Section 7: Treatment Facilities Evaluation

7.1 WWTP Treatment Facilities

The WWTP is one of two wastewater treatment plants owned and operated by the City to treat wastewater collected within its sewer service area. Located in West Bremerton along State Route 3, the facility provides year-round secondary wastewater treatment and discharges the treated effluent through an outfall into Sinclair Inlet.

The secondary treatment process was originally designed with a biofilter followed by an activated sludge system. However, the biofilter was placed offline due to the perceived lack of benefit and significant odor concerns. The City has no plans to return it to service without further evaluation at this time. Currently, wastewater undergoes screening, grit removal, and primary clarification, followed by processing in aeration basins and secondary clarifiers. The secondary effluent is then disinfected with sodium hypochlorite, dechlorinated, and discharged into Sinclair Inlet.

Primary sludge and thickened wasted activated sludge (WAS) are treated through anaerobic digestion and dewatered using centrifuges to produce Class B biosolids. The biosolids are transported and land applied on a City-owned silvicultural site.

7.1.1 References

All process components and system configurations referenced herein were gathered from staff interviews during the BRVA workshops and document review. The following documents were included in this review:

- 1985 Schedule A & B Wastewater Treatment Plant drawings
- 1986 Designer's Operations Memorandum
- 1995 Wastewater Treatment Plant Odor Control Project drawings
- 2005 Wastewater Comprehensive Plan Update
- 2008 Wet Weather Facilities Upgrade Project Drawings
- 2009 Major Facilities Upgrade
- 2010 Mixing Zone Study Update
- 2014 Wastewater Comprehensive Plan Update

7.1.2 Liquid Stream Processes

7.1.2.1 Headworks Screens

Influent at the City's WWTP first enters the headworks system equipped with three, 15 mm mechanical bar screens manufactured by Parkson. These screens operate automatically, responding to influent flow signals. Screen #3, installed in 2008 as the middle screen, is always open to assist with flow management, while screens #1 and #2, installed in 1995, transition to high flow mode when influent flow exceeds 20 MGD to handle the increased influent. The

design criteria for the headworks screens are provided in Table 7-1 **Error! Reference source not found.Error! Reference source not found.**below:

Table 7-1: Design Criteria for Headworks Screens

Headworks Screens #1 and #2	
Quantity	2
Type	Mechanical Bar Screens
Screen Channel Width	3 ft
Screen Opening Size	15 mm
Capacity, each	15 MGD
Motor	2 hp
Year Installed	1995
Screen #3	
Quantity	1
Screen Channel Width	4 ft
Opening Size	15 mm
Capacity	18 MGD
Motor	2 hp
Year Installed	2008

All three chambers have sluice gates to isolate them horizontally and vertically. City staff have noted that biosolids are generally clean, however, there was a significant amount of debris that was disposed of during a recent digester cleaning, indicative of moderate to below-average screening performance. The effluent from the three screens is collected in a common effluent channel, which then divides into two open channels that direct flow into two grit chambers.

7.1.2.2 Grit Removal

Raw wastewater from the headworks is conveyed into the aerated grit removal chambers. The operational set up includes two grit tanks and two Roots© blowers, configured for continuous operation without flow pacing. The system employs coarse air diffusers on manifolds positioned behind walls for optimized air distribution.

Grit removal is facilitated by a sloped floor directing debris to buckets on a chain, constructed from mild steel coated in polyurethane, with recent replacements ensuring functionality. Both chains operate simultaneously, allowing for maintenance on one during low flow periods. Design criteria for the grit removal chamber are provided in Table 7-2 below:

Table 7-2: Design Criteria for Grit Removal Chambers

Grit Removal Chamber	
Quantity	2
Type	Aerated Grit tank
Installed	1995
Volume, each	5,264 cu ft/each

Grit Removal Chamber	
Capacity	15 MGD/each

The grit removal system may not be functioning efficiently, as evidenced by periodic digester inspections revealing significant grit accumulation that necessitates approximately 60 vactor truck loads for removal every seven years. Operational strategies involve recycling the center well and hopper from the primary to the front of the screens to maximize grit capture, supplemented by grinders upstream to increase the service life of the sludge pumps.

The structure's integrity, particularly susceptible to hydrogen sulfide degradation above the waterline, should prompt consideration for recoating. Plans include bundling structural maintenance with the plant's CIP schedule.

7.1.2.3 Primary Clarifiers

The effluent from the grit process flows into the two primary clarifiers before flowing into aeration basins. The primary clarifiers are circular in design and equipped with aluminum covers. Adjacent to the north side of the weir lies the pre-aeration tank which is currently non-operational. Design criteria for the primary clarifiers are provided in Table 7-3 below:

Table 7-3: Design Criteria for Primary Clarifiers

Primary Clarifiers	
Number of Basins	2
Diameter	80 ft
Surface Area, each	5027 sq ft
Side Water Depth	10 ft
Installed	1986

The primary clarifiers consist of the influent line, dual flumes, paired tanks, odor control suction conduits, rakes, motors, and weirs, each contributing to the operational framework. New gear boxes were installed in 2016. Recoating of the rake outs was conducted in 2018, with periodic monitoring of overtorque via alarm systems linked to the Supervisory Control and Data Acquisition (SCADA) interface. Operational features include spray nozzles for scum layer manipulation, fiberglass baffles and v-notch weirs subject to annual cleaning due to delamination and breakage. Aluminum covers, each equipped with a separate P-trap drainage system remain in good condition. The primary clarifiers are not pressure washed, and instead are washed manually by hose on an annual basis.

7.1.2.4 Primary Effluent Pumps

Effluent from the primary clarifiers flows into a wet well at the biofilter, where it is transferred to the aeration basins by gravity. There are five primary effluent pumps, each with a capacity of 7,600 gpm at 50 feet of head (upgraded in 2013), that handle typical flows of 4 to 5 MGD under normal conditions and up to 30 MGD during storm events. During peak flows, three pumps are operational, and while the system is capable of running four pumps, it is avoided to protect the

pipe. Three pumps running at full speed achieve 33 MGD, which is the hydraulic capacity of the plant, and secondary bypass begins when flows reach 32.5 MGD.

The primary effluent pumps are FLYGT dry pit/submersible units, located in a ventilated and well-lit basement accessible by stairs. They include level control mechanisms, flow meters, transducers, and backup transducers to ensure efficient operation. Design specifications for these pumps are detailed in Table 7-4 below:

Table 7-4: Design Criteria for Primary Effluent Pumps

Primary Effluent Pumps	
Type	Dry pit/submersible from FLYGT
Installed	2013
Capacity, each	7,600 gpm at 50 ft head
Wet Well Surface Area	507 sq ft
Wet Well Volume	26,548 gallons

Flow meters are installed to measure the flow, with backup transducers available to ensure continuous monitoring. Pump fail alarms are integrated into the system to promptly alert operators in case of malfunction. Furthermore, a secondary bypass alarm diverts flows into the contact chamber in the event of an overflow, enhancing system resilience. Standard operating procedure dictates that during normal flows, one pump is utilized, with occasional activation of the second pump as dictated by demand. Activation of a second and third pump is automatic, while that of fourth and fifth is manual to protect the conveyance pipes before introducing higher flow rates.

