CITY OF LOWELL

LANE COUNTY, OREGON

WASTEWATER FACILITIES PLAN

City of Lowell

Civil West Engineering Services, Inc.

SEPTEMBER 2024

Wastewater Facilities Plan

City of Lowell

Lane County, Oregon

September 2024

City of Lowell

Civil West Engineering Services, Inc.

REGISTERED PROFESSIONAL ENGINEER
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OREGON
NOVEMBER 14, 2023
CLINTON M. CHENEY
RENEWS: 12/31/2025

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September 24, 2024

Max Baker, Public Works Director
City of Lowell
P.O Box 490
Lowell, OR 97452

Re: City of Lowell– DEQ Approval of 2024 Wastewater Facilities Plan
WQ – Lane County
NPDES# 101384 EPA Reference # OR002004-4

Dear Max Baker:

DEQ approves of the September 2024 revised Wastewater Facilities Plan for the City of Lowell. The revised plan was received electronically on September 17, 2024, from Clinton Cheney, P.E., with Civil West and adequately addresses DEQ's comments.

This approval is valid for five years. If implementation of this plan is not completed within five years of this letter, please consult DEQ Clean Water State Revolving Funds staff to ensure the proposed plan and issues are still timely. An update or new facilities plan may be required after five years.

Wastewater planning must comply with statewide land use goals and be consistent with locally acknowledged comprehensive land use plans. Please be aware that any land use or zoning changes may cause delays or require adjustment to the facility plan. In addition, DEQ will require an affirmative land use compatibility statement before reviewing the predesign report.

Overview of Plan

The plan includes a review of the existing wastewater facilities including the collection system, treatment system, and waste sludge management. The need for the project is based on:

1) New regulations – The plan does not anticipate any new regulations that would affect future treatment requirements during the planning period. While the receiving water is water quality limited for mercury and temperature, reduction of these pollutants is typically accomplished by either source reduction for mercury or effluent trading for temperature. However, the plan recommends implementing a treatment alternative that would reduce ammonia in anticipation of potential future requirements.
2) Aging infrastructure: The plan describes age-related deficiencies in the wastewater system, including:
    a. Deteriorated sewage collection system maintenance holes and gravity sewer pipe need to be replaced.
    b. The aeration basin and chlorine contact basin were last upgraded in 1990 and these units are well past the typical 20-year usable life.

    c. Much of the treatment plant was last upgraded in 2004 and while it has been well maintained and in good condition, the equipment is now past the typical 20-year useful life.
3) Growth – The system needs some growth-related improvements. Sections of the gravity sewer system have bottlenecks that need to be upsized. The Alder Street pump station needs to be upsized to accommodate future growth.

The recommended alternative for wastewater treatment is:

- Convert the treatment from trickling filters to conventional activated sludge.
- Install a new blower for the aeration system that serves the solids stabilizations.
- Decommission the trickling filter and build a new secondary clarifier in its place.
- Replace the existing disinfection system with ultraviolet disinfection.
- Sludge drying bed improvements.

The recommended alternative for collection system improvements is:

- Upgrade the capacity of the Alder Street lift station.
- Upgrade the capacity of Moss Street gravity sewer.
- Implement an inflow and infiltration reduction program.

DEQ agrees that the recommended alternative will meet current regulatory requirements. The plan also includes provisions to address foreseeable future regulatory requirements.

What does this approval mean?
DEQ's approval means that the plan satisfies the Clean Water State Revolving Fund planning requirement for an engineered planning document under Oregon Administrative Rule 340-054-0022(6)(a). This is not an approval for CWSRF funding. The City of Lowell may use this plan to apply for CWSRF funding for the projects in the plan. Funding agencies will need to review additional finance information to determine funding for the project(s). Additionally, DEQ's approval does not apply to USDA Rural Development, Business Oregon, and other funding agencies, who may require additional information and/or plan revisions.

Next steps
If you have not already done so, the next step is to request a One Stop Financing Roundtable to determine funding alternatives. See https://www.oregon.gov/biz/Publications/One-Stop.pdf for more information.

While the plan provides preliminary information on environmental issues, it does not meet the requirements for a National Environmental Policy Act review as required if state or federal funds are sought to fund this project. The city will need to prepare a separate environmental review document. The contents of the environmental review document vary by funding agency. See *Preparing Wastewater Planning Documents and Environmental Reports for Public Utilities* (https://www.oregon.gov/deq/FilterDocs/FacilitiesPlansGuidelines.pdf).

To avoid extra work and expense, DEQ recommends that the city wait to authorize final design until a pre-design report is reviewed and agreed upon by the city and DEQ. See DEQ's guidance document "Guidelines for Writing Wastewater Engineering Design and Pre-Design Reports" (https://www.oregon.gov/deq/FilterRulemakingDocs/div52-designrpts.pdf). Additionally, DEQ requires the following in the predesign report:

- Rehabilitation of the collection system should be a top priority to accommodate growth over the next 20-year planning period. Please include collection system improvements in the predesign report.

If the project is a wastewater treatment project greater than $10 million, please start the value engineering when the pre-design report is submitted to DEQ for review and approval.

DEQ looks forward to working with you on this project. Please address all submittals to my attention and contact me at 503-467-9441 or julie.ulibarri@deq.oregon.gov if you have any questions.

Respectfully,

Julie Ulibarri

Julie Ulibarri
Technical Specialist

cc: CWSRF File, DEQ Shared Folder

ec: Clinton Cheney, PE, FIRM
Kenzie Billings
Jon Gasik

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EXECUTIVE SUMMARY

The City of Lowell's wastewater facilities consist of approximately 7 miles of sewer pipe, a major pump station, and one centralized wastewater treatment plant. The City's sewer utility is fiscally conservative and offers reasonable service rates to its customers. This plan was prepared for the City to efficiently implement wastewater facility improvements that are protective of human health and the environment and comply with regulatory requirements for the upcoming planning period.

The facilities are competently operated and mostly in fair condition. However, infrastructure age has caused several issues to develop in the sanitary sewage collection system and the wastewater treatment plant (WWTP). Furthermore, the City is expected to grow considerably over the next 20 years, which will necessitate facility upgrades.

Since the City's previous Wastewater Facility Plan was adopted in 2001, the number of services has increased by over 35%, land use designations in the City have changed, and regulations have become more comprehensive. Because of this, an update to the City's wastewater facility plan was necessary. This document presents several technical evaluations of the City's wastewater facilities, an analysis of alternative improvement projects to address system deficiencies, and a wastewater utility Capital Improvement Plan (CIP) for a planning period ending in 2045.

Planning Criteria

Population growth, regulatory, and land use criteria consistent with the City's Comprehensive Plan were used to guide the development of this facilities plan. The City's population is expected to grow from approximately 1,250 people in 2023 to 1,620 in the year 2045. The City's characteristic wastewater flows are expected to grow commensurate with population (Table ES-1). A significant portion (45-85%) of the City's wastewater flow in the wet season originates from inflow and infiltration (I/I) of rainwater and groundwater throughout the collection system. A summary of current wastewater flowrates, projected 2045 flowrates, and current I/I estimates are provided in Table ES-1.

Table ES-1: Current (2023) and Projected (2045) Wastewater Flowrates in Million Gallons per Day

	2023 Flows	2045 Flows	I/I Flow
Base Sewerage	0.08	0.10	0.00
Average Dry Weather Flow	0.08	0.10	0.00
Average Wet Weather Flow	0.20	0.23	0.09
Maximum Monthly Average Dry-Weather Flow	0.29	0.32	0.18
Maximum Monthly Average Wet-Weather Flow	0.40	0.43	0.29
Peak Daily Average Flow	1.4	1.5	1.2
Peak Hour Flow	2.7	2.8	2.3

The City operates its wastewater facilities through the National Pollutant Discharge Elimination System (NPDES) under wastewater discharge permit #101384. This permit was issued in June, 2010 (Appendix A). NPDES permits in Oregon are generally issued for 5-year periods; when a permit lapses and a new permit is not issued, as is the case for the City, the permit is administratively extended until a new permit can be issued. The City is expected to have a new discharge permit issued in 2027. At the time this new permit is issued, any changes to federal and state regulations that occurred since the last permit are incorporated.

This plan evaluated the existing facility's capacity to treat current and future wastewater flows and pollutant loads to comply with current and expected regulatory requirements. The existing WWTP facility layout is presented in Figure ES-1. The WWTP processes generally consist of screening, primary clarification, trickling filter/solids contact biological treatment, secondary clarification, and chlorine disinfection. Biosolids are stabilized in an aerobic digester and dewatered in conventional sand drying beds. Dried biosolids are sent to an external facility for additional treatment prior to land application. Treated wastewater effluent is discharged 20 feet upstream of the Dexter Dam penstocks in the Middle Fork Willamette River.

Need for System Improvements

Several issues were identified in the City's wastewater facilities as in need of improvement:

- The WWTP has had multiple recent exceedances of the Biological Oxygen Demand and Total Suspended Solids limits specified in the NPDES permit. The existing biological treatment system lacks the flexibility and redundancy required for the substantial seasonal flow variations experienced by the City. Frequent violations of the permit necessitate corrective actions, including upgrades to the biological treatment system.
- The Alder Street Lift Station that conveys wastewater from the west and northwest areas of the City to the WWTP is under capacity for peak flows. This has resulted in sewage overflows, causing the City to receive civil penalties. This lift station should be upgraded to increase its firm capacity and prevent future overflows.
- Multiple areas of the City's collection system were determined to have direct sources of stormwater inflow or groundwater infiltration. This results in considerable volumes of water diluting the system and disrupting treatment during wet-weather events. A comprehensive I/I evaluation identified twenty-six direct sources of stormwater inflow and eight sections of the collection system piping with groundwater infiltration issues.
- The existing aerobic digestor that stabilizes biosolids is divided into two equally sized cells. Biological modeling of the treatment system indicates that just one of these cells would provide appropriate treatment capacity for the amount of biosolids produced at projected 2045 pollutant loads. The aeration system would need to be modified to provide operators flexibility to isolate the cells, but this would result in major electricity cost savings and more optimized solids processing.
- The existing solids dewatering process is not optimized from an operations and maintenance standpoint. The drainage pipes and bottom gravel layer in the drying beds have been severely damaged by dried solids removal activities due to a lack of guide walls and entry ramps for the machinery. The current drying beds are also oversized for the needs of the City.

- The existing system was not designed to treat ammonia, which could be required within the planning period in upcoming NPDES permit renewal cycles. Some degree of nitrification currently occurs in the treatment process, evidenced by drops in pH between the influent and effluent wastewater. Compliance with pH has been achieved by operators dosing with soda ash prior to dechlorination. Adding alkalinity prior to biological treatment would be a better solution to improve ammonia removal and provide buffering capacity against drops in pH.
- The hydraulic residence time in the chlorine contact basin is insufficient. This basin is a repurposed clarifier from the original WWTP design that experiences short circuiting due to a lack of baffling. This requires the operator to continuously add chlorine at high doses to compensate, resulting in the City overspending on disinfectant and dechlorination chemicals.

Improvement Recommendations

Multiple alternatives to address the listed issues were analyzed, and approximately $5.4 million worth of improvements are recommended. These recommendations are briefly described below. The projects are grouped as either I/I reduction projects, or facility improvement projects presented in recommended phases.

Inflow and Infiltration Reduction Projects

As determined via smoke testing, flow mapping, and television surveillance of the City's collection system, nine manholes were identified as having significant infiltration issues, two potential cross-connections with the stormwater drainage system were found, and the significant damage was observed on the two pipes that feed into the Alder Street Lift Station Wet Well. Recommended projects to reduce I/I sources in the City are listed below in order of priority.

- Patching/lining of the two main collector pipes for the Alder Street Lift Station,
- CCTV the potentially cross-connected stormdrains on the corners of Moss and Lakeview and Moss and Cannon. Make any necessary patches/repairs,
- Full manhole replacements: 1st and Wetleau, 4th and Hyland, Main and Pioneer,
- Various Manhole Patching/Grouting projects (Appendix D), and
- Consistent CCTV analysis as budget allows (Appendix D).

Phase 1: Immediate Facility Improvements

Phase 1 consists of a relatively low-cost project that would make considerable improvements to WWTP operation and reduce electricity expenditures. It is recommended to complete this project as soon as possible.

- **Aerobic Digester Improvements:** The existing aerobic digester consists of two 130-thousand-gallon aerobic cells that can only be operated simultaneously with the existing aeration equipment. The City should replace the existing blowers and add isolation valving to the air-pipe system to enable isolation of the cells. At projected design loads, one cell will provide enough capacity for effective solids stabilization. This would significantly decrease electricity costs.

Phase 2: High Priority Facility Improvements

The following Phase 2 improvements are recommended to be completed before 2030. The projects recommended in this phase will increase the capacity of the Alder Street Lift Station and improve the solids management system of the WWTP. Phase 2 consists of the following recommendations:

- **Upgrade Alder Street Lift Station:** The capacity of the lift station should be upgraded to meet DEQ redundancy and reliability standards. This will necessitate both pumps to be replaced. Each pump should be sized to meet a projected peak flow of 490 gpm.
- **Drying Bed Improvements:** This involves construction of concrete guide walls and replacement of the underdrain system to divide the existing drying bed pits into three 1,500 square foot cells. Each bed should have an entrance ramp to allow for ease of entry for machinery needed for maintenance and clearing of the drying beds.

Phase 3: Wastewater Treatment Facility Improvements

The recommendations in Phase 3 involve the conversion of the existing trickling filter/solids contact system into a conventional activated sludge system, and conversion of the chlorine disinfection system to a UV system. This project will improve WWTP effluent quality, reliability, and redundancy as well as simplifying operations at the WWTP. It is recommended to complete this upgrade prior to 2035. Specific timing will depend on the City’s ability to obtain funding, since this is the most expensive of the phases at an estimated cost of approximately $3.1 million. Phase 3 consists of the following recommendations:

- **Aeration Basins:** The existing primary clarifiers would be converted into parallel aeration basins. A fine-pore diffuser aeration system, including new blowers, would be installed.
- **Construct Secondary Clarifier:** A new secondary clarifier would be constructed in the pad of the existing trickling filter. This will also require construction of new clarified-water and solids process lines for the new clarifier. The existing secondary clarifier would be maintained, and a splitter box would allow operator flexibility in the operation of either clarifier, or both in parallel. The new clarifier would be sized for typical wastewater flows, and the larger existing clarifier would provide the necessary redundancy for peak wet-weather events.
- **Supplemental Alkalinity Addition:** To facilitate nitrification in the treatment system and to ensure compliance with pH standards of the City’s NPDES permit, a chemical feed system for magnesium hydroxide should be provided upstream of the new aeration basins.
- **UV Disinfection Conversion:** The City involves constructing a UV disinfection system and decommissioning the existing chlorine disinfection system. This project would save the City in hypochlorite and thiosulfate chemical costs.

Phase 4: Collection Facility Improvements

Phase 4 will increase the capacity of the gravity collection system in a growing part of the City. This phase would ideally begin before 2035 and conclude before the end of the planning period in 2045. Phase 4 consists of the following recommendation:

- **Collection System Capacity Upgrade:** This project would involve upgrading two pipes in the collection system that are undersized for future growth. The City's main 15" gravity collector on Moss Street would be extended up to 3rd Street, and minor pipe improvements would connect the properties in the north and east portion of town to this collector. This will have an additional benefit of moving approximately 20 properties from the lift station sewershed to the gravity-only system.

Itemized cost estimates and proposed timelines for the proposed CIP are provided in the following table:

Table ES-2: Recommended Wastewater Utility Capital Improvement Plan

Capital Improvement Plan: Budgetary Costs (2024$) and Schedule			
Collection System Improvements - I/I Reduction	**Budget Cost**	**Begin and Complete By**	
Collection System - Spot Repair of Sewer Pipe Voids	$24,000	2024	2026
Collection System - Cross-Connection Repair	$168,000	2024	2028
Collection System - Manhole Rehabilitation	$87,200	2024	2030
Collection System - CCTV Surveillance	$22,400	2024	2045
I/I Reduction Budget	**$301,600**	***2024***	***2045***
PHASE 1 - Aeration System Improvements			
WWTP - Aeration System Improvements	$296,000	2024	2026
Phase 1 Budget	**$296,000**	***2024***	***2026***
PHASE 2 - Lift Station Upgrade and Biosolids Improvements			
WWTP - Biosolids Management Improvements	$342,500	2025	2030
Collection System - Alder Street Lift Station Upgrades	$376,000	2025	2030
Phase 2 Budget	**$718,500**	***2025***	***2030***
PHASE 3 - Wastewater Treatment System Upgrades			
WWTP - Activated Sludge Improvement Project	$1,376,000	2028	2032
WWTP - Secondary Clarifier Construction	$1,507,000	2028	2032
WWTP - Supplemental Alkalinity System	$176,000	2028	2033
WWTP - UV Disinfection System Installation	$564,800	2033	2040
Phase 3 Budget	**$3,623,800**	***2028***	***2040***
PHASE 4 - Collection System Capacity Upgrades			
Collection System - Gravity Sewer Improvements	$469,200	2030	2045
Phase 4 Budget	$469,200	***2030***	***2045***
Total CIP Budget	**$5,409,100**		

Capital Implementation and Funding

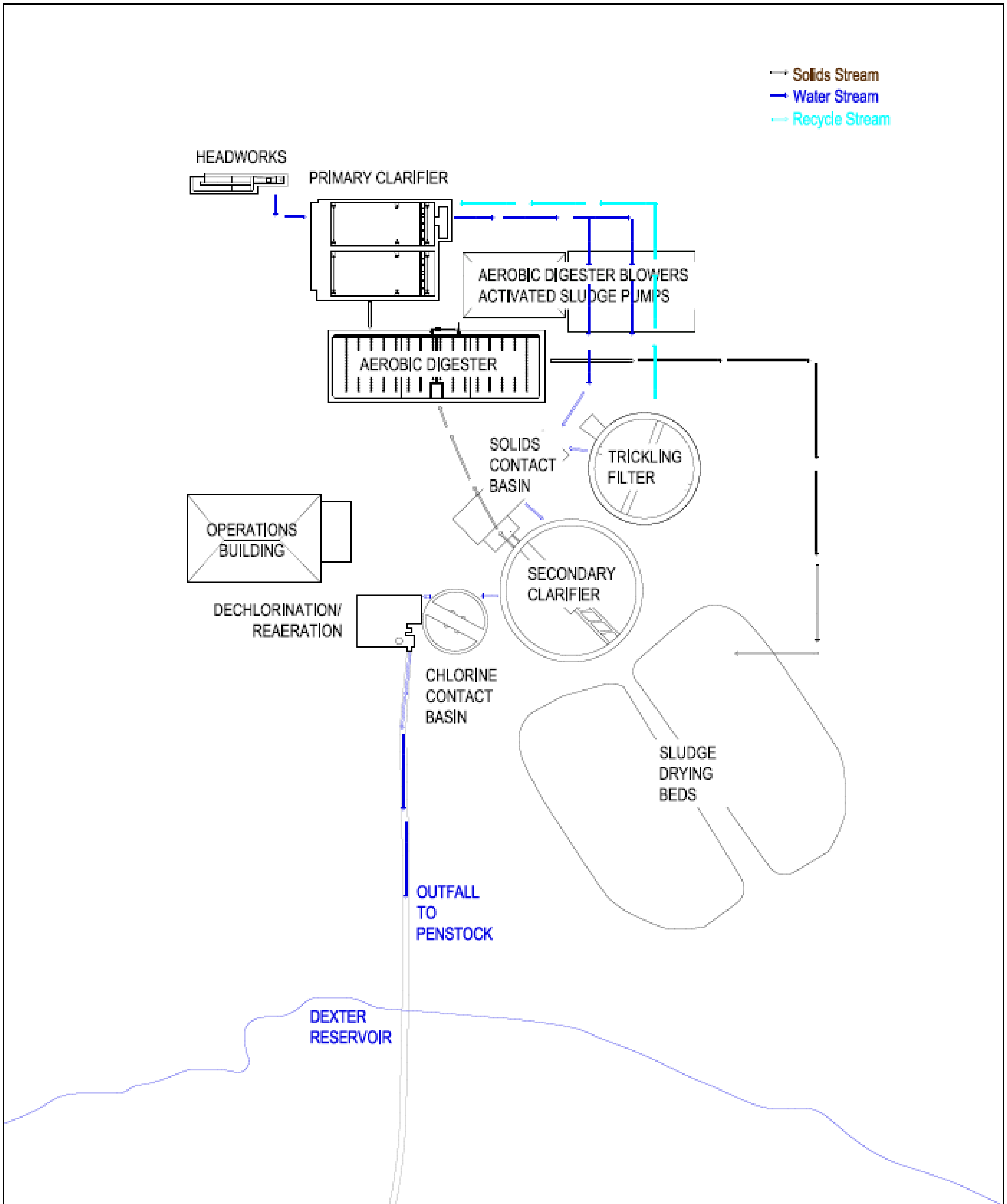
The recommended CIP will require a combination of budget funds, loans, and pursual of grant funds to complete all recommendations by the end of the planning period. A realistic goal for the City is to fund approximately $1.7 million through the City's budget, or approximately $81,000 annually (in 2024$). Most of the improvement projects listed in the CIP, aside from I/I reduction projects and optimization of the aerobic digester system, would be partially system development charge (SDC) eligible because they provide capacity for future development.

The City will likely require loans to fully fund the CIP. By pursuing grants and loans with forgivable portions, the City should aim to keep the annual debt service of the wastewater utility below $100,000 annually. Assuming a nominal 20-year loan at 3.5% interest, the City would need approximately $2 million in loans and $1.8 million in grants over the next 20 years to fully fund the CIP. These loan and grants funds are in addition to $1.7 million in funds from the City's capital improvement budget.

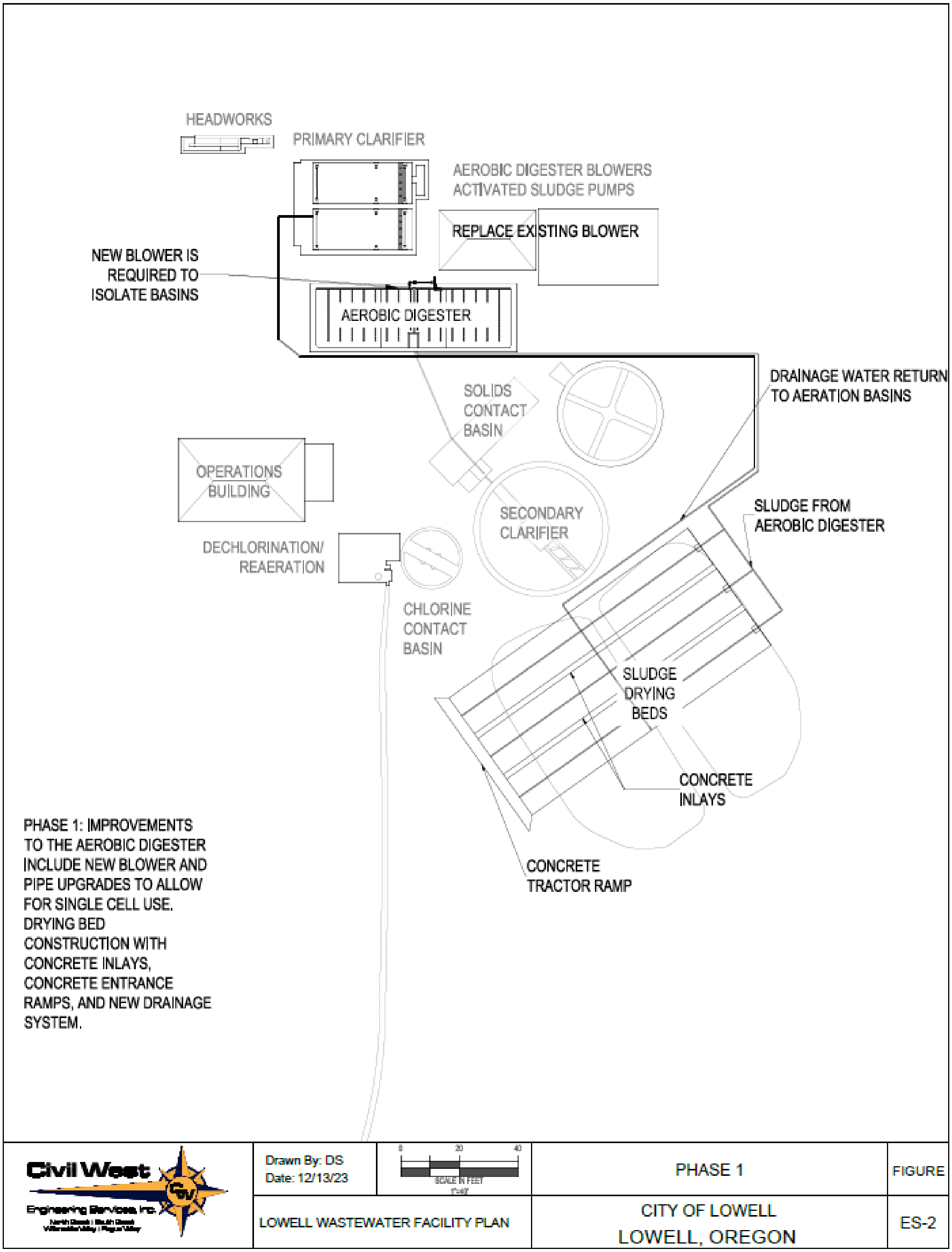
A summary of the recommended funding strategy and estimated impact on rate payers is shown in the table below.

Table ES-3: Funding Strategy and CIP Impacts on Rate Payers

Funding Strategy		
Debt Service:	$1,971,522	
Budgeted Capital Improvement Funds:	$1,659,745	
Grant/Forgivable Loan Funds:	$1,829,255	
Total Cost (2024$)	$6,177,994	
Sewer Rate Estimates		
Year	**Projected EDUs**	**Average Rate**
2024	545	$69
2025	551	$77
2030	585	$83
2040	658	$82
2045	697	$81



Civil West Engineering Services, Inc.	Drawn By: DS Date: 12/13/23	0 20 40 SCALE IN FEET 1"=40'	EXISTING WWTP	FIGURE
	LOWELL WASTEWATER FACILITY PLAN		CITY OF LOWELL LOWELL, OREGON	ES-1


HEADWORKS
PRIMARY CLARIFIER
AEROBIC DIGESTER BLOWERS
ACTIVATED SLUDGE PUMPS
REPLACE EXISTING BLOWER
NEW BLOWER IS REQUIRED TO ISOLATE BASINS
AEROBIC DIGESTER
DRAINAGE WATER RETURN TO AERATION BASINS
SOLIDS CONTACT BASIN
OPERATIONS BUILDING
SECONDARY CLARIFIER
SLUDGE FROM AEROBIC DIGESTER
DECHLORINATION/ REAERATION
CHLORINE CONTACT BASIN
SLUDGE DRYING BEDS
CONCRETE INLAYS
CONCRETE TRACTOR RAMP
PHASE 1: IMPROVEMENTS TO THE AEROBIC DIGESTER INCLUDE NEW BLOWER AND PIPE UPGRADES TO ALLOW FOR SINGLE CELL USE. DRYING BED CONSTRUCTION WITH CONCRETE INLAYS, CONCRETE ENTRANCE RAMPS, AND NEW DRAINAGE SYSTEM.
Civil West Engineering Services, Inc.
Drawn By: DS
Date: 12/13/23
0 20 40
SCALE IN FEET
PHASE 1
FIGURE
LOWELL WASTEWATER FACILITY PLAN
CITY OF LOWELL
LOWELL, OREGON
ES-2

1 PLANNING AREA

This section provides a detailed description of the location, environmental resources, and population trends in the City of Lowell. The provision of sewer collection and wastewater treatment services by the City is consistent with the Oregon Department of Land Conservation and Development (DLCD) land use goals and the City's local comprehensive plan. The environmental and socio-economic information provided in this section should be considered in evaluations for planning, design, and operation of the City's wastewater facilities.

1.1 Location

The City is located on the east side of the Southern Willamette Valley in Lane County on the hilly transitional terrain between the Willamette Valley and the Western Cascade Mountains. There are two prominent water features near the City: the Middle Fork Willamette River and Dexter Lake. A vicinity map is provided in Figure 1-1.

As described in the City's Comprehensive Plan (March 2023), the City is approximately 17 miles southeast of Springfield and 22 miles southeast of Eugene. The primary access route to Lowell is Oregon State Highway 58. This highway provides access to the City from a bridge and causeway across Dexter Lake. Two county roads, Jasper-Lowell Road and Pengra Road, provide access to Springfield on the east side of the Middle Fork Willamette River.

1.1.1 History

The area of Lowell was originally settled in 1852 and named Cannon at the time. The town was renamed in 1882 in response to the postal service's confusion with Cannon City, Oregon. The City of Lowell was officially incorporated in 1954. Lowell was primarily a timber town until the late 1980s. Early industries in the area were hop raising, stock raising, and logging. The first population boom occurred with the construction of Lookout Point Reservoir by the U.S. Army Corps of Engineers (ACE) in 1948. Much of the town was relocated when the dam was built. In recent years, Lowell's primary employers have been the U.S. Forest Service, ACE, and the Lowell School District. Because of the City's close proximity to the Eugene-Springfield urban area, it is less than a 30-minute commute to jobs in Eugene and Springfield. To a large extent, Lowell has become primarily a residential community.

1.1.2 Service Area

The City provides utility services, including water and wastewater, to over 1,000 year-round residents. The wastewater service area is limited to the City's Urban Growth Boundary (UGB). The UGB covers an area of approximately 762 acres (1.19 square miles), of which about 290 acres are undeveloped and about 200 acres includes Dexter Lake. The UGB extends from Dexter Lake to just north of Seneca Street from South to North, and from Lowell State Park to Orchard Park from West to East.

1.1.3 Topography

The topography of the service area ranges from relatively flat for most of the town to steeper slopes and hills to the north and west of the City. According to the City's comprehensive plan, Lowell is 741 feet above sea level. Elevations around the community range from 695 feet at the full pool elevation of Dexter Lake to 2,141 feet at the summit of Disappointment Butte, immediately northeast of Lowell. The developed area of Lowell occupies portions of a small plateau 45 feet above the lake. A topographical map of the City is provided in Figure 1-2.

1.1.4 Zoning and Land Use

Land use within the City is mostly residential, with some light commercial properties. The City's Comprehensive Plan defines land use within the City's UGB. The land use definitions and most recently available zoning map are discussed below. There are no land use issues that affect the existing wastewater treatment plant facility.

Most of the City's zoning consists of single-family residential homes. In 2022, the City of Lowell completed an update to its development code resulting in the new zoning districts being added to the City's zoning criteria. The current zoning types are listed as follows:

- Single-Family Residential
- Multi-Family Residential
- Commercial
- Light Industrial
- Public Land
- *Downtown Flex-Use 1*
- *Downtown Flex-Use 2*
- *Downtown Residential Attached*
- *Downtown Residential Detached*
- *Public Lands Downtown*

The zoning types listed above in italics were added to replace the now defunct "downtown commercial" zoning type in the 2022 update to the development code. The most recently available version of the City's zoning map (2012) does not reflect these changes. However, these reclassifications do not significantly affect the scope of this wastewater planning document, since the vast majority of existing and future wastewater flow and pollutant loads are from residential uses from the single-family residential zones.

Existing land use conditions were estimated from aerial photography and from information within the City's comprehensive plan. For simplification, single-family and multi-family residential zonings were combined into one residential classification since less than 5% of the residential zones are multi-family, and that is unlikely to change within this document's planning period. The commercial and downtown zoning criteria were also combined as one commercial zoning. A breakdown of developed and buildable area per zoning type, along with existing equivalent dwelling unit (EDU) estimates referenced from the City's Water Master Plan, is provided in Table 1-1. A copy of the most recent zoning map from 2023 is provided in Figure 1-3.

Table 1-1: Estimates of Developed Area and Buildable Land per Zoning Type

	Developed Area (acres)	EDU Estimate	Buildable Area (acres)
Residential	126	536	66
Commercial	8.25	4	1.59
Industrial	2.07	2	5.35
Public	35.7	4	0.71
Total	**172**	**545**	**74**

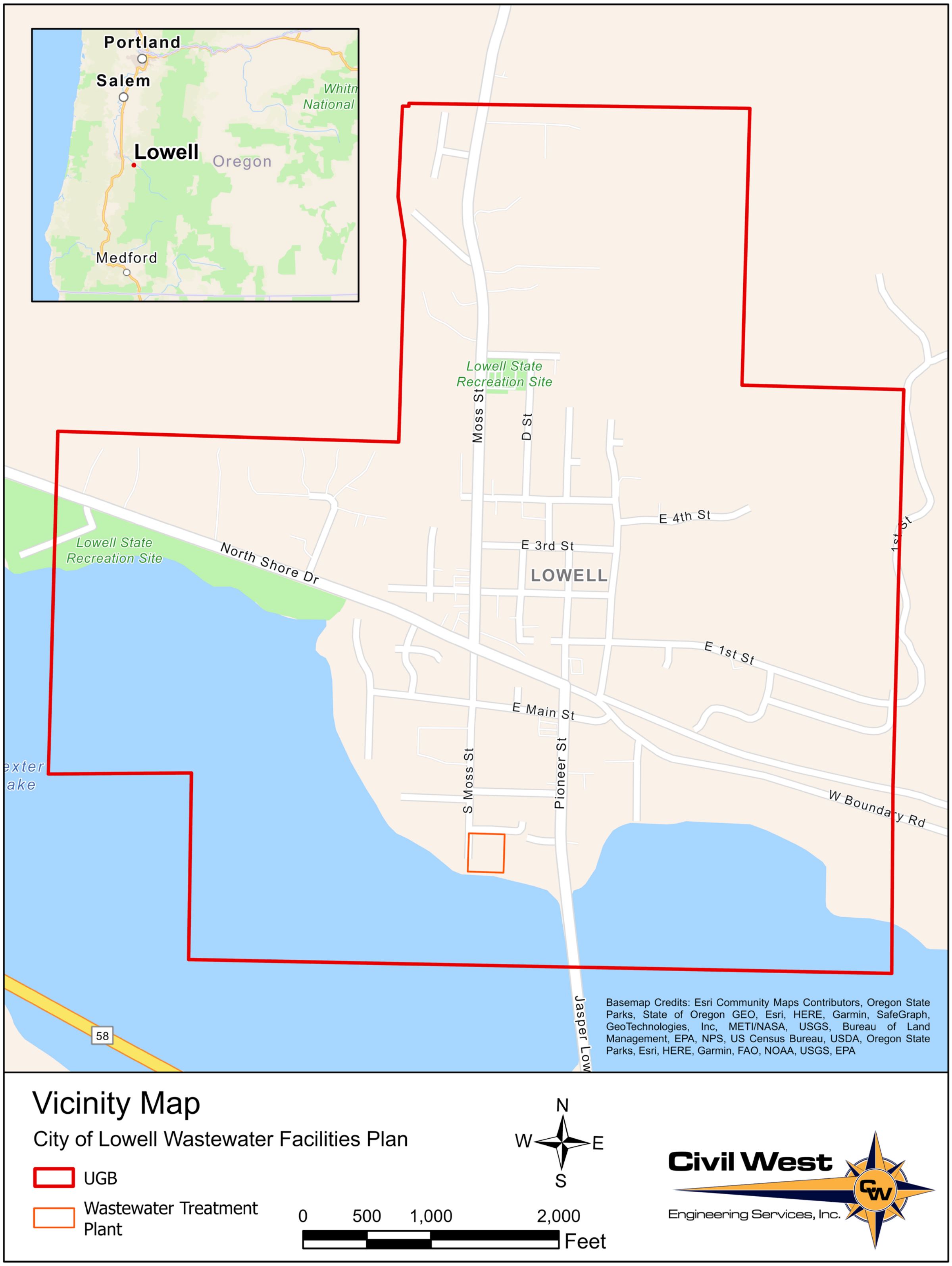


Figure 1-1: Vicinity Map of the City of Lowell

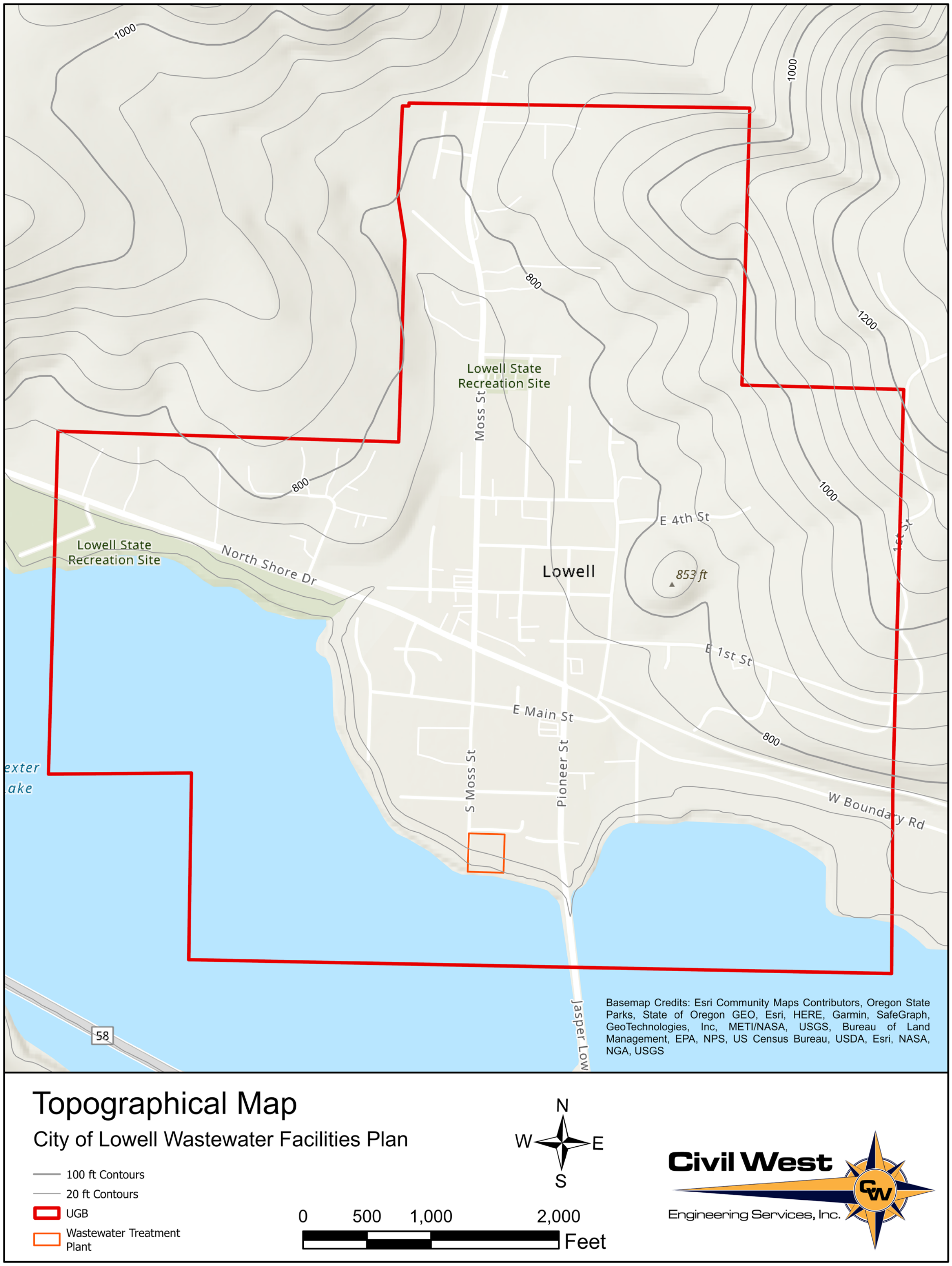


Figure 1-2: Topographical Map of the City of Lowell

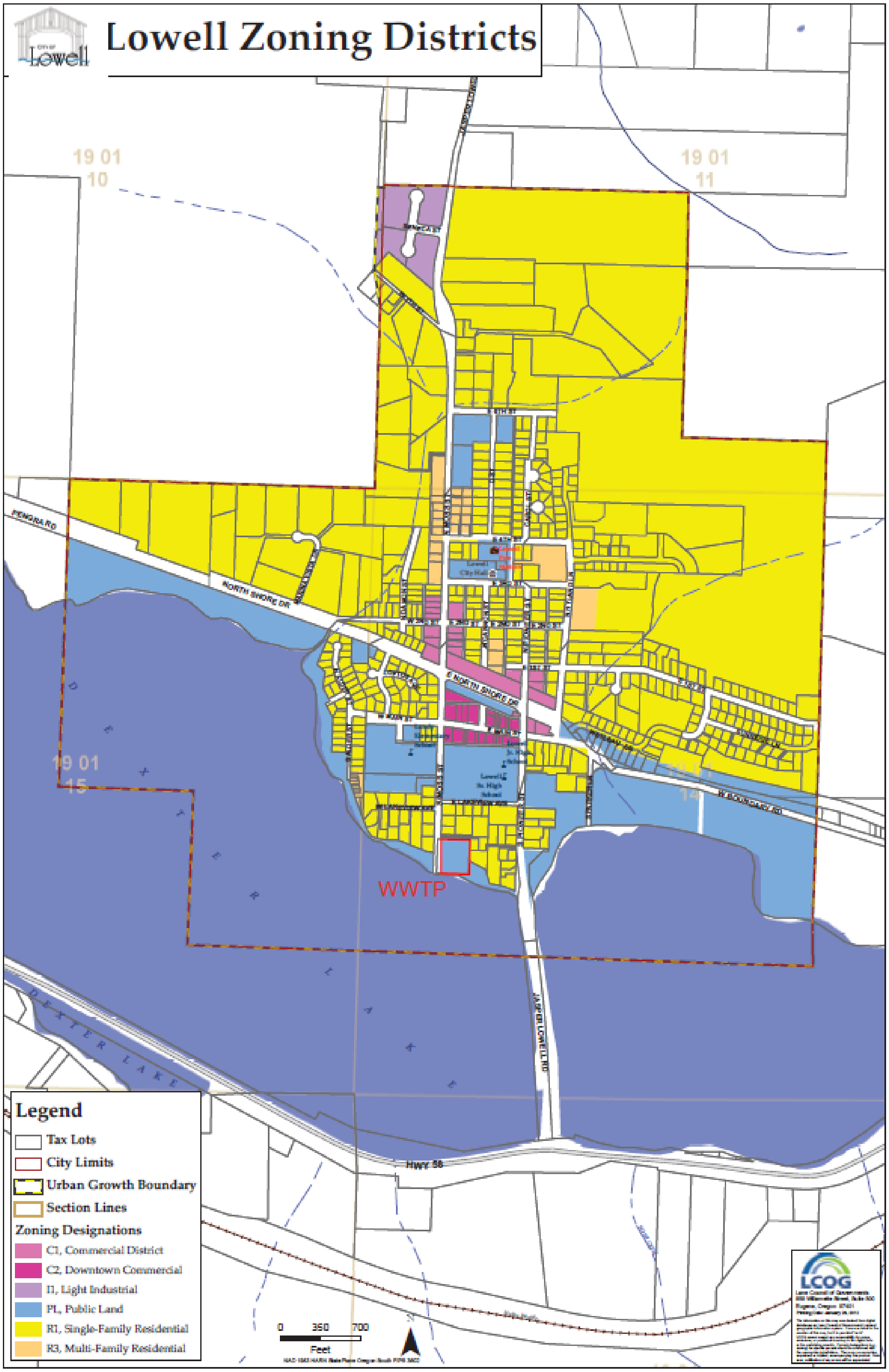


Figure 1-3: Zoning Districts in the City of Lowell

1.2 Environmental Resources

1.2.1 Water Bodies

The largest body of water near the City is Dexter Reservoir. An unnamed creek runs generally north to south toward Dexter Reservoir along the west side of the City near Moss Street. The creek confluences with a second creek that runs east to west north of East 6th Street. The City obtains its potable water from Dexter Reservoir on the east end of the City. The wastewater treatment plant (WWTP) is located on the west end of the reservoir (240 S Moss Street); the WWTP discharges treated wastewater 20 feet upstream of the Dexter Dam penstock.

