NEWPORT WASHINGTON 2023 WASTEWATER TREATMENT FACILITY PLAN

December 2023

Prepared by

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NEWPORT WASHINGTON
2023 WASTEWATER TREATMENT FACILITY PLAN

December 2023

Certification

This 2023 Wastewater Treatment Plant Facility Plan for the City of Newport has been prepared under the direction of the following Registered Professional Engineers in compliance with the Washington Department of Ecology Requirements for Engineering Reports, WAC 173-240-060

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ES-1 Purpose & Background

The City has experienced operational issues in recent years at the wastewater treatment plant (WWTP) and recognized a lack of reliability due to no redundancy in the major treatment processes. Additionally, two major treatment processes are off-line due the inability to repair or not being compliant with current standards.

The City commissioned this Facility Plan to evaluate the wastewater treatment facility and evaluate its ability to serve over the next 20 years. The nature of these goals, in addition to requirements in WAC 173-240-0600, requires that an engineering report be prepared and approved by the Washington State Department of Ecology (WSDOE). The City authorized J-U-B ENGINEERS, Inc. to undertake the WWTP Facilities Plan Update in 2019-2022.

The City of Newport (WWTP) provides biological treatment for incoming domestic and commercial waste. The WWTP is located north east of the City of Newport near the bank of the Pend Oreille River. The WWTP operates under NPDES Waste Discharge Permit No. WA-002232-2. This facility treats wastewater for the City of Newport and also from Oldtown, Idaho through the West Bonner Sewer District #1.

ES-2 Flows and Loads

Wastewater from the City of Newport Washington and Oldtown Idaho is treated by the City of Newport's WWTP. Influent flow data from January 2016 through December 2020 were analyzed. Flow and load data are summarized in Table ES-1.

Table ES-1: Flow and Load Summary by Year

Item	2016	2017	2018	2019	2020	Probable Existing 2022
Annual Average Day Flow (mgd)	0.22	0.22	0.21	0.20	0.19	0.20
Peak Day Flow (mgd)	0.35	0.54	0.36	0.33	0.36	0.40
Peak Hour Flow (mgd)	0.67	0.66	0.63	0.60	0.57	0.60
BOD Annual Ave. Day Load (ppd)	369	350	413	368	398	380
BOD Maximum Month Load (ppd)	493	571	522	616	498	557
BOD Peak Day Load (ppd)	628	579	631	675	522	600
TSS Annual Ave. Day Load (ppd)	327	310	242	335	410	372
TSS Maximum Month Load (ppd)	551	542	473	819	867	843
TSS Peak Day Load (ppd)	615	682	613	1305	944	1125

The existing wastewater treatment plant's Design Criteria are listed in Table ES-2 as well as the future projected flow and loading.

Table ES-2: Design Criteria in 2010 NPDES Permit & Projected

Parameter	Design	2041
Monthly Average Dry Weather	0.50 mgd	0.35 mgd
Maximum Month Flow	1.00 mgd	0.47 mgd
BOD_5 Influent Loading, Max Month	1330 lb/day	1,020 lb/day
TSS Influent Loading, Max Month	1,500 lb/day	1,423 lb/day

ES-3 Discharge Standards

The City discharges treated effluent into the Pend Oreille River under the National Pollution Discharge Elimination Permit WA-002232-2 which expired in 2010 and has been administratively extended. The effluent discharge limitations are shown in Table ES-3.

Table ES-3: Discharge Limits in 2010 NPDES Permit

Parameter	Average Monthly	Average Weekly
Biochemical Oxygen Demand		
Concentration	30 mg/l	45 mg/l
Load	125 pounds/day	188 pounds/day
Percent Removal	Greater than 85%	Greater than 85%
Total Suspended Solids		
Concentration	30 mg/l	45 mg/l
Load	125 pounds/day	188 pounds/day
Percent Removal	Greater than 85%	Greater than 85%
Flow	0.5 mgd	
Fecal Coliform Bacteria	100 cfu/100ml	200 cfu/100ml
Total Residual Chlorine	0.5 mg/l	0.75 mg/l
pH	6.0 < pH <9.0	

The permit requires the City to plan future facilities to maintain adequate capacity when any of the parameters listed in the above table reaches 85% of the design criteria for three consecutive months or are projected to reach design capacity within 5 years.

ES-4 Wastewater Treatment Plant

Newport's WWTP is an oxidation ditch type wastewater treatment plant that provides preliminary, primary and secondary treatment, and effluent disinfection. Treated effluent is discharged to the Pend Oreille River. Settled primary solids and biological solids generated within the treatment plant are stabilized via anaerobic and aerobic digestion, respectively. Stabilized solids are dewatered and hauled off-site for beneficial use via land application by third parties.

Biochemical Oxygen Demand (BOD), Total Suspended Solids (TSS), Chlorine Residual and Fecal Coliform are the primary effluent parameters used to determine overall plant performance. BOD and TSS have an effluent discharge limit of 30 mg/l. Between 2016 and 2020, BOD was only exceeded once and TSS was only exceeded 3 times. Fecal Coliform limits were violated eight times. A review of 2021 and 2022 effluent data found that 25 additional violations occurred. The violations are summarized as follows:

- 1 pH
- 3 Chlorine Residual
- 1 TSS, and
- 16 Fecal.

Performance has been good with most exceedances related to operational maintenance activities that required the function of some treatment component to be less than ideal.

Major unit processes at the WWTP need repairs; a brief summary follows:

- The primary clarifier is out of service and cannot be brought online without replacing all the mechanical and electrical components.
- The anaerobic digester is out of service and cannot be brought online without removing accumulated debris and replacing all the mechanical and electrical components.
- The oxidation ditch biological treatment tank has a cracked and damaged effluent weir box and cannot be readily fixed because there is not a redundant treatment unit.
- The aerobic digester building is badly rusted and therefore has questionable structural integrity.

ES-5 Alternatives Evaluated and Selected Improvements

The City evaluated four alternatives to continue providing wastewater treatment services.

1. Alternative A - Do nothing

   Summary
   City staff would continue to maintain facilities that are currently in operation by repairing and replacing parts and equipment as needed without an increase in plant reliability or performance. The plant has sufficient capacity to serve the projected population in the planning period; however, the existing plant lacks redundancy, and staff are unable to rehabilitate or maintain dilapidated parts and equipment due to lack of redundancy. The existing plant also has a number of safety concerns to staff operating it.

   Evaluation
   This alternative was not recommended due to the ever-increasing risk of permit violations as the plant service population grew and existing equipment continued to age.

2. Alternative B – Repair and Upgrade Existing Facilities:

   Summary
   This alternative would make operational upgrades to unit processes in the facility, including headworks improvements, lab/office renovations, improvements to the existing oxidation ditch, clarifier upgrades, pumphouse upgrades, and make repairs and upgrades to the aerobic digester. These repairs and upgrades would improve reliability, worker

safety, and address operational concerns, but would not provide adequate redundancy to all systems. The estimated cost for Alternative B is $30.4 M (2023 dollars).

Evaluation

Was a viable alternative in terms of treatment capacity with ability to meet water quality requirements. This alternative was the least cost alternative and was selected as the preferred alternative. This alternative has the potential for phased implementation based on the ability of the users to bear the costs of improvements.

3. Alternative C - Restore Primary Processes and Implement Repairs and Upgrades

Summary

Alternative C would repair the primary clarifier, anaerobic digester and aerobic digester, thereby recovering expected capacity and performance. This alternative would also make operational upgrades to other unit processes in the facility, including headworks improvements, lab/office renovations, belt filter press upgrades, improvements to the existing oxidation ditch, clarifier upgrades, pumphouse upgrades, replace chlorine disinfection with UV disinfection, and make repairs and upgrades to the aerobic digester. However, repairing the WWTP's components will not provide adequate redundancy to all systems, which maintains the increasing risk of failure. The single primary clarifier will not have ideal performance by the end of the planning period due to the shallow depth and being slightly overloaded; however, the extra solids flowing to the secondary system can easily be managed by the oxidation ditch and addition of a second primary clarify is not necessary during the planning period in this alternative. The estimated cost for Alternative C is $38.5 M (2023 dollars).

Evaluation

This was not considered a viable alternative because even with repairing and updating the current facilities, the improvements would not increase overall reliability due to a lack of redundancy in the primary and secondary treatment systems and the inability to take any unit process offline for heavy maintenance and repair.

4. Alternative D – Fully Convert to Secondary Treatment, and Implement Extensive Repairs and Upgrades

Summary

Alternative D would construct a second oxidation ditch and a third clarifier and remove the primary treatment system from service. The two oxidation ditches and three clarifiers would provide adequate redundancy allowing for unit processes to be taken offline for maintenance and future upgrades. This alternative would also make operational upgrades to other unit processes in the facility, including headworks improvements, lab/office renovations, belt filter press upgrades, improvements to the existing oxidation ditch, clarifier upgrades, pumphouse upgrades, replace chlorine disinfection with UV disinfection, and make repairs and upgrades to the aerobic digester. The estimated cost for Alternative D is $45.5 M (2023 dollars).

Evaluation

Was a viable alternative, addressing the deficiencies of the existing facility in a comprehensive manner. However, the cost was considered prohibitively expensive and it was not selected as the preferred alternative.

5. Alternative E - MBR Package Treatment Plant

Summary

Alternative E would provide a membrane bioreactor package treatment plant to fully treat the wastewater. This option entirely replaces the existing WWTP at a new location on nearby City property. A new influent lift station would intercept the existing flow and pump into a new building where the MBR treatment components would be housed. The existing WWTP components could be mothballed or repaired according to the needs of the City. The estimated cost for Alternative E is $37.0 M (2023 dollars).

Evaluation

Alternative E uses membrane filtration technology which is the tool used by most agencies to achieve very low effluent concentrations and is considered the best available technology. This technology, however, is not currently required to meet regulatory discharge limits and it is not considered likely that more stringent discharge limits will be implemented in the planning period. Alternative E was considered in relation to

Alternative D because it included advanced automation functions, a more secure barrier against discharges of improperly treated waste, the ability to provide water quality suitable for irrigation reuse with enhanced disinfection, and the ease and relative low cost of future expansion. Like Alternative E, this was not the least cost alternative and was not selected for implementation.

6. Alternative F – Land Application

<u>Summary</u>
Should compliance with the Pend Oreille River TMDL imposed heat load limit require effluent cooling, the cost to mechanically cool effluent could be more than the cost to land apply the effluent. About 80 acres and 4.5 million gallons of storage would be needed to dispose of effluent during the summer when heat loads could limit discharge. About 230 acres and 90 million gallons of storage would be needed to dispose of the effluent year-round.

<u>Evaluation</u>
At this time, the cost to implement land application is prohibitive when continued river discharge is likely; therefore, the land application alternative was eliminated from further consideration.

7. Alternative G – Gravity-Settling Package Treatment Plant

<u>Summary</u>
Alternative G would replace the current treatment processes (oxidation, ditch, clarifiers and pump houses) with a gravity-settling package treatment plant, with the objective of avoiding the costs of upgrading or expanding the existing processes eliminated by the package treatment plant. The headworks, belt filter press and chlorine contact chamber would be retained and the package treatment plant would be incorporated in the hydraulic flow path downstream of the headworks and upstream of the solids handling facilities. The estimated cost for Alternative G is $33.1M (2023 dollars).

<u>Evaluation</u>
This alternative does not provide any advantages over the existing processes and incorporation of a gravity-settling package treatment plant would require significant improvements and associated costs to be able to

configure the package plant within the site. This alternative is not the least cost alternative and was not selected for implementation.

ES-6 Capital Improvement Plan

It is estimated that the selected Alternative B could be implemented in the next 5-years as funding is secured:

- 2023 to 2025: Secure funding for permitting, environmental review, and design engineering (up to $2.3 M in 2023 dollars).
- 2025 to 2028: Secure funding for construction (up to $28.1 in 2023 dollars)

This selected Alternative B has the flexibility for phased implementation, which would allow for improvements to be prioritized by objectives and completed in separate phases based on funding availability and an evaluation of rate impacts and the ability of the wastewater customers to bear the costs of the improvements. The proposed phasing plan, developed through discussions with Newport administrative and WWTP staff, is as follows:

Preliminary Design Phase , 2024

This preliminary design phase has the following objectives:

1. Define the specifics of the improvements to be implemented,
2. Evaluate potential cost savings,
3. Confirm the phasing of improvements,
4. Refine the costs at an appropriate level to make funding requests.

Phase 1 Improvements , Design 2024/2025, Construct 2026

This phase will include the following elements:

1. Complete oxidation ditch upgrades,
2. Construct new Secondary Clarifier #3,
3. Complete Pumphouse #2 Upgrades,
4. Initiate purchasing for backup generator,

Phase 1 focuses on water quality compliance with the facility's discharge permit. It addresses the top priorities for improved redundancy and effectiveness in the treatment process. It is this phase that assures the treatment facility has the capacity to address the growth that may occur in the 20-year planning period. Subsequent phases address the maintenance issues typical of a treatment facility as it ages. Phase 1 also initiates the ability to provide power to the entire plant in the event of a power utility failure, which is absolutely essential to reliable treatment during emergency events requiring an alternate power source. The procurement of the backup generator occurs in this phase, while the final installation occurs in Phase 2.

Phase 2 Improvements, Design 2025/2026, Construct 2027/2028
This phase will include the following elements:

1. Overall site: backup generator/combine power sources, water line/hydrant
2. Headworks improvements,
3. Clarifier #1 and #2 mechanical equipment upgrades,
4. Pumphouse #1 upgrades,

Phase 2 finalizes the installation of emergency backup power, ensuring that the facility will operate if utility power is interrupted. Phase 2 also prioritizes maintenance issues that assure ongoing operational functionality and worker safety.

Phase 3 Improvements, Design 2027/2028, Construct 2029
This phase will include the following elements:

1. Overall site: Vactor truck purchase, yard valve replacement, SCADA system implementation
2. Aerobic digester and building improvements,
3. Belt filter press upgrades,
4. New shop/office/lab building

Phase 3 provides for maintenance upgrades of the existing facility and provides operational monitoring and control features that assure a rapid response by operations staff. It also addresses issues critical for worker safety and welfare and provides a facility for protection and maintenance of the vehicles and mobile equipment essential to facility operations.

CHAPTER 1 – PURPOSE & BACKGROUND

1.1 Purpose

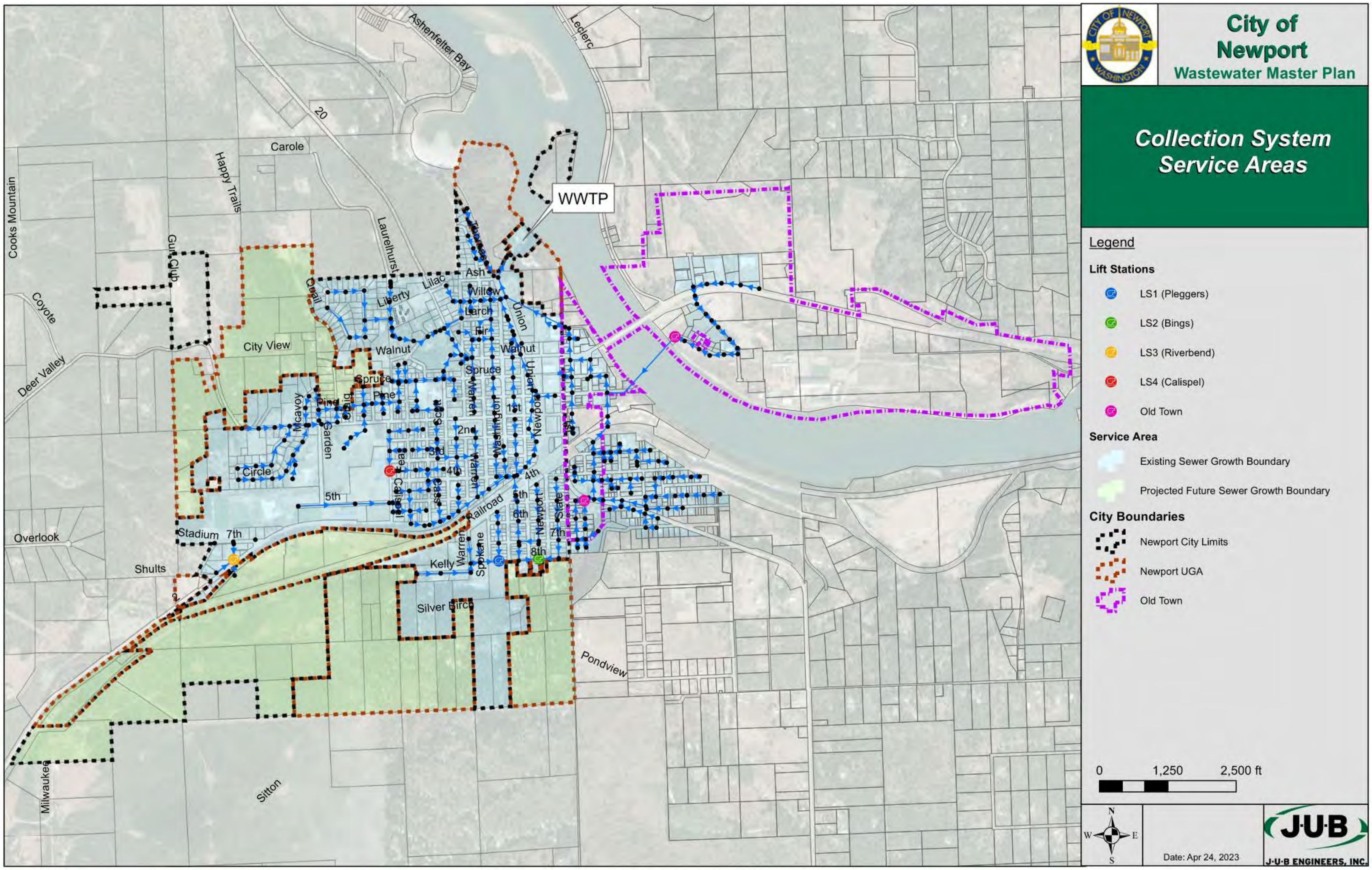
The City has experienced operational issues in recent years at the wastewater treatment plant (WWTP) and recognized a lack of reliability due to no redundancy in the major treatment processes. Additionally, two major treatment processes are offline due the inability to repair or not being compliant with current standards.

The City commissioned this Facility Plan to evaluate the wastewater treatment facility and evaluate its ability to serve over the next 20 years. The nature of these goals, in addition to requirements in WAC 173-240-060, requires that an engineering report be prepared and approved by the Washington State Department of Ecology (WSDOE). The City authorized J-U-B ENGINEERS, Inc. to complete the WWTP Facilities Plan Update in 2019-2023.

1.2 Background and Facility History

The City of Newport Wastewater Treatment Plant provides biological treatment for incoming domestic and commercial waste. There are no significant industries discharging wastewater that needs pre-treatment or require special handling. The WWTP is located in the northeastern corner of the City of Newport near the bank of the Pend Oreille River. The WWTP operates under NPDES Waste Discharge Permit No. WA-002232-2. This facility treats wastewater for the City of Newport and also from Oldtown Idaho through the West Bonner Water and Sewer District #1. The location of the WWTP is shown in Figure 1-1.

Figure 1-1: WWTP Location and Service Area



The facility currently includes the following major unit processes, discussed in Chapter 5:

- Influent Parshall Flume
- Receiving Station, (City no longer accepts septage due to operational concerns)
- Headworks with Bar Rack, Fine Screens, Grit Chamber
- Primary Clarifier
- Anerobic Digester
- Oxidation Ditch
- Secondary Clarifier(s)
- Aerobic Digester
- Belt Filter Press
- Chlorine Contact Basin
- Outfall

An aerial of the facility identifying the location of the processes listed above, as well as other critical components is outlined in Figure 5-1 of Chapter 5. The WWTP site consists of approximately 6 acres, adjacent to the Pend Oreille River. Access to the WWTP site is via a road aligned with Union Avenue north of Walnut Street.

Prior to 1950, wastewater was conveyed to a community septic tank and discharged to the Pend Oreille River. In 1950, unit process improvements were constructed to provide primary treatment that included a headworks facility (grit removal and comminutor), primary clarification (solids settling), disinfection and settled solids management by anaerobic digestion and drying (sand drying beds). The primary clarifier and anaerobic digester are the only unit process still on-call to provide service; however, both are non-functional due to breakdowns.

In 1972, an activated sludge aeration lagoon and secondary clarifier were constructed downstream of the primary clarifier to provide secondary treatment with activated sludge capture and return. Wasted solids were thickened on the sludge drying beds which were expanded as part of the 1972 upgrade. A new chlorine contact tank was also constructed in 1972 near the river outfall as well as upgrades to the outfall which extended the outfall about 60 feet and added a 4-port diffuser on the end. All the facilities constructed in 1972 are still in service except for the sand drying beds; however, the steel structure covering the aeration lagoon is badly corroded.

In 1984, an activated sludge oxidation ditch was constructed as well as another secondary clarifier. The old aeration lagoon was repurposed to serve as an aerobic digester to stabilize waste activated sludge from the oxidation ditch secondary treatment system. The facilities constructed in 1984 are still in service.

In 2003, the old headworks facility was removed and replaced with modern grit removal and screening facilities. A belt filter press dewatering unit process was constructed to thicken and dewater waste activated sludge replacing the sludge drying beds which were removed from service and demolished. The dewatered biosolids are loaded into trucks and hauled off-site to BarrTech for composting and subsequent sale for beneficial use as a soil amendment. The facilities constructed in 2003 are still in service.

1.3 Study Scope

Because of identified operational issues, concerns about process capacities, potential permit changes, and need to formerly prepare the City's Wastewater Facility Plan, the City of Newport authorizes J-U-B ENGINEERS, Inc. to complete this Wastewater Facility Plan with WSDOE requirements. J-U-B ENGINERS, Inc. is also concurrently working on preparing the City of Newport General Sewer Plan. The reader is referred to Chapter 1 and 2 of the General Sewer Plan for all population, demographics, land use, and planning information that was used in this analysis. Likewise, the General Sewer Plan refers the reader to this Facility Plan document for all information relating to the WWTP. These two planning documents are being submitted simultaneously to WSDOE for review and approval.

In order to qualify the identified WWTP improvements for WSDOE funding, the State Environmental Review Process (SERP) will be followed. The environmental review documents have been prepared in conjunction with the study.

This Wastewater Treatment Plant Facility Plan document contains the chapters and appendices listed below in Table 1-1.

Table 1-1: Wastewater Facility Plan Document Outline

Section	Title	Contents
Chapter		
1	Purpose and Background	Purpose, Scope, Organization, Planning Information
2	Existing Environment	Summary of Environmental Issues and SERP Process
3	Flows and Load Analysis	Summary of Existing and Projected Flows and Loads to WWTP
4	Discharge Standards	Summary of Current and Future Permit Requirements
5	Wastewater Treatment Plant	Evaluation of Existing Facilities at Existing and Future Conditions
6	WWTP Improvements Alternatives	Discussion of Various Alternatives for liquid stream process and other facility improvements
7	Biosolids Management	Discussion of Biosolids Management
8	Alternative Comparison, Selection and Capital Improvement Plan	Summary Evaluation of Alternatives, Cost, Prioritization and Improvement Plan
		-

Section	Title	Contents
Appendix		
A	Checklist for Facility Plan Contents	Documenting compliance with WDOE and WAC 173-240-060 requirements
B	FEMA Flood Insurance Rate Map	Floodplain documentation for SERP Cross-Cutter requirements
C	National Wetlands Inventory Map	Wetlands documentation for SERP Cross Cutter requirements
D	Farmland Classification Exhibit	Farmland documentation for SERP Cross Cutter requirements
E	NPDES Permit	The City's most recent discharge permit for the WWTP
F	WWTP Improvements Alternatives Preliminary Cost Estimates	Cost estimates for alternatives for improvements to the WWTP
G	WWTP Improvements Alternatives Exhibits	Exhibits for improvement alternatives for the WWTP
H	MBR Plant Sample Calculations and Life Cycle Costs	Design calculations from a MBR plant manufacturer, with equipment lift cycle costs
I	Gravity-Settling Package Treatment Plant Design Criteria Calculations	Design calculations from a gravity-settling package treatment plant manufacturer
J	Rate Analysis Sample Worksheets	Sample Rate Scenarios based on the preferred alternative
K	SEPA Checklist	SEPA Checklist for this facility plan

1.4 Compliance with Washington Department of Ecology and WAC Facility Plan Requirements

This document was prepared in accordance with Washington Department of Ecology and Washington Administrative Code (WAC) 173-240-060 requirements. Appendix A includes a summary of required facility plan contents from Orange Book Table G1-1 and the required contents for an engineering report listed in WAC 173-240-060 and further explained in Orange Book Table G1-2. The location of the required information is listed in these tables as an aid to the reader.