7.1.2.5 Aeration Basins

The primary effluent pumps transfer the primary effluent flow to the biofilter wet well, where the effluent then flows by gravity to the aeration basins. Air is supplied by Aerostrip diffusers, installed in 2001, a substantial portion of which underwent replacement in 2020. A divider wall was constructed in 2001 in each aeration basin to create an upstream anaerobic selector zone aimed at improving sludge settleability, and a floating mixer was installed at the same time to enhance mixing. Operating under the purview of a four-blower arrangement, the system was initially equipped with three, 200 hp Hoffman blowers, installed in 1985. Currently, one is in use as a backup. Two, 100 hp Neuros Turbo blowers (2014) and a 75 hp Roots blower (2015) have been added. Augmenting the structural integrity of the system, zinc-based cathodic protection mechanisms are integrated within the tank infrastructure. Design criteria for the aeration basins are provided in Table 7-5 below:

Table 7-5: Design Criteria for Aeration Basins

Aeration Basins	
Quantity	2
Volume, each	55,600 cu ft
Installed	1986
Anaerobic Length (assumed)	16 ft

Aeration Basins	
Anaerobic Area	1,500 sq ft
Aerobic Area	6,700 sq ft
Side Water Depth	13.5 ft
Anaerobic Zone Mixer	
Quantity	2
Motor Size	3 hp
Blowers	
Quantity	4
Positive Displacement Blowers (Hoffman)	One, 200 hp, 2,655 cfm at 7.1 psi
Turbo Blowers (added 2001)	Two, 100 hp, 2,500 cfm at 8.5 psi
Roots Blowers (added 2001)	One, 75 hp
Diffusers	
Type	Strip Aerators
Total Diffuser Area	674 sq ft
Specific Air Rate	1.43 scfm/sq ft

7.1.2.6 Secondary Clarifiers

The mixed liquor from the aeration basins flows into a secondary clarifier influent splitter box, where it is divided between two flumes, which then feed into the connected secondary clarifiers. The secondary clarifiers are equipped with circular rake arms and sludge withdrawal pipes, also known as organ pipes. The settled activated sludge is drawn up through the organ pipes and conveyed to the center RAS (Return Activated Sludge) collection box, then flows by gravity to an individual inlet box attached to the common RAS wet well at the Secondary Recirculation Pump Station. Both the splitter box and the RAS wet well contain a reserved channel and box for connecting an additional secondary clarifier in the future. Design criteria for the secondary clarifiers are provided in Table 7-6 below:

Table 7-6: Design Criteria for Secondary Clarifiers

Secondary Clarifiers	
Number of Basins	2
Diameter	110 ft
Basin Surface Area, each	9,503 sq ft
Side Water Depth	14 ft
Installed	1985

The organ pipes require daily flushing and manual operation via T-handles, necessitating maximal RAS flow for effective cleaning. Recent upgrades include drives and gearboxes installed in 2017, complementing the original 1985 rake arms, which were recoated in 2018. The scum removal system mirrors that of the primary clarifiers, with overtorque alarms providing additional safety measures. Occasionally, one clarifier may be taken offline during the summer months for maintenance in order to avoid a disruption of the sludge blanket caused by storm-induced large inflows.

7.1.2.7 RAS System

The settled activated sludge in the secondary clarifiers is drawn up through the organ pipes and conveyed to the RAS collection box, then flows by gravity to an individual inlet box attached to the common RAS wet well at the secondary recirculation pump station. The RAS system is responsible for returning a portion of the sludge from the secondary treatment process back to the aeration basin to maintain desired biomass population for biological treatment. The sludge is directed through organ-like pipes into an individual inlet box attached to a common wet well. When one secondary unit is taken offline, the system functions similarly to a splitter box, redirecting flow accordingly. Design criteria for the RAS pump are provided in Table 7-7 below:

Table 7-7: Design Criteria for RAS Pump

RAS Pump	
Type	Dry pit/ submersible pump
Quantity	2
Capacity, each	3,900 gpm
Motor	35 hp
Installed	2019

Two centrifugal pumps, installed in 2019, handle the flow within the RAS system. These pumps, which have a motor control center located in a dry pit, are vital to the operation. One pump typically runs with a VFD to control its speed, while the other pump serves as a backup. Based on the DMR data, RAS flow ranges between 1.7 to 5.7 MGD. The sludge is returned to the aeration basin through a single line that has a tee and two valves. This allows the flow to be directed either to the aeration basin splitter box or to a biofilter wet well. The biofilter is currently out of service. Maintenance of the pumps is largely handled in-house, with routine inspections focusing on components like impellers, which are pulled annually to ensure they are in good condition. The system is integrated into the plant's SCADA system, which alerts staff to any operational changes, such as fluctuations in power use, allowing them to address issues promptly.

7.1.2.8 Chlorine Disinfection Contact Chambers

The chlorine disinfection system uses sodium hypochlorite. Pump speed regulation is automated to maintain optimal performance. Chlorination is facilitated by a dedicated day tank containing hypochlorite solution, with dosing rates dynamically adjusted based on flow conditions, utilizing a flow-based dose approach. Chemical delivery logistics are managed by a reliable contractor, ensuring timely and reliable supply, with contingencies in place such as backup suppliers in Tacoma to mitigate potential supply chain disruptions.

Dechlorination follows the chlorination process and involves the utilization of sodium bisulfite. Initially stored in large fiberglass tanks, chemical storage has since transitioned to the use of plastic for hypochlorite while retaining fiberglass for sodium bisulfite. Hypochlorite is injected at the mixing box, while sodium bisulfite is injected at the final effluent weir. Design criteria for the disinfection chambers and equipment are provided in Table 7-8 below:

Table 7-8: Design Criteria for Chlorine Disinfection Contact Chambers

Chlorine Disinfection Contact Chambers	
Channels	2
Volume, each	28,000 cu ft
Side Water Depth	15 ft
Chlorination System	
Injection Pumps, each	2 pumps 31.8 GPH
Sodium Hypochlorite Tank Volume, each	6,000 gallons
Installed	1996
Dechlorination System	
Injection Pumps, each	2 pumps 8.5 GPH
Sodium Bisulfite Tank Volume, each	1,500 gallons

7.1.3 Solids Stream Processes

7.1.3.1 Primary Sludge Pumping

Primary sludge is pumped continuously as a thin sludge from the primary clarifiers directly to the anaerobic digesters. Two primary sludge pumps, each 4-inch, 35 gpm diaphragm pumps, are located in the primary sludge pump station in the primary clarifier building; both are equipped with grinders. The system operates on timed cycles with simultaneous utilization of both primary sludge pumps. The primary scum pumps are used as backups for the primary sludge pumps. During maintenance or downtime, the scum pumps are exclusively activated to provide redundancy and ensure continued operation. These pumps are calibrated to deliver sludge with a solids concentration ranging from 4-5%, obviating the need for additional thickening measures as a 3% concentration is targeted within the digesters. Gauging of system performance is facilitated by torque arm readings, typically falling within the 0-5 range. Situated in the primary effluent basement for accessibility, routine maintenance focuses on the wet ends, with built-driven mechanisms ensuring operational efficiency. Recent enhancements include upgraded pressure tanks for scum handling. Daily checks encompass flow and pressure monitoring, supported by a comprehensive check-sheet regimen and routine sample collection performed on-site three times weekly. Design criteria for the primary sludge pumps are provided in Table 7-9 below:

Table 7-9: Design Criteria for Primary Sludge Pump

Primary Sludge Pumps	
Type	Diaphragm Pump (equipped with grinders)
Quantity	2
Capacity, each	35 gpm
TDH	25 psig
Motor	5 hp
Installed	2010

Primary Sludge Pumps	
Primary Scum Pumps	
Type	Diaphragm Pump
Quantity	2
Capacity, each	35 gpm
TDH	25 psig
Installed	2007

7.1.3.2 Waste Activated Sludge System

The WAS system is located inside the secondary recirculation pump station and is thickened using the rotary drum thickeners prior to anaerobic digestion. The system was installed in 2008 and equipped with VFD technology. It operates under the guidance of laboratory test results, conducted three times weekly to assess sludge thickness, with a target of 0.8-1% solids concentration serving as the basis for waste determination. The system includes centrifugal pumps with dry pit configurations and mechanical seals to facilitate sludge transfer. Operational procedures entail continuous adjustments to avoid adverse effects on the activated sludge process. The system's operational parameters, including flow rates and wastage, are remotely monitored and controlled via the SCADA system. The system can sustain operations for approximately four days without wastage. Design criteria for the WAS system are provided in Table 7-10 below:

Table 7-10: Design Criteria for WAS System

WAS Pump	
Type	Vertical Non-Clog pump
Quantity	2
Capacity, each	225 gpm
TDH	16 ft
Motor	2 hp
Installed	2008

7.1.3.3 Thickening Process

The thickening process was updated in 2008, with a new thickening room added on the roof of the existing sludge thickener tank to accommodate two rotary drum thickeners (RDTs). The RDTs are operated to achieve solids concentrations in the range of 4-7% for the thickened WAS (TWAS). Polymer addition is facilitated through a dry poly-batch system. The mixing apparatus, likely original or replaced in the 1990s, is comprised of PVC components, including a 1,000-gallon mixing tank and a corresponding 1,000-gallon day tank, with polymer batches automatically initiated when levels are low. Utilized on a daily basis, the RDT represents a transition from a dissolved air floatation thickener (DAFT) system in 2008. As a precautionary measure, system redundancy is ensured through monthly rotation of backup equipment. Design criteria for the RDTs is provided in Table 7-11 below:

Table 7-11: Design Criteria for Thickening Process

Thickening process	
Type	Rotary Drum Thickeners
Number	2
Capacity	225 gpm
Motor, each	20 hp
Installed	2008
TWAS Pumps	
Capacity	36.97 gpm
Motor	7.5 hp
Installed	2009

Polymer procurement has not posed any significant challenges, with a typical pallet lasting approximately 1.5 months and two contractors supplying dry and liquid polymer variants, namely Polydyne and BASF, respectively. Operational oversight involves adjustments to feed and polymer rates, with periodic inspections supplemented by routine lead-driven assessments. The system is primarily automated, although occasional repairs to motors, mixers, and sprayers are undertaken, with particular attention directed towards belt and bearing maintenance. To mitigate odor concerns, a louver and exhaust fan are used, operating continuously, and modulated based on temperature conditions.

7.1.3.4 Anaerobic Digesters

The plant utilizes two anaerobic digesters to treat both the primary sludge and the TWAS; digesters are equipped with floating lids. Mixing within the digesters is accomplished using recycle pumps. Both digesters operate in parallel and treat to Class B biosolids standards. Gas generated during digestion is flared using a candlestick system, with the resulting digester gas utilized for heating the boilers without the use of scrubbers, maximizing gas utilization. Cleaning of digesters occurs approximately every seven years, triggered by grit level accumulation. A preferred hydraulic retention time (HRT) of 30-35 days is targeted, with shortened HRTs leading to odor issues. The gas system includes compressors directed towards boiler heating, with spare compressors available for immediate replacement if needed. Monitoring and maintenance of the gas system are conducted regularly to ensure operational compliance. Foaming issues are managed through drainage systems and daily checks. Design criteria for the anerobic digesters are provided in Table 7-12 below:

Table 7-12: Design Criteria for Anaerobic Digesters

Anaerobic Digesters	
Number of Units	2
Active Digestion Volume, Each	39,200 ft3
Sludge Storage Volume, Each	11,800 ft3
Total, Each	51,000 ft3
Diameter	50 ft
Maximum Side Water Depth	26 ft
Side Wall Depth	34.33 ft
Effective Gas Storage Volume	21,000 ft3
Installed	1986
Recirculation Pumps	
Capacity	350 gpm
Motor	4 hp
Installed	2019
Boiler	
Capacity	50 hp
Installed	2009
Heat Exchanger	
Type	Serpentine Heat Exchangers
Capacity	1,000,000 BTU(1)/hr
Installed	2014
Gas Burner	
Capacity	66 SCFH(2)
Installed	2009

(1) BTU – British Thermal Unit
(2) SCFH – Standard cubic feet per hour

7.1.3.5 Sludge Dewatering

The sludge dewatering system operates on a centrifuge-based setup, consisting of one centrifuge, two feed pumps, two cake pumps, and a polymer addition system. Liquid polymer, sourced from totes, is used for optimal processing efficiency. The centrifuge has a capacity of 150 gpm, producing an average cake concentration of 22-23%. Typically, one polymer tote is in use while a second is kept in reserve.

The centrifuge system is elevated on a pad to mitigate flooding risks and is powered by an adjacent Motor Control Center (MCC). The system receives sludge through piping connected to two basement pumps, each equipped with suction grinders. Operating hours generally span six hours per day, with flexibility to increase as flow rates rise. Regular maintenance and alarm monitoring are performed to ensure smooth operation. Contingency plans include maintaining an inventory of spare parts and monitoring for potential issues, such as pipeline bursts, which could impact operations. Discharge procedures route dewatered sludge to a wet cake pump which discharges to a dumpster before being loaded into trucks for designated disposal at City-owned forestlands. Since late 1992, 100% of the Class B biosolids produced at the WWTP have

been beneficially utilized on forestlands owned by the Bremerton Water Utility. The biosolids are stored in three storage ponds located in the permitted application area and application occurs an average of 3-5 days a month depending on quantity. The City Forestry division manages applications and the associated infrastructure including the three storage ponds, two applicator vehicles, and one front end loader.

Design criteria for sludge dewatering equipment are provided in Table 7-13 below:

Table 7-13: Design Criteria for Sludge Dewatering

Centrifuge Feed Pump	
Capacity	200 gpm at 39 psig
Motor	40 hp
Installed	2009
Centrifuge	
Type	Centrifuge
Capacity	150 gpm
Motor	100 hp
Installed	2009
Cake pump	
Capacity	50 gpm at 66 psig
Motor	3.72 hp
Installed	2009

7.1.4 Other Processes

The WWTP has two separate odor control systems: one for the Headworks and another for the Biofilter.

