City has the ability to store some of the influent wastewater in the FEB or divert wastewater to either the Mulberry WWTP or the North Regional WWTP. Therefore, a third secondary clarifier is not required at Island WWTP.

Figure C-5. State Point Analysis – Only Clarifier 1 Operating (Flow Rate is 2.5 mgd)



blue line = solids flux curve; red line = overflow line; green line = underflow line

Figure C-6. State Point Analysis – Only Clarifier 2 Operating (Flow Rate Reduced to 1.6 mgd)



blue line =solids flux curve; red line = overflow line; green line = underflow line

Table C-6. Secondary Clarifier Performance at 2040 Maximum Month Conditions

Input Parameter	Clarifier 1	Clarifier 2
Influent flow, mgd	2.5	1.6
RAS flow, mgd	2.5	1.3[a]
SOR, gpd/ft²	368	606
Secondary clarifier applied SLR lb/d/ft²	22.08	33.49
Secondary clarifier limiting SLR, lb/d/ft²	34.73	42.64
Secondary clarifier applied SLR to limiting SLR, %	63%	79%

[a] SPA analysis indicated a reduced RAS flow rate when Clarifier 2 is operating.

Return Activated Sludge Pump Station

The design criteria for the RAS Pump Station are summarized in Table C-7. The RAS pumps appeared to be in good condition during the site walk through. They have sufficient capacity to handle MM flows coming into the secondary treatment process.

Table C-7. RAS Pump Station Design Criteria

Design Criteria	RAS Pump Station
Number of Pumps	2
Type	Adjustable tube mounted screw

Table C-7. RAS Pump Station Design Criteria

Flow capacity each pump, mgd	2.66 at 5-foot TDH
Motor, hp	7.5 hp

TDH = total dynamic head

Secondary Effluent Pump Station

The Secondary Effluent Pump Station design criteria are summarized in Table C-8. The pump station was evaluated for its ability to treat peak hourly flows. The total capacity is approximately 5.4 mgd and the firm capacity is 2.9 mgd. Because the flow to the Secondary Treatment will be equalized in the near future, the maximum flow that the pumps need to handle is only 2.5 mgd. The pumps have sufficient capacity to be able to handle the flows with the largest pump offline.

Table C-8. Secondary Effluent Pump Station Design Criteria

Design Criteria	Secondary Effluent Pump Station
Number of Pumps	3
Type	Vertical Turbine
Flow Capacity, mgd	Pump 1 – 1.45 Pump 2 – 1.45 Pump 3 – 2.50
Motor, hp	Pump 1 and Pump 2 – 15 Pump 3 – 30

Recommendations

Jacobs' high-level analysis of secondary treatment at the Island WWTP indicates that there is sufficient capacity to treat the 2040 MM flows and loads. However, the analysis performed in this Wastewater Master Plan is high-level, and it is recommended that the City carry out a more detailed analysis of their secondary treatment to determine the actual capacity. It is also recommended that the operators pay close attention to the BOD_5 and ammonia loadings to the Island WWTP, to ensure that there are no disruptions to the nutrient removal processes.

The RAS Pump Station is anticipated to have sufficient capacity to meet the 2040 MM flows and loads. The Secondary Effluent Pump Station appeared to be in good condition during the site visit, and the operators have reported that the pumps are in good operating condition as well.

Operations staff suspect that there are air leaks in piping between the Blower Building and the center of the Schreiber activated sludge unit. These potential air leaks should be investigated, located, and repaired, if present.

Tertiary Filtration

Description of Existing Facilities

Tertiary filtration at the Island WWTP is provided by one sand filter and one cloth disk filter and is used to remove filterable solids from the secondary effluent.

The sand filter is divided into two concrete retaining basins, filter beds with under-drains, and a traveling bridge with a hood and a backwash pump. This filtration unit was first installed in 1968 and was subsequently retrofitted to include an automatic backwash system in 1985. The sand media is typically changed once every 2 years. The City also installed a fixed disk cloth filter by Five Star Filtration in 2016, which has been out of service since June 2018. The filter disk remains stationary while the center shaft rotates the backwash arms to remove solids from the filter. The two sand filters are typically operated in a parallel configuration. However, the two different filter types can be operated only in a parallel configuration. Currently, the backwash water generated from both the filters is sent back to the Headworks Building. The backwash water will be sent to the FEB after it is commissioned.

Capacity Evaluation

The existing filtration system characteristics are summarized in Table C-9. The sand filter design criteria were obtained from the 2003 Master Plan (Lake Havasu City 2003). The capacity of the tertiary filtration system was evaluated for its ability to treat peak hourly flow. Since the flow to Tertiary Filtration will be equalized in the near future, the maximum flow that the filters need to handle is only 2.5 mgd. This is the full capacity of the current filtration system with the cloth disk filters offline, meaning there is currently no redundancy to meet the design flow rate. The filter systems have sufficient capacity to handle the peak hourly flows in 2040 with the cloth disk filters online, therefore Jacobs recommends that they be repaired soon.

Table C-9. Tertiary Filtration Design Criteria

Design Criteria	Sand Filter	Cloth Disk Filter
Number of Filters	2	1
Average Design Flow each, mgd	-	0.75
Maximum Design Flow each, mgd	1.25	2.60
Filter Area, ft^2	450	288
HLR at Average Flow, gpm/ft^2	-	1.81
HLR at Maximum Design Flow, gpm/ft^2	1.93	6.50
Wash water rate, gpm/ft^2	12	-

gpm = gallon(s) per minute

Recommendations

The capacity analysis of the tertiary filtration system indicates that this system has sufficient capacity to handle the 2040 peak hourly flows as the flows coming into the WWTP are equalized and then sent to the downstream processes.

Cracks were observed in the walls of the sand filter (shown on Figure C-7) during the site walkthrough. As the sand filters are over 50 years old, it is recommended that the City pay close attention to the structural integrity of concrete and steel associated with the filters and make any necessary repairs.

The Five Star cloth disk filter has been inoperable since 2018 due to issues with the cloth media and internal filter flows. The City has received a quote for the rehabilitation of the cloth disk filter. Jacobs recommends that the cloth filter be repaired and used along with the sand filters for daily operations.

Figure C-7. Cracks Observed in the Concrete Walls of the Sand Filter

Disinfection

Description of Existing Facilities

The Island WWTP uses a Trojan UV3000Plus system to disinfect the filtered effluent to meet permit requirements for Class A+ Reclaimed Water. UV radiation can penetrate through the cell walls of remaining pathogenic organisms and damage the DNA or RNA strands. This results in the organism being unable to perform cellular functions, ultimately leading to the death of the organism. The disinfected effluent is then discharged to the storage ponds for reuse or percolation into the groundwater table, or to the non-potable process water system.

Capacity Evaluation

The UV system consists of 2 channels, 3 banks per channel, and 36 UV lamps per bank. The flow into the two channels is controlled by sluice gates. Space has been reserved to install a third channel of UV modules in the future to treat increased future flows. The UV disinfection design criteria are summarized in Table C-10. Based on the 2040 peak hourly flow projections for the Island WWTP, the UV disinfection system has sufficient capacity when all UV channels are online, as the flows coming into the WWTP are equalized and then sent to the downstream processes. If a channel is offline because of an emergency or maintenance, the overall capacity of the WWTP is reduced to 1.25 mgd. The Island WWTP could use its FEB to store the excess wastewater or divert some of the excess flows to either the Mulberry WWTP or the North Regional WWTP.

Table C-10. Disinfection Design Parameters

Design Criteria	UV Disinfection System
Type	Open Channel
Number of Channels	2
Number of Modules	3
Number of Lamps per Module	36
UV Transmittance	65% at 254 nanometers
Maximum Design Flow, Total Capacity	2.5 mgd
Disinfection Standards	*E. coli:* single maximum of 15 most probable number per 100 milliliters

Recommendations

The capacity analysis of the UV disinfection system indicates that the system has sufficient capacity to handle the 2040 peak hourly flows with both channels operating, because it will be equalized in the FEB in the near future. The capacity of the WWTP decreases if one of the UV channels is offline.

The UV disinfection system was installed in 2004 and has been in service for nearly 18 years. The typical lifespan of the Trojan UV3000Plus is 20 to 25 years with proper maintenance of the system. The system appeared to be in good condition during the site walkthrough. The operators indicated that they use third-party UV lamps instead of the recommended Trojan lamps because of budgetary constraints. This may result in performance issues such as reduced bulb life, as well as reduced UV dosage to properly disinfect the wastewater, potentially leading to noncompliance with ADEQ's water quality standards for Class A+ reuse. Jacobs recommends using Trojan bulbs for optimal UV system performance. Additionally, the City is working with Trojan to replace the lamp sleeve cleaning system, hydraulic controls, and effluent gates. Despite these maintenance activities, the City should anticipate having to replace the system in the next 10 years.

Water Reuse System

The filtered and disinfected effluent flows by gravity to the pond system at the Island WWTP. The system consists of two different types of ponds: reuse ponds and percolation ponds. The reuse ponds have two lined ponds, Pond A and Pond B, which are used to store the reuse water from the Island WWTP and can also receive reuse water from the Mulberry WWTP or the North Regional WWTP. These ponds are hydraulically connected. Percolation ponds include two ponds, Pond C and Pond D, which are used to recharge the groundwater table beneath the Island WWTP. The percolation ponds are not hydraulically connected with the reuse ponds, but there is piping installed to help transfer water between the two pond systems.

Pond A and Pond B have a usable volume of 0.8 million gallons each, and Pond C and Pond D have usable volumes of 2.25 million gallons and 1.5 million gallons, respectively. Pumps are used to send reuse water to the golf course and into the non-potable water system. Fairbanks Morse pumps are used to send water to the Mulberry WWTP or the North Regional WWTP. The design criteria for both pump stations are summarized in Table C-11 and Table C-12, respectively.

Table C-11. Golf Course Pump Station Design Criteria

Design Criteria	Golf Course Pump Station/Non-potable Water System
Number of Pumps	3
Type	Vertical Turbine
Flow Capacity, gpm	1,700 at 258 feet TDH
Configuration	2 Duty/ 1 Standby
Motor, hp	2 at 15, 1 at 30

Table C-12. Reuse Pump Station to Mulberry WWTP and North Regional WWTP Design Criteria

Design Criteria	Reuse Pump Station to Mulberry WWTP or North Regional WWTP
Number of Pumps	2
Type	Vertical Turbine
Flow Capacity, gpm	2,033 at 227 feet TDH

Table C-12. Reuse Pump Station to Mulberry WWTP and North Regional WWTP Design Criteria

Design Criteria	Reuse Pump Station to Mulberry WWTP or North Regional WWTP
Configuration	1 Duty/1 Standby
Motor, hp	2 at 15, 1 at 30

During the site walkthrough, Jacobs personnel observed that the liner of Pond A was in fair condition, and a floating aerator was operational in Pond A to aerate stored water. The liner will need to be replaced in the next 5 years. The liner of Pond B showed signs of significant degradation. The City is considering converting Pond B into a percolation pond to discharge additional reuse water into the groundwater table. Percolation Pond C and Pond D appeared to be in good condition. These percolation ponds are regularly cycled and are disked and windrowed four times each per year by WWTP staff to ensure there is sufficient volume and surface area available to enhance percolation. Jacobs recommends that the City convert Pond B into a percolation pond to allow greater flexibility in handling reclaimed water.

Solids Treatment

The solids treatment at the Island WWTP includes a waste activated sludge (WAS) pump station, a sludge holding tank, and dewatering equipment to process the WAS generated by the secondary treatment process.

Description of Existing Facilities

WAS from secondary treatment is diverted from a sump located next to the RAS pump station to the sludge holding tank via two submersible WAS pumps. The WAS pumps are manufactured by Fairbanks Morse and were replaced in 2003.

The Sludge Holding Tank at the Island WWTP is a 450,000-gallon concrete tank that was constructed in 2004. The holding tank is equipped with coarse bubble mixers and is aerated to mix the solids and provide minimal oxidation of the volatile solids remaining in the sludge. There are two positive displacement blowers that supply air to the sludge holding tank. Air is typically supplied for 14 to 16 hours per day. For the remaining time, the solids are allowed to settle in the bottom of the tank. The plant operators then decant the holding tank and route the decanted liquid back to the RAS pump station for further treatment. The solids in the tank are generally thickened up to 1 to 2 percent total solids.

The settled solids are drawn from the bottom of the holding tank by two belt filter press (BFP) feed pumps that discharge to the BFP. These pumps are progressive cavity type pumps manufactured by Moyno. The suction pipe of the BFP feed pumps also includes two sludge grinders by Moyno. These grinders help to break down rags and other stringy material, leading to better dewatering performance. A single 1-meter BFP manufactured by Andritz is located in the filter press room of the Headworks Building. The BFP is used to dewater the sludge following polymer addition to reduce its volume. The dewatered cake discharges into a chute, which in turn loads into a hauling truck, which then transports dewatered biosolids cake to the City's landfill for disposal. Sludge is conditioned with a polymer addition before dewatering. The filtrate from the BFP process is recycled back to Pump Station 3A or the discharge line of Pump Station 3A. The BFP was completely replaced in 2021 by Andritz. At the Island WWTP, sludge is typically dewatered 1 to 2 days per week, generating approximately two truckloads of dewatered cake weekly.

Capacity Evaluation

Waste Activated Sludge Load Projections

The WAS data collected between 2015 and 2021 was analyzed and 30-day rolling averages were determined for the WAS mass flow rate and concentrations. The assumptions used in the development of future BOD_5 loads at the Island WWTP were applied to estimate the WAS mass flow rate in 2040. The loads were escalated by compounding with an annual growth rate of 0.7 percent, which was obtained from the City's 2016 General Plan (Lake Havasu City 2016). Table C-13 summarizes the historical and future WAS flows.

Table C-13. Historical WAS Flow Rate and Anticipated WAS Flow Rate in 2040

Parameter	Historical Data	2040 Flows and Loads	Notes
WAS Mass Flow Rate, ppd	1,900	2,170	
WAS Average Concentration, mg/L	4,000	4,000	The WAS concentrations from historical data were assumed for the 2040 conditions as well.
WAS Flow Rate, gpd	57,000	65,000	

Waste Activated Sludge Pump Station

The WAS Pump Station design criteria are summarized in Table C-14. The pump station has sufficient capacity to convey the projected WAS flows to the sludge holding tank in 2040.

Table C-14. WAS Pump Station Design Criteria

Design Criteria	WAS Pump Station
Number of pumps	2
Type	Submersible
Flow capacity each pump, gpm	150 at 81 feet TDH 370 at 42 feet TDH
Configuration	1 Duty/ 1 Standby
Motor, hp	10 hp

Sludge Holding Tank

The sludge holding tank and blower characteristics are summarized in Table C-15. Wasting is typically performed 5 days a week, but the dewatering is performed only 1 to 2 days per week. The capacity of the sludge holding tank is determined by its hydraulic retention time (HRT). The HRT of the tank under current conditions is based on the historical data from 2015 to 2021 and was calculated at 7.1 days. With increased WAS flows in 2040, the HRT is reduced to 6.3 days. The tank has only an additional 1.3 days of storage if the BFP were to be offline because of an emergency or for maintenance.

Table C-15. Design Criteria for the Sludge Holding Tank and Blowers

Design Criteria	Sludge Holding Tank
Number of Tanks	1
Geometry	Circular
Total Volume, gallons	450,000
Usable Volume, gallons	406,000
Diameter, feet	62
Sidewater Depth, feet	18
Current HRT, days	7.1
HRT in 2040, days	6.3
Blowers	
Blower Type	Rotary Lobe
Number of Blowers	2
Capacity of Each Blower, scfm	1,500
Discharge pressure, psi	7.5
Motor, hp	15

Belt Filter Press Feed Pumps and Sludge Grinding Pumps

Table C-16 and Table C-17 summarize the design criteria for the BFP feed pumps and sludge grinder pumps, respectively. Because the sludge is slightly thickened in the sludge holding tank, the flow to the BFP is less than the WAS flow rates described above. The historical and anticipated feed rates to the BFP are presented in Table C-18. The assumptions made in the projections are provided in the table. Both pumps have sufficient capacities to process the sludge flows anticipated to be generated in 2040.

Table C-16. BFP Feed Pump Design Criteria

Design Criteria	BFP Feed Pumps
Number of Pumps	2
Type	Progressive Cavity
Flow Capacity each pump, gpm	25 at 131 feet TDH (minimum flow) 170 at 131 feet TDH (maximum flow)
Configuration	1 Duty/ 1 Standby
Motor, HP	15

Table C-17. Sludge Grinder Pump Design Criteria

Design Criteria	Sludge Grinder Pumps
Number of Units	2
Flow Capacity each pump, gpm	Average Flow – 150 Maximum Flow – 650
Configuration	1 Duty/ 1 Standby
Motor, HP	3

Table C-18. BFP Feed Rates – Historical and 2040 Projection

Parameter	Historical Data (2015-2021)	2040 Flows and Loads	Notes
Mass Flow Rate of Sludge to be Dewatered, lb/5 days	7,600	8,680	This is how much sludge remains in the bottom of the tank after 5 days. Assumed that 20 percent of the solids remains in the decant liquid.
Sludge Concentration, percent	1.4	1.4	1.4 percent was observed in the historical data. The same concentration is assumed for the 2040 projection.
Gallons of Sludge to be Dewatered, gallons/5 days	65,000	74,300	Gallons of sludge generated every 5 days needing to be dewatered.

Dewatering

The design criteria of the dewatering equipment at the Island WWTP are summarized in Table C-19.

Table C-19. Dewatering Equipment Design Criteria

Design Criteria[a]	BFP
Number of Units	1
Belt Width, meter	1
Motor, hp	3
Sludge Feed, percent	0.5 to 2
Solids Loading Rate, lb/hour	200 to 500
Cake Solids, percent	15 to 18
Design Solids Capture Rate, percent	92

[a] Design criteria for the BFP were obtained from the 2003 Master Plan (Lake Havasu City 2003).

The capacity of the dewatering equipment was evaluated based on the SLR to the BFP. The inlet flows to the BFP were developed in the previous section. The estimate assumed a run time of 5 hours per day for a total period of 2 days. The results of the capacity analysis for the historical data and 2040 conditions are presented in Table C-20. It appears that the SLR for both the conditions exceeds the manufacturer's design criteria. The increased SLR likely reduces the capture efficiency for producing a cake with lower total solids percentage than the design percentages. However, the BFP has sufficient capacity because the number of hours/days of operation could be increased to process the additional flows in 2040.

Table C-20. Dewatering Equipment Capacity Analysis

Parameter	Historical Data (2016-2021)	2040 Flows and Loads	Notes
Hours of Operation per day, hours	5	5	Based on the information provided by the operators. Same conditions have been assumed for 2040.
Number of Days BFP is operated, days	2	2	Based on the information provided by the operators. Same conditions have been assumed for 2040.
Sludge to be processed per day, ppd	3,800	4,300	Based on the mass flow rate estimated in the previous section.
Sludge to be processed per day, gpd	32,500	37,150	Based on the flow rate estimated in the previous section.
Solids Capture Rate, percent	85	85	Solids capture rate was reduced from 92 percent to 85 percent to account for inefficiencies with the BFP equipment.
Solids Loading Rate, lb/hour	760	868	
Cake Produced, dry pounds/day	3,200	3,700	
Cake Total Solids, percent	12	12	Average value based on the historical data (2015 to 2021). Same percentage has been assumed for the 2040 conditions.
Cake Produced, wet pounds/day	26,900	30,800	

Recommendation

The WAS flows that could be generated in 2040 were developed using the same methodology that was used to develop influent BOD_5 loads. Overall, the different unit processes of the solids treatment have sufficient capacity to process the 2040 flows and loads. The analysis assumed the same conditions observed historically would still be applicable in 2040. Any changes to the WAS stream resulting from operational changes or addition of new solids stream at the Island WWTP will require a reanalysis of the solids treatment.