1.2.2 Flora and Fauna

Biological resources in the area include numerous fishes, birds, insects, and plants. The U.S Fish and Wildlife Service Information for Planning and Conservation tool was used to identify species listed as endangered, threatened, or candidate and migratory birds that could potentially be affected by activities in Lowell. There were 6 listed species and 10 migratory birds determined to have habitats or migratory paths within the area. Table 1-2 presents the listed species in the planning area; Table 1-3 shows the migratory birds and their approximate breeding seasons.

Table 1-2: Endangered, Threatened, and Candidate Species with Habitats near the City of Lowell

Common Name	Scientific Name	Status
Birds		
Northern Spotted Owl	*Strix occidentalis caurina*	Threatened
Fish		
Bull Trout	*Salvelinus confluentus*	Threatened
Insects		
Fender's Blue Butterfly	*Icaricia icarioides fenderi*	Threatened
Monarch Butterfly	*Danaus plexippus*	Candidate
Flowering Plants		
Kincaid's Lupine	*Lupinus sulphureus ssp. kincaidii*	Threatened
Willamette Daisy	*Erigeron decumbens*	Endangered

Table 1-3: Birds with Migratory Paths near the City of Lowell

Common Name	Scientific Name	Breeding Season
Migratory Birds		
Bald Eagle (Non-BCC Vulnerable)	*Haliaeetus leucocephalus*	Jan 1 - Sep 30
Black Swift	*Cypseloides niger*	Jun 15 - Sep 30
California Gull	*Larus californicus*	Mar 1 - Jul 31
Clark's Grebe	*Aechmophorus clarkii*	Jun 1 - Aug 31
Evening Grossbeak	*Coccothraustes vespertinus*	May 15 - Aug 10
Golden Eagle (Non-BCC Vulnerable)	*Aquila chrysaetos*	Jan 1 - Aug 31
Olive-sided Flycatcher	*Contopus cooperi*	May 20 - Aug 31
Rofous Hummingbird	*selasphorus rufus*	Apr 15 - Jul 15
Western Grebe	*aechmophorus occidentalis*	Apr 15 - Jul 15
Wrentit	*Chamaea fasciata*	March 15 - Aug 10

1.2.3 Climate

Climate data was obtained from the Lookout Point Dam Weather Station located on Lookout Point Dam approximately one mile east of the City. According to the data gathered from National Oceanic Atmospheric Administration (NOAA) between 1999 and 2023, the maximum average monthly temperature of 81°F occurs in the months July and August whereas the minimum average monthly temperature of 37°F occurs in December. As shown in Figure 1-4, temperatures cycle annually with higher temperatures in the summer and lower temperatures in the winter. An annual average total precipitation of 42.3 inches was reported between the period of 1999 and 2023. As shown in Figure 1-5, December historically has the highest average precipitation (7.1 inches), and July has the lowest average precipitation (0.3 inches).

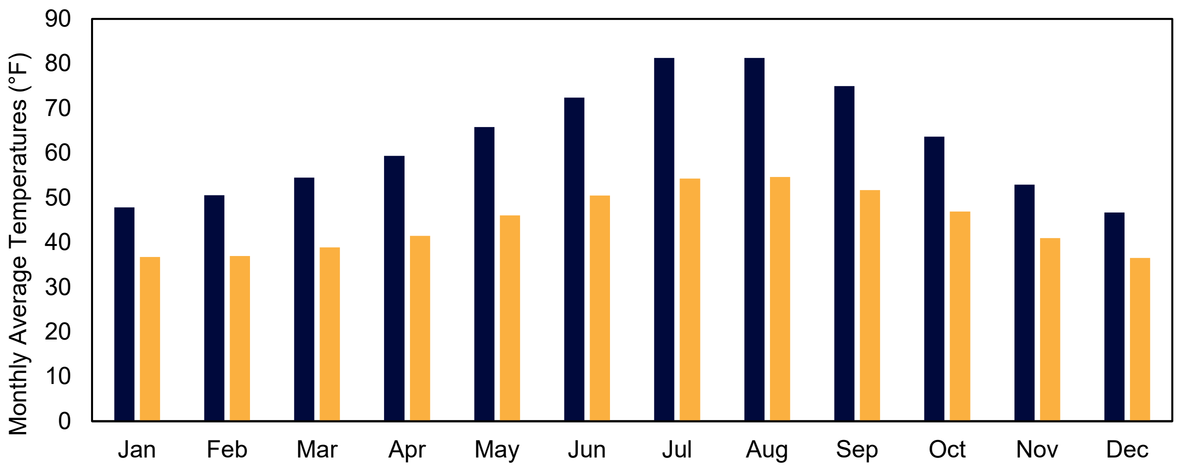


Figure 1-4: Monthly Average High (Dark Blue) and Low (Gold) Temperatures in the City of Lowell.

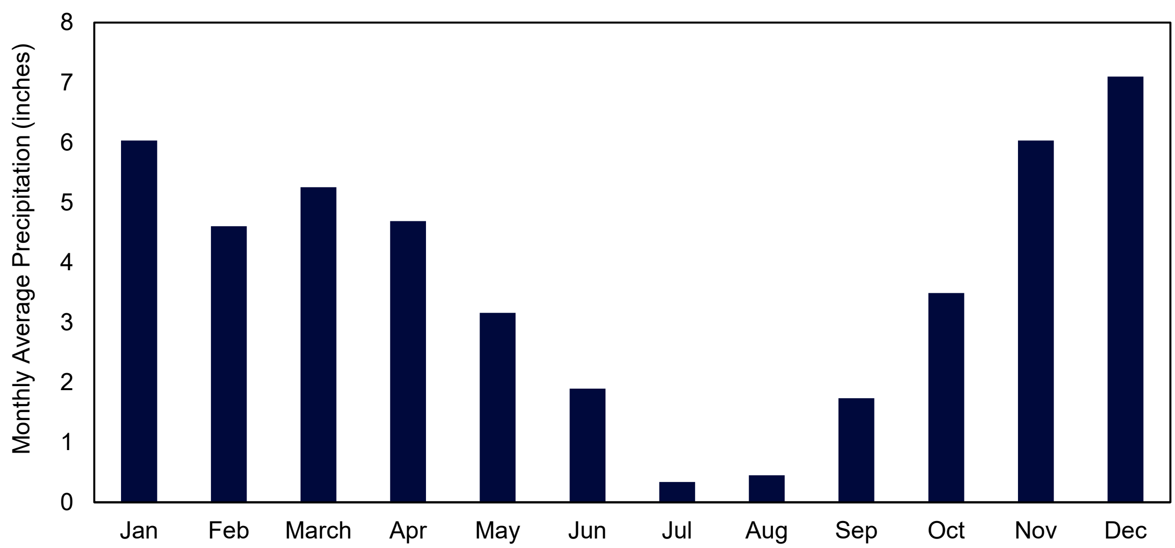


Figure 1-5: Monthly Average Cumulative Precipitation in the City of Lowell

1.2.4 Floodplains

The Federal Emergency Management Agency (FEMA) provides maps of flood zones for areas across the United States called "Flood Insurance Rate Maps" (FIRM). The FIRM detailing flood zones within and near the City of Lowell was acquired from FEMA's Flood Map Service Center Website and is provided in Appendix B. A 100-year flood zone was identified on this FIRM directly bordering Dexter Lake. No base flood elevation is defined for this flood zone.

1.2.5 Wetlands

The U.S. Fish and Wildlife Service manages the National Wetlands Inventory (NWI) for wetlands and other aquatic habitats that may be subject to regulation under Section 404 of the Clean Water Act or other State/Federal statutes. Future projects must take wetland and aquatic habitat impacts into consideration and avoid disruptions when possible. Figure 1-6 and Figure 1-7 show the location and boundaries of the wetlands near the City of Lowell based on National and Statewide wetland inventories respectively. Based on these maps, Dexter Lake is the only wetland in close proximity to the City's wastewater treatment facility. The hydric soils from the statewide inventory are primarily along Moss Street and the Lowell School District.

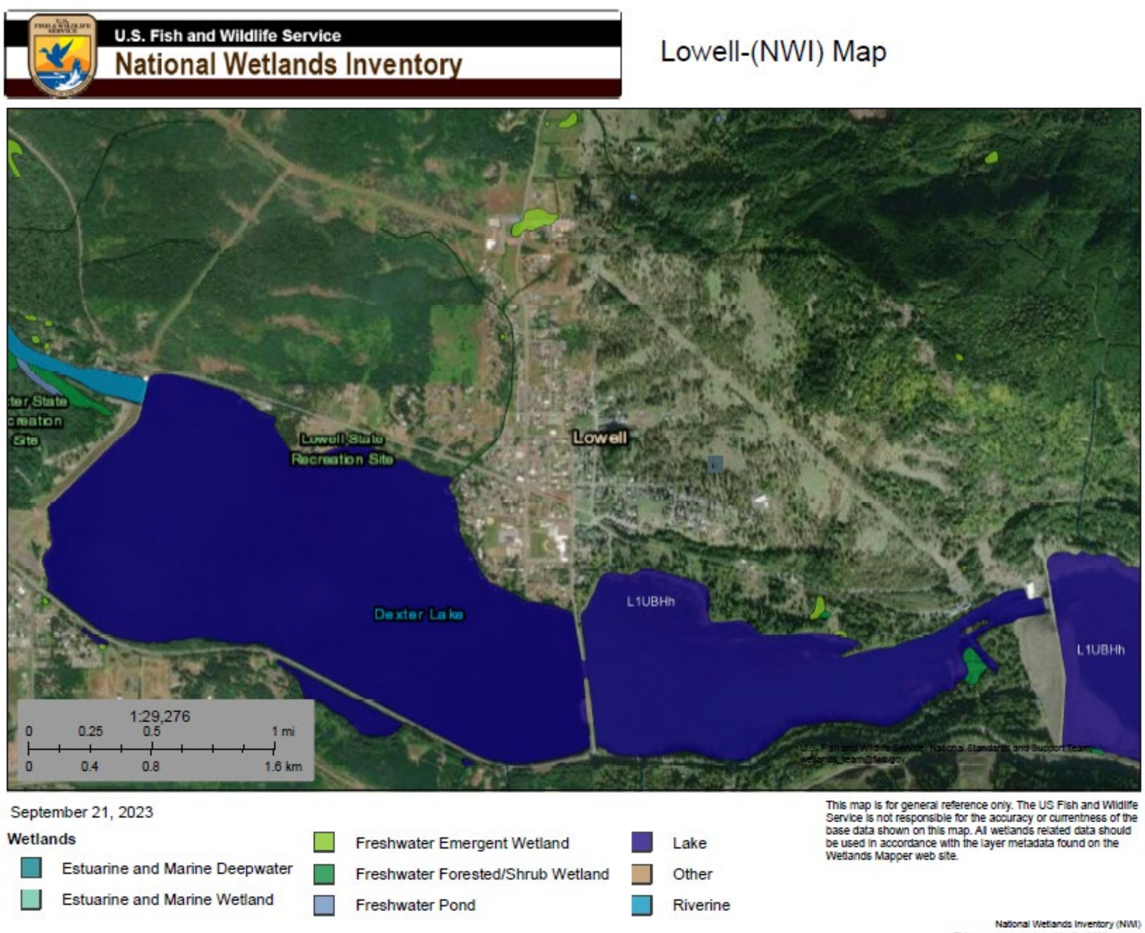


Figure 1-6: National Wetland Inventory for the City of Lowell

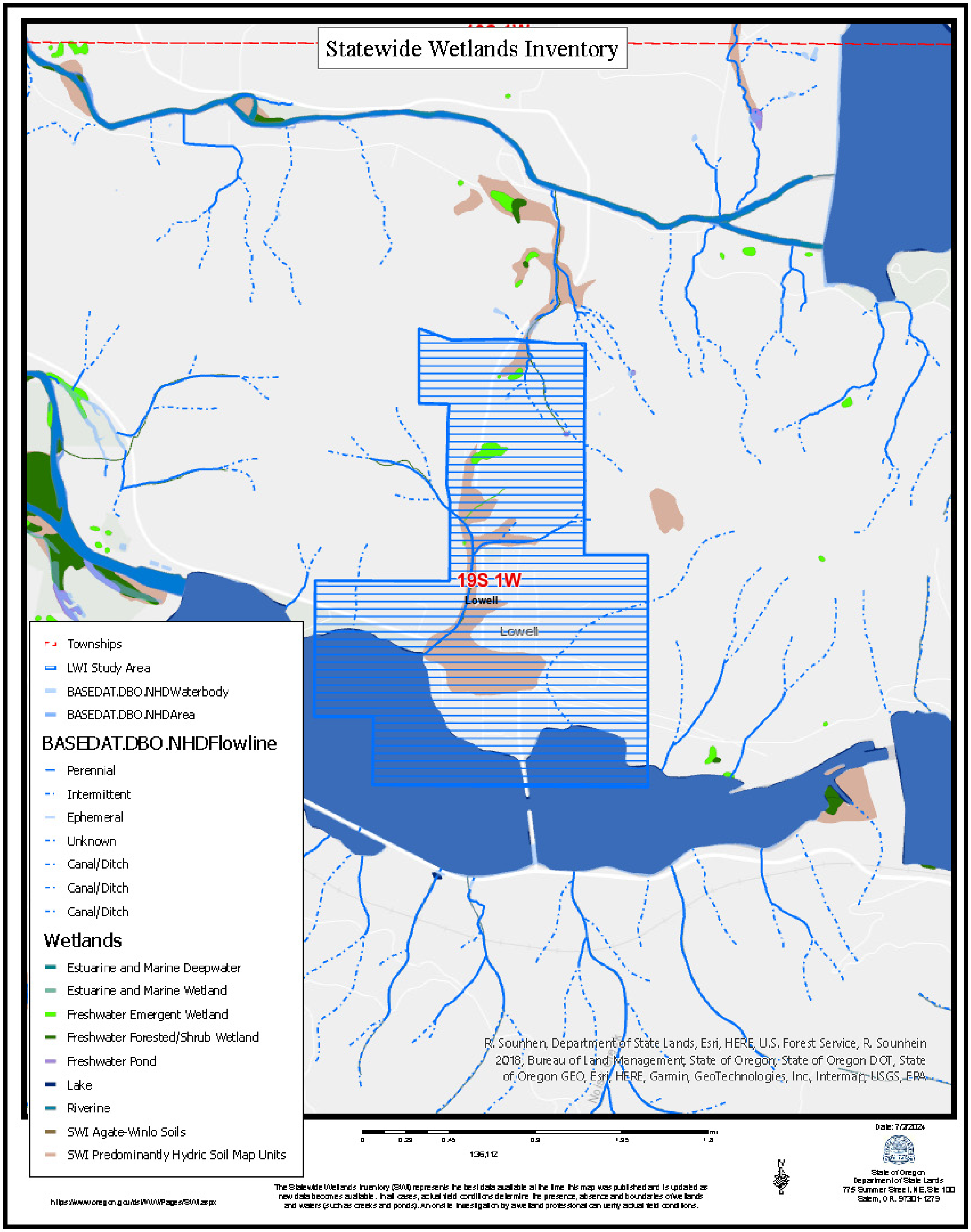


Figure 1-7: Oregon Statewide Wetlands Inventory

1.2.6 Soils

Soil data was obtained from the United States Department of Agriculture Natural Resources Conservation Service Soil Survey Web Mapper tool. A report was generated for the City's service area and is provided in Appendix C. The predominant soils in the area are Dixonville-Philomath-Hazelair complex and Hazelair silty clay loams. A summary map of soil types in the area is provided in Figure 1-8. A summary table of soil types is provided in Table 1-4.

1.2.7 Geological Hazards

Seismic hazard risks near and within the City were evaluated using the Oregon HazVu Statewide Geohazards Viewer maintained by DOGAMI. The majority of the City is classified with a “very strong” shaking hazard level. The greatest seismic risk to the region comes from the Cascadia Subduction Zone along the Pacific Coast due to the possibility of a massive earthquake. The nearest active fault is located about six miles southeast of the City at the highest point of Lookout Point Lake.

1.2.7.1 Landslides

A variety of events such as earthquakes and precipitation can cause landslides to occur. Landslide risks in the City were evaluated using the Oregon HazVu Statewide Geohazards Viewer maintained by DOGAMI. As shown in Figure 1-9, areas of moderate and high suscepitibility to landslides in the City are along the border of Dexter Lake. The Wastewater Treatment Plant is within this area.

1.2.7.2 Soil Liquifaction

Soil liquefaction, an event in which soil destabilizes and behaves more like a liquid than a solid, can be caused by strong seismic activity and can destabilize structures. Soil liquefaction risk was evaluated using the Oregon HazVu Statewide Geohazards Viewer maintained by DOGAMI. The majority of the City is at moderate risk of liquefaction.

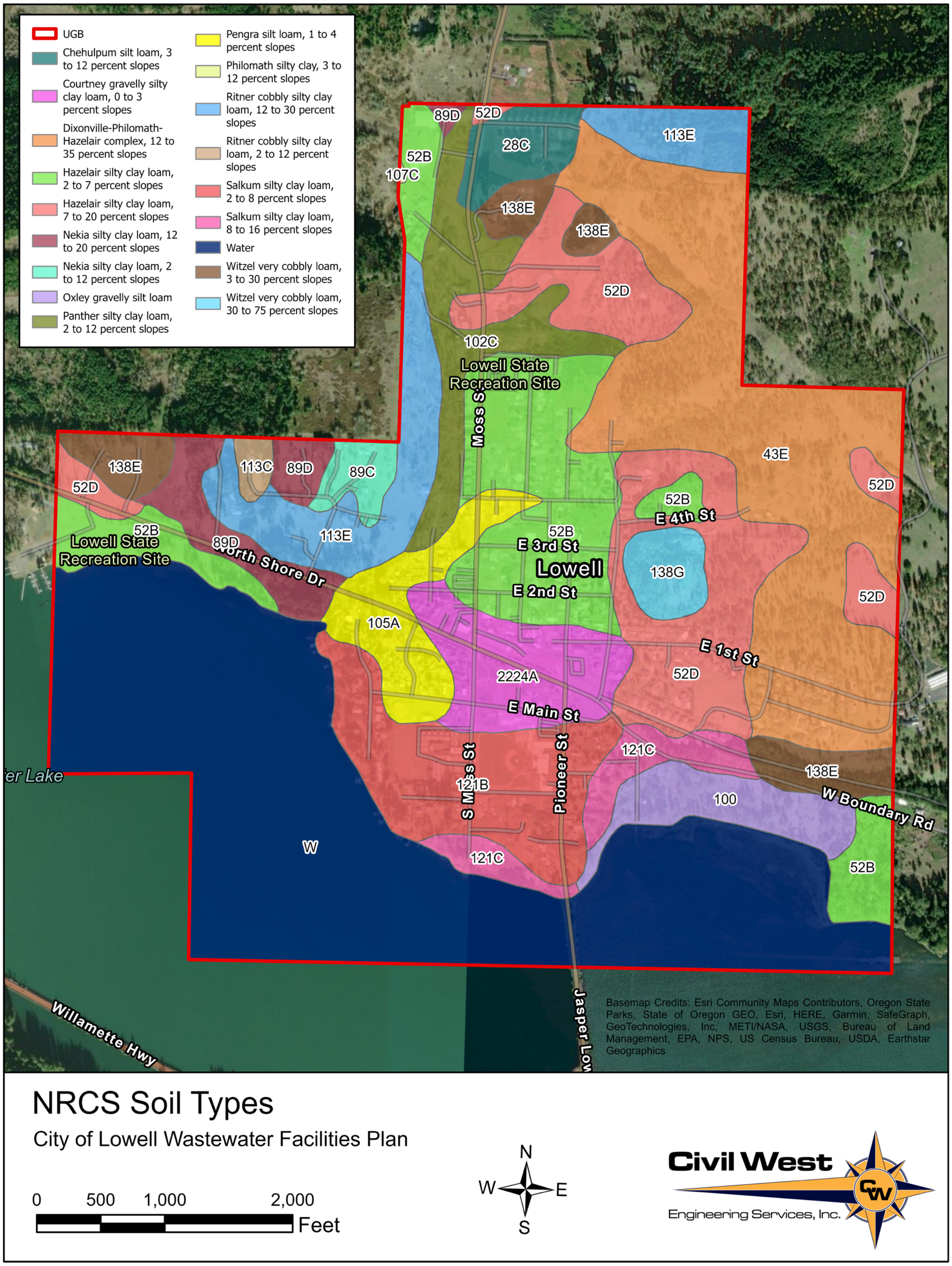


Figure 1-8: Soil Types within the City of Lowell

Table 1-4: Soil Types in the City of Lowell

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
28C	Chehulpum silt loam, 3 to 12 percent slopes	11.7	1.5%
43E	Dixonville-Philomath-Hazelair complex, 12 to 35 percent slopes	119.5	15.7%
52B	Hazelair silty clay loam, 2 to 7 percent slopes	82	10.8%
52D	Hazelair silty clay loam, 7 to 20 percent slopes	76.9	10.1%
89C	Nekia silty clay loam, 2 to 12 percent slopes	6.6	0.9%
89D	Nekia silty clay loam, 12 to 20 percent slopes	19.7	2.6%
100	Oxley gravelly silt loam	18.5	2.4%
102C	Panther silty clay loam, 2 to 12 percent slopes	29.5	3.9%
105A	Pengra silt loam, 1 to 4 percent slopes	22.9	3.0%
107C	Philomath silty clay, 3 to 12 percent slopes	0.2	0.0%
113C	Ritner cobbly silty clay loam, 2 to 12 percent slopes	2.9	0.4%
113E	Ritner cobbly silty clay loam, 12 to 30 percent slopes	41.1	5.4%
121B	Salkum silty clay loam, 2 to 8 percent slopes	46.6	6.1%
121C	Salkum silty clay loam, 8 to 16 percent slopes	15.9	2.1%
138E	Witzel very cobbly loam, 3 to 30 percent slopes	24	3.2%
138G	Witzel very cobbly loam, 30 to 75 percent slopes	9.1	1.2%
2224A	Courtney gravelly silty clay loam, 0 to 3 percent slopes	28.8	3.8%
W	Water	204.2	26.9%
	Totals	**760.1**	**100%**

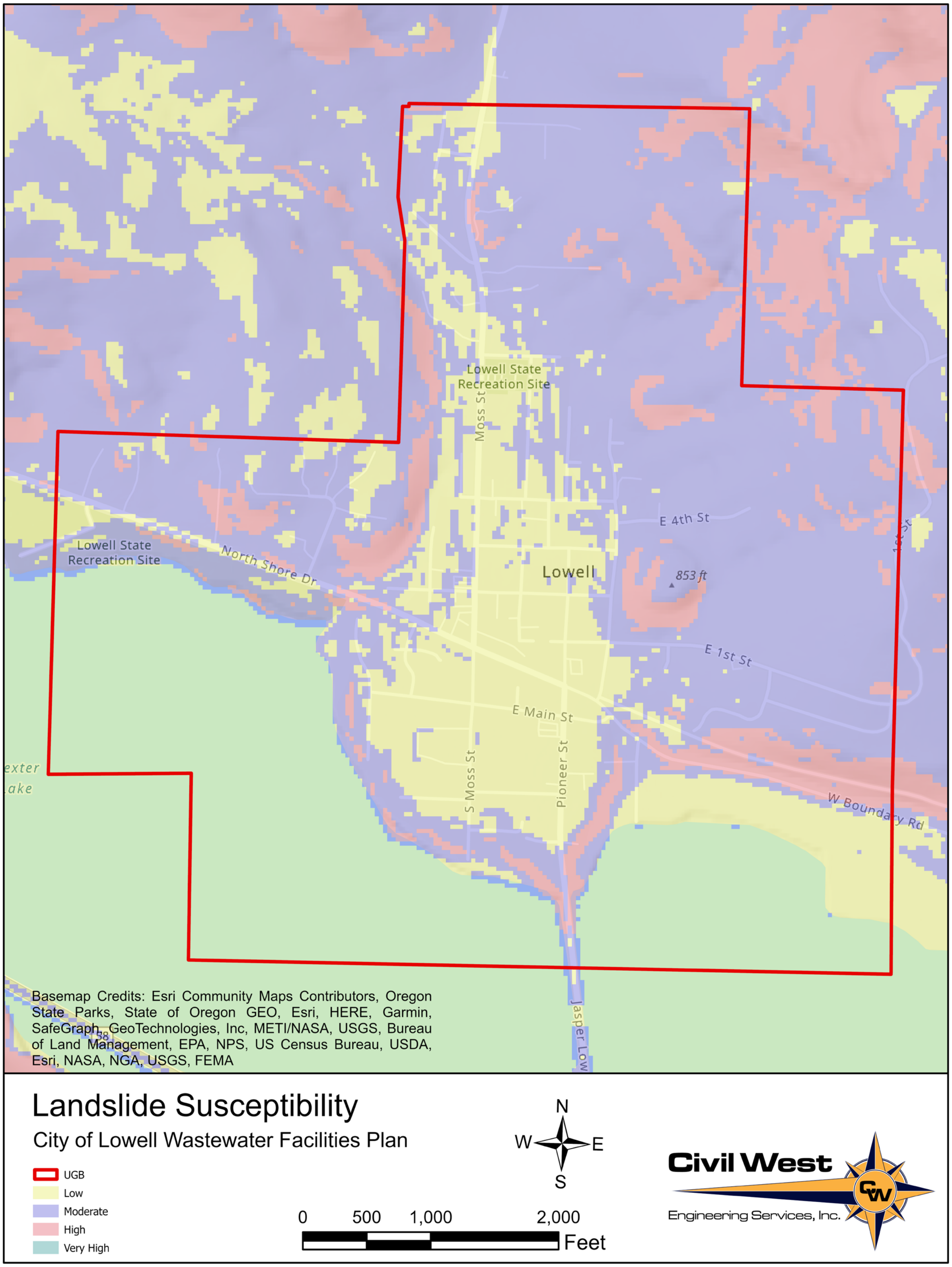


Figure 1-9: Landslide Susceptibility in the City of Lowell

1.3 Socio-Economic Resources

1.3.1 Population and Projections

Reported U.S. Census populations for the City of Lowell dating back to 1990 are presented in Table 1-5.

Table 1-5: Reported US Census Populations for the City of Lowell (1990 to 2020)

Census Year	Population
1990	785
2000	880
2010	1,045
2020	1,196

Population projections for the planning period were made using information from the Portland State University Population Research Center (PRC). The July 1, 2022 PRC population estimate was 1235 (Table 4: Populations for Oregon and Its Counties and Incorporated Cities and Towns, April 2023). The projected average annual growth rate (AAGR) for Lowell was estimated by the PRC as 1.2% for the period between 2020 to 2045 (Coordinated Population Forecast 2021 through 2071, Lane County Urban Growth Boundaries & Area Outside UGBs," June 30, 2021, Table 2). Based on the expected AAGR and 2022 certified population estimate, the population is estimated to be 1,618 people in 2045. Yearly population projections for Lowell are provided in Table 1-6.

Table 1-6: Population Projections from 2022 to 2045

Year	Population	Projected Population Increase
2022	1,235	
2023	1,250	15
2024	1,264	15
2025	1,279	15
2026	1,294	15
2027	1,310	15
2028	1,325	15
2029	1,341	16
2030	1,357	16
2031	1,373	16
2032	1,389	16
2033	1,405	16
2034	1,422	17
2035	1,439	17
2036	1,456	17
2037	1,473	17
2038	1,490	17
2039	1,508	18
2040	1,526	18
2041	1,544	18
2042	1,562	18
2043	1,580	18
2044	1,599	19
2045	**1,618**	**19**
Buildout	4,145	2,527

1.3.2 Cultural Resources

According to the National Register of Historic Places, there are two historic properties located in or near the City. These are listed as:

- Lowell Bridge - near Highway 58 on Dexter Lake
- Lowell Grange - 51 E 2nd St.

Potential impacts to cultural resources should be considered during planning for wastewater collection and treatment system improvements. The Legislative Commission on Indian Services was contacted to evaluate what Tribes may be potentially impacted by improvement projects in the planning area. The following Tribes were identified:

- Confederated Tribes of Coos, Lower Umpqua and Siuslaw Indians
- Confederated Tribes of Grand Ronde
- Confederated Tribes of Siletz Indians
- Confederated Tribes of Warm Springs Reservation of Oregon
- Coquille Indian Tribe
- Cow Creek Band of the Umpqua Tribe of Indians

Civil West reached out to each of the Tribes listed above to inquire about potential cultural sites in the area. The Tribes did not share this information due to confidentiality concerns. The City should plan to compile the following information to include in plan review submittals to the Tribes during design phase for future improvement projects: lead agency and their staff member with responsibility for oversight or permitting; funding source; and vertical and horizontal extent of any proposed ground disturbance, including spoils and staging areas.

1.3.3 Equivalent Dwelling Units

An Equivalent Dwelling Unit (EDU) is used in water and wastewater master planning to show typical monthly residential usage per connection. One EDU represents the average sewer use for a single-family residence or “equivalent dwelling”.

Based on water sales records from January 2016 to December of 2020, the average quantity of water sold to a dwelling on a standard residential meter is 4,716 gallons per month. This volume sold per month was the basis for Equivalent Dwelling Unit (EDU) calculations in the City’s Water Master Plan, with 1 EDU using 4,716 gallons of water per month based on metered sales.

Other users can then be described as an equivalent number of EDUs based on their relative water consumption. For example, a commercial business that had an average metered consumption of 9,432 gallons per month uses twice the amount of water as the typical single-family dwelling and can be considered 2 EDUs. Total water sold for the same period indicates the total number of system EDUs in the City is 545. A breakdown of EDU types (commercial, residential and industrial) was provided in Table 1-1.

1.3.4 Socioeconomic Conditions and Trends

According to the 2017-2021 American Community Survey (ACS) narrative profile, families make up most households in Lowell (65.71%) with an average household size of 2.51 people. An estimated 96.4% of the City’s population are born in the United States. More than 70% of the residents aged 25 and older had obtained a high school diploma or some community college or associate degree while slightly more than 18% had also obtained a bachelor's degree (or higher). Figure 1-10 presents the household income distribution for the City. The median household income is $52,431. Approximately 8.4% of the population lives in poverty.

The ACS reported that 417 of 442 housing units in the City were recorded as occupied, with approximately an 80% ownership rate. The biggest portion of housing units is comprised of single-family houses. The distribution of housing unit types is presented in Figure 1-11.

1.4 Community Engagement

This plan was generated with extensive engagement of the City's public works team. Specific activities included regular meetings, presentation of preliminary results (i.e., smoke testing, flow testing, flow analysis) and discussion of the results with the City, and regular site visits to observe operations.

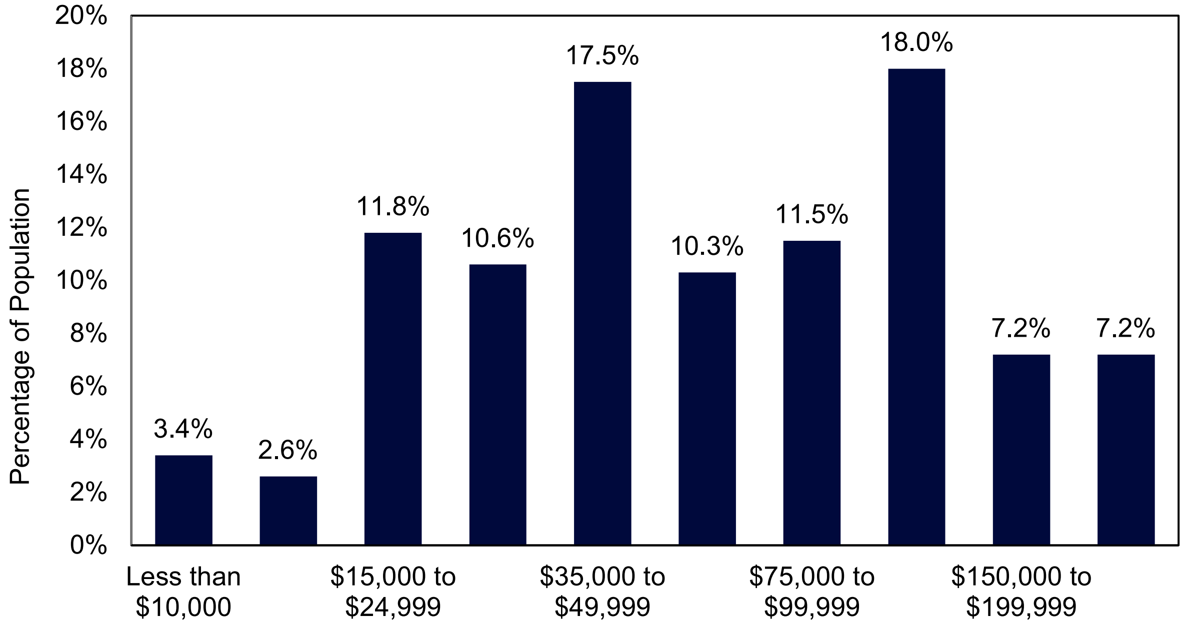


Figure 1-10: Income Distribution of the City's Population

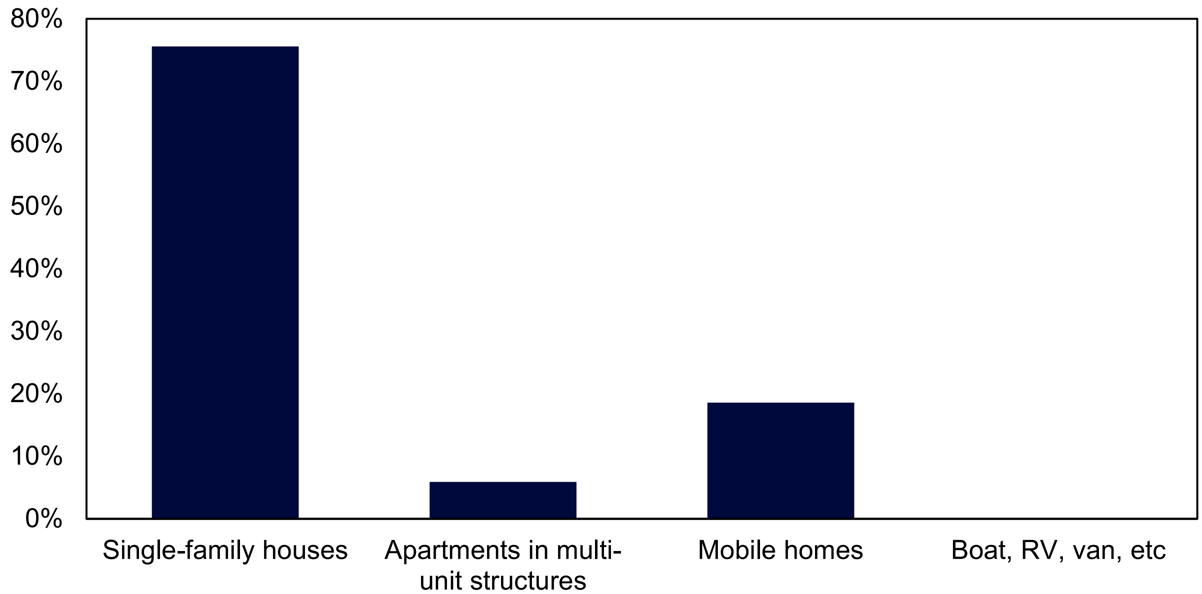


Figure 1-11: Housing Types in the City of Lowell

2 EXISTING FACILITIES

This section provides descriptions of each component of the City's existing wastewater collection and treatment facilities. Also included in this section is a description of the City's financial status with respect to the wastewater system, and an evaluation of the facilities.

2.1 Sanitary Sewer Collection System

2.1.1 Gravity Sewer

Sanitary sewer collection services are available to the population that live within the City's UGB. All wastewater collected within this area is conveyed to the WWTP for treatment and final discharge. The gravity sewer system consists of approximately 15,000 feet of 8- and 10-inch concrete pipe, 23,000 feet of 8-inch, 10-inch, and 12-inch polyvinyl chloride (PVC) pipe, and approximately 100 precast concrete manholes. The majority of the concrete pipe in the collection system was originally installed in 1950 by the U.S. Army Corps of Engineers. The system has since expanded with development and projects to replace pipe and manholes experiencing inflow and infiltration (I/I) issues. Despite the work done, the system still experiences substantial issues with I/I. An evaluation of areas in the collection system in need of immediate repair as determined by a technical I/I analysis is provided Appendix D. A comprehensive map of the collection system is provided in Figure 2-3.

2.1.2 Pressure Sewer

The Alder Street Pump Station was constructed along with the original collection system in 1950 and serves most of the properties in the City west of Moss Street, except for a few in the furthest northwest portion of the City. The pump station has essentially the same configuration today as the original construction with two pumps in a duplex submersible configuration. The pumps from the original facility were upsized around 2004.

The pumps have the firm capacity to pump 350 gpm with 1 pump operating. The pump control system includes a pressure transducer to monitor the wet well level, a control panel, back-up mercury floats for the high-level alarm, and a 20 kW stand-by generator for emergency power.

An 8" force main discharges from the Lift Station to the main 15" gravity collector on Moss Street.

Figure 2-1: Alder Street Pump Station

Figure 2-2: Alder Street Pump Station Wetwell and Intake (Left) and Emergency Overflow and Weir (Right)

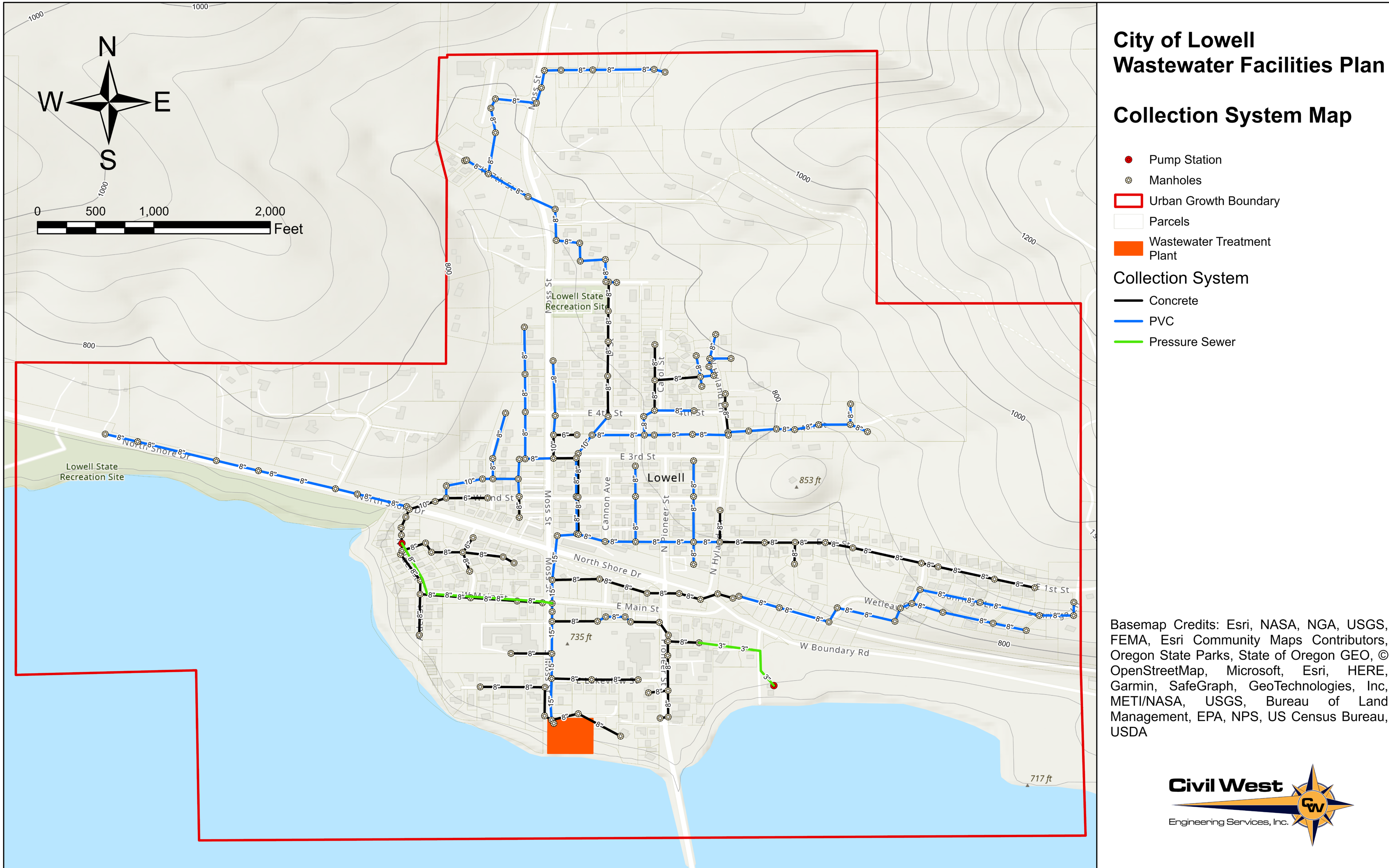


Figure 2-3: The City of Lowell Sanitary Sewer Collection System

2.2 Wastewater Treatment Plant

The WWTP treats domestic sewage by primary clarification, biological treatment, and chlorine disinfection prior to submerged discharge to Dexter Reservoir 20 feet upstream of the penstocks. An annotated aerial photo of the WWTP is provided in Figure 2-4. A hydraulic profile of the WWTP is provided in Figure 2-5.

The facility consists of headworks containing an inclined self-cleaning fine screen, a bypass channel with a bar screen, and a Parshall Flume for flow measurement. A rectangular primary clarifier removes solids, then primary clarified wastewater is biologically treated by a trickling filter and solids contact aeration reactor. Disinfection is accomplished by liquid sodium hypochlorite and excess chlorine is removed via calcium thiosulfate.

Final effluent is conveyed via a submerged outfall. The penstock, operated by the Army Corps of Engineers (ACE) controls the outflow of the dam into the Middle Fork Willamette River. Separated solids and wasted biological sludge is stabilized via an aerobic digester, and stabilized solids are stored in drying beds prior to being removed every two years for further treatment at Heard Farms near Roseburg, OR.

2.2.1 History

The original treatment facility was designed in 1950 by ACE. At the time, the facility consisted of a bar screen, an Imhoff tank, a trickling filter, a clarifier, a chlorine contact chamber, sludge drying beds and a 10-inch outfall to Dexter Reservoir approximately 75 feet south of the treatment plant.

The plant was upgraded in 1989 with the addition of a solids contact chamber and a new clarifier following the trickling filter. The original clarifier and chlorine contact chamber were converted to a new chlorine contact chamber and dechlorination chambers, respectively. The original outfall was replaced with the existing submerged outfall that discharges next to the dam's penstock.

In 2004, the original Imhoff tank was converted to an aerobic digester to stabilize solids. The original headworks were decommissioned, and new headworks and the primary clarifier were constructed. The sludge drying beds were deepened to increase volume and a new liner and subdrain system was installed at the base of the drying beds. The rock filter media in the trickling filter was replaced with plastic media. A scrubber was installed in the chemical dosing room to treat chlorine gas in the event of leakage, and a new baffle was installed in the chlorine contact tank to distribute flow along the entire circumference of the circular tank.

Around 2014, the gaseous chlorine disinfection system was retrofitted into a liquid sodium hypochlorite system.

City of Lowell Wastewater Treatment Plant

Wastewater Facilities Plan

0 25 50 100 Feet

N W E S

Civil West Engineering Services, Inc.