7.1.4.1 Headworks Odor Control System

The headworks odor control system is a two-stage wet scrubber designed to handle hydrogen sulfide at concentrations up to 150 parts per million (ppm), targeting a 99% removal efficiency. The system utilizes 12.5% sodium hypochlorite and 50% caustic solution as the primary chemical agents for neutralizing odors. The first stage achieves 80% removal, with the second stage serving as a polishing step to achieve the removal efficiency.

The scrubber uses plastic media, last replaced approximately in 2019. Media flushing is performed annually, and inspections include diffuser checks and monthly odor measurements using handheld devices. The performance remains consistent with minimal improvements after cleaning, and the media lifespan is estimated at around two years. The scrubber operates with two 6,000-gallon hypochlorite tanks, which require delivery every 1.5 weeks in the summer, and two 1,500-gallon caustic tanks, which last about 2-3 months during the summer and all winter on a single tank. Additionally, there is a 250-gallon acid tote used for cleaning the media. Probes monitor the system's pH, with a set point of pH 10.0, and keep track of abnormal sodium

hypochlorite usage, which could indicate leaks affecting the primary system or causing issues downstream.

7.1.4.2 Biofilter Odor Control System

The biofilter odor control system consists of a single-stage wet scrubber, designed for 1 ppm hydrogen sulfide removal with a 99% efficiency target. This system draws suction from the lid of the biofilter and underdrains, as well as the splitter box, which directs primary effluent into the aeration basins. Similar to the headworks system, the scrubber relies on sodium hypochlorite and caustic chemicals for odor treatment. Chemical storage includes two 1,000-gallon sodium hypochlorite tanks and a 500-gallon caustic tank.

The wet scrubber employs plastic media that was replaced in 2014.The media is flushed annually, with no signs of plastic degradation observed. Like the headworks scrubber, this system undergoes annual inspections and monthly odor checks.

7.2 WWTP System Analysis Overview

The following sections assess the plant's current capacity and performance under 2044 projected conditions in accordance with the with the Ecology's Criteria for Sewage Works Design (Orange Book), and ensure compliance with or exceed the design and construction standards referenced in WAC 173-240. This assessment was carried out using the following three primary methods:

- Plant Hydraulic Analysis: A hydraulic model was developed to assess the plant's hydraulic profile and overall capacity.
- Process Modeling Analysis: A process model was created to simulate the performance of the secondary biological process. Desktop calculations were performed to validate the model results and evaluate the capacity of other processes and major equipment directly related to the secondary process.
- Desktop Calculations: The remaining unit processes and major equipment were analyzed using desktop calculations, with their performance compared against design criteria from literature and regulatory guidelines.

Each method and its results are explained in detail in the following sections.

7.3 WWTP Hydraulic Capacity

A hydraulic model was developed using Visual Hydraulics Version 5.1 software by Innovative Hydraulics to evaluate the plant's overall hydraulic capacity. The goal of this assessment was to verify that the existing facilities can handle the design flows without overtopping the structures' walls and to confirm the proper operation of the flow split structure. The hydraulic profile was established from the headworks to the Sinclair Inlet. Field measurement data was collected by

plant operators on March 30 and May 20, 2024 and used to evaluate the current hydraulic conditions.

The hydraulic model of the existing WWTP was created based on record drawings provided by the City. Model results were then compared to field measurements. Water elevation estimates, calculated from freeboard height, were matched against the model-predicted water levels. The modeled water levels closely aligned with the field-measured data, except the following areas:

1. **Primary Clarifier (PC) Effluent Boxes:** The measured water level was 20 to 24 inches higher in PC1 and 8 to 12 inches higher in PC2. Additionally, the primary influent flow rates recorded in the DMR were unbalanced, with 47% going to PC1 and 53% to PC2. Uneven flow entry and mixing through the old screen channels may contribute to this imbalance, resulting in lower flow rates in PC1 and more severe grit accumulation along its flow path, further worsening the disparity. The outlet pipe from the PC effluent box to the primary effluent (PE) pump station wet well is also 27 feet longer on the PC1 side, which increases the likelihood of grit fouling in the outlet pipe. This fouling may further explain the greater difference between the modeled and measured water levels in PC1, highlighting potential operational issues not captured by the model.
2. **Biofilter Pump Station Wet Well:** The measured water level was 6 to 8 inches higher than the modeled level. The effluent box in this area is inaccessible, so no downstream measurements were available. Combined PE and RAS flow enters the wet well, passes through three submerged openings at the bottom of the divider wall, and exits through the effluent channel to the 42-inch aeration basin influent line. The first submerged opening is located near the pipe entrance and exit, which could cause higher headloss across the divider wall by allowing more flow through that opening. If any fine grit accumulates here, it could block the passage and increase headloss, leading to the higher water levels measured in this location.
3. **Aeration Basin Outlet Box**: The measured water level was 18 to 28 inches higher than the modeled level. The turbulent water movement and presence of air in the deep outlet box may have contributed to some of this discrepancy in measurements. Additionally, the flow velocity in the outlet pipe slows to around 1 fps under typical conditions, which could lead to grit accumulation in the pipe. This buildup might increase headloss and contribute to the higher water levels observed.
4. **Chlorine Contact Chamber Effluent Weir:** The measured water level was 7 to 11 inches higher than the modeled level. Since the effluent flow is clean, fouling on the exposed weir is not a concern. The difference may be due to elevation discrepancies from the record drawings.

Changes in elevation from the original design may have contributed to some discrepancies. To address this, it is recommended to survey key elevations and update the plant's hydraulic profile. Grit accumulation, especially in uneven structural areas, is also likely a major factor. Improvements should focus on redesigning these flow distribution areas for more balanced flow.

The model has not been calibrated for these differences yet, as these differences may be attributed to several factors:

- Higher water levels were measured in areas with sufficient remaining freeboard.
- Headloss issues may be resolved through inspection and cleaning.
- The availability of limited high-flow measurement data has precluded precise calibration.

These factors will be revisited as more data becomes available. However, the uneven flow distribution between the two primary clarifiers was simulated under future flow conditions to estimate the hydraulic conditions through PC2 at a higher proportion of the flow. The following scenarios listed in were checked to identify hydraulic bottlenecks in the plant hydraulic profile.

Table 7-14: Hydraulic Scenarios for Plant's Flow Capacity Assessment

Scenario	Summary of Operating Conditions	Purpose of Scenario
S1	• 2044 PHF = 59 MGD • Primary influent flow to PC2 = 33.6 MGD • Primary effluent flow pumped = 30 MGD • Record high tide at 127.96 ft	Assess the plant's hydraulic profile at projected PHF
S2	• Primary influent flow to PC2 = 57% of influent flow • Primary effluent flow pumped = influent flow if influent flow < 30 MGD, if not 30 MGD. • Effluent flume under submerged condition • At least 6-inch free board at chlorine contact chamber • Record high tide at 127.96 ft	Estimate the maximum flow capacity under record high tide and defined hydraulic conditions.
S3	• Primary influent flow to PC2 = 57% of influent flow • Primary effluent flow pumped = influent flow if influent flow < 30 MGD, if not 30 MGD. • Effluent flume under non-submerged condition • Record high tide at 127.96 ft	
S4	• Primary influent flow to PC2 = 57% of influent flow • Primary effluent flow pumped = influent flow if influent flow < 30 MGD, if not 30 MGD. • No overtopping or pressurized conditions through outfall pipes and manholes • Record high tide at 127.96 ft	
S5	• Primary influent flow to PC2 = 57% of influent flow • Primary effluent flow pumped = influent flow if influent flow < 30 MGD, if not 30 MGD. • Effluent flume under non-submerged condition • 22 nozzles at outfall pipe • Mean high water at 124.13 ft	Compare with the 32.4 MGD estimate from the 2008 Rerating Study where assumed: • Effluent flume under non-submerged condition • 20 nozzles at outfall pipe • Previous record high tide of 124.8 ft and 20 nozzles at the outfall pipe.