A concern identified during the capacity analysis is the cake thickness produced from the BFP equipment. The cake total percent solids is quite low compared to the typical industrial value of 15 to 18 percent from a BFP. This means that every truck load of cake hauled to the landfill has a higher water content than design conditions and results in higher hauling costs. The proposed solutions to this concern have been identified and are discussed in Section 6.7, Biosolids Management Plan. Solutions include operational changes and optimizing polymer solution and dosage.

Odor Control

The odor control unit at Island WWTP is a centralized Ecoverde biofilter that serves the Headworks Building and Pump Station 3A to prevent nuisance odors from being released into surrounding areas. Foul air is pulled from these areas and sent to the scrubber, where hydrogen sulfide is removed by bacteria and then discharged into the environment. The BFP room in the Headworks Building is not served by the existing odor control unit. The sludge holding tank has its own odor control unit, but it has been decommissioned.

It is recommended that the City extend the odor control to the BFP room in the Headworks Building to avoid hydrogen sulfide corrosion of mechanical and electrical equipment. The odor control unit at the sludge holding tank should either be replaced or connected to the existing unit if there is sufficient capacity.

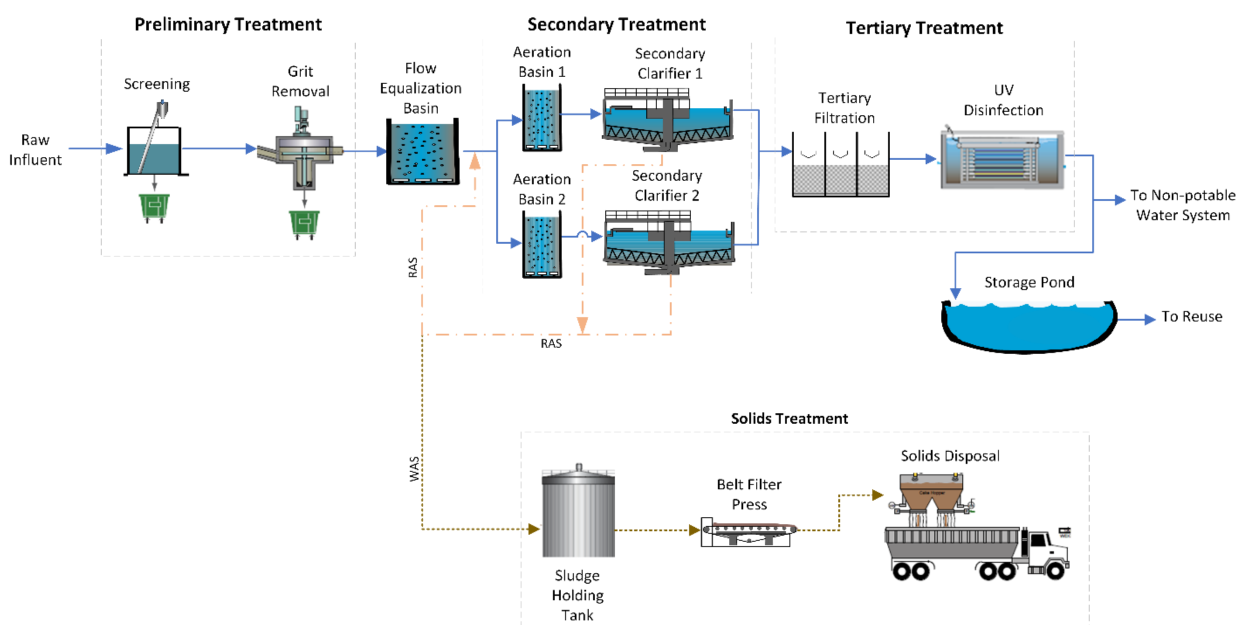
Attachment C1
Schreiber Basis of Design for Island Plant WWTP

Appendix D
Detailed Capacity Analysis of Mulberry WWTP

Appendix D. Detailed Capacity Analysis of Mulberry WWTP

An evaluation of each of the Mulberry WWTP main unit processes is described in the following sections. Each section discusses the current unit process and any operational problems or opportunities for optimization, provides an analysis of its capacity to treat flows and/or loads through the planning period, and is followed by recommended modifications. The liquids treatment is discussed first, progressing from influent raw sewage to filtered and disinfected effluent, followed by solids handling processes. The general process flow schematic is presented on Figure D-1.

Figure D-1. Mulberry WWTP Process Flow Schematic



Preliminary Treatment

Description of Existing Facilities

The preliminary treatment at the Mulberry WWTP is housed in the Headworks Building and consists of the following processes:

- Screening equipment – Huber multi-rake bar screen RakeMax and manual bar screen
- Grit removal – Smith and Loveless Pista Grit Chamber

The screen and grit removal systems had reached their end of useful life and were replaced with new equipment in 2014. The Mulberry WWTP has two types of screening equipment, including a multi-rake bar screen RakeMax from Huber used during normal operations. A manual bar screen is located in the adjacent channel and is used only during bypass conditions such as during maintenance or repair of the Huber screens. The screenings are dropped onto a conveyor belt and moved to a screening washer and compactor. The compacted screenings are then disposed of at the City landfill.

The screened wastewater flow is measured using a Parshall flume. The throat width of the flume is 12 inches. Level is monitored using an ultrasonic level recorder. The wastewater flows further downstream to the Pista Grit Chamber that is manufactured by Smith and Loveless, Inc. The unit includes a grit chamber, a grit concentrator, a turbo pump, and a screw conveyor. The grit is collected and disposed of at the City landfill.

Capacity Evaluation

The preliminary treatment capacities were evaluated for their ability to treat peak hourly flows at the 2040 conditions. The estimated peak hourly flow to the Mulberry WWTP in 2040 is 3.11 million gallon(s) per day (mgd).

Screens

The existing screening system characteristics are summarized in Table D-1. Based on the 2040 peak hourly flow projections for the Mulberry WWTP, both types of screening equipment are estimated to have sufficient capacities.

Table D-1. Screening Design Criteria

Design Criteria	Multi-Rake Bar Screen	Manual Bar Screen
Number of Screens	1	1
Bar spacing, inch	1/4	1/2
Channel Depth, feet	4.5	4.0
Channel Width, feet	3	2.5
Design Capacity, mgd	4	4.7

Parshall Flume

The total capacity of the Parshall Flume to record influent flows at the Mulberry WWTP is indicated to be 10.4 mgd, based upon the analysis carried out in the 2003 Master Plan (Lake Havasu City 2003) and appears to have sufficient capacity for the projected 2040 peak hourly flows.

Grit Removal System

The grit removal system characteristics are presented in Table D-2. The grit system has adequate capacity to handle the 2040 peak hourly flows of 3.11 mgd.

Table D-2. Grit Removal System Design Criteria

Design Criteria	Grit Removal System
Number	1
Design Capacity, mgd	7

Recommendation

The capacity analysis of the preliminary treatment indicated that the systems have sufficient capacity to handle the 2040 peak hourly flows. The screens and grit removal were replaced in 2014 and appeared to be in good condition during the site walkthrough. Operations staff have indicated that the preliminary treatment systems are performing well and do not have major operational or maintenance issues. It is recommended that the ultrasonic level recorder at the Parshall Flume be calibrated yearly to ensure accurate flow rate data are recorded.

Flow Equalization

Flow equalization is carried out to attenuate any peaks/valleys observed in the influent wastewater flows. This provides a nearly constant flow and load to the downstream processes, improving their performance. Equalization also provides operational flexibility for maintenance or repairs of the downstream processes.

Description of Existing Facilities

At the Mulberry WWTP, flow equalization is provided by a single flow equalization basin (FEB). At the end of the flow equalization basin are the influent pumps, which discharge into the secondary treatment.

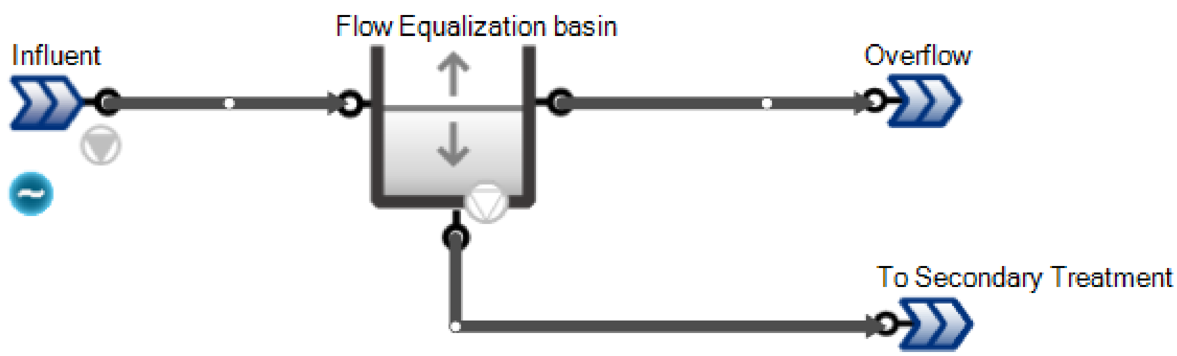
The FEB has a holding capacity of 300,000 gallons and is of a covered type. Mixing is provided through a coarse bubble mixer system to keep the solids suspended in the liquid. Three Fairbanks Morse submersible pumps are used to convey wastewater to the aeration basin splitter box for further treatment. The FEB system includes a bypass pipe from the grit removal system directly to the influent pumps, in case the FEB needs to be taken offline for maintenance.

Capacity Evaluation

The capacity of the FEB was analyzed using Dynamita's SUMO, a commercially available, whole-plant dynamic simulator. A simplified version of the FEB operation was simulated in SUMO at the peak day conditions as shown on Figure D-2. The model did not include any other WWTP processes. The following inputs were used in the model:

- The volume of the FEB is 300,000 gallons; however, because of potential solids accumulation the volume was reduced to 270,000 gallons.
- Initial volume at start of simulation is 100,000 gallons.
- 2040 peak day influent flows to the Mulberry WWTP were used. Peak day flows typically occur only once a year, but flows were simulated for 2 peak days to be conservative.
- Two conditions were simulated: one influent pump operating (1.25 mgd) and two influent pumps operating (2.25 mgd). Flows were limited to 2.25 mgd because this is the maximum flow that the disinfection system can handle.
- Pumps are operated at constant speed and turned off when the water level drops to 1 foot in the FEB.

Figure D-2. Sumo Model Configuration for the FEB at Mulberry WWTP



Note: No other process units were simulated.

The goal of the simulation was to observe the impact on the storage volume of the FEB at peak day flows and ensure there is no overflow. The overflow in the model represents any excess flows that cannot be stored in the FEB. Figure D-3 shows the response curve that was generated from the model when only one influent pump is operating. It was observed that the FEB would be at 100 percent of its capacity within half a day when only one influent pump is operating. The excess wastewater would be stored in the FEB because it has sufficient head space to prevent any spills onsite. The pump would need to be operated continuously for several hours to bring down the water level the next day. A second influent pump was turned on to process the increased flows. In this scenario, no overflow was observed. The response curve generated when two pumps are operating is shown on Figure D-4.

Overall, the FEB at the Mulberry WWTP has sufficient capacity to handle the 2040 peak day flows, based on the analysis described in this section. Removing solids accumulated at the bottom of the FEB will provide additional storage volume.

Figure D-3. Response Curve from the Sumo Model – One Influent Pump Operating

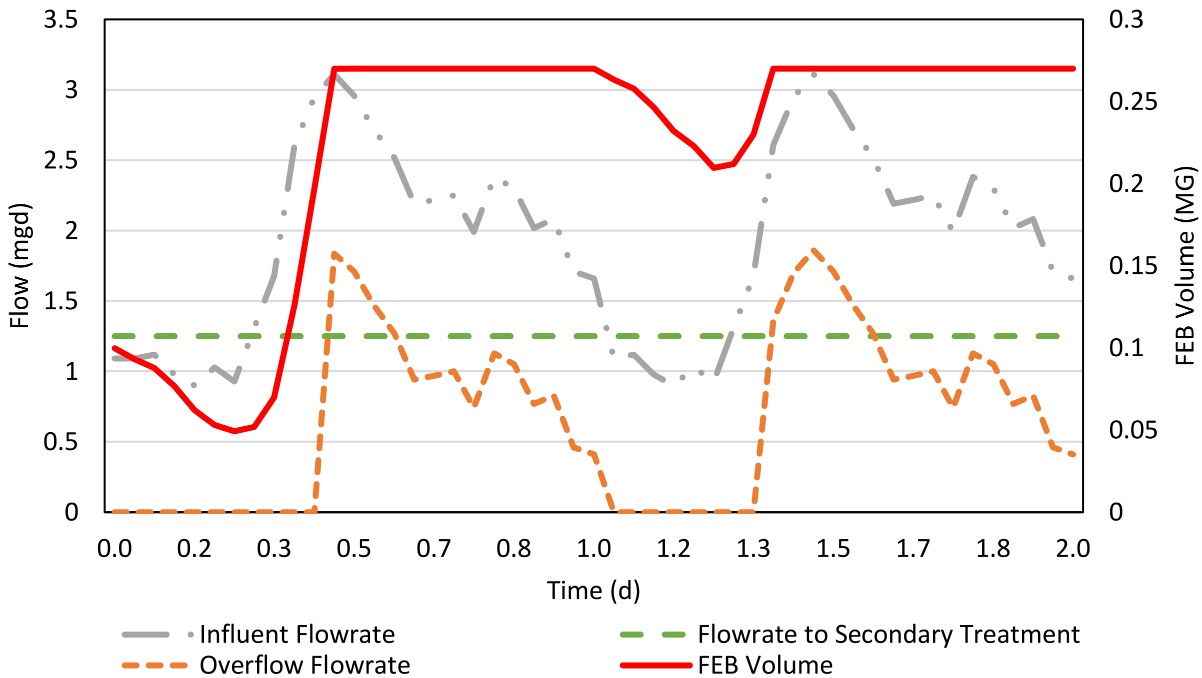
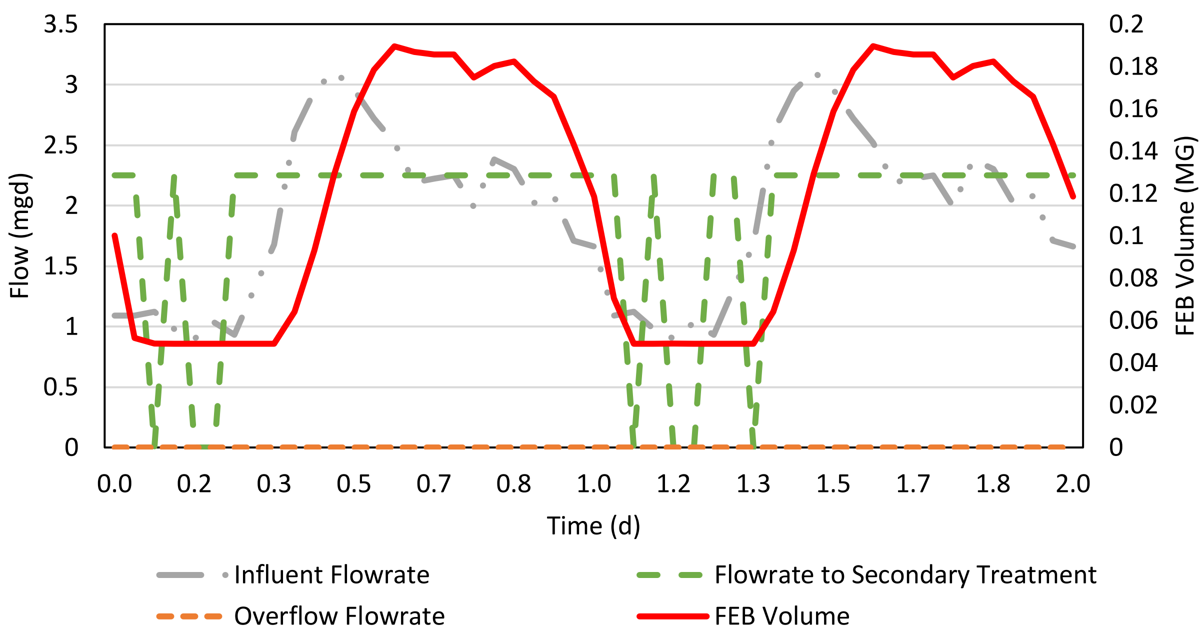


Figure D-4. Response Curve from the Sumo Model - Two Influent Pumps Operating



Influent Pump Station

The influent pump system design criteria are summarized in Table D-3. Based on the 2040 flow projections, the pumps have sufficient capacity to be able to handle the peak flows.

Table D-3. Influent Pump Station Design Criteria

Design Criteria	Influent Pump Station
Number of Pumps	3
Type	Dry well submersible
Flow Capacity, each, mgd	1.25 at 41.5 feet TDH
Configuration	2 Duty/ 1 Standby
Motor, hp	15
Adjustable Frequency Drive	No

hp = horsepower
TDH = total dynamic head

Recommendation

The capacity analysis of the FEB indicated that there is sufficient capacity to process the 2040 peak day flows. Additional storage volume could be available if the solids accumulated at the bottom of the FEB are removed. The FEB was last cleaned several years ago. It is recommended that the City remove accumulated solids from the FEB to fully use the equalization capacity. The influent pumps have sufficient capacity and appeared to be in good condition during the site walkthrough.

Secondary Treatment

Secondary treatment at the Mulberry WWTP encompasses biological treatment using an activated sludge process and settling out the sludge using a clarification process. The Mulberry WWTP has two aeration basins and two secondary clarifiers to remove organic material and nutrients. A distribution box is located upstream of the secondary treatment and is used to split the flows between the two aeration basins.

Description of Existing Facilities

Aeration Basin

The aeration basins were designed by Schreiber LLC, which is now part of Parkson Corporation. Aeration Basin 1 and Aeration Basin 2 were constructed in 1991 and were the Schreiber GRD model. The GRD model had two separate zones: aerobic and anoxic. These basins were upgraded to the GRO model in 2003 and include a half diameter rotating bridge. The GRO model is a continuous sequencing reactor system and can achieve nutrient removal in a single basin without the need for separate aerobic/anoxic zones. The system has two distinctive phases, aerobic and anoxic, occurring in a sequence to remove carbon and nitrogen.

In the aerobic phase, air is bubbled through the mixed liquor to allow nitrification to occur. The dissolved oxygen (DO) is typically maintained between 0.8 and 1.1 milligrams per liter (mg/L). The air is supplied through fine membrane diffusers, which are mounted on a rotating bridge that rotates around the aeration basin. The aerobic phase lasts for between 45 minutes and an hour. Air is supplied, then the air is turned off, and the basin turns anoxic, which allows denitrification to occur. The anoxic phase occurs for a time period similar to that of the aerobic phase.

Aeration Blowers

The aeration blowers are located in the Headworks Building. There are five Kaiser rotary lobe blowers, installed in 2003, that supply air to the headers mounted on the rotating bridges. Blowers 1 and 2 supply air to Aeration Basin 1. Blowers 4 and 5 supply air to Aeration Basin 2. Blower 3 is a standby blower and can supply air either to Aeration Basin 1 or Aeration Basin 2.