Basemap Credits: Maxar

Figure 2-4: Aerial Photography and Overview of the City's WWTP

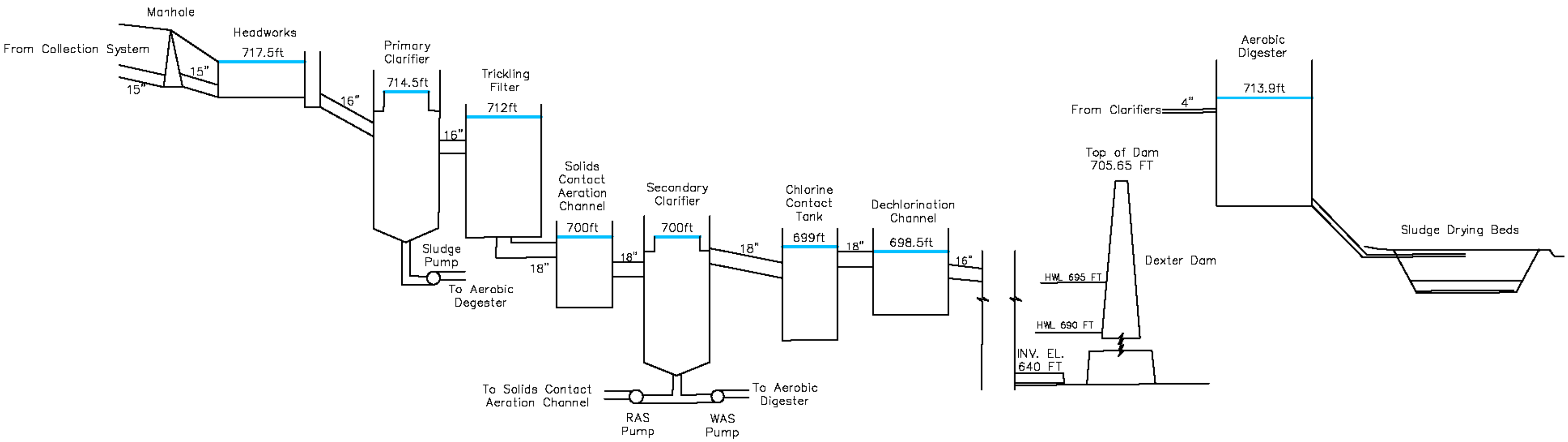


Figure 2-5: Hydraulic Profile of Existing WWTP

2.2.2 Influent Conveyance

The WWTP is gravity fed from the collection system. The lowest manhole in the collection system, with an approximate depth of 6', is located just before the headworks (Figure 2-6). This manhole has an invert elevation of 715.35'. A 15" line connects this manhole to the headworks channel. The water level in the manhole is 2.15' under design conditions.

Figure 2-6: Manhole upstream of the WWTP Headworks

2.2.3 Headworks

The primary purpose of the headworks is to provide initial screening of the influent wastewater. It is necessary to remove rags and large debris that could negatively impact downstream treatment processes, especially the clarifier sludge pumps and the trickling filter. The components located in the headworks include a fine screen, a bypass channel, an influent sampler, and a flow meter. The headworks channel has a design hydraulic head of 717.5'.

2.2.3.1 Fine Screen

The fine screen was supplied by *Treatment Equipment Company*, manufactured by *Parkson Corporation*. It is an automated inclined fine screen with a screen size of ¼". The peak flow capacity of the screen is 2.6 MGD. An ultrasonic sensor upstream of the screen monitors the depth of the channel. When the channel is above a 4' setpoint (which can be controlled by the plant operator), a mechanical brush clears the screenings automatically. The screenings are automatically washed and compacted to prepare for transport to landfill.

Figure 2-7: Inclined Fine Screen

2.2.3.2 Bypass Channel

Flow can be routed around the fine screen through a bypass channel via manually opened stop-gates. There is a grate to provide coarse screening in the bypass channel. Debris smaller than a 2" nominal diameter can pass through the grate in the bypass channel, resulting in much poorer performance than the ¼" fine screen. The bypass channel converges with the main headworks channel, the channel housing the fine-screen, prior to the Parshall flume.

2.2.3.3 Influent Flow Measurement

After the convergence of the fine screen and bypass channel, influent wastewater passes though a 9" Parshall flume for flow measurement. This flume was installed in the 2004 plant upgrades, however it does not currently have a water level sensor installed and is therefore not collecting data. Flow data for the plant is currently collected by a similar flume near the effluent of the disinfection system.

2.2.3.4 Influent Sampler

An influent sampler (*Hach Sigma AWRS*) collects influent samples in the fine screening channel by vacuum through 3/8" flexible tubing. The sampler is automated and collects samples several times per day to make 24-hour composites as required by the plant's NPDES permit. The sample bottle is contained in a temperature-controlled cabinet set at 4°C.

Figure 2-8: Influent Sampler and Fine Screen Control Panel

2.2.4 Primary Clarifier

The primary clarifier was constructed as part of the 2001 WWTP improvements for the purpose of removing excess solids detrimental to the tickling filter. Originally planned as a circular clarifier, the design was finalized as two parallel rectangular clarifiers with a depth of 12', and a combined surface area of 952 square feet. The design overflow rate at the design PDAF is 2027 gpd/sq-ft, and the 2721 gpd/sq-ft at the design PHF. Flow control slide gates control the operation of the clarifier, with both cells able to be taken offline via closing the respective slide gate. Generally, only one of the cells is in operation; both cells are used only during high flow periods that generally coincide with rain events. Prior to the overflow weir, a scum collection pipe scrapes fat, oil, and grease that collects on the surface of the water and discharges into the aerobic digester. Chain driven flights scrape the settled solids to four sump areas that pump to the aerobic digester.

Figure 2-9: Primary Clarifier Weir and Scum Collector (Left) and Sludge Scraping Mechanism (Right)

2.2.5 Secondary Treatment

2.2.5.1 Trickling Filter

The plant's trickling filter receives flow from the primary clarifier and passes it through polypropylene media for biological treatment. Forced air is pulled through the filter media by a constantly running exhaust fan to supply oxygen to the bacteria growing as a film on the filter media. The trickling filter is 8 feet deep and 33 feet in diameter, with a media volume of 6,840 cubic feet. The capacity of the trickling filter is 868 gpm; flows over this are diverted to the solids contact channel. The hydraulic loading rate is 0.35 gpm/square foot for the design MMDWF and 1.57 gpm/square foot for the design PDF. The average and maximum BOD loading rates are 0.049 lbs/day/cubic foot and 0.089 lbs/day/cubic foot respectively. The hydraulic and BOD loading rate classifies the trickling filter within the range of a "High Rate" filter for the MMDWF, meaning that the expected BOD removal is between 70-90% for plastic media filters (Metcalf and Eddy, 5th Edition).

During the dry weather period, the trickling filter typically operates with a recirculation ratio over 3. This is higher than typical for these types of trickling filters and is due to the large difference between typical summer flowrates and the max month design flow that the trickling filter was designed for. The high recirculation ratio is necessary in the summer months to keep the hydraulic distribution arms of the trickling filter spinning fast enough to wet the entire media surface and maintain the biological film's activity. In contrast, in the winter when flows are high during heavy storm events, the trickling filter does not recirculate at all.

Figure 2-10: Trickling Filter with Polypropylene Media

2.2.5.2 Solids Contact Channel

The Solids Contact Channel is an aerobic bioreactor that treats the trickling filter effluent and the decant water from the aerobic digester. The reactor is split into two sections: the reaeration section and the contact section. In the reaeration section, the return activated sludge (RAS) from the bottom of the secondary clarifier is aerated to supply oxygen to the heterotrophic bacteria in the RAS and to improve the flocculation properties of the sludge. In the contact section, the aerated RAS is mixed with the trickling filter effluent and suspended via fine bubble diffusers. The solids retention time (SRT) of the bioreactor can be adjusted by opening and closing gates that separate the two sections, which allows for controlled short circuiting between the reaeration and contact sections of the reactor. The reactor is typically operated with a SRT of 0.7 to 2 days for wet weather and dry weather flows respectively.

Figure 2-11: Solids Contact Channel

2.2.5.3 Secondary Clarifier

The secondary clarifier receives flow from the solids contact aeration channel. The clarifier has a 40-foot diameter and a 14-foot depth. The original design, as stated in the 1990 O&M Manual, had a design surface overflow rate of 1488 gpd/square foot at peak flow. The length of the overflow weir is 126 feet, and the design peak weir loading rate is 14,840 gpd/foot. There are six uptake pipes with telescoping valves along the collection arm, with three pipes on each side of the center column for sludge collection. The sludge is partitioned into waste activated sludge (WAS) and RAS. WAS is pumped to the aerobic digester via a 1 hp sludge pump with a capacity of 80 GPM. RAS is recirculated through the solids contact aeration channel via two 3 hp sludge pumps, with capacity ranging from 200 to 600 gpm. The catwalk of the secondary clarifier does not reach across the entire diameter of the clarifier, which has created issues with maintenance for the operators.

Figure 2-12: Secondary Clarifier

2.2.6 Disinfection

2.2.6.1 Chlorine Dosing

Chlorine dosing equipment is housed in the control building. A 2” pipe is routed from the chlorine contact chamber through the control building to serve as the dosing point for the contact chamber. Chlorine is dosed via a liquid solidum hypochlorite solution stored in 55-gallon drums. There are two chemical dosing pumps (ProMinent gamma/L 1601 Metering Pumps) each with a capacity of 7 gpd. Typically, the feed rate is 2-3 gpd. During normal conditions (wastewater flows around 50,000 gpd), only one pump is in operation with the second one as a backup. The pump feed rates are set manually by the operator based on the constantly monitored chlorine residual at the contact chamber effluent.

2.2.6.2 Chlorine Contact Chamber

The chlorine contact chamber is constructed inside of the WWTP’s original secondary clarifier. Water is fed at the circumference of the converted clarifier and flows inward to a weir in the center of the chamber. The total volume of the basin is 31,400 gallons and the contact time as stated in the facility’s operation and maintenance manual varies between approximately 20 and 100 minutes at peak day flow and maximum-month dry-weather flow respectively. However, as discussed in Section 2.4.7, these contact times are probably significantly overestimated due to the lack of baffling in the basin.

2.2.6.3 Dechlorination

Dechlorination is accomplished via the addition of a purchased calcium thiosulfate solution (Captor). The Captor is stored in a 55-gallon drum and is pumped via one chemical dosing pump of the same type as the chlorine dosing pumps. Carrier water for the Captor dose is supplied via the City’s potable water. Under normal conditions, approximately 1 gpd is needed to reduce the chlorine residual to levels acceptable according to the plant’s NPDES permit. The Captor dosing rate is set manually by the operator based on the chlorine analyzer results at the end of the basin. The chlorine residual is usually maintained at 1.5 mg/L, and the Captor is dosed at approximately 1/3 of the residual. The reaction time for Captor to remove the residual chlorine is instantaneous. The dechlorination reaction occurs in the reaeration chamber, located in the facility’s original chlorine contact chamber.

2.2.6.4 Effluent Sampler

The effluent sampler withdraws samples by vacuum through 3/8” flexible tubing. Sampler is automated and collects samples several times per day to make 24-hour composites as required by the plant’s NPDES permit. The sample bottle located in a temperature-controlled cabinet set at 4°C.

2.2.6.5 Effluent Flow Measurement

Similar to the Parshall flume in the headworks discussed in Section 2.2.3.3, effluent flow is recorded near the end of the dechlorination channel. However, this flume is smaller than the influent flume, with a capacity of 2 MGD.

2.2.7 Outfall

The final effluent discharges to Dexter reservoir within 20 feet of the dam's penstock intake trash racks. This outfall was constructed in 1989 and consists of a 16-inch pipe that drains via gravity from the dechlorination chamber to the dam. The discharge location is considered a river discharge into the Middle Fork Willamette for permitting purposes.

2.2.8 Solids Treatment

2.2.8.1 Aerobic Digester

In 2004, the Imhoff tank of the original WWTP was converted into a dual celled aerobic sludge digester. The purpose of the digester is to stabilize the primary sludge and waste activated sludge (WAS) from secondary treatment; by supplying oxygen via fine bubble aeration, the digester reduces pathogens, the quantity of volatile suspended solids in the sludge, and the total volume of solids discharged into the sludge drying beds. The fine bubble aeration system consists of one 30 hp positive displacement blower, air piping, and eighty-eight 9-inch membrane diffusers arranged in a grid at the base of the digester. The original design originally consisted of two 30 hp blowers, with one as a backup. However, operators have expressed frustration with the maintenance and electricity costs associated with the blower configuration. When one of the blowers went out of commission around 2020, the City decided to not replace the blower in-kind after with the hopes that the system can be improved in the near future.

Each cell of the digester contains an adjustable weir to decant or overflow water at the surface back to the Solids Contact Aeration Basin. Decant usually occurs on a daily basis in the winter and 3 times a week during the summer, and discharges about 800 to 1,000 gallons per decant cycle to secondary treatment.

2.2.8.2 Sludge Drying Beds

Figure 2-13: Eastern Sludge Drying Bed

The sludge drying beds receive stabilized solids from the aerobic digester. There are two drying beds with a combined volume of 119,000 gallons. There is a 60 mil HDPE liner at the base of the drying beds with an underdrain system that drains to a sump near the secondary clarifier for treatment through the solids contact aeration basin. The available detention time in the drying beds is 1.8 years, assuming 20 pounds of sludge per capita per year at 5% solids. Solids are ultimately hauled by and disposed to Heard Farms near Roseburg, OR. Heard Farms performs further treatment to the solids to meet Class B biosolids Criteria as required by their Water Pollution Control Facilities Permit (Permit #102449).

2.3 Design Criteria of Existing Facilities

A summary of design criteria for each of the components of the WWTP is provided in Table 2-1. Information compiled in this table was sourced from the City's Operations and Maintenance manual, the 2004 pre-design report, and discussions with operators.

Table 2-1: Design Criteria of WWTP Processes

City of Lowell - Wastewater Treatment Plan Design Criteria	
Headworks	
Fine Screen	
Type	Automated Inclined Fine Screen
Screen Size	0.25 inches
Peak Flow Capacity	2.6 MGD
Screenings Washing and Compaction	Yes
Channel Width	2 feet
Max Depth	4 feet
Design Channel Depth	3 feet
Bypass	
Type	Course Bar Screen
Screen Size	2"
Cleaning	Manually-Cleaned
Flow Diversion	Manually-Operated Stop Gates
Channel Width	2 feet
Max Channel Depth	4 feet
Flow Measurement	
Type	Parshall Flume
Size	9 inches
Flow Measurement	Transducer (Not Installed)
Influent Sampler	
Type	Automated Composite
Temperature	4°C
Primary Treatment	
Primary Clarifier	
Type	Rectangular
# of Cells	2 in parallel
Total Surface Area	952 square feet
Side Water Depth	12 feet
PDAF Surface Overflow Rate	2027 gpd/square foot
PHF Surface Overflow Rate	2721 gpd/square foot

Table 2-1: Design Criteria of WWTP Processes

Primary Sludge Pump	
Type	Progressing Cavity
# of Pumps	2 (1 redundant)
Design Capacity	20 gpm
Average Sludge Production	2400 gpd
Typical Operating Time	2 hour/day
Secondary Treatment	
Trickling Filter	
Type	High-rate, plastic media
Diameter	33 feet
Media	Polypropylene
Media Area	855 square feet
Media Depth	8 feet
Media Volume	6,840 cubic feet
Average BOD Loading	0.049 lbs/day/cubic feet
Max BOD Loading	0.089 lbs/day/cubic feet
Hydraulic Loading at Design MMDWF	0.35 gpm/square foot
Hydraulic Loading at Design PDF	1.57 gpm/square foot
Air Supply	1 HP Exhaust fan
Solids Contact Basin	
Type	Activated Sludge
Basin Depth	6 feet
Contact Channel Width	6 feet
Contact Channel Volume	8600 cubic feet
Design Hydraulic Detention Times:	
ADWF	12 minutes
MMDWF	25 minutes
MMWWF	16 minutes
PDF	6.4 minutes
Reaeration Channel Depth	6 feet
Reaeration Channel Width	2.5 feet
Reaeration Channel Volume	3590 gallons
# of Blowers	2
Blower Capacity	40-150 scfm, each
Design Solids Retention Time	0.7 - 2 days
Secondary Clarifier	
Type	Circular
Total Surface Area	1260 square feet
Diameter	40 feet
Side Water Depth	14 feet
Design Surface Overflow Rate at Peak Flow	1488 gpd/square foot
Design Weir Loading Rate at Peak Flow	14840 gpd/ft
Design Detention Time at ADWF	21.4 hours
Design Detention Time at PIF	1.7 hours

Table 2-1: Design Criteria of WWTP Processes

Activated Sludge Pumps	
# of Return Activated Sludge Pumps	2
RAS Pump Capacity	600 gpm, each
# of Waste Activated Sludge Pumps	1
WAS Pump Capacity	80 gpm
Disinfection	
Disinfection Dosing	
Type	Liquid Sodium Hypochlorite
Chemical Storage	55-gallon drums
Number of Injection Valves	2
Pump Capacity	7 gpd, each
Average Feed Rate	2-3 gpd
Feed Control	Manual
Chlorine Flash Mixer	
Motor Size	1 HP
Velocity Gradient	500 sec^{-1}
Impeller Diameter	13.2"
Chlorine Contact	
Type	Circular Tank with Baffles
Total Volume	31,400 gallons
Contact Time at MMDWF	103 minutes
Contact Time at PDF	23 minutes
Dechlorination	
Type	Calcium Thiosulfate (Captor)
Chemical Storage	55-gallon drums
Number of Injection Valves	1
Pump Capacity	7 gpd
Average Feed Rate	1 gpd
Feed Control	Manual
Effluent Sampler	
Type	Automated Composite
Temperature	4°C
Solids Treatment	
Digestion	
Type	Aerobic
Number of Basins	2
Volume	130,000 gallons, each
Solids Yield	358 lbs/day
SRT (Average at 2% solids)	81 days
Volatile Solids Destruction	38%
Aeration	Fine Bubble Diffusers
Blowers	2, each at 30 HP
Air Rate	580 scfm (at 6.5 psi)
Mixing Air Provided	25 scfm per 1000 cubic feet

Table 2-1: Design Criteria of WWTP Processes

Solids Dewatering	
Type	Gravity Thickening, Drying Beds
Depth	3 feet
Volume	119,000 gallons
Solids Content	5%
SRT	6 months
Design Flowrates and Loads	
Flowrates	
Average Dry Weather Flow	0.22 MGD
Max-Month Dry Weather Flow	0.44 MGD
Max-Month Wet Weather Flow	0.76 MGD
Peak Day Flow	1.93 MGD
Peak Hour Flow	2.60 MGD
Pollutant Loads	
Average BOD	448 lb/day
Maximum BOD	761 lb/day
Average TSS	377 lb/day
Maximum TSS	642 lb/day

2.4 Condition of Existing Facilities

2.4.1 Collection System

One of the City's biggest issues with their wastewater system is the excessive amount of wet weather associated inflow and infiltration (I/I) into the collection system. This results in excessively high wet weather flows relative to dry weather flows, which makes operation of the WWTP more difficult since none of the operations are flow paced, and the operator has to adjust operations to accommodate higher flows during storm events. A more comprehensive analysis of I/I in the collection system is discussed in Section 3.2.1.1, and an I/I study was conducted as part of this planning effort to identify areas that can be improved to mitigate this issue (Appendix D).

2.4.1.1 Alder Street Pump Station

The pumps in the Alder Street Pump Station were upgraded in 2004 to accommodate flows of 350 gpm with just one pump in operation. The wet well and the pumps seem to be in good condition. There was one recent overflow into Dexter Reservoir in 2021, which may be indicative of a need to upsize these pumps again. A more comprehensive evaluation of pump capacity is provided in Section 3.3.4.1

2.4.2 Headworks

The headworks were recently updated in 2004. In general, the individual unit operations of the headworks are in good condition and the operator is satisfied with the fine screen that is in place. According to the current operator's experience, the bypass channel is only needed to accommodate excessive influent flow during severe storm events, usually those close to the area's 5-year storm.

The main area of concern with the headworks is that the influent flow measuring Parshall flume is not actively recording data, as the plant reports effluent flow to fulfill monitoring requirements. It is recommended to install the transducer in the flume to collect influent flow data so that each part of the headworks operates as designed.

2.4.3 Primary Clarifier

The primary clarifier was constructed as part of the 2004 upgrades. The clarifier seems to operate well, with generally one cell in operation in the summer and both during high flow events in the winter. There have been no issues reported with the scum removal or sludge collection mechanisms. The sump pumps of the clarifiers have exceeded the typical useful life of pumps (10-20 years).

2.4.4 Trickling Filter

The exhaust fans of the trickling filter have never been replaced and seem to be in poor to fair condition based on rust on the external housing.

The operation of the trickling filter is most affected by the large variation of flows experienced by the WWTP. A high recirculation ratio (over 4 times the WWTP influent flow) is often necessary in the summer to sustain the required hydraulic loadings, which results in low BOD loadings to the solids contact aeration basin. Nitrification occurring in the aeration basin may explain the low effluent pH values that occur in the dryer part of the year (See Section 4.5.3).

In contrast, the trickling filter is not recirculated as designed during high flow wet-weather periods, resulting in poor treatment efficiency. High flows generally cooccur with lower temperatures, which decreases biological activity, and the lack of recirculation decreases retention time in the biological reactor. Furthermore, most of the excess flow during the high flow, wet-weather periods is from rain and groundwater entering the collection system, resulting in lower influent BOD concentrations. The WWTP generally meets its current permit limits despite the process inefficiencies; however, there have been multiple exceedances in the past 5 years (See Section 3.2.5).

Overall, the trickling filter, designed for a flow that statistically only occurs a couple weeks each year, performs far outside of its design criteria during most of the year because of the large variation in flows.

2.4.5 Solids Contact Aeration Basin

Based on operational sludge testing records, the aeration basin operates consistently within 20% of the design MLSS concentration of 1,800 mg/L. While the opening of different gates in the channel between the contact and reaeration sections can be used to modify the solids retention time, the unit has only been operated with the first gate open (the “default” configuration as designed) in the current operator’s experience.

There have not been any issues reported with the aeration system, although the blowers are past their expected useful life of 20-25 years. One point of concern with the aeration basin is the lack of redundancy; if the basin was to be taken offline to replace diffusers or too otherwise maintenance the basin, the plant would have to rely only on the trickling filter for treatment. This may be insufficient to meet current or future permit requirements.

2.4.6 Secondary Clarifier

The City is out of compliance with the Oregon Department of Environmental Quality's (DEQ) redundancy requirements by only having one secondary clarifier (See Section 3.2.5). Also, there are some safety concerns with the existing clarifier associated with the catwalk only extending half the diameter of the basin. The WAS and RAS pumps, installed in 1989, are past their design life.

The mechanism of the clarifier is well maintained and in good condition. The City has a replacement drive unit available and ready to be replaced whenever needed, since the unit is over the theoretical design life of 20 years for mechanical equipment. There are no major apparent structural issues with the concrete basin.

2.4.7 Disinfection

The nominal contact time of the chlorine contact chamber of 23 minutes at a flow of 1.93 MGD is greatly overestimated. This contact time was calculated as the volume of the reactor divided by flow rate which inherently assumes plug flow behavior, like in a serpentine contact chamber. This assumes that no short circuiting within the chamber occurs, which is highly unlikely given the tank's indeterminant length to width ratio. The City's chlorine contact chamber is a repurposed circular clarifier from the original ACE WWTP. The flow moves from a point on the circumference of the clarifier towards the center of the basin, and then over a weir prior to the dechlorination chamber. The issue with this is that the reactor is unbaffled, and the assumption that the entire contact chamber volume is effectively used for disinfection contact is incorrect.

According to the EPA's Disinfection Profiling and Benchmarking Guidance Manual, a tank with a single baffle or multiple unbaffled inlets or outlet, and with no intra-basin baffles, a baffling factor of 0.3 should be applied to the basin's volume to correct for short circuiting. This would make the effective contact time at the design flow approximately 8 minutes, which is insufficient to meet DEQ requirements.

2.4.8 Solids Management

The aerobic digester was constructed as part of the 2004 WWTP upgrades. According to the operator, the maintenance requirements of the rotary lobe blower used for aeration are excessive, requiring full oil and gasket replacements monthly and constant maintenance due to overheating in the warm, summer months. The electricity costs of this blower are $1,500 per month, more than half the typical operating costs of the entire WWTP.

The aerobic digester has some signs of concrete deterioration on the outer side of the tank. This is not entirely surprising given the old age of the structure, which was originally the Imhoff tank of the original ACE WWTP.

At the most recent hauling of solids from the drying beds in 2023, it was discovered that one of the underdrain pipes was broken and the felt layer that separates the underdrain layer from the sand buffer and solids storage section has deteriorated. This is likely a sign that the underdrain system has experienced wear throughout the scraping and hauling cycles. It may be feasible to add some guide walls in the solids drying beds to make it easier to perform removal of solids without damaging the underdrain system, although this would likely decrease the total available solids storage volume.

2.5 Finanacial Status of Existing Facilities

Financial data for the City was obtained from *Independent Audit Reports* that are publicly available on the City's website. In these reports, the main accounting method that has been used is modified-cash basis accounting and the data below is based on City of Lowell's Actual and Budget Statements regarding Sewer Funds. The City's Sewer Fund consist of three sub-accounts: an Operating Fund, a System Development Charges (SDC) Fund, and the Reserve Fund. The City invests in the Local Government Investment Pool (LGIP) which is managed by the State Treasurer's office, and the City records the earnings from this pool as Investment Earnings.

Table 2-2 includes the information regarding the city's resources and expenditures. The overall picture is positive. The Debt Service amount reflects the Sewer Revenue Loan annual payments that have been taken from USDA Rural Utilities Service beginning in 2012 with an interest rate of 2.75% and a maturity date of April 6, 2052. Table 2-3 summarizes each sub-account of the Sewer Funds by showing the net change for each of the past few years.

Table 2-3 shows the change in values of the City's wastewater facility assets and liabilities over the past few years, and the net position of the City's sewer utility. This can be an indicator to determine if the City's sewer balance sheet is improving or deteriorating. The numbers show decreasing positive value, which is mostly reflective of accumulated depreciation.

Table 2-2: Resources and Expenditures of the City's Wastewater Facilities

Resources	2019	2020	2021	2022
Sewer Operating Fund	**$376,382**	**$376,664**	**$418,914**	**$430,970**
Charges for Services	$342,844	$361,249	$386,433	$406,487
Sewer Connections and Permits	$1,610	$805	$575	$3,795
Intergovernmental	-	-	$24,364	-
Reimbursement of SDC fees	$6,891	$5,241	$3,090	$19,158
Investment Earnings	$4,655	$1,899	$734	$633
Miscellaneous	$8,212	$2,187	$3,589	$897
Other Financing Sources (balance)	$12,170	$5,283	$129	-
Sewer Reserve Fund	**$1,576**	**$1,580**	**$5,925**	**$21**
Investment Earnings	$1	$5	$5	$21
Other Financing Sources (balance)	$1,575	$1,575	$5,920	-
Sewer SDC Fund	**$128,017**	**$11,728**	**$6,414**	**$33,752**
SDC Fees	$11,942	$9,082	$5,355	$33,201
Investment Earnings	$158	$2,646	$1,059	$551
Other Financing Sources (balance)	$115,917	-	-	-
Total Revenues	**$505,975**	**$389,972**	**$431,253**	**$464,743**
Expenditures				
Current Account				
Personal Services	$133,446	$169,294	$184,402	$189,970
Materials and Services	$121,939	$139,337	$149,747	$215,559
Debt Service				
Principal	$28,489	$29,013	$29,563	$30,139
Interest and other changes	$23,419	$22,220	$20,980	$19,698
Capital Outlay	$14,558	$42,745	-	$23,377
Total Expenditures	**$321,851**	**$402,609**	**$384,692**	**$478,743**

Table 2-3: Wastewater Facility Account Balances

Sewer Operating Fund Summary	2019	2020	2021	2022
Beginning	$121,619	$176,150	$150,205	$184,427
Net Change	$54,531	-$25,945	$34,222	-$100
Ending	$176,150	$150,205	$184,427	$184,327
Sewer Reserve Fund Summary	**2019**	**2020**	**2021**	**2022**
Beginning	$6,670	$8,246	$9,826	$15,751
Net Change	$1,576	$1,580	$5,925	$21
Ending	$8,246	$9,826	$15,751	$15,772
Sewer SDC Fund	**2019**	**2020**	**2021**	**2022**
Beginning	-	$128,017	$139,745	$146,159
Net Change	$128,017	$11,728	$6,414	-$13,921
Ending	$128,017	$139,745	$146,159	$132,238

Table 2-4: Modified Cash Basis of Wastewater Facility Assets

Sewer Fund Balance Sheet (Modified Cash Basis)				
	2019	**2020**	**2021**	**2022**
Assets				
Cash and Cash Equivalents	$312,411	$299,776	$346,337	$330,562
Other current Assets	-	-	-	$1,775
Land	$11,000	$11,000	$11,000	$11,000
Buildings and Facilities	$81,869	$89,114	$89,114	$89,114
Vehicles and Rolling Stock	$34,064	$21,780	$21,780	$21,780
Equipment and Furniture	$33,629	$68,935	$68,330	$91,707
Infrastructure	$4,708,963	$4,708,963	$4,708,963	$4,708,963
Accumulated Depreciation	-$2,757,719	-$2,860,791	-$2,974,881	-$3,090,135
Total non-current Assets	$2,111,806	$2,039,001	$1,924,306	$1,832,429
Total Assets	**$2,424,217**	**$2,338,777**	**$2,270,643**	**$2,164,766**
Liabilities				
Current Liabilities				
Bonds, notes and loan payable	$29,013	$29,563	$30,139	$35,743
Non-current Liabilities				
Bonds, notes and loan payable	$576,682	$547,120	$516,981	$481,238
Total Liabilities	**$605,695**	**$576,683**	**$547,120**	**$516,981**
Position				
Net Investment in Capital Assets	$1,506,111	$1,462,318	$1,377,186	$1,315,448
Restricted for Debt Service	$8,246	$9,826	$15,751	$15,772
Restricted for Capital Projects (SDC)	$128,017	$139,745	$146,159	$132,238
Unrestricted	$176,148	$150,205	$184,427	$184,327
Net Position	**$1,818,522**	**$1,762,094**	**$1,723,523**	**$1,647,785**

2.5.1 Water, Energy, and Waste Audits

The City has not completed any water, energy or waste audits in the past five years.

3 NEED FOR PROJECT

Drivers for wastewater facility capital improvement projects typically fall into one of three categories:

- Protection of human and environmental health;
- Replacing or rehabilitating infrastructure and equipment nearing or exceeding its useful life;
- Accommodation of expected growth in the planning area.

This section describes the current factors influencing each of these drivers with regard to the City of Lowell.

3.1 Health, Sanitation, Environmental Regulations, and Security

Many State and Federal regulations have been established to ensure the health, safety, and security of the public. This section discusses the relevant regulations governing the City's wastewater system facilities.

The federal Clean Water Act (CWA) requires permits for all discharges of wastewater to waters of the state. The CWA is delegated to the State of Oregon and enforced through Oregon Revised Statutes (ORS 468B.050). The City operates its wastewater system under the jurisdiction of the Oregon Department of Environmental Quality (DEQ), with a National Pollutant Discharge Elimination System (NPDES) Waste Discharge Permit (Permit No. 101384) which was issued June 30, 2014 (Appendix A).

NPDES permits in Oregon are issued for 5-year periods. When a permit lapses and a new permit is not issued, as is the case with the City's wastewater treatment plant, the permit is administratively extended until a new permit can be issued. The City of Lowell is expected to have a new permit issued in 2028 according to DEQ's Statewide Permit Issuance Plan. At the time a new permit is issued, any changes to federal and state regulations that occurred since the last permit are incorporated.

3.1.1 Collection System Requirements

Performance requirements for collection system pipelines and lift stations are provided in the appendices to OAR 340-052-0020. These guidelines generally require that collection system infrastructure be designed with adequate capacity to convey the peak flow rates. Additional requirements, including redundancy and reliability requirements for pumping systems, are outlined in those appendices. Regarding the operator certification level required to work on the City's collection system, the City's collection system is classified as "Class II."

3.1.2 Treatment System Requirements

Treatment system performance requirements for facilities that discharge to surface waters are heavily influenced by the need to protect the quality of the receiving water for other beneficial uses. This section reviews the current requirements that the City's WWTP is subject to and future requirements that will impact the WWTP performance requirements. The treatment system is classified pursuant to OAR 340-049 as "Class III."

3.1.2.1 Current Wastewater Treatment Plant Discharge Requirements

Treated effluent quality from the WWTP is governed by the facility's NPDES permit. The current permit was issued in 2010 and was set to expire in 2014; the permit is currently administratively extended. Final effluent is discharged 20 feet upstream of the penstocks of Dexter Reservoir, located on the Middle Fork Willamette River at River Mile 15.7. The quality of the final effluent has to meet water quality criteria that varies seasonally, based on effluent flowrates of 0.15 MGD (average dry weather) and 0.23 MGD (average wet weather). Table 3-1 provides the waste discharge limits required under the facility's current NPDES permit.

Table 3-1: NPDES Permit Limits for the City of Lowell Wastewater Treatment Plant

Seasonal Effluent Limits					
May 1 - October 31					
	Concentration (mg/L)		**Loading (lb/day)**		
	Monthly Average	Weekly Average	Monthly Average	Weekly Average	Daily Maximum
BOD_5	10	15	13	19	26
TSS	10	15	13	19	26
November 1 - April 30					
	Concentration (mg/L)		**Loading (lb/day)**		
	Monthly Average	Weekly Average	Monthly Average	Weekly Average	Daily Maximum
BOD_5	30	45	58	87	120
TSS	30	45	58	87	120
Year-Round Limits					
E. coli	Must not exceed 126 organisms per 100 mL monthly geometric mean; no single sample can exceed 406 organisms per 100 mL				
BOD_5 and TSS Removal Efficiency	Must not be less than a monthly average of 85%				
pH	Must be within the range of 6.0 and 9.0				
Chlorine, total residual	Must not exceed a monthly average of 0.5 mg/L				

In addition to complying with effluent quality limitations, the NPDES permit also requires regular sampling of the influent for BOD_5, TSS, and pH, and the effluent for flow, BOD_5, TSS, pH, *E. coli*, temperature, and chlorine. The permit allows for land application of biosolids provided that Class B pathogen reduction standards are achieved.

3.1.2.2 Supplemental Requirements

Supplemental water quality requirements for locations in the Willamette River basin are established in OAR 340-041-0340. These are as follows:

- Water quality in the Willamette Basin must be managed to protect the designated beneficial uses shown in Table 3-2.

- Designated fish uses to be protected in the Willamette Basin are shown in Figure 3-1 and Figure 3-2. While the City's discharge location is technically upstream of Dexter Dam, the discharge location has historically been treated as a river discharge due to its proximity (20 feet upstream) to the penstocks, with more conservative mixing zone assumptions than typical river discharges (5% instead of 25% for chronic dilution zone, and 1% instead of 5% for acute dilution zone). With regard to OAR 340-041-0340, the end of the mixing zone is at the outlet of the penstocks, putting the discharge within "Core Cold Water Habitat" zone for salmonids (Figure 3-1), and in the September 15 to June 15 salmon and steelhead spawning use zone (Figure 3-2).

Table 3-2: Designated Beneficial Uses - Willamette Basin

Beneficial Uses	Willamette River Tributaries						Main Stem Willamette River			
	Clackamas River	Molalla River	Santiam River	McKenzie River	Tualatin River	All Other Streams & Tributaries	Mouth to Willamette Falls, Including Multnomah Channel	Willamette Falls to Newberg	Newberg to Salem	Salem to Coast Fork
Public Domestic Water Supply[1]	X	X	X	X	X	X	X	X	X	X
Private Domestic Water Supply[1]	X	X	X	X	X	X	X	X	X	X
Industrial Water Supply	X	X	X	X	X	X	X	X	X	X
Irrigation	X	X	X	X	X	X	X	X	X	X
Livestock Watering	X	X	X	X	X	X	X	X	X	X
Fish & Aquatic Life[2]	X	X	X	X	X	X	X	X	X	X
Wildlife & Hunting	X	X	X	X	X	X	X	X	X	X
Fishing	X	X	X	X	X	X	X	X	X	X
Boating	X	X	X	X	X	X	X	X	X	X
Water Contact Recreation	X	X	X	X	X	X	X[3]	X	X	X
Aesthetic Quality	X	X	X	X	X	X	X	X	X	X
Hydro Power	X	X	X	X	X	X	X	X		
Commercial Navigation & Transportation							X	X	X	

[1]With adequate pretreatment and natural quality that meets drinking water standards.

[2]See also Figures 340A and 340B for fish use designations for this basin.

[3]Not to conflict with commercial activities in Portland Harbor.

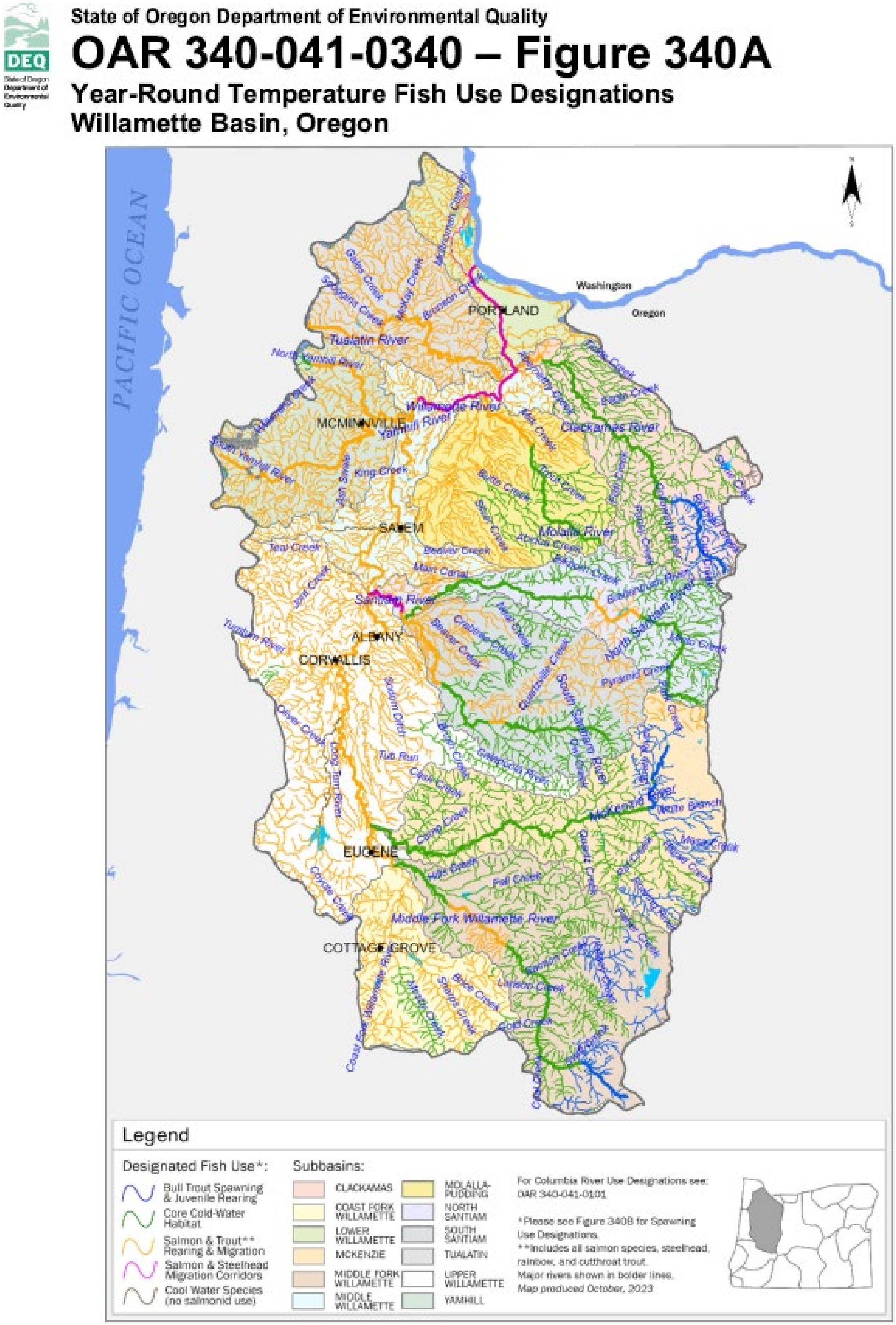


Figure 3-1: Fish Use Designations - Willamette Basin

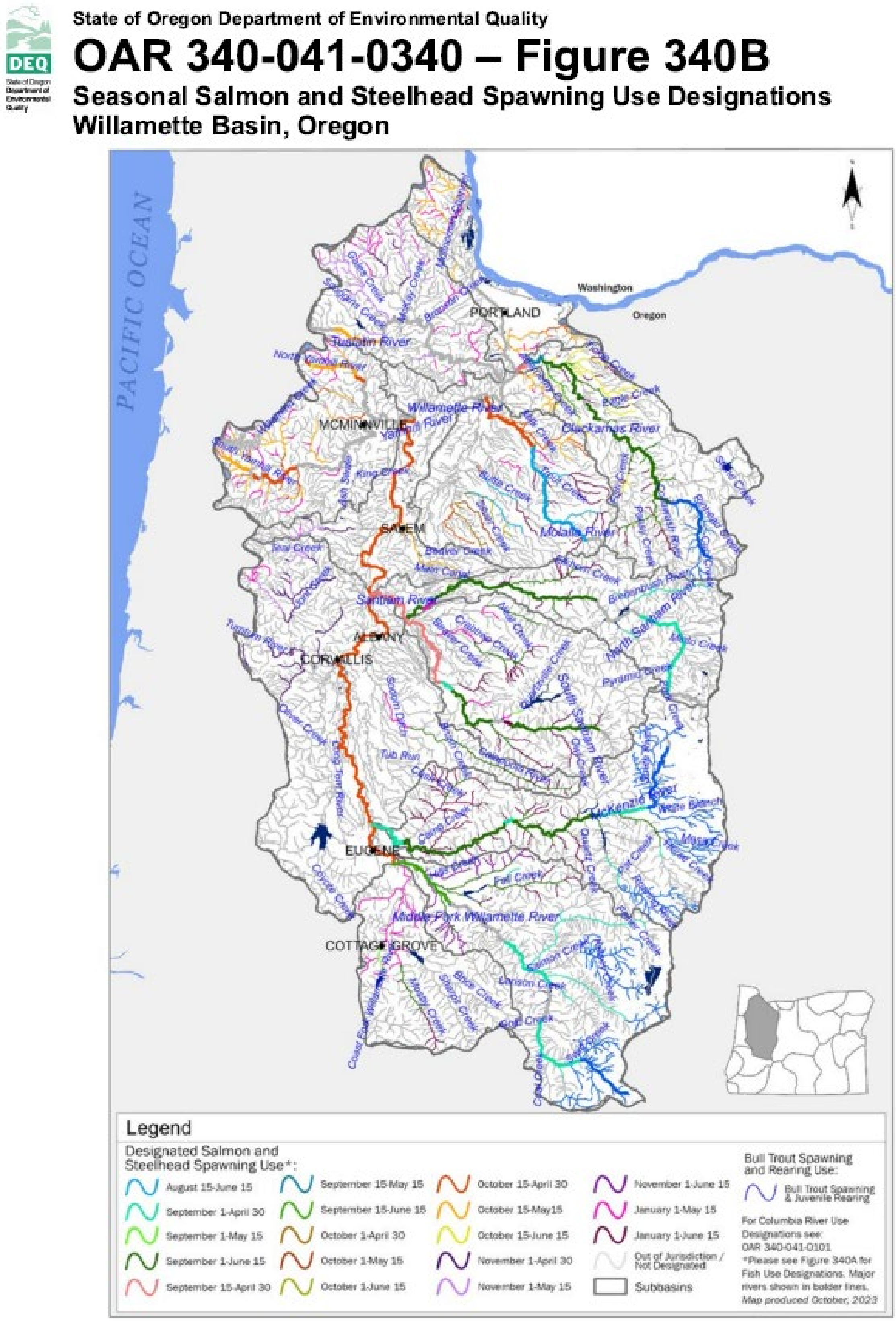


Figure 3-2: Salmon and Steelhead Spawning Use Designations - Willamette Basin

3.1.2.3 Management of Sewage Sludge/Biosolids

The wastewater treatment process results in the production of solids referred to as sewage sludge. Sewage sludge that has been treated to comply with pollutant and other limitations established by the State and Federal governments is referred to as biosolids. Management of sewage sludge is regulated under 40 CFR part 503.