1.5 Acknowledgements

Many people were extremely helpful in providing documentation, information, and input throughout the course of this project. We wish to thank the City of Newport Mayor and City Council. We wish to especially thank the City of Newport's staff who contributed to this report: Abby Gribi, Dave North, Josh Howard, and Bryce Seaney were instrumental in collecting data, presenting improvement ideas, evaluating alternatives, expressing system concerns, and giving timely, pointed feedback. Their assistance was appreciated. The authorized owner's representative of the treatment plant is:

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CHAPTER 2 – EXISTING ENVIRONMENT

2.1 Public Health

For information regarding public health issues within the service area, the reader is directed to Chapter 2 of the 2023 General Sewer Plan. This includes information regarding planning area, land use, water systems, unsewered areas, onsite sewer systems, and service area policies.

In summary:

- The service area for the WWTP includes the City of Newport, WA and Oldtown, ID.
- Land uses for all areas within the UGA boundary have been established by the City Planning Department.
- The City of Newport owns and operates a water system that provides potable water service to all areas within the UGA boundary. There is not a separate irrigation system.
- The General Sewer Plan identifies proposed extensions of sewer interceptors to serve all areas within the UGA that are currently unsewered.
- No onsite sewer systems are used within the City's boundary.

2.2 Physical Characteristics

2.2.1 Topography

The topography of the service area in Newport is between elevations 2110 and 2180 feet above sea level, with slopes ranging between 1% and 6%. Newport sits in a depression, with hills and forested areas located to the north, west, and south. The City of Newport is located on the west banks of the Pend Oreille River. The east boundary of Newport is shared with the west boundary of Oldtown, Idaho.

The WWTP is located on the west banks of the Pend Oreille River. The site is stepped into the hillside, where structures and features are generally level and surrounding topography is on a 3:1 slope.

2.2.2 Climate

Newport, Washington is located in a semi-arid climate within the rain shadow of the Cascade Mountain Range. The mean annual temperature is 44.8 °F. Average daily temperature is 66.0 °F in July and 25.0 °F in January. Average annual precipitation 24.7 inches and average annual snowfall 54 inches. Most precipitation occurs from November through May.

2.2.3 Geology

The City of Newport is set on gravelly silt loam and silt loam, with the Pend Oreille River on the east boundary of city limits. Some well bore holes have reported encountering granite at 100- or 150-feet depths below ground surface in some areas, with a majority of bore holes encountering topsoil, sand, gravel, and silt closer to the surface.

Depth to groundwater depends on the location in Newport and the proximity to the Pend Oreille river, but is generally encountered 50 feet or greater below ground surface.

2.2.4 Soils

The City of Newport is set on gravelly silt loam and silt loam, with the Pend Oreille River on the east boundary of city limits.

2.3 Study Boundary

The WWTP is located on the west banks of the Pend Oreille River. The WWTP currently discharges approximately 0.25 million gallons per day (mgd) of treated wastewater effluent into the Pend Oreille River. The WWTP operates under NPDES Waste Discharge Permit No. WA-002232-2, as discussed in Chapter 4. The WWTP is situated on property that is owned by the City of Newport. All planned improvements for the WWTP will occur within the property or on nearby property owned by the City of Newport. All environmental analysis herein are limited to the area of potential effects (APE) or study boundary area shown in Figure 2-1.

Figure 2-1: WWTP Study Boundary

2.4 Sensitive Areas

2.4.1 Flood Plains

To assess the flooding potential associated with the WWTP Facility, the most recent Federal Emergency Management Agency (FEMA) flood insurance rate map was obtained and reviewed (see Appendix B). The WWTP area is designated as Zone X floodplain, areas determined to be outside the 500-year floodplain. Proposed improvements being considered are in Zone X.

2.4.2 Shorelines

The Pend Oreille River is located adjacent to the northeast boundary of the WWTP. The Pend Oreille River and the WWTP share approximately 465 linear feet of frontage along the property line.

2.4.3 Wetlands

The National Wetlands Inventory (NWI) map identifies no wetland area within the WWTP boundary (see Appendix C). No evidence of wetland features (i.e. hydric soils, hydrophytic vegetation or wetland hydrology) exist within this area.

2.4.4 Prime or Unique Farmland

The Farmland Classification (FC) map identifies no areas rates as “farmland of statewide importance” (see Appendix D).

2.4.5 Archaeological and Historical Sites

Within the City of Newport, there are approximately 23 historical properties, with 3 on the national historical register. The properties listed include Newport City Hall, Pend Oreille County Courthouse, Roxy Theater, Hope Congregational Church, and other buildings such as depots, firehouses, homes, and barns.

The nearest Tribal lands are northwest of Newport, near Cusick, Washington (Kalispel Tribe). Newport falls within the Kalispel Tribe area of interest.

2.4.6 Wild and Scenic Rivers Act

The Pend Oreille River is not listed on the National Wild and Scenic Rivers System.

2.5 Species and Habitats

As alternatives for improvements to the WWTP are considered, the potential impacts to endangered species or essential fish habitats will be evaluated. No negative impact is anticipated.

2.6 SERP

In order to qualify the identified WWTP improvements for WSDOE funding, the State Environmental Review Process (SERP) will be followed. This effort is limited to the APE as depicted in Figure 2-1. This will include preparation of a SEPA Checklist, public notice, SERP Checklist, Federal Cross-Cutter Checklist, SERP Cover Sheet, Biological Assessment, and Cultural Resources Survey.

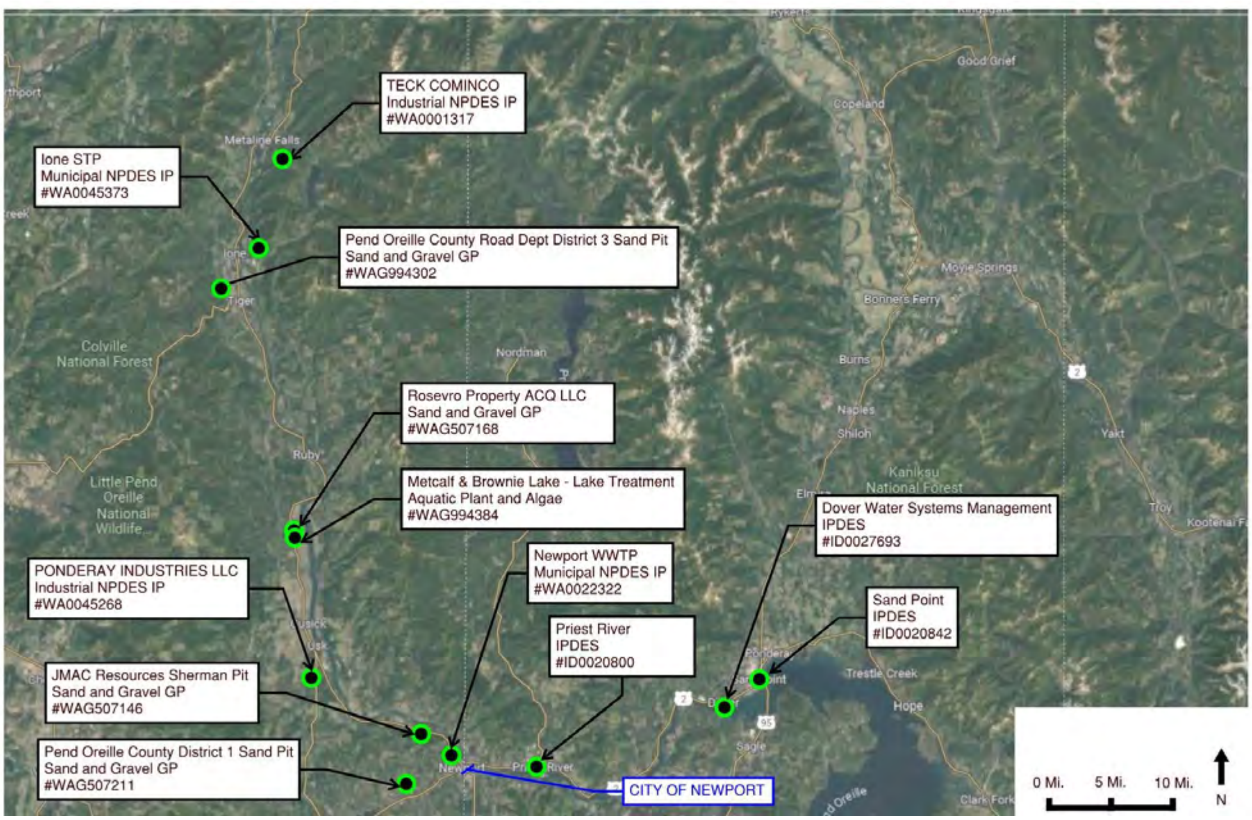
2.7 Estuaries

No salt water or estuaries could be affected by the City's WWTP discharge.

2.8 Adjacent Dischargers

The Idaho Department of Environmental Quality Issued Permits and Water Quality Certifications data base and the Washington Department of Ecology Water Quality Permitting and Reporting Information System (PARIS) were searched for upstream and downstream discharges to the Pend Oreille River. The results are illustrated in Figure 2-2.

Figure 2-2: Permitted Discharges in Vicinity of Newport



CHAPTER 3 – FLOW AND LOAD ANALYSIS

3.1 Introduction

Wastewater from the City of Newport Washington and Oldtown Idaho is treated by the City of Newport's WWTP. Flow from Oldtown enters Newport's gravity wastewater collection system at a single point. The combined flow is measured by a 9-inch Parshall flume flow meter located in the headworks building. The wastewater currently consists primarily of residential discharges with a few other types of connections. Daily flow volumes are logged for record keeping purposes. Influent wastewater is periodically sampled and analyzed for constituents of concern. Influent samples are collected just upstream of the bar rack via a temperature-controlled auto sampler. Historic wastewater parameters were used to estimate the current flow and strength of wastewater entering the WWTP for treatment. Historic parameters were projected to the end of the planning period to estimate future performance needs at the WWTP.

Data from November 2015 through December 2020 were used for this study. Definitions and descriptions of the averaging periods used in this analysis are as follows:

- Average Day: The average annual flow rate observed at the facility in a given year. (e.g., total flow for a year divided by 365 days). The average rate is used to estimate annual average pumping and chemical costs, solids production, and organic loading rates.
- Maximum Month: The largest 30-day moving average observed during the analysis period. This condition is typically used to design unit processes for permit compliance.
- Peak Day: The largest flow or load observed in any one day during the analysis period. The peak day condition is used to size processes for peak events occurring over a 24-hour period.
- Peak Hour: The largest flow or load condition expected to occur during any 60-minute averaging interval throughout the planning period. The peak hour conditions are used to size processes for peak events (e.g. pump stations, hydraulics, oxygen demand).
- Peaking Factors: Ratios of maximum events to average events (e.g., a maximum month peaking factor is obtained by dividing the maximum month value for a selected parameter by a baseline value, typically the average day value).
- Population Equivalent (PE): The amount of flow and load on a per capita basis. The PE is typically calculated by dividing total flow or load by the service population. The PE is used to project future flows and loads in conjunction with population projections.

3.2 Existing Flows and Loads

3.2.1 Flows

Influent flow data from January 2016 through December 2020 were reviewed to determine recent trends. Influent flow data are shown in Figure 3-1. The data set from January 2016 through December 2020 were analyzed to determine the conditions discussed in the previous section, which are summarized in Table 3-1. Of note is the decline in gallons per capita per day due to water conservation efforts mainly due to more efficient household fixtures.

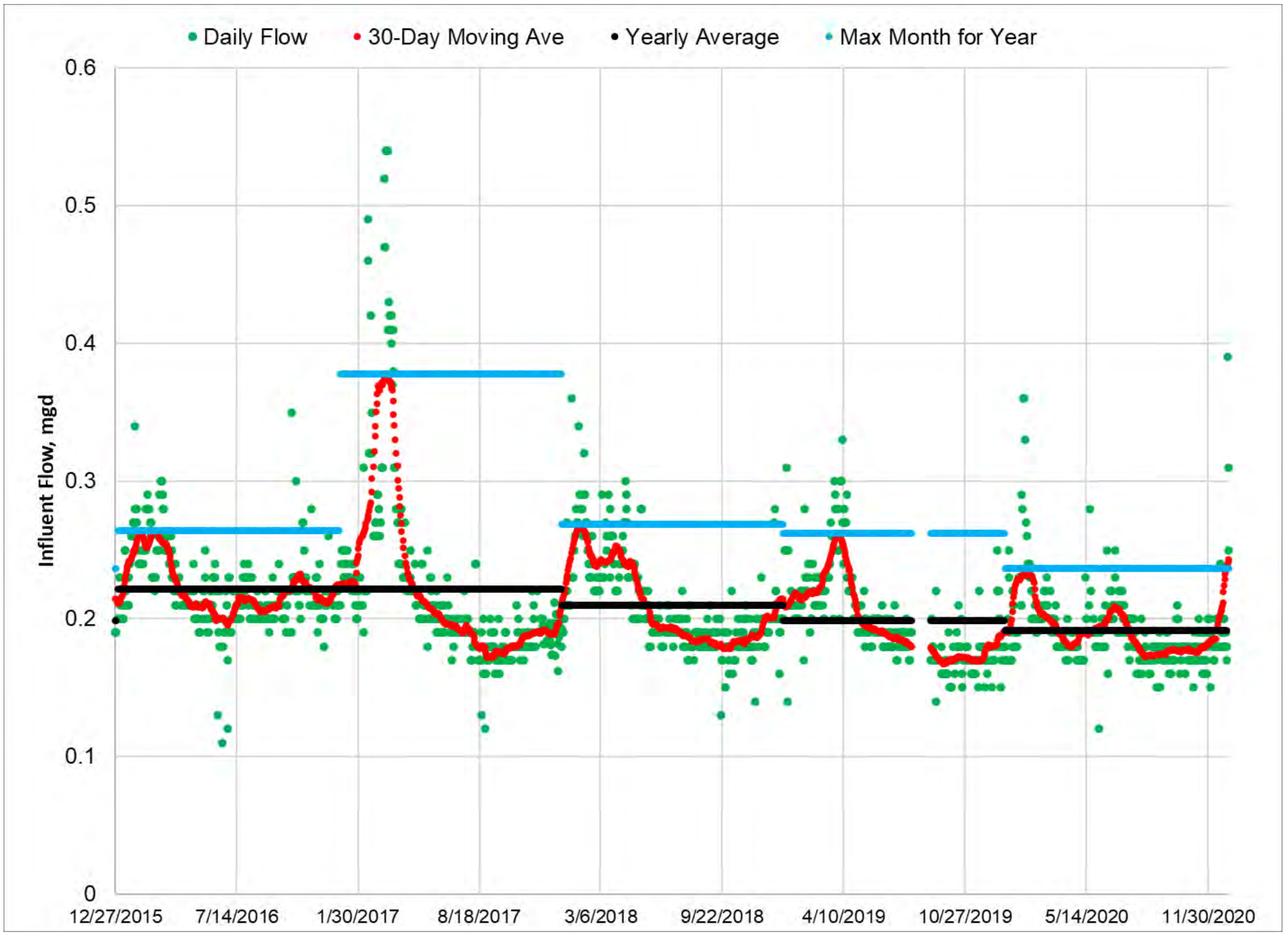
Table 3-1: Flow Summary by Year

Item	2016	2017	2018	2019	2020	Probable Existing 2022[b]
Annual Average Day Flow (mgd)	0.22	0.22	0.21	0.20	0.19	0.20
Population Equivalent (gpcd) [a]	95	93	88	82	78	82
Maximum Month Flow (mgd)	0.26	0.38	0.27	0.26	0.24	0.26
Peaking Factor	1.19	1.71	1.28	1.32	1.24	1.32
Peak Day Flow (mgd)	0.35	0.54	0.36	0.33	0.36	0.40
Peaking Factor	1.58	2.44	1.71	1.66	1.88	2
Peak Hour Flow (mgd)	0.67	0.66	0.63	0.60	0.57	0.60
Peaking Factor	3	3	3	3	3	3

(a) *Based on an estimated population, Newport+Oldtown, See General Sewer Plan*

(b) *Largest of last two years. Peak day and peak hour flows were rounded up to nearest 1/10th.*

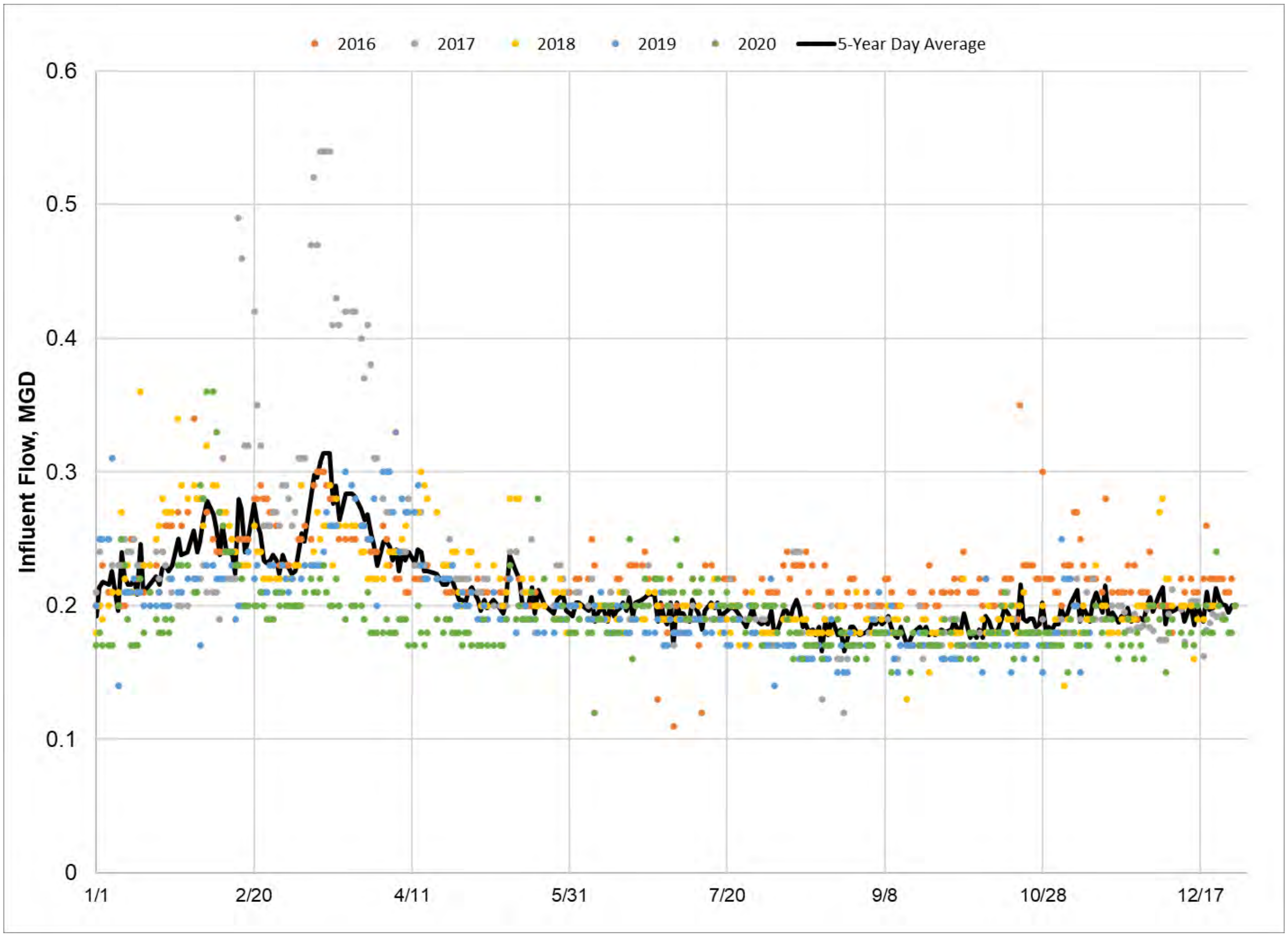
Figure 3-1: Influent Flow (2015-2020)



The average daily flow varies seasonally as shown in Figure 3-2. Flows are generally higher during January, February and March and lowest during July, August and September (average = 0.24 and 0.19 MGD respectively). The average daily flow during the wet season is 28% greater than the dry season. The wet season flow amounts to a ~22 gallon per person per day increase for a total wastewater contribution of 99 gpcd during the wet season compared to a 77 gpcd contribution during the dry season. Peak day flows from direct inflow contributions result in a per capita flow of 221 gpcd. While inflow and infiltration extraneous flow contributions are significant, the flows are not considered excessive per EPA guidance[1]. However, the City recognizes that extraneous flow due to inflow and infiltration has negative impacts on the WWTP and is actively investigating sources of inflow and infiltration to reduce extraneous flows. Reducing extraneous flow to the WWTP will make capacity available for users and reduce operational difficulties.

[1] EPA Infiltration/Inflow Analysis and Project Certification, US EPA, May 1985, Ecology Publication No. 97-03

Figure 3-2: Influent Flow Seasonal Variation



3.2.2 Biochemical Oxygen Demand

The average day, maximum month, and peak day BOD loading for 2016 through 2020 are summarized in Table 3-2.

Table 3-2: BOD Summary by Year

Item	2016	2017	2018	2019	2020	Estimated Existing Value 2022
Annual Average Day Load (ppd)	369	350	413	368	398	383 [a]
Annual Ave. Day Concentration (mg/L)	198	185	239	229	256	242 [a]
Population Equivalent (ppd/person) [b]	0.15	0.14	0.17	0.15	0.16	0.15 [b]
Maximum Month Load (ppd)	493	571	522	616	498	557 [a]
Peaking Factor	1.33	1.63	1.26	1.67	1.25	1.45 [c]
Peak Day Load (ppd)	628	579	631	675	522	600 [a]
Peaking Factor	1.70	1.66	1.53	1.83	1.31	1.56 [c]

(a) *2019 & 2020 average*

(b) *Based on an estimated population, Newport+Oldtown, See General Sewer Plan*

(c) *The peaking factor is calculated as the selected maximum divided by the annual average day load*

A probable current average value of 380 ppd for BOD loading for the City was selected based on the 2019 and 2020 average, which equates to 0.15 pounds per capita per day (ppcd) using a sewered population of 2,539. This is within the typical range of 0.11 to 0.26 ppcd expected for residential loading (Metcalf and Eddy).

3.2.3 Total Suspended Solids

The average day, maximum month, and peak day TSS loading for 2016 through 2020 are summarized in Table 3-3.

Table 3-3: TSS Summary by Year

Item	2016	2017	2018	2019	2020	Estimated Existing Value 2022
Annual Average Day Load (ppd)	327	310	242	335	410	372 (d)
Annual Ave. Day Concentration (mg/L)	174	172	141	206	261	233 (d)
Population Equivalent (ppd/person) (e)	0.13	0.13	0.10	0.13	0.16	0.15 (d)
Maximum Month Load (ppd)	551	542	473	819	867	843 (d)
Peaking Factor	1.68	1.74	1.96	2.45	2.12	2.27 (f)
Peak Day Load (ppd)	615	682	613	1305	944	1125 (d)
Peaking Factor (e)	1.88	2.20	2.54	3.90	2.31	3.10 (f)

(d) *2019 & 2020 average*

(e) *Based on an estimated population, Newport+Oldtown, See General Sewer Plan*

(f) *The peaking factor is calculated as the selected maximum divided by the annual average day load*

A probable current average value of 372 ppd for TSS loading for the City was selected based on 2019 and 2020 data, which equates to 0.15 pounds per capita per day (ppcd) using a sewered population of 2,539. This is within the typical range of 0.11 to 0.26 ppcd expected for residential loading (Metcalf and Eddy).