Secondary Clarification

Secondary clarification is provided by two 72-foot-diameter clarifiers with a 12.75-foot sidewater depth. The secondary clarifiers are also provided by Schreiber and were last upgraded in 2003. Each aeration basin is served by one clarifier. The clarified effluent flows by gravity to the tertiary filters.

The settled sludge at the bottom of the tank is pumped back to the aeration basins using the return activated sludge (RAS) pumps. The RAS pump station has three screw-type pumps provided by Schreiber. Two RAS pumps were completely replaced in 2013 and a third RAS pump was installed in 2014. WAS is drawn off from the RAS piping and sent to the sludge holding tank. The scum collected from the clarifiers is also sent to the sludge holding tank.

Capacity Evaluation

Aeration Basin

The design criteria for the aeration basins were obtained from the Schreiber Corporation and are included in Attachment D1. The design criteria are summarized in Table D-4.

Table D-4. Aeration Basin Sizing

Parameter	Value
Geometry	Circular
Number of Units	2
Diameter, feet	126
Sidewater depth, feet	16
Volume each basin, million gallons	1.49
Maximum Month Flow, mgd	2.2
Maximum Day Flow, mgd	2.5
Maximum BOD_5 Loading, ppd	5,688 (310 mg/L)

Table D-4. Aeration Basin Sizing	
Maximum TSS Loading, ppd	9,633 (525 mg/L)
Maximum Ammonia Loading, ppd	1,027 (56 mg/L)
Hydraulic Retention Time at Maximum Month Flow, hours	32.56
BOD_5 Loading Rate, lb. BOD_5/1,000 ft^3	14.26
Design MLSS concentration, mg/L	3,600
Design SRT, days	28

BOD_5 = 5-day biochemical oxygen demand
ft^3 = cubic feet
lb = pound(s)
mg/L = milligram(s) per liter
MLSS = mixed liquor suspended solids
ppd = pounds per day
SRT = solids retention time

The overall capacity of the aeration basins is dependent on several factors, including aeration capabilities, overall basin volume, primary effluent mass loading, and the solids retention time or the MLSS concentrations. In this analysis, no process model was developed to provide a detailed estimate of the capacities to treat future flows and loads. Instead, the BOD_5 loading rate and the various influent loads were compared against the design loads to provide a capacity estimate. If the City has concerns about the secondary treatment performance or is looking to expand or modify the secondary treatment, Jacobs recommends that the City conduct a detailed analysis of the secondary treatment using more detailed influent characterization and dynamic simulations.

The capacity of the aeration basins was evaluated using the 2040 maximum month influent loads presented in Section 6.1.3. The estimated BOD_5 loading rate is 5 lb. BOD_5/1,000 ft^3, which is well below the design criteria, indicating that the aeration basins have sufficient volume. The BOD_5, TSS, and ammonia loading to the Mulberry WWTP are well below the design criteria. It is important to note that because the long retention times in the sewer collection system, the BOD degrades within the sewer pipes, resulting in less BOD_5 reaching the Mulberry WWTP. The lack of carbon can severely impact the denitrification process. However, because of the long sludge age in the aeration basins it is possible that some of the slowly biodegradable carbon is consumed during nitrification. It recommended that the BOD_5 and ammonia loadings be closely monitored to minimize any impacts on the nitrogen removal process.

Aeration Blowers

The existing blower system characteristics are summarized in Table D-5.

Table D-5. Aeration System Sizing

Parameter	Value
Number of Units	5
Blower Type	Rotary Lobe
Airflow each Blower, scfm	1,064

Table D-5. Aeration System Sizing

Discharge pressure, psi	10
Motor, hp	75

hp = horsepower
psi = pounds per square inch
scfm = standard cubic feet per minute

The theoretical air demand for carbon and nitrogen removal at the 2040 MM condition was calculated to evaluate the capacity of the existing blowers. The calculation is presented in Table D-6. The parameters used in the calculations were similar to what Schreiber had used to estimate the airflow demand during the aeration basin design. The aeration system at the Mulberry WWTP has sufficient capacity at the 2040 MM conditions because the BOD_5 and ammonia loading are well below the design criteria.

Table D-6. 2040 Air Demand for Maximum Month Flow and Loads

Parameter	Schreiber Design Criteria	At 2040 Maximum Month Condition	Notes
O_2 required for BOD_5, lb O_2/day	8,532	3,000	Assumed 1.5 lb O_2/lb of BOD_5
O_2 required for nitrification, lb O_2/day	4,726	2,107	Assumed 4.6 lb O_2/lb of ammonia
Total AOR, lb O_2/day	13,258	5,107	Credits for denitrification were not considered to generate a conservative value
SOTR, lb O_2/hour	1,064	410	
Airflow, scfm	4,254	1,640	

AOR = actual oxygen requirement
O_2 = oxygen
SOTR = standard oxygen transfer rate

Secondary Clarification

A summary of the secondary clarification system at the Mulberry WWTP is presented in Table D-7.

Table D-7. Secondary Clarifier Design Criteria

Parameter	Value
Geometry	Circular
Number of Clarifiers	2
Diameter, feet	72
Surface Area, ft^2	4,070
Sidewater depth, feet	12.5
SOR at Maximum Month Flow, gpd/ft^2 [a]	270

Table D-7. Secondary Clarifier Design Criteria

SOR at Maximum Day Flow, gpd/ft^2 [a]	307
SVI, mL/g[b]	183

[a] The surface overflow rate is the same for average flow and at peak flow conditions because there is flow equalization.

[b] SVI data was analyzed from 2015 to 2021 and the 95th percentile value was used for the capacity analysis.

ft^2 = square feet
gpd = gallons per day
mL/g = milliliter per gram
SOR = surface overflow rate
SVI = sludge volume index

The capacity of the secondary clarifiers is defined by two criteria:

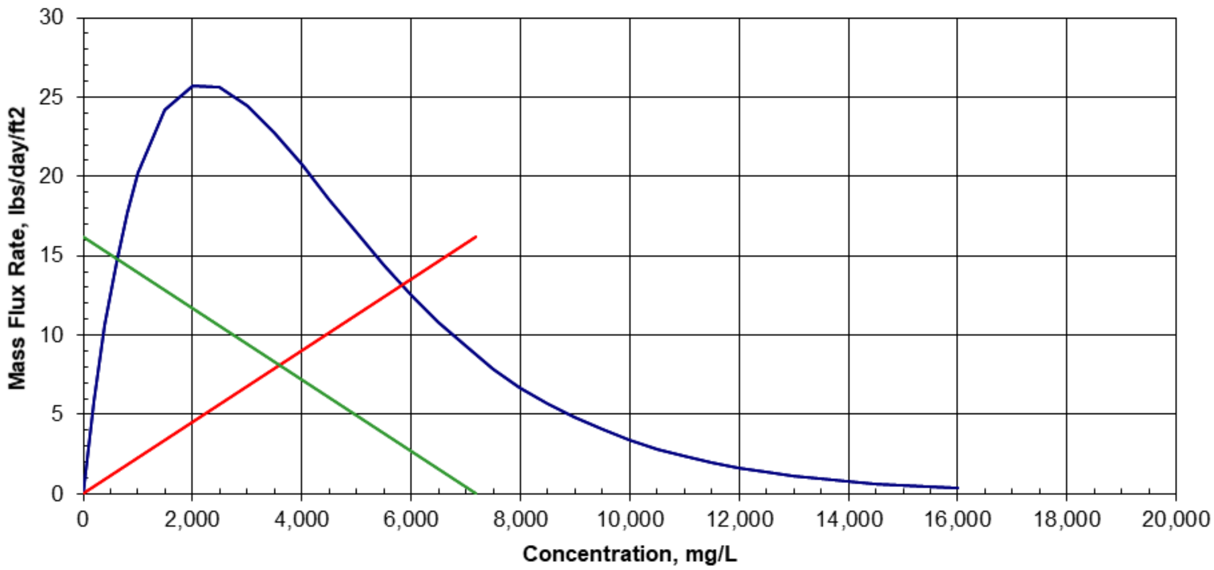
- Solids loading rate (SLR) on the secondary clarifier: This is controlled by the MLSS concentration of the system, the solids settleability, and the hydraulic flow through the system. MLSS is controlled by the operating SRT and the solids load to secondary treatment.
- Hydraulic loading rate (HLR) on the secondary clarifier: During wet weather events, flow rates through the secondary clarifier can be high enough to cause solids not to settle or scour solids from the sludge blanket surface.

The analysis of solids loading rate on the secondary clarifiers was performed using Jacobs' PClarifier tool. PClarifier uses solid flux theory and state point analysis (SPA) to determine the limiting conditions on the clarifier. SPA is based on solids mass balances around the clarifier. The solids flux curve represents the settling characteristics of a particular MLSS concentration per unit area of the clarifier. The overflow line starts at the origin and the slope of the line is equal to the SOR. The underflow line starts at the calculated SLR on the y-axis and the slope of this line is equal to the RAS flow rate. Where the underflow line intersects with the x-axis, that represents the expected RAS concentration. The intersection of the overflow line and underflow line is defined as the state point. The state point represents the operating point of the clarifier and helps to determine if the clarifier is underloaded, critically loaded, or overloaded (Water Environment Federation [WEF] Manual of Practice [MOP] Number 8). For this analysis, the following assumptions were used:

- MLSS concentration of 3,600 mg/L
- RAS flow was assumed to be equal to the influent flow to the secondary clarifier
- SVI was set to 187 mL/g, which was the 95th percentile value observed in the historical data (2015 to 2021)
- The Daigger 1995 SVI model was used for flux correlation
- Analysis was conducted for two conditions:
  - Maximum Month Flows –Two clarifiers online
  - Maximum Month Flows – One clarifier offline

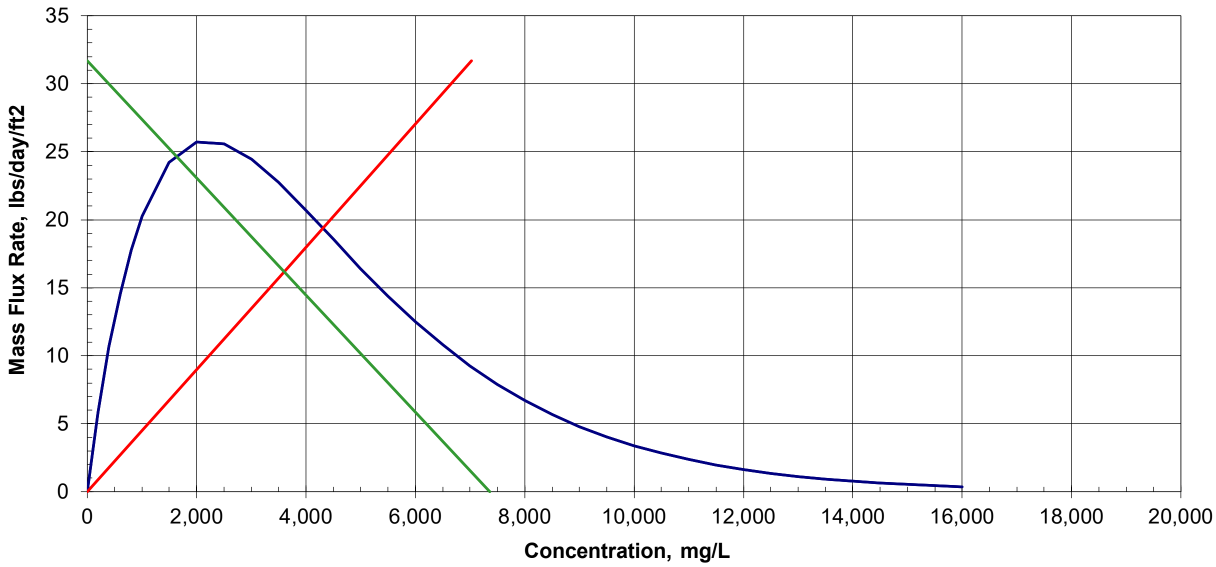
For the SPA analysis, the maximum allowed SLR is only 80 to 90 percent of the theoretical SLR. This is done to account for inefficiencies associated with the clarifiers. Because the clarifiers at the Mulberry WWTP are of the shallow type, the maximum SLR is set to 80 percent. The SPA for the secondary clarifiers for the two flow conditions is shown on Figure D-5 and Figure D-6. The results from the SPA analysis are presented in Table D-8. The secondary clarification system has sufficient capacities to treat maximum month flows when both clarifiers are operational. If one of the clarifiers must be taken offline for maintenance, the remaining clarifier will be able to handle flows up to 2.2 mgd. The clarifier will be operating at capacity in terms of SLR and SOR as shown on Figure D-6. Operating the clarifier in this condition is recommended for only short time periods. Additionally, the City may be able to reduce the flow rate from the FEB to the downstream processes or divert flows to other WWTPs to ensure optimal treatment if one clarifier is offline.

Figure D-5. State Point Analysis – Maximum Month Flow Condition (Two Clarifiers Online)



blue line = solids flux curve; red line = overflow line; green line = underflow line

Figure D-6. State Point Analysis – Maximum Month Flow Condition (One Clarifier Offline)



blue line =solids flux curve; red line = overflow line; green line = underflow line

Table D-8. Secondary Clarification System Performance at 2040 Maximum Month Conditions

Input Parameter	Maximum Month Flow Condition – Two Clarifiers Online	Maximum Month Flow Condition – One Clarifier Offline
Influent flow, mgd	2.2	2.2
RAS flow, mgd	2.2	2.1
SOR, gpd/ft²	270	540
Secondary clarifier applied SLR lb/d/ft²	16.21	31.68
Secondary clarifier limiting SLR, lb/d/ft²	24.75	37.92
Secondary clarifier applied SLR to limiting SLR, %	65%	84%

Return Activated Sludge Pump Station

The design criteria for the RAS Pump Station are summarized in Table D-9. The RAS pumps appeared to be in good condition during the site walk through. They have sufficient capacity to handle maximum month flows coming into the secondary treatment.

Table D-9. RAS Pump Station Design Criteria

Design Criteria	RAS Pump Station
Number of Pumps	3
Type	Adjustable Tube Mounted Screw

Table D-9. RAS Pump Station Design Criteria

Flow Capacity, mgd	2.45 at 13.75 feet TDH
Motor, hp	20

Recommendations

The initial analysis of the secondary treatment at the Mulberry WWTP indicated that there is sufficient capacity to treat the 2040 maximum flows and loads. However, the analysis provided in this Wastewater Master Plan is very brief. It is recommended that the City carry out a detailed analysis of the secondary treatment to determine the actual capacity. It is also recommended that the operators pay close attention to the BOD_5 and ammonia loadings to Mulberry WWTP, to ensure that there are no disruptions to the nutrient removal processes.

During the site visit, cracks were observed in the top wall of the aeration basins as shown on Figure D-7. The wall of the aeration basins is not completely circular and as a result the weight of the rotating bridge is not evenly distributed. Jacobs recommends that the City repair the concrete in the top wall before cracking accelerates. Additionally, the City is considering piloting submerged fine bubble diffusers with three mixers in one of the aeration basins. If the pilot is successful, the City may consider making the installations permanent and removing the rotating bridge.

Figure D-7. Cracks Observed on the Edge of the Aeration Basin Concrete Wall

The operators noted that if the secondary clarifiers overflow, the clarified wastewater has the potential to flow offsite into the Daytona Wash, which is connected to the Colorado River. The site is significantly sloped on the southeast side of the Mulberry WWTP near Aeration Basin 2. One such overflow event occurred several years ago that washed away soil and damaged the property fence as shown on Figure D-8. Jacobs recommends that curbing be designed and installed near the clarifiers to divert any potential overflow to an onsite catchment.

Figure D-8. (a) Southeast Corner of Mulberry WWTP near Aeration Basin, (b) Impact on the Site from a Clarifier Overflow Event

The RAS pump station has sufficient capacity, but there are no flowmeters on the RAS piping. The operators use the pump run time to get an approximate daily value. Jacobs recommends installing a flowmeter on the RAS discharge piping to allow for RAS flow quantification.

The top of the RAS sump pit walkway is protected by only a chain railing as shown on Figure D-9. This represents a fall hazard, with the drop almost 20 feet, and it is recommended that the chain be replaced with a more effective and substantial railing.

Figure D-9. Chain Railing Installed at the Top of the RAS Sump Pit; RAS Pumps are Located to the East

Tertiary Filtration

Description of Existing Facilities

Tertiary filtration is provided by three Schreiber Fuzzy Filters. These filters use a compressible media filtration system, and the porosity of the media can be adjusted to suit the influent characteristics. Two of the filters were installed in 2004 and were upgraded in 2020 with new media, valves, pressure sensors, and turbidity control system. The third filter was installed in 2020. The filtered effluent is collected in a common channel and then sent to the disinfection system.

To backwash the filter media, filtration is discontinued but the influent is still allowed into the filter at a reduced rate. The influent is used as the wash water medium. During the backwash, the media is agitated by high-velocity air provided by two Kaiser rotary blowers. This agitation causes the trapped solids to be released from the media pores. The backwash liquid is then discharged to the FEB, where it is reintroduced into the process.

Capacity Evaluation

The existing filtration system characteristics are summarized in Table D-10. Based on the 2040 peak hourly flow projections for the Mulberry WWTP, the filtration system has sufficient capacity.

Table D-10. Tertiary Filtration Design Criteria

Design Criteria	Fuzzy Filters
Number of Filters	3
Average Design Flow, each, mgd	0.73
Maximum Design Flow, each, mgd	0.83
Filter Size, feet	5 by 5
Filter Area per Vessel, ft^2	25
Hydraulic Loading Rate at Average Design Flow, gpm/ft^2	20.37
Hydraulic Loading Rate at Maximum Design Flow, gpm/ft^2	23.14
Wash water rate, gpm/ft^2	10-20
Air required for washing per vessel, scfm	375
Tertiary Filtration Blowers	
Blower Type	Rotary
Number of Blowers	2
Capacity of Each Blower, scfm	370
Discharge pressure, psig	8.0

gpm = gallon(s) per minute
psig = pounds per square inch gauge

Recommendation

The capacity analysis of the tertiary filtration system indicated that this system has sufficient capacity to process the 2040 peak hourly flows as the flows coming into the WWTP are equalized and then sent to the downstream processes. The filters were upgraded in 2020 and appeared to be in good condition during the site walkthrough. Operations staff have indicated that the filters perform well and are easy to maintain.

Disinfection

Description of Existing Facilities

The Mulberry WWTP uses the Trojan UV3000Plus to disinfect the filtered effluent to meet permit requirements for Class A+ Reclaimed Water. Ultraviolet (UV) radiation can penetrate the cell walls of the pathogenic organisms and damage the DNA or RNA strands. This results in the organism being unable to perform cellular functions, ultimately leading to the death of the organism. The disinfected effluent is then discharged into a lined pond for reuse or diverted to the non-potable process water system.

Capacity Evaluation

The UV system consists of 2 channels, with 3 banks per channel and 24 UV lamps per bank. The flow into the two channels is controlled by sluice gates. Space has been left to install a third channel of UV modules to treat increased flows in the future. The UV disinfection design criteria are summarized in Table D-11. Based on the 2040 peak hourly flow projections for the Mulberry WWTP, the UV disinfection system is estimated to have sufficient capacity with all UV channels operating. If a channel is offline because of emergency or maintenance, the overall capacity of the WWTP is reduced to 1.1 mgd. The Mulberry WWTP could use the FEB to store excess wastewater or divert some of the flows to either the Island WWTP or the North Regional WWTP.