The key results of the simulated scenarios are summarized below:

- Scenario 1, the WWTP is expected to be unable to handle the projected 2044 Peak Hour Flow (PHF) at the record high tide due to:
  - Water levels overtopping walls or finish grade in secondary processes, the chlorine contact chamber, and outfall manholes/pipes.
  - Submerged conditions at the influent flumes of the primary clarifiers.
  - Approximately 1 foot of freeboard remained at the screen channels in the headworks.
- Maximum Flow Capacities Under Different Scenarios:
  - Scenario 2: 34.0 MGD to maintain at least a 6-inch freeboard at the chlorine contact chamber.
  - Scenario 3: 30.5 MGD to keep the effluent flume in non-submerged conditions.
  - Scenario 4: 23.5 MGD to prevent overtopping or pressurizing the outfall pipes and manholes.
- Scenario 5: Validates the 2008 Rerating Study findings, with the plant's maximum flow capacity estimated at 34 MGD, aligning closely with the 32.4 MGD estimated in the 2008 study.
- Outfall System as a Hydraulic Bottleneck:
  - The outfall system limits hydraulic capacity and causes constraints in upstream processes.
  - Instances of high flow exceeding rated capacities (23.5 MGD, 30.5 MGD, 34 MGD) occurred 210, 66, and 41 times, respectively, between February 2019 and December 2023.
- Operational Observations:
  - The plant's O&M team resealed manhole covers following recent storm events, highlighting the need for further investigation and repair of the outfall system to address potential leaks or pressure-related issues..
  - These storm events caused manhole covers to lift off, indicating pressurized flow or elevated water levels in the outfall piping. If pressure release points are absent, hydraulic issues could transfer upstream, potentially causing the effluent flume to become submerged.

Based on these findings, the effluent outfall is the primary hydraulic bottleneck limiting the plant's capacity. Recommendations include:

- Conducting a thorough investigation and repair of the outfall system to address leaks and pressurization, particularly in the section between the southeast corner of the facilities and the manholes located between State Highway 3 and the State Highway 204 ramp. This includes the section from the Onsite Manhole through Manhole 1 to Manhole 2, as well as from Manhole 2 to Manhole 4.
- Implementing measures to enhance the hydraulic capacity of the outfall and improve overall WWTP performance.

The original design peak flow was 29.5 MGD, as indicated by 1986 record drawings. The 2008 Rerating Study also identified the outfall system and effluent flume as capacity bottlenecks. Historical data suggest that previous flow exceedances likely occurred during times of lower water levels in Sinclair Inlet, minimizing apparent impacts.

Given the frequency of high flow events, coupled with potential risks from severe storm conditions, addressing the hydraulic constraints in the outfall system is critical for long-term WWTP operation.

7.4 WWTP Secondary Process Performance and Capacity

A biological process simulation was performed to assess the performance of the plant's secondary process, supported by desktop calculations. A plant process model was created using BioWin Simulator Version 6.2, developed by Envirosim. The BioWin model leverages complex biological interactions to predict material transformations and pollutant removal across various processes within a plant, offering detailed insights into wastewater characteristics at each stage of treatment.

7.4.1 Process Model Calibration

The process model was configured to represent the existing WWTP processes, as shown in the Figure 7-1. Following the City's decision to remove the biofilter, which was out of use for over 10 years due to odor and performance issues, the biofilter was excluded from the secondary process capacity evaluation. Plant daily monitoring data from January 2018 through September 2023, along with additional sampling data collected between October 16 and December 18, 2023, were used to develop the detailed influent wastewater characteristics for the model.

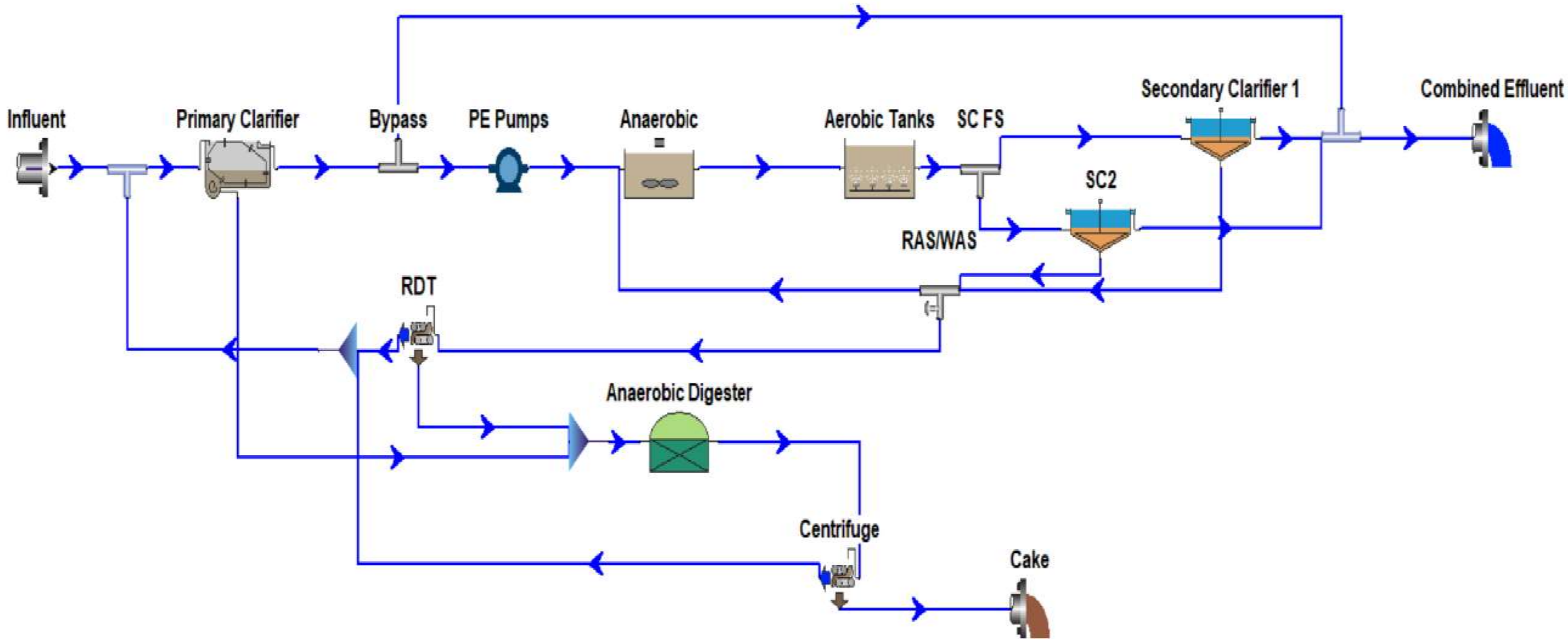

Figure 7-1: BioWin Simulator Layout for Bremerton WWTP

7.4.1.1 Existing Wastewater Characterization

An existing wastewater characterization was conducted to improve the accuracy of the biological process modeling. Additional water quality samples were collected from the plant influent from October 16 through December 18, 2023. Total Kjeldahl nitrogen (TKN) was also measured from grab samples collected from the centrate, from the centrifuge five times during the same period.