3.2.4 Total Kjeldahl Nitrogen

Total Kjeldahl Nitrogen (TKN) data are used to estimate oxygen demand required for nitrification which is the biological conversion of ammonia-nitrogen to nitrate-nitrogen if the WWTP needed to nitrify. Historic TKN data are not available so a typical TKN concentration of 40 mg/l was used to estimate existing nitrogen loading (typical range of 20 to 85 with 40 mg/l being the medium expected for domestic wastewater, Metcalf and Eddy). The estimated existing average day and maximum month TKN loading are summarized in Table 3-4. At this time, the WWTP does not need to nitrify the effluent.

Table 3-4: TKN Summary by Year

Item	Probable Existing 2022
Annual Average Day Load (ppd)	67
Annual Average Day Concentration (mg/L)	40
Population Equivalent (ppd/person) [a]	0.026
Maximum Month Load (ppd)	133
Peaking Factor	2
Peak Day Load (ppd)	200
Peaking Factor	3

[a] *Based on an estimated population, Newport+Oldtown, See General Sewer Plan*

3.2.5 Summary of Current Flows and Loads

The existing flow and load data presented above are summarized in Table 3-5.

Table 3-5: Existing Flows and Loads Summary

Item		Value
Flow (mgd)	Average Day	0.20
	Maximum Month	0.26
	Peaking Factor	*1.32*
	Peak Day	0.40
	Peaking Factor	*2*
BOD (ppd)	Average Day	383
	Maximum Month	557
	Peaking Factor	*1.45*
	Peak Day	600
	Peaking Factor	*1.56*
TSS (ppd)	Average Day	372
	Maximum Month	843
	Peaking Factor	*2.27*
	Peak Day	1125
	Peaking Factor	*3.10*
TKN (ppd)	Average Day	67
	Maximum Month	133
	Peaking Factor	*2*
	Peak Day	200
	Peaking Factor	*3*

3.3 Projected Flows and Loads for Year 2041

As discussed in the General Sewer Plan, the service area population is estimated to grow at 2.5%. Future flows are also projected to grow at 2.5%. Future loads are based on the projected average day flow and the estimated per-person loading discussed in previous sections. Peak flows and load are estimated using observed historical peaking presented above. The corresponding projected flows and loads for 2041 are summarized in Table 3-6 and shown graphically in Figure 3-3.

Table 3-6: Projected Flows and Loads Summary (2041)

Item		Probable Existing Value 2022	Estimated Future Value 2041
Flow (mgd)	Average Day	0.20	0.35
	Population Equivalent	82	82
	Maximum Month	0.26	0.47
	Peaking Factor	1.32	1.32
	Peak Day	0.40	0.71
	Peaking Factor	2	2
	Peak Hour	0.60	1.06
	Peaking Factor	3	3
BOD (ppd)	Average Day	383	647
	Maximum Month	557	940
	Peaking Factor	*1.45*	*1.45*
	Peak Day	600	1011
	Peaking Factor	*1.56*	*1.56*
TSS (ppd)	Average Day	372	628
	Maximum Month	843	1423
	Peaking Factor	*2.27*	*2.27*
	Peak Day	1125	1949
	Peaking Factor	*3.10*	*3.10*
TKN (ppd)	Average Day	67	112
	Maximum Month	133	224
	Peaking Factor	*2*	*2*
	Peak Day	200	336
	Peaking Factor	*3*	*3*

Figure 3-3: Projected Flows

3.4 Infiltration and Inflow

Infiltration is the term for groundwater that enters the system through faulty joints, leaking manholes and cracks in the collection system. Infiltration flow is observed at the WWTP as wet weather conditions increase groundwater, which flows into the collection system through faulty joints and cracks. Groundwater could come from seasonal weather conditions, excessive irrigation or contributions of infiltrated flows from drywells and stream management features. Storm associated infiltration increases during a storm event then slowly subsides as groundwater retreats over a period of several hours or days depending on soil conditions. Irrigation associated infiltration usually increases with the irrigation season and remains until the end of the season when groundwater subsides.

Inflow is the term for extraneous flow that enters the system during a storm event, usually through a direct connection such as a manhole lid and other miscellaneous direct connections *i.e.,* roof drains, foundation drains and storm drainage structures.

During a storm event, inflow suddenly enters the collection system as rainfall is collected from the illicit connection drainage area and conveyed into the wastewater main line. Rainfall associated inflow rapidly decreases when precipitation ends. Snow melt associated inflow is related to snow accumulation and temperature and could take several days to run its course.

Infiltration and inflow (I/I) affect the sewer system by increasing the volume of flow that must be collected and treated at the WWTP. This results in reduction of efficiencies in biological processes and increases the cost of unit processes that are sized based on detention time or surface overflow rates. Therefore, it is desirable to minimize I/I.

Newport's service area experiences a relatively minor amount of infiltration throughout the year. Infiltration is considered non-excessive if the average daily flow rate (with infiltration) is less than 120 gallons per capita per day (gpcd) during a dry period when there is seasonally high groundwater and no rainfall. During the wet season the per capita contribution is estimated to be 95 gpd which is less than 120 gpcd; therefore, infiltration is not considered to be excessive per EPA standards.

The determination of whether inflow is non-excessive is made using the highest daily flow recorded during a storm event and the estimated population at the time. If the total daily flow during the high rainfall day is less than 275 gpcd (with inflow), the EPA considers the system to have non-excessive inflow. During the last five years the peak per capita contribution was 213 gallons per day which is less than 275 gpcd; therefore, the inflow component of the City's sewer flow is not considered "excessive" per EPA standards.

However, as stated above, the City recognizes that extraneous flow negatively impacts the treatment system and consumes capacity and is actively looking for sources and diverting extraneous flow from the collection system. If the City can remove a substantial volume of extraneous flow prior to needed treatment plant upgrades, peak flow management facilities may not be required (*e.g.* flow equalization storage).

CHAPTER 4 – DISCHARGE STANDARDS

The City discharges treated effluent into the Pend Oreille River under the National Pollution Discharge Elimination Permit WA-002232-2 at River Mile 87.7 (48°11’ 25.66” N : 117°02’ 29.25” W). The River’s 7Q10 flow is 4653.36 million gallons per day, as stated in the discharge permit fact sheet and TMDL. Also stated in the fact sheet are the following ambient river data:

Parameter	Value used
Temperature (highest annual 1-DADMax)	23.6 °C
Temperature (highest annual 7-DADMax)	22.9 °C
Flow (7Q10)	4653 MGD
pH (Maximum / Minimum)	8.7 / 7.7 s.u.
Dissolved Oxygen	10.7 mg/L
Total Ammonia-N	0.05 mg/L
Fecal Coliform 1/100 mL dry weather	(21/100 mL storm related)
Turbidity	7.1 NTU
Hardness	84 mg/L as CaCO3
Alkalinity or Salinity	87.6 mg/L as CaCO3

Due to the high volume of available dilution water in the river, Ecology has concluded that toxic effects cause by unidentified pollutants in the effluent are unlikely and whole effluent toxicity testing (WET) is not required.

The EPA maintains a list of 129 priority pollutants that could be toxic, could bioaccumulate or could be persistent in the environment. Priority pollutants have historically been associated with industrial or manufacturing operations; however, some are coming from consumer goods which could have a future impact to the City. At this time, the City does not have to sample effluent for priority pollutants.

Discharge standards are discussed below.

4.1 Federal Water Quality Standards

The principal authority for the water pollution control programs is the Clean Water Act (33 U.S.C. 1251 et seq.). The aim of the act is to "restore and maintain the chemical, physical, and biological integrity of the nation’s waters." This act set forth the following national goals:

- Eliminate the discharge of pollutants into navigable waters by 1985.
- Set interim goals of water quality which will protect fish and wildlife and will provide for recreation by July 1, 1983.

- Prohibit the discharge of pollutants in quantities that might adversely affect the environment.
- Construct publicly owned waste treatment facilities with federal financial assistance.
- Establish waste treatment management plans within each state.
- Establish the technology necessary to eliminate the discharge of pollutants.
- Develop and implement programs for the control of non-point sources of pollution to enable the goals of the act to be met.

These goals were to be achieved by a legislative program which includes permits under the National Pollutant Discharge Elimination System (NPDES). Key provisions of the act include the development of such permit systems and effluent standards as well as state and local responsibilities.

The Clean Water Act emphasizes that state governments are to use the minimum federal standards, guidelines, and goals, and establish individual pollution control programs and enforcement procedures. When the state has completed its programs for waste treatment management, its implementation plans for preserving or restoring water quality, and the Environmental Protection Agency (EPA) has approved those programs, the state assumes enforcement responsibilities. The Washington State Department of Ecology (WSDOE) has been delegated with these responsibilities by EPA.

4.2 Washington State Surface Water Quality Standards

The State of Washington' s surface water quality standards are given in the Washington Administrative Code (WAC) Chapter 173-201A, the Water Quality Standards for Surface Waters of the State of Washington, and WAC Chapter 173-204, Sediment Management Standards.

WAC 173-201A strives to establish surface water quality criteria which are consistent with public health and public enjoyment, and the propagation and protection of fish, shellfish, and wildlife, pursuant to the provisions of chapter 90.48 of the Revised Code of Washington (RCW). The surface water quality standards establish specific water quality criteria based on the Aquatic Life and Recreational Use designations. Use designations for the Pend Oreille River (from Idaho to the Canadian Border) are defined in WAC 173-201A Table 602 as follows:

- Aquatic Life Uses: Spawning/rearing
- Recreational Uses: Primary Contact
- Water Supply Uses: Domestic Water, Industrial Water, Agricultural Water, and Stock Water
- Miscellaneous Uses: Wildlife Habitat, Harvesting, Commerce/Navigation, Boating, and Aesthetics

In accordance with the direction of EPA, WSDOE has pursued compliance with surface quality standards based on a watershed management approach. The emphasis of watershed management is to monitor, analyze, and protect water quality on a geographic basis. The watershed management strategy was implemented as a means to:

- Identify and address high priority water quality issues.
- Tie NPDES permit conditions more closely to localized water quality conditions.
- Improve coordination among state, tribal and local environmental programs.
- Target activities to attain state water quality standards.

In 1970, under WAC 173-500-040 and the Water Resources Act of 1971 (RCW 90.54), WSDOE partitioned the state into 62 (WRIAs). These WRIAs are the administrative underpinning of WSDOE’s business activities and provide the framework for the watershed approach which is embodied in the Section 303(d) process. The Pend Oreille River at Newport is located within WRIA 62 (Pend Oreille).

4.2.1 303(d) List

The Federal Clean Water Act (Section 303(d)) and federal regulation 40 CFR Part 130.7 require states to develop a 303(d) list. The primary purpose of the 303(d) listing is to describe the health of rivers, coastal waters, estuaries and lakes. In Washington, WSDOE submits this listing of “troubled waters" to EPA for approval and uses it to monitor water quality trends and establish priorities for protection.

Water bodies must meet two criteria to be placed on the 303(d) list:

- Current water quality does not meet the state water quality requirements.
- Technology-based controls are not sufficient to achieve water quality requirements.

Monitoring data to determine which water bodies should be identified on the 303(d) list are gathered from several sources, including WSDOE's own monitoring, and project-specific monitoring conducted by resource agencies, tribes, and other sources. Monitoring information submitted to the WSDOE is evaluated to ensure that the data was collected and analyzed using quality assurance/quality control methods and that data was tested by a state accredited laboratory.

The Pend Oreille River near Newport is on the State’s 303(d) list as temperature and pH impaired. Upon listing as temperature impaired, the Washington State Department of Ecology (Ecology) initiated a total maximum daily load (TMDL) study which was complete in 2011 and approved by the EPA in 2019. The TMDL study determines the extent of the water quality problem(s) and the underlying causes, and then specifies a limit on the amount of pollutants to improve water quality and return the surface water to criteria, thereby achieving its beneficial uses.

4.2.2 Temperature TMDL

For the River near Newport, the TMDL concluded that, the Pend Oreille River near Newport complies because Albeni Falls Dam discharges deeper, cooler water from the lake to the Pend Oreille River which buffers sources of river warming so that river temperatures are cooler now than before the dam was built. Overall, maximum temperatures observed at most of the upper river reaches are cooler now than what occurred naturally. Because Newport's effluent is highly diluted upon discharging into the River, there is not a reasonable potential for Newport's discharge to cause a temperature water quality exceedance. However, a wasteload allocation of 47,600,000 kcal/d was assigned to Newport to protect against future temperature increases. The wasteload allocation was calculated based on the permitted capacity of 0.5 MGD and an effluent temperature of 22.9 °C. The heat wasteload allocation limit is not expected to require any facility improvements over the next 20 years; however, effluent temperature should be closely monitored to track heat discharged to determine when mitigation may be required.

TMDL listings for temperature for the Pend Oreille river are show in Figure 4-1 and Figure 4-2.

Figure 4-1: Category 4A Listing for Pend Oreille River (Source: WSDOE)

Listing ID: 48352

Main Listing Information

Listing ID: 48352
Waterbody Name: PEND OREILLE RIVER
Medium: Water
Parameter: Temperature
WQI Project: Pend Oreille River Temperature TMDL
Designated Use: Aquatic Life - Salmonid Spawning, Rearing, and Migration

Draft Category: 4A

View Category History

Assessment Unit

Assessment Unit ID: 17010216000238_001_001
Size: 4.487 Kilometers
Associated Components(s): Reach: 17010216000238 0% - 100%, Type: Rivers/Streams
County: Pend Oreille
WRIA: 62 - Pend Oreille

Basis Table

No Basis Table data.

Basis Statement

HISTORICAL INFORMATION

Location ID: 1020 -- In 2004, between 6/26/2004 and 10/19/2004, the 1-day maximum values exceeded the criterion for this waterbody (20°C) on 55 of 116 days (47%); The maximum exceedance during this period was 23.9°C on 8/13/2004;

Remarks

Assessment Cycle 2018 - A historic Category 4A determination was carried forward from a previous assessment or administrative decision. See remarks for more information.

Impairment addressed by Pend Oreille River Temperature TMDL, EPA approved 12/31/2020.

Special Condition: Temperature shall not exceed a 1-day maximum (1-DMax) of 20.0 deg C due to human activities.

Data Sources

Study Id	Location Id	Source Database
PPIC0006	1020	EIM

Map Link

Map Link

Figure 4-2: Category 5 listing for Pend Oreille River (Source: WSDOE)

Listing ID: 48352

Main Listing Information

Listing ID: 48352
Waterbody Name: PEND OREILLE RIVER
Medium: Water
Parameter: Temperature
WQI Project: None
Designated Use: None

Current Category: 5

View Category History

Assessment Unit

Assessment Unit ID: 17010216000238 **County:** Pend Oreille
WRIA: 62 - Pend Oreille

Basis Statement

Location ID: 1020 -- In 2004, between 6/26/2004 and 10/19/2004, the 1-day maximum values exceeded the criterion for this waterbody (20°C) on 55 of 116 days (47%); The maximum exceedance during this period was 23.9°C on 8/13/2004 ;

Remarks

WQ Standards for this reach of the Pend Oreille state that temperature shall not exceed a 1-DMax of 20.0°C due to human activities. -B. Conklin

Data Sources

Study Id	Location Id
PPIC0006	1020

Map Link

Map Link

4.2.3 Future TMDLs

The WSDOE Surface Water Quality Standards website includes Current Rule Activities with the recent update on 'Human Health Criteria and Implementation Tools Rulemaking.' This rulemaking is focusing on water quality standards for toxics. This Rule could substantially reduce allowable concentrations of toxins in the effluent primarily due to an increase in fish consumption rates. Most toxins accumulate in the fatty portions of edible fish. For example, the current Washington WQS for Polychlorinated Biphenyls (PCBs) is 7 pg/l. This concentration is too low to be quantified. Some WWTPs try to achieve an effluent limit less than 170 pg/l which is difficult to consistently meet at a conventional wastewater treatment plant with advanced secondary treatment. There is not currently any data on Newport's concentrations, but typical influent concentrations range from 1,000 – 10,000 pg/l.

Ecology's current PCB position is that small communities will not have to test for priority pollutants unless an industry that may discharge toxics locates in Newport (or Oldtown). Understanding the far-reaching impacts such an industry may have on the cost of wastewater treatment should be fully understood before allowing an industry to locate within Newport's service area.

If new rules require extensive treatment to remove toxics, it could trigger advanced oxidation processes after secondary treatment with filtration – which could double or triple the cost of treatment.

4.3 Existing Discharge Standards

The City of Newport's WWTP operates under NPDES Waste Discharge Permit No. WA-002232-2, included in Appendix E. The permit was effective on May 1, 2010 and expired on April 30, 2013. The 2010 permit has been administratively extended. The permit was issued for only 3 years rather than the typical 5 years in case the TMDL (which was being planned in 2010) recommended additional effluent limits. The City has applied for renewal of the permit and has been following the terms and conditions of the existing permit in the interim. The wastewater treatment plant's Design Criteria from the 2010 NPDES Permit are listed in Table 4-1. The effluent discharge limitations are shown in Table 4-2, which are typical standards for Technology-Based Effluent Limits achieved at Newport via secondary treatment facilities.

Table 4-1: Design Criteria in 2010 NPDES Permit

Parameter	
Monthly Maximum Flow	1.00 mgd
Average Daily Flow	0.50 mgd
BOD_5 Influent Loading	1330 lb/day
TSS Influent Loading	920 lb/day

Table 4-2: Discharge Limits in 2010 NPDES Permit

Parameter	Average Monthly	Average Weekly
Biochemical Oxygen Demand [A]		
Concentration	30 mg/l	45 mg/l
Load	125 pounds/day	188 pounds/day
Percent Removal	Greater than 85%	Greater than 85%
Total Suspended Solids [A]		
Concentration	30 mg/l	45 mg/l
Load	125 pounds/day	188 pounds/day
Percent Removal	Greater than 85%	Greater than 85%
Flow	0.50 mgd	
Fecal Coliform Bacteria [B]	100 cfu/100ml	200 cfu/100ml
Total Residual Chlorine [A]	0.5 mg/l	0.75 mg/l
pH [B]	6.0 < pH <9.0	

A Technology Based Limit

B Water Quality Based Limit

The 7Q10 flow of the receiving Pend Oreille River is 4,353.36 MGD (7,200 cfs).

The permit requires the City to plan future facilities to maintain adequate capacity when any of the parameters listed in the above table reaches 85% of the design criteria for three consecutive months or are projected to reach design capacity within 5 years.

The permit requires the wastewater treatment plant to be operated by a Group II operator.

4.4 Historical Performance

Effluent samples are collected just downstream of the chlorine contact basin via a temperature-controlled auto sampler. Effluent sample data were reviewed for permit compliance. In general, performance at the WWTP has been very good.

Biochemical Oxygen Demand (BOD), Total Suspended Solids (TSS), Chlorine Residual and Fecal Coliform are the primary effluent parameters used to determine overall plant performance. BOD and TSS have an effluent discharge limit of 30 mg/l. Between 2016 and 2020[2], BOD was only exceeded once and TSS was only exceeded 3 times. Fecal Coliform limits were violated eight times. A review of 2021 and 2022 effluent data found that 25 additional violations occurred. The violations are summarized as follows:

- 1 pH
- 3 Chlorine Residual
- 1 TSS, and
- 16 Fecal Coliform.

Performance has been good with most exceedances related to operational maintenance activities that required the function of some treatment component to be less than ideal.

The plant operators believe the Fecal Coliform violations were mainly due to the inability to flow pace the chlorine dose. Recently, City staff installed an effluent weir monitor on the effluent V-notch weir allowing the flow to be measured. The flow data were integrated into the chlorine dose control equipment which allowed flow paced chlorine dosing. Operators will evaluate performance to determine if additional facilities are needed (de-chlorination).

The average effluent BOD and TSS concentrations were 10.4 and 9.3 mg/l, respectively, demonstrating excellent performance.

The violations are not associated with high flows (all flows ~0.2 MGD) and continue to stem from difficulty flow pacing chlorine (freezing conditions), operating when one clarifier is out of service and splitting flow evenly to the two secondary clarifiers. The operators continue to optimize the functionality of the equipment available.

4.5 Mixing Zone

Newport discharges into the Pend Oreille River wherein the effluent is mixed into the bulk river flow. The river volume where mixing occurs is the mixing zone. Within mixing zones, pollutant concentrations may exceed water quality standards if beneficial uses are maintained. Water quality standards include both aquatic life-based criteria and human health-based criteria.

There are two aquatic life criteria concentrations for toxic constituents, chronic and acute. Acute mixing zones are small to minimize contact time with high concentrations of toxic constituents that may be instantly lethal. Chronic mixing zones are larger to

[2] This project started in the Spring of 2021, the prior 5 years of data were analyzed.

allow a bit more contact time with lower concentrations of toxic constituent that may be lethal with time. Newport's chronic mixing cannot exceed more than 300 feet plus the depth of water above the outlet ports downstream, or more than 100 feet upstream, cannot be wider than 25% of the river's width nor take up more that 25% of the rivers flow. The acute mixing zone cannot exceed more than 10% of the chronic mixing zone.

Carcinogenic and non-carcinogenic pollutants are evaluated for human health-based water quality criteria.

During the last permitting cycle, using the facility design flow of 0.5 MGD, Ecology estimated the available mixing zone dilution factors to be:

- Acute Aquatic Life Criteria 117.3
- Chronic Aquatic Life Criteria 2327.7
- Human Health Criteria - Carcinogen 3258.3
- Human Health Criteria - Non-carcinogen 11634.4

Ecology uses dilution factors with the water quality criteria to calculate reasonable potentials to exceed criteria at permit limits and establish lower limits should a reasonable potential to exceed criteria exist.

Ecology concluded that the discharge does not have a reasonable potential to cause the loss of sensitive or important habitat, substantially interfere with existing or characteristics uses, result in damage to the ecosystem, or adversely affect public health if the permit limits are met.

The actual mixing was not modeled nor measured in the field since the potential to exceed criteria (in the maximum sized mixing zone allowed, see above) is very low. Future permits may require the mixing zone to be modeled or measured.

4.6 Expected Future Discharge Standards

Based upon inquires made to the WSDOE Staff, work may begin on the discharge permit renewal process in 2023. Discharge limits in the new permit are expected to remain largely unchanged; however, pending rule making may require monitoring of additional contaminants of concern; specifically, polychlorinated biphenols (PCBs) may need to be monitored as PCBs are an emerging pollutant that may need regulated in Newport's discharge. The current policy is that small community dischargers will not have to monitor for toxics unless there is an industry in town that could discharge toxics.

Should toxic testing be required, and results find toxics reductions are necessary, the City should review existing dischargers to find potential source control opportunities with an eye towards reducing the compliance effort to meet potential discharge limits for toxics. While the permit conditions that will result from current rule-making efforts are far from clear, the evidence points toward more stringent standards. Involvement with the rule-making is critical to provide as much compliance flexibility as possible plus reasonable compliance schedules for any required upgrades. Additionally, as stated above, toxics impacts by industries wishing to locate in Newport's service area may have on wastewater facilities should be well understood before allowing industries to locate in the area.

As discussed in Chapter 3, the flow that will be generated upon complete buildout of the existing Urban Growth Area (UGA) boundary as well as the proposed UGA boundary expansion has been calculated. Based on this expanded service, average day flows are projected to be 0.35 mgd. Applying a maximum month peaking factor of 1.32 to this value yields a projected maximum month flow of 0.47 mgd. In lieu of the flow calculated based upon OFM population projections for the 20-year study period, this UGA buildout flow should be considered for permitting.

The predicted flows at end of the planning period are less than the flows allowed in the current permit. The future discharge is not expected to have a measurable change in the concentration of constituents of concern in the Pend Oreille River near the outfall.

CHAPTER 5 – WASTEWATER TREATMENT PLANT

5.1 General

When Newport's wastewater treatment plant (WWTP) is fully operational, the combined facilities function as an oxidation ditch type wastewater treatment plant that provides pretreatment, primary and secondary treatment, and effluent disinfection. Treated effluent is discharged to the Pend Oreille River. Settled primary solids and biological solids generated within the treatment plant are stabilized via anaerobic and aerobic digestion, respectively. Stabilized solids are dewatered and hauled off-site for beneficial use via land application by third parties. Screened material and grit removed in the pretreatment process are hauled to a landfill for disposal. Automated portions of the facilities can be controlled and monitored in a control building. Limited constituent analysis can be performed by City staff in an onsite Laboratory located in the control building.