Table D-11. Disinfection Design Parameters

Design Criteria	UV Disinfection System
Type	Open Channel
Number of Channels	2
Number of Modules	3
Number of Lamps per Module	24
UV Transmittance	65% at 254 nanometers
Maximum Design Flow, Total capacity	2.2 mgd
Disinfection Standards	≤ 23/100 milliliter fecal coliform based on 1 day maximum

Recommendation

The capacity analysis of the disinfection system indicates that the UV disinfection system has sufficient capacity to handle the 2040 peak hourly flows with all UV channels operating, as the flows coming into the WWTP are equalized and then sent to the downstream processes.

The UV disinfection system was installed in 2004 and has been in service for nearly 18 years. The typical lifespan of the Trojan UV3000Plus is 20 to 25 years with proper maintenance of the system. The system appeared to be in good condition during the site walkthrough. The operators indicated that they use third-party UV lamps instead of the recommended Trojan lamps because of budgetary constraints. This may potentially result in performance issues such as reduced UV dosage to properly disinfect the wastewater. Jacobs recommends using Trojan bulbs for optimal UV system performance. Additionally, the disinfection system controller is not connected to the plant SCADA system. Connecting the UV disinfection system to the SCADA system will provide greater system and operator control. Despite these maintenance activities, the City should anticipate having to replace the system in the next 10 years.

Water Reuse System

The disinfected effluent is discharged into a lined pond where it is stored and then can be used as reuse water for irrigation at the Lake Havasu Golf Club or pumped to the Island WWTP to supplement the WWTP's water reuse demand. The total capacity of the pond is 2.3 million gallons, of which 1.6 million gallons is the useful volume available for storage. Mulberry WWTP can also accept reuse water from Island WWTP and blend it with its own effluent to meet irrigation demands. Water from Lake Havasu can also be pumped into this pond for blending with reuse water to meet irrigation and water quality demands. Flygt pumps are used to send reuse water to the golf course. Fairbanks Morse pumps are used to send water to the Island WWTP or the North Regional WWTP. The design criteria for both pump stations are summarized in Table D-12 and Table D-13.

Table D-12. Golf Course Pump Station Design Criteria

Design Criteria	Golf Course Pump Station
Number of Pumps	3
Type	Vertical Turbine
Flow Capacity, gpm	900 at 317 feet TDH
Configuration	2 Duty/1 Standby
Motor, hp	100

Table D-13. Reuse Pump Station Design Criteria

Design Criteria	Reuse Pump Station to Island WWTP or North Regional WWTP
Number of Pumps	2
Type	Vertical Turbine
Flow Capacity, gpm	1,530 at 203 feet TDH
Configuration	1 Duty/ 1 Standby
Motor, hp	125

A portion of the reuse water is pumped back through the WWTP's non-potable water system. The non-potable water is used for routine floor and equipment washing, irrigation of landscape within the WWTP property, and for BFP wash water. The system includes a steel hydropneumatic tank and two Peerless vertical turbine pumps. The design criteria for the pumps are summarized in Table D-14

Table D-14. Non-potable Water System Design Criteria

Design Criteria	Non-potable Water Pump Station
Number of Pumps	2
Type	Vertical Turbine
Flow Capacity, gpm	350 at 207 feet TDH
Configuration	1 Duty/ 1 Standby
Motor, hp	30

Solids Treatment

The sludge handling at the Mulberry WWTP includes a sludge holding tank and dewatering equipment to process the WAS generated by the secondary treatment process.

Description of Existing Facilities

WAS from secondary treatment is diverted from the RAS piping to a sludge holding tank, which is a covered, circular concrete basin. The holding tank is equipped with coarse bubble mixers and is aerated to mix the solids and provide minimal oxidation of the volatile solids in the sludge. The Kaiser blowers for the sludge holding tank are located in the Headworks Building. Air is typically supplied for 12 to 14 hours each day. For the remaining time, the solids are allowed to settle in the bottom of the tank. The plant operators then decant the holding tank and route the decanted liquid back to the FEB for further treatment. The holding tank is completely covered and is served by the odor control system to reduce nuisance odors. The solids in the tank are generally thickened up to 1 to 2 percent total solids.

The settled solids are drawn from the bottom of the holding tank by submersible belt filter press (BFP) feed pumps that discharge to the BFP. The BFP feed pumps are located in the sludge holding tank and manufactured by Hydromatic. A single 1-meter BFP manufactured by Andritz is located in the Headworks Building. The BFP was installed in the 1990s and was completely rebuilt several years ago. The BFP is used to dewater the sludge to reduce its volume. The dewatered cake drops into a chute to load up the hauling truck, which then transports the cake to the City's landfill for disposal. Sludge is conditioned with polymer addition prior to dewatering. The polymer feed system consists of a 55-gallon drum and a PolyBlend feed system to feed polymer to the BFP. The filtrate from the BFP process is recycled back to the FEB. At the Mulberry WWTP, sludge is typically dewatered 2 to 3 days per week, generating approximately two truckloads of dewatered cake weekly.

Capacity Evaluation

Waste Activated Sludge Load Projections

The WAS data collected between 2015 and 2021 was analyzed and 30-day rolling averages were determined for the WAS mass flow rate and concentrations. The assumptions used in the development of future BOD_5 loads at the Mulberry WWTP were applied to estimate the WAS mass flow rate in 2040. The loads were escalated by compounding with an annual growth rate of 0.7 percent, which was obtained from the City's 2016 General Plan (Lake Havasu City 2016). The historical and future WAS flows are summarized in Table D-15.

Table D-15. Historical WAS Flow Rate and Anticipated WAS Flow Rate in 2040

Parameter	Historical Data	2040 Flows and Loads	Notes
WAS Mass Flow Rate, ppd	2,200	2,512	
WAS Average Concentration, mg/L	4,670	4,670	The WAS concentrations from historical data were assumed for the 2040 conditions as well.
WAS Flow Rate, gpd	56,450	64,500	

Sludge Holding Tank

The sludge holding tank and blower characteristics are summarized in Table D-16. Wasting is typically performed for 5 days a week, but the dewatering is performed only 2 to 3 days per week. The capacity of the sludge holding tank is determined by its hydraulic retention time (HRT). The HRT of the tank under current conditions is based on the historical data from 2015 to 2021 and was calculated at 3.2 days. With increased WAS flows in 2040, the HRT is reduced to 2.8 days. The sludge holding tank has very limited capacities for the current conditions as well as the future conditions. In the event that the BFP is offline because of an emergency or maintenance, the Mulberry WWTP will have less than a week's worth of sludge storage.

Table D-16. Design Criteria for the Sludge Holding Tank and Blower

Design Criteria	Sludge Holding Tank
Number of Tanks	1
Geometry	Circular
Diameter, feet	
Sidewater Depth, feet	
Volume, gallons	180,000
Blowers	
Blower Type	Rotary Lobe
Number of Blowers	2
Airflow each Blower, scfm	1,064
Discharge pressure, psi	10
Motor, hp	75

Belt Filter Press Feed Pumps

The design criteria for the BFP feed pumps are summarized in Table D-17. The design criteria were obtained from the 2003 Master Plan (Lake Havasu City 2003). Because the sludge is slightly thickened in the sludge holding tank, the flow to the BFP is less than the WAS flow rates described above. The historical and anticipated feed rates to the BFP are presented in Table D-18. The assumptions made in the projections are provided in the table. The BFP feed pumps have sufficient capacity to process the current flows but may be operating at capacity for the 2040 projected sludge flows.

Table D-17. BFP Feed Pump Design Criteria

Design Criteria	BFP Feed Pumps
Number of Pumps	3
Type	Submersible, Centrifugal
Flow Capacity, gpm	95 at 25 feet TDH
Configuration	2 Duty/1 Standby
Motor, hp	5

Note:

Design criteria for the BFP feed pumps were obtained from the 2003 Master Plan (Lake Havasu City 2003).

Table D-18. BFP Feed Rates – Historical and 2040 Projection

Parameter	Historical Data (2015–2021)	2040 Flows and Loads	Notes
Mass Flow Rate of Sludge to be Dewatered, lb/5 days	8,800	10,050	This represents how much sludge is remaining in the bottom of the tank after 5 days. Assumed that 20 percent of the solids remains in the decant liquid.
Sludge Concentration, percent	1.3	1.3	1.3 percent was observed in the historical data. The same concentration is assumed for the 2040 projection.
Gallons of Sludge to be Dewatered, gallons/5 days	81,100	86,000	Gallons of sludge generated every 5 days needing to be dewatered.

Dewatering

The design criteria of the dewatering equipment at the Mulberry WWTP are summarized in Table D-19.

Table D-19. Dewatering Equipment Design Criteria

Design Criteria	BFP
Number of Units	1
Belt Width, meter	1
Motor, hp	3
Sludge Feed, percent	0.5 to 2
Solids Loading, lb/hour	200 to 500

Table D-19. Dewatering Equipment Design Criteria

Design Criteria	BFP
Design Cake Solids, percent	15 to 18
Solids Capture Rate, percent	92

Note:
Design criteria for the BFP were obtained from the 2003 Master Plan (Lake Havasu City 2003).

The capacity of the dewatering equipment was evaluated based on the solids loading rate (SLR) to the BFP. The inlet flows to the BFP were developed in the previous section. The estimate assumed a run time of 5 hours per day for a total period of 3 days. The results of the capacity analysis for the historical data and 2040 conditions are presented in Table D-20. It appears that the SLR for both the conditions exceeds the manufacturer's design criteria. The increased SLR likely reduces the capture efficiency and produces a cake with a lower total solids percentage than the design percentages. However, the BFP has sufficient capacity because the number of hours/days of operation could be increased to handle the additional flows in 2040.

Table D-20. Dewatering Equipment Capacity Analysis

Parameter	Historical Data (2016-2021)	2040 Flows and Loads	Notes
Hours of operation per day, hours	5	5	Based on the information provided by the operators. Same conditions have been assumed for 2040.
Number of days BFP is operated, days	3	3	Based on the information provided by the operators. Same conditions have been assumed for 2040.
Sludge to be processed per day, ppd	2,950	3,350	Based on the mass flow rate estimated in the previous section.
Sludge to be processed per day, gpd	27,000	30,900	Based on the flow rate estimated in the previous section.
Solids Capture Rate, percent	85	85	Solids capture rate was reduced from 92 percent to 85 percent to account for inefficiencies with the BFP equipment.
Solids Loading Rate, lb/hour	587	670	
Cake Produced, dry pounds/day	2,500	2,850	
Cake Total Solids, percent	11	11	Average value based on the historical data (2015 to 2021). Same percentage has been assumed for the 2040 conditions.
Cake Produced, wet pounds/day	22,700	25,900	Hauled to the City landfill.

Recommendation

The WAS flows that could be generated in 2040 were developed using the same methodology that was used to develop influent BOD_5 loads. The sludge holding tank is at capacity under current conditions and cannot store sludge for more than 3 days on average. The rest of the components of the solids treatment system have sufficient capacity to process the 2040 flows and loads. The analysis assumed the same conditions observed historically would still be applicable in 2040. Any changes to the WAS stream resulting from operational changes or the addition of a new solids stream at the Mulberry WWTP will require a reanalysis of the solids treatment.

Similar to the Island WWTP, a major concern identified during the capacity analysis is the cake thickness produced from the BFP equipment. The cake total percent solids generated is low compared to the typical value of 15 to 18 percent from a BFP. This means that every truck load of cake hauled to the landfill has a higher water content than design conditions and results in higher hauling costs. The proposed solutions to this concern have been identified and are discussed in Section 6.7, Biosolids Management Plan. Solutions include operational changes and optimizing polymer solution and dosage.

Odor Control

The odor control unit at the Mulberry WWTP is a centralized Ceco Environmental Duvall wet scrubber located adjacent to the Headworks Building. The odor control unit serves the Headworks Building, FEB, and sludge holding tank to prevent nuisance odors being released into surrounding areas. Foul air is pulled from these areas and sent to the scrubber, where hydrogen sulfide and ammonia are removed by caustic solution and carbon filters, and then discharged into the environment.

The odor control unit has reached the end of its useful life and needs to be replaced. Jacobs recommends installing a biological odor control unit in place of the wet scrubbers. The biological filter would eliminate the need for the City to procure chemicals, resulting in significant savings. The unit is also relatively easier to operate and is more energy efficient.

Attachment D1
Schreiber Basis of Design for Mulberry Ave. WWTP

SCHREIBER®
Improving Water Through Technology

Basis of Design
for

Mulberry Ave. WWTP Expansion

Lake Havasu City, AZ
2.200 MGD ADF Facility

with
Filtration

Engineer
Burns & McDonnell

Representative
Enviro-Line

July 08, 2003

Mulberry Ave. WWTP Expansion

A. GIVEN OR ASSUMED	Design Year			
1A. DESIGN PARAMETERS	2021			
Design flows (Avg. 24 hr) --	**2.200**	**mgd**	1,528	gpm
Maximum Daily Flow --	**2.500**	**mgd**	1,736	gpm
Ratio: (Maximum / Daily) --	1.14			
1B. FLOWS FOR HEADWORKS				
Design flows (Avg. 24 hr) --	2.200	**mgd**	1,528	gpm
Maximum Daily Flow --	2.500	**mgd**	1,736	gpm
1C. FLOWS FOR CLARIFIER(S)				
Design flows (Avg. 24 hr) --	2.200	**mgd**	1,528	gpm
Maximum Daily Flow --	2.500	**mgd**	1,736	gpm
1D. FLOWS FOR FILTER(S)				
Design flows (Avg. 24 hr) --	2.200	**mgd**	1,528	gpm
Maximum Daily Flow --	2.500	**mgd**	1,736	gpm
2. DESIGN LOADINGS				
BOD --	310.00	mg/l		
lb BOD /day = mgd x mg/L x 8.34 lb/gal ----------------	5,688	**lb/day**		
Suspended Solids ---	525.00	mg/l		
lb Suspended Solids /day = mgd x mg/L x 8.34 lb/ga	9,633	**lb/day**		
Ammonia (NH3-N) ---	56.00	mg/l		
lb Ammonia (NH3-N) /day = mgd x mg/L x 8.34 lb/ga	1,027	**lb/day**		
B. EFFLUENT CRITERIA				
1. BOD ---	30.00	mg/l		
2. Suspended Solids --	30.00	mg/l		
3. Total Nitrogen --	10.00	mg/l		
C. ANTICIPATED PERFORMANCE	Less Than:			
1. BOD ---	10.00	mg/l		
2. Suspended Solids --	12.00	mg/l		
3. Total Nitrogen --	10.00	mg/l		

This Preliminary Basis of Design has been prepared exclusively for the convenience of the design engineer. The design parameters used have been obtained from other sources. Schreiber assumes no responsibility for the accuracy of the design parameters.

The design engineer is responsible for the final facility design. Schreiber LLC's sole function is to supply equipment based upon the final facility design.

Summary of Equipment and Design Basis

GRO Aeration System

Model :	GRO	
No. of Units :	2	
Diameter :	126	feet
SWD :	16.0	feet
F:M Ratio :	0.064	
MLSS Concentration :	3,600	mg / L
Hydraulic Detention :	32.56	hours
Biological Loading Rate :	14.26	lb. BOD / 1000 cu. ft.
Linear feet of diffuser media :	1,680	feet (PER BASIN)

Secondary Clarification

Model :	TBS	
No. of Units :	2	
Diameter :	72	feet
SWD :	12.5	feet
Surface Settling Rate @ Avg Flow :	270	gpd / sq. ft.
Surface Settling Rate @ Peak Flow :	307	gpd / sq. ft.

Fuzzy Filter

Model:	5' x 5'	
No. of Units :	2	
Total Surface Area :	50	sq. ft.
Actual Loading Rate :	34.72	gpm / sq. ft.
Maximum Operating Height	16'-10"	
Blower	1 Duty PD - 241	
Blower HP :	15.0	hp

Aeration Systems for all Processes

A. Type: Counter Current Aeration & Low - Load Process

B. Design Criteria:

1. 0.05 to 0.1 lb BOD/ lb MLSS (Low-Load Process)
2. MLSS Operating Level - 2500 to 7500 mg/L
3. MLVSS assumed to be 0.75 x MLSS
4. 1.2 to 1.5 lb Oxygen / lb BOD and 4.6 lb Oxygen/lb of Ammonia (NH3-N), all adjusted for temperature, altitude, humidity, D.O., alpha, beta to SOR.

C. Aeration Reactor Capacity Required:

1. Total lb of MLSS in Reactor

$$= \frac{5{,}688 \text{ lb BOD}}{0.064 \text{ lb BOD / lb MLSS}} = 88{,}873 \text{ lb MLSS}$$

2. lb MLSS = Volume in mil. gal x concentration mg/L x 8.34 lb / gal

Assume 3,600 mg/L Concentration

$$\text{Vol mil. gal} = \frac{88{,}873 \text{ lb MLSS}}{3{,}600 \text{ mg/L} \times 8.34} = 2.960 \text{ mil. gal}$$

$$= 395{,}731 \text{ cu ft}$$

D. Aeration Equipment Sizing and Selection:

Model - GRO		
Configuration -Dual Train		
Number of Units Required -------------------------------------	**2**	
Diameter ---	**126**	**ft.**
Side Water Depth -----------------------------------	**16.0**	**ft.**

	PER REACTOR		**TOTAL**	
2. Hp of each drive motor-----------------------	7.50	hp	15.00	hp
Bhp required at design-----------------------	5.63	bhp	11.25	bhp
3. Volume of each unit -------------------------	199,504	cu ft	399,007	**cu ft**

Actual Detention Time at Design Flow:

$$= \frac{399{,}007 \text{ cu ft} \times 7.48 \text{ gal/cu ft} \times 24 \text{ hr}}{2{,}200{,}000 \text{ gal}} = 32.6 \text{ hours}$$

Biological Loading Rate lbs BOD / 1000 cu. ft.:

$$= \frac{5687.88 \text{ (lb BOD)}}{399.007 \text{ (1000 cu ft)}} = \mathbf{14.26} \textbf{ lb BOD / 1000 cu ft}$$

E. Air Requirements - AOR:

1. Classification - Rotary positive displacement blowers and fine bubble diffusers.
2. Oxygen Requirements

a. O_2 Required for BOD =BOD lb/day * Lb of O_2/ Lb BOD

	DESIGN	OPERATING	
BOD	5,688	5,688	lb/day
Lb of O_2/ Lb BOD	1.50	1.20	
O_2 Required for BOD	8,532	6,825	lb/day

b. O2 Required for Ammonia (NH3-N) = Ammonia (NH3-N) lb/day * 4.6

	DESIGN	OPERATING	
Ammonia (NH3-N)	1,027	1,027	lb/day
Lb of O2/ Lb Ammonia (NH3-N)	4.60	4.60	
O2 Required for Ammonia (NH3-N)	4,726	4,726	lb/day

c. Total O_2 Required (AOR)

	DESIGN	OPERATING	
BOD	8,532	6,825	lb/day
Ammonia (NH3-N)	4,726	4,726	lb/day
Total O_2 Req'd (AOR)	13,258	11,552	lb/day

d. Credit for Denitrification Recovery of Oxygen

	DESIGN	OPERATING	
Ammonia (NH3-N)	4,726	4,726	lb/day
% Denite	0%	0%	
O2 Required for Ammonia (NH3-N)	0	0	lb/day
Total O_2 Req'd (AOR)	13,258	11,552	lb/day

F. Air Requirements - SOTR:

$$\text{SOTR} = \frac{(\text{AOR})\,(C^*_{20})}{(\alpha)\,(\theta^{T-20})\,[\,(\tau)\,(\beta)\,(\Omega)\,(C^*_{20})-C]}$$

		DESIGN	OPERATING	
AOR	Actual Oxygen Requirement	13,258	11,552	lb/day
C^*_{20}	Assumed from Previous Clean Water Tests	10.5	10.5	mg/l
α	Alpha	0.7	0.7	
θ	Theta = Oxygen Transfer Coefficient	1.024	1.024	
T	° C Temperature of Water	28	28	° C
C^*_{st}	Oxygen Saturation at T	7.83	7.83	mg/l
C^*_{s20}	Oxygen Saturation at 20 ° C	9.09	9.09	mg/l
τ	Tau = Oxygen Saturation Value- C^*_{st}/c^*_{s20}	0.86	0.86	
β	Beta	0.95	0.95	
P_b	Atmospheric Pressure	14.441	14.441	P.S.I.A.
P_s	Standard Atmospheric Pressure	14.696	14.696	P.S.I.A.
Ω	Omega = Pb/Ps	0.983	0.983	
C	Dissolved Oxygen Concentration	2.00	0.70	mg/l
SOTR	Standard Oxygen Transfer Rate	25,532	18,511	lb/day
SOTR	Standard Oxygen Transfer Rate	1,064	771	lb/hour

G. Air Requirements - SCFM:

Air Required

@ 0.0174799 lb Oxygen / cu ft of Air.