The following parameters were provided for the plant influent:

- Chemical oxygen demand (COD)
- Filtered COD (fCOD)
- 5-day carbonaceous biochemical oxygen demand ($CBOD_5$)
- Total suspended solids (TSS)
- Volatiles suspended solids (VSS)
- Total Kjeldahl nitrogen (TKN)
- Ammonia nitrogen (NH_3-N)
- Total phosphorus (TP)
- Oil and grease
- Alkalinity as calcium carbonate ($CaCO_3$).

The analytical results were used to establish ratios between normally monitored influent and effluent parameters, such as $CBOD_5$, and parameters that are not routinely monitored, but required as the inputs in process modeling, such as COD. Table 7-15 contains a list of the wastewater characteristics ratios which are used as the modeling inputs. Table 7-16 contains a summary of results from the sampling and analysis.

Table 7-15: Ratios Used to Establish BioWin Modeling Input Parameters

Parameter	Value	Source	Sampling Average	Typical
COD to $CBOD_5$	1.90[(1)]	Calibrated	1.75	1.9 - 2.2[(2)]
fCOD/COD	0.32[(1)]	Calibrated	0.28	0.3 - 0.5[(2)]
VSS/TSS	0.76	Sampling	0.76	-
NH_3-N /TKN	0.67	Sampling	0.67	0.5 - 0.8[(2)]
TKN/$CBOD_5$	0.20[(3)]	Historical	0.21	0.17 - 0.20[(4)]
TP/$CBOD_5$	0.022	Sampling	0.022	0.027 - 0.036[(4)]
Alkalinity, mg/L as $CaCO_3$	280	Sampling	280[(5)]	-

Notes:

(1) Adjusted to the selected value during calibration to match the biomass yield for the historical data.
(2) Typical range provided in Influent Specifier Template for raw wastewater provided by EnviroSim.
(3) Average from 2019 through 2022 historical data.
(4) Typical range estimated for raw wastewater (Table 3-18 and Table 3-21 in Metcalf & Eddy / AECOM Wastewater Engineering Treatment and Resource Recovery, 5th Ed, 2014).
(5) Average of the supplemental samples after removing the lowest and highest value.

Table 7-16: Water Quality Characteristics for Influent and Centrate from Supplemental Sampling

Influent Parameter	COD	fCOD	$CBOD_5$	TKN	NH_3-N	TP	TSS	VSS	Alkalinity
10/16/2023	-	-	-	-	23.9	-	-	-	-
10/23/2023	-	-	-	-	34.3	-	-	-	-
10/30/2023	-	-	-	-	29.8	-	-	-	-
11/6/2023	260	46	107	25	18.6	2.6	174	139	5.5
11/7/2023	230	ND	104	31	-	3.2	171	125	6.4
11/9/2023	280	58	-	34	-	3.6	-	-	10.3
11/13/2023	-	-	-	-	21.4	-	-	-	-
11/14/2023	260	32	164	33	-	3.4	185	133	11.1
11/15/2023	320	100	-	33	-	4.1	-	-	7.8
11/19/2023	-	-	124	-	-	-	181	137	-
11/20/2023	-	-	161	-	30	-	192	140	-
11/26/2023	430	100	216	40	-	4.1	262	212	17.6
11/27/2023	280	78	257	38	22.9	4.1	223	186	12.7
11/28/2023	280	70	188	42	-	4.5	247	197	20.8
12/3/2023	120	75	159	28	-	2.7	187	131	13.9
12/4/2023	-	-	-	-	18.1	-	-	-	-
12/6/2023	100	ND	68	15	-	1.8	106	73	6.2
12/10/2023	-	-	-	-	21.2	-	-	-	-
12/18/2023	-	-	-	-	22.6	-	-	-	-
Centrate	11/6/2023	11/7/2023	11/9/2023	11/14/2023	11/15/2023				
TKN	1,200	1,300	1,400	1,400	1,500				

(Unit: mg/L)

7.4.1.2 Calibration and Validation Results

The WWTP BioWin model was calibrated for the average conditions observed between April 16 and May 15, 2022. Model validation was performed by applying the calibrated model for the average conditions for November 16 and December 15, 2022 and comparing the model results to measured values. Table 7-17 summarizes a list of key calibration and validation modeling inputs and outputs.

Table 7-17: Secondary Process Modeling Calibration Results

		Calibration for April 16 to May 15, 2022			Validation for November 16 to December 15 2022		
Parameters	**Unit**	**Model**	**DMR[1]**	**% Diff.[2]**	**Model**	**DMR[1]**	**% Diff.[2]**
Model Input Parameters							
Influent Wastewater							
Flow	MGD	4.04	4.04	0.0	4.64	4.64	0.1
COD	mg/L	308	NA	-	323	NA	-
$CBOD_5$	mg/L	162	162	0.0	170	170	0.0
TSS	mg/L	211	210	0.4	211	211	0.1
VSS	mg/L	160	NA	-	190	NA	-
TKN	mg N/L	40.5	NA	-	32.0	32	0.0
Ammonia	mg N/L	27.3	27	0.0	28.5	28	0.1
TP	mg P/L	3.6	NA	-	3.8	NA	-
Alkalinity	mg/L as $CaCO_3$	280	NA	-	280	NA	-
Process							
Primary Clarifier Area	sf	10,053	10,053	0	10,053	10,053	0
Aeration Basin Volume	MG	0.83	0.83	0	0.83	0.83	0
Secondary Clarifier Area	sf	19,007	19,007	0	19,007	19,007	0
Anaerobic Digester Volume	MG	0.65	0.65	0	0.68	0.68	0
Model Operating Parameters							
Temperature	°C	14.4	14.4	0.2	13.7	13.7	0.1
AB SOTE[3]	%	26%	NA	-	26%	NA	-
RAS Flow	MGD	3.02	3.02	0.0	2.96	2.96	0.1
Aeration DO	mg/L	4.1	4.10	0.0	4.9	4.92	0.1
Model Output Parameters							
Primary Clarification							
Primary Sludge Flow	gpd	15,500	14491	7.0	14,200	13,835	2.6
TSS Removal Rate[4]	%	71%	NA	-	67%	NA	-
$CBOD_5$ Removal Rate	%	41%	NA	-	38%	NA	-
TKN Removal Rate	%	13%	NA	-	7%	NA	-
TSS	mg/L	61.3	NA	-	69.2	NA	-
$CBOD_5$	mg/L	96.4	NA	-	105.9	NA	-
TKN	mg/L	40.7	NA	-	33.0	NA	-
Aeration Basins							
SRT[5]	days	3.9	NA[5]	-	3.3	NA[5]	-
HRT	hours	4.9	NA	-	4.3	NA	-
MLSS	mg/L	1,196	1,228	2.6	1,338	1,261	6.0
MLVSS	mg/L	983	NA	-	1,127	NA	-
MLVSS/MLSS	-	0.82	0.84	2.0	0.84	0.82	2.9