All biosolids must be in compliance with the pollutant concentration and loading limits established in 40 CFR part 503.32, the vector attraction standards established in 40 CFR part 503.32, and the pathogen reduction standards established in 40 CFR part 503.33. Final biosolids are classified by 40 CFR part 503.33 as Class A or Class B. Class A biosolids are required to be treated to a higher pathogen removal standard, commonly achieved by raising pH and/or temperature above certain levels for extended periods of time.

The City currently sends stabilized and dried solids to a regional treatment facility near Roseburg for final treatment. Regulations pertaining to the land application of biosolids in Oregon are located in OAR 340-050.

3.1.3 Water Quality Status of the Receiving Waterbody

Per OAR 340-041-0004, the Antidegradation Policy guides decisions that affect water quality such that unnecessary further degradation from new or increased point and nonpoint sources of pollution is prevented and enhances existing surface water quality to ensure the full protection of all existing beneficial uses.

Dexter Reservoir on the Middle Fork Willamette River is the receiving waterbody for treated effluent from the WWTP. For surface water discharge, the City of Lowell is required to comply with Sections 442, 445, and 455 of OAR 340-041, which pertain to the Willamette Basin. The Willamette Basin is far-reaching, conveying water from the Cascade Mountains in the east, Coast Range in the west, and south of Eugene/Springfield north to the Columbia River.

Section 305(b) of the Clean Water Act (CWA) requires DEQ to assess water quality in Oregon and report on the overall condition of waters. DEQ assigns an assessment status category to each water body where data are available to evaluate. Water bodies that do not meet water quality standards are Water Quality Limited and are assigned Category 4 or Category 5. Water bodies in Category 5 need pollutant Total Maximum Daily Loads (TMDLs) developed and comprise the Section 303(d) list.

3.1.3.1 2022 Integrated Report

DEQ presented the Section 305(b) required report most recently 2022 as a story map, available on the department's mapping website (Oregon 2022 Integrated Report - Final (arcgis.com)). This story map shows all of the State water bodies and their status as impaired (not meeting water quality criteria) or attaining (meeting water quality criteria). In this report, Dexter Reservoir was listed as impaired for harmful algal blooms. The Dexter Reservoir-Middle Fork Willamette watershed unit (HUC12: 170900010703) was listed as impaired for dissolved oxygen, *E. coli* and temperature. Directly downstream of Dexter Reservoir, the Middle Fork Willamette River is listed as impaired for dissolved oxygen and temperature.

3.1.3.2 Mercury TMDL

The Middle Fork Willamette River is 303(d) listed for mercury from RM 0 to 82.2. The total mercury load from all minor sewage treatment plant facilities (population < 10,000) was estimated to be essentially 0 percent of the total mercury load in the Willamette Basin. As a minor sewage treatment plant facility, the City of Lowell will not be expected to perform additional mercury control or monitoring at the wastewater treatment plant. Mercury monitoring and treatment requirements may be required if/when the City's population surpasses 10,000 people, flow exceeds 1 million gallons per day, or if a major potential industrial source begins discharging into the City's sewer system, at which point the City would be considered a major sewage treatment plant facility. Compliance with the Mercury TMDL is currently accomplished through a TMDL implementation plan managed by the City's stormwater drainage program.

3.1.3.3 Temperature

The Middle Fork Willamette River is 303(d) listed for temperature from RM 0 to 15.6. This essentially is the confluence of the Middle Fork and Coast Fork to Dexter Dam. Reservoirs and lakes are vitally important to control the temperatures of downstream reaches. According to DEQ's 2006 temperature TMDL, the "load allocation" for Dexter reservoir is essentially no increase beyond the natural thermal potential temperatures, presented as "Monthly Target Temperatures" or seven-day average temperatures. These background temperatures in Dexter Reservoir are 6.5°C in April, 8.6°C in May, 13.2°C in June, 17.4°C in July, 16.5°C in August, 13.9°C in September, 10.2°C in October and November. The City of Lowell is required to monitor effluent temperature, but no load allocation or temperature limit has been defined in the City's most recent NPDES permits. This will likely change due to the upcoming Temperature TMDL replacement (See Section 3.1.4.1).

3.1.3.4 Bacteria

The Middle Fork Willamette River was in attainment of bacteria water quality criteria as of the 2006 bacteria TMDL. New and existing point source dischargers are required to meet the bacteria water quality standard (126 *E. coli* organisms per 100 mL for a monthly log-mean, and not in excess of 406 *E. coli* organisms per 100 mL in any single sample) prior to discharge.

3.1.4 Potential Future Regulations

In addition to the currently applicable regulations previously discussed, several additional regulations may be relevant to the facility during the planning period. This section provides brief discussions of those potential regulations.

3.1.4.1 Temperature: Thermal Load

Excessive water temperature concerns in the Willamette Basin are expected to be addressed through the issuance of a new Willamette Subbasins Temperature TMDL. According to discussions with DEQ, Lowell's WWTP is expected to be given a waste load allocation (WLA) for thermal load as part of the new TMDL. The WLA is the maximum amount of heat energy that the City's WWTP could discharge into the Middle Fork Willamette without violating the temperature TMDL. The details of this load allocation are provided in Table 3-3. Note that these are subject to change with the finalization of the TMDL.

Table 3-3: Proposed Thermal Waste Load Allocation from Draft Temperature TMDL

NPDES Permittee	Allocated Human Use Allowance (°C)	WLA Period Start	WLA Period End	7Q10 River Flow (CFS)	Effluent Discharge (MGD)	Effluent Discharge (CFS)	WLA (kcal/day)
Lowell STP	0.03	1-May	15-Nov	998.4	1.96	3.03	73,505,100

The "7Q10 River Flow" was determined as the flow from the Dexter penstock, and the effluent discharge was the maximum effluent flow reported by DEQ's review team from September 2017. The City should dispute this effluent flowrate being used as the basis for imposing future temperature limits on the City, as the flow records from that date show an average flow closer to 0.05 MGD (Appendix E). It is likely that this flowrate was a mistake during data entry.

The amount of heat energy actually discharged by the WWTP is called the "Excess Thermal Load" or ETL. The ETL must be lower than the WLA for the City to be in compliance with the temperature TMDL. ETL is defined by the following equation:

ETL = QE x (TE-TR) x 3.785

Where,
ETL = Excess Thermal Load, million kcals/day
QE = Daily average effluent flow, MGD
TE = Daily maximum effluent temperature, °C
TR = Applicable criterion, °C (will be listed in the TMDL and in permit renewal)
3.785 = Conversion factor

The past five years of data during the WLA period as reported by the City's DMRs was evaluated using the equation above to calculate ETL based on flowrate and effluent temperature, and an assumed temperature criterion of 12.3°C. This criterion was listed in the draft TMDL as the lowest criterion temperature for the Middle Fork Willamette River; although the actual criterion temperature may end up being slightly higher in the finalized TMDL, this was used for a conservative estimate. As shown in Figure 3-3, the City's ETL has consistently been much lower than the 73.5 million kcal/day WLA.

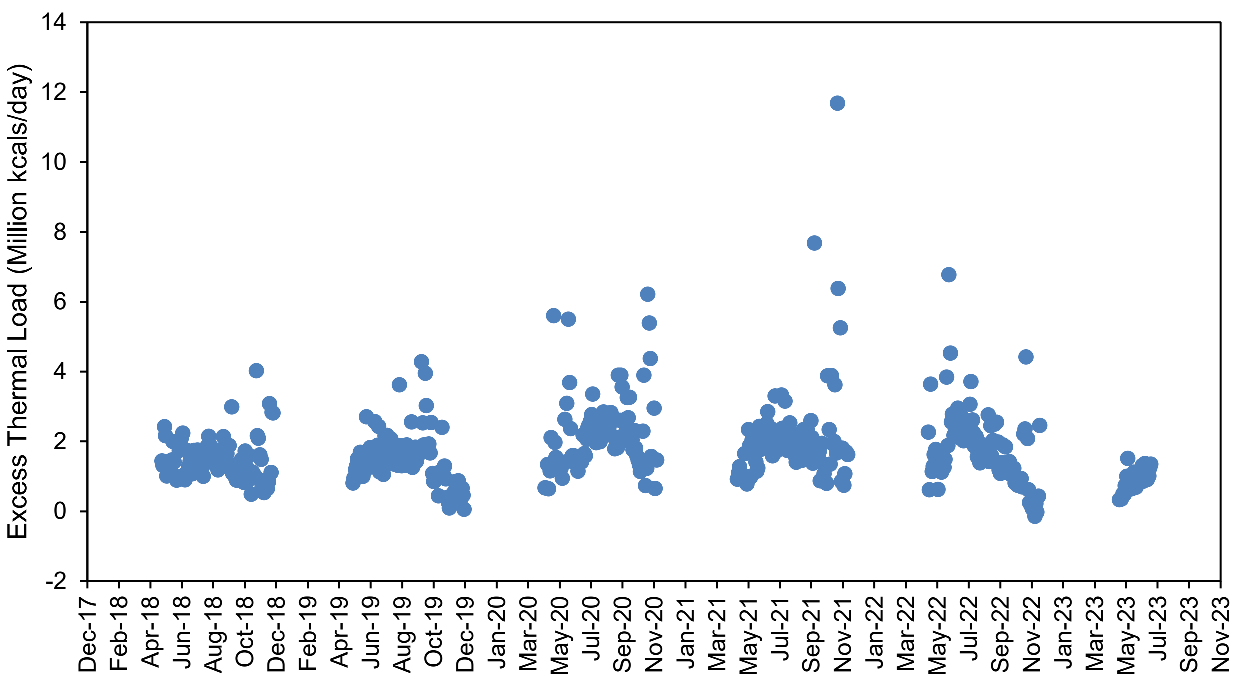


Figure 3-3: Calculated Excess Thermal Load from Previous Five Years of DMRs

To determine if the City will be given a temperature or thermal load limit in future permit renewal cycles, DEQ will most likely perform a “reasonable potential analysis,” or RPA. The thermal load from the WWTP is inherently a function of effluent flowrate and temperature. The RPA will involve comparing plant and ambient flows and temperatures to determine if there is a potential for the City to exceed the WLA in future years. While the details are not available yet on what an updated RPA will look like after the new TMDL is finalized, it is unlikely that the City’s WWTP’s temperatures or flowrates will increase in the next planning period to result in an 85% increase in thermal load from what was seen in the past five years.

After the final temperature TMDL is published, the City should ensure that the plant flowrates used in the RPA are reflective of the plant’s design flowrate or consistently observed flows during the WLA period. The 1.96 MGD flowrate used in the draft WLA is not representative of the plant’s typical flowrates; no flowrate in the WLA period has been near this level in the past five years (2018 – 2023). The maximum flowrate during the WLA period observed in the DMR review period was 0.8 MGD (corresponding with the outlier datapoint in Figure 3-3 in November 2021).

A more appropriate flowrate to use in an RPA would be the max month dry weather flow, or the design dry weather flow of an upgraded WWTP. It should be considered that when flows are over 1 MGD, it is likely because of I/I issues associated with a storm event. During storm events, it should be expected that effluent temperatures are lower, and the receiving stream are at higher flows than the 7Q10 low level used in determining the WLA.

3.1.4.2 Temperature: Thermal Plume Limitations

In addition to the thermal load requirements discussed in Section 3.1.4.1, the City must also comply with the requirements of OAR 340-041-0053(2)(d)(A) through (D) to minimize adverse effects to salmonids inside the effluent plume created by the outfall. There are four effects that must be evaluated based on the relevant OAR: impairment of spawning areas, acute impairment/instantaneous lethality, thermal shock, and migration blockage. A reasonable potential analysis spreadsheet was provided by DEQ to evaluate each of these.

OAR 340-041-0053(2)(d) Part A: Spawning Impairment

Based on interpretation of DEQ Figure 340B (provided in Figure 3-2), the physical discharge location is located 20 feet upstream of Dexter Dam, which is in the “No Designation” area for spawning. However, in the past the discharge location has been treated like a river discharge due to its proximity to the dam’s penstocks. This means that the discharge should be evaluated right at the effluent of the penstocks, which is within the Salmon and Steelhead spawning use designation zone of DEQ Figure 340B. Therefore, the evaluation in DEQ’s Thermal Plume part A spreadsheet was performed. This evaluation requires a dilution factor, an ambient temperature, a maximum effluent temperature, and the applicable temperature criterion.

Based on the Draft Temperature TMDL Technical Support Document, the applicable 7Q10 flow for Lowell’s discharge is 998.4 cfs, with the source defined by DEQ as: *“USGS Gage 14150000. Assumed flow in the penstock as measured by USGS gage defined flow available for mixing.”* This ambient flow rate is less than the value ACE has stated as the minimum flow through the penstocks and what was used by DEQ in previous mixing zone analyses (1200 cfs, See Permit Fact Sheet in Appendix A). In the interest of being consistent with the new temperature TMDL, the 998.4 cfs value was used in the analysis.

The facility’s design dry weather flowrate of 0.22 MGD (which is greater than projected the 2045 design average dry weather flowrate, see Section 3.3.2) was used to calculate a dilution factor using the following equation: $(Q_r \times 0.05)/Q_e + 1$. This assumes that the effluent is fully mixed with 5% of the flow prior to the spawning location downstream of the dam, which is very conservative considering the turbulence experienced within the penstocks and turbines. The resulting dilution factor is 148. This is less than the value of 177 used by DEQ in the previous mixing zone analysis (Appendix A) due to the lower ambient flowrate.

The applicable spawning criterion temperature is 13°C. Rather than a max seven-day arithmetic mean, a more conservative maximum recorded temperature from the City's effluent during the 2018-2022 review period was used (24.4°C from June 28, 2021). The result was a temperature increase of 0.1°C from the criteria of 13°C and No Reasonable Potential. The results of the RPA are summarized in Table 3-4.

Table 3-4: Results of Thermal Plume RPA, Part A

OAR 340-041-0053(2)(d)(A): Active Spawning Area Impairment 13.0 deg C at location of active spawning area			
			Data Metric/Source
Dilution at Spawning Area =	148		Ambient Flow = 5% * 998.4 cfs Effluent Flow = 0.22 MGD
Ambient Temperature =	13	ºC	Applicable Criterion
Max. 7dAM Effluent Temperature =	24.4	ºC	Max Recorded Temperature (6/28/2021)
Applicable Temperature Criterion =	13	ºC	
DT at Spawning Area=	**0.1**	**ºC**	**No Reasonable Potential**
Temp. at Spawning Area=	**13.1**	**ºC**	

OAR 340-041-0053(2)(d) Part B: Acute Impairment

Acute impairment or instantaneous lethality is prevented or minimized by limiting potential fish exposure to temperatures of 32.0°C (89.6°F) or more to less than 2 seconds). 32°C is well above the maximum temperature recorded from the City's effluent data (24.4°C). There is little possibility of acute impairment.

OAR 340-041-0053(2)(d) Part C: Thermal Shock

Thermal shock caused by a sudden increase in water temperature is prevented or minimized by limiting potential fish exposure to temperatures of 25.0°C (77.0°F) or more to less than 5 percent of the cross section of 100 percent of the 7Q10 low flow of the water body. Since the maximum effluent temperature is below 25°C, thermal shock caused by the discharge is prevented or minimized.

OAR 340-041-0053(2)(d) Part D: Migration Blockage

Migration blockage is prevented or minimized by limiting potential fish exposure to temperatures of 21.0°C (69.8°F) or more to less than 25 percent of the cross section of 100 percent of the 7Q10 low flow of the water body. DEQ's RPA analysis requires a 7Q10 ambient flow, an ambient temperature, an effluent flowrate, and a maximum effluent temperature. The 7Q10 flow was set as the same as discussed in Part A, the ambient temperature was set as the maximum allowable input of 21°C, and the effluent flow and temperature was set as the same as Part A. A more conservative 5% of stream flow was used instead of 25%, consistent with previous permit evaluations (See Permit Fact Sheet in Appendix A). A “No Reasonable Potential” result was obtained; see Table 3-5 for a summary of the RPA.

Table 3-5: Results of Thermal Plume RPA, Part D

OAR 340-041-0053(2)(d)(D): Migration Blockage 21 deg C at 25% of the stream cross section			
			Data Metric/Source
7Q10 =	998.4	cfs	Draft Willamette Subbasins Temperature TMDL
Ambient Temperature =	21	ºC	Maximum allowed input temperature
Effluent Flow =	0.22	mgd	Design ADWF (Lowell WWTP O&M Manual)
Max 7dAM Effluent Temperature =	24.4	ºC	Max Recorded Temperature (6/28/2021)
5% of 7Q10 =	49.9	cfs	
5% dilution =	148	dilution = (Qr*0.25)/Qe + 1	
Temperature at 5% cross section =	**21.0**	**ºC**	**No Reasonable Potential**
ΔT at 5% Stream Flow=	**0.0**	**ºC**	

**Note, analysis was performed at 5% of stream flow instead of 25% to be consistent with the City’s most recent permit fact sheet*

3.1.4.3 Ammonia

Considering that Dexter Reservoir is listed as impaired for harmful algal blooms, it would be reasonable to prepare for future NPDES permit cycles to impose nutrient limits, likely in the form of an ammonia nitrogen limit. Ammonia is also a toxic substance, and effluent loads of ammonia cannot cause the receiving water body to exceed water quality criteria outside of an established mixing zone. Water quality criteria for ammonia is dependent on pH and temperature.

As the City grows over the next planning period, it is likely that DEQ will require testing of ammonia in the plant effluent, and ammonia, pH, alkalinity, and temperature in the receiving water to support an ammonia RPA. DEQ has indicated that they will likely require data for this analysis in approximately 2026. While this data is not available to make a useful estimate on what a future limit might be, it is recommended to implement a treatment alternative that can support nitrification if an ammonia limit is imposed in future permit revisions.

3.1.4.4 Biocriteria

Addressing concerns associated with biocriteria impairment could take many forms when a TMDL is developed, but may include addressing issues related to temperature, bacteria, pH, and nutrient loading to the waterbody. A biocriteria TMDL for the Middle Fork Willamette subbasin is not listed in the 2022 Integrated Report’s TMDL priority list. Therefore, it is unlikely that one will be developed prior to April 2030.

3.2 Aging Infrastructure

Multiple issues that arise for wastewater collection and treatment systems are the result of infrastructure age. Infrastructure aging can lead to a decrease in treatment efficacy and increases in operations and maintenance (O&M) costs. Table 3-6 presents estimates of wastewater infrastructure useful lifespans.

Table 3-6: Typical Useful Life of Common Wastewater Infrastructure

Component	Useful Life (Years)
Collections	80-100
Concrete Structures	50
Mechanical and Electrical	15-25
Force Mains	25
Information obtained from EPA-816-R-02-020	

The following sections summarize age-related deficiencies in the wastewater facilities. It should be noted that detailed structural evaluations were not completed during the development of this planning document. The City should budget for structural evaluations during any design phase for improvement projects that involve significant structures as defined by ORS 672.002 to 672.325. The condition assessments in the following subsections are based on preliminary site inspections, a comparison of the date of construction to the theoretical useful life, and the authoring engineer's judgement. The operating conditions and maintenance history can significantly impact the actual useful life of a structure.

3.2.1 Collection System

As discussed in Section 2.1, the collection system was originally constructed in the 1950s. Since that time, the system has expanded and sections of piping have been replaced, resulting in the current collection system which consists of multiple pipe sizes, materials, and ages. The pipe original to the system bult in the 1950s is expected to near the useful life of 80-100 years towards the end of this planning period. The City should rehabilitate or replace pipes as determined by system evaluations, such as smoke testing, flow testing, and/or CCTV analysis. A comprehensive I/I evaluation was conducted as part of this facility planning process, and the results are provided in Appendix D.

3.2.1.1 Collection System Inflow and Infiltration

Many communities in Oregon struggle with the issue of inflow and infiltration (I/I) within their wastewater collection systems. Inflow and infiltration are defined as follows:

- Inflow: Flows that enter the collection system through above ground paths. Inflow is often related to building downspouts being improperly connected to sanitary sewer service laterals, cross connections with storm drain systems that have not been separated, water flowing over manholes and entering in through the openings in the lids, or area drains being connected to the sewer system, and other surface water sources.
- Infiltration: Flows that enter the collection system through underground paths. Infiltration can be caused by high groundwater levels, rain-induced groundwater, and other sources. Infiltration flows make their way into the collection system through cracks in pipe, open or offset pipe joints, broken piping sections, leaks in manholes, and other below-grade openings in the collection system.

When combined, I/I can result in a significant increase in flow rates during the winter, particularly during prolonged storm events.

Based on EPA I/I guidance documents, the determination of "excessive" or "non-excessive" infiltration is based on a comparison of the highest average daily flow rate recorded during high groundwater conditions relative to benchmark flow volumes. Average daily flowrates during periods of high ground water exceeding 120 gpcd are indicative of excessive infiltration, and average daily flowrates during periods of significant rainfall that exceed 275 gpcd are indicative of excessive inflow (EPA, I/I). Due to western Oregon groundwater remaining low between June and December, the excessive infiltration analysis only considers the months of January through May.

Average per capita flows when precipitation was minimal during the high groundwater period have ranged from 60 to 145 gpcd, with a total average of 80 gpcd, in the past five years. While the total average is below the benchmark of 120 gpcd, there were exceedances of the benchmark within the period of available data. This indicates that infiltration of groundwater could be an issue in the City's collection system but may not be excessive compared to other areas in the Pacific Northwest. In contrast, the average flow during heavy periods of precipitation (antecedent five days with cumulative precipitation over 1 inch) was 501 gpcd. This is well above the 275 gpcd benchmark, indicating the City experiences excessive levels of stormwater inflow.

The I/I analysis resulted in the recommendation of rehabilitating multiple manholes, and CCTV surveillance of 6,300 linear feet of pipe. Most of the pipe recommended to CCTV is original to the system built in 1950s, so it is likely that this pipe has deteriorated over the years. Based on the results of CCTV analysis, a plan should be developed to replace segments of the pipe.

3.2.2 Wastewater Treatment System

Site visits and discussions with WWTP operations staff were used to identify several issues with the current treatment facilities.

3.2.3 Headworks

The headworks are generally in good condition. The Parshall flume at the headworks does not have a transducer in operation, so plant influent flows are not being recorded currently. Electrical and mechanical components, such as the bearings of the influent screen and the influent sampler will exceed their useful life during the planning period. Replacement of these components should be completed.

3.2.4 Primary Clarifier

The mechanisms and piping in the primary clarifier are in good condition. The sump pumps will exceed their useful life during the planning period and should be planned to be replaced at that time.

The structure itself appears to be in good condition. There are a couple of small cracks near the bottom of the wall separating the two cells (Figure 3-4). This is not an immediate concern, but the City should observe these cracks to make sure they aren't increasing in size every year and resulting in the empty cell being filled with water.

Figure 3-4: Small Cracks in Clarifier Wall

3.2.5 Secondary Treatment

On July 1, 2021, DEQ sent a warning letter to the City detailing nineteen violations of the City's NPDES permit. Multiple of these were violations of the City's dry-season monthly average BOD_5 concentration limit of 10 mg/L (Table 3-7), violation of the dry-season weekly average limit of 15 mg/L, and one violation of the 85% minimum BOD_5 removal efficiency. This warning letter was sent with the indication that future violations of the BOD_5 limits may be inferred to DEQ's Office of Compliance and Enforcement for corrective action, including civil penalties for each day of violation. These violations have not been ongoing in recent years (2022 – 2024), which the operator credits to more careful control of RAS flows and reducing slug loads from the Alder Street lift station by operating at lower flowrates during the summer.

The majority of BOD_5 is removed via biological treatment. The previous facilities plan recommended the addition of tertiary filtration to the WWTP to meet projected effluent BOD_5 and TSS limits. However, the City has been hesitant on the addition of filtration units because the process would require extensive pumping, filter replacements, and associated operations costs. Upgrading the biological treatment system to handle current and future limits may be a more cost-effective alternative for the City.

Table 3-7: Violations of the City's Monthly Average BOD5 Limit

Monthly Average BOD5 Effluent Concentration (mg/L):					
	2018	**2019**	**2020**	**2021**	**2022**
January	3.2	3.0	4.4	6.1	3.4
February	3.0	2.0	2.8	8.1	5.5
March	2.0	2.3	4.1	4.4	3.7
April	2.8	2.3	6.6	11.2	6.3
May	4.8	3.2	4.3	20.0	4.5
June	3.8	3.8	2.6	15.6	3.7
July	7.5	8.2	8.5	14.1	3.2
August	10.4	10.3	6.8	4.3	5.3
September	11.3	7.0	10.4	6.5	5.6
October	13.8	8.8	7.8	2.6	7.0
November	11.8	12.9	11.6	4.9	5.3
December	3.8	6.9	2.9	3.2	2.7
Monthly Average BOD5 Effluent Loading (ppd):					
	2018	**2019**	**2020**	**2021**	**2022**
January	7.4	3.4	13.0	15.8	7.1
February	3.8	5.8	3.7	16.1	4.6
March	2.8	4.0	3.7	5.6	10.8
April	2.8	4.5	8.2	6.2	9.8
May	3.0	2.2	2.8	14.9	4.7
June	1.5	2.3	1.3	9.3	3.5
July	2.3	3.8	3.7	6.4	2.1
August	4.2	4.5	4.1	2.1	2.2
September	5.3	6.0	7.1	2.9	2.7
October	7.4	4.5	5.0	1.7	3.2
November	9.5	7.5	17.4	12.5	6.3
December	7.0	7.5	5.5	5.7	3.5

The existing solids contact aeration basin lacks control capabilities. Operators have indicated that the small blower for the aeration basin is inadequate for summer flows; when the blower is in operation, solids are observed to settle in the basin. It has also been observed that the digester decant cycle results in sloughing in the trickling filter. The decant line discharges to the head of the solids contact channel where the trickling filter recirculation pump is located. It is possible that the digester decant is being recirculated to the trickling filter via proximity.

Currently, operators have to harness-and-fall-restrain in order to deploy the decant pump in the secondary clarifier. Ideally, the catwalk should extend across the basin with access stairs from both sides.

The existing trickling filter/solids contact system is not optimal for the high variation in flows experienced by the City, especially given the lack of redundancy with only one filter and one clarifier. During high flow events, colder temperatures inhibit biological activity, requiring the relatively small solids contact basin to act as the primary treatment unit. During the dry season, the trickling filter must be over-recirculated (over a factor of 4) to keep the distribution arms spinning at a rate to keep the entire filter area wetted. Furthermore, with only one secondary clarifier, the facility is out of compliance with DEQ reliability and redundancy criteria. The existing clarifier was sized for peak flows, which makes it oversized for typical dry season flows. Oversized secondary clarifiers can result in settled activated sludge going septic and losing biological activity. This can cause the solids contact aeration basin to not perform to its design standards, even if routine process testing indicates that mixed liquor suspended solids concentrations are in an optimal range.

At a minimum, a more appropriately sized redundant secondary clarifier should be added so that the existing clarifier can be taken offline and properly maintained. While I/I reduction could help the WWTP perform in mitigating issues caused by extreme flow variations, it will likely prove to be more cost-effective for the City to transition to a treatment configuration that is more robust and with a higher degree of operational flexibility.

3.2.6 Disinfection

As discussed in Section 2.4.7, the existing contact chamber has an overestimated contact time and is not in compliance with DEQ redundancy and reliability requirements. Furthermore, the structure itself was constructed in the original WWTP and will approach the 100-year design life for a concrete structure in the planning period. The lack of controls or flow-pacing for chlorine and thiosulfate dosing is also a concern. At worst, this puts the City at risk of exceeding the chlorine residual limit of 0.5 mg/L. At best, it results in overdosing of chemicals and the City wasting money.

3.2.7 Solids Handling

Electricity expenditures for the existing aerobic digester blower are over $1,500 per month, and the digester cells are not able to be isolated with the existing blower configuration. After discussion with vendors, a turbine-style positive displacement blower would be more appropriate to meet the mixing and air requirements for the aerobic digester than the existing rotary lobe. A new blower should be able to operate just one of the cells at a time (with the other being empty) and save the City considerable electricity costs. A turbine-style blower would be appropriate for the digester given the turndown flexibility.

The existing underdrain piping in one of the solids drying beds was observed to be exposed and broken in multiple places. A full rehabilitation of the underdrain system is recommended for at least a short-term fix. The operators have expressed concerns about the difficulty in removing sludge from the drying beds because the plastic liner is completely exposed around the drying bed rim, and the side slopes of the beds are too steep to drive in a tractor without providing gravel fill before clearing the beds. An alternative to modify the drying beds should be evaluated to add runner-walls to help guide equipment during bed clearing and to improve access of equipment into the beds.

3.3 Reasonable Growth

The planning period for this document will end in 2045. During this period, the population of the City is expected to grow significantly. The anticipated population growth will increase the total wastewater volume and pollutant load that must be treated at the WWTP. To estimate future wastewater flow rates and pollutant loading rates during the planning period, the existing flow and loading rates were scaled with the projected population growth rates.

3.3.1 Current Flow Rates

An evaluation of flowrates via the DEQ guidance document "*Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon*" was performed using flowrates recorded in the facility's DMRs and precipitation data. The data used in this analysis is documented in Appendix E. A summary of this analysis is provided in the following subsections.

3.3.1.1 Characteristic Flowrate Definitions

The following terms are used to describe characteristic flowrates:

- Dry Weather Period: Defined as the period when the precipitation and streamflow are low. This period is defined as May 1 through October 31.
- Wet Weather Period: Defined as the period when precipitation and streamflow are low. This period is defined as November 1 through April 30.
- Average Annual Flow (AAF) or Average Daily Flow (ADF): Total wastewater flow for a 12-month period, from January 1 through December 31, divided by the total number of days for which data was available (between 363 and 366 days).
- Base Sewerage: Average daily flow for the period between June 1 and September 31. This is used as a basis to evaluate inflow and infiltration (I/I).
- Average Dry Weather Flow (ADWF): Total wastewater flow for the dry-weather period divided by the number of days in the period for which data was available.
- Maximum Month Dry Weather Flow (MMDWF): Total wastewater flow for the month with the highest flow during the dry-weather period, divided by the number of days in the month.
- Average Wet Weather Flow (AWWF): Total wastewater flow for the wet-weather period divided by the number of days in the period for which data was available.
- Maximum Month Wet Weather Flow (MMWWF): Total wastewater flow for the month with the highest flow during the wet-weather period, divided by the number of days in the month.
- Peak Day Average Flow (PDAF): Total flow for the day with the highest wastewater flow during the year.
- Peak Hour Flow (PHF): The maximum flow observed during the peak day.

The following terms will be used in the statistical analysis of flowrates:

- Ten-year Maximum Month Dry Weather Flow ($MMDWF_{10}$): The monthly average dry weather flow with a 10% probability of occurrence.
- Five-year Maximum Month Wet Weather Flow ($MMWWF_5$): The monthly average wet weather flow with a 20% probability of occurrence.
- Five-year Peak Day Average Flow ($PDAF_5$): The peak day average flow associated with a five-year storm event.
- Max Week Flow (MWF): The average weekly flow during a five-year storm event.

3.3.1.2 Max Month Flowrates

Monthly average flows were plotted against monthly cumulative precipitation (Figure 3-5). A linear fit of the data was created and flowrates were estimated at precipitation values of 6.08 inches and 8.69 inches. These precipitations correspond to the 90th percentile May precipitation and the 80th percentile January precipitations at Lookout Point Dam respectively (NOAA Climatography of the United State Number 20, 2001, Appendix F). The flowrates corresponding to these precipitations are equal to $MMDWF_{10}$ (occurs once every 10 years) and $MMWWF_{5}$ (once in five years) respectively. Data was limited to the most recent year (2023) to avoid population growth from skewing the correlation analysis. The 5-year high of January 2020 was plotted for reference, but not included in the correlation.

3.3.1.3 Peak Day Flow

Daily flowrates were plotted against daily precipitation totals for days where the following criteria were met: antecedent five days prior to the record date had over 1” of cumulative rainfall, and the event occurred during the high groundwater period (January – March). Based on a linear fit of this dataset, the flowrate associated with the precipitation corresponding to a 5-year, 24-hour storm (4.25”, NOAA Atlas 2 Volume X, Appendix G) was calculated at 1.01 MGD as shown in Figure 3-6.

3.3.1.4 Peak Hourly Flow

Peak hourly flow (PHF) was estimated assuming a probability of occurrence once every 8,760 hours (0.011%). The probability of occurrence associated with the other flows shown on Figure 3-7 are as follows: peak monthly (MMWWF5) occurs once every 12 months (8.3%), max weekly (MWF) occurs once every 52 weeks (1.9%), and peak daily (PDAF5) occurs once every 365 days (0.27%).

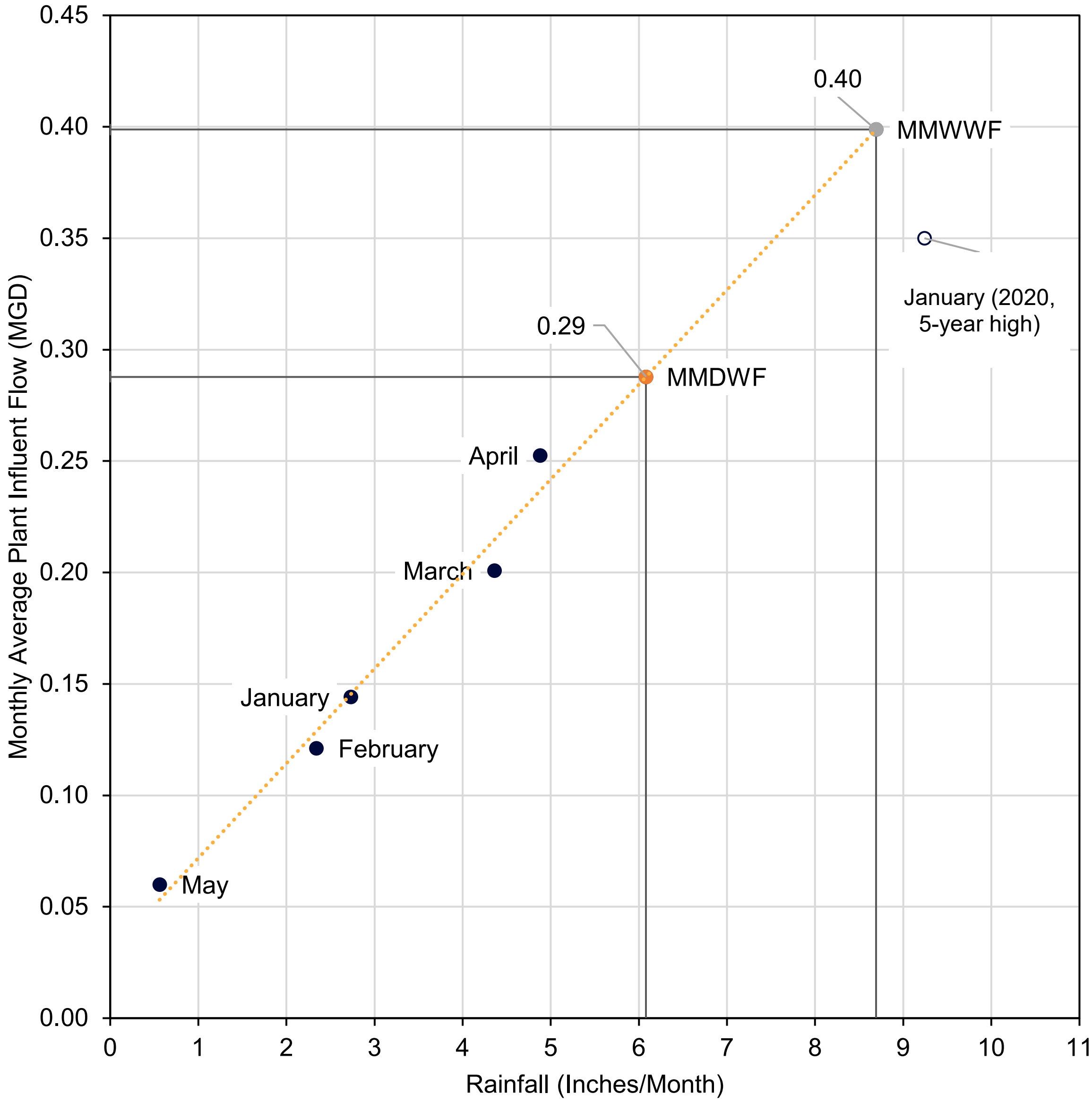


Figure 3-5: DEQ Graph #1, Monthly Average Flowrates and Monthly Precipitation Correlations

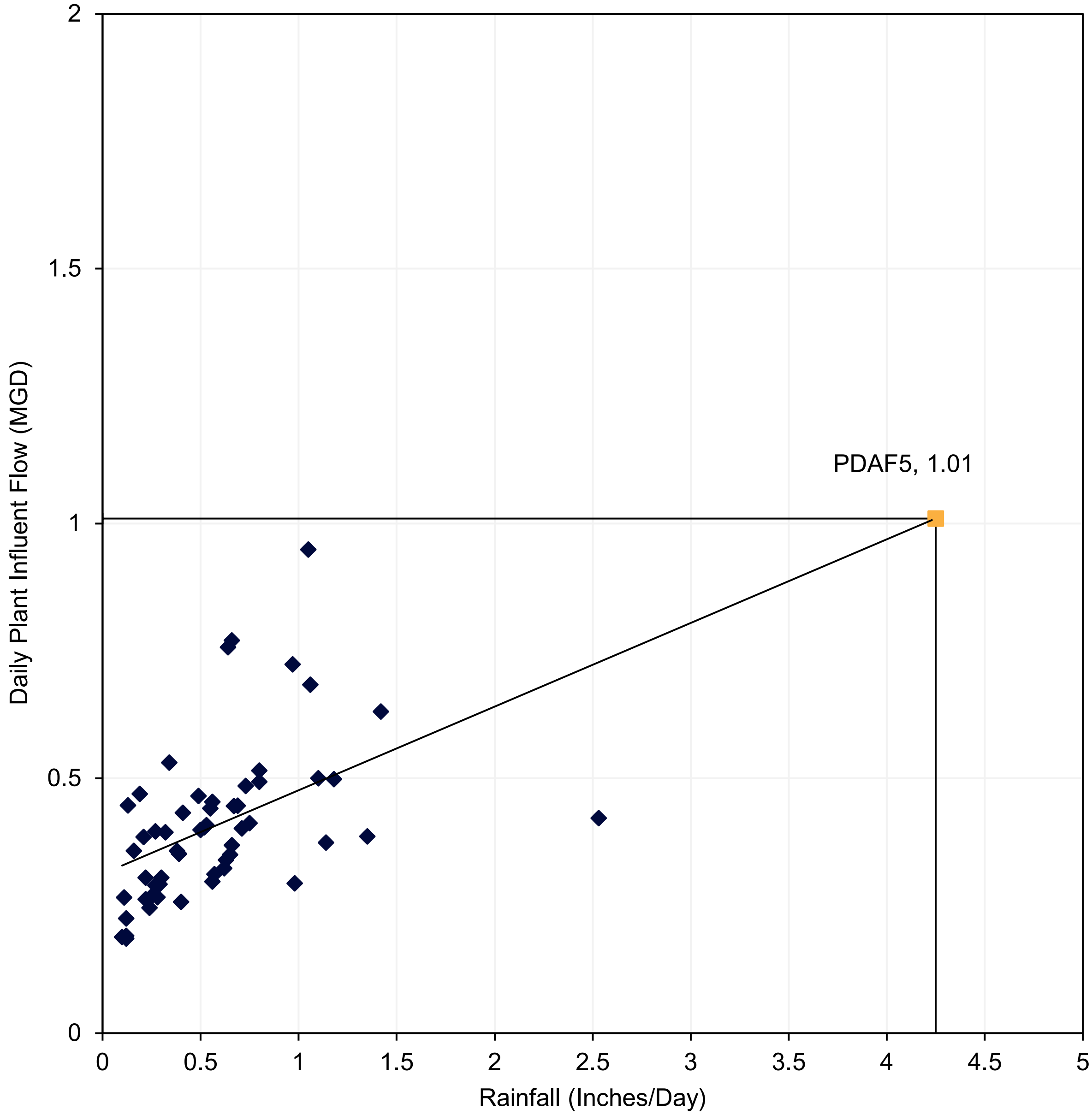


Figure 3-6: DEQ Graph #2, Daily Average Flow correlated to Daily Precipitation

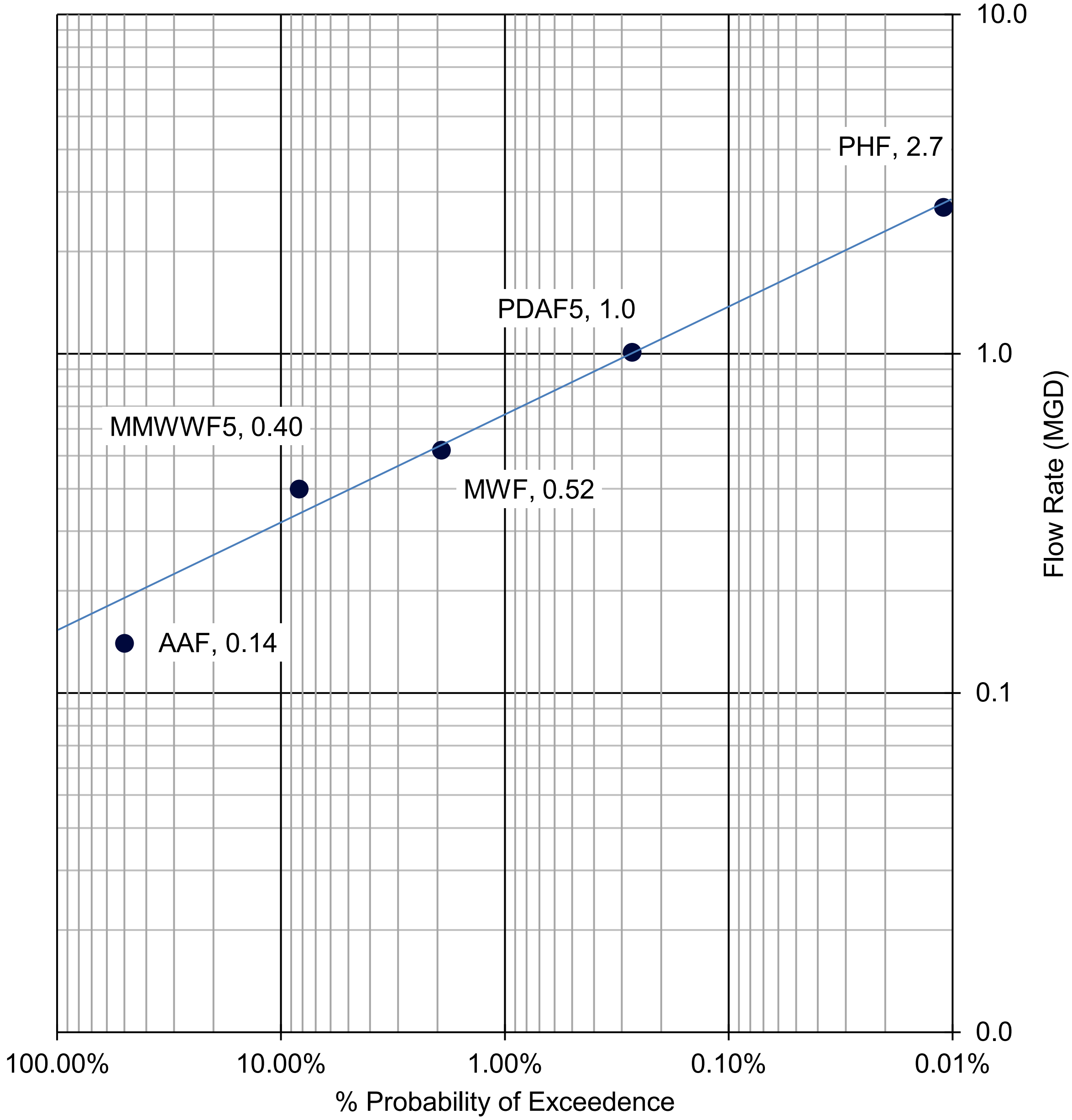


Figure 3-7: DEQ Graph #3, Flow Projections as a function of Exceedance Probability

3.3.1.5 Evaluation of Estimated Current Flows

In addition to the flowrates estimated via DEQ methods, the ADWF and AWWF were estimated by evaluation of the facility's DMRs from the previous 5-year period (2018 – 2023). The AWWF was determined to be 0.20 MGD and ADWF was determined as 0.08 MGD. Together, these average to the AAF of 0.14 MGD. The DEQ estimated flows were compared to all flows recorded in the past five years in Figure 3-8. The estimated flows are generally in agreement with recorded flows. The PDAF estimated using DEQ method seems to underestimate the maximum day average flow by one to four hundred thousand gallons per day. Therefore, the estimated flow was adjusted to the value of the highest observed flow of 1.4 MGD to be conservative. A summary of all current flows are provided in Table 3-8.