@ 5.10% efficiency per meter immersion depth, or 1.55% efficiency per foot

@ 4.67 meters immersion depth of diffusers, or 15.33 feet immersion depth

24% Transfer Efficiency

V required = (SOTR lb/hr) / (0.0174799 X Transfer Efficiency)

	DESIGN	OPERATING	
V required in cu ft/hour	255,269	185,074	cu ft/hour
V_1 (required) - SCFM	**4,254**	**3,085**	SCFM

H. Blower Equipment Sizing and Selection:

1. Adjust to Site Conditions for Schreiber PD Blowers

Note: These calculations are valid for Schreiber PD Blowers **ONLY**!

		DESIGN	OPERATING	Units
P_{std}	Standard Atmospheric Pressure	14.696	14.696	P.S.I.A.
P_{site}	Atmosheric Pressure Due to Altitude	14.441	14.441	P.S.I.A.
P_{inlet}	Inlet Pressure Loss			
P_{v1}	Vapor Pressure of Water at Std Temperature	0.339	0.339	P.S.I.A.
P_{v2}	Vapor Pressure of Water at Inlet Temperature	0.949	0.949	P.S.I.A.
RH_1	Rel. Hum. for Std. Blower Curves	0%	0%	
RH_2	Relative Humidity at Site	30%	30%	
P_1	Std. Atmospheric Pressure Adjusted for Humidity			
	$=P_{std} - P_{V1} X RH_1$	14.70	14.70	P.S.I.A.
	$=P_{std}$	14.70	14.70	P.S.I.A.
P_2	Atmospheric Pressure at Inlet Adjusted for Humidity			
	$=P_{site} - P_{V2} x RH_2 - P_{inlet}$	14.16	14.16	P.S.I.A.
	$=P_{site} - P_{V2} x RH_2$	14.16	14.16	P.S.I.A.
T_1	Standard Inlet Temperature (68 ° F + 460)	528	528	° R (Rankine)
T_2	Actual Inlet Temperature (° F + 460)	560	560	° R (Rankine)
V_2	Volume			
	$=V_1*(P_1/P_2)*(T_2/T_1)$	4,684	3,396	ICFM

I. Summary:

Description	DESIGN	OPERATING	
D.O.	2.00	0.70	mg/L
Lb of O_2/ Lb BOD	1.5	1.2	lb
Relative Humidity	30%	30%	%
Inlet Temperature	100	100	° F
Waste Temperature	28	28	° C
Blower Time / Day	24	24	hours
% Denitrification Achieved	0%	0%	
Total O_2 Req'd (AOR)	13,258	11,552	lb / day of O_2
SOTR - Standard Oxygen Transfer Rate	25,532	18,511	lb / day
SOTR -Standard Oxygen Transfer Rate	1,064	771	lb / hour
V_1 (required) - SCFM	**4,254**	**3,085**	**SCFM**
V_2=V1*(P1/P2)*(T2/T1)	4,684	3,396	ICFM

AUTOMATIC BLOWER CONTROL

The following process control systems are available to provide for the cycling of the blowers and the creating of process sequences to achieve the necessary effluent limits. The control systems are:

A. Dissolved Oxygen Monitoring and Sequencing:

A D.O. probe is installed in each aeration reactor to continuously monitor the change in concentration. The simplest control would be to have a low D.O. value, such as 0.70 mg/l, bring blowers on and a higher value, such as 1.4 mg/l, shut blowers off. This would keep the basin in the oxic mode at all times, but matches the oxygen applied to the biological demand of the reactor. Anoxic conditions can be created with the use of timed intervals, such as shutting all blowers off after the peak D.O. was achieved for intervals up to 1 or 2 hrs. Then bring the blowers on and use D.O. to control again.

This process control will achieve strict BOD and Ammonia limits. The use of timed intervals with the blowers off can have the benefit of partial denitrification resulting in reduced nitrates and enhancement of biological Phosphorus removal.

B. ORP (Oxidation Reduction Potential) monitoring with D.O. control:

An ORP probe is installed in the aeration reactor to continuously monitor the change in potential express as mA. The ORP curves indicate the achievement of nitrification in the oxic phase and the occurrence of denitrification in the anoxic phases. From the observed points of rate change, timed intervals can be used to set the ending of the oxic phase by stopping the blowers and the ending of the anoxic phase with the starting of blowers.

The D.O. control would be the same as described in A and would provide for blower operation in the oxic phase only. This matches the oxygen demand to the actual loading and makes process adjustments even when the biological loading changes. Achieving Total Nitrogen reduction can be accomplished.

C. Ammonia monitoring with D.O. control:

Continuous monitoring of the ammonia concentration can be used to maintain a very strict ammonia limit, but provide for anoxic phases to achieve denitrification with the resulting recovery of alkalinity and oxygen up to 50% of what was needed for nitrification. The concentration and the rate of change that occurs in the reactor basin can be monitored by withdrawing a mixed liquor sample from the reactor, filtering the sample and use a continuous on line monitor. A low value is used to start the controlled anoxic phase and resulting denitrification by shutting off the blowers and a high value will end the anoxic phase by turning on the blowers and starting the oxic phase. By controlling the high value, the required effluent values will be achieved.

The D.O. control would be the same as described in A and would provide for blower operation in the oxic phase only. This matches the oxygen demand to the actual loading and makes process adjustments even when the biological loading changes. Achieving strict Total Nitrogen or Nitrate limits can be done.

D. Nitrate monitoring with D.O. control:

Continuous monitoring of the nitrate concentration and the rate of change that occurs in the reactor basin can be monitored by withdrawing a mixed liquor sample from the reactor, filtering the sample and use a continuous on line monitor. The nitrate value curves indicate the achievement of nitrification in the oxic phase, the achievement of denitrification in the anoxic phase, and indicate the beginning of the anaerobic phase. If denitrification is not achieved to the required degree, this will be indicated, and an outside carbon source can be added to continue the process.

The D.O. control would be the same as described in A and would provide for blower operation in the oxic phase only. This matches the oxygen demand to the actual loading and makes process adjustments even when the biological loading changes. Achieving strict Total Nitrogen or Nitrate limits can be done.

E. Phosphorus monitoring in conjunction with Nitrate monitoring:

To achieve the highest degree of biological P removal, direct P monitoring can be added. The P monitoring will confirm that an anaerobic condition has been achieved and that P is increasing in that stage. If the increase is not sufficient, then a carbon can be added to continue the process. Subsequent aeration in the Oxic mode will remove the highest degree of P in the biological cell matter.

The D.O. control would be the same as described in A and would provide for blower operation in the oxic phase only. This matches the oxygen demand to the actual loading and makes process adjustments even when the biological loading changes. Achieving strict Total Nitrogen or Nitrate limits can be done.

Note: The simplest process control is A, but as more stringent limits are required, the system is upgraded by just adding to the basic control system. No additional tankage needs to be constructed.

All the above blower control systems enable the plant to optimize energy conservation while maintaining a high degree of nutrient removal.

For this application we recommend the following :

___X___ **A. Dissolved Oxygen Monitoring and Sequencing:**
________ **B. ORP (Oxidation Reduction Potential) monitoring with D.O. control**
________ **C. Ammonia monitoring with D.O. control**
________ **D. Nitrate monitoring with D.O. control**
________ **E. Phosphorus monitoring in conjunction with Nitrate monitoring**

Secondary Clarification System

A. Criteria:

1. Secondary clarifier design is based upon a parameter called Reference Sludge Volume for Surface Settling Rate determination.
   (RSv in ml/g = SVI in ml/g x MLSS in g/L)
2. Calculation of SWD shall include Zone for Thickening (H-1), Zone for Storage at Maximum Flow Rates (H-2), Zone for Separation (H-3), and Zone for Clean Water (H-4)
3. Maximum Allowable Surface Settling Rates are at peak sanitary flow, Q-16 hr.
4. SVI is assumed @ 100 ml / g.
5. MLSS @ design is 3,600 mg/L (See Aeration Design)
6. Calculations for Depth:

$$\text{H-1} = \text{Zone for Thickening} = \frac{\text{TSr} \times \text{Isv}}{1{,}000} = \text{meters} \times 3.28 = \text{ft.}$$

H-2 = Zone for Storage -- Assuming RAS Rate is not changed.

$$= \frac{(\text{MLSS @ avg.} - \text{MLSS @ peak}) \times \text{Vol. Aeration in cu ft} \times \text{SVI}}{1000 \times \text{Surface Area Final Clarifiers in sq ft}}$$

H-3 = Zone for Separation = 3'-4" if H-2 is less than 3 ft.,
otherwise H-3 = 1' - 8"

H-4 = 1'- 8" minimum.

B. Maximum Allowable Surface Settling Rate:

Reference Sludge Volume RSv = MLSS x SVI
= 3.60 x 100 = 360 ml /L

Allowable Surface Settling Rate qf = 0.80 m / hr
(Enclosure I, Graph 1)

$$\text{Q16 hr.} = \frac{24\text{ hr.}}{16\text{ hr.}} \times \text{Q avg.} \quad (\ 91{,}667 \text{ gal / hr}) = 137{,}500 \text{ gal/ hr}$$

$$= 520.44 \text{ cu m/hr}$$

$$\text{Minimum Surface Area} = \frac{520.44 \text{ cu m/hr}}{0.80 \text{ m/hr}} = 651 \text{ sq m}$$

$$= 7{,}003 \text{ sq ft}$$

C. Required Depth of Clarifier(s):

Total Depth = H-1 + H-2 + H-3 + H-4

$$\text{H-1} = \text{Zone for Thickening} = \frac{\text{MLSS in Aeration in g / L} \times \text{SVI in ml / g}}{1{,}000}$$

$$= \frac{3.60 \times 100.00}{1{,}000} = 0.36 \text{ meters } \times \mathbf{1.18 \text{ ft}}$$

$$\text{H-2} = \text{Zone for Storage} = \frac{(\text{MLSS @ avg.} - \text{MLSS @ peak}) \times \text{Vol. Aeration} \times \text{SVI}}{1{,}000 \times \text{Clarifier Surface Area}}$$

$$= \frac{(3.60 - 3.30) \times 399{,}007 \times 100}{1{,}000 \times 8{,}143} = 1.47 \text{ ft}$$

H-3 = Zone for Separation ------------	3.33	ft
H-4 = Zone for Clean Water ---------	1.67	ft minimum
Total Depth Required (minimum) ---	7.65	ft
Recommended Side Water Depth	12.50	ft

D. Clarifier Equipment Sizing and Selection:

1. Clarifier Model ----------------------	**TBS**	
Number of Units Required -----	**2**	
Diameter -----------------------------	**72**	**ft**
Side Water Depth ----------------	**12.5**	**ft**

	Each Clarifier		**Total**	
2. HP of each drive motor-------------	0.50	hp	1.00	hp
Bhp required at design-------------	0.40	bhp	0.80	bhp
3. Surface area of each unit ---------	4,072	sq ft	8,143	sq ft
4. Volume of each unit ----------------	50,894	cu ft	101,788	cu ft
5. Approx. Length of overflow weir -	214	ft	427	ft

E. Surface Settling Rate Provided:

$$@\ \ Q\ \text{avg.} = \frac{2{,}200{,}000\ \text{gal}}{8{,}143\ \text{sq ft}} \qquad 270\ \ \text{gal / day / sq ft}$$

$$@\ \ Q\ \text{max.} = \frac{2{,}500{,}000\ \text{gal}}{8{,}143\ \text{sq ft}} \qquad 307\ \ \text{gal / day / sq ft}$$

F. Detention Time Provided:

$$@\ \ Q\ \text{avg.} = \frac{101{,}788\ \text{cu ft}\ \times\ 7.48\ \times\ 24\ \text{hr.}}{2{,}200{,}000\ \text{gal / day}} = \qquad 8.31\ \ \text{hours}$$

$$@\ \ Q\ \text{max.} = \frac{101{,}788\ \text{cu ft}\ \times\ 7.48\ \times\ 24\ \text{hr.}}{2{,}500{,}000\ \text{gal / day}} = \qquad 7.31\ \ \text{hours}$$

G.Weir Overflow Rate:

$$@\ \ Q\ \text{avg.} = \frac{2{,}200{,}000\ \text{gal / day}}{427\ \text{ft. weir}} \qquad 5{,}149\ \ \text{gal / ft / day}$$

$$@\ \ Q\ \text{max} = \frac{2{,}500{,}000\ \text{gal / day}}{427\ \text{ft. weir}} \qquad 5{,}851\ \ \text{gal / ft / day}$$

Fuzzy Filter System Calculations

A. Description of System :

High-rate upflow filtration system that utilizes compressible, synthetic fiber spheres as the medium for filtration. Standard system includes painted steel vessel, galvanized steel internals, air supply for washing filter media, and PLC controls. Media is held in place between a fixed lower perforated plate and an upper moveable perforated plate.

B. Design Criteria :

1. Design Flow (Peak Flow)
   Q = 2.500 MGD = 1,736 gpm

Loading Rates are variable as subject to media compression rate

2. Criteria for Loading Rate of Media Surface Area
   a. System sized for a filtration rate of 34.72 gpm / sq.ft.
   b. Upper hydraulic limit ~ 45 gpm / sq.ft.
3. Air Requirement for Wash System = 15 scfm / sq.ft. of media surface area

C. Total Required Area of Media at Recommended Loading Rate During Filtration :

Surface Area Required = 1,736 gpm / 34.72 gpm / sq.ft. = 50.0 sq.ft.

D. Filter Equipment Sizing and Selection :

Filter Model Size ------------------------	**5' x 5'**
No. of Filter Units -----------------------	**2**
Media Wt. (lbs. / unit) -----------------	**703**
Max. Dirty Headloss (inches) --------	**70**
Max. Operating Height ----------------	**16'-10"**

E. Provided Surface Area of Media & Resulting Loading Rates at Design Flow :

1. Total Surface Area Provided
   2 unit(s) x (5' x 5') = 50 sq.ft.
2. Loading Rate at Design Flow (Peak Flow)
   1,736 gpm / 50 sq.ft. = 34.72 gpm / sq.ft. At Average Flow =30.56 gpm/sq. ft.
3. Loading Rate at Design Flow with 1 Unit(s) Washing or Otherwise Out of Service (Peak Flow)
   (washing cycles typically run for 30 minutes)
   1,736 gpm / 25 sq.ft. = 69.44 gpm / sq.ft. At Average Flow = 61.11 gpm / sq.ft.

** Note: The loading rate at peak flow with 1 unit(s) washing or otherwise out of service is within the hydraulic limits of the filter, but since the loading rate is above that which is recommended, performance may decrease during this period of operation. Depending on factors such as site-specific effluent requirements and plant holding capability, an extra filter unit may be desired.

F. Filter Blower and Drive Motor Data :

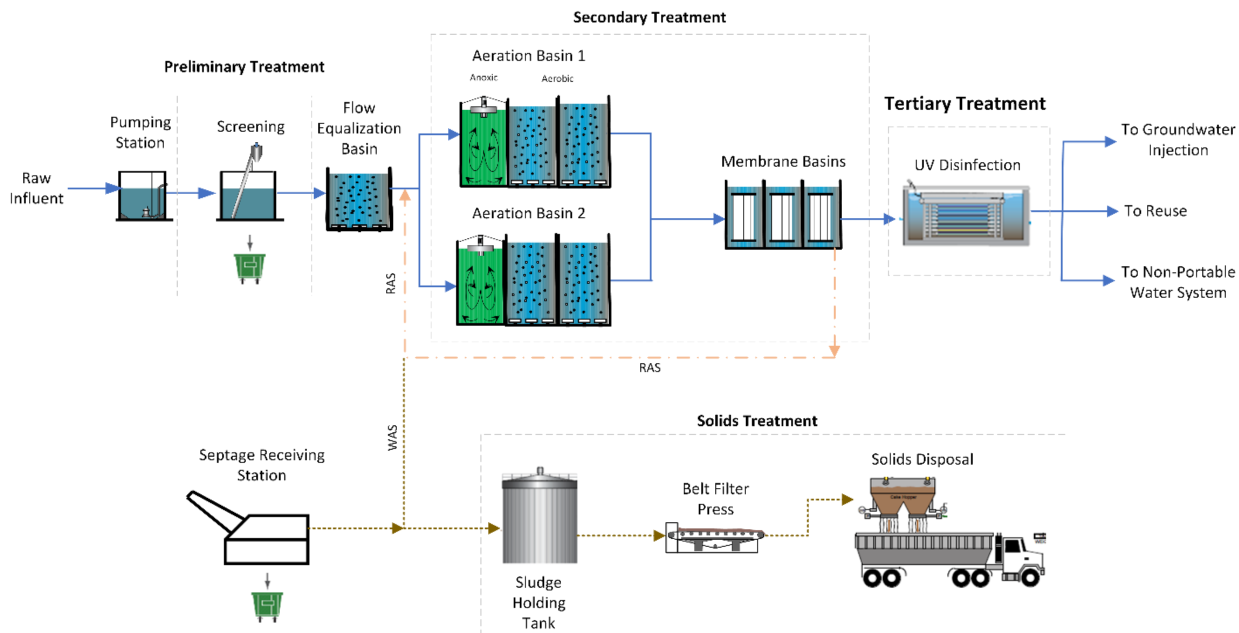
Number of Duty Blowers Required --------------------------------	1	Schreiber PD - 241
Blower HP --	15	HP (13.1 BHP)
Capacity of Each Blower--	375	scfm (@8 psi)
Total Capacity of Duty Blowers ------------------------------------	375	scfm (@8 psi)
Traveling Plate Motor HP ---	7.5	HP

Appendix E
Detailed Capacity Analysis of North Regional WWTP

Appendix E. Detailed Capacity Analysis of North Regional WWTP

An evaluation of each North Regional WWTP main unit process is described in the following sections. Each section discusses the current unit process and any operational problems or opportunities for optimization, provides an analysis of its capacity to treat flows and/or loads through the planning period, and is followed by recommended modifications. The liquids treatment is discussed first, progressing from influent raw sewage to filtered and disinfected effluent, followed by solids handling processes. The general process flow schematic is presented on Figure E-1.