An aerial view of the WWTP with the major components identified is shown in Figure 5-1. A process flow schematic is shown in Figure 5-2. A Hydraulic Grade Line (HGL) schematic is included in Figure 5-3.

5.2 Historical Performance

Biochemical Oxygen Demand (BOD) and Total Suspended Solids (TSS) are the two primary effluent parameters used to determine overall plant performance. From Section 4.3 above, both parameters have an effluent discharge limit of 30 mg/l. Effluent flow data from November 2015 through December 2020 were reviewed to determine recent trends in BOD and TSS discharge concentrations. Over the 5-year period, BOD was only exceeded once and TSS was only exceeded 3 times. Performance has been good with most exceedances related to operational maintenance activities that required the function of some treatment component to be compromised.

An evaluation of each unit process is presented in the following sections of the report.

Figure 5-1: WWTP Major Component Aerial View

Figure 5-2: WWTP Flow Schematic (From Record Drawings)

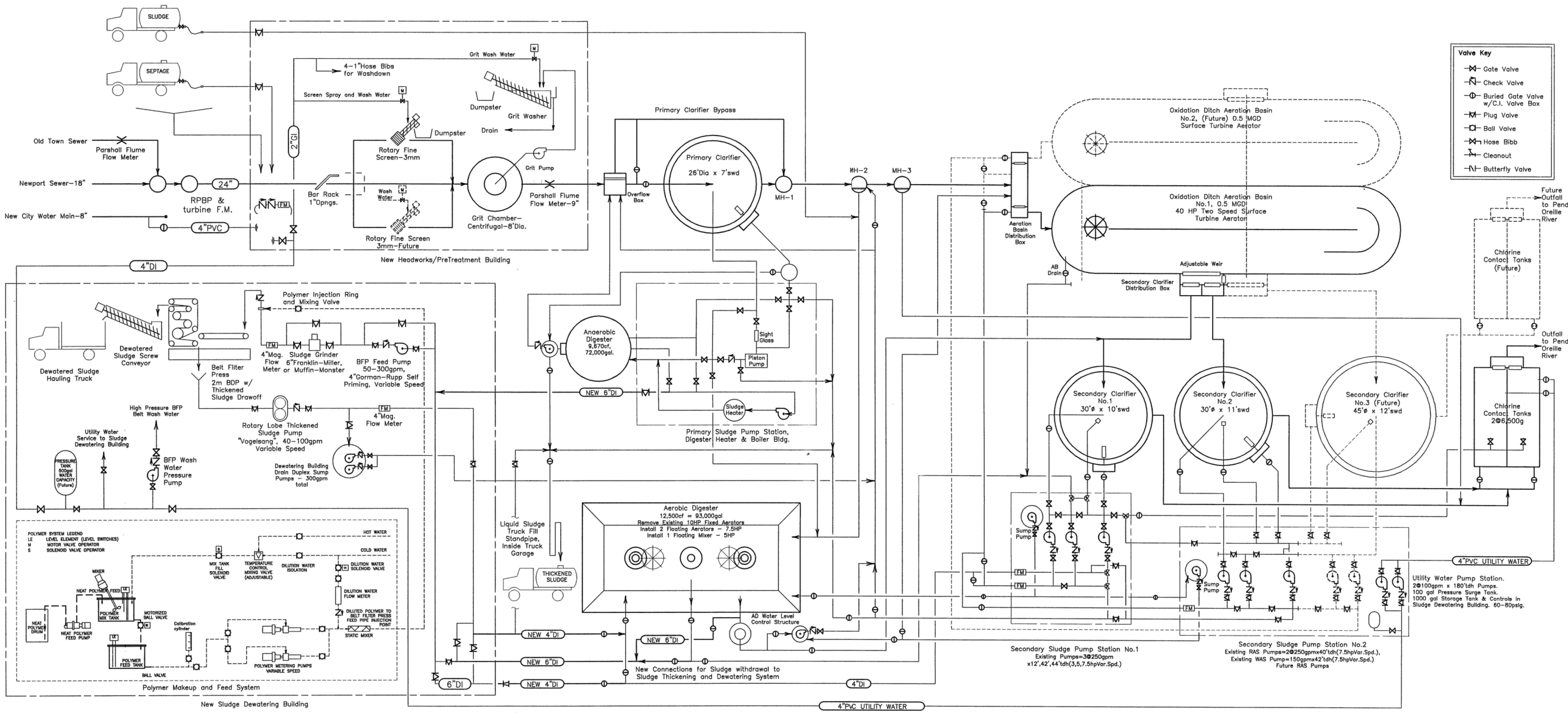
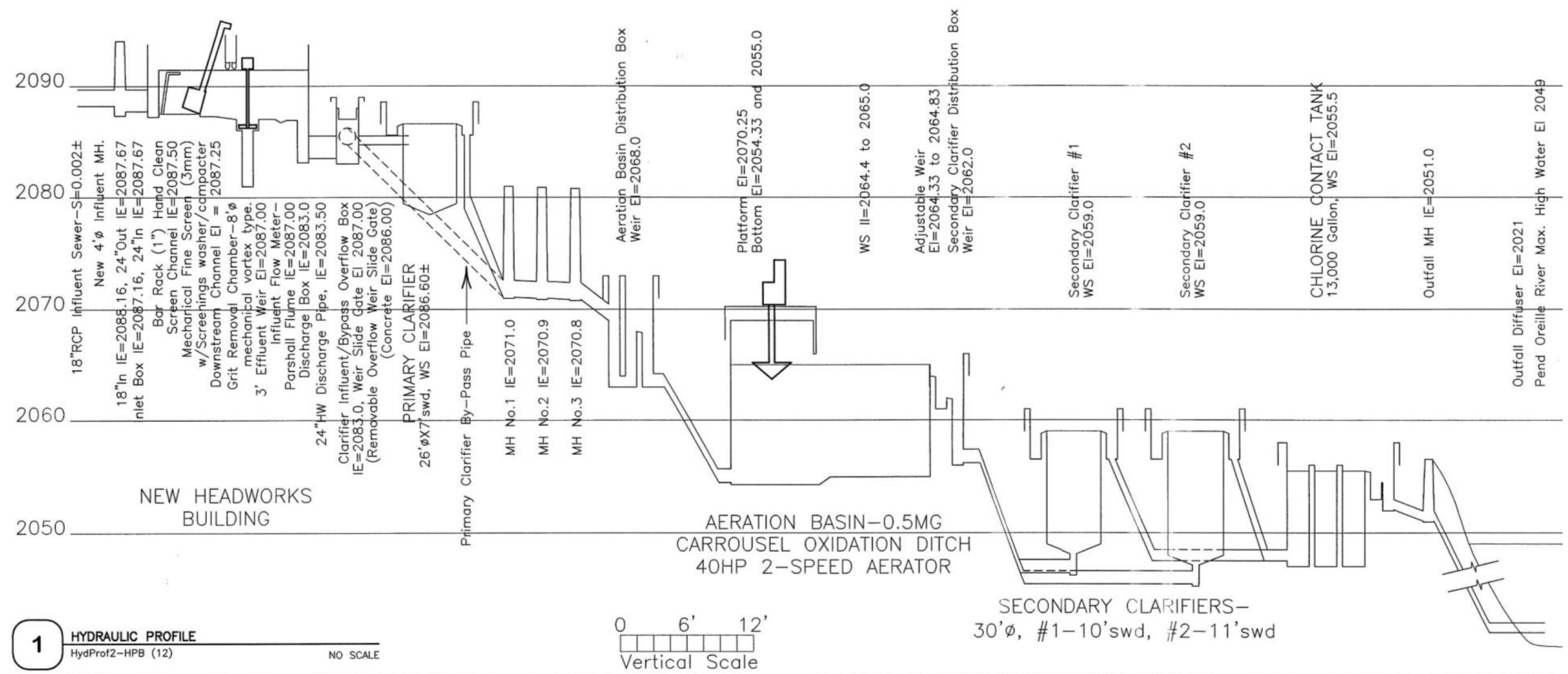


Figure 5-3: HGL Schematic



1 HYDRAULIC PROFILE
HydProf2–HPB (12) NO SCALE

DESIGN CRITERIA

	PRESENT	AFTER UPGRADE
WASTEWATER FLOW (MGD)		
AVERAGE DAILY	0.5	0.5
MAXIMUM DAY	1.0	1.0
PEAK HYDRAULIC	1.5	2.5
BOD (LB/DAY) – MAX MONTH	1,330	1,330
TSS (LB/DAY) – MAX MONTH	920	1,500

EFFLUENT LIMITATIONS (PERMIT)

PARAMETER	AVERAGE MONTHLY	AVERAGE WEEKLY
WASTEWATER FLOW	0.5MGD	– – – –
BOD (5 DAY)	30mg/L	45mg/L
	125lb/day	188lb/day
TSS	30mg/L	45mg/L
	125lb/day	188lb/day
FECAL COLIFORM BACTERIA	200/ 100 ml	400/ 100 ml
pH	Minimum 6.0	Maximum 9.0
TOTAL RESIDUAL CHLORINE	0.5mg/L	0.75mg/L
BOD & TSS REMOVAL	Minimum 85%	

TREATMENT PROCESS UNIT CRITERIA

		EXISTING / PRESENT	NEW / FOLLOWING UPGRADE (20 YR PROJECTION)
HEADWORKS		NA	
FINE SCREEN	2.3MGD CAPACITY		ROTATING FINE SCREEN WITH WASHER / COMPACTER
GRIT CHAMBER			8' DIA w/ GRIT WASHER/CLASSIFIER
VELOCITY	AVG	0.4 FT/SEC	1.0 FT/SEC – MECHANICALLY INDUCED
	MAX	0.5 FT/SEC	
PRIMARY CLARIFIER	26' DIA x 7' SWD		
SURFACE OVERFLOW RATE	AVG ANN	530GPD/SF	650GPD/SF
	MAX MO	640GPD/SF	800GPD/SF
AERATION BASIN	500,000 GAL "CARROUSEL" OXIDATION DITCH – 40 HP 2 SPEED VERTICAL TURBINE AERATOR		
F:M (MLSS = 3,000 mg/L)	AVG ANN	0.03D^(–1)	0.04D^(–1)
	MAX MO	0.05D^(–1)	0.07D^(–1)
SRT (SLUDGE AGE)	AVG ANN	62Da	42Da
	MAX MO	29Da	20Da
AOR (ACTUAL OXYGEN DEMAND RATE)	MAX DAY	1,350 #/DA	1,740 #/DA
SECONDARY CLARIFIERS	2 @ 30' DIA, 1 – 10' SWD, 1 – 11' SWD		
SURFACE OVERFLOW RATE	MAX MO	350 GPD/SF	350 GPD/SF
	MAX DAY	700 GPD/SF	700 GPD/SF
CHLORINE CONTACT TANK13,000 GALLON	13,000 gallons		
DETENTION TIME	MAX MO	37 MIN	37 MIN
	MAX DAY	19 MIN	19 MIN
ANAEROBIC DIGESTER	9,670 CU FT – HEATED, MIXED		
VOLATILE SOLIDS LOADING	AVG ANN	0.013LB Vm/DAY–CU FT	0.018LB Vm/DAY–CU FT
	MAX MO	0.026LB Vm/DAY–CU FT	0.040LB Vm/DAY–CU FT
DETENTION TIME	AVG ANN	99Da	72Da
	MAX MO	50Da	33Da
AEROBIC DIGESTER	12,500 CU FT – 2 – 10 HP FIXED AERATORS2 – 7.5 HP FLOATING AERATORS + 7.5 HP MIXER		
VOLATILE SOLIDS LOADING	AVG ANN	0.021LB Vm/DAY–CU FT	0.016LB Vm/DAY–CU FT*
	MAX MO	0.044LB Vm/DAY–CU FT	0.031LB Vm/DAY–CU FT*
DETENTION TIME	AVG ANN	32Da	INITIAL 64Da* 20 YEAR 44Da*
	MAX MO	16Da	32Da* 22Da*
			* SLUDGE THICKENED TO 4% DS
SLUDGE DEWATERING			2 METER COMBINATION BELT FILTER PRESS / GRAVITY BELT THICKENER
THICKENING RATE			1,500 LB DS/HR
DEWATERING RATE			800 LB DS/HR
THICKENING TIME	AVG ANN		INITIAL: 6 HR/WK 20 YEAR: 8 HR/WK
	MAX MO		11 HR/WK 16 HR/WK
DEWATERING TIME	AVG ANN		5 HR/WK 8 HR/WK
	MAX MO		10 HR/WK 15 HR/WK

2 DESIGN CRITERIA
HydPr–SC (12) NO SCALE

5.3 Wastewater Treatment Timeline Overview

Prior to 1950, wastewater was conveyed to a community septic tank and discharge to the Pend Oreille River. In 1950, unit process improvements were constructed to provide preliminary treatment that included a headworks facility (grit removal and comminutor), primary clarification (solids settling), disinfection and settled solids management by anaerobic digestion and drying (sand drying beds). The primary clarifier and anaerobic digester are the only remaining unit processes still on-call to provide service; however, both are non-functional due to breakdowns.

In 1972, an activated sludge aeration lagoon and secondary clarifier were constructed downstream of the primary clarifier to provide secondary treatment with activated sludge capture and return. Wasted solids were thickened on the sludge drying beds which were expanded as part of the 1972 upgrade. A new chlorine contact tank was constructed near the river outfall and river outfall improvements were made by adding a 4-port diffuser. All the facilities constructed in 1972 are still in service except for the sand drying beds; however, the steel structure covering the aeration lagoon is badly corroded.

In 1984, an activated sludge oxidation ditch was constructed as well as another secondary clarifier. The old aeration lagoon was repurposed to serve as an aerobic digester to stabilize waste activated sludge from the oxidation ditch secondary treatment system. The facilities constructed in 1984 are still in service.

In 2003, the old headworks facility was removed and replaced with modern grit removal and screening facilities. A belt filter press dewatering unit process was constructed to thicken and dewater waste activated sludge replacing the sludge drying beds which were removed from service and demolished. The dewatered biosolids are loaded into trucks and hauled off-site to Barr-Tech for composting and disposal via beneficial use. The facilities constructed in 2003 are still in service.

5.4 Overall Site Plan

The wastewater treatment plant site has systems and components that serve the overall site. Site deficiencies are noted below.

- Retaining wall repairs are needed to maintain structural integrity.
- The chain link security fence needs to be repaired due to tree damage and age.
- Foot travel paths need sidewalks and stairs for safety.
- A fire hydrant is needed onsite to be closer to critical infrastructure.
- Fire extinguisher boxes are weather damaged and need replacement.

- A SCADA system is needed to monitor the site with security cameras to reduce incidences of vandalism.
- The two main line power feeds should be combined into one feed to facilitate the incorporation of standby power.
- A standby generator is needed for the facility to continue to operate during utility outages.
- Most of the yard piping valves need to be replaced since they cannot isolate flow and the valves leak.
- The site reclaimed water system needs to be replaced. It is experiencing an excessive number of leaks due to failing glued joints in the PVC pipe.
- The primary clarifier needs to be mothballed and made safe.

5.5 Liquid Wastewater Treatment Unit Process Evaluation

Major process components that make up the overall wastewater treatment facility were evaluated to assess their condition and ability to continue providing reliable service. Individual unit process evaluations are discussed below.

5.5.1 Influent Flow Meters

<u>Component Description and Operations</u>
The City receives wastewater from Oldtown Idaho as well as the City of Newport service area in Washington. Flow from Oldtown is measured via Parshall flume (near the Idaho-Washington border) that is owned and operated by the City of Newport. The total combined flow is measured by a 9-inch Parshall flume flow meter located in the headworks building. Flow from the City of Newport is estimated by calculating the difference between the two flow meters.

The combined flow meter at the plant headworks is installed in an 18-inch wide by 40-inch-deep channel providing accurate flow measurements from 0.059 to 5.7 million gallons per day at approach depths between 1.2 and 24 inches, respectively. The flow meter has sufficient capacity and functions adequately to serve throughout the planning period.

Daily flow data are summarized by a digital data logger and displayed for the operators to note in their daily log sheets.

<u>Operational Deficiencies</u>
No operational deficiencies were noted during facility tours and discussions with operations staff.

Summary

- Performance: No concerns reported by operators.
- Reliability: No historical issues.
- Safety: No observed safety issues.

5.5.2 Receiving Station

Component Description and Operations

The WWTP has a receiving station that allows septage (septic tank solids) and sludge to be dumped at the WWTP for treatment and disposal, shown in Figure 5-4. When in service, the receiving station functioned adequately; however, the City no longer accepts septage or sludge due to operational issues associated with septage and portable toilet waste. The receiving station is a concrete pad on the west side of the headworks building with 4-inch and 6-inch cam-lock quick disconnect fittings to receive septage and sludge, respectively, from haul trucks. Septage flows by gravity from the delivery haul truck to the influent channel just upstream of the bar rack in the headworks building. Sludge also flows by gravity from the delivery haul truck to the aerobic digester.

Figure 5-4: Receiving Station (Not in Use)

Operational Deficiencies

The following deficiencies were noted during facility tours, discussions with operations staff, and the preceding assessment:

- Not currently in service.
- A possible inlet for stormwater flow if not regraded or dammed off.

Summary

- Performance: Not in use.
- Reliability: No historical issues.
- Safety: No observed safety issues.

5.5.3 Headworks – Bar Rack, Fine Screens, Grit Removal

The Headworks facility was constructed in 2003 and provides large debris screening via a bar rack, redundant fine screens, and grit removal. The influent flow meter discussed above is in the headworks building.

5.5.3.1 Bar Rack – Manually Cleaned

Component Description and Operations

The headworks receives wastewater from the 24-inch ductile iron influent sewer pipe. The influent pipe discharges into a 48”X48” concrete transition structure within the headworks building. The transition structure has a 24-inch exit channel conveying flow to the influent splitter that divides the flow between two 18-inch channels feeding the fine screens. A bar rack (1/4” bars) with 1-inch clear space openings sits within the 24-inch channel protecting the downstream unit process from large debris, shown in Figure 5-5. The bar rack is hand-cleaned by raking captured debris off the rack onto a perforated drip plate. After dripping a while, debris is forked into a dumpster for disposal via landfill. The bar rack is inspected daily and cleaned, as necessary. When cleaned periodically, the bar rack provides adequate service protecting the downstream mechanical fine screens from large debris that could damage the equipment.

Operational Deficiencies

No operational deficiencies were noted during facility tours and discussions with operations staff.

Summary

- Performance: No concerns reported by operators.
- Reliability: No historical issues.
- Safety: No observed safety issues.

Figure 5-5: Manually Cleaned Bar Rack

5.5.3.2 Influent Fine Screens

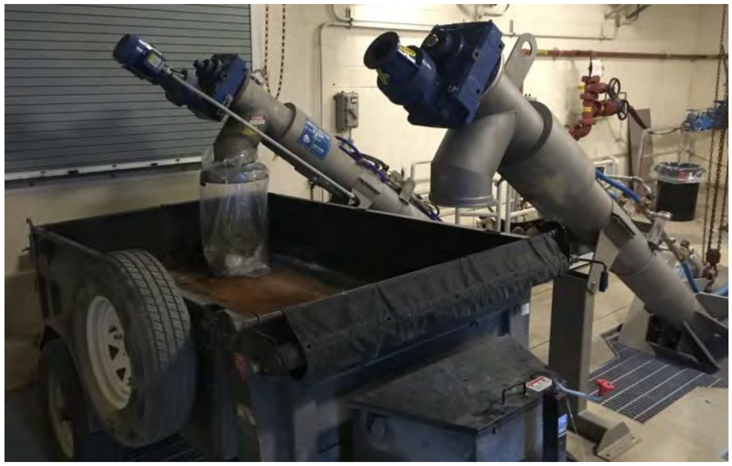
<u>Component Description and Operations</u>

The influent splitter divides flow to two channels. The two channels are sized and shaped to allow the installation of mechanical fine screening equipment. The headworks was constructed with one Lakeside Raptor Rotating Wedge Wire Drum Screen with 3 mm openings. The City has installed a second fine screen. The second fine screen is the same make and model as the original screen, with some structural and operational improvements by the manufacturer, shown in Figure 5-6.

Figure 5-6: Mechanical Fine Screen

Each screen is served by a dedicated washer/compactor to dewater screenings prior to disposal. Compacted solids are discharged into a bag and stored in a trailer for disposal via a landfill, shown in Figure 5-7.

Figure 5-7: Screenings Washer Compactor



A 24-inch channel within the headworks allows either fine screen to be removed from service for maintenance.

Operating conditions and design criteria associated with the fine screen are listed in Table 5-1. Based on the available data, the Headworks design criteria appear adequate for current conditions. Staff indicate the older fine screen operated and performed well when it was new; however, something damaged the drum causing it to be “out-of-round” which allows flow to leak past the filtering screen. The older screen is currently not performing adequately, which is the primary reason the City installed a second fine screen. The new screen is operating adequately. The old screen can be taken off-line and repaired or replaced.

Table 5-1: Fine Screen Operating Conditions and Design Criteria

Item	Actual / Observed Condition	Typical Design Condition or Range	Reference
Type of Screen	Lakeside Raptor Wedge wire Model 40RDS	-	
Screen Opening (mm)	3	2 - 6	Orange Book T3-3.1.1.A.2.c
Number of Screens	2 (1 duty, 1 standby)	-	
Peak Capacity, each (mgd)	2.0	-	Manufacturer Information
Number of Washer Compactors	One dedicated to each screen	-	

<u>Observed Deficiencies</u>

The following deficiencies were noted during facility tours, discussions with operations staff, and the preceding assessment:

- The oldest screen's drum is out-of-round, allowing unscreened flow downstream. The damage was caused by an accumulation of "wet wipes" within the drum which became very heavy and bent the drum which is not designed to support such weight. To prevent future damage, the screen will rotate more often and the wash spray will be cycled every time the drum rotates. The current electrical control of the screens does not allow each screen to operate independently of the other screen. Both screens are either on or off. Due to this, there is not an operable redundancy in the screens. Wiring and control logic need to be revised to remedy this situation.
- The existing electric furnace is old, functions poorly and is not adequate. It has been determined that this furnace needs to be replaced with an upsized and efficient heat pump unit.
- The existing bar rack channel rectangular shape tends to accumulate solids in the corners of the sampling zone, which then cause these solids to be drawn up into the sampler and cause non-representative samples. The channel needs to be reshaped so that it doesn't collect solids in the sampling zone.
- Influent channel slide gates are damaged and misshapen, very difficult to operate and therefore cannot properly isolate flow. The slide gates and rails need to be replaced.
- The staff cannot directly access the screen debris management area from ground level to remove debris. Access should be created to allow access from ground level by relocating the motor control center (MCC) room and installing a new access door in the current MCC room location.

- A traveling bridge crane is needed to lift the heavy screening and grit removal equipment in the headworks room for repairs or replacement.
- Odor control improvements are needed by covering the channels and retrofitting the ventilation fans with carbon filters.
- The potable water system needs an air gap system for proper backflow prevention of the potable water system.
- The electrical components need to be brought up to current code.

Summary

- Old components need to be replaced and/or made functional.
- Performance: The new fine screen operates well and has capacity to process flows up to 2.0 mgd which is sufficient to process projected peak flows at the end of the planning period. The older, damaged screen originally had this same capacity, but currently, the damaged drum on the older screen has limited effectiveness at removing screenable material. Adequate performance is expected when the old screen is replaced. Some carryover of debris has also been observed if the cleaning brush is not adjusted.
- Reliability: The older screen has become unreliable and does not provide adequate redundancy for the newer screen. Replacement of the older screen will restore that reliability and redundancy.
- Safety: No safety issues were identified during a site tour and discussions with the operators.

5.5.3.3 Vortex Grit Chamber

Grit increases wear on mechanical components such as pumps and clarifier rakes; therefore, it is beneficial to remove grit to prolong the service life of equipment. Additionally, grit settles in treatment basins and occupies treatment volume and is difficult to remove.