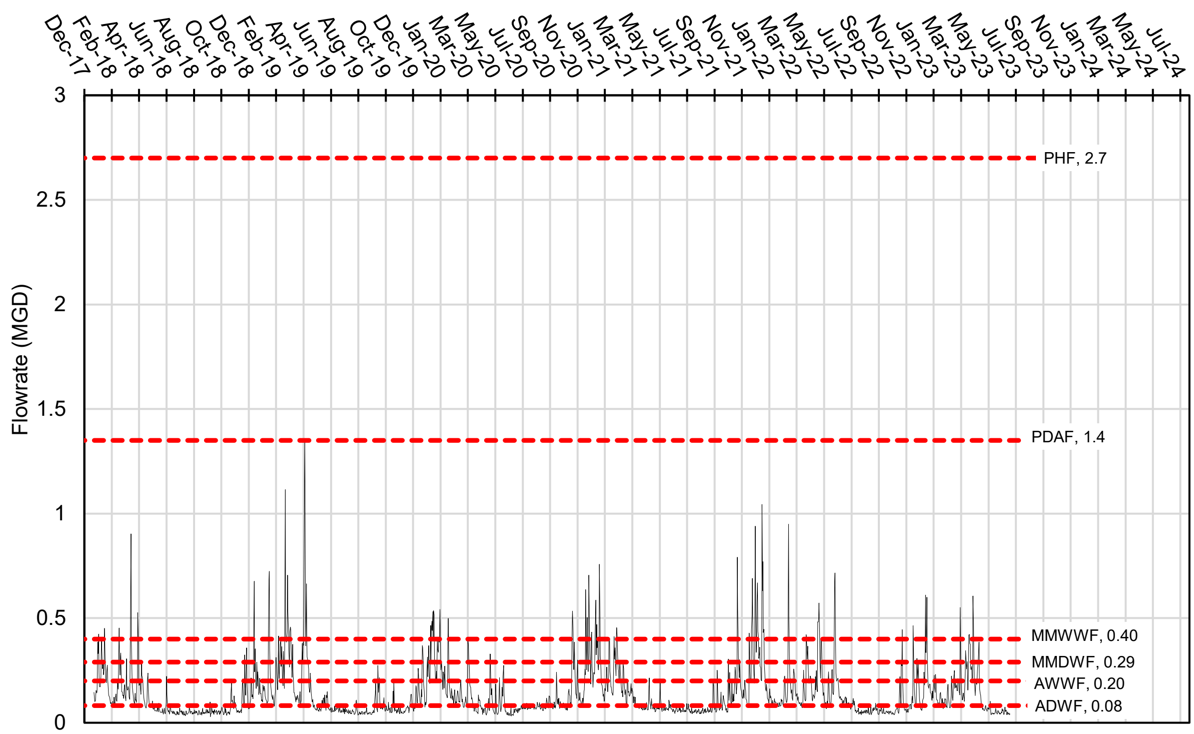


Figure 3-8: Comparison of Estimated Flows with Recorded Flows from Past Five Years

Table 3-8: Summary of Current Flow Estimates

	2023 Flow Estimates (MGD) Population = 1250	Per Capita Flow (GPCD)	Evaluation Method
Base Sewerage	0.08	62	Average between 6/1 and 9/31
Average Dry Weather Flow (ADWF)	0.08	66	Average Flow between 5/1 and 10/31
Average Wet Weather Flow (AWWF)	0.20	158	Average Flow between 11/1 and 4/30
Maximum Monthly Average Dry-Weather Flow (MMDWF)	0.29	230	DEQ Graph 1
Maximum Monthly Average Wet-Weather Flow (MMWWF)	0.40	319	DEQ Graph 1
Peak Daily Average Flow (PDAF)	1.40	1120	Highest Daily Average Flow in past 5-years
Peak Hourly Flow (PHF)	2.70	2160	DEQ Graph 3

3.3.2 Projected Flowrates

To estimate 2045 flowrates, the City's base sewerage (defined as average flow from June 1 to September 1) of 0.08 MGD was scaled commensurately with the expected population growth. A unit per capita sewerage of 62 gal/capita/day was calculated by dividing the base sewerage by the current population of 1,250.

Peaking factors of 2 and 5, based on the water demand peaking factors from the City's Water Master Plan, were used to scale the base sewerage for peak day and peak hour flows respectively. A peaking factor of 1.4, determined by the quotient of average wet weather and average annual flow, was applied to average wet weather and max month sewerages. The expected increase in base sewerage was then added to each of the characteristic flows. A summary of existing plant flowrates and projected 2045 flowrates is provided in Table 3-9.

A brief evaluation of how enrollment growth at the Lowell School District may affect wet weather flows was performed since the school is the largest non-residential discharger to the City's wastewater system during the wet season. The Lowell School District reported an enrollment of 889 students in 2020, and the National Center for Education Statistics reported an eleven-year growth rate of 1% for total public school enrollment between 2010 and 2021 in Oregon. This growth rate was extrapolated for an expected enrollment of 909 students in 2045. A unit flow of 19 gal/student/day was applied to the expected student enrollment growth for an additional flow of 388 gallons per day in 2045. This is less than 2% of the projected increase in AWWF.

As discussed previously, the contributions of I/I volumes are considerable in the City's collection system. The flow projections in Table 3-9 are made under the assumption that I/I volumes will not increase throughout the planning period. This assumption is valid if the City makes efforts to repair the identified sources of I/I from this planning effort, maintains a program to identify and repair I/I sources, and ensures new developments and additions to the collection system are not adding new I/I sources.

Table 3-9: Summary of Projected Flow Rates

	2023 Flows (MGD)	Sewerage Peaking Factor	Per Capita Sewerage (GPCD)	Estimated I/I Volume (MGD)	2045 Flows (MGD)	% I/I
Base Sewerage	0.08	1.0	62	0.00	**0.10**	0%
Average Dry Weather Flow (ADWF)	0.08	1.0	62	0.00	**0.10**	4%
Average Wet Weather Flow (AWWF)	0.20	1.4	86	0.09	**0.23**	39%
Maximum Monthly Average Dry-Weather Flow (MMDWF)	0.29	1.4	86	0.18	**0.32**	56%
Maximum Monthly Average Wet-Weather Flow (MMWWF)	0.40	1.4	86	0.29	**0.43**	68%
Peak Daily Average Flow (PDAF)	1.4	2.0	123	1.25	**1.5**	86%
Peak Hourly Flow (PHF)	2.7	5.0	310	2.31	**2.8**	82%

3.3.3 Pollutant Load Projections

A thorough review of the City's WWTP discharge monitoring reports (DMRs) from 2018 to 2023 was conducted to project influent pollutant loads for the 2045 design year. A full summary of the data from DMRs is provided in Appendix E.

The City's current NPDES permit requires monitoring influent and effluent BOD_5 and TSS for treatment compliance. Influent concentrations and flowrates for BOD_5 and TSS were used to calculate average, max month, and peak day pollutant loads. These loads were divided by Lowell's 2023 population of 1,250 to calculate unit loadings. These unit loadings were then multiplied by expected population in 2045 to calculate design year loads.

An estimate of ammonia loadings was made using a concentration of 20 mg/L as nitrogen, typical of medium strength domestic wastewater (Metcalf and Eddy, 5th edition). A peaking factor of 1.85, calculated from BOD_5 data, was applied to estimate max month ammonia loads. A summary of current and design year loads is provided in Table 3-10.

Table 3-10: Current and Projected Pollutant Loads

	lb/day 2023 (Pop. 1,250)	lb/capita/day	lb/day 2045 (Pop. 1,618)
Five-Day Biochemical Oxygen Demand (BOD_5)			
Annual Average	114	0.091	148
Max Month	213	0.170	276
Peak Day	423	0.338	548
Total Suspended Solids (TSS)			
Annual Average	129	0.103	167
Max Month	235	0.188	304
Peak Day	502	0.402	650
Ammonia			
Annual Average	14	0.011	18
Max Month	25	0.020	33

3.3.4 Collection System Capacity

3.3.4.1 Alder Street Pump Station Design Flows

Based on the number and zoning type of properties connected to the Alder Street Pump Station sewerage basin, this lift station serves approximately 147 EDUs. Using the Lane County average of 2.3 people per EDU and the unit flow of 62 gal/capita/day, this results in a base sewerage of 0.02 MGD. This is approximately 25% of the City's base sewerage. This is reasonable, since the 2001 facilities plan estimated 19% of the City's flow was sourced from the pump station, and since that plan was published a sizable collection system expansion was added on North Shore Drive northeast of the Alder Street Pump Station. Assuming that I/I is constant throughout the City, a constant ratio of 0.25 was used to determine flows to the lift station relative to the City's projected 2045 average flow and PHF. These flows are presented in Table 3-11.

Table 3-11: Alder Street Pump Station Current and Projected Flows

	Gallons/Minute	Million Gallons/Day
2023 Average Flow	24	0.04
2023 Peak Flow	469	0.68
2045 Average Flow	29	0.04
2045 Peak Flow	489	0.70

There are two submersible pumps each with a 350 GPM capacity in the Alder Street Pump Station. The total capacity of the station (700 GPM) is nominally enough to handle these flows, however, DEQ reliability standards require that the firm capacity of the pump station be sized for peak flow. Firm capacity is defined as the capacity of the pump station with the largest pump out of service. Therefore, the existing lift station's firm capacity is deficient to the projected 2045 flow by 139 gallons per minute. There has also been an overflow at the lift station relatively recently (2021), which may indicate the pumps in the lift station are not performing to their design criteria. Since these pumps are past the typical 20-year design life, the City should plan to upgrade the lift station pumps relatively soon in the planning period. Furthermore, multiple I/I sources were identified in the sewershed of the lift station during the I/I evaluation. Rehabilitating these manholes and pipes should be prioritized since they present the most risk for unpermitted discharge from the lift station overflow. A map of the lift station sewershed and I/I sources identified is presented in Figure 3-9.

3.3.4.2 Gravity Sewer Capacity

The City's gravity sewer pipes should be sized for the capacity associated with PHF. Using the Lane County average of 2.3 people per household, the City's current EDU total of 545, and an expected population growth of 368 people, approximately 705 EDUs are expected to be served by the City's wastewater facilities in the 2045 design year. At a projected 2045 PHF of 2.81 MGD, this equates to approximately 2.8 gpm per EDU. Assuming that flow is even distributed throughout the City, the number of properties upstream of a pipe in the collection system can be used to estimate wastewater flow during PHF, and this can be compared to the receiving pipe capacity as determined using Manning's equation. If the estimated wastewater flow is greater than the pipe capacity, then that pipe should be upsized.

The main collector truck along Moss Street that connects to the WWTP was upgraded in the early 2000s to a 15" PVC sewer main. This upgraded main has a capacity of about 2100 gpm. With a projected PHF of approximately 1950 gpm, this collector is large enough for the City's expected growth for the planning period.

There are two substantial bottlenecks in the collection system upstream of the main collector that are likely undersized for future growth. Both of these are 8" pipes that serve a significant number of properties in the City, one located in the alleyway between Moss Street and Cannon Avenue, and the other located at the west end of 1st street up to the Moss/Cannon alleyway. The location of these pipes and the areas they serve is shown in Figure 3-10. At a nominal slope of 0.3%, the capacity of an 8" line with an assumed Manning's coefficient of 0.015 is approximately 260 gpm. Estimating the flow for 90 properties with a unit flow of 2.8 gpm per EDU results in about 252 gpm. Both of these lines respectively serve over 90 properties. The areas served by these lines are also the most likely to experience new development, since most of the buildable land within the UGB is located in the northeast portion of the City.

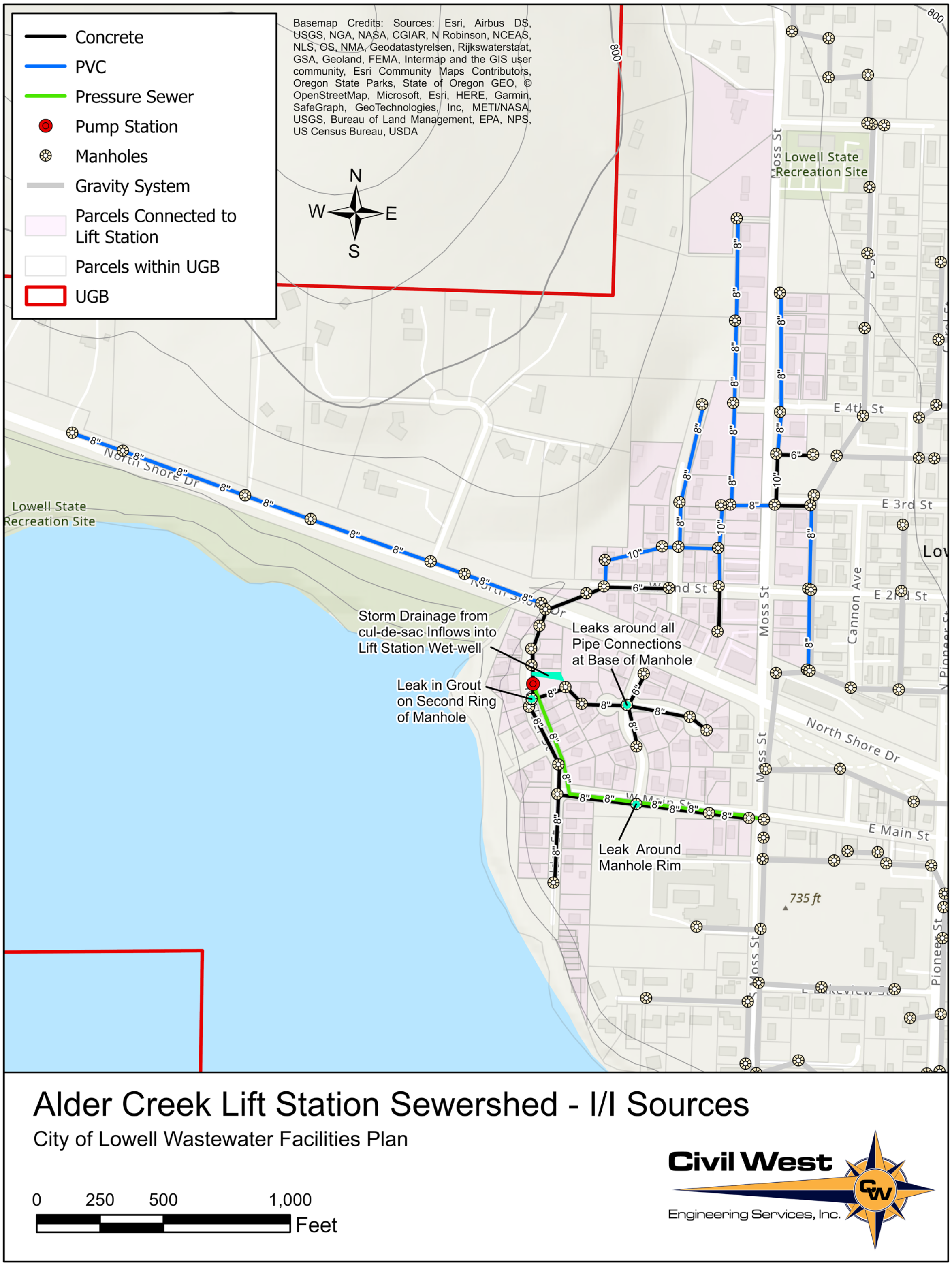


Figure 3-9: Alder Street Lift Station Sewershed and Identified I/I Sources

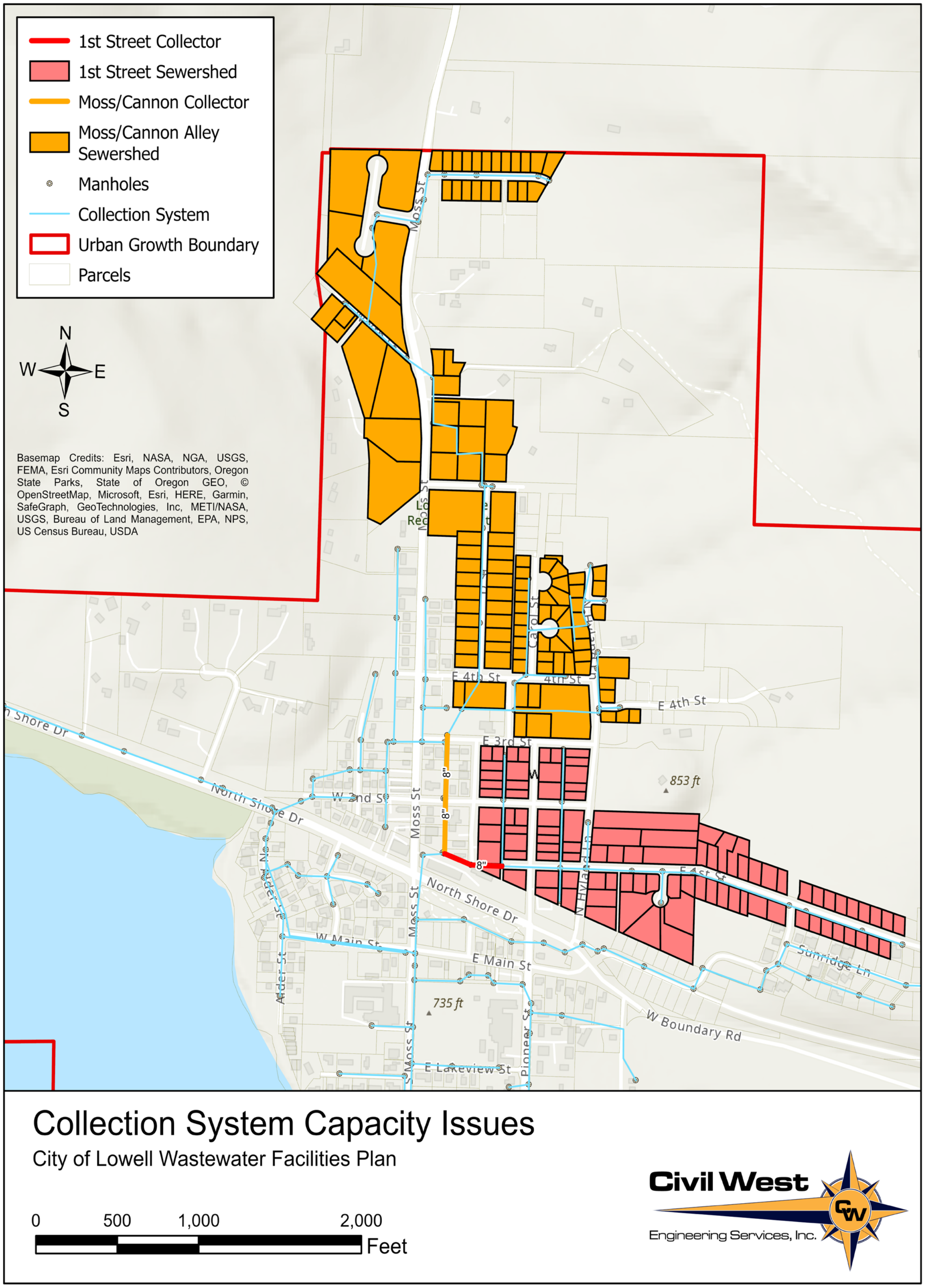


Figure 3-10: Collection System Pipes with Capacity Concerns

3.3.5 Treatment System Capacity

The WWTP's last major upgrades occurred in the early 2000s with replacement of trickling filter media, construction of the aerobic digesters, and installation of primary clarifiers. Another minor upgrade occurred in 2014 with a switch from gas chlorine to liquid chlorine for the disinfection system. A summary of design capacity data is provided to compare with projected 2045 flowrates and loads in Table 3-12. 1990 Facilities refer to facilities that were part of the 1990 upgrades: aeration basin, chlorine contact chamber, and the secondary clarifier. The 2001 Facilities include: headworks (channel and fine screen), primary clarifiers, trickling filter, aerobic digester, and sludge drying beds.

Table 3-12: Comparison of Current and Projected Design Parameters

Comparison of Current Facility Design Parameters with Projected Flows and Loads			
Parameter	**1990 Facilities**	**2001 Facilities**	**Projected 2045 Flows and Loads**
ADWF	0.15 MGD	0.22 MGD	0.10 MGD
MMDWF	--	0.44 MGD	0.32 MGD
MMWWF	--	0.76 MGD	0.43 MGD
PDAF	1.25 MGD	1.9 MGD	1.5 MGD
PHF	--	2.6 MGD	2.8 MGD
Average Day BOD	223 lb/day	448 lb/day	148 lb/day
Max Month BOD	335 lb/day	761 lb/day	276 lb/day
Average TSS	223 lb/day	377 lb/day	167 lb/day
Max Month TSS	335 lb/day	642 lb/day	304 lb/day

In theory, most of the existing WWTP has excess capacity for the remainder of the planning period, except for the facilities that were last upgraded in 1990 (aeration basin and chlorine contact chamber). However, multiple BOD permit exceedances during storm events and the large PHF to ADWF ratio also indicates that the facilities struggle to meet the City's treatment goals during periods of excessive rain. Without parallel unit operations, the facility is unable to optimize treatment for the variable flow conditions.

This is most apparent with the biological treatment system. During the summer, the trickling filter is operated at high recirculation ratios (greater than 3) so that the hydraulic distributor arms rotate. This results in very low strength wastewater entering the aeration basin and "starving" the activated sludge, which makes the facility vulnerable to shock flows and loads during the rare dry season storm. In contrast, during the high groundwater period (January – March) the trickling filter is not recirculated at all. This requires the aeration basin to provide most of the biological treatment, which was sized for flows smaller than what the City regularly sees during the wet season.

It should be noted that the only projected flow that is larger than the results of the previous flow analysis (2001) is the peak hour flow. This is likely indicative of a worsening I/I situation as the collection system pipes age. Rehabilitation of the collection system should be a top priority for the next planning period. However, because the existing plant was constructed without redundancy, major reductions in I/I will significantly reduce the high wet season flows that the facility was designed for. This could make operations even more difficult by exacerbating the issues discussed above.

4 ALTERNATIVES CONSIDERED

The following issues should be addressed within the next planning period:

- Multiple sources of I/I were discovered in the sanitary sewer collection system;
- The Alder Street Lift Station does meet firm capacity requirements for current and projected peak flow events;
- Two significant sewersheds in the City have collector pipes that are undersized for future projected growth;
- Multiple recent BOD_5 and TSS violations of the City's NPDES permit indicate need for biological treatment system upgrades at the WWTP;
- The facility is not in compliance with State redundancy requirements with only one secondary clarifier;
- The biosolids aeration system is not optimized, costing the City unnecessary electricity and O&M expenditures;
- The underdrains of the sludge drying beds have been damaged over time by regular sludge removal, and;
- The existing disinfection system is undersized for current and 2045 design flows.

Several alternatives were considered to address these issues. This section presents a description of every alternative considered and a discussion of their technical feasibility. Each technically feasible alternative is discussed with respect to planning-level design criteria, cost estimates, environmental impacts, land requirements, and potential construction issues.

4.1 Basis for Cost Estimates

Itemized cost estimates for each technically feasible alternative considered in this section are provided in Appendix I. These cost estimates include capital costs, operations and maintenance costs, and salvage values. Capital costs are typically comprised of four components: construction cost, engineering cost, contingency, and administrative costs. Operations and maintenance costs consist of disposables (chemicals, oil, parts), labor costs, and electricity costs. Salvage values are estimated as the value of each tangible item (i.e., not including installation costs) at the end of the planning period after accounting for design life.

These cost estimates are preliminary and based on the level and detail of planning presented in this study. The goal of planning-level cost estimates is to establish a reasonably conservative budget and to allow fair cost comparisons of alternatives. As projects proceed, site-specific information becomes available, and these estimates should be updated.

4.1.1 Construction Costs

Estimated construction costs were based on construction bidding results from similar work, published cost guides, budget quotes obtained from equipment suppliers, and other construction cost experience. Construction costs are preliminary estimates for budgeting purposes.

Future changes in the cost of labor, equipment, and materials may justify comparable changes in the cost estimates presented herein. For this reason, common engineering practices usually tie the cost estimates to an index that varies in proportion to long-term changes in the national economy. The Engineering News Record (ENR) construction cost index (CCI) is most commonly used. This index is based on the value of 100 for the year 1913. Average values for the past 10 years are summarized in Table 4-1.

Table 4-1: ENR Construction Cost Index History

Year	Average CCI	% Change/Year
2010	8801	2.70%
2011	9070	3.06%
2012	9309	2.64%
2013	9547	2.55%
2014	9807	2.72%
2015	10036	2.34%
2016	10331	2.95%
2017	10681	3.39%
2018	11062	3.56%
2019	11281	1.98%
2020	11457	1.55%
2021	12149	6.04%
2022	13007	7.06%

4.1.2 Contingencies

A contingency factor equal to approximately twenty percent of the estimated construction cost was added to the construction cost estimate. In recognition that the cost estimates presented are based on conceptual planning, allowances must be made for variations in final quantities, bidding market conditions, adverse construction conditions, unanticipated specialized investigation and studies, and other difficulties which cannot be foreseen at this time but may tend to increase final costs. Upon completion of final design, the contingency can be reduced to 10%. A contingency of at least 10% should always be maintained going into a construction project to allow for variances in quantities of materials and unforeseen conditions.

4.1.3 Engineering and Technical Services

Engineering and technical services for major projects typically include surveying, preliminary and final design, preparation of contract/construction drawings and specifications, bidding services, construction management, inspection, start-up services, and the preparation of operation and maintenance manuals. Depending on the size and type of project, engineering costs may range from 18 to 25% of the contract cost when all the above services are provided. The lower percentage applies to large projects without complicated mechanical systems. The higher percentage applies to small or complicated projects.

Engineering costs for basic design and construction services presented in this section were estimated at 20% of the estimated total construction cost. Other engineering costs such as specialized geotechnical explorations, hydro-geologic studies, easement research and preparation, pre-design reports, and other services outside the normal basic services will typically be in addition to the basic engineering fees charged by firms. When it was suspected that a specific project in this report may need any special engineering services, an effort has been made to include additional budget costs for such needs. Specific efforts required for individual basic engineering tasks such as surveying, design, construction management, etc. vary widely depending on the type of project, scheduling and timeframes, level of service desired during construction, and other project/site-specific conditions however an approximate breakdown of the 20% engineering budget is as follows:

- Surveying and Data Collection – 0.5%
- Civil/Mechanical Design – 8%
- Electrical/Controls Design – 1.5%
- Bid Phase Services – 1%
- Construction Management – 4%
- Construction Observation (Inspection) – 5%

4.1.4 Administrative and Legal Services

An allowance of five percent (5%) of construction cost was added for legal and other project management services. This is intended to include internal project planning and budgeting, funding program management, interest on interim loan financing, legal review fees, advertising costs, wage rate monitoring, and other related expenses associated with the project that could be incurred.

4.1.5 Operations and Maintenance Costs

Operations and maintenance (O&M) costs were simplified to include the following:

- Electricity costs for major pieces of equipment were based on the rated horsepower of representative equipment, the anticipated equipment runtime, and an estimated market price for electricity.
- Chemical consumption costs were based on estimated consumption rates for the identified chemical.
- Fees for outside services (such as tipping fees for the landfilling of biosolids) were based on quoted prices.
- Staff hours were estimated using *The Northeast Guide for Estimating Staffing at Publicly and Privately Owned Wastewater Treatment Plants* prepared by the New England Interstate Water Pollution Control Commission (New England Interstate Water Pollution Control Commission, 2008). An hourly labor cost of $30 per hour was used as a base rate.
- Materials costs were estimated from anticipated lifespans and replacement costs for commonly replaced materials.

4.2 General Treatment Alternatives

The alternatives in this subsection describe and discuss the feasibility of general, overarching modifications to the City's wastewater treatment facilities.

4.2.1 Regionalization

Regionalization involves coordinating with nearby wastewater utilities to consolidate resources and provide treatment at a centralized location. The nearest cities to Lowell (Oakridge, Jasper, Springfield) are too small and/or too far away to be considered feasible for conveying the City's wastewater to a regional treatment facility. Any costs saved from capital investments to improve the existing WWTP would be dwarfed by conveyance costs and the costs to construct a new or upgrade a receiving facility.

The nearby unincorporated community of Dexter recently evaluated regionalization to convey septage to Lowell's treatment facility as part of a recent planning effort. Civil West assisted with this evaluation to provide preliminary cost estimates for their alternative analysis (Appendix I. The estimated cost for a storage basin and lift station to convey septage to the City's WWTP was estimated at $850,000 (2022$). The final results of Dexter's analysis were not made available to the City or Civil West at the time of this plan being finalized. Therefore, it was assumed that Dexter decided on a different alternative than conveying septage to the City's WWTP. The City is open to receiving septage from Dexter, provided the Dexter community is able to fund the necessary conveyance infrastructure.

4.2.2 New Treatment Plant

This alternative involves purchasing new property and constructing a new WWTP. This alternative is not necessary since the existing property is ideally located for the City's WWTP at the lowest elevation in the area. The collection system and outfall would have to be completely redone since there is no suitable property available near the existing site. If the City was to move the location of the WWTP, ownership of the existing property would revert to ACE as it is within their ownership rights associated with the reservoir. This alternative is not feasible compared to rehabilitation of the existing facilities.

4.2.3 Rehabilitate Existing Treatment Plant

Deficiencies in the existing facilities would be corrected and facilities expanded to accommodate design flows and loads. This alternative is the most feasible for the City compared to regionalization and constructing a new WWTP. Several alternatives to upgrade this existing facility are provided in the remainder of this section.

4.3 Headworks Improvements

The existing headworks system consists of a mechanical fine screen, a bypass channel with a manually cleaned bar rack, and a Parshall Flume for flow measurement. The design capacity of each unit is summarized in Table 4-2. The existing headworks system has the capacity to handle the projected peak flow events throughout the planning period, with the caveat that the manually cleaned bar rack will need to be used occasionally during intense rain events. Efforts to eliminate I/I sources in the system should reduce the peak flow events and prevent overwhelming of the City's resources. The following subsections discuss the alternatives of maintaining the existing headworks as-is throughout the planning period (the "no construction" alternative) and adding a parallel fine screen unit to reduce the facility's use of the bar rack.

Table 4-2: Capacity of Existing Headworks Units

Unit Operation	Capacity
Fine Screen	2.6 MGD
Bar Rack	2.6 MGD
Parshall Flume	3.3 MGD

4.3.1 "No Construction" – Optimize Existing Facilities

4.3.1.1 Description

The existing headworks system would continue to be used throughout the planning period without any major changes aside from periodically replacing short-lived assets. This would result in more frequent use of the bypass channel as the City expands and flows increase.

4.3.1.2 Design Criteria

The existing fine screen channel was designed for a maximum flow of 2.6 MGD. Flows over this are designed to overflow into the bypass screen channel. For this preliminary planning effort, it is assumed that the bypass channel would have to be used for 5% of the wet season as a result of not upgrading the headworks.

4.3.1.3 Environmental Impacts

There are no major environmental impacts associated with this alternative.

4.3.1.4 Land Requirements

This alternative would not require additional land.

4.3.1.5 Potential Construction Problems

This alternative would not require construction to implement.

4.3.1.6 Cost Estimates

Detailed cost estimates are provided in Appendix I. As the "do-nothing" alternative, this alternative would have zero associated capital cost. However, the increased use of the bar rack would result in more labor hours. Based on approximately 65 hours per year for maintenance of the fine screen, 20 hours for maintenance of the bar rack, $500 per year for replacement parts for the headworks components, and 6,000 kWh for electrical demand for the existing headworks components, the annual O&M cost for this alternative is estimated at approximately $4,400 per year.

4.3.2 Increase Screening Capacity

4.3.2.1 Description

This alternative involves construction of a parallel channel in the headworks and installation of a second fine screen to increase screening capacity.

4.3.2.2 Design Criteria

The maximum flow rate of the existing fine screen is 2.6 MGD (1805 GPM). Within the 2’ wide channel, the maximum upstream water level for the screen is 29 inches, associated with a headloss of 16 inches.

This alternative assumes that an identical screen unit would be installed in a channel constructed adjacent to the existing bypass channel, effectively doubling the screening capacity. This alternative would retain the existing bypass channel with the manually screened bar rack, to provide screening in case of total power loss to the headworks and also provide 3’ clearance between the two screens for maintenance.

4.3.2.3 Map

A conceptual drawing of this alternative is provided in Figure 4-1.

4.3.2.4 Environmental Impacts

There are no major environmental impacts associated with this alternative.

4.3.2.5 Land Requirements

This alternative would be constructed within the property of the existing treatment plant. No additional land is required.

4.3.2.6 Potential Construction Problems

This project would involve construction onto the existing headworks structure. A full structural evaluation will need to be performed as part of this project to ensure the headworks will be sound during and after construction. Consideration should be made to perform construction during the dry-season.

4.3.2.7 Cost Estimates

Detailed cost estimates are provided in Appendix I. This alternative would have capital costs associated with the construction of a parallel channel for a new screen unit, the screen unit itself, and installation and electrical fees. The capital cost is estimated at approximately \$470,000. The salvage value is estimated at approximately \$18,000 based on a 20-year planning period (2043\$). Based on approximately 65 hours per year for maintenance of the fine screen, 5 hours for maintenance of the bar rack, \$750 per year for replacement parts for the headworks components, and 6000 kWh for electrical demand for the headworks components, the annual O&M cost for this alternative is estimated at approximately \$4,000 per year.

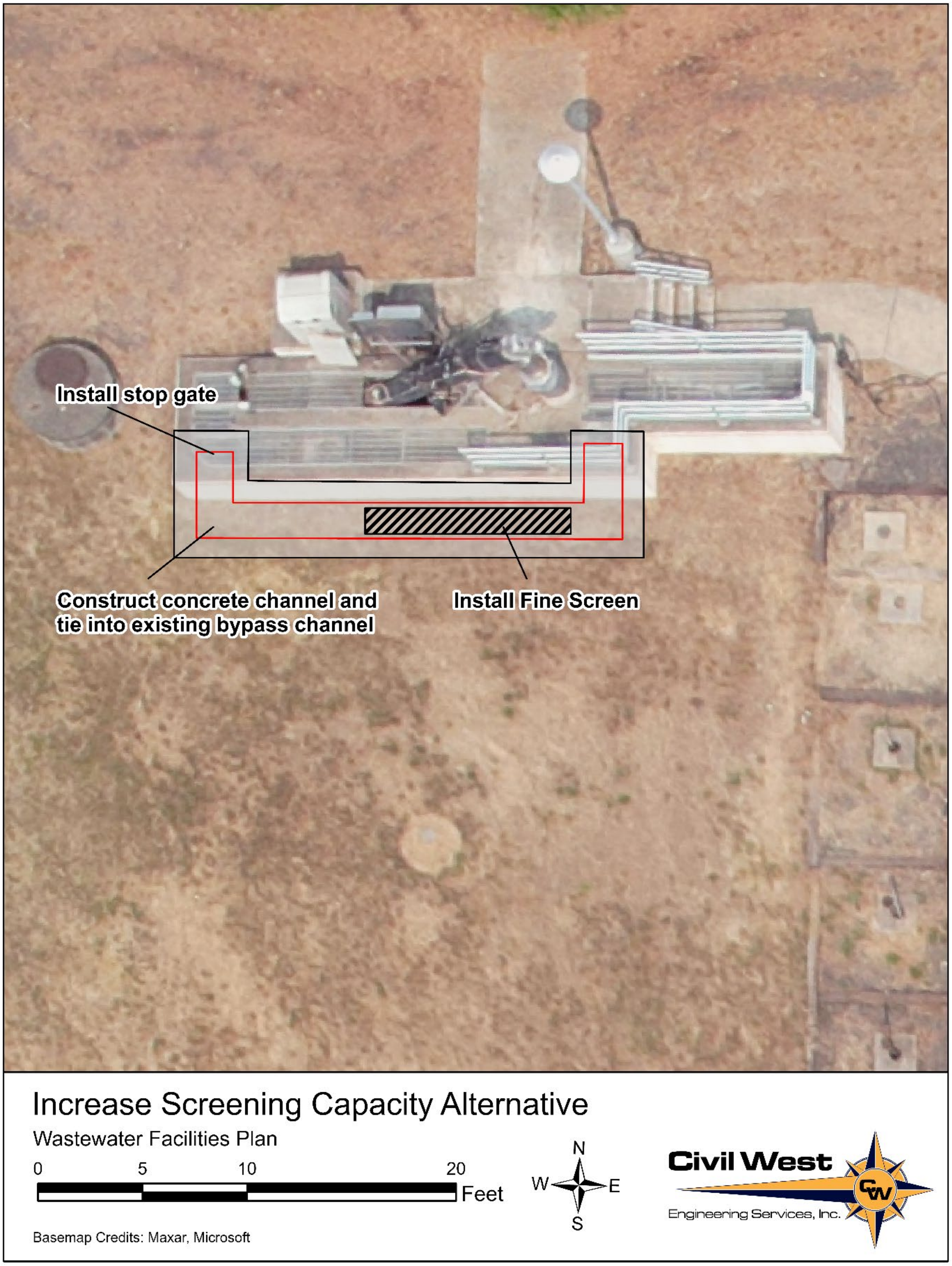


Figure 4-1: Conceptual drawing of adding an additional fine screening unit

4.4 Primary Treatment

Primary clarifiers should be sized for peak daily flow according to DEQ facilities plan guidelines. The current clarifiers, based on 2023 peak daily flow, have an overflow rate of 1376 gpd/ft^2. At 2043 design flow, the overflow rate is projected to be 1515 gpd/ft^2. These are within acceptable ranges based on typical design values of 1,200 to 2,000 gpd/ft^2 (Metcalf and Eddy). The detention time of the clarifiers are approximately 1.5 hr at the design flow. While detention times closer to 2 hr are ideal, over 1 hr is acceptable for a sedimentation process ahead of secondary treatment (Metcalf and Eddy, 5th ed.).

The primary clarifiers are not vital to the overall treatment process due to relatively small TSS loads in the City's influent wastewater. The main purpose of constructing the primary clarifiers was to protect the trickling filter process from unnecessary solids loading. However, since the primary clarifiers have been in operation, they have not been instrumental in helping the City meet its permit limits as evidenced by the multiple permit violations in the past decade.

Furthermore, operation of the primary clarifiers is arguably more trouble than its worth for the City because of the nuisance conditions (scum and odors) created by primary sludge and significant O&M demands. Continuation of operating the primary clarifiers is only recommended for the option of expanding the existing trickling filter/solids contact system. The conversion of the primary clarifiers to biological aeration reactors was considered as an alternative for secondary treatment improvements, as discussed in Section 4.5.6.2.

4.5 Secondary Treatment Improvements

4.5.1 General Process Considerations

The existing biological treatment system has had issues meeting BOD_5 and TSS removal targets to comply with the City's NPDES permit. Furthermore, multiple deficiencies with the existing system necessitate the consideration of process improvements. These deficiencies include the following:

- A lack of redundant secondary clarification capacity, inconsistent with DEQ and EPA reliability requirements,
- The existing secondary clarifier is oversized for typical summer flows, resulting in sludge retention times that risk the activated sludge going septic in the clarifier bottoms,
- During high flow events, water ponds in the trickling filter and colder temperatures inhibit biological activity, causing the filter to act more as an equalization tank for the small aeration channel than as its own treatment unit. During the dry season, the trickling filter has to be recirculated by a factor of over 4 to keep the arms spinning at a rate so that the entire filter area is wetted.
- The solids contact aeration channel does not provide adequate aeration volume on its own.
- The existing system was not designed to treat ammonia, which could be required within the planning period based on DEQ's analysis in upcoming NPDES permit renewal cycles.

Multiple alternatives were determined in an initial review to be technically infeasible given land availability, operations capacity, and treatment requirements. A brief description of these alternatives and the rationale behind their infeasibility is presented below:

- *Lagoons* – While lagoons are an attractive choice for small communities like Lowell due to the low O&M requirements, they require a substantial amount of area to construct. The topography of the area around Lowell is very hilly, leading to a lack of suitable land to construct a lagoon system. The expected treated effluent quality of lagoons would likely be insufficient to meet the City's NPDES requirements for TSS and BOD, necessitating the City to consider effluent reuse for summer discharge, which would have a high cost for piping and land purchase. Due to the high anticipated costs, land requirements, and likely decrease in effluent quality, lagoons were not considered a feasible alternative.
- *Oxidation Ditch* – Oxidation ditches are an extended aeration system consisting of long, continuous channels that are continuously aerated to treat BOD and ammonia. These systems can provide good treatment, but require a large footprint compared to other extended aeration systems and conventional activated sludge systems. An oxidation ditch would likely require the City to purchase land for a new treatment plant site or demolish many of the existing structures to make room. With these considerations, the oxidation ditch was not considered viable for the City.
- *Rotating Biological Contactor* – RBCs are a fixed-film technology that, similar to a trickling filter, pass primary-clarified wastewater over a zoogleal film to remove BOD and nutrients. Instead of the film growing on filter media like in a trickling filter, the microorganisms grow on rotating plastic discs. These proprietary units have small footprint requirements. However, their performance is highly dependent on temperature and flowrate as those parameters affect biological activity and biofilm-shearing. Given the highly variable nature of Lowell's climate and wastewater flowrates, a complex system of parallel RBC treatment trains would need to be designed for all possible conditions. This would pose a concern given the slow start-up time of fixed-film biological reactors, requiring a high degree of attention by the operators to keep the biology active on the reactors. Because of these considerations, RBCs were considered technically infeasible compared to more conventional biological treatment technologies.
- *Membrane Bioreactor* – These units consist of a conventional activated sludge aeration basin with membrane filters in lieu of secondary clarifiers. The main benefits of membrane bioreactors are that they require a smaller footprint than conventional biological treatment alternatives, they retain larger biomass concentrations in the bioreactors for theoretically better treatment of dissolved organic matter, and they produce effluent with similar quality to plants with tertiary filtration treatment processes. However, they do require extensive pumping and electrical control systems to operate properly, and therefore require more oversight by the operator. Since one of the primary concerns with the existing facilities is the extensive O&M requirements of existing electrical and mechanical systems, it was decided that a system heavily reliant on pumps and mechanical units would compound the City's existing issues. This was therefore considered not a viable alternative for the City.
- *No Construction* – the "no action" alternative in this case is not feasible as it would leave the plant out of compliance with redundancy requirements and the City has had NPDES permit compliance issues with the existing treatment system. At a minimum, the City should have a plan to increase clarification capacity and upgrade the biological treatment system to have the capacity to meet design year flows and loads. A cost estimate for the "No Construction" alternative is provided (Appendix I) for Net Present Value comparisons in Section 5.1.