Figure E-1. North Regional WWTP General Process Flow Schematic



Preliminary Treatment

Description of Existing Facilities

The pumped wastewater from the influent pump station (IPS) is received in the Headworks Building, where it is screened to remove rags and other debris. The preliminary treatment consists of the following processes:

- Coarse screen
- Fine screen

The coarse screen at the North Regional WWTP is a continuous multi-rake bar screen manufactured by JWC Environmental. This screen was installed in 2020 to replace the old coarse screen. The fine screen is also manufactured by JWC Environmental, is a Bandscreen Monster model, and was installed in 2019. This screen is a center flow type, which typically captures small particles that otherwise can foul the MBR. There is also a bypass channel with a bar screen that be used in case both sets of screens are offline at the same time. The screenings removed from the coarse and fine screens are sent to the washer and compactor and then disposed of in the City's landfill.

Capacity Evaluation

The existing screening system characteristics are summarized in Table E-1. Based on the 2040 peak hourly flow projections of 7.64 million gallons per day (mgd), both the coarse and fine screening equipment do not have sufficient capacities. In such a scenario, the bypass screening could be used for short time periods.

Table E-1. Screening Design Criteria

Design Criteria	Coarse Screen
Number of screens	2
Bar spacing, inch	1/4
Channel depth, feet	9
Channel width, feet	4
Design capacity each screen, mgd	3.5
Design Criteria	**Fine Screen**
Number of screens	2
Mesh size, mm	2
Channel depth, feet	4
Channel width, feet	9
Design capacity each screen, mgd	3.5

Recommendations

The capacity analysis indicated that both the coarse and fine screens systems do not have sufficient capacity to handle the 2040 peak hourly flows. However, if current projections match actual growth, any potential expansion does not need to occur until close to 2040. When properly maintained, the screen equipment has a typical lifespan of about 15 to 20 years. The screens have been only recently installed and should be able to function until 2040. If the City is looking to expand or modify the existing screening system, it is recommended that the peak hourly projections be revised based on the historical data to reassess the capacity before the expansion or modification. The current peak hour flow projections are based upon literature values, not on actual influent data. Therefore, Jacobs recommends that the City monitor the hourly flows to the North Regional WWTP and based on accumulated data decide on the need for expansion of the screening system. Alternatively, if the peak North Regional WWTP flows do exceed the capacity of the screens in the future, the City has the option to divert some of the flows to the Island WWTP or the Mulberry WWTP in such a scenario.

The screen area was previously poorly ventilated and shows signs of significant hydrogen sulfide (H_2S) corrosion (Figure E-2). Plant operations staff developed a ventilation layout for the screening area, which they had installed approximately a year ago, which has helped significantly lessen H_2S concerns.

The fine screens were installed only several years ago, but the steel enclosure of the screens already shows signs of H_2S corrosion. Severe corrosion was also observed in electrical conduits and in the fire sprinkler system (Figure E-3). Jacobs recommends that a thorough inspection be performed of the electrical systems and piping networks. The grating over the influent channels in the Headworks Building is in poor condition because of historical H_2S corrosion and could potentially pose a safety hazard to the operators. Jacobs recommends that the grating be replaced as soon as possible.

The preliminary treatment at the North Regional WWTP does not include grit removal equipment. Grit removal typically precedes secondary treatment in those treatment plants that do not have primary clarification. Removal of grit prevents unnecessary abrasion and wear of mechanical equipment, grit deposition in pipelines and channels, and accumulation of grit in the flow equalization basin (FEB), aeration basins, and sludge holding tanks. Jacobs recommends including a grit removal system when the Headworks Building is upgraded.

Figure E-2. Screen Enclosure Showing Signs of Corrosion

Note newer ventilation piping along wall

Figure E-3. Typical H_2S Corrosion of the Electrical System and Fire Sprinklers

Septage Receiving Facility

North Regional WWTP accepts septage from hauling/Vactor trucks and has a septage receiving station designed to receive such flows. The facility includes a Honey Monster septage receiving station manufactured by JWC Environmental. The septage station screens and dewaters incoming material, and generates solids that are disposed of in the City's landfill. The remaining liquid and soft solids are discharged to the sludge holding tank via the waste activated sludge (WAS) pump station. The capacity of this facility is approximately 600 gallons per minute (gpm) at peak flow conditions.

Flow Equalization Basin

Flow equalization is carried out to attenuate any peaks/valleys observed in the influent wastewater flows. This provides a nearly constant flow and loads to the downstream processes, improving their performance. Equalization also provides operational flexibility for maintenance or repairs of the downstream processes.

Description of Existing Facilities

At the North Regional WWTP, flow equalization is provided by a single FEB. Flow from the Headworks Building enters the FEB by gravity. At the end of the FEB are the influent pumps, which discharge into the secondary treatment.

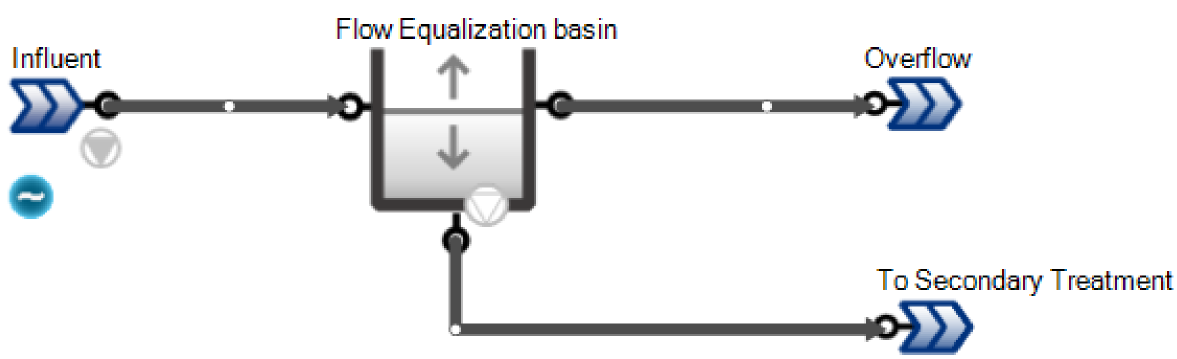
The FEB has a holding capacity of 890,000 gallons and is of a covered type. Mixing is provided through a coarse bubble mixer system to keep the solids suspended in the liquid. Three Flygt submersible pumps are used to convey the wastewater from the tail end of the FEB to the aeration basin splitter box for further treatment.

Capacity Evaluation

The capacity of the FEB was analyzed using Dynamita's SUMO, a commercially available, whole-plant dynamic simulator. A simplified version of the FEB operation was simulated in SUMO at the peak day conditions as shown on Figure E-4. The model did not include any other WWTP processes. The following inputs were used in the model:

- The volume of the FEB is 890,000 gallons; however, because of potential solids accumulation the volume was reduced to 801,000 gallons.
- Initial volume at the start of simulation was 100,000 gallons.
- 2040 peak day influent flows to the North Regional WWTP were used. Peak day flows typically occur only once a year, but flows were simulated for 2 peak days to be conservative.
- Two conditions were simulated: one influent pump operating (1.75 mgd) and two influent pumps operating (3.5 mgd).
- The flow to the downstream equipment is limited to 3.5 mgd, because this is the maximum flow that the disinfection system can handle.
- Pumps are operated at constant speed and turned off when the water level drops to 1 foot in the FEB.

Figure E-4. Sumo Model Configuration for the FEB at North Regional WWTP



Note: No other process units were simulated.

The goal of the simulation was to observe the impact on the storage volume of the FEB at peak day flows and ensure there is no overflow. The overflow in the model represents any excess flows that cannot be stored in the FEB. Figure E-5 shows the response curve that was generated from the model when only one influent pump is operating. It was observed that the FEB would be at 100 percent of its capacity within half a day when only one influent pump is operating. Even when the second influent pump was turned on, overflow was observed in the model response curves as shown on Figure E-6. The magnitude of overflow was lower when two pumps were operating. The flows cannot be increased beyond 3.5 mgd by turning on the third pump because the disinfection system is sized to handle flows up to only 3.5 mgd and the pumps may run into hydraulic issues.

Figure E-5. Response Curve from the Sumo Model – One Influent Pump Operating

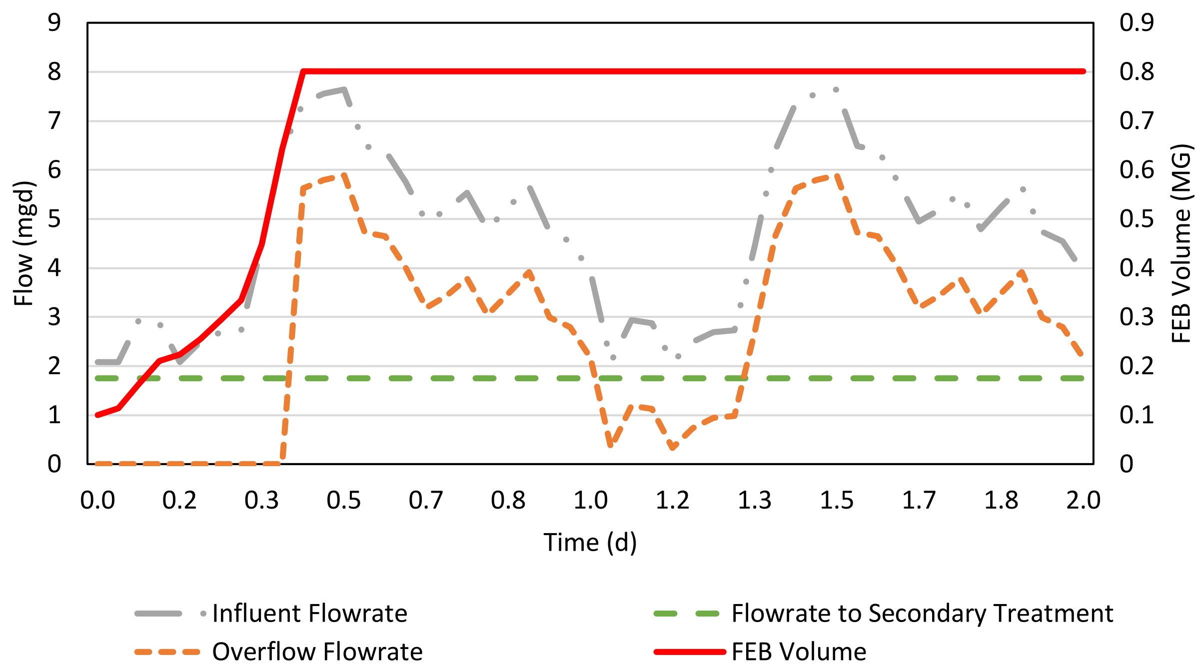
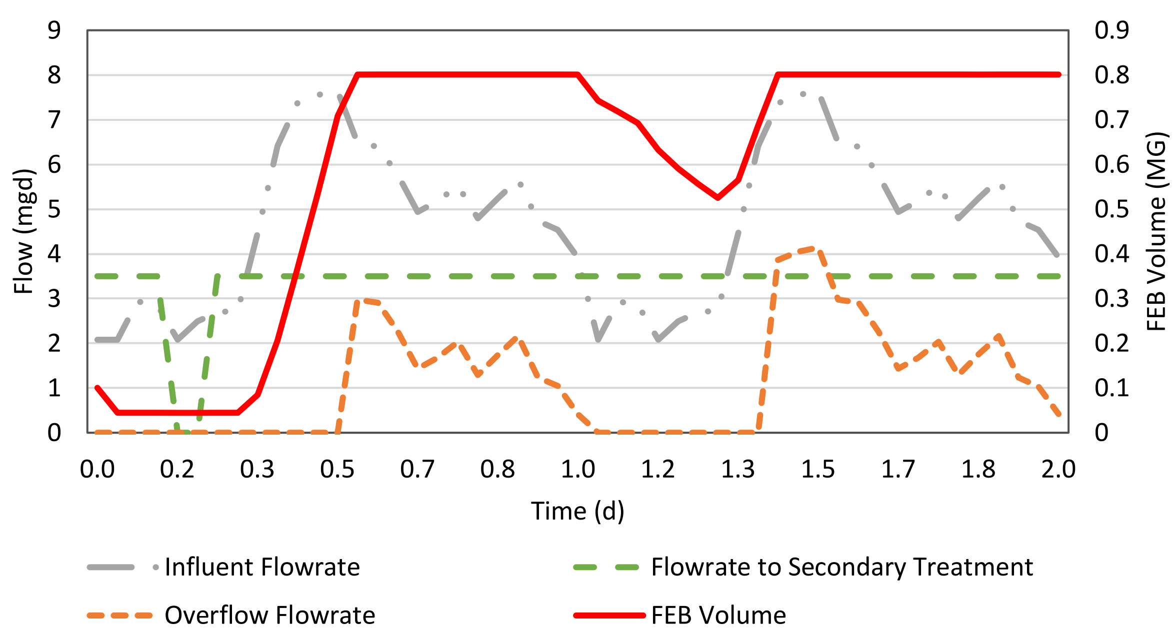


Figure E-6. Response Curve from the Sumo Model – Two Influent Pumps Operating



Recommendation

Overall, the FEB at North Regional WWTP has insufficient capacity to process even one day of the 2040 peak day flows, based on the preceding analysis. The chances of consecutive peak days occurring is low but was selected as a conservative approach to better understand the capacity of the FEB. The FEB has 3 feet of freeboard that may be used in this situation to store some of the excess water, but this is not recommended as a long-term solution. Alternatively, the City may divert some of the flows to the Island WWTP or the Mulberry WWTP in such a scenario. It is recommended that the City carefully monitor the peak flows to the North Regional WWTP and water levels in the FEB, and then decide, based on accumulated data, whether another FEB is needed. Space is available next to the existing tank to install the new FEB if warranted.

Jacobs recommends removing and quantifying the solids accumulated at the bottom of the FEB to provide additional storage volume. Solids have not been removed from the FEB since startup of the North Regional WWTP over 15 years ago. Additionally, Jacobs recommends installing a bypass line around the FEB. Having a permanent bypass line would facilitate FEB cleaning and other maintenance activities and provide operational flexibility.

Flow Equalization Basin Pump Station

The FEB pump system design criteria are summarized in Table E-2. The pump station was evaluated for its ability to treat the 2040 peak hourly flows. Based on the 2040 flow projections and downstream process capacities, the pumps have sufficient capacity to handle the peak flows.

Table E-2. FEB Pump Station Design Criteria

Design Criteria	FEB Pump Station
Number of pumps	3
Type	Dry well submersible
Flow Capacity, each, mgd	1.75 at 35 feet TDH
Configuration	2 Duty/1 Standby
Motor, hp	20
Adjustable Frequency Drive	Yes

hp = horsepower
TDH = total dynamic head

Overall Recommendation

The capacity analysis of the FEB indicated that there is insufficient capacity to process the 2040 peak day flows. Solids accumulated at the bottom of the FEB have not been removed since startup of the North Regional WWTP in 2006. This decreases the useful volume available for wastewater storage and equalization. It is recommended that the City remove the solids from the FEB to fully use the equalization capacity.

The FEB pumps are reported to be operating well and have sufficient capacity to process the 2040 flows. However, the lack of grit removal and accumulation of solids in the FEB may tend to wear out the pumps faster than their typical lifespan. It is recommended that operators pay close attention to the condition of the pumps and undertake necessary maintenance activities to keep the pumps operational. The FEB does not include a bypass pipe, making it difficult to take the unit offline for maintenance. If the FEB experiences any emergency, temporary bypass piping would have to be installed very quickly. It is recommended that the City install a permanent bypass line from the Headworks Building to the secondary treatment, which would provide more flexibility for the operators.

Secondary Treatment

Secondary treatment at the North Regional WWTP uses the membrane bioreactor (MBR) treatment technology to degrade and remove organic compounds and nutrients. MBR includes biological treatment followed by ultrafiltration membranes to remove solids.

Description of Existing Facilities

Aeration Basin

Flow from the FEB is pumped into the aeration basin flow control structure, where the flow is distributed between two aeration basins. Each basin is divided into three zones: one anoxic and two aerobic. Each anoxic zone has three submersible mixers to keep the solids in suspension. Air is provided in the two aerobic zones by fine bubble membrane diffusers.

Membrane Bioreactor

Mixed liquor from the aeration basins enters the membrane basin inlet channel by gravity. The MBR system consists of three ZeeWeed Ultrafiltration Hollow-Fiber trains manufactured by Suez. The membrane filtration system eliminates the need for secondary clarifiers, which are typically found in conventional activated sludge systems, as well as tertiary filtration needed to meet Class A+ water quality requirements.

The membrane cassettes are directly in contact with the mixed liquor, and a vacuum is applied to the header connecting the membranes. The pore size of the membranes is such that only water can be drawn into the hollow fibers by the applied vacuum, resulting in filtration of solids. The permeate from the MBR is extracted by permeate pumps and discharged to the UV system for disinfection. A portion of the permeate water is sent to the permeate storage tank and is used for cleaning the membranes.

Fouling of the membranes can occur during normal operations, reducing the ability to filter out solids. In order to ensure optimum performance of the MBR system, the membranes are often physically cleaned by backpulsing with permeate water. Air scouring is also performed to keep the solids in suspension and away from the membranes. Over a period of time, backpulsing with permeate water and air scouring is not sufficient to clean the membranes and they require a recovery clean. The recovery clean process uses sodium hypochlorite to remove organic material and citric acid to remove inorganic material. These chemicals are stored in tanks at the treatment plant. Jacobs recommends performing recovery cleaning at least 2 to 3 times per year.

Recycle Pump Station

The sludge that is left behind flows by gravity into a common wet well area at the end of the membrane basin. The recycle pump station has three submersible Flygt pumps that convey the sludge back to the aeration basin influent splitter box. A portion of the sludge flow is wasted from the discharge header and sent to the WAS pumps. The pump station has a submersible mixer to keep the solids in suspension.

Aeration Blowers

The aeration blowers are located in the West Blower Building. There are three multistage Continental centrifugal blowers, providing the air required for carbon and nitrogen removal.

Capacity Evaluation

Aeration Basin

A summary of the aeration basin sizing is provided in Table E-3.

Table E-3. Aeration Basin Sizing

Parameter	Value
Geometry	Rectangular
Number of passes per basin	2
Dimensions per basin, feet	154 by 21
Sidewater depth, feet	18.5
Freeboard, feet	2
Volume each basin, million gallons	0.89
Diffusers per basin	1,649
Design SRT, days	15
Design MLSS, mg/L	8,000 to 10,000
Design F:M Ratio, lb. BOD_5/lb. MLVSS	0.33

BOD_5 = 5-day biochemical oxygen demand
F:M = food to mass ratio
ft^3 = cubic feet
lb = pound(s)
mg/L = milligram(s) per liter
MLSS = mixed liquor suspended solids
MLVSS = mixed liquor volatile suspended solids
ppd = pounds per day
SRT = solids retention time

The overall capacity of the aeration basins is dependent on several factors, including aeration capabilities, overall basin volume, primary effluent mass loading, and the solids retention time or operating MLSS concentration. The hydraulic retention time (HRT) is 3.1 hours when one basin is operating and 6.2 hours with both basins operating. The HRT of the basins meets the minimum requirements of 2 to 3 hours as stated in Water Environment Federation (WEF) Manual of Practice

(MOP) Number 8. Additionally, the current and future loadings the aeration basins are less than the design criteria. Therefore, the aeration basins are anticipated to have sufficient capacities to treat the 2040 plant flows and loads.