Heavier grit particles are removed in a mechanical 8-foot diameter vortex grit chamber. Removed grit is pumped to an inclined screw classifier and conveyed into a garbage can for landfill disposal, shown in Figure 5-8.

Figure 5-8: Grit Washer Conveyor

Operating conditions and design criteria associated with grit removal are listed in Table 5-2. The grit chamber is adequate for future conditions.

Table 5-2: Vortex Grit Chamber Operating Conditions and Design Criteria

Item	Actual / Observed Condition	Typical Design Condition or Range	Reference
Type	Vortex	-	
Diameter (feet)	8	-	
Number of Units	1 duty with bypass	-	Orange Book T1-1.5.6.B Manufacturer Information
Peak Capacity, (mgd)	2.3	-	
Number of Washer	1	-	

Operations staff indicate the grit removal chamber is operating well.

Observed Deficiencies

The following deficiencies were noted:

- The Smith and Loveless grit pump has difficulty priming, due to age and the freeze-prone location of the pump priming system on the exterior of the building, requiring regular additional operator attention. A new grit pump is needed, and the priming system needs to be relocated inside the heated building.
- The grit chamber piping support is structurally inadequate and needs to be replaced. During the start of the cycle, the pipe moves excessively, which over time is likely to damage the piping.

Summary

- Performance: Adequate to serve throughout the planning period with stated improvements.
- Reliability: The newer fine screen performs adequately, but there are issues with the existing fine screen and priming issues with the grit pump.
- Safety: No observed safety issues, with the exception of the need for an overhead crane for safe handling of screening and grit removal equipment, to protect worker safety.

5.5.4 Primary Clarifier

Component Description and Operations

Discharge from the headworks flows via a 24-inch ductile iron pipe to the primary clarifier overflow box where it is routed to the clarifier inlet line. Overflow is routed to the secondary treatment system (oxidation ditch). At this time, the primary clarifier is off-line; therefore, all of the flow is routed to the secondary treatment, bypassing primary treatment.

Primary clarification is a gravity settling unit process wherein a quiescent environment is provided to allow some solids to settle out of the bulk liquid. Settled solids are removed from the bottom of the basin and pumped to the anaerobic digester. Floating scum is pushed to the scum trough and conveyed to the aerobic digester via a 6-inch gravity line. The clarifier should remove 30 to 35 percent of the influent BOD and 50 to 60 percent of the influent TSS. Typical operating conditions (if operational) and design criteria are listed in Table 5-3 based on flows listed in Table 3-6.

Table 5-3: Primary Clarifier Operating Conditions and Design Criteria

Item	Condition			Typical Design Condition or Range	Reference
	Existing 2020	Future 2041	Design		
Diameter, feet	26				
Side Water Depth, feet	7			8-14 **	
Volume, gallons	28,800				
Area, ft^2	530				
Ave loading, gpd/sf*	377	660	650	800-1200	Orange Book T2-2.1.1
HRT, hours	3.34	1.91	1.93	2.5	Metcalf & Eddy
Max Month Loading, gpd/sf*	490	885	800		
HRT, hours	2.57	1.42	1.57		
Peak Loading, gpd/sf*	753	1337	2000	2000-3000	Orange Book
HRT, hours	1.67	0.94	0.63	1.5	T2-2.1.1

**Unit not in operation, value given assumed normal operation.*
*** Typical to achieve desired HRT*

Of note is the design average loading value of 650 gpd/sf is lower than the typical design range of between 800 and 1200 gpd/sf. The lower expected design value is due to the rather shallow depth of the primary clarifier and insufficient volume to provide adequate detention time for proper function. The primary clarifier does not have adequate capacity to serve through the end of the planning period. Additionally, the clarifier mechanical equipment is badly worn and currently off-line, shown in Figure 5-9.

The concrete is structurally sound and generally serviceable with minor surface pitting. The concrete basin could be retrofitted and brought on-line for continued service.

The rotating mechanism that pushes settled solids to the sump pit and floating scum to the scum trough cannot be repaired and has been removed from service. Replacement parts are not available due to the age of the equipment and custom-made parts cannot reliably be fitted to the mechanism due to the fragility of the old metal.

To bring the primary clarifier on-line, all the mechanical parts will need to be replaced.

Figure 5-9: Primary Clarifier (Not Operational)

Observed Deficiencies

The following deficiencies were noted during facility tours, discussions with operations staff, and the preceding assessment:

- No redundancy is available. When the primary clarifier is off-line for service, the secondary treatment facilities must manage 100 percent of the BOD and TSS load.
- The concrete basin is structurally sound and could be retrofitted for service; however, performance will be reduced due to the unit not being adequately size for future flow.
- The mechanical parts are not functional and need to be replaced.
- The primary sludge pumps are out of date and need to be replaced.
- The unit is too small to provide adequate service at future flows.
- Electrical components require replacement to meet current code.

Summary

- Performance: The unit process is not functional and currently out of service.
- Reliability: When operational, the unit process had a low percent of time in service due to mechanical failure.

- Safety: There were no safety concerns when the unit process was operational.
- The unit does not have capacity to serve to the end of the planning period.

5.5.5 Secondary Treatment

Secondary treatment is a biological treatment process that uses bacteria to consume (metabolize) organic matter in wastewater. Wastewater is retained in an aerated basin (aeration basin) where oxygen is mechanically added to the liquid. Bacteria use the oxygen to metabolize the wastewater organic matter. Bacteria leaving the aeration basin are captured in a secondary clarifier and returned to the aeration basin to be used several times for treatment. The clarified effluent is disinfected and discharged to the Pend Oreille River.

Since the primary clarifier is off-line, flow is routed from the primary clarifier overflow box directly to the secondary treatment system flow splitter box. When the secondary system was constructed, a second oxidation ditch was planned for future expansion and a splitter box was built to allow the flow to be split between the two ditches. Flow from the splitter box enters the southwest end of the aeration basin.

5.5.5.1 Aeration Basin - Oxidation Ditch

Component Description and Operations

The secondary treatment aeration basin is a 380,000-gallon “racetrack” type basin with a dividing wall in the middle as shown in Table 5-8. Flow circulates around the ditch providing conditions similar to a continuously stirred tank reactor. The mechanical aerator has a spinning impeller underwater that induces circulation around the ditch in addition to adding oxygen to the water.

Figure 5-10: Aeration Basin

The oxidation ditch aeration basin functions well; however, the concrete discharge structure is damaged and cannot be repaired without taking the unit process off-line, which cannot be done without a redundant unit process. The damage prevents the function of the discharge weirs, which makes it difficult to isolate the downstream clarifiers and ensure an even flow split between the two clarifiers.

Operating conditions and design criteria associated with oxidation ditch aeration basins are listed in Table 5-4. Based on the available data, the aeration basin appears adequate for current conditions and has capacity to serve to the end of the planning period.

Aeration is provided by one 40-hp two speed vertical turbine mechanical aerator. A spare motor is kept on site for quick repair. In general, the aerator can supply three times the amount of oxygen normally required to treat the influent BOD.

The ability of the oxidation ditch to serve to end of planning period is adequate with or without the primary clarifier in use; however, performance will need to be verified.

Observed Deficiencies

The following deficiencies were noted during facility tours, discussions with operations staff, and the preceding assessment:

- There is no redundant oxidation ditch, which makes it impossible to take this process offline for repairs.
- The basin has an unknown volume of settled material in the bottom.
- The concrete outlet structure is damaged and prevents the proper function of the outlet weirs, causing more flow to be sent to secondary Clarifier #1. It is not possible to adjust the flows to the clarifiers when influent flow varies.
- The 40-horsepower aerator/mixer does not have variable speed control and often works harder than needed and is not very efficient.
- The shed covering the aerator needs to be replaced.
- The aerator does not have back up power and therefore does not provide adequate treatment during a power outage.
- There is no redundant aerator in the event of aerator failure.
- There is limited freeboard due to the construction of the ditch channel walls.
- The foam spray system needs to be replaced.

Summary

- Performance: Good with or without primary clarifier, performance should be monitored.
- The effluent distribution box needs repair to allow for flow distribution.
- Reliability: Good history but cannot be taken off-line. Aeration unit not functional during a power outage.
- Safety: No observed safety issues.

Table 5-4: Oxidation Ditch Operating Conditions and Design Criteria

Item		Condition			Typical Design Condition or Range	Reference
		Existing 2020	Future 2041	Design [A]		
Oxidation Ditch						
Active Volume (MG)		0.38		0.38		
Number in Operation		1		1		
Flow, MGD						
Average Day		0.20	0.35	0.5		
Maximum Month		0.26	0.47	1.0		
Peak Day		0.4	0.71	1.3		
Hydraulic Residence Time (days)						
Average Day		1.9	1.09	0.79	0.33 - 1.5	M&E (2013)
Maximum Month		1.46	0.81	0.38		
Peak Day		0.95	.71	0.29		
Solids Residence Time, SRT (days)					10 - 30	M&E (2013)
Average Day	With Primaries	104	60	42		
	Without Primaries	43	24	17		
Maximum Month	With Primaries	79	43	20		
	Without Primaries	39	22	10		
MLSS Concentration (mg/L)		3,000	3,000	3,000	1,500 - 5,000	M&E (2013)
Yield (lb biomass/lb BOD removed)					0.4 - 0.8	M&E (2013)
Average		0.6	0.6	0.6		
Maximum Month		0.7	0.7	0.7		
Food:Microorganism (F:M)					0.05 - 0.3	M&E (2013)
Average	With Primaries	0.016	0.028	0.04		
	Without Primaries	0.039	0.068	0.1		
Max Month	With Primaries	0.018	0.033	0.07		
	Without Primaries	0.036	0.066	0.14		
Volumetric Load (lb BOD/1,000 ft^3/day)					5 - 30	M&E (2013)
Average	With Primaries	3.0	5.2	7.5		
	Without Primaries	7.3	12.8	18		
Max Month	With Primaries	3.4	6.1	13.1		
	Without Primaries	6.8	12.3	26		

Item		Condition			Typical Design Condition or Range	Reference
		Existing 2020	Future 2041	Design [A]		
Aeration System						
Aerator Type, 2 Speed Vertical Turbine						
Horsepower						
Average, ~ at 46.5 Hz		31		31		Record Drawings
Maximum Month, ~ at 60 Hz		40		40		Record Drawings
Oxygen Supply Rate, lb O_2/ Hp / Hr		1.82	1.82	1.82	1.2 – 2.4	M&E (2013)
Oxygen Supply Rate, lb O_2/day						M&E (2013)
Average, ~ at 46.5 Hz		1350	1350	1350		Calculated
Maximum Month, ~ at 60 Hz		1740	1740	1740		Calculated
Pounds of Oxygen per pound of BOD					0.90 – 1.3	Orange Book
Oxygen Supplied per Pound of BOD						
Average [B]	With Primaries	8.9	5.1	3.6		Calculated
	Without Primaries	3.6	2.1	1.5		
Max Month [B]	With Primaries	10.1	5.6	2.6		Calculated
	Without Primaries	5.0	2.9	1.3		

[A] Design Values from Record Drawings
[B] Actual Oxygen Supply Rates

The oxidation ditch has sufficient capacity to serve to the end of the planning period

5.5.5.2 Secondary Clarifiers

<u>Component Description and Operations</u>

Discharge from the oxidation ditch flows through a distribution box then to the secondary clarifiers where the biological solids are removed from the wastewater via a gravity settling process. Clarified wastewater exits the process and flows to the disinfection unit process prior to discharge to the Pend Oreille River. Biological solids that settle out of the wastewater are raked to a sump in the center of the clarifier and are removed for reuse and/or additional processing prior to disposal by the activated sludge pump stations, shown in Figure 5-11. Flow purposed for reuse is returned to the oxidation ditch to “seed” the influent with biological mass.

The wastewater treatment plant has two 30-foot diameter clarifiers. Clarifier No. 1 has a 10-foot side water depth and clarifier No. 2 has an 11-foot side water depth. Typical operating conditions and design criteria are listed in Table 5-5.

Figure 5-11: Secondary Clarifier

Observed Deficiencies

The following deficiencies were noted during facility tours, discussions with operations staff, and the preceding assessment:

- The concrete in both secondary clarifiers is serviceable; however, the metal weirs need to be resurfaced and installed level.
- Clarifier #2's effluent channel has an adverse slope which needs to be fixed.
- There is insufficient capacity to take one of the clarifiers offline.
- The mechanical parts in both clarifiers are deteriorated and need to be replaced.
- Mechanical system failure rate is too high, causing only one clarifier to be in service while repairs are made, which reduces performance and sends solids to the chlorine contact basin.
- Uneven flow split overloads Clarifier #1, causing solids to flow to the chlorine contact basin.
- Electrical components need to be brought up to code.

Summary

- Performance: No concerns reported by operations staff when working properly and flow is split evenly; however, frequent repairs without sufficient redundancy degrade overall performance in addition to the decreased performance due to uneven flow splitting.

- Reliability: Poor due to age of mechanical equipment and lack of redundancy. Units have a good history when newer but aging equipment increases operational down time.
- Shallow depth likely decreases performance; however, uneven flow split seems to have a more noticeable performance impact.
- Safety: No observed safety issues.

Table 5-5: Secondary Clarifier Operating Conditions and Design Criteria

Item	Condition [A]			Typical Design Condition or Range	Reference
	Existing 2020	Future 2041	Design		
Diameter, feet	30				
Side Water Depth, feet, #1 and #2	10 and 11				
Hydraulic Loading, gpd/sf					
Ave loading	141	248	350	200-400	M&E (2013)
Maximum Month	184	332			
Peak Loading	283	502	700	700	Orange Book T3-3.1.1.B
Solids Loading, lb/sf/hr					M&E (2013)
Ave loading	0.29	0.52	0.74	1.0	
Maximum Month	0.38	0.69			
Peak Loading	0.6	1.05	1.47	1.4	

[A] Two clarifiers in service

The secondary clarifiers have sufficient capacity to perform throughout the planning period; however, a clarifier cannot be taken out of service during peak flow season. Additionally, both clarifiers are shallow and suffer poor performance, making it difficult to maintain permit compliance when one clarifier is offline; therefore, a third clarifier is needed to maintain reliable service with one unit offline.

5.5.6 Chlorine Contact Basin and Chlorine Injection Equipment

<u>Component Description and Operations</u>

Clarified secondary effluent flows to the chlorine contact basin for disinfection prior to discharge to the Pend Oreille River. Chlorine gas is used to disinfection the clarified effluent. Chlorine is drawn from a gas cylinder using a vacuum regulator. A vacuum switching valve is used to switch from the duty cylinder to the standby cylinder when the duty cylinder is empty. Chlorine dose is controlled via a dosing regulator with a servomotor based on the effluent flow rate and the operator's desired chlorine concentration. Gas chlorine is injected into the chlorine feed line via a vacuum injector.

The chlorination feed line is final effluent that is pumped from the downstream end of the contact basin to the chlorine injection point then routed to the upstream side of the contact basin; thereby delivering chlorine prior to the contact tanks.

Chlorine gas and equipment are housed in a special room with a gas detection sensor that alarms should chlorine gas be detected.

The gas system has the capacity to serve throughout the planning period. Flow is split between two parallel channels, shown in Figure 5-12. Typical operating conditions and design criteria are listed in Table 5-6.

Figure 5-12: Chlorine Contact Basin

Table 5-6: Chlorine Contact Basin Operational Condition and Design Criteria

Item	Condition			Typical Design Condition or Range	Reference
	Existing 2020	Future 2041	Design		
Volume, gallons	13,000				
Detention Time, minutes				15-45	M&E (2013)
Average	94	53	37		
Max Month	72	40			
Peak Hour	31	18	19		“

The chlorine contact basin has sufficient capacity to serve to the end of the planning period.

<u>Observed Deficiencies</u>

The following deficiencies were noted during facility tours, discussions with operations staff, and the preceding assessment:

- The concrete is cracked but the cracks do not interfere with normal operations.
- The gas chlorine system was installed in 1972. While reliable and safe, the equipment needs frequent repairs.
- The auto-sampler freezes in the winter.
- Gas chlorine is a potential safety issue but, to date, there have been no issues.
- Frequent repairs to the secondary clarifiers have caused solids to flow into and settle in the basins.
- Uneven flow splitting from the oxidation ditch outlet structure overloads Clarifier #1, which reduces solids capture, causing solids to flow into and settle in the chlorine contact basins.

<u>Summary</u>

- Performance: Adequate through the planning period; spikes in the collection system inflow and infiltration (max day flows) should be reduced for continued reliability for chlorine contact treatment process.
- Reliability: Good due to parallel channels
- Safety:
  - o No issues associated with the concrete basin
  - o Chlorine gas used for disinfection is highly toxic and extremely hazardous if not properly handled and managed with care. Dangers are (may not be inclusive):
    - ▪ Fatal if inhaled

- Skin burns
- Eye damage
- Toxic to aquatic life
- Cylinder may explode if heated
- May cause or intensify fire (oxidizer)

5.5.7 River Outfall

Component Description and Operations

The outfall manhole receives treated effluent from the chlorine contact tank and has 10-inch outfall pipe. The 10-inch diameter cast-iron pipe runs down the riverbank at a steep angle (slope ~ 0.345 ft/ft) about 50 feet and transitions to a newer 8-inch ductile iron pipe for about 114 feet then connects to a 4-port diffuser and discharges effluent into the Pend Oreille River under the lowest water surface level elevation. The diffuser is 15 feet long with a 3-inch port at 0 feet (point of connection), a 4-inch port at 5 feet, a 5-inch port at 10 feet, and a 4-inch port at 15 feet. The outfall has an estimated capacity over 1.5 MGD which is sufficient to serve throughout the planning period. The outfall diffuser is about 69 feet from the riverbank at low flow and about 89 feet at high flow between 13 and 24 feet deep. The diffuser is 15 feet long.

The riverbank at the cast-iron outfall pipe is showing signs of erosion. Riverbank stabilization will be necessary as the erosion continues to encroach on the outfall pipe and the aeration basin. The City and a consulting Engineer have a complete set of plans for this project and the Joint Aquatic Resource Permit Application (JARPA) process has been initiated.

Observed Deficiencies

- Riverbank erosion at outfall location.

Summary

- Performance: No concerns reported by operations staff.
- Reliability: No historical issues.
- Safety: No issues.

5.6 Solids Wastewater Treatment Unit Process Evaluation

Primary and secondary solids are separated from the bulk liquid via gravity settling unit processes (secondary clarifiers, see above discussion). Once removed, solids are further treated for stabilization prior to disposal. If the primary clarifier were in operation, primary solids would be pumped from the bottom of the primary clarifier to the anaerobic digester where a portion of the volatile solids would be biologically converted to methane, carbon dioxide, biomass and water, thereby decreasing the volatile fraction of the solids (stabilization). Secondary solids are pumped from the bottom of the secondary clarifier to the aerobic digester where a portion of the volatile solids are biologically converted to carbon dioxide and water, thereby decreasing the volatile fraction of the solids (stabilization). Once stabilized to the extent practical, the solids are dewatered and hauled off-site to be land applied for beneficial reuse. These facilities are discussed below.

5.6.1 Primary Sludge Pump Station

5.6.1.1 Primary Solids Pumps

<u>Component Description and Operations</u>
Solids settled in the primary clarifier, if the clarifier were in operation, are raked to the center sump where they are conveyed via a 6-inch cast iron pipe to a pump pit. From the pump pit, primary solids are pumped to the anaerobic digester by one positive displacement piston pump.

The piston pumps are original equipment (1950). The pumps are currently off-line due to their poor condition, which was a consideration when evaluating whether to repair the primary treatment process.

The pumps are outdated and cannot be made operational. The primary solids pumps need to be replaced. Duplex solids handling pumps are required to replace the primary pumps with a capacity to convey around 2,000 gallons per day of primary solids (at 3% solids).

<u>Observed Deficiencies</u>
The following deficiencies were noted during facility tours, discussions with operations staff, and the preceding assessment:

- Pumps are outdated, not in use, and cannot be made operational. Pumps need to be replaced, if/when the Primary Clarifier is rehabilitated.

<u>Summary</u>

- Performance: Out of service.
- Reliability: Out of service
- Safety: Confined space to access.

5.6.2 Anaerobic Digester, Standard Rate

Component Description and Operations

Solids removed in the primary clarifier are pumped to the anaerobic digester for biological stabilization. The anaerobic digester is a large underground tank with an above ground process control building next to the digester, shown in Figure 5-13. The digester has never had a significant overhaul to update or modernize the unit process; therefore, the ancillary equipment (pumps, pipe, valves and all of the biogas system) needed to keep the digester functioning is old and out of date. Additionally, the old electrical components do not meet current codes for their installed location. Repair and replacement parts are difficult to find. Operating and maintaining the anaerobic digester, while meeting current requirements, is nearly impossible; therefore, the unit process is off-line.

Figure 5-13: Anaerobic Digester (Underground) and Primary Sludge Pump Station

Should the anaerobic digester be made fully operational by replacing all of the ancillary equipment and electrical components the operating conditions and design criteria are listed in Table 5-7. Should the digester be made operational, the unit process has the ability to serve through the end of the planning period.

Table 5-7: Anaerobic Digester Operating Conditions and Design Criteria

Item	Condition			Typical Design Condition or Range	Reference
	Existing 2020	Future 2041	Design		
Volume, gallons*	72,000				
Diameter, feet, inside	28				
Side Water Depth, feet	16				
Detention Time, days					
Average*	180	103	72	30-60	Orange Book, Table S-4
Max Month*	127	71	33	30	“
Mixing, hp / 1000 gallons				0.025 – 0.04	M&E (2013)
Average Loading, lb VS/ft^3-day*	0.007	0.012	0.018	0.03 - 0.3	Orange Book, Table S-4

**Unit not in operation, value given assumed normal operation.*

Observed Deficiencies

The following deficiencies were noted during facility tours, discussions with operations staff, and the preceding assessment:

- The digester is difficult to clean and therefore has never been cleaned. To gain full capacity the digester will need to be cleaned out of accumulated and settled solids.
- The mixing pumps and control panels are out of date and need to be replaced with modern pumps that meet current electrical codes.
- The gas management equipment is out of date and needs to be replaced.
- The boiler is unreliable and out of date and needs to be replaced.
- All of the electrical equipment does not meet current codes.

Summary

- Performance: Adequate when online; however, the process has been offline since 2018. Overall condition of ancillary equipment is poor due to being out-of-date. Full inspection is not available due to confined space concerns.
- Reliability: Out of service
- Safety: Confined space prohibits full inspection and operational access. Out-of-date ancillary equipment is a concern for fire or electrical risk to operators. Indications of an explosion and fire in the pump room are evident.

5.6.3 Activated Sludge Pump Station 1 and 2

Component Description and Operations

Activated Sludge Pump Station 1 was constructed to serve clarifier 1 when the aeration lagoon (now aerobic digester) and clarifier 1 were constructed. The station is outfitted with 3 Vaughan vortex pumps each capable of pumping 250 gpm, shown in Figure 5-14. Two pumps are dedicated to pumping return activated sludge (RAS) back to the aeration basin and one pump is dedicated to pumping waste activated sludge (WAS) to the dewatering facility. The pumps are interconnected to allow all three pumps to pump either WAS or RAS. The pump station is adequately sized to serve throughout the planning period and to the capacity of the treatment plant in conjunction with sludge pump station 2.

Figure 5-14: Activated Sludge Pump Station Pump Room

Sludge pump station 2 was constructed to serve clarifier 2 when the oxidation ditch and clarifier 2 were constructed. The station is outfitted with 3 Vaughan vortex pumps each capable of pumping 250 gpm. The three 250 gpm pumps are dedicated to pumping return activated sludge back to the aeration basin. The pump station is adequately sized to serve throughout the planning period and up to the capacity of the treatment plant in conjunction with sludge pump station 1.

Observed Deficiencies (both sludge pump stations)

The following deficiencies were noted during facility tours, discussions with operations staff, and the preceding assessment:

- Ground water leaks into the pump room.
- The sump pump often fails and plugs.
- Numerous valves do not work well and need significant repairs or replacement.
- The pump bleeder valves plug shut (blockages).
- The pump room in each building has poor ventilation.
- The pump room areas do not comply with current confined space entry requirements, including ingress and egress provisions and proper air exchange requirements.
- Current Class 1 Division 1 Electrical Standards will have to be met.
- No SCADA monitoring system is currently installed and monitoring must therefore be conducted onsite in the facility.
- Main line power is fed into the electrical panel and then feeds other parts of the wastewater treatment plant. If the pump station power panel has service issues, power to the WWTP may be lost.