Multiple alternatives for upgrading the biological treatment systems were determined to be technically feasible and were evaluated in detail in the following sections. In addition to these broad treatment system alternatives, an analysis of adding a supplemental alkalinity addition system to improve nitrification capacity of the WWTP was evaluated.

4.5.2 Redundant Secondary Clarifier

4.5.2.1 Description

The existing treatment system is out of compliance with redundancy requirements because the WWTP has only one secondary clarifier. This means that the existing clarifier cannot be effectively maintained. Furthermore, the existing clarifier was built for capacity associated with peak day flows. This makes the clarifier oversized for typical dry weather flows, which creates issues associated with sludge age.

It is recommended that a redundant clarifier should be constructed to optimize treatment of summer flows. The existing secondary clarifier is in relatively good condition and is appropriately sized to handle peak and max month wet weather flows. With this alternative, a smaller clarifier would be in operation during the dry season, and the operator could divert flows from the aeration basin to the larger clarifier when the plant's flows increase in the wet season. The operator would also have the flexibility to operate both clarifiers in parallel, although this would probably not be necessary given the projected future flows.

4.5.2.2 Design Criteria

It is recommended to size a new clarifier for the 2045 design MMDWF of 0.3 MGD. A typical design point for dry season flows is 500 gpd/sqft. Applying the design point to the MMDWF results in a clarifier area of 620 sqft, or an equivalent clarifier diameter of 28'. Assuming an MLSS concentration in the aeration basin of 2,500 mg/L, a 28' clarifier results in a solids loading rate of 10 lb/day/sqft at MMDWF.

To meet reliability class II requirements for sedimentation basins, the smaller clarifier unit must be able to handle a capacity of at least 50% of peak day flow. The existing clarifier was sized at 1538 gpd/sf, which is a good design point for a secondary clarifier for peak day flowrates. 50% of the peak day flow is 0.71 MGD. Applying that same design point results in a minimum clarifier diameter of 25 feet (assuming a circular footprint).

With both of these design considerations, a 28' diameter clarifier is appropriately sized for dry-season flows throughout the planning period.

4.5.2.3 Location

The location of a new redundant clarifier will depend on the alternative selected for secondary treatment improvements. The location of the clarifier is clearly noted on each of the applicable alternatives in later subsections.

4.5.2.4 Environmental Impacts

There are no major environmental impacts as a result of this alternative.

4.5.2.5 Land Requirements

This alternative would not require the City to purchase additional land as it would be located on the existing treatment plant lot.

4.5.2.6 Potential Construction Problems

This alternative could be constructed outside of the flood zone, however, projects that involve underground piping should be planned to be constructed in the dry-season to avoid difficulties with managing high groundwater levels.

4.5.2.7 Cost Estimates

Detailed cost estimates are provided in Appendix I. This alternative would have capital costs associated with the construction of a flow splitter, the clarifier, mechanical mechanisms, piping improvements, and RAS/WAS pump improvements. The capital cost is estimated at approximately $1.2 million. The salvage value is estimated at approximately $105,000 based on a 20-year planning period (2043$). Based on approximately 80 hours per year for maintenance and operator labor, and $850 per year for the electricity for the clarifier drive and pump components, the annual O&M cost for this alternative is estimated at approximately $4,000 per year.

4.5.3 Supplemental Alkalinity Addition

4.5.3.1 Description

The City's NPDES permit requires the effluent pH to be between the values of 6.0 and 9.0. The effluent pH has been at the low end of this range at the end of summer and early fall in the last five years, while the influent pH tends to be slightly basic (Figure 4-2). It is likely that nitrification is occurring in the secondary treatment system during low flow periods, which would explain the drop in pH between influent and effluent. Another common cause for pH drops is the use of acidic chemicals (like bisulfite) for dechlorination; however, Lowell uses a non-acidic calcium thiosulfate solution.

The operators have resorted to dosing the secondary effluent with lime to raise the pH prior to discharge to meet permit criteria. It would be more beneficial for the WWTP to dose alkalinity prior to biological treatment. This would improve nitrification in the secondary treatment system and help the City meet potential ammonia limits, and help the City maintain compliance with its NPDES permit.

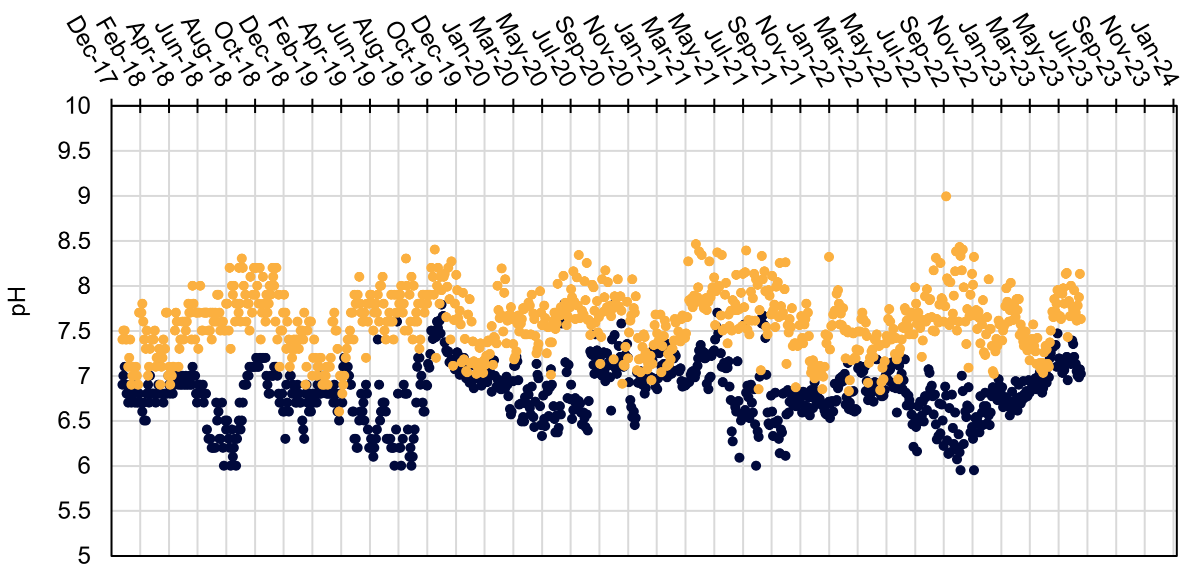


Figure 4-2: Reported pH in WWTP influent (Gold) and effluent (Blue)

4.5.3.2 Design Criteria

This alternative assumes the use of magnesium hydroxide as a supplemental alkalinity source, which is preferred over other alternatives since the solubility characteristics of the chemical reduce the risk of burning out downstream biology. MgOH provides about 13.38 lb of alkalinity as calcium carbonate per gallon. The amount of MgOH required per day to treat an assumed Total Kjeldahl Nitrogen (TKN) loading of 5 lb/day is calculated as shown below. Note that this analysis conservatively assumes that all organic nitrogen will degrade to ammonia and have an alkalinity demand of 7.14 lb alkalinity/lb nitrogen.

$$\text{Influent TKN Loading} = 5\,\frac{\text{lb}}{\text{day}}$$

$$\text{Alkalinity Demand} = 5\,\frac{\text{lb}}{\text{day}} * \frac{7.14\text{ lb Alkalinity Consumed}}{\text{lb Nitrogen}} = 35.7\,\frac{\text{lb Alkalinity Consumed}}{\text{day}}$$

$$\text{MgOH Feed Rate} = \frac{35.7\,\frac{\text{lb Alkalinity Consumed}}{\text{day}}}{13.38\,\frac{\text{lb Alkalinity}}{\text{gal MgOH}}} = 2.7\,\frac{\text{gal MgOH}}{\text{day}}$$

Assuming a 4-month supply of MgOH would be kept on hand, a 500-gallon drum that a mixer can be installed in is recommended. Heating equipment should be provided on the drum and chemical feed lines to prevent freezing during cold weather months. A mixer/agitator should be sized after conferring with chemical suppliers to confirm the level of agitation required to keep the slurry well mixed.

4.5.3.3 Location

A supplemental alkalinity system would be added prior to the secondary treatment system to provide alkalinity required for ammonia removal via nitrification. A logical location for the dosing point would be towards the end of the headworks channel after the influent Parshall Flume, prior to the primary clarifier. The chemical feed equipment could be placed in the existing chemical storage area next to the laboratory.

4.5.3.4 Environmental Impacts

A chemical addition treatment step would result in the need to transport the chemicals on site to the treatment plant. However, the impact of discharging acidic effluent to the river would have larger and more immediate impacts to the natural environment.

4.5.3.5 Land Requirements

This alternative would not require the City to purchase additional land as it would be located on the existing treatment plant lot.

4.5.3.6 Potential Construction Problems

This alternative could be constructed outside of the flood zone, however, projects that involve underground piping should be planned to be constructed in the dry-season to avoid difficulties managing high groundwater levels.

4.5.3.7 Cost Estimates

Detailed cost estimates are provided in Appendix I. This alternative would have capital costs associated with the construction of chemical dosing system, chemical feed piping, electrical and controls, and installation. The capital cost is estimated at approximately $175,840. The salvage value is estimated at approximately $600 based on a 20-year planning period (2043$). Based on 32 hours per year for maintenance and labor, approximately $200 per year for electricity, and

1,000 gallons of MgOH slurry per year, the annual O&M cost for this alternative is estimated at approximately $4,500 per year.

4.5.4 Expand Existing Trickling Filter/Solids Contact System

4.5.4.1 Description

The existing biological treatment system consists of a trickling filter with plastic media and an aeration basin. Expansion of the existing system would involve the construction of a redundant aeration channel and a redundant secondary clarifier. It is not recommended to construct a new, or expand the existing, trickling filter since the existing unit already has issues during low flow periods turning the hydraulic distributor.

4.5.4.2 Design Criteria

The recommended total aeration basin volume is 41,000 gallons based on biological process modeling. Accounting for treatment provided by the existing trickling filter, doubling the current aeration volume in the existing solids contact aeration basin would be sufficient. The secondary clarifier should be designed following the criteria as described in Section 4.5.2.2.

4.5.4.3 Location

A conceptual site plan for the construction of the aeration basin and secondary clarifier is provided in Figure 4-3.

4.5.4.4 Environmental Impacts

Biological treatment is where the majority of BOD and TSS removal occurs in a standard WWTP. Without meeting redundancy requirements, the components of the system cannot be taken offline for full maintenance, potentially leading to effluent quality issues. Undersized unit operations could also lead to poor effluent quality. Upgrading the treatment system would ensure effluent quality targets can be met throughout the year.

4.5.4.5 Land Requirements

This alternative would not require the City to purchase additional land as it would be located on the existing treatment plant lot.

4.5.4.6 Potential Construction Problems

This alternative could be constructed outside of the flood zone, however, projects that involve underground piping should be planned to be constructed in the dry-season to avoid difficulties managing high groundwater levels.

4.5.4.7 Cost Estimates

Detailed cost estimates are provided in Appendix I. This alternative would have capital costs associated with the construction of a second aeration basin, piping upgrades, a flow splitter for aeration basin selection, and electrical and controls. Capital costs are estimated at approximately $1.2 million. Salvage value is estimated at approximately $55,500 based on a 20-year planning period (2043$). Based on approximately 1300 hours per year for O&M, and $12,000 for electricity associated with aeration, the annual O&M cost for this alternative is estimated at approximately $64,000 per year.

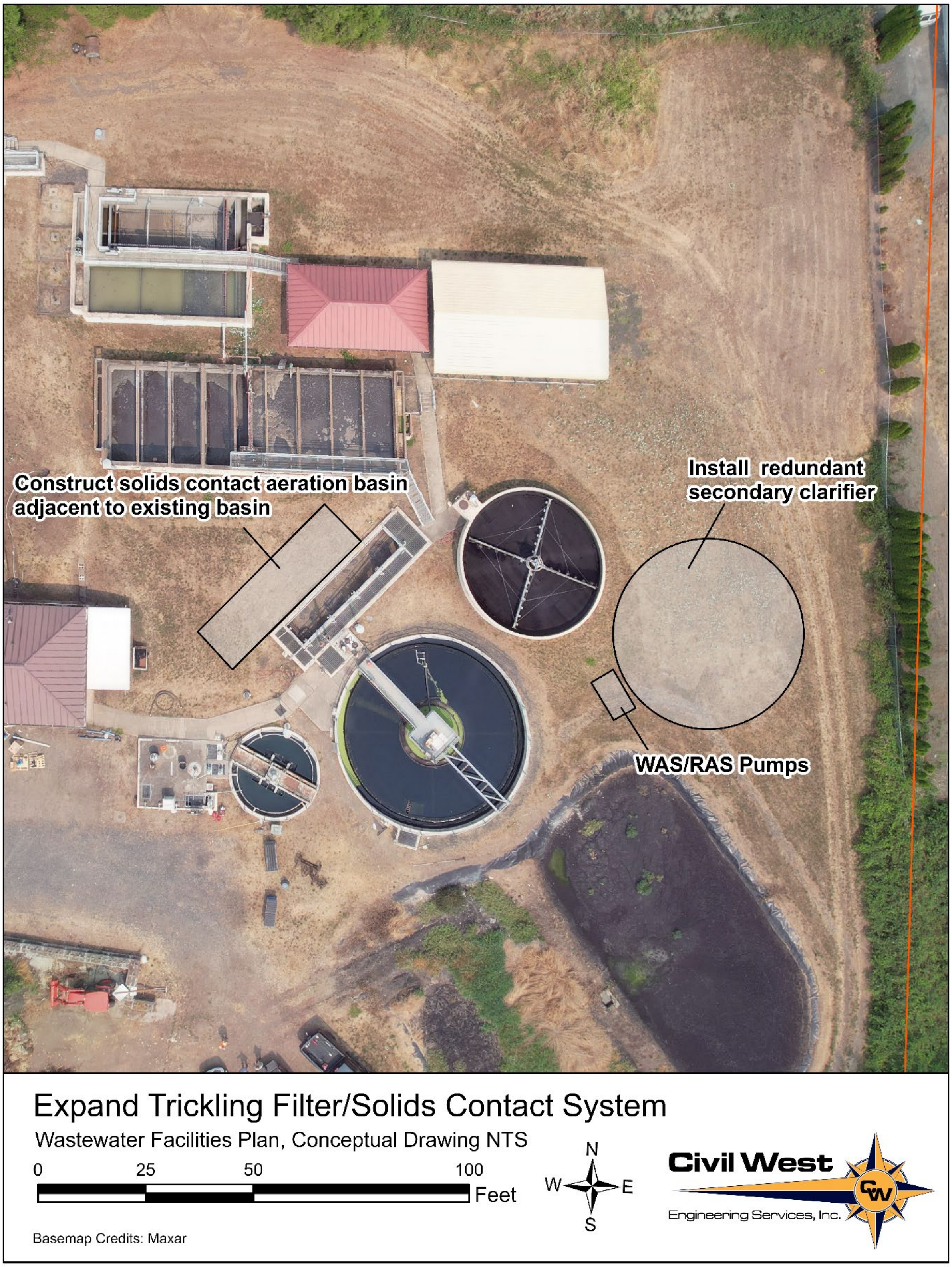


Figure 4-3: Expand Existing Biological Treatment System Alternative

4.5.5 Sequencing Batch Reactors

4.5.5.1 Description

Sequencing Batch Reactors (SBRs) are biological reactors that operate in a sequence of fill – react – settle – decant – idle. These batch systems are attractive compared to continuously mixed or plug flow processes because the reaction and clarification steps both occur within the footprint of one structure. The downsides are that at least two parallel units are required to operate continuously, an equalization basin is required upstream of the reactors to attenuate diurnal flow variations, and a complex controls system and a competent operator are needed to operate effectively. All of these units would require extensive demolition of existing units and regrading of the site.

4.5.5.2 Design Criteria

Planning level design criteria for this alternative are provided in Table 4-4.

Table 4-4: Design Criteria for Sequencing Batch Reactor Alternative

Design Criteria - Sequencing Batch Reactors	
Equalization Basins (Pre and Post)	
Max Water Depth	13.5 ft
Freeboard	1.5 ft
Surface Area	1225 sqft
Reactor Basins	
Number	2
Max Water Depth	13.5 ft
Freeboard	1.5 ft
Surface Area	1225 sqft
Treatment Cycle Duration	5 h
MLSS Concentration	3000 mg/L
Hydraulic Retention Time	1 day
Solids Retention Time	15 days
Air Requirement	150 scfm

4.5.5.3 Location

A conceptual site map of this alternative is provided in Figure 4-4.

4.5.5.4 Environmental Impacts

Upgrading the treatment system would ensure effluent quality targets can be met throughout the year.

4.5.5.5 Land Requirements

This alternative would not require the City to purchase additional land as it would be located on the existing treatment plant lot.

4.5.5.6 Potential Construction Problems

The relatively large depth requirements of the SBR basins would require substantial excavation to keep the water level between the primary clarifier and disinfection unit operations. Temporary treatment facilities or holding tanks would likely need to be installed during construction to provide treatment, because the footprint of the required reactors would necessitate demolishing the entire existing biological treatment system.

4.5.5.7 Cost Estimates

Detailed cost estimates are provided in Appendix I. This alternative would have capital costs associated with the construction of the equalization basins, reactor basins, piping upgrades, and electrical and controls. The capital cost is estimated at approximately $3.8 million. The salvage value is estimated at approximately $480,000 based on a 20-year planning period (2043$). Based on approximately 1200 hours per year for O&M, and $6,700 for electricity associated with aeration and pumping, the annual O&M cost for this alternative is estimated at approximately $54,000 per year.

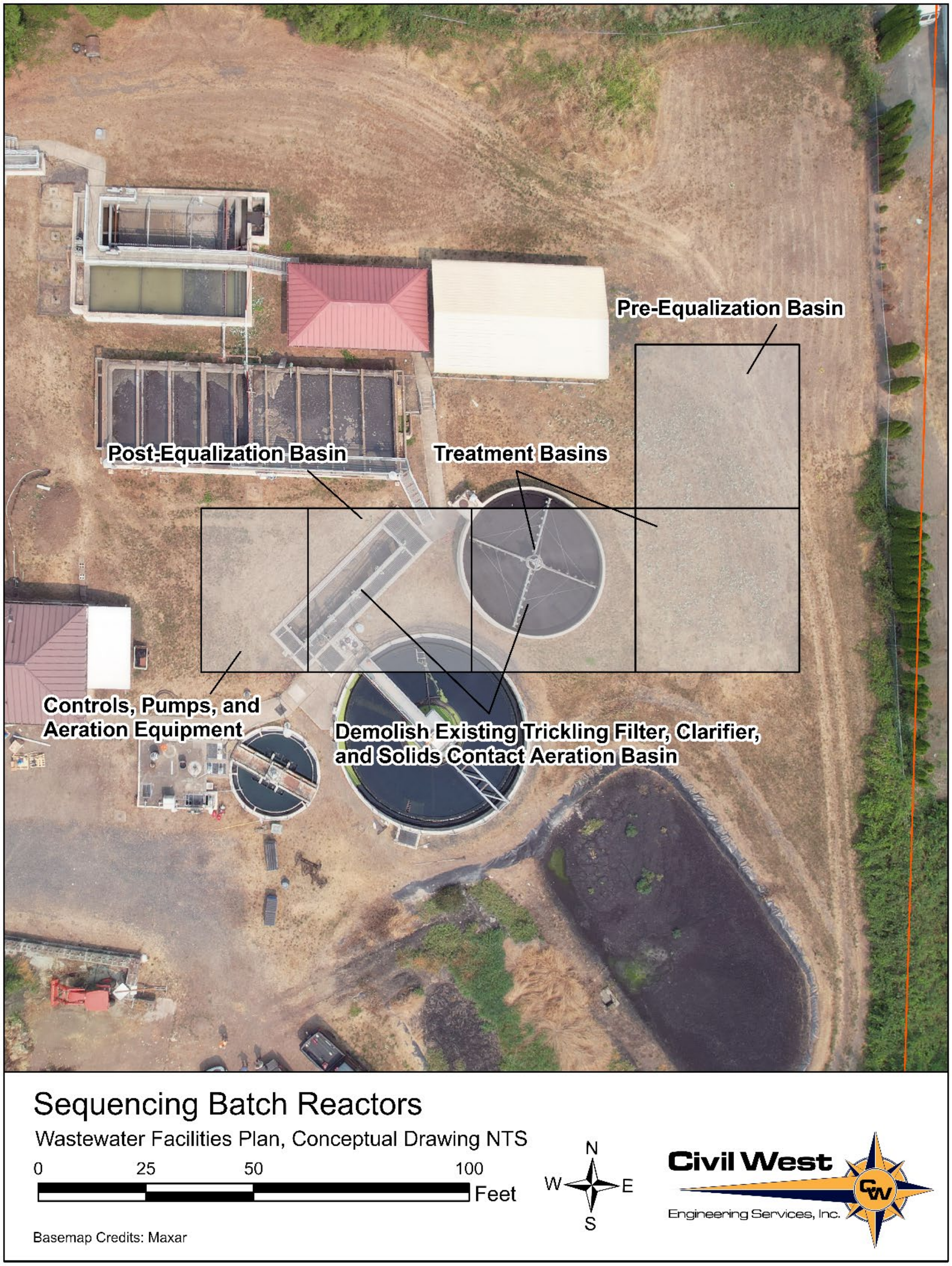


Figure 4-4: Conceptual Site Map for Sequencing Batch Reactors Alternative

4.5.6 Conventional Activated Sludge

4.5.6.1 Description

A conventional activated sludge system consists of aeration basins and secondary clarifiers. The aeration basin should be sized for nitrification to occur given potential for future ammonia limits. There should be a minimum of two aeration basins and two clarifiers for redundancy and to handle a range of seasonal flow variations.

4.5.6.2 Design Criteria

A logical location for the aeration basins is to convert the existing primary clarifiers into aeration basins. With this alternative, the flow path is already established, and no hydraulic changes are necessary. As discussed in Section 4.4, the primary clarifier would not be necessary if the trickling filter was decommissioned, and the treatment system was converted to an activated sludge configuration. The primary clarifier would be split into two equally sized basins with the idea that one basin could provide the appropriate treatment capacity for typical flows, and both basins could be used during high flow events. A secondary clarifier would be constructed as described in Section 4.5.2 for redundancy and for use during dry-season flows. Piping for return activated sludge would be routed to the top of the new aeration basins. Because the trickling filter would be decommissioned as part of this alternative, the pad that the trickling filter sits on could be used for the new clarifier location.

A biological model was prepared to estimate the required aeration capacity for treatment of BOD_5 and ammonia, and to estimate biosolids production rates at the design pollutant loads. The model was evaluated to meet BOD_5 limit of 10 mg/L, TSS limit of 10 mg/L, and an ammonia limit of 1 mg/L given the loadings in Table 3-10. The model was evaluated with and without nitrification (removal of ammonia). The results of this model are presented in full in Appendix H, and a summary of design criteria from the modeling is provided in Table 4-5. These criteria were used to evaluate biological treatment improvement alternatives in Section 4. Generally, mixing requirements were limiting regarding aeration rates, except for projected 2045 max month flows for the full nitrification alternative.

Table 4-5: Estimated Aeration Requirements for Conventional Activated Sludge

Aeration Basin Air Requirements (SCFM)	
BOD Treatment Only	
Aeration Volume	41,300 gal
Average Dry Weather	138
Max Month	153
BOD Treatment + Full Nitrification	
Aeration Volume	82,600 gal
Average Dry Weather	276
Max Month	319

4.5.6.3 Location

A conceptual site plan of this alternative is provided in Figure 4-5.

4.5.6.4 Environmental Impacts

Upgrading the treatment system would ensure effluent quality targets can be met throughout the year.

4.5.6.5 Land Requirements

This alternative would not require the City to purchase additional land as it would be located at the existing treatment plant.

4.5.6.6 Potential Construction Problems

The existing trickling filter would need to be demolished to make room for the new secondary clarifier. The trickling filter pad is approximately the same size as required for the clarifier.

4.5.6.7 Cost Estimates

Detailed cost estimates are provided in Appendix I. This alternative would have capital costs associated with decommissioning the trickling filter, solids contact basin, and primary clarifier, installing aeration equipment and piping, and sludge piping improvements. The capital cost is estimated at approximately $820,000. The salvage value is estimated at approximately $72,000 based on a 20-year planning period (2043$). Based on approximately 960 hours per year for O&M, and $18,500 for electricity associated with aeration, the annual O&M cost for this alternative is estimated at approximately $57,000 per year.

Figure 4-5: Conceptual Site Map of Conventional Activated Sludge Alternative

4.5.7 Proprietary/Package Extended Aeration System

4.5.7.1 Description

Proprietary biological treatment systems, such as the Biolac© system by Parkson, have become attractive options for small cities such as Lowell with numerous case studies showing these units to be successful, and relative ease of construction and installation. There's also a significant benefit in the operations support available by the supplier for these units after construction. This alternative would involve purchasing and constructing a proprietary treatment unit. For this analysis, the Biolac© system was evaluated.

4.5.7.2 Design Criteria

Design criteria for this alternative is provided in Table 4-6.

Table 4-6: Design Criteria based on budgetary quote provided by Parkson for a Biolac© System.

Design Criteria - Proprietary Activated Sludge System	
Number of Aeration Basins	1
Approximate Dimensions at Grade (ft)	64x63
Approximate Bottom Dimensions (ft)	49x24
Basin Volume (MG)	0.17
Clarifier Size	65x23
Number of Clarifiers	1
Estimated SOR (lbs/hr)	42
Estimated SCFM	269

4.5.7.3 Map

A conceptual site plan for this alternative is provided in Figure 4-6.

4.5.7.4 Environmental Impacts

Upgrading the treatment system would ensure effluent quality targets can be met throughout the year.

4.5.7.5 Land Requirements

This alternative would not require the City to purchase additional land as it would be located on the existing treatment plant lot.

4.5.7.6 Potential Construction Problems

The existing biological treatment system (trickling filter and solids contact chamber) would likely need to be demolished to make room for the new treatment system.

4.5.7.7 Cost Estimates

Detailed cost estimates are provided in Appendix I. This alternative would have capital costs associated with decommissioning the trickling filter, solids contact basin, and primary clarifier, installing the new system, and sludge piping improvements. The capital cost is estimated at approximately $2.5 million. The salvage value is estimated at approximately $150,000 based on a 20-year planning period (2043$). Based on approximately 960 hours per year for O&M, and $18,500 for electricity associated with aeration, the annual O&M cost for this alternative is estimated at approximately $57,000 per year.

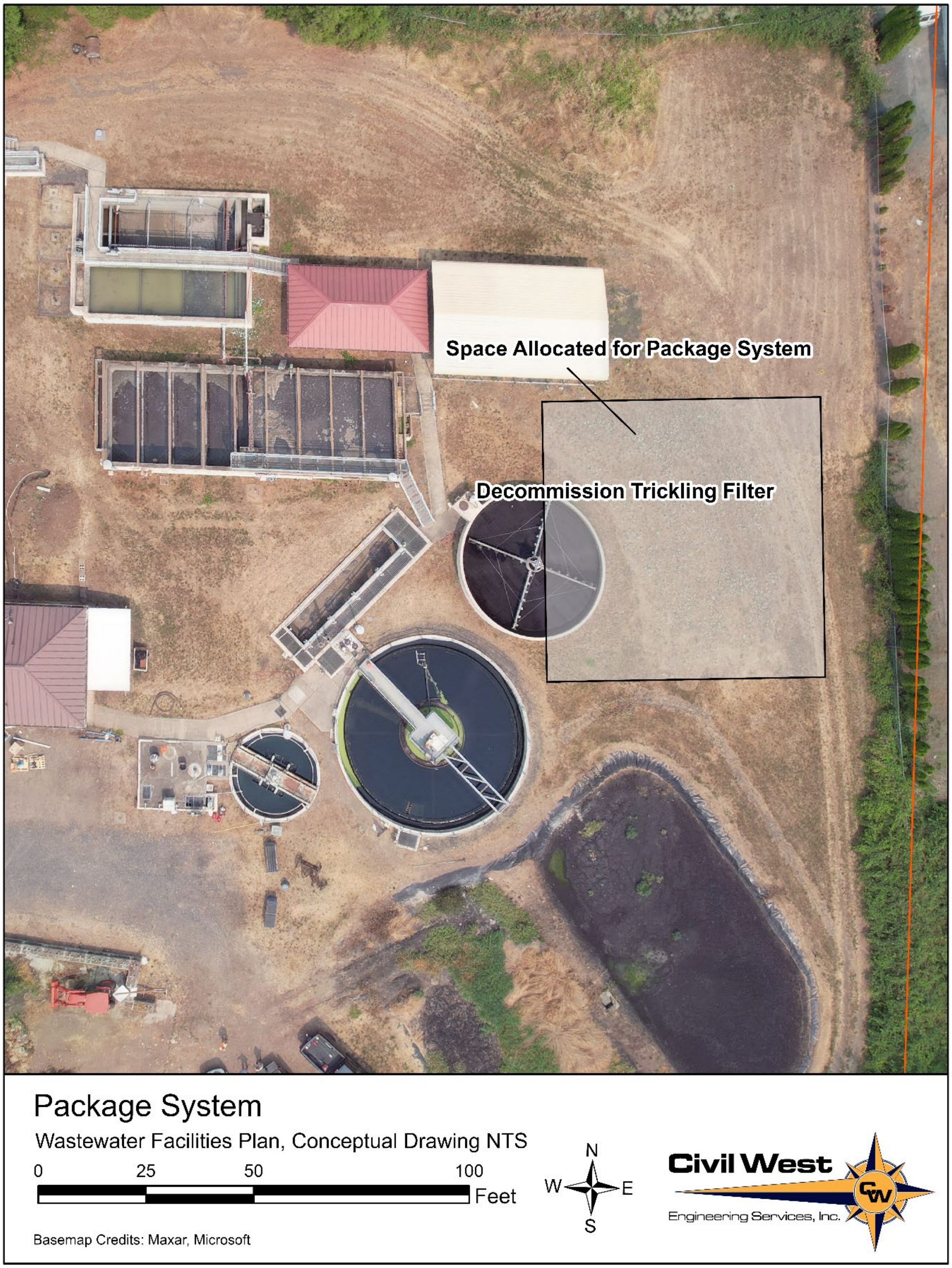


Figure 4-6: Conceptual Site Map for Package System Alternative

4.6 Disinfection Improvements

4.6.1 No Construction – Optimizing Existing Facilities

This alternative involves no changes to the existing disinfection system. The City would continue to use sodium hypochlorite as the disinfectant and calcium thiosulfate as the dechlorination chemical. The existing chlorine contact and dechlorination basins would be unchanged. A summary of O&M costs associated with the current system is provided in Table 4-7.

Table 4-7: Approximate Operations and Maintenance Costs of Existing Disinfection System

Disinfection "Do-Nothing" Alternative – Current Operations & Maintenance Costs					
No.	**Item Description**	**Quantity**	**Units**	**Unit Cost ($)**	**Item Cost ($)**
1	Operator Labor	511	h	$40	$20,400
2	Replacement Parts	1	LS	$1,000	$1,000
3	Hypochlorite	2000	gal	$4.00	$8,000
4	Thiosulfate	750	gal	$4.00	$3,000
5	Electricity Usage	5000	kWh	$0.08	$422
				Estimated Annual O&M (2023$)	**$32,862**

4.6.2 Construct New Chlorine-Based Disinfection System

4.6.2.1 Description

Liquid hypochlorite is one of the most commonly used wastewater disinfection methods. Liquid hypochlorite can be added as a solution formed from sodium hypochlorite or calcium hypochlorite. For this planning effort, it was assumed that bulk 12.5% liquid sodium hypochlorite would be purchased and delivered to the WWTP.

Alternative methods of liquid hypochlorite production could also be used. For example, calcium hypochlorite erosion feeders dissolve tablets to produce a dilute (~ 1.2%) solution of calcium hypochlorite. Also, electrolytic cell-based systems can be used to convert salt brine solutions into 0.8% solutions of sodium hypochlorite. This lower concentration solution is more stable than the 12.5% bulk solution, helping to ensure that a consistent hypochlorite dosage is introduced to the effluent stream. Further consideration of alternative methods of hypochlorite solution production and delivery should be considered during predesign work if a liquid hypochlorite approach is the recommended disinfection alternative.

4.6.2.2 Design Criteria

Design criteria for this alternative is provided in Table 4-8.

4.6.2.3 Map

A conceptual site plan for this alternative is provided in Figure 4-7.

4.6.2.4 Environmental Impacts

Chlorine will need to be removed prior to final discharge to meet NPDES permit requirements and prevent chlorine toxicity to aquatic life downstream of the WWTP.

Table 4-8: Design Criteria for Chlorine Disinfection and Dechlorination System

Hypochlorite Disinfection Preliminary Design Criteria	
Chlorination	
Chemical	Sodium Hypochlorite
Assumed Stored Chemical Concentration	12.5%
Design Dose	10 mg/L
Number of Chemical Feed Pumps	2
Consumed Per Day @ Design AAF	6.24 gal
Target Residual	1 mg/L
Chlorine Contact Chamber	
Number of Contact Basins	2
Minimum Basin Volume	2100 ft^3
Min. Effective Length of Channel	150 ft
Channel Width	4 ft
Length: Width Ratio	30:1
Dechlorination	
Chemical	Calcium Thiosulfate
Design Dose	3 ppm
Number of Chemical Feed Pumps	2
Consumed Per Day	3 lb

4.6.2.5 Land Requirements

This alternative would not require the City to purchase additional land as it would be located on the existing treatment plant lot.

4.6.2.6 Potential Construction Problems

This alternative could be constructed outside of the flood zone, however, projects that involve underground piping should be planned to be constructed in the dry-season to avoid difficulties managing high groundwater levels.

4.6.2.7 Sustainability Considerations

Hypochlorite disinfection is a chemical-intensive process requiring one chemical to disinfect and a second chemical to dechlorinate. During low flow periods, the sodium hypochlorite usage rate may drop. Sodium hypochlorite stability decreases as the concentration of the solution increases, potentially resulting in the degradation of purchased chemical prior to use if it is not used relatively quickly. This results in economic inefficiency and the potential for under-disinfected wastewater if the effluent chlorine residual is not regularly checked.

4.6.2.8 Cost Estimates

Detailed cost estimates are provided in Appendix I. This alternative would have capital costs of site preparation, excavation, site restoration, chlorine basin construction, equipment installation, and electrical and controls installation. The capital cost is estimated at approximately $550,000. The salvage value is estimated at approximately $54,000 based on a 20-year planning period (2043$). Based on approximately 460 hours per year for O&M, and $8,000 for chemicals, the annual O&M cost for this alternative is estimated at approximately $28,000 per year.

Figure 4-7: Conceptual Site Plan of new Chlorine Disinfection System Alternative

4.6.3 Construct UV Disinfection System

4.6.3.1 Description

Disinfection by ultraviolet (UV) light works by exposing microorganisms to wavelengths of light that damage DNA, limiting the ability of the microorganism to reproduce. One of the primary benefits of UV disinfection is that no chemicals are used. This eliminates the need for both chlorination and dichlorination chemicals that are required for hypochlorite-based disinfection systems.

Wastewater UV disinfection is achieved through two styles: open channel and closed vessel. Open channel UV disinfection places ultraviolet bulbs in racks that are submerged in a channel filled with secondary effluent. Closed vessel disinfection mounts the ultraviolet bulbs in a housing slightly larger than the diameter of the pipe. Closed vessel UV systems are particularly well-suited for situations where installation space is limited as the systems can be installed into a pipe; however, the systems typically have a higher capital cost relative to packaged open channel systems.

4.6.3.2 Design Criteria

Planning level design criteria for this alternative is provided in Table 4-9.

Table 4-9: Design Criteria for UV Disinfection System

UV Disinfection Design Criteria	
Style	Open Channel
Number of Banks	2
Minimum Dose @ PHF (All units on)	30 mJ/cm^2
Minimum Dose @ MMDWF	30 mJ/cm^2
Redundancy	Ballast and Controls
Minimum UV Transmittance	65%

4.6.3.3 Map

A conceptual site plan for this alternative is provided in Figure 4-8.

4.6.3.4 Environmental Impacts

Unlike a chlorine disinfection system, UV disinfection requires no chemicals. Additionally, UV does not leave residual chlorine that could be toxic to a receiving waterbody.

UV disinfection systems require regular maintenance and replacement of UV bulbs. UV bulbs contain mercury amalgam and require proper disposal methods to be followed.

4.6.3.5 Land Requirements

This alternative would not require the City to purchase additional land as it would be located on the existing treatment plant lot.

4.6.3.6 Potential Construction Problems

No significant construction problems have been identified for this project alternative.

4.6.3.7 Sustainability Considerations

UV disinfection requires a considerable amount of electricity compared to alternative disinfection methods.

4.6.3.8 Cost Estimates

Detailed cost estimates are provided in Appendix I. This alternative would have capital costs of site preparation, excavation, site restoration, UV basin construction, equipment installation, and electrical and controls installation. The capital cost is estimated at approximately $565,000. The salvage value is estimated at approximately $15,000 based on a 20-year planning period (2043$). Based on approximately 208 hours per year for O&M, and $1,000 for replacement parts, the annual O&M cost for this alternative is estimated at approximately $9,700 per year.

Figure 4-8: Conceptual Site Plan of UV Disinfection Alternative

4.7 Solids Treatment Improvements

4.7.1 No Construction – Optimize Existing System

This alternative would involve no changes to the existing solids treatment system. The existing drying bed underdrains have deteriorated, reducing treatment efficacy. It is not feasible to leave the City without a properly functioning underdrain system for the next planning period.

4.7.2 Rehabilitate Drying Bed Underdrains

4.7.2.1 Description

Sludge drying beds are an EPA and DEQ approved process that significantly reduce pathogens, provided the solids have been drying for at least three months. Sludge drying beds require low capital cost and energy consumption while requiring minimal operator skill and attention. T

The current drying beds have some design flaws, including being too deep for a tractor to easily remove solids. Operators have to unload a few yards of gravel to make temporary ramps whenever the beds are being emptied. The underdrains have also been damaged from use in the past planning period, making dewatering not as effective. This alternative involves replacing the bottoms of the drying beds as existing, including the underdrains, gravel fill, fabric layer, and sand.

4.7.2.2 Design Criteria

This alternative would replace the drying bed underlain materials in kind. The design detail from the most recent design (Tetra Tech, 2003) is provided in Figure 4-9.

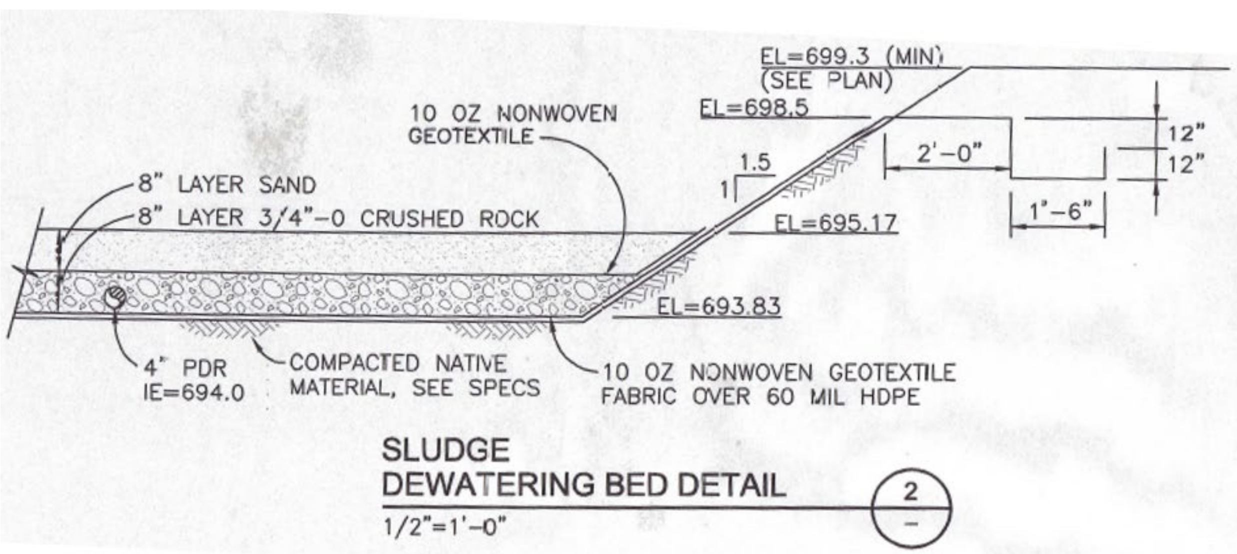


Figure 4-9: Design Details of Existing Sludge Drying Beds

4.7.2.3 Location

The location of the drying beds would remain unchanged from existing conditions.

4.7.2.4 Environmental Impacts

Rehabilitating the drying beds would improve dewatering capabilities of the drying beds.

4.7.2.5 Land Requirements

This alternative would not require the City to purchase additional land as the improvements would be located on the existing treatment plant lot.

4.7.2.6 Potential Construction Problems

No significant construction problems have been identified for this project alternative.

4.7.2.7 Sustainability Considerations

Sludge drying beds require little energy as compared to mechanical dewatering system.

4.7.2.8 Cost Estimates

The City is projected to produce approximately 100 to 200 lb/day of dried solids. Annual hauling fees are approximately $2,500. Along with operator labor and replacement part costs, the total annual O&M is approximately $4,300. The capital costs for rehabilitation were estimated at approximately $39,000 with a salvage value of $8,750.

4.7.3 Reconstruct Drying Beds

4.7.3.1 Description

This alternative involves replacing the bottoms of the drying beds as existing, including the underdrains, gravel fill, fabric layer, and sand, installing guide walls of concrete, and installing concrete ramps to allow easy entry for tractors cleaning the beds.

4.7.3.2 Design Criteria

The two pit-style drying beds will be converted into three smaller beds with a smaller overall footprint. The 3 new bed would be separated by two feet thick concrete walls spaced 15 feet apart. Sludge from the aerobic digester will be fed into the bed along the east side of the bed. Concrete ramps will be installed on the west side of the beds allowing for ease of solids removal. Specific design criteria are provided in Table 4-10.

Table 4-10: New Drying Bed Design Criteria

Construct Improved Drying Beds - Design Criteria	
Length (ft)	100
Width (ft)	50
Channel Width (ft)	15
Surface Area (ft^2)	4500
*Avg Loading Rate (lbs/ft^2*y)	14.6
*Peak Loading Rate (lbs/ft^2*y)	25.55
*Loading Rate per person (ft^2/person)	1.85

**Loading rates calculated assuming two of the three available channels of the drying beds in use. This assumes that one channel will be available for emergency emptying of the aerobic digester.*

4.7.3.3 Location

The new drying beds would be constructed in the footprint of the existing drying beds.

4.7.3.4 Environmental Impacts

Rehabilitating the drying beds would ensure that the solids treatment would be more efficient. Installation of the ramps will prevent tearing of the liner when machinery enters the beds.

4.7.3.5 Land Requirements

This alternative would not require the City to purchase additional land as the improvements would be located on the existing treatment plant lot.