If the City has concerns about the secondary treatment performance or is looking to expand or modify the secondary treatment, it is recommended that the City carry out a detailed analysis of the secondary treatment using more detailed influent characterization and dynamic simulations.

Membrane Bioreactor

The design criteria for the MBR system and permeate pumps are presented in Table E-4. The membranes have sufficient capacity to treat the 2040 flows and loads.

Table E-4. Membrane Bioreactor Design Criteria

Parameter	Value
Annual Average Flow, mgd	3.5
Maximum Month Flow, mgd	3.5
Peak Hour Flow, total capacity, mgd	3.5
Number of Membrane Trains	3
Number of Cassettes per Train	6
Number of Elements per Cassette	48
Nominal Membrane Area per Element, ft^2	340
Total Membrane Surface Area, ft^2	293.76
Membrane Flux – All Trains Online, gpd/ft^2	12.75
Membrane Flux – One Train Offline, gpd/ft^2	19.12
Design Criteria	FEB Pump Station
Number of Pumps	3
Type	Dry well submersible
Flow Capacity, mgd	1.75 at 35 feet TDH
Configuration	2 Duty/1 Standby
Motor, hp	20
Adjustable Frequency Drive	Yes

ft^2 = square feet
gpd = gallons per day

The permeate pump system design criteria are summarized in Table E-5. The total capacity of the pump station is 5.6 mgd and the firm capacity with one pump offline is 3.76 mgd. The permeate pumps have sufficient capacity to handle the flows coming into the secondary treatment.

Table E-5. Permeate Pump System Design Criteria

Design Criteria	Value
Number of Pumps	3
Type	End Suction Centrifugal
Flow Capacity, mgd	1.88 at 62 feet TDH
Configuration	2 Duty/1 Standby
Motor, hp	40
Adjustable Frequency Drive	Yes

Recycle Pump Station

The recycle pump station design criteria are summarized in Table E-6. These pumps have sufficient capacity to handle sludge flows generated at 2040 conditions.

Table E-6. Recycle Pump Station Design Criteria

Parameter	Value
Number of Pumps	3
Type	Dry well submersible
Flow Capacity, mgd	10.6 at 20 feet TDH
Configuration	2 Duty/1 Standby
Motor, hp	60
Adjustable Frequency Drive	Yes

Aeration Blowers

The existing blower system characteristics are summarized in Table E-7.

Table E-7. Blower System Characteristics

Parameter	Value
Number of Units	3
Blower Type	Multistage Centrifugal
Airflow each Blower, scfm	3,500
Discharge pressure, psi	8.74
Motor, hp	200

psi = pounds per square inch
scfm = standard cubic feet per minute

The theoretical air demand for carbon and nitrogen removal at the 2040 maximum flows and loads was calculated, and it was determined that the existing blowers will meet future demands. The calculation is presented in Table E-8. The parameters used in the calculations were similar to what was used for calculating airflow demand at the Island WWTP and the Mulberry WWTP. The aeration system at the North Regional WWTP has sufficient capacity at the 2040 maximum month conditions because the BOD_5 and ammonia loading are anticipated to be below the design criteria.

Table E-8. 2040 Air Demand for Maximum Month Flow and Loads

Parameter	At 2040 Maximum Month Condition	Notes
O_2 required for BOD_5, lb O_2/day	4,743	Assumed 1.5 lb O_2/lb of BOD_5
O_2 required for Nitrification, lb O_2/day	3,680	Assumed 4.6 lb O_2/lb of ammonia
Total AOR, lb O_2/day	8,423	Credits for denitrification were not considered to generate a conservative value
SOTR, lb O_2/hour	410	Estimated using same factors applied at Island WWTP and Mulberry WWTP
Airflow, scfm	2,300	

AOR = actual oxygen requirement
O_2 = oxygen
SOTR = standard oxygen transfer rate

Recommendations

The initial analysis of the secondary treatment indicated there is sufficient capacity to treat the 2040 maximum flows and loads. The analysis provided in this Wastewater Master Plan is at a high level, and it is recommended that the City carry out a detailed analysis of the secondary treatment to determine the actual capacity. It is also recommended that the operators pay close attention to the BOD_5 and ammonia loadings to the North Regional WWTP, to ensure that there are no disruptions to the nutrient removal processes.

The aeration basins appeared to be in good condition during the site walkthrough. Only one out of the three centrifugal blowers was in operation at the time of the site visit. The City is working on repairing the remaining blowers to ensure redundancy. It is recommended that the City further automate the airflow system at the North Regional WWTP by including automatic valves on the header pipe going to the two aeration basins, and additional dissolved oxygen (DO) probes in the basins. This will allow for greater control over the air being delivered to the aeration basins and provide potential cost savings for energy.

Over time, the ultrafiltration membranes have reached the end of their useful life and are being replaced one membrane train at a time. The recovery clean process is being performed only once every 1.5 years instead of once every 4 to 6 months, as recommended. The operators have indicated that the Island WWTP and the Mulberry WWTP are unable to deal with the flows diverted from the North Regional WWTP during the cleaning cycle, which typically lasts for a couple of days. The lack of regular recovery cleaning will lead to reduced membrane life and the need for more frequent replacements.

Disinfection

Description of Existing Facilities

The North Regional WWTP uses a low-pressure high intensity Trojan UV3000Plus system to disinfect the filtered effluent to meet permit requirements for Class A+ Reclaimed Water. Ultraviolet (UV) radiation can penetrate the cell walls of pathogenic organisms and damage the DNA or RNA strands. This results in the organism being unable to perform cellular functions, ultimately leading to the death of the organism. The disinfected effluent is then discharged into the reuse tank, and subsequently sent to either the vadose wells for groundwater injection or to the non-potable process water system.

Capacity Evaluation

The UV facility consists of 1 channel with 2 banks and 48 UV lamps per bank. The flow from the permeate pumps enters a common header and is distributed to the UV channel. Space has been provided to install a second channel of UV modules to treat increased flows in the future. The UV disinfection design criteria are summarized in Table E-9. Based on the 2040 peak hourly flow projections for the North Regional WWTP, the UV disinfection system is estimated to have sufficient capacity as the incoming wastewater is equalized and then sent to the downstream processes. However, the facility lacks redundancy because there is only one UV channel. If the UV system is offline because of an emergency or maintenance, the entire WWTP would need to be shut down.

Table E-9. Disinfection Design Parameters

Design Criteria	UV Disinfection System
Type	Open Channel
Number of Channels	1
Number of Modules	2
Number of Lamps per Module	48
UV Transmittance	65% at 254 nanometers
Maximum Design Flow	3.5 mgd
Disinfection Standards	≤ 23/100 milliliter fecal coliform based on 1 day maximum

Recommendation

The capacity analysis of the UV disinfection system indicates that the system has sufficient capacity to handle the 2040 peak hourly flows but lacks redundancy.

The UV disinfection system was installed in 2006 and has been in service for nearly 16 years. The typical lifespan of the Trojan UV3000Plus is 20 to 25 years with proper maintenance of the system. The system appeared to be in good condition during the site walkthrough. The operators have indicated that they use third-party UV lamps instead of the recommended Trojan lamps because of budgetary constraints. This may potentially result in performance issues such as reduced UV dosage. Jacobs recommends using Trojan bulbs for optimal UV system performance. Additionally, the disinfection system controller is not connected to the plant supervisory control and data acquisition (SCADA) system. Jacobs recommends that the UV lamps be replaced with Trojan lamps to ensure optimum performance. The City should anticipate having to replace the entire system in the next 10 years. Jacobs recommends constructing additional UV channels to ensure that this system has sufficient redundancy.

Solids Handling

The solids treatment at the North Regional WWTP includes a WAS pump station, a sludge holding tank, and dewatering equipment to process the WAS generated by the secondary treatment.

Description of Existing Facilities

WAS from the secondary treatment is diverted from the recycled pump station discharge pipe to the sludge holding tank using two submersible Flygt pumps. The WAS pump station also receives flows from the septage receiving facility.

The sludge holding tank at the North Regional WWTP is a 397,000-gallon concrete tank. The holding tank is equipped with coarse bubble mixers and is aerated to mix the solids and provide minimal oxidation of the volatile solids in the sludge. There are two positive displacement blowers that supply air to the sludge holding tank. Air is typically supplied only for 30 to 60 minutes per day. For the remaining time, the solids are allowed to settle in the bottom of the tank. The plant operators then decant the holding tank and route the decanted liquid back to the plant drain pump station. The solids in the tank are generally thickened up to 1 to 2 percent total suspended solids.

The settled solids are drawn from the bottom of the holding tank by two belt filter press (BFP) feed pumps and discharged to the BFP. These pumps are progressive cavity type manufactured by the Netzsch Corporation. A single 2-meter BFP by Ashbrook Simon-Hartley (now part of Alfa Laval) is located in the northern portion of the Headworks Building. The BFP is used to dewater the sludge to reduce its volume. The dewatered cake is discharged into a chute and loaded into the hauling truck, which then transports the cake to the City's landfill for disposal. Sludge is conditioned with polymer addition prior to dewatering. The filtrate from the BFP process is recycled back to the plant drain pump station. At the North Regional WWTP, sludge is typically dewatered 5 days per week, generating approximately two truckloads of dewatered cake per day.

Capacity Evaluation

Waste Activated Sludge Load Projections

The WAS data collected between 2015 and 2021 was analyzed and 30-day rolling averages were determined for the WAS mass flow rate and concentrations. The assumptions used in the development of future 5-day biochemical oxygen demand (BOD_5) loads at the North Regional WWTP were applied to estimate the WAS mass flow rate in 2040. The loads were escalated by compounding with an annual growth rate of 0.7 percent, which was obtained from the City's 2016 General Plan (Lake Havasu City 2016). Table E-10 summarizes the historical and future WAS flows. There is no data available on the volume or strength of liquids generated by the septage receiving station, which is anticipated to be only a small portion of the overall WAS flows. Therefore, this flow is not considered in the analysis.

Table E-10. Historical WAS Flow Rate and Anticipated WAS Flow Rate in 2040

Parameter	Historical Data	2040 Flows and Loads	Notes
WAS Mass Flow Rate, ppd	5,200	5,940	Only includes sludge generated by secondary treatment. Liquid from the septage receiving station is not considered in the analysis because it is only a small portion of the WAS flows.
WAS Average Concentration, mg/L	7,000	7,000	The WAS concentrations from historical data were assumed for the 2040 conditions as well.
WAS Flow Rate, gpd	89,000	101,700	

Waste Activated Sludge Pump Station

The WAS pump station design criteria are summarized in Table E-11. The pump station has sufficient capacity to convey the projected WAS flows to the sludge holding tank in 2040.

Table E-11. WAS Pump Station Design Criteria

Design Criteria	WAS Pump Station
Number of Pumps	2
Type	Submersible
Flow Capacity each pump, gpm	450 at 53 feet TDH
Configuration	1 Duty/1 Standby
Motor, hp	10 hp

Sludge Holding Tank

The sludge holding tank and blower characteristics are summarized in Table E-12. Wasting is typically performed for 5 days per week and the dewatering equipment is operated 4 to 5 days per week. The capacity of the sludge holding tank is determined by its HRT. The HRT of the tank under current conditions is based on the historical data from 2015 to 2021 and was calculated at 4.5 days. With increased WAS flows in 2040, the HRT reduces to 3.9 days. Because dewatering is typically done 4 to 5 days per week, if the BFP has to be taken offline there is approximately 1 weeks' worth of storage. The sludge holding tank has sufficient capacity to handle the projected 2040 WAS flows. The sludge holding tank is reportedly aerated between 30 and 60 minutes per day, which is significantly less than the aeration times of the Island WWTP or the Mulberry WWTP.

Table E-12. Design Criteria for the Sludge Holding Tank and Blowers

Design Criteria	BFP
Number of Tanks	1
Geometry	Circular
Diameter, feet	65

Table E-12. Design Criteria for the Sludge Holding Tank and Blowers

Design Criteria	BFP
Sidewater Depth, feet	15.5
Volume, gallons	397,000
Blowers	
Blower Type	Positive Displacement
Number of Blowers	2
Capacity of Each Blower, scfm	1,000

Belt Filter Press Feed Pumps

The design criteria for the BFP Feed Pumps are summarized in Table E-13. Because the sludge is slightly thickened in the sludge holding tank, the flow to the BFP is less than the WAS flow rates described in previous sections. The historical and anticipated feed rates to the BFP are presented in Table E-14. The assumptions made in the projections are provided in the table. The BFP Feed Pumps have sufficient capacity to handle the 2040 projected sludge flows.

Table E-13. BFP Feed Pump Design Criteria

Design Criteria	BFP Feed Pumps
Number of Pumps	2
Type	Progressive Cavity
Flow Capacity each pump, gpm	250 at 30 psi
Configuration	1 Duty/1 Standby
Motor, hp	15

Table E-14. BFP Feed Rates – Historical and 2040 Projection

Parameter	Historical Data (2015-2021)	2040 Flows and Loads	Notes
Mass Flow Rate of Sludge to be Dewatered, lb/5 days	20,800	23,800	This represents how much sludge is remaining in the bottom of the tank each day. Assumed that 20 percent of the solids remains in the decant liquid.
Sludge Concentration, percent	1.5	1.5	1.3 percent was observed in the historical data. The same concentration is assumed for the 2040 projection.
Gallons of Sludge to be Dewatered, gallons/5 days	166,200	189,800	Gallons of sludge generated every 5 days needing to be dewatered.

Dewatering

The design criteria of the dewatering equipment at the North Regional WWTP are summarized in Table E-15.

Table E-15. Dewatering Equipment Design Criteria

Design Criteria	BFP
Number of Units	1
Belt Width, meter	2
Sludge Feed, percent	0.5 to 2
Hydraulic Capacity, gpm per meter	100
Solids Loading, lb/hour	1,400
Cake Solids, percent	16
Solids Capture Rate, percent	95

Note:

Design criteria for the BFP were obtained from the specifications developed during the construction of the North Regional WWTP.

The capacity of the dewatering equipment was evaluated based on the solids loading rate (SLR) and hydraulic loading rate (HLR) to the BFP. The inlet flows to the BFP were developed in the previous section. The estimate assumed a run time of 6 hours per day for a total of 5 days. The results of the capacity analysis for the historical data and 2040 conditions are presented in Table E-16. The SLR and HLR for both the conditions are below the manufacturer's design criteria. The cake total solids produced at the North Regional WWTP is higher than what is produced at the other two WWTPs, but it can be further improved to reduce hauling costs. The BFP has sufficient capacity to handle the additional flows in 2040.

Table E-16. Dewatering Equipment Capacity Analysis

Parameter	Historical Data (2016-2021)	2040 Flows and Loads	Notes
Hours of operation per day, hours	6	6	Based on the information provided by the operators. Same conditions have been assumed for 2040.
Number of days BFP is operated, days	5	5	Based on the information provided by the operators. Same conditions have been assumed for 2040.
Sludge to be processed per day, ppd	4,160	4,752	Based on the mass flow rate estimated in the previous section.
Sludge to be processed per day, gpd	33,200	38,000	Based on the flow rate estimated in the previous section.
Solids Capture Rate, percent	95	95	Provided by the BFP manufacturer.
Solids Loading Rate, lb/hour	693	792	
Hydraulic Loading Rate, gpm	92	106	

Table E-16. Dewatering Equipment Capacity Analysis

Parameter	Historical Data (2016-2021)	2040 Flows and Loads	Notes
Cake Produced, dry pounds/day	3,950	4,520	
Cake Total Solids, percent	14	14	Average value based on the historical data (2015 to 2021). Same percentage has been assumed for the 2040 conditions.
Cake Produced, wet pounds/day	28,230	32,250	Hauled to the City landfill.

Recommendations

The WAS flows that could be generated in 2040 were developed using the same methodology that was used to develop influent BOD_5 loads. Overall, the different unit processes of the solids treatment have sufficient capacity to process the 2040 flows and loads. The analysis assumed the same conditions observed historically would still be applicable in 2040. Any changes to the WAS stream resulting from operational changes or the addition of a new solids stream at the North Regional WWTP will require a reanalysis of the solids treatment.

The cake total percent solids generated at the North Regional WWTP is higher than Island WWTP or the Mulberry WWTP but still below the typical value of 15 to 18 percent from a BFP. This means that every truck load of cake hauled to the landfill has a higher water content than design conditions and results in higher hauling costs. The proposed solutions to this concern have been identified and are discussed in Section 6.7, Biosolids Management Plan. Solutions include operational changes and optimizing polymer solution and dosage.

Similar to the screening room, severe H_2S corrosion was observed in the BFP dewatering room in the Headworks Building. During the site visit, severe corrosion was observed in the overhead pipes, pipe fittings, flanges, BFP overhead supports, and electrical conduits as shown on Figure E-7. It is recommended that the City immediately improve the ventilation in the room, and subsequently replace corroded piping and electrical fixtures.

Figure E-7. Hydrogen Sulfide Corrosion on the Fire Sprinkler Piping and BFP Overhead Supports

Odor Control

The odor control unit at the North Regional WWTP is a centralized Siemens wet scrubber located adjacent to the aeration basins. The odor control unit serves the Headworks Building, FEB, aeration basin flow control structure, septage receiving station, and sludge holding tank to prevent nuisance odors being released into surrounding areas. Foul air is pulled from these areas and sent to the scrubber, where hydrogen sulfide and ammonia are removed by caustic solution and sodium hypochlorite, and then discharged into the environment.

The odor control unit has reached the end of its useful life, maintenance and chemical costs are rapidly increasing, and the unit needs to be replaced in the near future. Jacobs recommends installing a biological odor control unit in place of the wet scrubbers. The biological filter would eliminate the need for the City to procure chemicals, resulting in significant savings in chemical costs. A biological unit would also be relatively easier to operate and would be more energy efficient.

Non-Potable Water System

A portion of the reuse water is pumped back through the WWTP's non-potable water system. The non-potable water is used for routine floor and equipment washing, irrigation of landscape within the

WWTP property, and for BFP wash water. The system includes a hydropneumatic tank and two vertical turbine pumps. The design criteria for the Fairbanks pumps are summarized in Table E-17.

Table E-17. Non-potable Water System Design Criteria

Design Criteria	Non-potable Water Pump System
Number of Pumps	2
Type	Horizontal Split Case Centrifugal
Flow Capacity, gpm	748 at 208 feet TDH
Configuration	1 Duty/1 Standby
Motor, hp	75

Plant Drain Pump Station

The plant drain pump station receives flows from different parts of the WWTP. Flows from the sludge holding tank, BFP, screens washpactor, waste chemical from the odor control unit, and from the administration building are collected and then pumped to the inlet channel of the screens in the Headworks Building. The design criteria for the Flygt pumps are summarized in Table E-18.