Aeration Basin DO	mg/L	4.1	4.10	0.0	4.9	4.92	0.1
Airflow(6)	scfm	1,218	NA	-	1,699	NA	-
Other Operating Parameters							
RAS TSS	mg/L	2,781	NA	-	3,418	NA	-
WAS Flow	MGD	0.09	0.09	0.6	0.099	0.09	7.6
WAS TSS	ppd	2,111	NA	-	2,822	NA	-
Secondary Effluent							
$CBOD_5$	mg/L	8.0	9.0	11.5	8.1	8.0	1.9
TSS	mg/L	12.6	11.6	8.3	12.1	12.2	1.4
pH		7.1	7.2	0.9	7.1	7.3	3.4
Ammonia		33.7	33.1	1.8	25.9	30.0	13.6
TKN		36.1	NA	-	27.8	NA	-
NO_3-N		0.0	NA	-	0.0	NA	-
Alkalinity	mg/L as $CaCO_3$	304.0	NA	-	273.2	NA	-
RDT(7)							
Thickened TSS (TS)	%	7.5%	(7.4%)	-	7.0%	(7.5%)	-
Thickened TSS	ppd	1,938	NA	-	2,622	NA	-
Thickened VSS	ppd	1,579	NA	-	2,191	NA	-
Anaerobic Digestion							
SRT (HRT)	days	35.6	NA	-	36.3	NA	-
TSS (TS)	%	2.8%	(2.5%)	-	3.3%	(3.0%)	-
VSS	%	1.5%	NA	-	1.7%	NA	-
VSS/TSS (VS/TS)	%	51.7%	(65.5%)	-	52.1%	(65.7%)	-
Gas	scfd	39,561	NA	-	50,930	NA	-
Gas/VS Fed	scf/lbs	7.3	NA	-	8.0	NA	-
VSS destruction	%	58%	NA	-	58%	NA	-
Centrifuge(8)							
Dewatered TSS (TS)	%	22%	(22%)	-	22%	(22%)	-
Cake Dry TSS	ppd	3,672	(3,229)	-	4,206	(4,829)	-
Centrate TKN	ppd	1,536	NA(9)	-	973	NA(9)	-
Centrate Flow		16,700	NA	-	16,400	NA	-

Notes:

(1) Average of the operating data for the selected period, unless otherwise noted.

(2) % Difference = (Measured Value - Modeled Value) / Measured Value x 100

(3) SOTE was assumed to be 2%/feet water depth with 13 ft submergence. SOTE provided in the product submittal by Sanitaire in 2001 indicated that the SOTE of the Type T3 strip aerators was 35.6%, assuming 13.5 ft of submergence.

(4) Historical TSS removal rate was recorded only from 3 January 2019 through 5 July 2020 and varied from 29% to 75%. In the calibration and validation, the TSS removal rates were adjusted to get a good match for MLSS, MLVSS/MLSS in the aeration basins and TS and VS/TS in the anaerobic digestion.

(5) Estimated MCRTs are provided in DMR and the average was 1.5 in 2022, calculated using the following formula: MCRT = AB MLSS inventory/(Final effluent TSS + WAS TS). Typical WAS TS can be ranged from 4,000 mg/L to 12000 mg/L and is much greater than WAS TSS per typical ranges provided for untreated activated sludge (Table 13-5 in Metcalf & Eddy / AECOM Wastewater Engineering Treatment and Resource Recovery, 5th Ed, 2014). Using WAS TSS could result in a much shorter MCRT.

(6) Typically, the plant has run two blowers. Severe air leakage was observed during the site visit for condition assessment.

(7) TSS capture rate is assumed to be 95% for RDTs.

(8) TSS capture rate is assumed to be 97% for centrifuge.

(9) TKN ranged from 1,200 to 1,500 mg/L from additional sampling grab samples

The objective of model calibration is to minimize the error between the simulator predictions and a selected data set with minimal change to model input, kinetic, and stoichiometric parameters. An acceptable calibration is achieved when simulation results are within 20% of actual values

from two different data sets at steady state operating conditions. Using this criterion, an acceptable calibration was achieved using the April through May 2022 data sets. The calibration was validated against average operating conditions for separate period of November and December 2022. The calibrated parameters were used in the modeling for secondary process capacity estimation.

7.4.1.3 Limitations of BioWin Modeling

Certain aspects of biological process design are beyond the predictive capabilities of a biological process simulator, and it is not feasible to fully examine performance of a wastewater treatment facility using biological simulation alone. It should be noted that this evaluation and the process simulations may not fully account for:

- Potential for changes in sludge quality and secondary clarifier performance with changing process mean cell residence time (MCRT) and food to microorganism ratio.
- Potential for hydraulic limitations of infrastructure and capacity limitations of equipment.
- Potential for changes to digester performance due to changes in loading.
- Potential for complex chemical and biological interactions that may occur in a full-scale system, such as foaming, solids bulking, mixing limitations, tank geometry, short circuiting, poor solids distribution, and impacts to microbiology by inhibitors.

These potential impacts are beyond the scope of this evaluation or beyond the ability to predict using a biological process simulator.

7.4.2 Process Modeling and Capacity Assessment

The primary role of the secondary process at the WWTP is to remove $CBOD_5$ and TSS. Table 7-18 summarizes the average monthly effluent quality goals based on the current NPDES permit limitations on these parameters.

Table 7-18: Effluent Quality Goal

Design Criteria	Units*	NPDES Permit Requirements
$CBOD_5$ Average Monthly	mg/L	25
TSS Average Monthly	mg/L	30
pH Average Monthly		6.0 to 9.0

The current NPDES permit doesn't include any limits on nitrogen or phosphorus, but the City has made an effort to optimize nitrogen removal and planning activities required by the Puget Sound Nutrient General Permit (PSNGP), which went into effect on January 1, 2022. The current planning doesn't include any direct improvements on the secondary process but monitoring and investigating the side-stream nitrogen load and its reduction as potential methods to reduce the total inorganic nitrogen load in the final effluent. An anaerobic selector

zone was added to the existing aeration basins to improve sludge settleability. Therefore, it is recommended that the plant maintain an optimal solids retention time (SRT) range of 4 to 5 days that is sufficient for $CBOD_5$ removal while avoiding excessively long SRTs that could inhibit nitrification.

The treatment capacity of the secondary process was assessed based on the design SRT required to meet the effluent $CBOD_5$ goals. Additionally, the typical secondary clarifier operation parameters were considered to meet the TSS goal. Table 7-19 summarizes the key operation criteria used for the assessment.