Summary

- Performance: The pumps perform adequately; operation of valving is difficult or in some cases not possible.
- Reliability: No historical issues.
- Safety: The spiral staircases are narrow and steep and must be used with caution. The pump room space is tightly packed with pipes/pumps/valves and appurtenant equipment which makes working in the pump room difficult. Poor ventilation is a concern for operators. These issues create concerns about safe confined space entry.

5.6.4 Aerobic Digester / Solids Holding Lagoon

Component Description and Operations

The 93,000-gallon basin was constructed in 1974 to serve as an aeration basin and was converted to an aerobic digester in 1983 when the oxidation ditch aeration basin was brought on-line. The 1983 project also covered the basin with a metal building. The large size of the converted aeration basin provides more than enough hydraulic detention time to provide typical aerobic digestion treatment for the waste activated sludge. The aerobic digester is 10 feet deep and is currently outfitted with two 7.5-hp floating aerators and one 7.5-hp floating mixer. The all-metal building is corroding severely and is in poor condition, as shown in Figure 5-15. Typical operating conditions and design criteria are listed in Table 5-8. The aerobic digester is lightly loaded and has plenty of capacity to produce Class B biosolids throughout the planning period, however, the City disposes of dewatered biosolids at Barr-Tech without classification and therefore does not sample biosolids.

Figure 5-15: Aerobic Digester and Steel Building Enclosure

Table 5-8: Aerobic Digester Operating Conditions and Design Criteria

Item	Condition			Typical Design Condition or Range	Reference
	Existing 2020	Future 2041	Design		
Volume, gallons	93,500				
Detention Time, days				10-15 *	Orange Book, Table S-4
With Primaries	256	142	67		
Without Primaries	128	71	33		
Oxygen Field Transfer, lb O_2/hp-hr			1.8	1.2 -2.0	M&E (2013)
Existing O_2 supplied, lb/day	652	652	652		
WAS digested/stabilized	42.5	77	163		
Oxygen Supplied, lb O_2/hp-hr	15	8.5	4.0		
Mixing, hp / 1000 ft^3	1.8	1.8	1.8	0.75 – 1.5	M&E (2013)
Loading, lb VS/ft^3-day				0.1-0.3	Orange Book, Table S-4
With Primaries	0.007	0.013	0.018		
Without Primaries	0.018	0.031	0.045		

** Increase for cold temperature operation*

<u>Observed Deficiencies</u>

The following deficiencies were noted during facility tours, discussions with operations staff:

- The steel building covering the aerobic digester is badly corroded and needs to be replaced. The aerator hoist beam attached to the building is also badly corroded and does not extend far enough for extraction of the aerator at the far end of the beam.
- Concrete basins have not been inspected for leaks for an extended period of time.

<u>Summary</u>

- Performance: No concerns reported by operations staff.
- Reliability: No historical issues.
- Safety: Building may not be safe.

5.6.5 Belt Filter Press (Dewatering) / Gravity Belt Thickener

<u>Component Description and Operations</u>

The solids thickening/dewatering unit process has two components:

- A gravity belt thickener that increases:

    - The solids content of waste activated sludge to about 4% prior to pumping the thickened flow to the aerobic digester
    - The solids content of digested biosolids (both anaerobic and aerobic) to about 4% prior to dewatering in the belt filter press.
- A belt filter press that receives thickened digested biosolids from the gravity belt thickener and dewaters the biosolids to about 15% solids content.

The belt filter press components (manufactured by EBT) are housed in a CMU building, shown in Figure 5-16.
The dewatered biosolids are dropped onto a conveyor whereon they are carried outside the building and dumping to a truck and hauled off-site for disposal via beneficial reuse.

Typical operating conditions and design criteria are listed in Table 5-9.

Figure 5-16: Belt Filter Press

Table 5-9: Belt Filter Press / Gravity Belt Thickener Operating Conditions

Item	Condition		Typical Range	Reference
	Existing 2020	Design		
Width, meter		2		
Thickening Rate, lb dry solids / hour		1500		
Time, hours / week Average	3.3	8	Hrs < 12	Engineering opinion for normal work hours when unit thickens and dewaters. Overtime could increase hours.
Time, hours / week, Max Month	6	16	Hrs < 20	
Dewatering Rate, lb dry solids / hour		800		
Time, hours / week Average	3.3	8	Hrs < 12	
Time, hours / week, Max Month	5.7	15	Hrs < 20	

<u>Observed Deficiencies</u>

The following deficiencies were noted during facility tours, discussions with operations staff:

- Conveyor leaks / drops solids. The shower boxes that rinse the sludge off the conveyor belt spray the solids up where they get trapped. These need to be replaced with shower boxes that spray down as well as up and can effectively clear the conveyor.
- Polymer room heaters do not work. All of the heaters in the building are due for replacement.
- An eye wash station is needed in the polymer room.
- The lighting in the building is approaching the end of its useful life and requires replacement.
- Spray nozzles plug (spray system uses utility reclaimed water system). Either upgrades to the reclaimed water system or a connection to the external domestic water feed line would resolve this issue.
- Polymer system and control panel upgraded in 2021.
- The bladder pressure tank supplying storage for the wash water system needs replacement.
- The sludge thickener system is not functional and will require rebuilding.
- The pumps in the lift station outside the building have reached the end of their expected life and need replacement, along with full length pump rails for extraction.
- The wall around the conveyor for the dewatered sludge requires repairs prevent heat loss from the main section of the building.

Summary

- Performance: The performance of the processes is generally functional, but not optimal.
- Reliability: Intermittent in that it works well when operational but when out of service it takes a long time to repair since it takes at least two people to work on such a big piece of equipment and it takes some time to get parts. Process is reliable because the aerobic digester has extra volume to store WAS.
- Safety: Good.

5.7 Miscellaneous

5.7.1 Yard Piping

Component Description and Operations

A network of pipes connects the different unit processes. A schematic of the piping between the processes is shown in Figure 5-2.

Observed Deficiencies

Many of the yard piping control valves throughout the plant are no longer operational, making it difficult or impossible to re-route or bypass flows, or to shut down systems if leaks occur or operational changes are needed.

Summary

- Performance: Adequate under normal operation, as long as changes in flow are not required.
- Reliability: Numerous valves are no longer functional, which could result in challenges if there are operational issues.
- Safety: Good, no issues reported.

5.7.2 Water Use

Component Description and Operations

The WWTP uses approximately 171,000 gallons per month of municipal water for day-to-day operations. This use is high because the reclaimed water system is not functional. The WWTP uses City water at the headworks building, sludge pump station, and belt filter press. There are five 3/4-inch double check valve backflow prevention assemblies and one two-inch double check valve backflow prevention assembly installed at the headworks. However, backflow prevention assemblies do not meet current cross connection control requirements for Washington State.

<u>Observed Deficiencies</u>
Even though the WWTP has processes in place to reuse water on site for other treatment processes, that system experiences an excessive number of leaks and needs to be replaced. Replacement of the reclaimed water system would greatly reduce the use of domestic water for the plant processes.

Backflow prevention does not meet current standards and needs to be upgraded.

<u>Summary</u>

- Performance: The domestic water system performs reasonably, but the reclaimed water system has excessive leaks.
- Reliability: the reclaimed water system is no longer reliable, resulting in the need to use domestic water.
- Safety: No concerns observed or expressed by staff.

5.7.3 Electrical Service

<u>Component Description and Operations</u>
The Newport WWTP has two power feeds. One feed comes from a pole mounted utility transformer at the laboratory and a second feed enters via a buried vault transformer at the lower end of the facility. Both electrical services are provided by Pend Oreille Utility District. The two feeds do not back each other up.

The WWTP is not outfitted with any standby power generator; therefore, no backup power is available the plant to maintain treatment during a power outage. Combining the power feeds into one would facilitate the installation of a backup power system.

<u>Observed Deficiencies</u>

Regarding utility power:

No operational deficiencies were noted during facility tours and discussions with operations staff.

Regarding standby power:

Power during a utility power outage is not provided and therefore deficient.

<u>Summary</u>

- Performance:
  - Utility power: Good.
  - Standby: None, therefore Poor

- Reliability:
  - Utility power: Good.
  - Standby: None, therefore Poor
- Safety:
  - Utility power: Good.
  - Standby: None, therefore Poor

5.8 Current Operation

As stated above, the primary clarifier and the anaerobic digester are off-line due to the age and difficulty keeping the old equipment operational. The organic and solids load that would normally be removed in the primary tank are treated in the secondary biological treatment process (oxidation ditch) which increases the load to the secondary system. Therefore, the secondary system does not have the capacity it normally would if the primary tank and anaerobic digester were on-line. However, even with primary treatment off-line the secondary system is not expected to be overloaded throughout the planning period. The expected capacity of the WWTP remains at the design capacity of 0.5 MGD max month flow and 1.0 MGD peak day flow.

Primary and secondary treatment work together to optimize treatment. To gain the full potential capacity of the WWTP, both unit process would need to be fully operational.

5.9 Evaluation Summary

A summary of each unit process and the associated building facilities is included in Table 5-10 based on discussions in the preceding sections of this chapter.

Table 5-10: Summary of Observed Unit Process and Building Conditions

Item	Observed Conditions
Influent Flow Meters	• Measures inflow between 0.059 MGD to 5.700 MGD • Recording log antiquated
Receiving Station	• Not in use for septic haulers to dump • May need to be regraded to block storm water flow from entering (I/I)
Headworks	• The channel upstream of the bar rack where sampling occurs accumulates grit and solids, which impact the sampling process. This channel needs to be re-shaped. The sampler, located outside the building, has experienced freezing and needs to be enclosed or moved indoors. • Normal operation for bar rack, but upstream channel gates are very difficult to operate and require replacement.

Item	Observed Conditions
	• The original mechanical fine screen had issues with warping and not being fully efficient, requires replacement with a newer version of the screen. • Second mechanical fine screen installed, 2.0 MGD capacity per screen. • The electrical controls for the screens require that both screens be shut off if service is required. The controls need to be updated so that one screen can continue to run while the other screen is offline. • Vortex grit chamber operating well, some difficult priming, pump piping needs support. • Heating system is problematic and requires replacement. • Configuration of building makes maintenance of equipment difficult and potentially hazardous to workers due to challenging access and lack of lifting equipment. Reconfiguration of the building and the addition of a traveling bridge crane would resolve these concerns. • The retaining walls on the exterior of the building are deteriorated in several sections and require repair. • An air gap skid system is needed for the domestic water feed into the building to prevent cross-contamination of the drinking water system.
Primary Clarifier	• Offline, not in use • Would require extensive rehabilitation to resume use. • All components need replacement
Secondary Treatment – Oxidation Ditch	• Functions well, plenty of capacity. • Concrete outlet structure damaged, in need of repair • 40-hp aerator/mixer does not have variable speed control, often works harder than needed, not efficient, requires upgrade. • Aerator shed needs replacement. • Grating on the inlet distribution box is badly corroded and needs replacement. • No backup power in event of power outage. • Foam spray system is inadequate and needs replacement.
Secondary Clarifier(s)	• Concrete in both clarifiers is serviceable. • Aeration basin concrete damage preventing proper function of outlet weirs. • Metal weirs should be replaced or resurfaced and reinstalled level. • Secondary clarifiers cannot be taken offline for maintenance. • Mechanical parts of secondary Clarifier #1 need to be replaced. • Mechanical parts of secondary Clarifier #2 need to be resurfaced.
Chlorine Contact Basin	• Adequately sized for use during planning period. • Concrete is cracked, but does not interfere with operation • Auto-sampler freezes in the winter. • Reduction of spike inflow and infiltration (max day flow) would help regulate performance. • Flow paced dosing recently installed
River Outfall	• Discharges below low water level for Pend Oreille River. • No issues noted. • Slope erosion concerns.
Primary Sludge Pump Station	• Pumps outdated, not in use, and cannot be maintained. Confined space to access. • Replacement or rehabilitation should be coordinated with work on the Primary Clarifier.
Anaerobic Digester	• Not in use, solids need to be removed. • All components need to be replaced

Item	Observed Conditions
Activated Sludge Pump Station 1 and 2	• Groundwater leaks into pump room. Sump pump often plugs and fails. Basement walls require sealing. • Numerous valves do not work well, requiring repairs or replacement. • Poor ventilation in pump room, must be upgraded to meet Class 1, Division 1 requirements • Sump pump system requires upgrades for capacity. • No SCADA monitoring system. • No backup power, could affect rest of WWTF. • The restroom in Pump Station #2 has rotting/moldy wall, deteriorated flooring and requires a complete remodel. • Confined space entry measures are required for safe ingress/egress and extraction. The existing spiral staircase is not acceptable for ingress/egress or extraction and an improved access is needed. • The gas chlorination system will require upgrades if not replaced with UV disinfection equipment.
Aerobic Digester	• Steel building covering the aerobic digester is badly corroded, needs replacement. Walkways are connected to the building and will also require replacement. • The decant pipe system needs to be replaced and the waste line supports need to be replaced. • The digester lift station currently only has one pump and requires an upgrade with two pumps on a rail system. • Concrete basins have not been inspected for leaks.
Belt Filter Press	• Conveyor leaks and drops solids. Spray boxes need to be upgraded to a more effective model. • Polymer room heaters do not work. Main area heaters also need to be replaced. • Spray nozzles plug. • The pressure bladder tank requires replacement. • Lift station pumps and rails are due for replacement. • Polymer system and control panel upgraded in 2021. • The sludge thickener system is now longer functional and requires repair and upgrade. • An external domestic water feed line is needed, with air gap skid system for backflow prevention. • An overhead traveling bridge hoist is needed to lift and replace heavy equipment items. • The floor drains need to be repaired. • The wall around the conveyor system needs to be repaired.
Yard Piping	• No deficiencies reported. • Due for inspection.
Electrical Service	• Facility processes do not currently have backup power during power outages. • Two feeds come from the single power utility and do not provide backup to each other. Combination into a single feed will facilitate backup power provisions.
Lab and Office Operations Building	• The existing lab and office space is undersized and both facilities are included in the same room. A break room is not included, resulting in workers having to eat at their desks and on the same counters where wastewater samples are being handled and where lab work is being performed. • The last improvements to the facility were in the 1980's and ceilings, windows, hallways, finishes, fixtures, furnishings and restroom facilities are in need of significant repairs and upgrades. It is highly likely that there is asbestos in the mastic of the flooring, the roof has leaks that have been repaired multiple times,

Item	Observed Conditions
	there are ceiling tiles that periodically fall down and several electrical and plumbing issues. • There has been a recent risk assessment of the facility with a recommendation that major improvements need to be done for both OSHA and L&I compliance. • The parking accommodations for this facility are insufficient and do not provide protection to the existing and future needed fleet and maintenance vehicles. • As an accredited laboratory, the current facility does not provide the proper separated spaces for the laboratory testing procedures. • The process site utilities severely restrict the expansion and improvement of the existing facility. A new constructed space in a new location would allow for separation of lab and office/breakroom. A new facility would provide health and safety benefits to the operators as well as a more controlled site appropriate for the accredited laboratory. The option of a shop structure integrated into the new facility would provide the needed storage and maintenance space for rolling stock, equipment and vehicles needed to properly run the facility and collection system.

CHAPTER 6 – WWTP IMPROVEMENTS ALTERNATIVES

Several deficiencies were identified in the liquid stream treatment process. The City has six general options to improve unit process:

1. Alternative A - Do Nothing
2. Alternative B - Repair and Upgrade Existing Facilities
3. Alternative C - Restore Primary Processes
4. Alternative D - Convert to a Secondary Treatment Plant
5. Alternative E - Construct a Membrane Biological Reactor Package Plant
6. Alternative F – Land Application
7. Alternative G – Gravity-Settling Package Treatment Plant

These liquid stream improvement alternatives are discussed below. Biosolids management for is discussed in Chapter 7. The cost estimates and alternative evaluations are discussed in Chapter 8.

6.1 Alternative A – Do Nothing

The “do nothing” Alternative A would attempt to maintain WWTP performance with a de minimis effort. In general, City staff would continue to maintain facilities that are currently in operation by repairing and replacing parts and equipment, on an as needed basis, without an overall increase in performance or reliability. Since secondary treatment facilities have sufficient capacity to serve for the planning period if projected growth is not exceeded, this option is viable; however, ongoing permit violations are likely and there is an ever-increasing increases risk of failure due to the lack of redundant secondary systems and the inability to rehabilitate or maintain dilapidated equipment and systems. There are also a number of worker safety concerns that need to be addressed for the welfare of the staff and others working at or visiting the plant, which this alternative would not address.

Although no upfront costs may be incurred with the do nothing alternative, the City is at risk for emergency repairs to the existing facilities to keep them running without the benefit of providing redundancy to the system.

6.2 Alternative B – Repair and Upgrade Existing Facilities

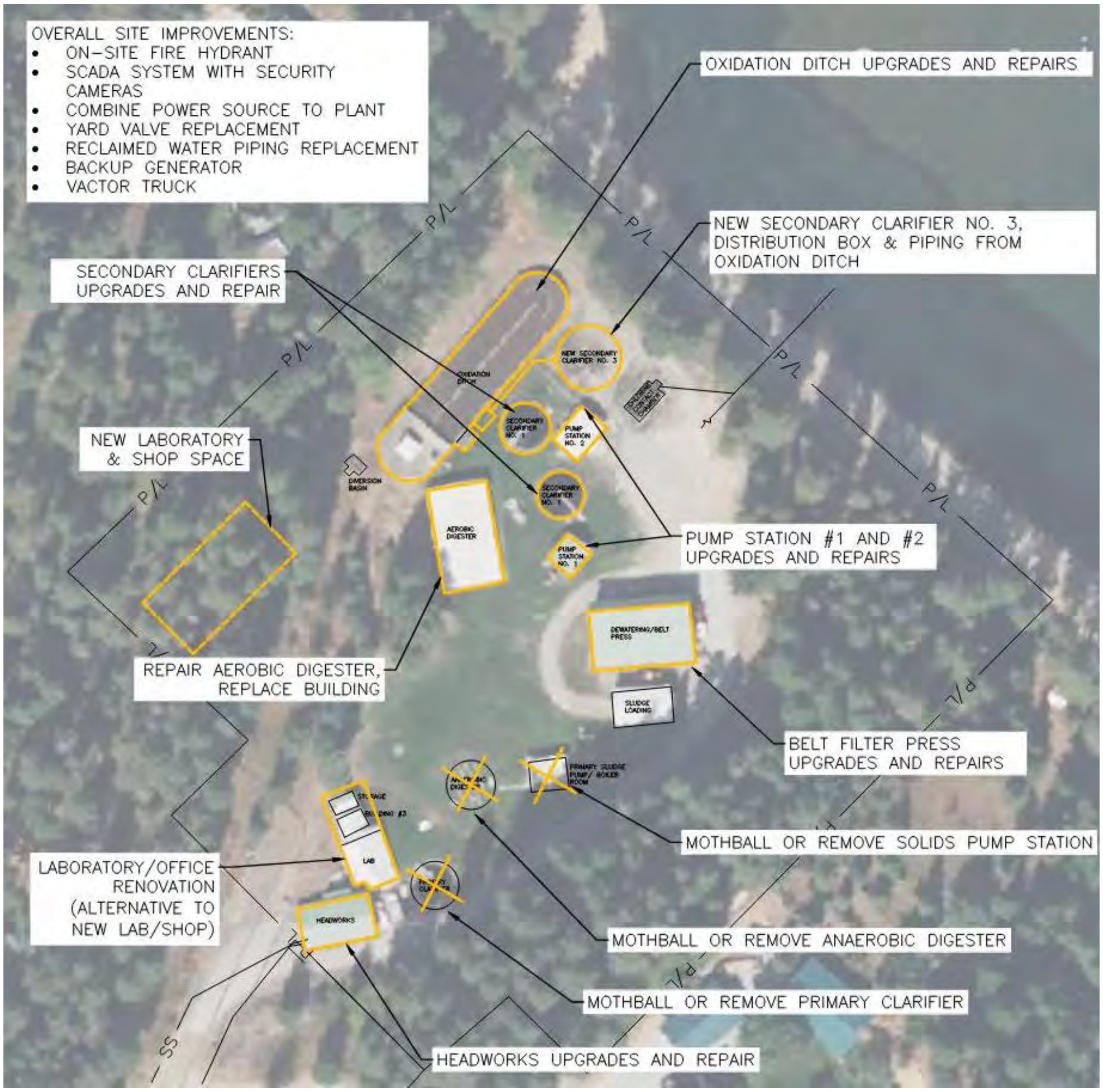
Alternative B would repair and upgrade the existing facilities to improve reliability to meet compliance requirements with the existing processes by restoring the function of the existing processes and adding redundant processes, address operational challenges and address worker safety concerns, see Figure 6-1.

This alternative would make operational upgrades to other unit processes in the facility, including headworks improvements, lab/office renovations, improvements to the existing oxidation ditch, clarifier upgrades, pumphouse upgrades, and make repairs and upgrades to the aerobic digester.

This alternative would meet the wastewater treatment capacity needs for the City of Newport through the end of the 20-year planning period. It would meet the current and anticipated water quality limits through the planning period.

The estimated cost for Alternative B to repair and upgrade the Existing Facilities is estimated at $30.4 M (2023 dollars) including contingency, sales tax, design engineering, construction management engineering, and legal and administrative costs. See Appendix F for a detailed cost estimate for Alternative B. See Appendix G for the exhibit for Alternative B.

Figure 6-1: Alternative B to Repair and Upgrade Existing Facility

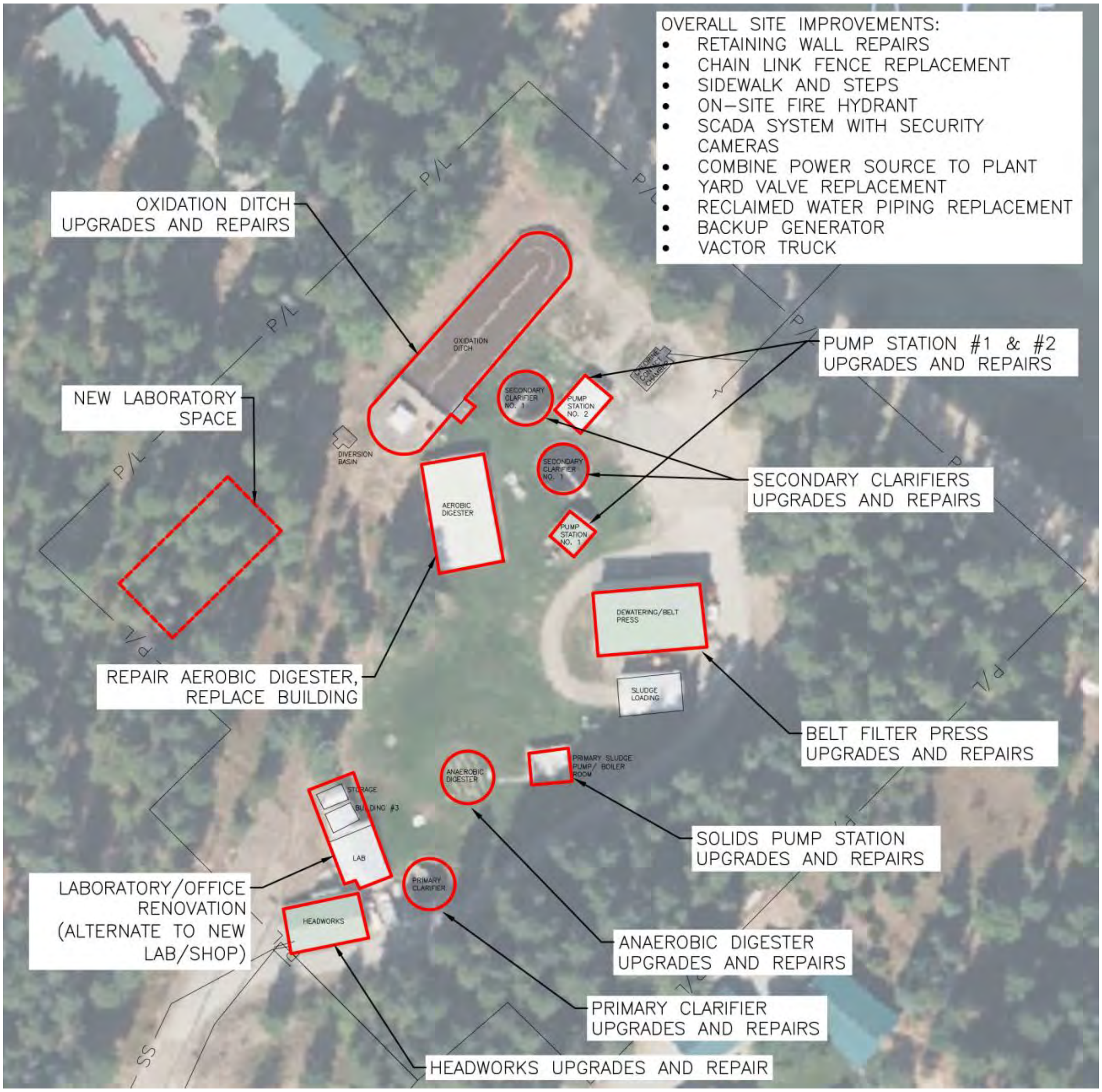


6.3 Alternative C – Restore Primary Processes

Alternative C would repair the primary clarifier, anaerobic digester and aerobic digester, thereby recovering expected capacity and performance, see Figure 6-2. Additionally, the lighter loading to the secondary system may allow the overall capacity to be increased if studies confirm performance. However, repairing the WWTP's components will not provide any redundancy, which maintains the increasing risk of failure. Additionally, at the end of the planning period, the single primary clarifier will not have ideal performance due to the shallow depth and being slightly overloaded; however, the extra solids flowing to the secondary system can easily be managed by the oxidation ditch. At this time, a second primary clarifier is not recommended. This alternative would also make operational upgrades to other unit processes in the facility, including headworks improvements, lab/office renovations, belt filter press upgrades, improvements to the existing oxidation ditch, clarifier upgrades, pumphouse upgrades, replace chlorine disinfection with UV disinfection, and make repairs and upgrades to the aerobic digester.