4.7.3.6 Potential Construction Problems

No significant construction problems have been identified for this project alternative.

4.7.3.7 Sustainability Considerations

Sludge drying beds require little energy as compared to mechanical alternatives to dewatering.

4.7.3.8 Cost Estimates

Detailed cost estimates are provided in Appendix I. The City is projected to produce approximately 100 to 200 lb/day of dried solids. Annual hauling fees are approximately $2,500. Along with operator labor and replacement part costs, the total annual O&M is approximately $3,700. The capital costs of the new construction is approximately $342,500, with a salvage value of $117,900.

4.7.4 Rehabilitate Aerobic Digester Aeration System

4.7.4.1 Description

The existing positive displacement rotary lobe blowers require exorbitant maintenance, break frequently, and cost $6,000 per blower to replace. These blowers make up most of the WWTP's short-term asset expenses, having been replaced approximately every 5 years since the system was installed. With the current aeration piping system, it is not possible to isolate aeration basins even though only one is needed. This project would install valving on the aeration system to be able to isolate the digester cells and replace the existing blower with two, smaller sized turbine blowers.

4.7.4.2 Design Criteria

The current blowers are oversized for the system, requiring both aeration basins to be run at all times. Downsizing the blowers will allow for basin isolation and improve the energy costs of the blowers. For this project, it is assumed that the existing blowers would be replaced with turbine-style positive displacement blowers with a design point of 300 scfm at 6.5 PSIG. This alternative would include two new blowers for redundancy.

4.7.4.3 Environmental Impacts

There are no major environmental impacts because of this alternative.

4.7.4.4 Land requirements

This alternative would not require the City to purchase additional land as the improvements would be located on the existing treatment plant lot.

4.7.4.5 Potential Construction Problems

No significant construction problems have been identified for this project alternative.

4.7.4.6 Sustainability Impacts

Reducing the size of the blowers and having the capabilities of running an isolated aeration basin will allow for a reduction in the energy consumption of the system.

4.7.4.7 Cost Estimates

Detailed cost estimates are provided in Appendix I. This alternative would include the capital costs of replacing the blowers and valving improvements. The capital cost is estimated at approximately $216,000. The salvage value is estimated at approximately $21,000 based on a 20-year planning period (2043$). Based on approximately 52 hours per year for O&M, and $500 for replacement parts, the annual O&M cost for this alternative is estimated at approximately $2,600 per year.

4.8 Collection System Improvements

4.8.1 Rehabilitate Inflow and Infiltration Sources

Multiple areas of the collection system were identified to have issues during the I/I investigation, the results of which are provided in Appendix D. Twenty-six locations were identified as likely sources of stormwater inflow and eight sections of the collection system were identified as likely sources of groundwater infiltration.

It is recommended that the City prioritize two instances of direct connection between the storm drainage system and the collection system, as identified by smoke testing. Specifically, a curb inlet on the corner of Moss Street and Lakeview Street, and a culvert on 2nd street between Moss Street and Cannon Avenue. As a first step, these lines should be CCTV surveyed to identify the direct cause of the cross connection. The City should budget $1,400 to CCTV these lines as soon as possible in the planning period. After the issues are more clearly identified, it is recommended to use the City's stormwater fund for rerouting the problematic storm lines. Assuming that the storm lines will have to be repaired to fix these cross connections, a budgetary estimate for repair is $120,000. A new estimate should be made once CCTV data is available. The City should also plan for CCTV surveillance of approximately 6,300 linear feet of pipe in the collection system. CCTV prioritization should be organized as followed, based on unaccounted for flow volumes measured during flow testing:

1. Alder Street, South of the Lift Station to Main Street
2. 1st Street, West of Cannon Avenue to N Hyland Drive
3. East of Moss Street, from 3rd Street to North of 4th Street to first manhole on D Street.
4. Between 3rd and 4th Streets, West of Pioneer Street to N Hyland Drive
5. South of Main Street, from Moss Street to the first manhole by the School
6. 6th Street to second manhole on 7th Street.
7. North end of Alder Street to 2nd Street, and 2nd Street to Damon Street

8. North end of Cannon Street to Pioneer Street (pipe south of North Shore Drive)

Multiple manholes in the collection system were identified with leaks. Figure 4-10 indicates the location and the specific issue observed with each of these manholes. The recommended reparation project varies for each manhole from simple regrouting to full replacement; a budgetary cost estimate for each manhole is provided in Table 4-11. The identifying numbers in Table 4-11 correspond to the labeled numbers in Figure 4-10.

Table 4-11: Budgetary Cost Estimates for Manhole Reparation Projects

Manhole Number	Type of Repair	Cost Estimate
68	Full Replacement	$15,000
79	Full Replacement	$15,000
17	Full Replacement	$15,000
7	Regrout Ring	$1,500
136	Patch Holes/Regrout Ring	$2,000
126	Regrout Ring	$1,500
57	Patch Holes/Regrout Ring	$2,000
12	Patch Cracks	$1,000
80	Regrout Ring	$1,500
	Total Cost Estimate:	**$54,500**

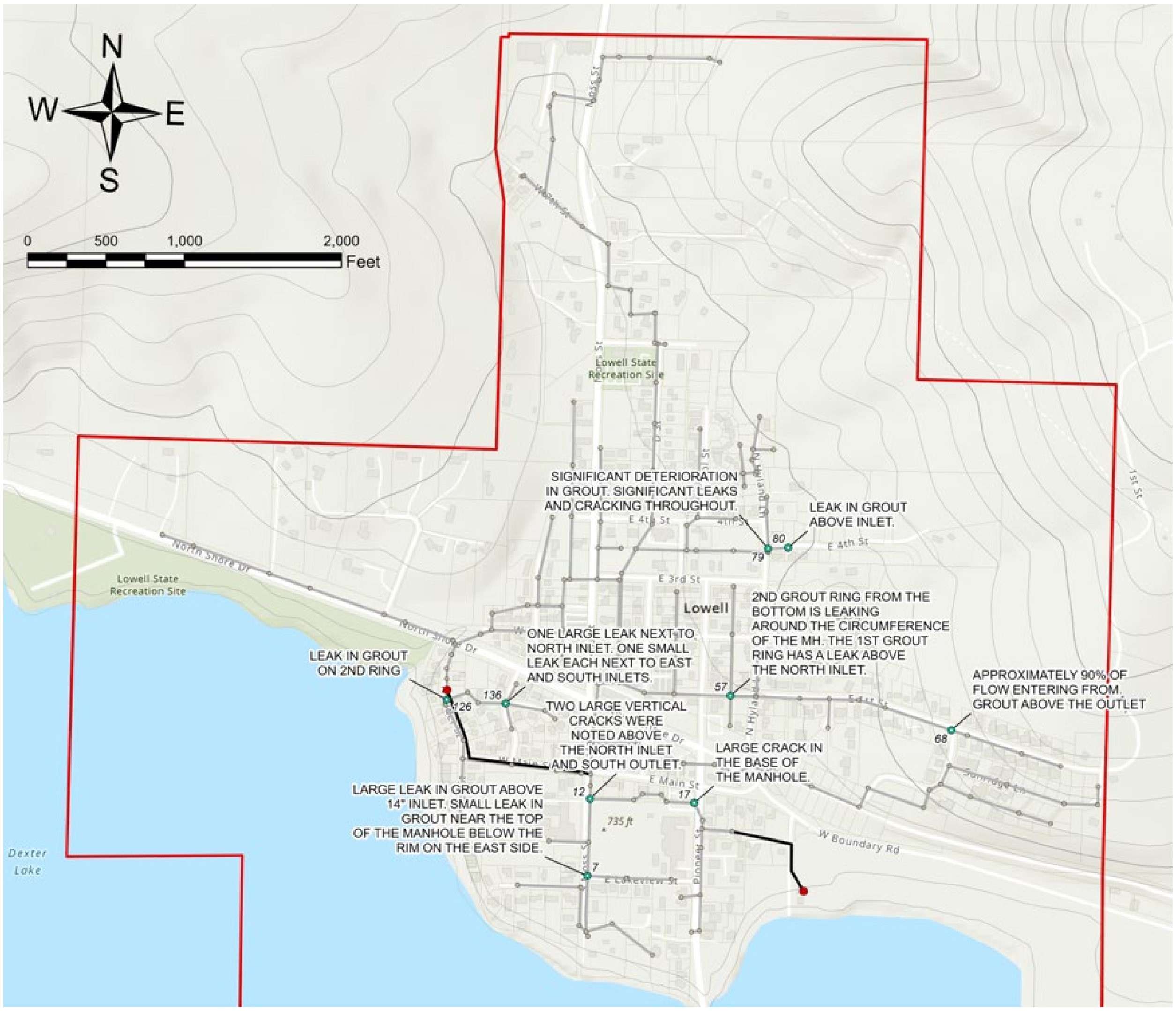


Figure 4-10: Locations of Manholes to Rehabilitate

4.8.2 Upgrade Alder Street Lift Station

The pumps in the lift station should be upgraded to have a capacity of 490 gpm with a total head of 43 feet. The upgraded system should include two pumps that fit in the existing mounts. The pump station building and wet well are in relatively good condition and should be maintained. A budgetary cost estimate for this project is approximately $390,000.

4.8.3 Collection System Capacity Upgrades

As discussed in Section 3.3.4.2, two pipes in the collection system that serve a significant number of properties are undersized for the City's growth projections. To address this, two alternatives were considered. These alternatives are discussed in the following subsections.

4.8.3.1 Alternative 1 – Cannon Avenue

A new 12" line would connect to the junction of 8" lines at the south end of the Moss/Cannon sewershed. This line would run down Cannon Avenue until it meets the existing 8" line that collects the 1st Street sewershed. A new 12" line would replace the undersized 8" line from 1st street to Cannon avenue, and then a new 15" line would collect both the 1St Street and Moss/Cannon sewershed flows. This 15" line would then connect to the existing 15" collector along Moss Street. A conceptual drawing of this alternative is provided in Figure 4-11. Detailed cost estimates are provided in Appendix I. This alternative would have capital costs of site preparation, 12" and 15" PVC gravity sewer line, manhole assemblies, and ACP decommissioning. The capital cost is estimated at approximately $473,000.

4.8.3.2 Alternative 2 – Moss Street

A new 10" line would connect to the manhole at the north end of the undersized 8" collector of the Moss/Cannon sewershed. Then, a new 15" line would be constructed down Moss Street to connect the manhole at the intersection of 3rd Street and Moss Street to the north end of the 15" main collector on Moss. The 8" line that currently drains the manhole at this intersection to the Alder Street Lift Station sewershed would be abandoned, and the new 15" line would drain the Moss/Cannon sewershed and the approximately twenty properties that currently are served by the lift station to the main gravity collector. A new 12" gravity line would be constructed to replace the undersized 8" collector of the 1st street sewershed. A conceptual drawing of this alternative is provided in Figure 4-12. Detailed cost estimates are provided in Appendix I. This alternative would have capital costs of site preparation, 12" and 15" PVC gravity sewer line, manhole assemblies, and ACP decommissioning. The capital cost is estimated at approximately $470,000.

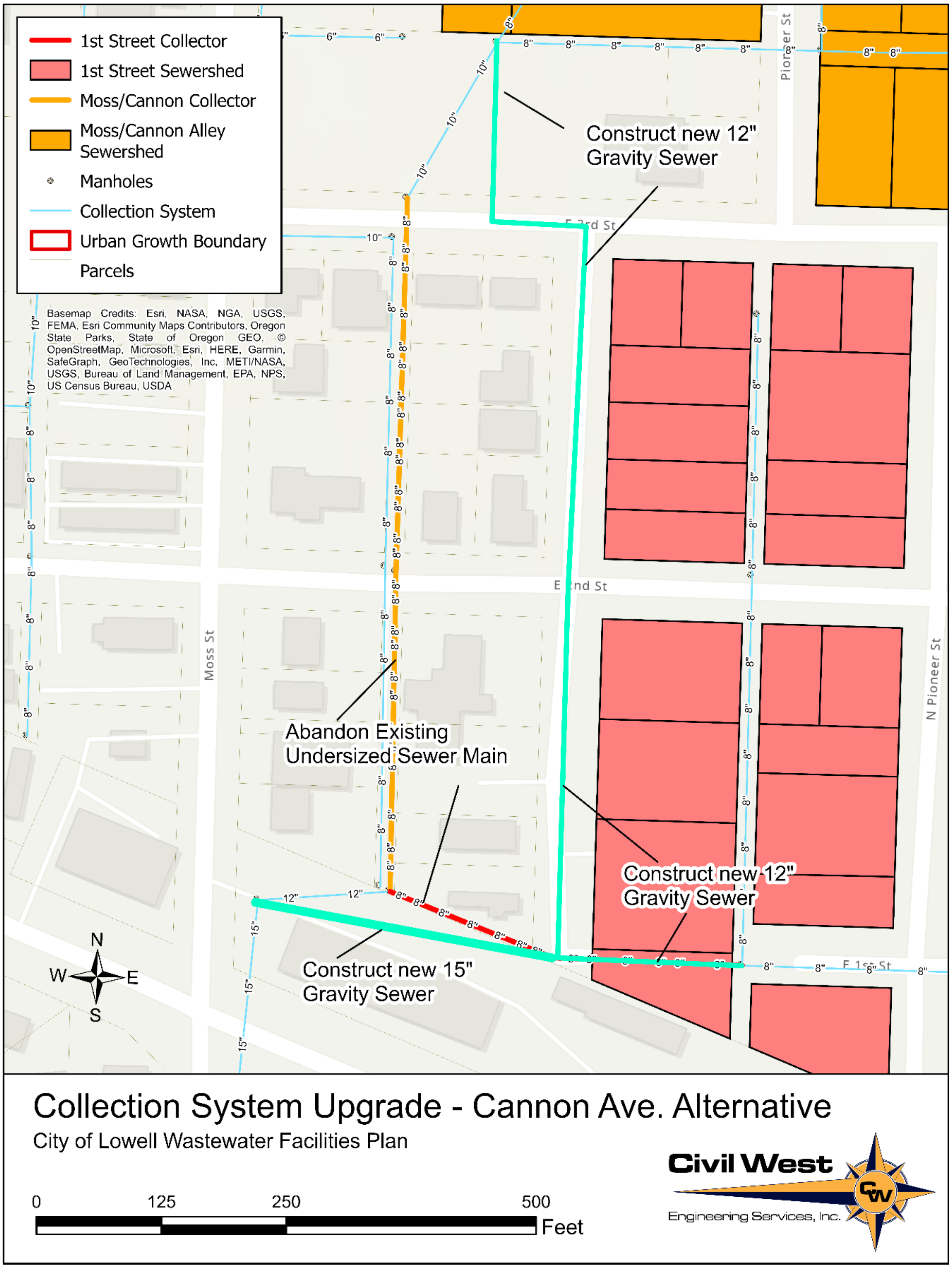


Figure 4-11: Cannon Avenue Collection System Alternative Conceptual Map

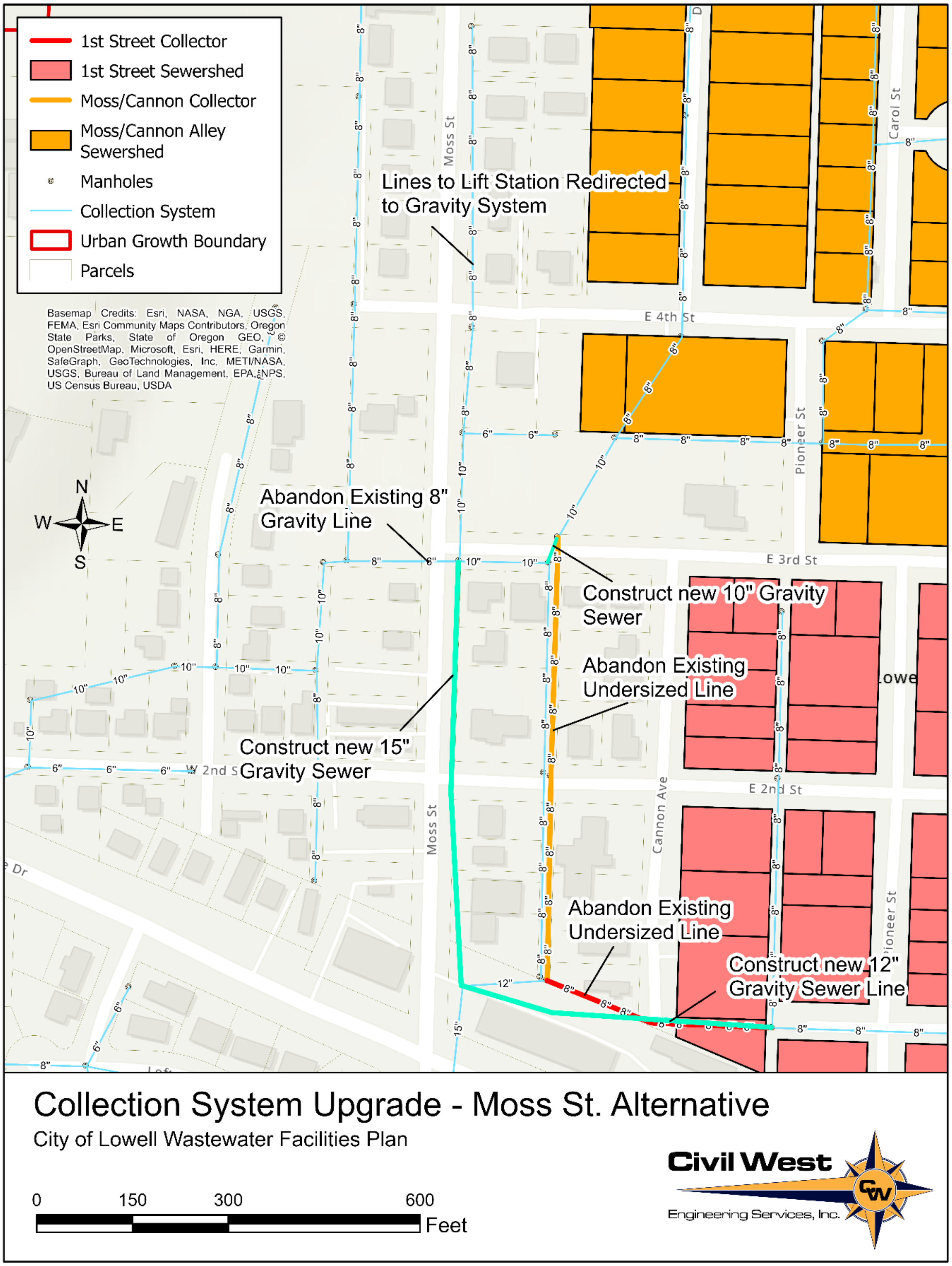


Figure 4-12: Moss Street Collection System Alternative Conceptual Map

5 Selection of Alternatives

This section presents the results of a life cycle cost analysis of alternatives discussed in Section 4, explains the scoring criteria for selection of the best alternatives for the City, and summarizes the results of the alternative evaluation.

5.1 Net Present Value Analysis

Table 5-1 summarizes the total life cycle cost (net present value) of each alternative. These costs consider O&M costs (chemical, electrical, and labor), capital costs, and salvage value of equipment. The net present value was calculated for each alternative as the sum of capital cost and the uniform series of annual O&M costs, minus present worth of the salvage value. Itemized estimates for each of these costs for each alternative are provided in Appendix I.

Table 5-1: Summary of Net Present Values (in 2024$) for each Viable Alternative

Alternative	Capital Costs	O&M Uniform Series	Salvage Present Worth	Net Present Value
Headworks				
"No Construction"	$0	$72,041	$0	**$72,041**
Add Redundant Fine Screen	$467,360	$66,318	$12,113	**$521,565**
Biological Treatment				
"No Construction"	$0	$950,799	$0	**$950,799**
Supplemental Alkalinity Addition	$175,840	$73,430	$404	**$248,866**
Redundant Secondary Clarifier	$1,506,640	$66,109	$70,662	**$1,502,087**
Trickling Filter - Activated Sludge Rehabilitation	$1,888,320	$1,048,908	$158,485	**$2,778,744**
Sequencing Batch Reactors	$4,280,000	$1,082,890	$444,161	**$4,918,729**
Conventional Activated Sludge	$1,376,000	$929,841	$149,400	**$2,156,441**
Extended Aeration System	$3,910,400	$982,165	$484,539	**$4,408,026**
Disinfection				
"No Construction"	$0	$537,333	$0	**$537,333**
Chlorine Disinfection - New Chlorine Contact Basin	$548,000	$452,165	$36,340	**$963,824**
Construct UV Disinfection System	$836,800	$233,245	$10,095	**$1,059,950**
Solids Management				
"No Construction"	$0	$58,538	$0	**$58,538**
Aerobic Digester Aeration System Improvements	$296,000	$200,706	$14,132	**$482,573**
Rehabilitate Drying Bed Underdrains	$46,520	$87,153	$5,885	**$127,788**
Reconstruction of Drying Beds with Guide Walls	$342,520	$61,154	$79,374	**$324,301**
Collection System				
Collection System - I/I Reduction	$301,552	$22,892	$22,006	**$302,438**
Alder Street Lift Station Upgrade	$376,000	$29,433	$16,824	**$388,608**
Capacity Upgrades - Cannon Avenue Alternative	$472,800	$8,830	$110,031	**$371,599**
Capacity Upgrades - Moss Street Alternative	$469,200	$8,830	$109,122	**$368,907**

5.2 Evaluation Criteria

The alternative improvement projects to include were evaluated using monetary and non-monetary considerations for inclusion in the City's Capital Improvement Plan (CIP).

5.2.1 Monetary Factors

Recommended improvement projects should be evaluated with regards to construction costs, operations and maintenance costs, and any retained value of infrastructure beyond its design life. The Net Present Value analysis summarized in Table 5-1 are inclusive of these costs. These cost estimates are at the planning level and have an inherent level of uncertainty, therefore, evaluation of non-monetary factors should also be performed.

5.2.2 Non-monetary Factors

For non-monetary factors, professional judgement was used as the basis of evaluation. Non-monetary criteria considered during the alternative evaluation process included:

- Ease of Operation – based on the annual O&M requirements and classification criteria (OAR 340-049-0025)
- Constructability – Based on estimated construction timelines, land requirements, and bypass treatment requirements
- NPDES Compliance – Based on the confidence of the alternative to meet the City's discharge permit obligations
- Permitting Requirements – Based on the type of permits likely required for construction
- Sustainability – Based on system resiliency, resource efficiency, and provision for future growth in the City
- Reduction of Greenhouse Gas Emissions – Based on electricity usage and requirement for transport of materials/chemicals

5.3 Alternative Selection

For many of the alternatives discussed in Section 4, including secondary treatment, solids management, disinfection, and collection system capacity improvements, it was necessary to choose the best alternatives for inclusion in the City's capital improvement plan (CIP) using the evaluation criteria described in Section 0. Three alternatives did not have feasible counterparts for comparison but are recommended to be included in the City's CIP. These projects are discussed below in terms of why the projects are necessary in lieu of full comparison of alternatives.

- **Alder Street Lift Station Capacity Upgrade** – The existing pumps in the lift station are undersized for the current and projected peak flows associated with storm events. These pumps are also past the typical design life of 20 years. The firm capacity of the existing lift station is not sufficient for current peak flows as evidenced by overflows in recent years. It is expected that similar issues will only become more frequent if the pumps are not upgraded. To comply with DEQ reliability requirements, each pump should be sized for the peak hour flow of 490 gpm.

- **Inflow and Infiltration Reduction Projects**: As part of the facility planning process, a thorough investigation of the collection system for direct sources of I/I was conducted (Appendix D). This analysis discovered direct sources of I/I and recommended direct fixes and further follow-up activities, including CCTV surveillance. These projects should be completed as part of regular wastewater facility maintenance throughout the next planning period. The implementation of an I/I reduction program will benefit the City's treatment process significantly, avoiding disruptions of the biological treatment system during extreme rain events. The City should start with the rehabilitation projects identified in this plan and continue to monitor I/I regularly to fix leaking pipes and manholes throughout the collection system as they arise.
- **Supplemental Alkalinity Addition**: The existing method of dosing soda ash at the end of the treatment train for pH compliance is not efficient or beneficial to the treatment process in general. By implementing an alkalinity addition system upstream of biological treatment, the City will enhance the nitrification capabilities of the WWTP and be better prepared for potential ammonia limits in the future. It is recommended at this time to use magnesium hydroxide because of the inability to overdose and burn out downstream biology, but alternative chemicals could be considered during the pre-design phase.

For the remainder of the alternatives listed in Table 5-1, many would otherwise not be necessary if one is chosen over others. The selection of the alternatives to include in the City's CIP are discussed in the following subsections. The subsections are ordered according to the type of facility: headworks, biological treatment, disinfection, solids management, and the collection system.

5.3.1 Headworks

5.3.1.1 Monetary Considerations

The economic costs of alternatives to address the effect of peak flow events on the existing headworks are summarized in Table 5-2. The two alternatives involved the "no construction" alternative, which involves keeping the existing headworks structure as is throughout the planning period and installing an additional fine screening unit to reduce use of the bypass channel.

Table 5-2: Life Cycle Costs of Headworks Alternatives

	No Construction	Additional Fine Screen
Capital Cost	$0	$467,360
Annual O&M Costs	$4,406	$4,056
Salvage Value	$0	$18,000
Net Present Value	$72,041	$521,565

5.3.1.2 Non-monetary Considerations

<u>Ease of Operation:</u> The scenario where the City would need two mechanical screens in operation at the same time is very rare, occurring only during the 5-year storm during peak hour based on how the characteristic flows are defined. The reduced maintenance costs for a second

mechanical screen compared to manually cleaning the bar-screen in the parallel channel does not outweigh the high capital cost considering the rarity of this scenario. It is possible that a reduction in I/I could eliminate the need for more screening capacity.

Constructability: Bypass treatment would be required while the headworks channel is modified to include a new channel. It is estimated that construction could be completed in one dry season, with a bypass treatment duration of about 3 months.

Compliance Issues: There are no significant differences for these two alternatives.

Permit Requirements: Coordination with ACE would be required for all work performed on the WWTP lot since the City leases the land. They would require an ACE Individual Permit for any construction activities on the property.

Sustainability: There are no significant differences in sustainability considerations for these two alternatives.

Reduction of Greenhouse Gas Emissions: There are no significant differences for these two alternatives.

5.3.1.3 Alternative Selected

The costs of expanding the headworks do not outweigh the benefits considering how infrequently both units would be necessary, and the City has adequate screening capacity with both the mechanical screen and the bypass channel. Use of the bypass channel reduces ease of operation because the bar racks must be manually cleaned; however, given the low frequency that the bypass channels used, the improvements to ease of operation are not significant enough to justify the capital expense. It is recommended that the City prioritize I/I reduction by following the recommendations in this plan to reduce severity of peak flow events.

5.3.2 Biological Treatment

Issues with the existing biological treatment system include a lack of redundant clarification capacity, observed treatment performance issues, insufficient hydraulic capacity in the aeration basin, and the need for alkalinity addition. Six alternatives were considered feasible, not including the “no construction” option since the existing system does not consistently meet current requirements. These alternative projects were evaluated in four groups depending on necessity:

- **TF/AS – Trickling Filter Activated Sludge System:** Involves construction of an aeration basin the same size as the existing solids contact aeration channel, and construction of a new, redundant secondary clarifier. The trickling filter would continue to be operated as existing, the aeration basin capacity would be expanded, and a more appropriately sized clarifier would be constructed. This includes the cost of alkalinity addition, a redundant secondary clarifier, and the trickling filter/activated sludge rehabilitation alternatives.
- **SBRs – Sequencing Batch Reactors**: Involves complete decommissioning of the existing biological treatment system and construction of sequencing batch reactors. Secondary clarifiers are not necessary with this type of system. This includes the cost of the sequencing batch reactors and alkalinity addition.
- **CAS – Conventional Activated Sludge System:** Involves decommissioning the primary clarifiers and converting them into activated sludge aeration basins and construction of a new secondary clarifier. The costs are inclusive of converting the

primary clarifiers into aeration basins, constructing a new secondary clarifier, and constructing an alkalinity addition system.

- **EA/AS – Extended Aeration Activated Sludge**: This would involve converting the WWTP into an activated sludge system with an extended aeration configuration that uses larger aeration basins and longer solids retention times. The primary clarifiers would be maintained, and proprietary units would be constructed in the footprint of the existing trickling filter/solids contact system. The redundant clarifier alternative is not necessary with this option because a clarifier is included as part of the proprietary unit. This includes the cost of package/proprietary system and alkalinity addition.

5.3.2.1 Monetary Considerations

The alternatives to upgrade the biological treatment system to provide necessary treatment capacity and redundancy are summarized in Table 5-3.

Table 5-3: Life Cycle Costs of Biological Treatment Alternatives

	TF/AS	SBRs	CAS	EA/AS
Capital Cost	$3,571,000	$4,456,000	$3,058,000	$4,086,000
Annual O&M Costs	$73,000	$71,000	$65,000	$65,000
Salvage Value	$341,000	$661,000	$328,000	$721,000
Net Present Value	$4,530,000	$5,168,000	$3,907,000	$4,657,000

5.3.2.2 Non-monetary Considerations

<u>Ease of Operation:</u> The CAS and EA/AS options are tried and tested technologies for wastewater treatment plants of similar size to Lowell. Many resources are available for operation and troubleshooting of these systems. OAR 340-049-0025 considers Activated Sludge systems as 15 Points towards the total system classification score.

The TF/AS alternative would address concerns with redundancy, but adding multiple units of the same type as existing may only exacerbate the operational difficulties that the facility currently experiences. The doubling of aeration basin volume would likely cause the classification of the system to change from Trickling Filter – Solids Contact to Trickling Filter – Activated Sludge; this could result in a classification score of 22 (Low Rate Trickling Filter + Activated Sludge).

SBR technology has become more prevalent over the past couple of decades and there are many technical resources available now to help operate these systems, but they do require more complex controls and operator attention to various reaction cycles. It is a concern that Lowell's small Public Works team may not have the bandwidth for a system that should have frequent supervision. SBRs do not have their own classification grouping; it is assumed that they would be scored the same as Activated Sludge system (15 points).

<u>Constructability:</u> The CAS option would require bypass primary clarification while the primary clarifiers are converted into aeration basins to prevent overloading of solids to the trickling filter. It is estimated that this conversion, which would involve minor concrete work and installation of aeration equipment, could occur over one dry season. No bypass treatment would be required to decommission the trickling filter and construct a secondary clarifier, because the existing clarifier could be used until the new one is brought online. This would likely require another dry season. The full conversion would take about two years, including approximately 5 months of bypass treatment.

The EA/AS system would require full bypass treatment for the entire construction period because the existing system would need to be decommissioned to make room for the new basins. These system are typically done in lagoon conversions because of the footprint required. It is estimated that this would take about three years to construct, with bypass treatment occurring for at least two years.

TF/AS system could likely be constructed without the need for any bypass treatment. There would also be no need to decommission any of the existing infrastructure. Due to the amount of grading and concrete work, it is expected that this could be constructed over two dry-seasons.

Constructing SBRs would have similar issues as the EA/AS alternative due to the limited space available on the existing lot. The City should plan to have bypass treatment for the entire project duration to decommission existing system and make room for the new basins. It is estimated that this would take about three years to construct, with bypass treatment occurring for at least two years.

<u>Compliance Issues:</u> All of these alternatives would be able to meet the City's discharge permit obligations. The TF/AS system, which is similar to the system that is underperforming currently, would have the most issues as flows increase due to growth. While the addition of aeration capacity will help with redundancy and peak flows, the impacts of seasonal flow variability on the trickling filter would not be addressed by this alternative.

<u>Permit Requirements:</u> Coordination with ACE would be required for all work performed on the WWTP lot since the City leases the land. They would require an ACE Individual Permit for any construction activities on the property.

A building permit filed with Lane County would be required for modifications or erection of building structures, as would be required to house the aeration equipment for all of these alternatives. It is possible that the CAS alternative could avoid construction of a blower building and use surface mounted aerators; this would need to be evaluated in more detail during a pre-design phase.

<u>Sustainability:</u> The electricity costs of the CAS, EA/AS, and SBR alternatives are comparable, with the TF/AS system having an advantage because drafting air up through the trickling filter is less energy intensive than blowers and diffusers.

<u>Reduction of Greenhouse Gas Emissions:</u> There are no significant differences between these alternatives.

5.3.2.3 Alternative Selected

The lowest cost alternative is conversion to a conventional activated sludge system. For the non-economic criteria considered, the TF/AS system is better than the low-cost alternative with regard to not requiring bypass treatment and lower electricity usage. However, the CAS system would involve less unit operations and is the simplest to operate of all alternatives considered.

With all of these considerations, the conversion to conventional activated sludge is the recommended alternative. The primary clarifiers are not necessary for the City given the low solids loading rates in the influent wastewater. Furthermore, conversion of existing infrastructure is much more cost effective compared to the major excavation and installation costs associated with an entirely new system. The ability for the City to operate one aeration basin or two in parallel given the high flow variations experienced at the WWTP would provide much greater flexibility than the existing system.

5.3.3 Disinfection

5.3.3.1 Monetary Factors

The costs associated with the two alternatives to upgrade the existing disinfection system are presented in Table 5-4.

Table 5-4: Life Cycle Costs of Disinfection Alternatives

	Ultraviolet	Chlorine
Capital Cost	$837,000	$548,000
Annual O&M Costs	$14,000	$28,000
Salvage Value	$15,000	$54,000
Net Present Value	$1,060,000	$964,000

5.3.3.2 Non-monetary Considerations

Ease of Operation: Generally, UV systems are easier to operate because they only require power; the City could avoid inventory management of chlorination and dechlorination chemicals. Liquid chlorine disinfection plus dechlorination has a total system classification score of 6, and UV disinfection systems have a score of 5.

Constructability: Because both proposed alternatives include building a new structure, no bypass treatment or significant construction issues are expected. It is possible that some cost savings could be realized by using the existing dechlorination channel for placing the UV equipment because it is an appropriate size (2 feet wide and 4 feet deep), but this would require bypass treatment. This would need to be evaluated further during pre-design; at this time the more conservative cost estimate for building a new structure for the UV system is evaluated.

Compliance Issues: Both of these alternatives would be able to meet the City's discharge permit obligations. Verification of 65% UV transmittance would be required prior to final design of a UV system.

Permit Requirements: Coordination with ACE would be required for all work performed on the WWTP lot since the City leases the land. They would require an ACE Individual Permit for any construction activities on the property.

Sustainability: While the electricity costs of UV systems are higher than what is required for chemical dosing pumps, the total O&M costs are overall lower because the City would not be purchasing hypochlorite and dechlorination chemicals. A UV system would reduce the City's reliance on outside vendors for chemical sales; given the market instability in chlorine products since the COVID-19 pandemic, it is recommended to use other disinfection alternatives when possible.

Reduction of Greenhouse Gas Emissions: There are no significant differences between these alternatives. The higher cost of electricity for the UV system is offset by emissions from the manufacturing and transport of chemicals for the chlorine-based system.

5.3.3.3 Alternative Selected

While the estimated costs of the UV system are approximately 10% larger than the chlorine disinfection alternative, the reduced O&M costs, ease of operation, sustainability considerations, and reducing the City's reliance on an unstable chlorine market make the UV system the recommended alternative.

5.3.4 Biosolids Management

The City's issues with the existing biosolids system include an unoptimized aeration system in the aerobic digester, and deep-pit sand drying beds that are difficult to maintain. The feasible alternatives were grouped together as follows for evaluation:

- **Optimize Existing System:** Maintains the drying beds as they currently exist, replace the broken underdrain piping, and improve the aerobic digester aeration system as recommended previously. This includes the cost of the "No Construction" solids management alternative, the underdrain replacement project, and the aerobic digester aeration upgrades as listed in Table 5-1.
- **Improve Drying Beds**: Improves the aeration system of the digester and constructs new drying beds with concrete guide walls that are easier to maintain within the footprint of the existing system. This includes the cost of the new drying beds and the aerobic digester aeration upgrade alternatives.

5.3.4.1 Monetary Factors

The life cycle costs for these alternatives are provided in Table 5-5.

Table 5-5: Scoring of Biosolids Management Alternatives

	Optimize Existing System	Improve Drying Beds
Capital Cost	$340,000	$640,000
Annual O&M Costs	$21,000	$16,000
Salvage Value	$30,000	$140,000
Net Present Value	$670,000	$810,000

5.3.4.2 Non-monetary Considerations

<u>Ease of Operation:</u> The existing drying beds are very difficult to maintain, especially when removing the final biosolids products because of inadequate protection of the underdrain piping and the plastic liner. Installation of guide walls and underdrain protection would significantly reduce labor requirements.

<u>Constructability:</u> The City has ample aerobic digester capacity to store biosolids to implement either of these alternatives.

<u>Compliance Issues:</u> Both of these alternatives would be able to meet the City's discharge permit obligations.

<u>Permit Requirements:</u> Coordination with ACE would be required for all work performed on the WWTP lot since the City leases the land. They would require an ACE Individual Permit for any construction activities on the property.

<u>Sustainability:</u> Optimization of the aerobic digester aeration system will save the City significantly in electricity costs, making it so that the cells could be isolated as originally

intended. The labor requirement of the existing drying beds are significant, and because of inadequate protection of the underdrain system, replacement costs are a reoccurring liability.

Reduction of Greenhouse Gas Emissions: There are no significant differences between these alternatives.

5.3.4.3 Alternative Selected

The recommended alternative for the City is to improve the aeration system and construct improved sludge drying beds. Ease of operation and equipment sustainability are the prime factors that justify the expense of new drying beds over the existing deep pits the City currently uses. The current dried solids removal process has resulted in damage to the under-drain system and liner in the past planning period due to the difficulty entering the bed with the excavator. By constructing a ramp for the excavator to enter the bed, and protecting the underdrains with steel grates or rails, the risk of damage to the liner and underdrain system is reduced. Second, with the proposed three-channel configuration of the proposed drying beds, the WWTP would have the capacity to use two beds year-round and have a third bed available for contingency.

5.3.5 Collection System

As discussed in Section 4.8.3, there are two feasible alternatives to handle the capacity issues associated with the collector pipes that serve the north and east areas of the City.

5.3.5.1 Monetary Factors

The costs associated with these alternatives are presented in Table 5-6. Given the uncertainty of cost estimates at the planning level, these projects are considered equal (<1% difference) in terms of life cycle cost.

Table 5-6: Scoring of Collection System Capacity Alternatives

	Cannon Avenue	Moss Street
Capital Cost	$476,000	$470,000
Annual O&M Costs	$500	$500
Salvage Value	$164,000	$162,000
Net Present Value	$372,000	$369,000

5.3.5.2 Non-monetary Considerations

Ease of Operation: There are no significant differences between these alternatives.

Constructability: There are no significant differences between these alternatives.

Compliance Issues: The Cannon Avenue alternative will reduce the number of properties that drain to the Lift Station, reducing the chance of overflows and violation of the City's NPDES permit.

Permit Requirements: There are no significant differences between these alternatives.

Sustainability: There are no significant differences between these alternatives.

Reduction of Greenhouse Gas Emissions: There are no significant differences between these alternatives.

5.3.5.3 Alternative Selected

The recommendation is the Cannon Avenue alternative. It is advantageous in that it would result in a significant number of properties being rerouted from the Alder Street Lift Station sewershed and onto the gravity system. This would help with the reduction of flows to the lift station, reducing pump run times and reducing the risk of overflows.

6 PROPOSED PROJECTS

This section summarizes the proposed wastewater facility improvement projects recommended for inclusion in the City's Capital Improvement Plan (CIP). A recommended phasing and funding plan is presented, as well as a summary of funding sources available to the City for implementing the CIP over the next planning period.

6.1 Improvement Project Recommendations

Through the analyses that were completed during this planning effort, numerous project recommendations have been developed. These recommendations include improvements to the WWTP and collection system. The current plant flow diagram can be seen in Figure 6-1.

6.1.1 Wastewater Treatment Plant

6.1.1.1 Activated Sludge Aeration Basins

The primary clarifiers will be converted to aerated basins. This will change the City's treatment paradigm from trickling filter/solids contact to conventional activated sludge, which is appropriate for a growing City that experiences major seasonal variations in flow. Both basins would be equipped with fine-pore air diffusers and two new blowers for aeration, in addition to underground air and sludge piping. A conceptual drawing of this project is provided in Figure 6-2. Additionally, the supplemental alkalinity dosing system would be installed at the time of this project to provide ammonia removal capacity in the new aeration basins.

6.1.1.2 Aerobic Digester Aeration Improvements

A new blower for the aeration system that serves the solids stabilization process is recommended to allow the operator to isolate the digester basins. The current configuration requires the diffusers for both basins to be run in conjunction. A conceptual plan for this project is presented in Figure 6-3. This will save the City considerably in electricity expenditures throughout the planning period, so it is recommended to complete the project as soon as possible.

6.1.1.3 Secondary Clarifier

The trickling filter is to be decommissioned, demolished, and a new secondary clarifier would be constructed in the available pad. This new clarifier would have an internal diameter of 28 feet, appropriately sized for the City's typical flows throughout the planning period. Activated sludge recycle and waste streams will be directed to the existing solids contact aeration channel where the RAS and WAS splitter box is currently located, and RAS and WAS will be sent to the new aeration basins or aerobic digester respectively. Figure 6-4 shows these recommendations in a conceptual drawing.

6.1.1.4 UV Disinfection

The existing chlorine disinfection system is to be replaced with a UV disinfection system, as shown in Figure 6-5. The UV disinfection basin will consist of two channels, each two feet wide.

The basin is to be located south of the chlorine contact chamber. The use of UV disinfection will significantly reduce chemical expenditures in the treatment process improving the sustainability of the system.

6.1.1.5 Sludge Drying Bed Improvements

This project involves construction of concrete guide walls and replacement of the underdrain system to divide the existing pit-style drying beds into three 1,500 square-foot beds. Each bed will have an entrance ramp to allow for ease of entry for machinery needed for solids removal, and the guide walls will provide protection for the liner and underdrain system. Figure 6-6 shows these recommendations.

6.1.2 Collection System

6.1.2.1 Alder Street Lift Station Upgrades

The City will upgrade the capacity of the lift station to meet DEQ's reliability standards. This will necessitate replacement of both pumps. Each pump will be sized to meet a projected peak flow of 490 gpm and be equipped with variable frequency drives.