Table E-18. Plant Drain Pump Station Design Criteria

Design Criteria	Plant Drain Pump Station
Number of Pumps	2
Type	Vertical Turbine
Flow Capacity, gpm	500 at 44 feet TDH
Configuration	1 Duty/1 Standby
Motor, hp	10

Appendix F
North Regional Wastewater Sub-Area Master Plan Technical Memorandum

North Regional Wastewater Sub-Area Master Plan Technical Memorandum

Date:	November 21, 2022
Project name:	Lake Havasu City Wastewater As-Needed
Project no:	W9Y34401
Attention:	Jarod Thiele
Client:	Lake Havasu City
Prepared by:	Jacobs
Document no:	PPS0725221736RDD
Revision no:	Draft

Jacobs
1501 West Fountainhead Parkway
Suite 401
Tempe, AZ 85282
United States
T +1 480 966 8188
F +1 480 966 9450
www.jacobs.com

In November 2022, Jacobs finalized the 2022 Wastewater Master Plan (WWMP) for Lake Havasu City (LHC outlining sewer infrastructure needs over the next 20 years to support the City's planned growth. The Victoria Farms Development, located in the northeast portion of the City, is undergoing the development review process and has requested LHC provide a plan for regional sewerage that can be implemented within its development footprint and aligns with the 2022 WWMP. This technical memorandum supplements the 2022WWMP (and is included in Appendix F) and identifies the recommended backbone infrastructure for the North Regional Sub-Area, which encompasses the Victoria Farms Development.

LHC anticipates an increase in development activity in the northern portion of the City on the west and east sides of State Route (SR) 95. As aprt of finalizing this technical memorandum several proposed development projects were reviewed in the area for potential future sewer service and optimization of the City's sewer collection system. The timing of several of these proposed development sis unknown and will be contingent on the sale and auction of State Lands. Based on a review of the development plans and meetings with the City, several modifications were made to the North Regional Sewer system, primarily future lift station locations.

The proposed sewer consolidation and optimization projects and cost estimates are included in the2022 WWMP capital plan as follows:

- Lift Station Optimization (Alternative 2) as "North End Wastewater System Expansion"
- Victoria Farms as "Area A Expansion"
- Proposed Development west of SR 95 as "Area B and Area C Expansions"

1. Purpose

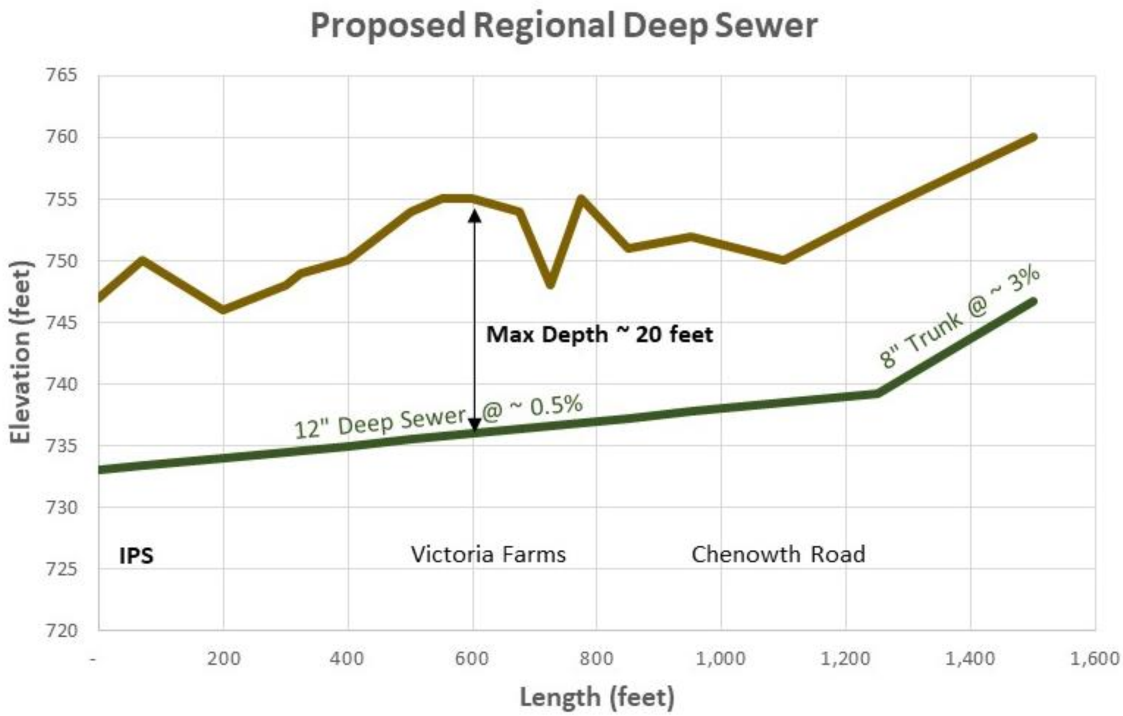
The purpose of this technical memorandum is to identify the backbone sewer system in the North Regional Sub-Area (documented as Area A in the WWMP), and to evaluate alternatives for optimizing the existing regional pump system by offloading the Bombay Sewer Lift Station (Bombay SLS). Lift station optimization alternatives include (1) extending a new force main across Highway 95 along Chenoweth Drive to the new backbone sewer and (2) determining whether the Canterbury Sewer Lift Station (Canterbury SLS), Refuge Sewer Lift Station (Refuge SLS), and North Palo Verde Sewer Lift Station (North Palo Verde SLS) can be replumbed to the new force main extension.

2. Sewer Facilities Concept Layout

The WWMP presents a conceptual layout for future backbone sewerage expansion in four areas. Area "A" is the area of interest for this study. The conceptual layout provides for maximizing gravity flows to the Influent Pump Station (IPS) and incorporating the Victoria Farms proposed circulation network. The conceptual layout consists of a gravity sewer connection to the IPS, a new 20-foot-deep 12-inch-diameter trunk sewer, two 8-inch-diameter trunk sewer extensions, and two local lift stations and force mains. Figure 1 presents the conceptual layout. Figure 2 presents the anticipated profile of the deep sewer.

Figure 1. Sewer Facilities Concept Layout

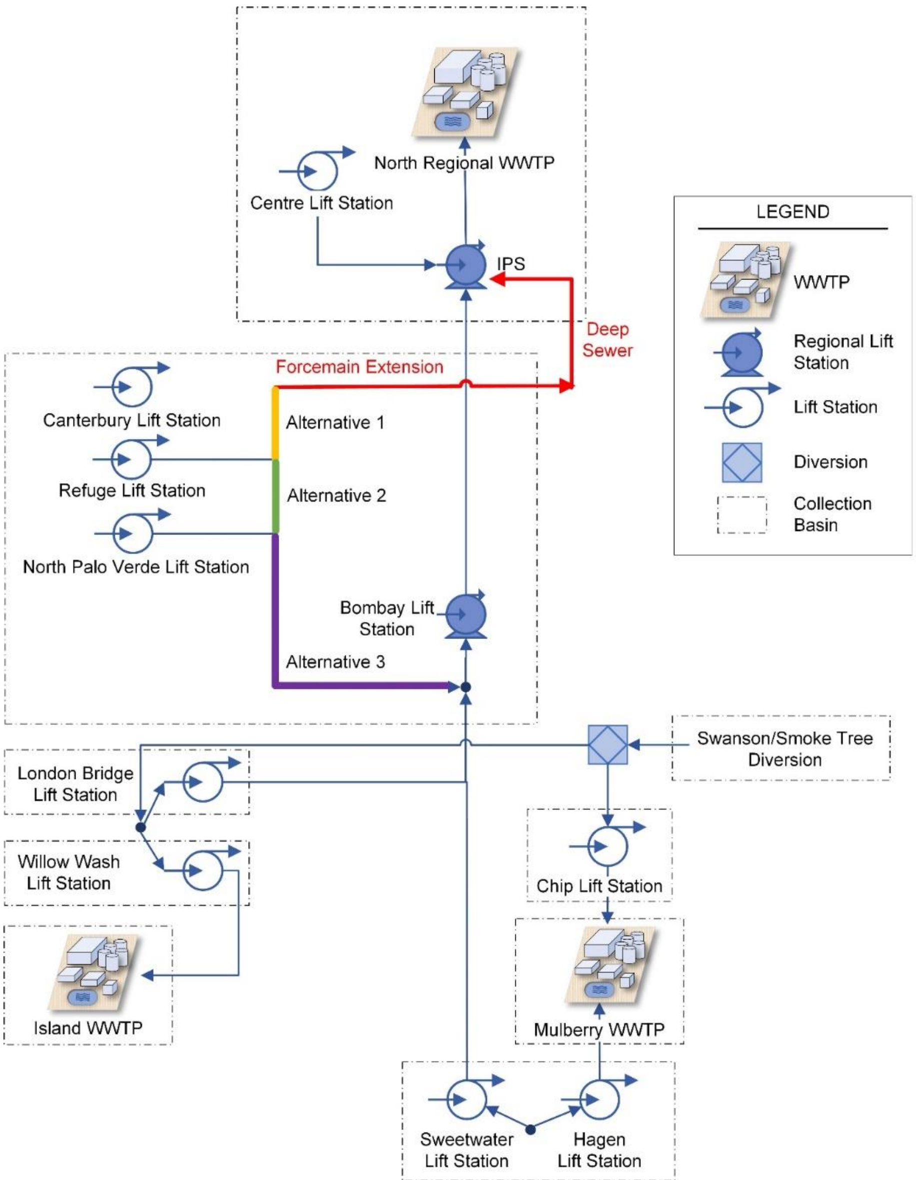
Figure 2. Proposed Deep Sewer Profile



3. Lift Station Optimization

With the conceptual sewer layout in place, an optimization evaluation was performed to assess whether the three lift stations on the west side of Highway 95 could divert flows to the new backbone sewer and away from the Bombay SLS . The benefits of this optimization include reducing energy costs from re-pumping, and potentially abandoning sections of the combined force main west of Highway 95, which has experienced numerous breaks over the past several years. Optimization alternatives are described in the following paragraphs. Figure 3 presents the regional sewerage system highlighting the proposed optimization alternatives.

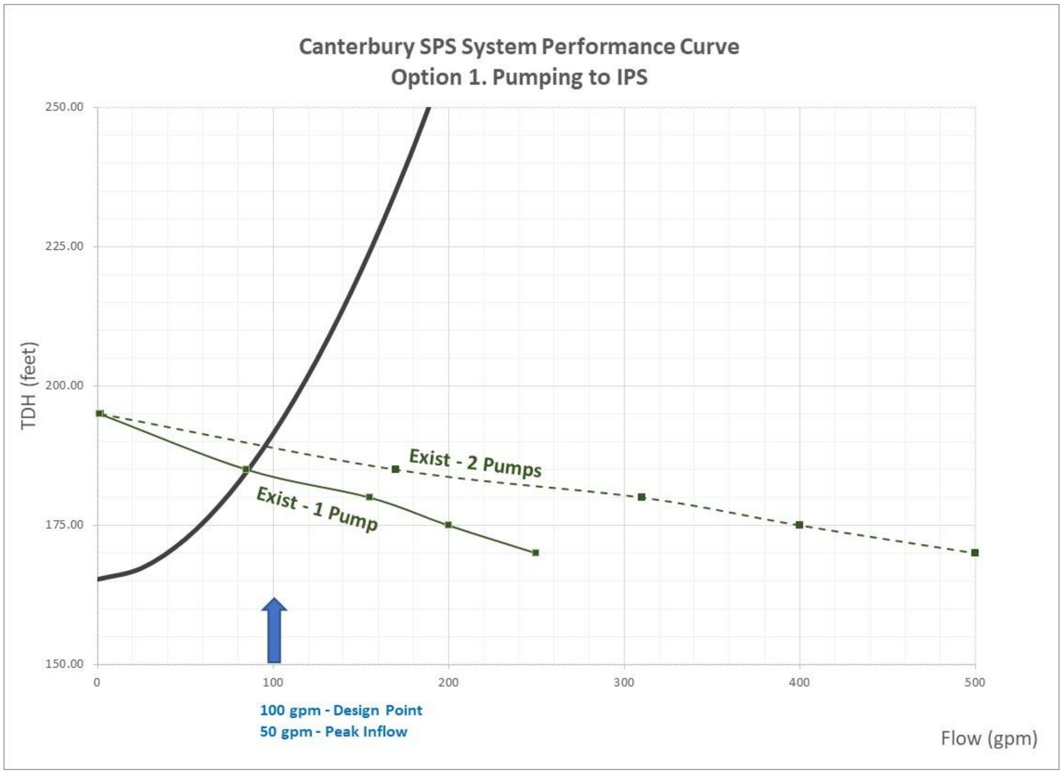
Figure 3. Lift Station Optimization Alternatives



3.1 ALTERNATIVE 1: CANTERBURY LIFT STATION REPLUMB

Alternative 1 would divert flows from the Canterbury SLS through the force main extension to the new backbone sewer. At the 90-degree bend along Chenoweth Drive, a cap would be placed along the force main, which would result in abandoning 3,900 feet of the 8-inch combined force main between the Canterbury SLS and the Refuge SLS force main connection. From the bend along Chenoweth Drive, a 1,700-foot extension of the force main would allow a connection to the proposed backbone sewer. An evaluation of the existing pumps at the Canterbury SLS for this changed condition was conducted as shown on Figure 4. Based on these calculations, no improvements will be required to connect the Canterbury SLS to the proposed backbone sewer. The maximum pressure in the existing force main is anticipated to be approximately 85 pounds per square inch (psi), which should be within the pipe's pressure rating.

Figure 4. Alternative 1 Canterbury Lift Station System Performance Curve



3.2 ALTERNATIVE 2: CANTERBURY AND REFUGE LIFT STATION REPLUMB

Alternative 2 would divert flows from the Canterbury SLS and the Refuge SLS through the force main extension to the new backbone sewer. This alternative proposes to cap the southside of the common force main connection with the Refuge SLS force main and abandon approximately 7,000 feet of existing 8-inch force main south of the Refuge SLS to the connection point with the North Palo Verde SLS force main. An evaluation of the existing pumps at the Refuge SLS and Canterbury SLS are shown on Figures 5 and 6. It appears that no improvements would be required to connect the Refuge SLS and Canterbury SLS to the proposed backbone sewer, and it would allow for the abandonment of portions of the common force main. The maximum pressure in the existing force main is anticipated to be approximately 150 psi, which is high enough to warrant verifying the existing force main's pressure rating. A pre-design report is recommended to confirm the maximum operating pressure. A few segments of force main may need to be replaced.

Figure 5. Alternative 2 Canterbury Lift Station System Performance Curve

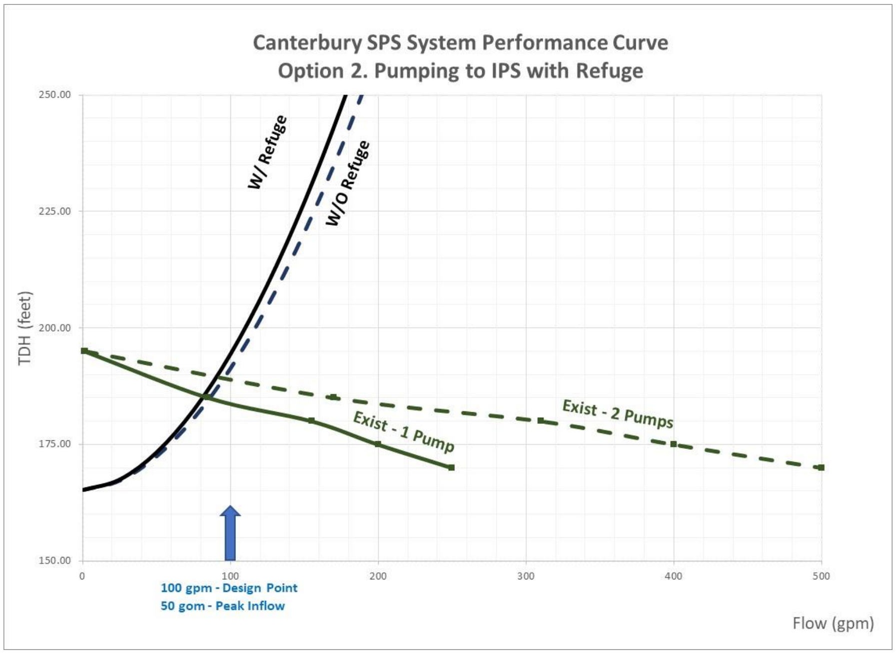
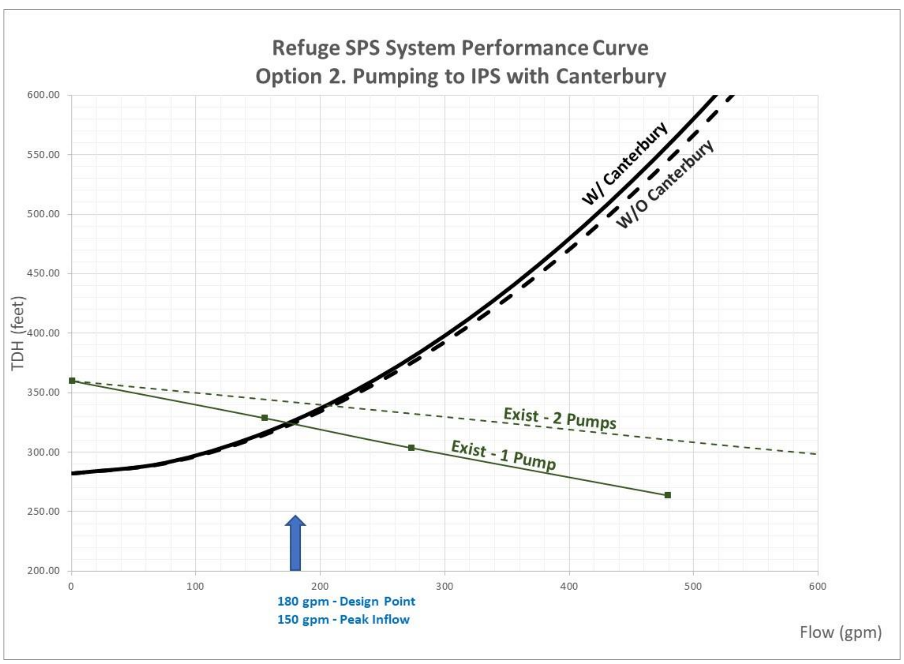


Figure 6. Alternative 2 Refuge Lift Station System Performance Curve



3.3 ALTERNATIVE 3: CANTERBURY, REFUGE, AND NORTH PALO VERDE LIFT STATION REPLUMB

Alternative 3 would divert flows from the Canterbury SLS, Refuge SLS, and the North Palo Verde SLS through the force main extension to the new backbone sewer. Alternative 3 would place a cap at the south end of the connection point at North Palo Verde SLS with the common force main. The additional static head needed to divert flows from the North Palo Verde SLS is too great and would likely require the pump station to be rebuilt with a series pump operation or an intermediate pump station. It is recommended that the North Palo Verde SLS continue to pump to the Bombay SLS.

4. Recommendations

Based on the analysis presented in this technical memorandum, it is recommended that Area "A" be planned for the backbone infrastructure identified and that the City implement Alternative 2 lift station optimization. The recommended improvements are summarized as follows:

- Backbone Infrastructure
  - 1,250 feet of 12-inch-diameter deep sewer
  - 5,000 feet of 8-inch-diameter trunk sewer extensions
  - North Local Pump Station and 750 feet of 6-inch-diameter force main
  - South Local Pump Station and 2,500 feet of 6-inch-diameter force main
- Lift Station Optimization
  - Hydraulic analysis of existing force mains to confirm pressure class rating are not exceeded
  - 1,700 feet of 8-inch force main extension
  - Abandon West Site combined force main between the Refuge SLS and North Palo Verde SLS