The estimated cost for Alternative C to restore primary processes and repair and upgrade the existing WWTP components is estimated at $38.5 M (2023 dollars) including contingency, sales tax, design engineering, construction management engineering, and legal and administrative costs. See Appendix F for a detailed cost estimate for Alternative C. See Appendix G for the exhibit for Alternative C.

Figure 6-2: Alternative C to Restore Primary Processes

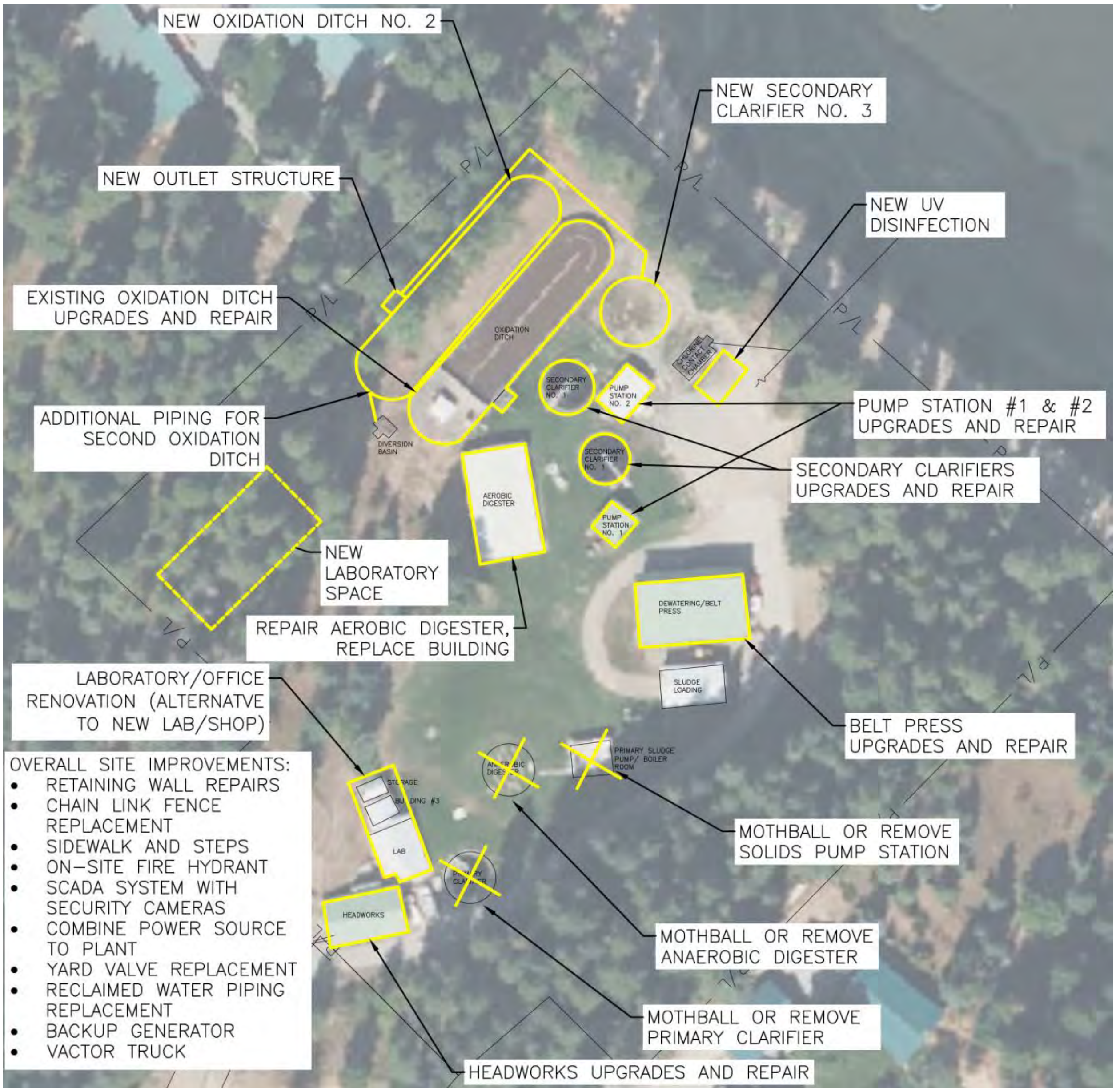


6.4 Alternative D – Convert to Secondary Treatment Only

Alternative D would construct a second oxidation ditch and a third clarifier and remove the primary treatment system from service, see Figure 6-3. The two oxidation ditches and three clarifiers would backup each other, thereby providing much needed redundancy. Once the second ditch was on-line, then the first ditch could be removed from service and repaired. This alternative would also make operational upgrades to other unit processes in the facility, including headworks improvements, lab/office renovations, belt filter press upgrades, improvements to the existing oxidation ditch, clarifier upgrades, pumphouse upgrades, replace chlorine disinfection with UV disinfection, and make repairs and upgrades to the aerobic digester.

The estimated cost for Alternative D to construct a second oxidation ditch and a third secondary clarifier and make the additional improvements at the existing WWTP is estimated at $45.5 M (2023 dollars) including contingency, sales tax, design engineering, construction management engineering, and legal and administrative costs. See Appendix F for a detailed cost estimate for Alternative D. See Appendix G for the exhibit for Alternative D.

Figure 6-3: Alternative D to Convert to Secondary Treatment



6.5 Alternative E – New Membrane Biological Reactor Package Plant

Membrane biological reactor (MBR) package treatment plants have recently become available in the wastewater treatment industry. These stainless steel tank treatment plants use membranes to separate the biological consortium from the treated water which allow a dense biomass to work in a smaller footprint to provide treatment. Alternative E would provide a package treatment plant to fully treat the wastewater, as outlined in Figure 6-4 and Figure 6-5. A new influent lift station would intercept the existing flow and pump into a new structure where the MBR treatment components would be housed. The treated effluent would be piped to the existing outfall pipe that discharges to the Pend Oreille River. Many of the existing WWTP components could be mothballed or repaired according to the needs of the City. A process flow schematic of a potential package treatment is shown in Figure 6-5. See Appendix H for calculations regarding sizing the MBR plant and estimated life cycle costs for maintenance of the equipment from a manufacturer.

The estimated cost for the MBR plant, equalization storage, new lift station, and new pipe to connect to the existing discharge pipe is estimated at $37.0 (2023 dollars) including contingency, sales tax, design engineering, construction management engineering, and legal and administrative costs. The cost above is for present cost comparison and does not include escalations for inflation which could occur over the years of implementation. See Appendix F for a detailed cost estimate for Alternative E. See Appendix G for the exhibits for Alternative E.

Figure 6-4: Alternative E Overview with MBR Plant and New Pipe to Discharge

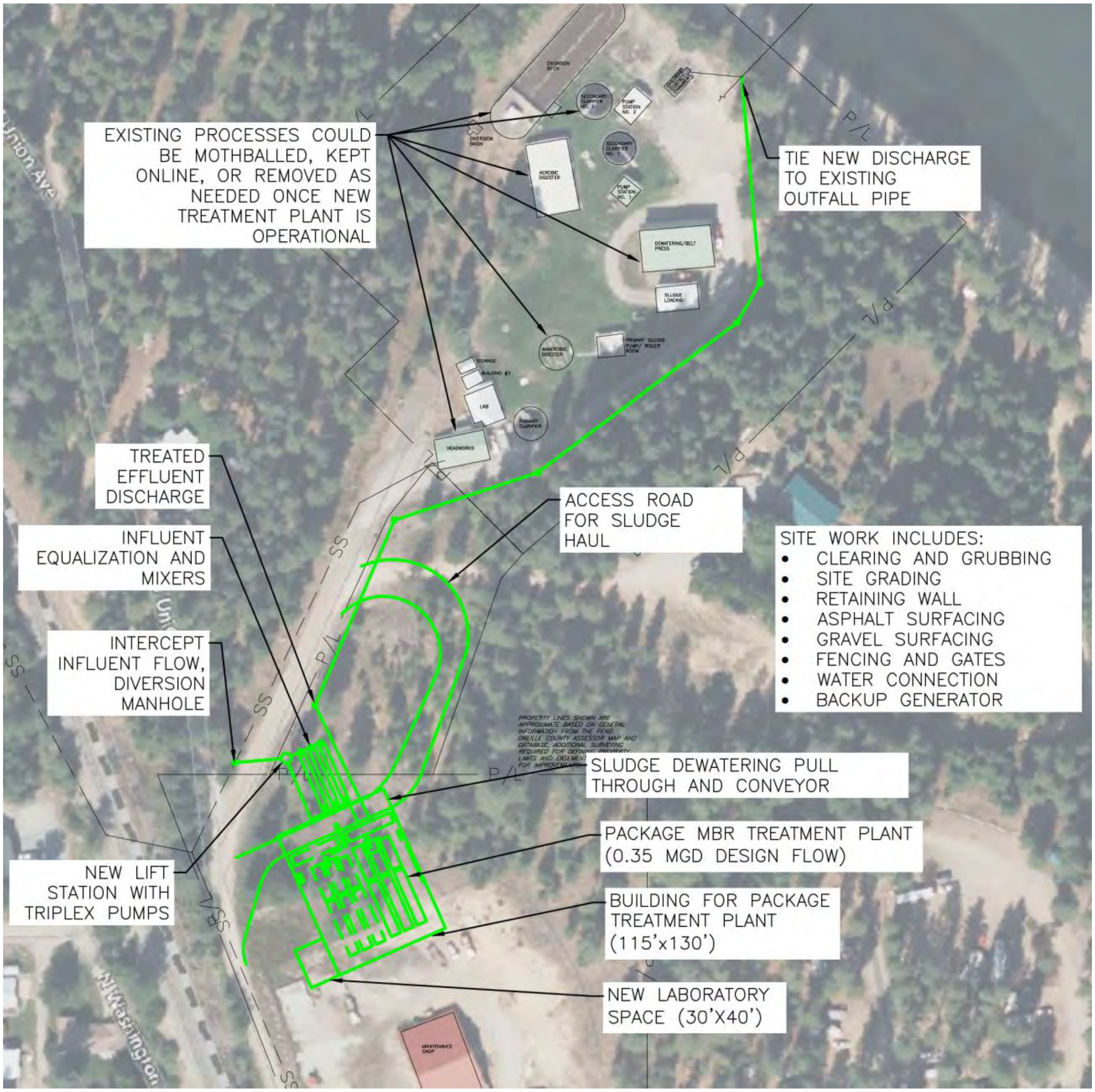
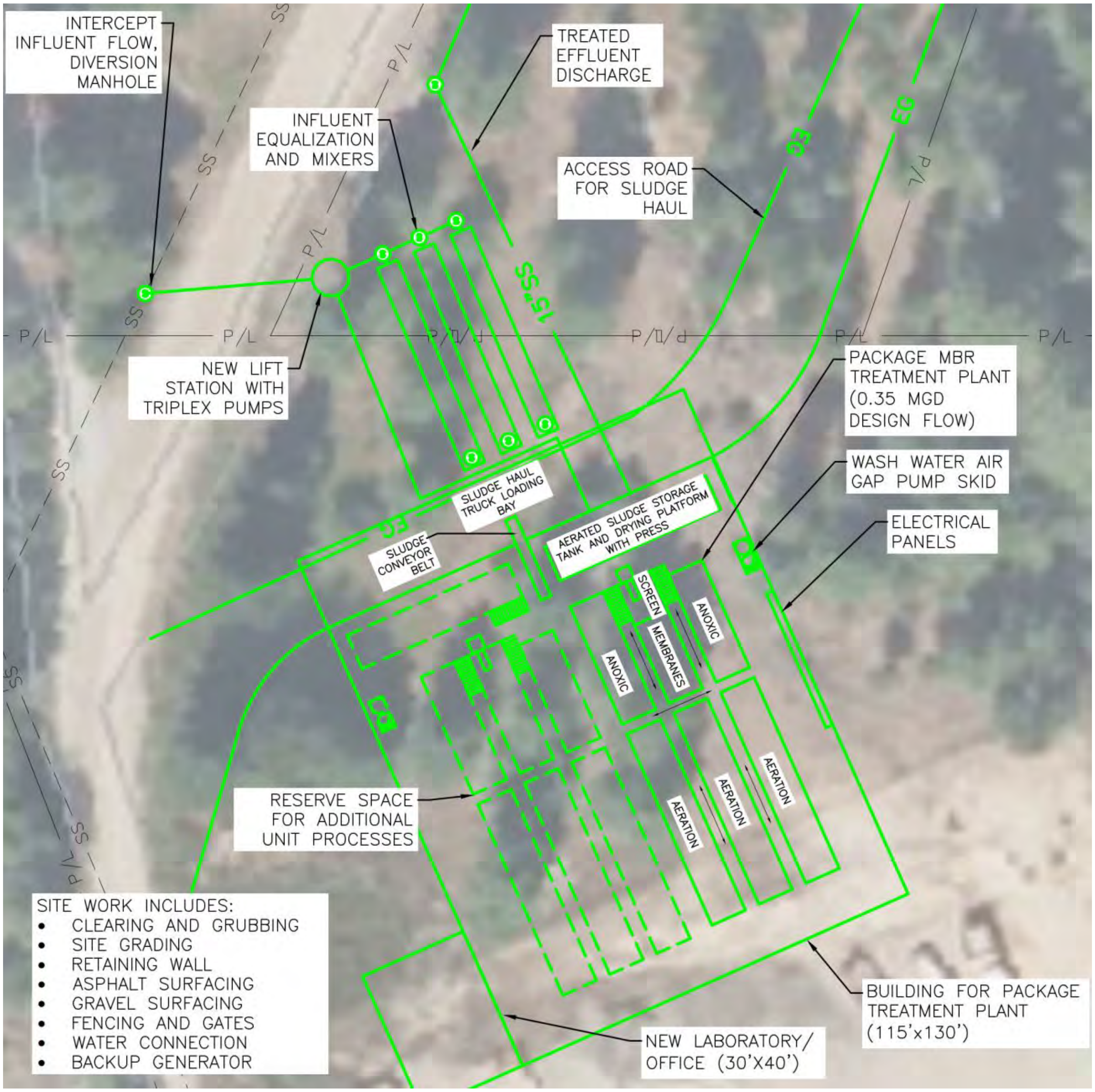


Figure 6-5: Alternative E Proposed MBR Plant at City Shop Site



A flow schematic of the proposed package treatment plant is included in Figure 6-6 and Figure 6-7.

Figure 6-6: Alternative E Flow Schematic

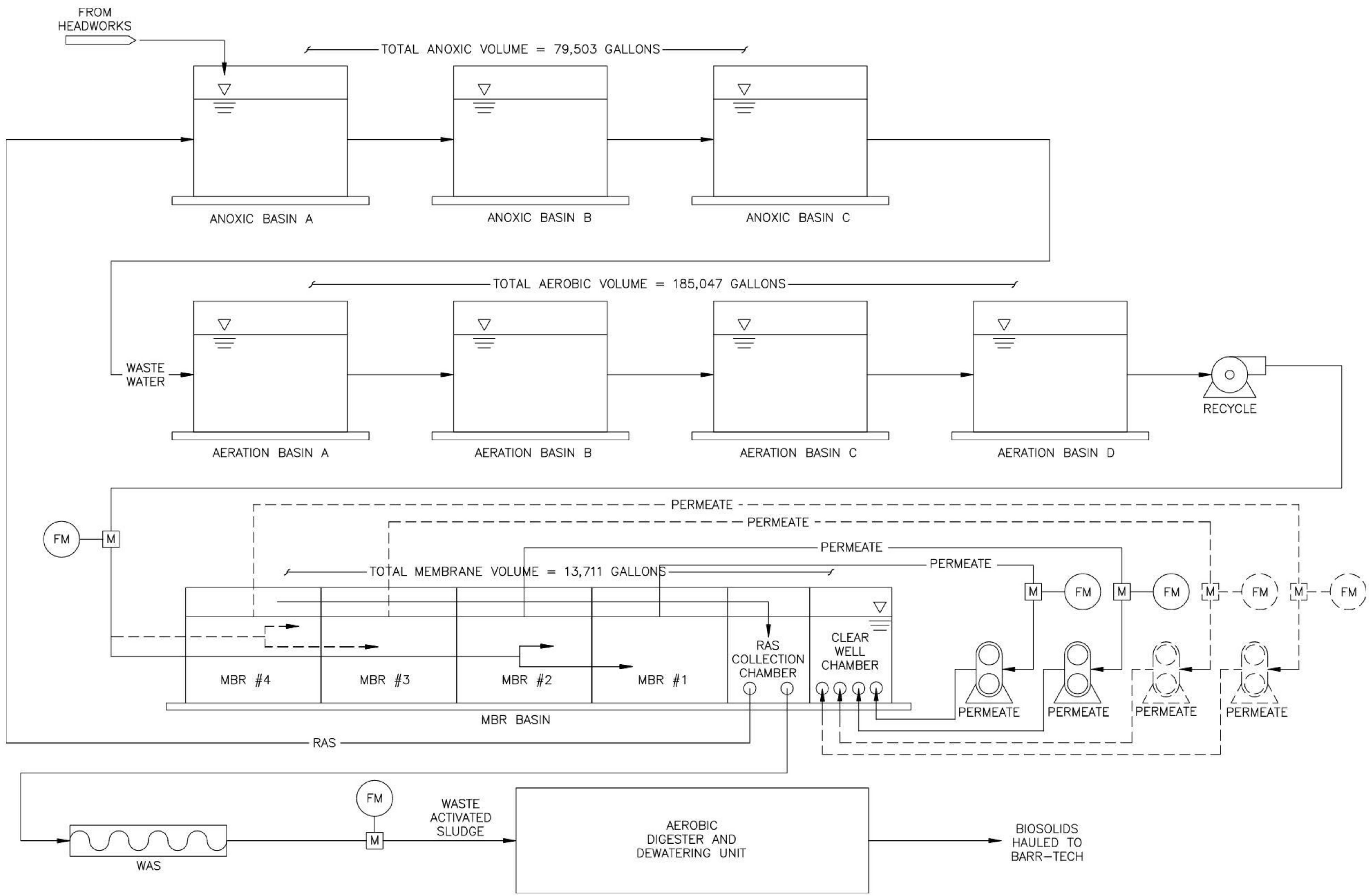
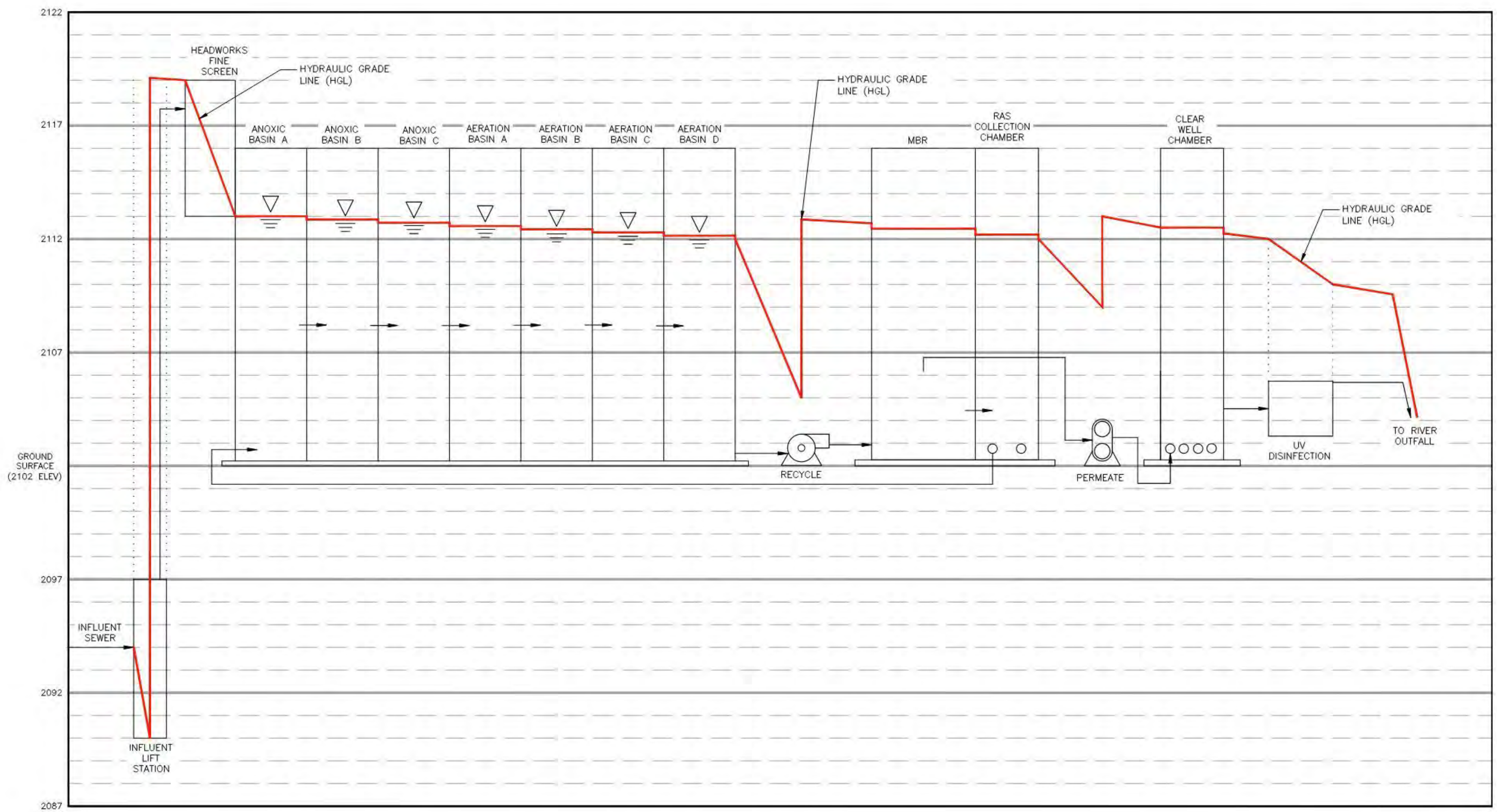