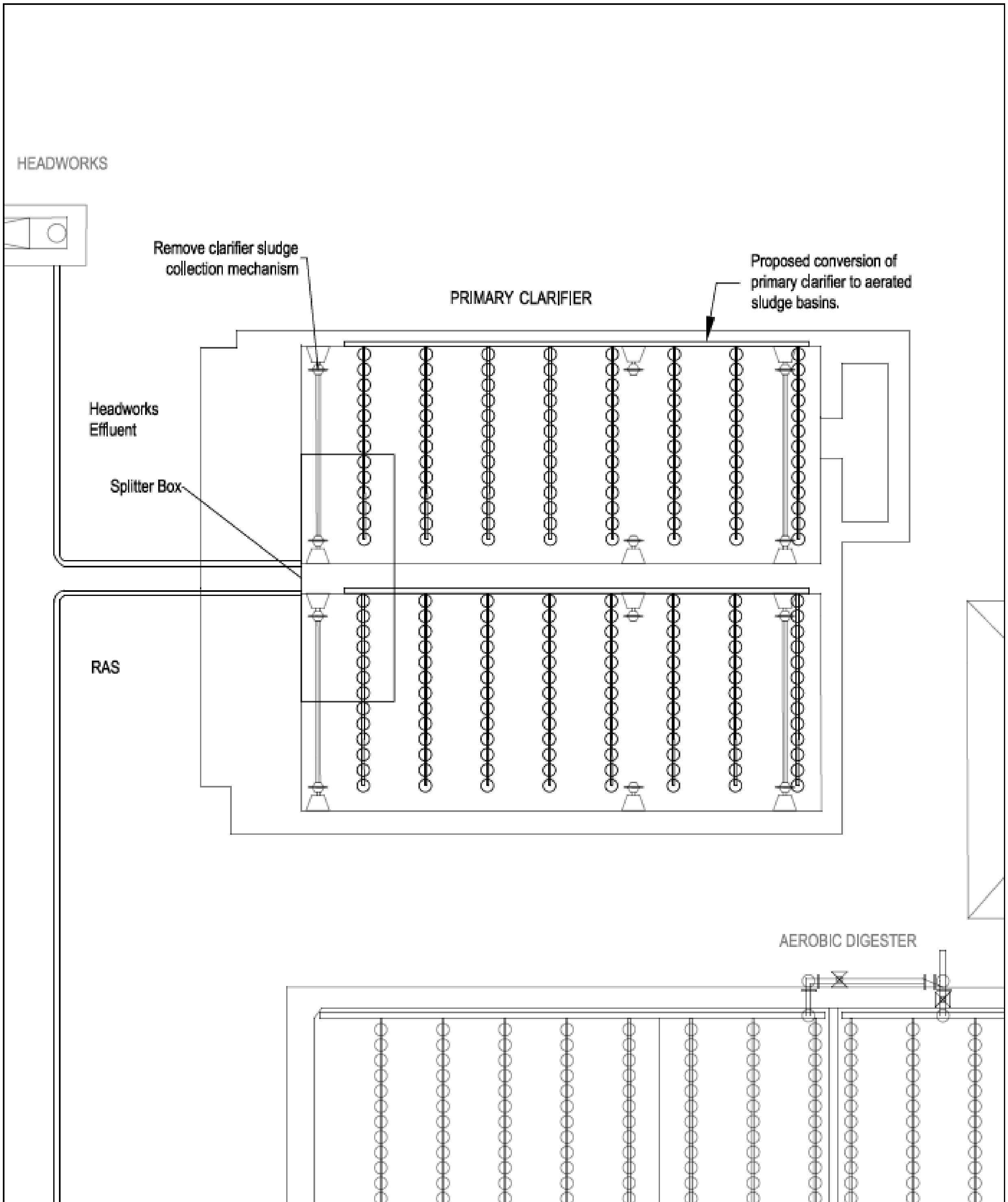
6.1.2.2 Moss Street Gravity Sewer Capacity Upgrades

This project would involve upgrading two pipes in the collection system that are undersized for future growth, while also transitioning approximately 20 properties from the lift station basin to the gravity collection system. The City's main 15" gravity collector on Moss Street would be extended up to 3rd Street, and minor pipe improvements would connect the properties in the north and east portion of town to this collector.

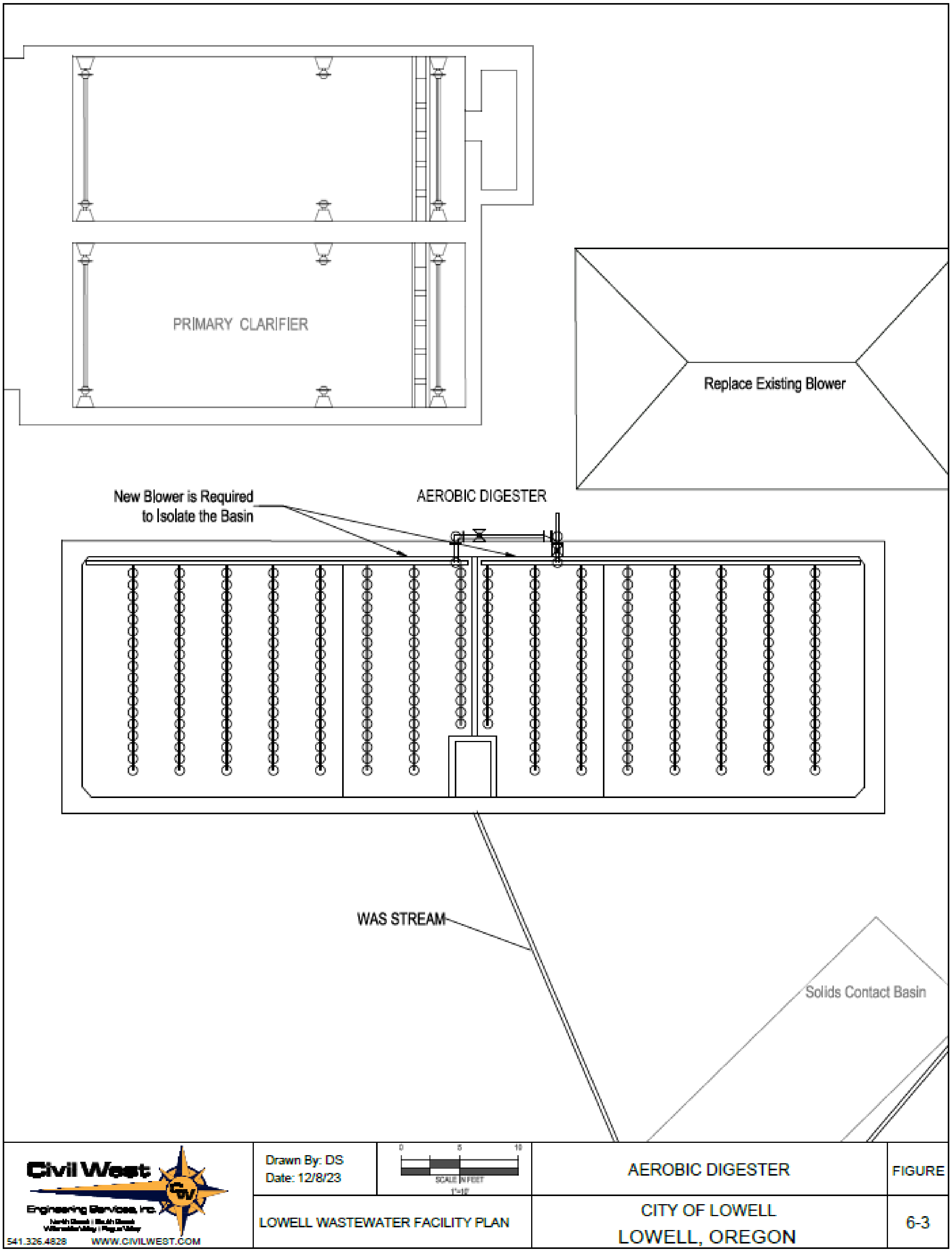
6.1.2.3 Inflow and Infiltration Reduction Program

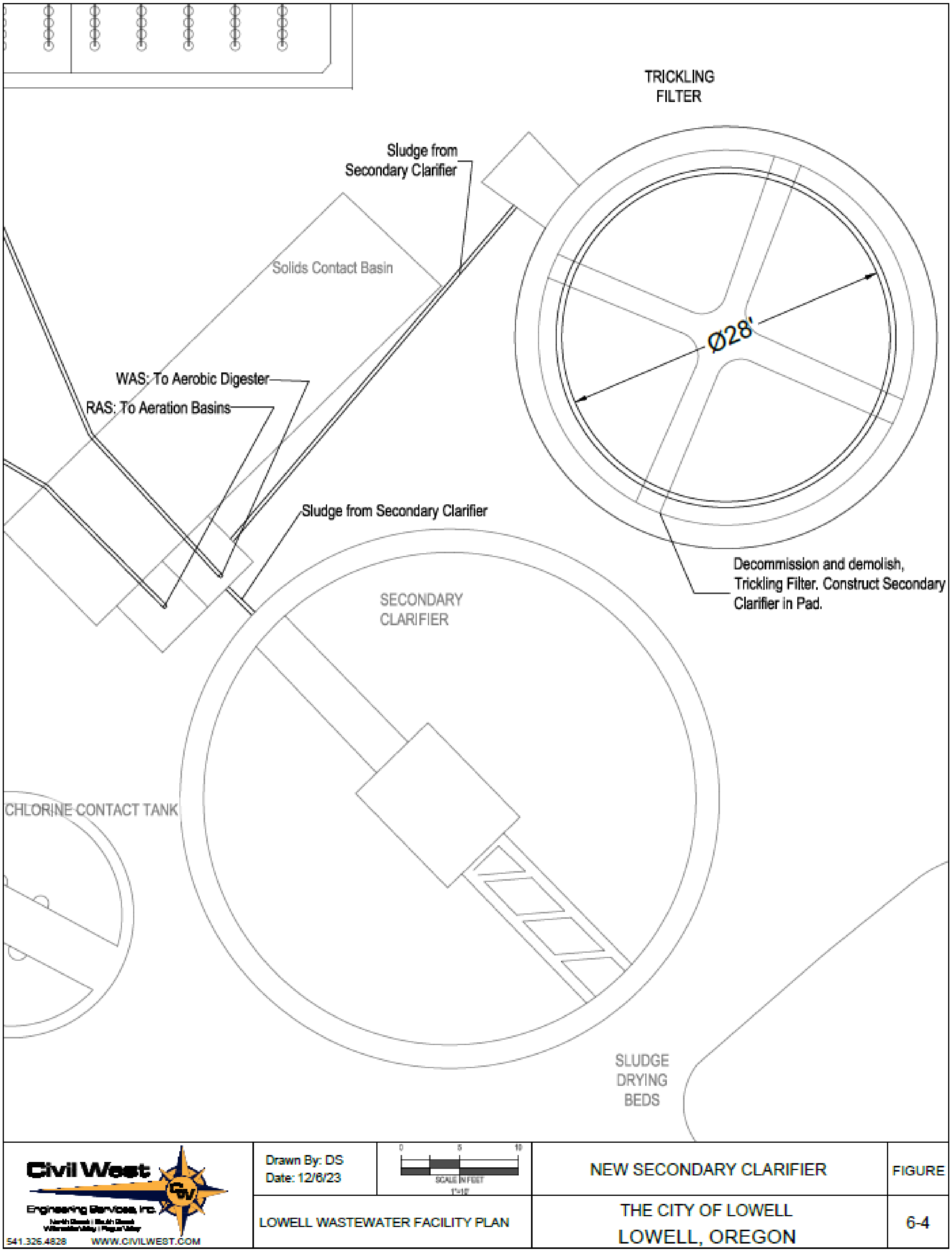
The City should budget approximately $25,000 per year for the period 2024-2028 to fix the identified I/I sources in the collection system. This includes pipe-lining projects near the Alder Street Lift Station, repair of cross-connected storm drains on Moss Street, and manhole replacement projects in the gravity sewer collection system.

The City should continue to budget approximately $13,000 annually for I/I reduction from 2028-2045. This will involve routine CCTV surveillance of pipes and repairs to pipes and manholes as needed. The City has the authority to require repairs and rehabilitation of private sewer laterals, which is necessary because these systems comprise a large portion of the sewer collection system. The requirement of private laterals to be maintained to ensure I/I is minimized is prohibited in Title 4 Code 4.215. Violations of this code carry penalties of a Class B Violation and may be cited into the Lowell Municipal Court. Regular smoke testing will help the City identify private laterals in need of repair.



Civil West Engineering Services, Inc. 541.326.4828 WWW.CIVILWEST.COM	Drawn By: DS Date: 12/11/23	SCALE IN FEET	PRIMARY CLARIFIERS	FIGURE
	LOWELL WASTEWATER FACILITY PLAN		CITY OF LOWELL LOWELL, OREGON	6-2


PRIMARY CLARIFIER
Replace Existing Blower
New Blower is Required to Isolate the Basin
AEROBIC DIGESTER
WAS STREAM
Solids Contact Basin
Civil West Engineering Services, Inc.
541.326.4828 WWW.CIVILWEST.COM
Drawn By: DS
Date: 12/8/23
SCALE IN FEET
AEROBIC DIGESTER
FIGURE
LOWELL WASTEWATER FACILITY PLAN
CITY OF LOWELL
LOWELL, OREGON
6-3


TRICKLING FILTER
Sludge from Secondary Clarifier
Solids Contact Basin
Ø28'
WAS: To Aerobic Digester
RAS: To Aeration Basins
Sludge from Secondary Clarifier
Decommission and demolish, Trickling Filter. Construct Secondary Clarifier in Pad.
SECONDARY CLARIFIER
CHLORINE CONTACT TANK
SLUDGE DRYING BEDS
Civil West
Engineering Services, Inc.
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WWW.CIVILWEST.COM
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Date: 12/6/23
0
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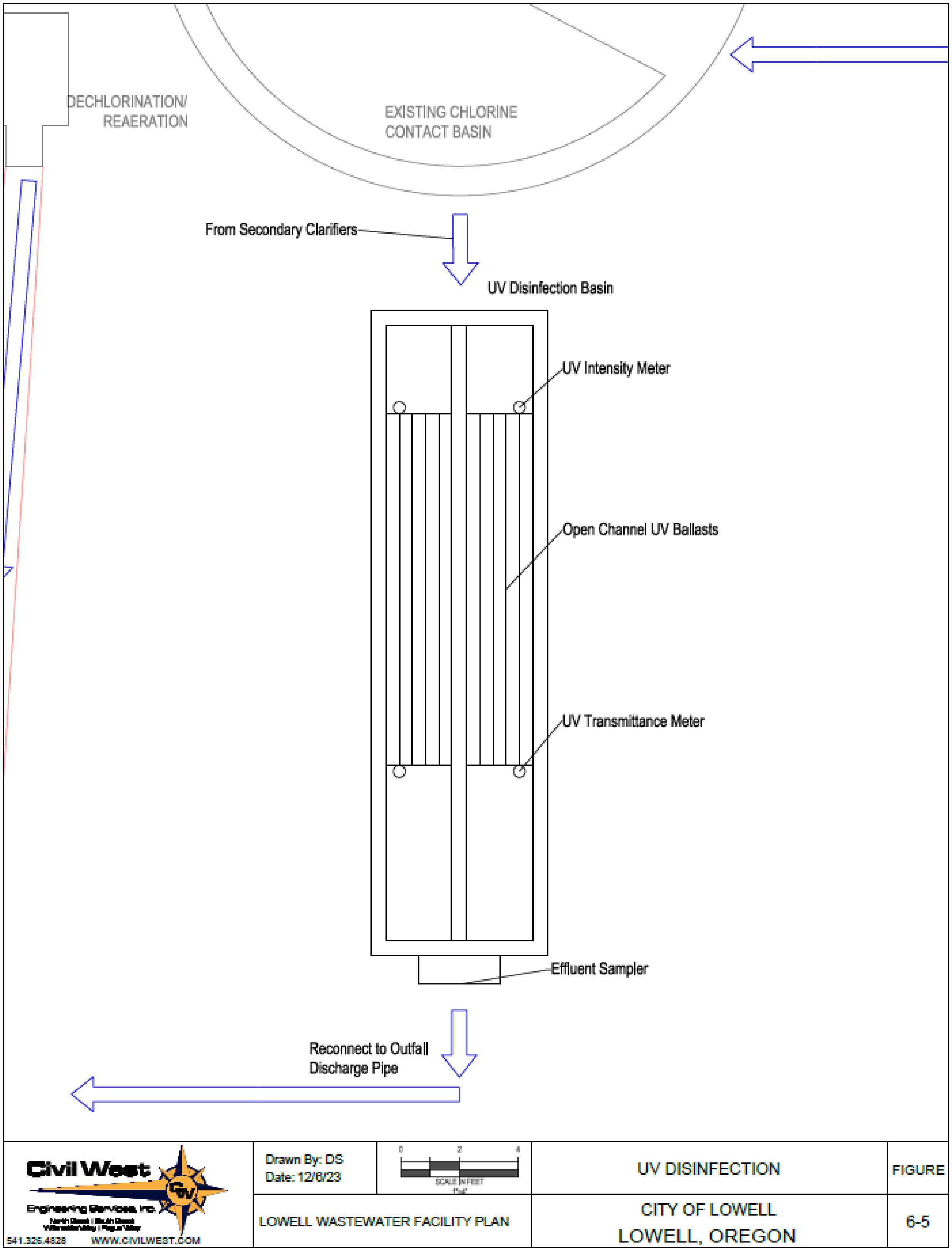
10
SCALE IN FEET
NEW SECONDARY CLARIFIER
FIGURE
LOWELL WASTEWATER FACILITY PLAN
THE CITY OF LOWELL
LOWELL, OREGON
6-4


DECHLORINATION/
REAERATION
EXISTING CHLORINE
CONTACT BASIN
From Secondary Clarifiers
UV Disinfection Basin
UV Intensity Meter
Open Channel UV Ballasts
UV Transmittance Meter
Effluent Sampler
Reconnect to Outfall
Discharge Pipe
Civil West
Engineering Services, Inc.
541.326.4828
WWW.CIVILWEST.COM
Drawn By: DS
Date: 12/6/23
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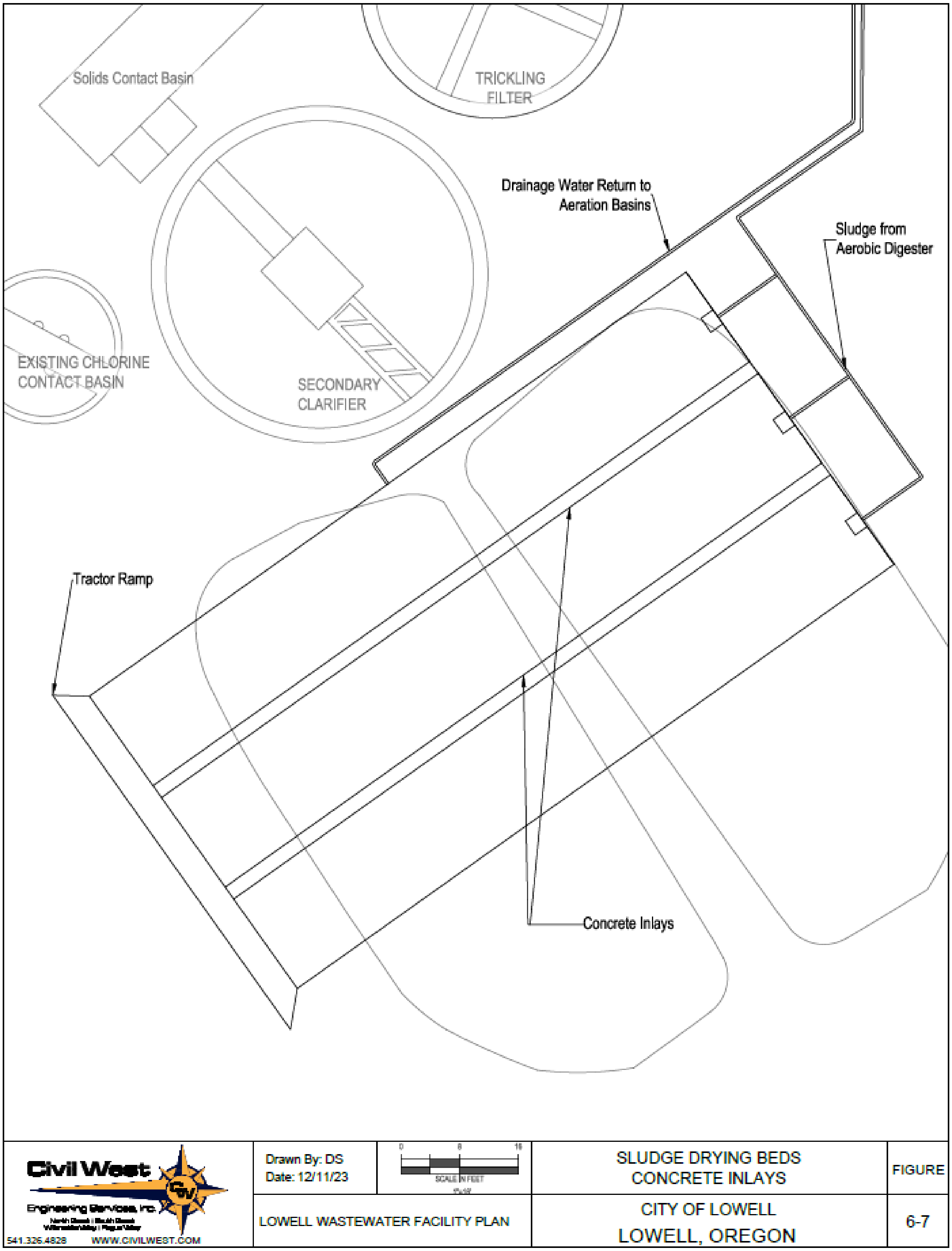
UV DISINFECTION
FIGURE
LOWELL WASTEWATER FACILITY PLAN
CITY OF LOWELL
LOWELL, OREGON
6-5


Solids Contact Basin
TRICKLING FILTER
Drainage Water Return to Aeration Basins
Sludge from Aerobic Digester
EXISTING CHLORINE CONTACT BASIN
SECONDARY CLARIFIER
Tractor Ramp
Concrete Inlays
Civil West
Engineering Services, Inc.
541.326.4828
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SCALE IN FEET
SLUDGE DRYING BEDS
CONCRETE INLAYS
FIGURE
LOWELL WASTEWATER FACILITY PLAN
CITY OF LOWELL
LOWELL, OREGON
6-7

6.2 Capital Improvement Plan

The recommended CIP for the City's wastewater utility is summarized in Table 6-1.

Table 6-1: Recommended Capital Improvement Plan

Capital Improvement Plan: Budgetary Costs (2024$) and Schedule			
Collection System Improvements - I/I Reduction	**Budget Cost**	**Begin and Complete By**	
Collection System - Spot Repair of Sewer Pipe Voids	$24,000	2024	2026
Collection System - Cross-Connection Repair	$168,000	2024	2028
Collection System - Manhole Rehabilitation	$87,200	2024	2030
Collection System - CCTV Surveillance	$22,400	2024	2045
I/I Reduction Budget	**$301,600**	***2024***	***2045***
PHASE 1 - Aeration System Improvements			
WWTP - Aeration System Improvements	$296,000	2024	2026
Phase 1 Budget	**$296,000**	***2024***	***2026***
PHASE 2 - Lift Station Upgrade and Biosolids Improvements			
WWTP - Biosolids Management Improvements	$342,500	2025	2030
Collection System - Alder Street Lift Station Upgrades	$376,000	2025	2030
Phase 2 Budget	**$718,500**	***2025***	***2030***
PHASE 3 - Wastewater Treatment System Upgrades			
WWTP - Activated Sludge Improvement Project	$1,376,000	2028	2032
WWTP - Secondary Clarifier Construction	$1,507,000	2028	2032
WWTP - Supplemental Alkalinity System	$176,000	2028	2033
WWTP - UV Disinfection System Installation	$564,800	2033	2040
Phase 3 Budget	**$3,623,800**	***2028***	***2040***
PHASE 4 - Collection System Capacity Upgrades			
Collection System - Gravity Sewer Improvements	$469,200	2030	2045
Phase 4 Budget	$469,200	***2030***	***2045***
Total CIP Budget	**$5,409,100**		

6.2.1 Improvement Project Phasing

The recommended projects were grouped into two categories: I/I reduction projects and improvement projects. Improvement projects were further divided into four phases to help the City plan and fund the capital projects in a sensible and cost-effective way.

The recommended I/I reduction projects should start with repairing the most egregious I/I issues identified in the collection system: repair of the broken pipes that go into the Alder Street Lift Station wet well, repair of cross-connections with the storm drainage system, and manhole repairs in the order presented in Table 4-11. The highest priority I/I improvements should be completed by end of year 2028. The City should continue to budget for I/I reduction projects after this and complete until all of the recommended manhole and pipe rehabilitation projects identified in the I/I evaluation are completed, and also continue to implement an I/I reduction program via routine CCTV surveillance of sewer pipe and repairing issues as they are identified throughout the entirety of the planning period.

The first phase of improvement projects is considered “low hanging fruit” in the sense that the total estimated cost is relatively low, and the benefits would be immediately beneficial to the City’s wastewater facilities. Phase 1 consists of optimizing the aerobic digester aeration system to save considerable O&M costs. It is recommended to begin engineering and design in 2024 and complete the aeration improvements by Summer 2025

Phase 2 improvement projects are considered high priority. These projects address capacity issues with the Alder Street Lift Station and improve the sludge drying beds to dramatically improve the WWTP’s solids management system by reducing labor and material requirements required for maintaining the existing drying beds. It is recommended to begin design and engineering of Phase 2 projects by end of year 2025, and finish construction before 2030.

Phase 3 projects are those associated with the upgrade of the WWTP to convert the existing trickling filter/solids contact system into a conventional activated sludge system. This will involve the conversion of the primary clarifier to aeration basins, the construction of a new secondary clarifier, installation of the supplemental alkalinity system, and construction of a new UV disinfection system. The beginning date of this project will likely depend on the City’s ability to obtain funding, but it is recommended to begin working on this phase prior to 2030 and complete the treatment system conversion before 2040.

Phase 4 involves the final CIP items for the City to implement in the second half of the planning period. This includes completion of the recommended gravity sewer capacity upgrades. This phase should be completed before the end of the planning period in 2045.

6.2.1.1 Permit Requirements

Building permits and grading permits will be required for each project involving rehabilitation of existing or construction of new structures on the wastewater treatment plant property, and at the Alder Street Lift Station. Plans for traffic control will be required for manhole rehabilitation projects and any pipe-laying work done for collection system capacity upgrades.

6.3 Financing

6.3.1 Annual Operating Budget

A review of the previous four years of the City’s sewer fund was presented in Section 2.5. The City generally budgets between $400,000 to $500,000 for the City’s sewer facilities, inclusive of capital projects, debt service, and operations and maintenance costs of the WWTP and collection system.

6.3.1.1 Income

Income for the facilities is provided from rates charges to customers. The rates are charged by equivalent dwelling unit (EDU). The basic monthly service charge per EDU is $68.51 with a greywater disposal fee per gallon of $0.17.

6.3.1.2 Annual Operations and Maintenance Costs

An itemized estimate of O&M costs of proposed projects is presented in Table 6-2. The existing wastewater system’s annual O&M costs are estimated at approximately $209,000. With the system upgrades, this should reduce to approximately $183,000. The expected decrease in O&M costs is a function of simplifying the treatment system, reducing chemical costs from disinfection, improving efficiency in the solids dewatering system, and reducing air requirements from the solids digestion process.

Table 6-2: Estimated Operations and Maintenance Costs of Proposed Wastewater Facilities

Operations & Maintenance - Headworks					
No.	**Item Description**	**Quantity**	**Units**	**Unit Cost ($)**	**Item Cost ($)**
1	Operator Labor - Existing Fine Screen	65	h	$40	$2,600
2	Operator Labor - Bar Rack Maintenance	20	h	$40	$800
3	Replacement Parts	1	LS	$500	$500
4	Electricity Usage	6000	kWh	$0.08	$506
Operations & Maintenance - Supplemental Alkalinity System					
No.	**Item Description**	**Quantity**	**Units**	**Unit Cost ($)**	**Item Cost ($)**
1	Operator Labor	32	h	$40	$1,280
2	Electricity Usage	2500	kWh	$0.08	$211
3	MgOH Costs	1000	gal	$3.00	$3,000
Operations & Maintenance - Secondary Clarification (New and Existing Clarifiers)					
No.	**Item Description**	**Quantity**	**Units**	**Unit Cost ($)**	**Item Cost ($)**
1	Operator Labor	160	h	$50	$8,000
2	Electricity Usage	10000	kWh	$0.08	$843
Operations & Maintenance - Activated Sludge System					
No.	**Item Description**	**Quantity**	**Units**	**Unit Cost ($)**	**Item Cost ($)**
1	Operator Labor	958	h	$50	$47,900
2	Electricity Usage	220000	kWh	$0.08	$18,546
Operations & Maintenance - UV Disinfection					
No.	**Item Description**	**Quantity**	**Units**	**Unit Cost ($)**	**Item Cost ($)**
1	Operator Labor	300	h	$50	$15,000
2	Replacement Parts	1	LS	$1,000	$1,000
3	Electricity Usage	15000	kWh	$0.08	$1,265
Operations & Maintenance - Solids Management					
No.	**Item Description**	**Quantity**	**Units**	**Unit Cost ($)**	**Item Cost ($)**
1	Operator Labor	40	h	$50	$2,000
2	Electricity Usage	90000	kWh	$0.08	$7,587
3	Replacement Parts	1	LS	$200	$200
4	Solids Hauling	1	LS	$2,500	$2,500
Operations & Maintenance - Collection System					
No.	**Item Description**	**Quantity**	**Units**	**Unit Cost ($)**	**Item Cost ($)**
1	Operator Labor	100	h	$50	$5,000
2	Replacement Parts	1	LS	$1,000	$1,000
Operations & Maintenance - Administrative and Laboratory					
No.	**Item Description**	**Quantity**	**Units**	**Unit Cost ($)**	**Item Cost ($)**
1	Operator Labor	750	h	$50	$37,500
2	Misc. Materials and Services	1	LS	$25,000	$25,000
		Estimated Annual O&M (2023$)			**$182,237**

6.3.1.3 Debt Repayments

The City's sewer fund, as of June 30, 2023, has $481,238 of debt service. This is inclusive of two Business Oregon loans and one United States Department of Agriculture Rural Utilities Loan. The City generally budgets $60,000 annually for loan repayments.

6.3.1.4 Debt Service Reserve

The completion of the projects described in this section will require the City to identify potential funding sources. These sources will each have unique program requirements including the need to maintain a debt service reserve.

6.3.1.5 Short-Lived Asset Reserve

Items are identified as short-lived assets if their replacement is likely to occur within the 20-year planning period of the facility. As a result, their replacement should be planned for by making an annual deposit into an equipment replacement fund. For reference, Table 6-3 lists the items included in the City's short-lived asset inventory, their replacement timeframe, and estimated costs.

Table 6-3: Short Lived Asset Replacement Costs and Recommended Replacement Periods

Equipment	Replacement Period			Replacement Cost
	0-5 Years	*6-10 Years*	*11-15 Years*	
RAS/WAS Pumps		X		$10,000
Chemical Feed Pumps		X		$7,500
Pump Controls		X		$3,500
Headworks Thrust Bearings		X		$700
Headworks Auger Support Bearings		X		$3,000
Aeration Blowers		X		$45,000
Aeration Diffusers	X			$100
Mechanical Mixers			X	$75,000
UV Lamps	X			$400
UV Electrical Ballast		X		$1,000
Pressure Transducers		X		$750
SCADA Hardware		X		$11,000
Flow Meters			X	$13,000
Laboratory Equipment			X	$50,000
Office Computer and Misc. Equipment		X		$7,500

6.3.2 Financing Options

To implement all of the improvement projects included in the proposed CIP, the City will likely need to secure funding from external sources. Some grant funding may be available to the City, however, loans or the use of available cash reserves may be required for a significant portion of the cost. A description of funding sources available for the City is provided below, followed by an evaluation of a few funding scheme alternatives.

6.3.2.1 External Funding Resources

Some amount of outside funding assistance in the form of grants or low interest loans may be necessary to make the proposed improvement projects affordable for the City. The amount and types of outside funding will dictate the amount of local funding that the City must secure. In evaluating grant and local programs, the major objective is to select a program or combination of programs that is available and the most beneficial for the planning project.

It is recommended that the City schedule a “One-Stop” meeting as a first step after this plan’s approval to find the available alternatives for external funding. Potential funding programs that the City may be eligible for include Oregon’s Water/Wastewater Financing Program, the Clean Water State Revolving Fund, Oregon Department of Energy Small Scale Energy Loan Program, and the Special Public Works Fund. Information gained through the One-Stop meeting can then be used to select the funding sources that the City would then apply for.

6.3.2.2 Local Funding Resources

Several local funding sources are available to the City for sharing the cost of the planned wastewater system improvements. The amount and type of local funding obligations for infrastructure improvements will depend in part on the amount of grant funding anticipated and the requirements of potential loan funding. Local revenues sources for capital expenditures include various types of bonds, capital construction funds, system development charges (SDC), system user fees, and ad valorem taxes. Local revenue sources for operating costs include system user fees and ad valorem taxes.

Any potential sewer rate adjustment will depend on funding packages secured by the City. Interest rates, payback periods on loans, adjusted construction costs after pre-design phases, and many other variables could impact sewer rates. All of the projects included in the CIP, excluding I/I improvement projects, are partially SDC eligible as they provide for increased capacity for future development.

6.3.2.3 Funding Alternatives

To evaluate the impact of implementing the CIP on the City’s capital budget, debt service, and user rates, three funding approaches were evaluated. All dollars are in terms of 2023$ and do not account for inflation. The three alternative funding strategies evaluated were:

- Fully funded via loans at a nominal 20-year payback period and an interest rate of 3.5%;
- Mostly funded via loans at same terms, with approximately $2.3 million secured from grants or forgivable loan portions. Assumes that Phase 1, 2 and 4 projects would be fully loan funded, Phase 3 would be 25% loan funded and 75% funded via grants, and all I/I reduction would be funded internally through the City budget;
- Budget for capital improvements at approximately $90,000 annually for total of $2 million over the planning period, obtain approximately the same amount in loans, and obtain approximately $1.5 million in grant funds or forgivable loan portions. This assumes Phase 1 and all I/I reduction projects would be fully funded by the City’s budget, Phase 2 would be 100% grant funded, and Phases 3 and 4 would be 40% budget funded, 40% grant funded, and 20% loan funded.

A summary of debt service requirements, capital fund budget requirements, and the required grant/forgivable loan funds for each of these alternatives in presented in Table 6-4. The estimated impact on user rates for each alternative is also shown.

Table 6-4: Evaluation of Funding Strategy Alternatives

	Funding Strategy		
	Fully Loan Funded	Loan and Partially Grant Funded	Capital Investment, Loan, and Grant Funded
Total Debt Service:	$6,971,143	$3,815,987	$1,971,522
Budgeted Capital Funds:	$0	$301,600	$1,659,745
Grant Funds/Forgivable Loans:	$0	$2,717,850	$1,829,255
Total Cost (2023$)	$6,971,143	$6,835,437	$6,177,994

Sewer Rate Estimates				
Year	**Projected EDUs**	**Estimated Monthly Sewer Rates**		
2024	545	$75	$75	$69
2026	558	$83	$82	$77
2028	571	$83	$82	$76
2030	585	$113	$89	$83
2033	606	$111	$88	$82
2040	658	$112	$91	$82
2045	697	$109	$90	$81

As shown in the above table, it is possible for the City to pursue a funding strategy that combines budgeted capital improvement funds, loans, and grants to implement the proposed CIP while maintaining reasonable sewer rates to customers. It is recommended to obtain loans strategically throughout the planning period to keep the City's annual debt service under $100,000. A total grant income of at least $1 million over a 21-year period is a reasonable goal for the City. Any grant funds obtained in excess of this, or loans obtained at more competitive rates, would help the City keep service rates as low as possible. The following subsections describe some of the available programs that the City should consider pursuing to partially fund the proposed CIP.

6.3.2.3.1 Economic Development Administration Public Works Grant Program

The EDA Public Works Grant Program, administered by the U.S. Department of Commerce, is aimed at projects which directly create permanent jobs or remove impediments to job creation in the project area. Thus, to be eligible for this grant, a community must be able to demonstrate the potential to create jobs from the project. Potential job creation is assessed with a survey of businesses to demonstrate the prospective number of jobs that might be created if the proposed project is completed.

Projects must be located within an EDA designated Economic Development District. Priority is given to projects that improve opportunities for the establishment or expansion of industry and which create or retain both short-term and long-term private sector jobs. Communities that can demonstrate that the existing system is at capacity (i.e. moratorium on new connections) have a greater chance of being awarded this type of grant. EDA grants are usually in the range of 50 to 80 percent of the project cost. Therefore, some type of local funding also is required. Grants typically do not exceed one million dollars.

6.3.2.3.2 Water and Waste Disposal Loans and Grants

The Rural Utilities Service administers a water and wastewater loan and grant program designed to improve the quality of life and promote economic development in rural America. The Rural Utilities Service programs provide needed facilities to ensure health and safety and stimulate local economy by allowing access to new and advanced services and job opportunities. Program funds can be used for water, sewer, solid waste, and storm drainage projects. The most common uses are to restore deteriorating water supplies, or to improve, enlarge, or modify inadequate water or waste facilities.

Eligible applicants for Rural Utilities funds include public bodies and Indian Tribes. Non-profit corporations with significant ties to the local rural community may also be eligible. Funding is targeted to rural areas with populations of 10,000 or less. Applicants must be unable to obtain commercial financing at reasonable rates and terms or finance the project from existing resources.

The proposed project must serve a rural area not likely to decline in population below that for which the project is designed. The project should serve the present population and provide for foreseeable growth. Proposed projects should be necessary for orderly community development consistent with a comprehensive community or county development plan. Facilities must be modest in design, size, and cost. Water meters, a primary instrument for promoting conservation, are required by the agency. All water and wastewater systems must meet the standards set by the State Department of Environmental Quality.

The Rural Utilities staff review each project to determine need based on various priority points. Prioritization is necessary due to limited funding and to make sure the most deserving projects receive assistance. When possible, loan funds are combined with other federal and state financing to reduce the end cost to users of the system. Depending on median household income (MHI) and need, communities may qualify for grant funds of up to 75% of the eligible project costs. These grants can help reduce water and waste disposal rates to reasonable levels. Rural Utilities loans have a term of up to 40 years or for the useful life of the facility, whichever is less.

Grant fund eligibility is determined based on population, MHI, and user rates. Priority for grant funding is given to projects with populations of less than 5,500. Communities with low MHI may receive grant funding to reduce user costs to a reasonable level for rural residents. User rates are considered reasonable if they are less than or equal to existing prevailing rates in similar communities with similar systems. There are other restrictions and requirements associated with these loans and grants. If the City becomes eligible for grant assistance, the grant will apply only to eligible project costs. Additionally, grant funds are only available after the City has incurred long-term debt resulting in an annual debt service obligation equal to 0.5% of the MHI. In addition, an annual funding allocation limits the Rural Development funds. To receive a Rural Development loan, the City must secure bonding authority, usually in the form of general obligation bonds or revenue bonds.

6.3.2.3.3 Special Public Works Fund

The Special Public Works Fund program provides funding for the infrastructure that supports job creation in Oregon. Loans and grants are made to eligible public entities for the intent of studying, designing and building public infrastructure that leads to job creation or retention. The public entities or "municipalities" that are eligible to apply for Special Public Works Fund assistance include:

- Cities
- Counties
- Domestic water supply districts organized under ORS chapter 264
- Sanitary districts organized under ORS 450.005 to 450.245
- Sanitary authority, water authority or joint water and sanitary authority organized under
- ORS 450.600 to 450.989
- County service districts organized under ORS chapter 451
- Tribal Councils of Indian Tribes in Oregon
- Airport district organized under ORS Chapter 838
- A district as defined in ORS 198.010

To be eligible, the proposed project must be owned by a public entity that is an eligible applicant. The Special Public Works Fund is comprehensive in terms of the types of project costs that can be financed. As well as actual construction, eligible project costs can include costs incurred in conducting feasibility and other preliminary studies and for the design and construction engineering. The Fund is primarily a loan program. Grants can be awarded, up to the program limits, based on job creation or on a financial analysis of the applicant's capacity for carrying debt financing.

The total loan amount per project cannot exceed $10 million. The department can offer very attractive interest rates that typically reflect low market rates. In addition, the department absorbs the associated costs of debt issuance thereby saving applicants even more on the overall cost of borrowing. Loans are generally limited to the usable life of the contracted project, or 25 years from the year of project completion, whichever is less.

For infrastructure projects, grants are offered to projects creating or retaining jobs and are eligible for up to $5,000 per job created or retained. If a grant is offered it cannot exceed 85 percent of the project cost or $500,000, whichever is less. Additional grants may be awarded if there is a gap between the grant for jobs plus the loan and the total project costs.

6.3.2.3.4 Water/ Wastewater Financing Program

The Water/Wastewater Fund was created by the Oregon State Legislature in 1993. It was initially capitalized with lottery funds appropriated each biennium and with the sale of state revenue bonds since 1999. The purpose of the program is to provide financing for the design and construction of public infrastructure needed to ensure compliance with the Safe Drinking Water Act or the Clean Water Act.

The public entities that are eligible to apply for the program include: Cities, Counties, County Service districts (organized under ORS Chapter 451), Tribal Councils of Indian tribes, Ports, and Special Districts as defined in ORS 198.010.

Eligible activities include reasonable costs for construction improvement or expansion of drinking water, wastewater or storm water systems. Eligible projects include those related to drinking water source, treatment, storage and distribution; wastewater collection and capacity; stormwater system; purchase of rights-of-way and easements necessary for construction; and design and construction engineering. All projects must ensure that municipal water and wastewater systems comply with the Safe Drinking Water Act or the Clean Water Act.

To be eligible a system must have received, or is likely to soon receive, a Notice of Non-Compliance by the appropriate regulatory agency, associated with the Safe Drinking Water Act or the Clean Water Act. Projects also must meet other state or federal water quality statutes and standards.

The Fund provides both loans and grants, but it is primarily a loan program. The loan/grant amounts are determined by a financial analysis of the applicant's ability to afford a loan (debt capacity, repayment sources and other factors). The Water/Wastewater Financing Program's guidelines, project administration, loan terms and interest rates are similar to the Special Public Works Fund program. The maximum loan term is 25 years, or the useful life of the infrastructure financed, whichever is less. The maximum loan amount is $10,000,000 per project through a combination of direct and/or bond funded loans. Loans are generally repaid with utility revenues or voter approved bond issues. A limited tax obligation pledge may also be required. "Credit worthy" borrowers may be funded through sale of state revenue bonds.

Grant awards can be awarded up to a maximum of $750,000 depending on a financial review. An applicant is not eligible for grant funds if the annual median household income in the affected area is equal or greater than 100 percent of the state average median household income for the same year. Technical assistance funding for preliminary planning, engineering studies and economic investigations are available to municipalities with populations under 15,000 residents. Technical assistance projects must be done in preparation for an eligible construction project and can be awarded loans of up to $50,000 or grants of up to $20,000 per project.

6.3.2.3.5 Clean Water State Revolving Fund (CWSRF)

The Clean Water State Revolving Fund (CWSRF) Loan Program administered by DEQ provides low-cost loans for the planning, design and construction of a variety of projects that address water pollution. The loans through the CWSRF program are available to Oregon's public agencies, including cities, counties, sanitary districts, soil and water conservation districts, irrigation districts and various special districts.

There are several different types of loans available within the program. These include traditional planning, design and construction loans. Each of these loan types has different financial terms and is intended to provide communities with choices when financing water quality improvements. Interest rates are based on the nation's bond buyer's index and fluctuate quarterly. The interest rates of various loans are substantially discounted from the bond rate. For example, with a quarterly bond rate of 5.0%, the CWSRF interest rates (depending on the type of loan) would range from 0.97% to 3.88%. Loan payback periods vary, ranging from 5 to 30 years. Loans do include an annual loan fee of 0.5% of the outstanding balance. Planning loans are exempt from this fee. Eligible projects include:

- Wastewater system plans and studies
- Secondary or advanced wastewater treatment facilities
- Irrigation improvements
- Infiltration and inflow correction
- Major sewer replacement and rehabilitation
- Qualified storm water control
- Onsite wastewater system repairs
- Matching funds for some U.S. Department of Agriculture conservation programs

- Estuary management efforts
- Various nonpoint source projects (stream restorations, animal waste management, conservation easements)
- Qualified brownfields projects

All eligible proposed projects are ranked based upon their application information and entered on the program's Project Priority List. Points are assigned based on specific ranking criteria. Newly ranked projects are integrated into the priority list on a regular basis. The Project Priority List is incorporated within DEQ's annual Intended Use Plan which indicates the proposed use of the funds each year. Projects are funded based on the availability of loan monies. If monies are insufficient to fund all the approved projects, funds are distributed to as many projects as possible based on the Project Priority List. Each time new monies become available, those monies are allocated to as many unfunded or partially funded projects as possible.