Figure 6-7: Alternative E Process Hydraulic Grade Line



6.6 Alternative F – Land Application

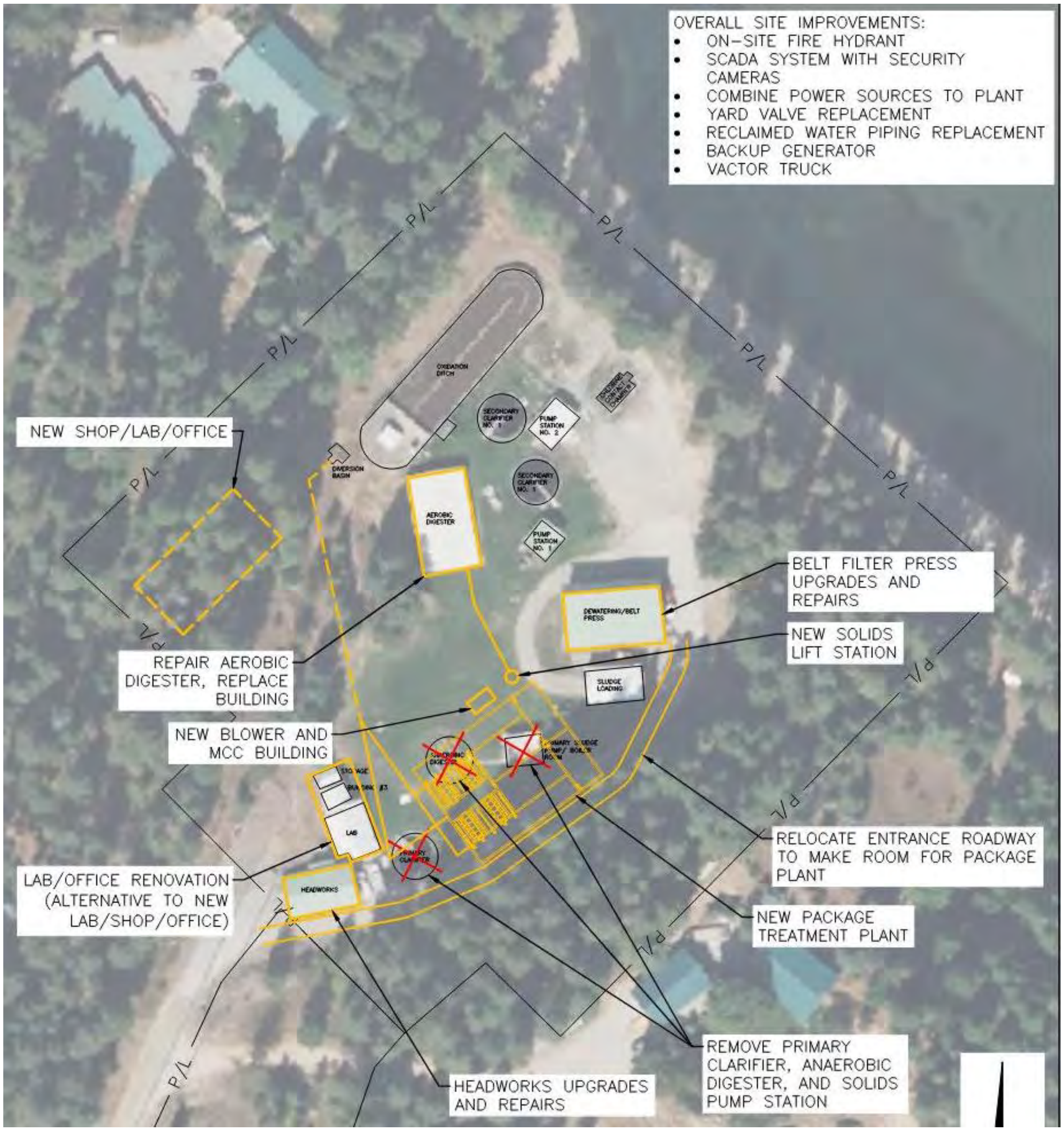
Disposing of treated effluent by land application is a potential option should discharge to the Pend Oreille River become untenable. For example, should compliance with the TMDL imposed heat load limit require effluent cooling the cost to mechanically cool effluent could be more than the cost to land apply the effluent. To dispose of effluent during the summer when heat loads could limit discharge, about 80 acres would be needed and 4.5 million gallons of storage. To dispose of the effluent year-round about 230 acres would be needed and 90 million gallons of storage. Land application facilities would be needed in addition to the treatment improvements discussed above. At this time, the cost to implement land application is prohibitive when continued river discharge is likely; therefore, the land application alternative was eliminated from further consideration.

6.7 Alternative G – Gravity-Settling Package Treatment Plant

An additional alternative that has been evaluated is the replacement of key treatment processes with a gravity-settling package treatment plant, which would provide water quality results comparable to the existing treatment processes. The Aero-Mod package treatment plant was used as the basis for this evaluation, with sizing, configuration and costs based on proposal by the manufacturer. Other manufacturers could also provide a package plant which would produce similar results to the Aero-Mod system and with a similar footprint based on the capacity needed. This alternative was evaluated to determine if it may be more cost effective than upgrades to the existing processes, as outlined previously in Alternative B. In this alternative, the Aero-Mod core treatment processes (aeration stages and clarification) would replace the existing oxidation ditch and the existing clarifiers and pumping facilities, as shown in Figure 6-8 below. The headworks, aerobic digester, belt filter press and chlorine contact chamber are considered adequate, with some maintenance upgrades, to be retained through the planning period to save cost. The Aero-Mod process package would be inserted in the hydraulic flow path downstream from the existing headworks. Although the existing oxidation ditch would no longer be used for treatment of the wastewater, it was evaluated for equalization storage, with a pump station incorporated to return stored excess flows through the new treatment processes. This alternative is technically viable, but requires removal of existing unused primary treatment facilities and the existing topographical characteristics of the site would require a number of site improvements to be able to fit the package treatment plant within the available area in the appropriate location for operation. In order to protect piping and structures critical to the operation of the plant, insertion of the package treatment plant would require deep excavation into the hillside and associated shoring and retaining structures to relocate the existing access roads and existing essential utilities. The operation of the sludge dewatering belt filter press requires the storage of sludge in the existing aerobic digester. Plumbing improvements have been incorporated to accommodate the transfer of sludge from the package treatment plant to the aerobic digester. These improvements are outlined in detail in the cost estimates provided in Appendix F discussed in the next section.

The estimated cost for the incorporation of a package treatment plant into the existing screening, solids dewatering and disinfection processes is estimated at $33.1M (2023 dollars) including contingency, sales tax, design engineering, construction management engineering, and legal and administrative costs. The cost above is for present cost comparison and does not include escalations for inflation which could occur over the years of implementation. See Appendix F for a detailed cost estimate for Alternative G. See Appendix G for the exhibits for Alternative G. See Appendix I for the design criteria calculations and sizing for the package treatment plant from the supplier of the Aero-Mod package plant used as a basis for this evaluation.

Figure 6-8: Alternative G Gravity-Settling Package Treatment Plant



CHAPTER 7 BIOSOLIDS MANAGEMENT

At the end of the planning period, the City will have to dispose of about 950 pounds of biological solids per day wasted from the treatment process (assuming a yield coefficient ~0.68). The City currently stabilizes waste activated biosolids from the secondary system in the aerobic digester. Dewatered biosolids are hauled offsite to Barr-Tech for composting and beneficially used, which is the City's preferred disposal method. Barr-Tech accepts unclassified biosolids; therefore, the City does not test their biosolids for classification to save on sampling and laboratory cost. Stabilized biosolids (unclassified) are dewatered using the belt filter press prior to hauling off site for beneficial use. As noted above, the primary clarifier and anaerobic digester are offline; therefore, only secondary biosolids are currently produced and managed.

The liquid stream alternatives discussed above require biosolids management. The overall strategy to manage biosolids for each alternative is the same; waste biosolids to an aerobic holding tank (aerobic digester) until they can be dewatered and hauled off-site to Barr-Tech for compositing and beneficial use. Biosolids management (aeration, dewatering, disposal) is similar for each alternative and each alternative will produce a similar product; therefore, there is little distinction between the alternatives (pros and cons).

Necessary improvements to manage biosolids are discussed in Chapter 6 and the cost to manage biosolids are included in the alternative cost estimates.

CHAPTER 8 – ALTERNATIVE COMPARISON, SELECTION AND CAPITAL IMPROVEMENT PLAN

Alternatives that will allow the City to continue providing wastewater treatment services are evaluated below. Specific biosolids management options discussed above are part of respective liquid stream alternatives.

8.1 Summary of Alternatives

The management and improvement alternatives, advantages and disadvantages, viability, capital costs and operation/maintenance costs are summarized in Table 8-1, except for Alternative F which was eliminated for not being viable for implementation without other treatment processes taking place and because of exceptionally high projected costs and lack of available land for implementation.

Table 8-1: Wastewater Treatment Alternatives Summary

Alternative #	Alternative Description	Advantages	Disadvantages	Viable? Yes/No	Present Capital Cost	Annual O&M Cost	20-Year O&M Present Worth
Alternative A	Do Nothing	• May defer the costs of improvements for a period of years.	• Does not provide redundancy in several key processes, creating the risk of violating discharge requirements and incurring fines or lawsuits. Does not repair the ditch effluent splitter box, thereby perpetuating the difficulty splitting flow evenly to the clarifiers and causing solids flow, resulting in potential disinfection violations. Risks the inability to rehabilitate and maintain dilapidated equipment and systems. • Does not address worker safety concerns created by facilities and systems that do not comply with current codes and standards. • Inoperable reclaimed water system means high demand for potable water from City water system.	Yes, capacity met, but periodic violations likely	$ 0	$536,616.00	$11,244,534.59
Alternative B	Repair and Upgrade Existing Facilities	• Would reduce risk to the City by protecting workers and improving the ability to meet safety and water quality compliance requirements. • Would reduce potable water use by repairing reclaimed water system. • Would provide redundancy in secondary clarifier processes, allowing for maintenance of each of these processes while continuing to operate the facility. • Current operator certifications (Group II) are sufficient for this alternative.	• This alternative perpetuates and exacerbates space constraints on the existing site which impact the ease and cost of maintenance and operation of the facility. • At the end of the planning period, existing structures and equipment will have further deteriorated and it may no longer be feasible to continue using these structures and equipment.	Yes, capacity met, but periodic violations possible	$ 30,411,000	$488,348.00	$10,233,101.47
Alternative C	Restore Primary Processes and Implement Repairs and Upgrades	• Would reduce risk to the City by protecting workers and improving the ability to meet safety and water quality compliance requirements, due to the repairs and upgrades to existing operating processes. • Would reduce potable water use by repairing reclaimed water system.	• Even though primary treatment functions would be restored, the overall operation would not experience a net benefit in redundancy to treatment processes. • Adds additional O&M costs associated with primary treatment processes placed back in operation.	No	$ 38,519,000	$590,224.00	$12,367,864.89
Alternative D	Fully Convert to Secondary Treatment, and Implement Extensive Repairs and Upgrades	• Would reduce risk to the City by protecting workers and improving the ability to meet compliance requirements, due to the repairs and upgrades to existing operating processes. • Would provide redundancy in key secondary treatment processes, allowing for maintenance of each of these processes while continuing to operate the facility. • Would reduce potable water use by repairing reclaimed water system. • Current operator certifications (Group II) are sufficient for this alternative.	• At the end of the planning period, existing structures and equipment will have further deteriorated and it may now longer be feasible to continue using these structures and equipment. • This alternative perpetuates and exacerbates space constraints on the existing site which impact the ease and cost of maintenance and operation of the facility. • Does not provide the best available technology to maximize removal of potential pollutants of concern, compared to Alternative E.	Yes	$ 45,512,000	$595,992.00	$12,488,730.60
Alternative E	New Membrane Biological Reactor Package Treatment Plant	• Would provide a facility that would meet current worker safety codes and provide the best level of treatment. • Has the lowest staff labor requirement of all the alternatives as a result of advanced automated functions, thereby allowing staff to perform other functions. • Produces the highest quality effluent and meets redundancy requirements. • Provides the ability to expand treatment capacity in the future at a lower cost than the other alternatives. • Would reduce potable water use by repairing reclaimed water system.	• Requires a higher level of operator certification (Group III). • Requires advanced instrumentation. • Has a high chemical feed demand. • Adds electrical demand for new treatment processes.	Yes	$ 36,986,000	$708,796.00	$14,852,485.10

Alternative #	**Alternative Description**	• **Advantages**	• **Disadvantages**	**Viable? Yes/No**	**Present Capital Cost**	**Annual O&M Cost**	**20-Year O&M Present Worth**
Alternative F	Land Treatment	• If the discharge to the Pend Oreille River were to become exceptionally challenging and expensive, this could be a potential option. • If heat loads were to be imposed on the river discharge, this land application option would mitigate that impact by removing all or a portion of the flows from the river.	• Prohibitively expensive and challenging to acquire land for storage and land application • Would require Class A treatment as associated costs and permitting	No	$75M Plus	N/A	N/A
Alternative G	Gravity Settling Package Treatment Plant	• Would reduce risk to the City by protecting workers and improving the ability to meet safety and water quality compliance requirements. • Would reduce potable water use by repairing reclaimed water system. • Current operator certifications (Group II) are sufficient for this alternative. • Would provide fully redundant treatment processes if existing facilities were left intact.	• More pumping required, additional power cost • Would have to pump excess flows back from existing oxidation ditch and clarifiers if excess flows were stored there. • This alternative perpetuates and exacerbates space constraints on the existing site which impact the ease and cost of maintenance and operation of the facility.	Yes	$33,144,000	$544,156.00	$11,402,532

The Operation and Maintenance (O&M) Annual Costs shown in the alternative comparison table were evaluated for each capital improvement alternative. O&M Costs are typically annual labor, equipment, chemical, utility, and other costs associated for operating the wastewater treatment plant. Labor hour estimates were estimated for each unit process at the Newport WWTP based on feedback from City staff, generally assuming a $53 per hour aggregate cost for WWTP staff and operators. Equipment costs were estimated for each unit process, accounting for the service life and potential replacement cost of critical equipment. Chemical costs for the Newport WWTP generally include the chlorine for the disinfection contact chamber and polymer cost for the sludge dewatering and belt filter press. Utility costs include water and electricity costs for the overall plant, based on existing utility billing information. Other costs associated with O&M for the WWTP were identified by historical budget information provided by the City of Newport. The City of Newport staff and operators reviewed the O&M costs presented for each alternative and provided feedback to help these numbers reflect real costs. By comparison, Alternative A (Existing) O&M cost estimates correlated within 95 percent of the historical WWTP operating costs documented in the City of Newport's financial system. For Alternative E, O&M costs also include the costs of membrane replacement and chemical costs associated with that process. The detailed O&M cost analysis for each alternative can be found in Appendix F.

The City had the following primary ranking criteria:

- Ability to reliably meet current permit requirements
- Ability to reliably and cost effectively meet future permit requirements
- Ability to meet worker and public safety standards and regulations
- Cost, both Capital Cost and Annual Operation and Maintenance Costs

Alternative A was eliminated due to the ever-increasing risk of permit violations as the plant service population grew and existing equipment continued to age. Alternative A also does not address key worker safety or key operational and reliability challenges.

Alternative B would repair and upgrade the existing facilities to address worker safety concerns, restore function, address operational challenges and improve reliability to meet compliance requirements with the existing processes. This alternative addresses a less comprehensive list of repairs and upgrades than Alternative D, which was intended to address all issues to make Alternative D most comparable to Alternative E, both considered full restoration of the plant components. Alternative B was also considered for partial or phased implementation to meet the highest priority needs, with the ability to make additional improvements in the future if current funding does not allow full implementation.

Alternative C was not considered a viable alternative because even with repairing and updating the current facilities, the plant would not increase overall reliability due to a lack of redundancy in the primary and secondary treatment systems and the inability to take any unit process offline for heavy maintenance and repair.

Alternative D and Alternative E were considered in depth with City staff with the goal of providing a full rehabilitation of the existing plant (Alternative D) to a level that would be most comparable to the new facility (Alternative E). Alternative D has a capital cost approximately $8.5M more than Alternative E, but 20-year present worth O&M cost approximately $2M less than Alternative E. Alternative E has advantages over Alternative D and Alternative E has the least disadvantages of the two alternatives, as discussed in the table above. The key attractive features of Alternative E included the advanced automation functions, a more certain barrier (physical) to separate the biology used for treatment thereby eliminating the need to select biology that settles well, the ability to provide water quality suitable for irrigation reuse with enhanced disinfection, and the ease and relative low cost of future expansion. After full consideration of these two alternatives, neither Alternative D nor Alternative E was considered economically viable due to high cost and unreasonable impacts on user rates.

Alternative F, the effluent land application alternative was eliminated from further consideration due to the exceptionally high cost.

Alternative G would replace the current treatment processes (oxidation, ditch, clarifiers and pump houses) with a gravity-settling package treatment plant, with the objective of avoiding the costs of upgrading or expanding the existing processes eliminated by the package treatment plant. The headworks, belt filter press and chlorine contact chamber would be retained and the package treatment plant would be incorporated in the hydraulic flow path downstream of the headworks and upstream of the solids handling facilities. This alternative does not provide any advantages over the existing processes and incorporation of a gravity-settling package treatment plant would require significant improvements to be able to configure the package plant within the site. This alternative is not the least cost alternative and was not selected for implementation.

After consideration by the City of Newport, Alternative B, the least cost alternative, was selected for implementation.

Staffing levels for the preferred alternative will be the same as the current wastewater treatment facilities.

A summary of the improvements included in Alternative B is included in **Table 8-2** below:

Table 8-2: Alternative B Summary of Improvements

Improvement Area	Improvements
Overall Site Improvements	• Onsite domestic fire hydrant • SCADA system with security cameras • Combine power sources into one feed • Yard valve replacement throughout plant • Reclaimed water system replacement, including piping and filtration • Backup generator • Vactor truck
Headworks	• New influent fine screen • Electrical improvements of independent screen operation • Replace/support grit chamber piping
Belt Filter Press	• Replace heaters • Replace 1100-gallon pressure tank • Upgrade shower boxes on gravity and press section • Domestic water feed line • Air gap skid system • Upgrade lift station with new pumps • Sludge trailer
Existing Oxidation Ditch	• Upgrade drive, gear box, paddle system • Replace building over drive system • New control panel with VFD for drive system w/DO control • Construct backup aeration system on opposite end of basin • Replace grating on influent distribution box • Repair outflow distribution box
Secondary Clarifiers	• Clarifier #1 equipment replacement • Clarifier #1 concrete inspection and repairs • Clarifier #2 equipment replacement • Clarifier #2 concrete inspection and repairs • Reshape Clarifier #2 trough and level weir
New Clarifier #3	• Construct new clarifier with appurtenant equipment, deeper than existing clarifiers for improved solids removal efficiency. • Upgrade piping and valving for new clarifier operation • Upgrade chlorination system • Electrical and instrumentation
Pumphouse #1	• Replace piping and valving • Ventilation with monitoring for basement area • Excavate and seal basement walls with drainage • New sump pump system (WAS line and flows from PH #2) • Replace conduits with water issues • Overhead hoist in basement • Construct basement extension for entrance with overhead building extension • Underdrain with outlet pipe • Yard piping modifications • Electrical and instrumentation
Pumphouse #2	• Replace 8" valves RAS-AUX-WAS • Ventilation with monitoring for basement area • Excavate and seal basement walls with drainage • New sump pump system (WAS line and flows from PH #2) • Replace conduits with water issues

Improvement Area	Improvements
	• Overhead hoist in basement • Construct basement extension for entrance with overhead building extension • Underdrain with outlet pipe • Yard piping modifications • Electrical and instrumentation
Aerobic Digester	• Demolish existing structure • Construct new building with overhead hoist • Building lighting • Digester lift station upgrade for dual pumps on rails
New Shop/Office/Lab	• Clearing and grubbing • Site excavation • Steel shop/lab/office building, 50'X100' • Finish office/lab space, 30'X50' • Extend 8" water line to shop • Fire hydrant • Extend sewer line to shop/office/lab • Crushed surfacing top course for driveway • Hot mix asphalt for driveway

8.2 Capital Improvement Plan

It is estimated that the selected Alternative B could be implemented in the next 5-years as funding is secured:

- 2023 to 2025: Secure funding for permitting, environmental review, and design engineering (up to $2.3 M in 2023 dollars).
- 2025 to 2028: Secure funding for construction (up to $28.1 in 2023 dollars)

This selected Alternative B has the flexibility for phased implementation, which would allow for improvements to be prioritized by objectives and completed in separate phases based on funding availability and an evaluation of rate impacts and the ability of the wastewater customers to bear the costs of the improvements. The proposed phasing plan, developed through discussions with Newport administrative and WWTP staff, is as follows:

Preliminary Design Phase, 2024

This preliminary design phase has the following objectives:

1. Define the specifics of the improvements to be implemented,
2. Evaluate potential cost savings,
3. Confirm the phasing of improvements,
4. Refine the costs at an appropriate level to make funding requests.

Phase 1 Improvements, Design 2024/2025, Construct 2026

This phase will include the following elements:

1. Complete oxidation ditch upgrades,
2. Construct new Secondary Clarifier #3,

3. Complete Pumphouse #2 Upgrades,
4. Initiate purchasing for backup generator,

Phase 1 focuses on water quality compliance with the facility's discharge permit. It addresses the top priorities for improved redundancy and effectiveness in the treatment process. It is this phase that assures the treatment facility has the capacity to address the growth that may occur in the 20-year planning period. Subsequent phases address the maintenance issues typical of a treatment facility as it ages. Phase 1 also initiates the ability to provide power to the entire plant in the event of a power utility failure, which is absolutely essential to reliable treatment during emergency events requiring an alternate power source. The procurement of the backup generator occurs in this phase, while the final installation occurs in Phase 2.

Phase 2 Improvements , Design 2025/2026, Construct 2027/2028
This phase will include the following elements:

1. Overall site: backup generator/combine power sources, water line/hydrant
2. Headworks improvements,
3. Clarifier #1 and #2 mechanical equipment upgrades,
4. Pumphouse #1 upgrades,

Phase 2 finalizes the installation of emergency backup power, ensuring that the facility will operate if utility power is interrupted. Phase 2 also prioritizes maintenance issues that assure ongoing operational functionality and worker safety.

Phase 3 Improvements, Design 2027/2028, Construct 2029
This phase will include the following elements:

1. Overall site: Vactor truck purchase, yard valve replacement, SCADA system implementation
2. Aerobic digester and building improvements,
3. Belt filter press upgrades,
4. New shop/office/lab building

Phase 3 provides for maintenance upgrades of the existing facility and provides operational monitoring and control features that assure a rapid response by operations staff. It also addresses issues critical for worker safety and welfare and provides a facility for protection and maintenance of the vehicles and mobile equipment essential to facility operations.

A rate study (under separate cover) has been completed to estimate the rate impacts of the implementation of the treatment plant preferred alternative, in addition to collection system improvements outlined in the General Sewer Plan. Sample worksheets of this rate analysis are included in **Appendix J**. This rate study will be updated as funding options are finalized and the study will be used to guide City Council actions on the implementation of rate adjustments for the improvement projects.

8.3 Compliance with SEPA

In order to qualify the identified collection system improvements for WSDOE funding, the State Environmental Policy Act (SEPA) will be followed. This will include preparation of a SEPA Checklist, public notice, State Environmental Review Process (SERP) Checklist, Federal Cross Cutter Checklist, SERP Cover Sheet, Biological Assessment, and Cultural Resources Survey.

Appendix K includes the SEPA Checklist and Determination of Non-Significance for this planning document.

REFERENCES

City of Newport; Comprehensive Plan 2021

HDR; *Treatment Technology Review and Assessment for Association of Washington Cities*; December 2013.

City of Newport Wastewater Treatment Plant Facilities Improvements Plan Engineering Report, 2003

Metcalf & Eddy | AECOM; *Wastewater Engineering: Treatment and Resource Recovery, Fifth Edition*; 2013.

NPDES Waste Discharge Permit No. WA-002232-2

State of Washington Department of Ecology; *Criteria for Sewage Works Design (Orange Book)*; August 2008.