Table 7-19: Operation Criteria for Secondary Process Capacity Assessment

Design Criteria	Unit	Value
Aeration Basins		
Total SRT	days	4 to 5(1)
Targeted DO Concentration in Aerobic Zone	mg/L	2 (1)(2)
Anaerobic Selector Zone Volume%		20%(3)
Process MLSS	mg/L	1,500 or 2,500(4)
Current RAS Pumping Capacity	MGD	1.7 to 5.7(5)
Secondary Clarifier		
Side Water Depth of Current Clarifiers	ft	14
Sludge Volume Index	mL/g	250(6)
Solids Loading Rate- Average	lbs/sf/day	19 to 28(7)
Solids Loading Rate- Peak	lbs/sf/day	48(7)
Surface Overflow Rate - Average	gal/sf/d	400 to 600(7)
Surface Overflow Rate – Peak	gal/sf/d	1,000 to 1,200(7)

Notes:

(1) For BOD removal, a typical SRT of 3 to 5 days is recommended, depending on the process temperature. At temperatures around 10°C, an SRT of 5 to 6 days is recommended for BOD removal only. The original SRT of the aeration basins was estimated to be 2 to 3 days when the process was designed as a contact basin following the biofilter. The equivalent SRT of the biofilter in the original design was estimated to be about 1.3 days, based on the correlation curve provided for BOD loading versus equivalent SRT for trickling filters, sourced from "Wastewater Engineering: Treatment and Resource Recovery" (5th Edition, 2014, by Metcalf & Eddy).

(2) Minimum DO at 1 mg/L is assumed to be maintained under peak day BOD loading conditions.

(3) No record drawing is available for the anaerobic zone, and the volume is assumed to be 20% of the aeration basin by roughly estimating the horizontal length of the anaerobic basin using Google Maps.

(4) Due to poor settleability, the maximum MLSS in the aeration basins is currently limited to 1,500 mg/L, which is the limit that plant operations staff currently consider feasible given high SVI. The assessment was provided for two MLSS concentrations: one for the current limit and the other at 2,500 mg/L. Typical MLSS concentration could be 3,000 to 4,000 mg/L for the Anaerobic/Oxic (A/O) process, according to Metcalf & Eddy .

(5) Historical RAS pumping rates from the recent DMR.

(6) In the last two years, the annual average SVI and the maximum monthly average SVI at the plant were 250 and 440 milliliters per gram (mL/g), respectively. The annual average SVI was assumed to assess the secondary clarifier capacity.

(7) Typical design ranges for the activated sludge process are provided. A higher range is recommended for the activated sludge process with a properly functioning selector.

Table 7-20 summarizes the operating scenarios evaluated through process modeling, along with additional scenarios used in desktop calculations to assess aeration and clarification capacity.

Table 7-20: Operating Scenarios for Modeling and Desktop Calculations

Scenario	Summary of Operating Conditions	Purpose of Scenario
Assess Existing Aeration Basin Capacity (Modeling and Desktop Calculations)		
S1	2044 MML – Most Conservative Monthly Average • Two aeration basins and two secondary clarifiers in service • Primary clarification – 46% TSS and 25% CBOD removal assumed • Lowest monthly average temperature=11.5°C • Aeration basin max MLSS at 2,500 • Desktop calculations only for S1b with MLSS at 1,300 mg/L	Determine if the SRT can reach the target of 5 days under the most conservative monthly loading condition and the lowest monthly average temperature.
S2	2044 AAL – Typical Condition • All aeration tanks in service • Annual average temperature=17°C • Aeration basin max MLSS at 2,500 mg/L • Desktop calculations only for S2b with MLSS at 1,300 mg/L	Investigate whether the SRT can be maintained at 4 days under average annual wastewater load and temperature.
S3	2044 AAL – Maintenance • One aeration basin offline • Annual average temperature=17°C • Aeration basin max MLSS at 2,500 mg/L • Desktop calculations only for S3b with MLSS at 1,300 mg/L	Assess if one aeration basin can be taken offline during maintenance while still maintaining an SRT of 4 days.
Assessment of Aeration Capacity (Desktop Calculations Only)		
S4	2044 MML • Aerobic tank DO at 2.0 mg/L • Maximum average monthly temperature=22°C	Estimate the aeration demand under MML during dry months.
S5	2044 PDL and PD DWF • Aerobic tank DO at 1.0 mg/L • Maximum average monthly temperature=22°C	Estimate the aeration demand under PDL during dry months.
S6	2044 PDL and PD WWF • Aerobic tank DO at 1.0 mg/L • Average annual temperature=17°C • PE flow to secondary limited at 22.8 MGD	Estimate the aeration demand under PDL during wet months.
Assessment of Secondary Clarifier Capacity (Desktop Calculations and State Point Analysis)		

Scenario	Summary of Operating Conditions	Purpose of Scenario
S7	2044 MM WWF • All secondary clarifiers online • MLSS = 2,500 mg/L • RAS = 5.7 MGD • SVI = 250 mL/g	Verify that the clarifiers operate within the typical operational ranges recommended for average conditions under 2044 MM WWF.
S8	2044 AAF - Maintenance • One secondary clarifier offline • MLSS = 2,500 mg/L • RAS rate = 75% of AAF • SVI = 250 mL/g	Confirm that the clarifiers operate within typical operational ranges during maintenance.
S9	22.8 MGD (Limit recommended from 2008 Rerating Study) • All secondary clarifiers online • PE flow to aeration basins = 22.8 MGD • MLSS = 1,500 mg/L • RAS = 5.7 MGD • SVI = 175 mL/g	Validate the capacity assessed in the previous study with the assumed conditions.
S10	MMF and PHF Capacity assuming • All secondary clarifiers online • MLSS = 2,500 mg/L • RAS = 5.7 MGD • SVI = 250 mL/g	Estimate the peak flow that the secondary clarifiers can handle under assumed conditions.
S11	MMF and PHF Capacity assuming • All secondary clarifiers online • MLSS = 1,300 mg/L • RAS = 5.7 MGD • SVI = 250 mL/g	Estimate the peak flow that the secondary clarifiers can handle under assumed conditions.

The results for secondary treatment capacity depend on the performance of primary clarification. Therefore, the estimated flow capacity can be increased if additional treatment capacity is added to the primary treatment. For the existing capacity assessment, no expansion was assumed for the primary clarifier. The performance of the primary clarifier was estimated based on the effluent TSS concentrations reported in DMR's for 2019 and 2020. A trace line between hydraulic retention time and primary TSS removal rates was used to determine the primary clarifier TSS and the corresponding $CBOD_5$ removal rates for each scenario. The following section summarizes the results of the secondary capacity assessment based on these scenarios.

7.4.3 Aeration Basin Capacity Assessment

Table 7-21 summarizes the results of the aeration tank capacity assessment and predicted operating and effluent conditions from the modeling. The detailed modeling results are provided in Appendix D.

Table 7-21: Summary of Aeration Basin Capacity Assessment

Parameter	Unit	S1 Design MML	S2 Design AAL	S3 Maintenance
Aeration Basin Volume, in Service	gallons	828,200	828,200	414,100
Total Design SRT	days	5.0	4.0	4.0
Anaerobic SRT	days	1.0	0.8	0.8
Aerobic SRT	days	4.0	3.2	3.2
Secondary Process HRT	hours	1.5	3.3	1.7
Estimated Observed Yield	lbs MLSS /lbs PE BOD	0.77	0.70	0.70
Max MLSS Concentration	mg/L	2,500	2,500	2,500
PE $CBOD_5$ Load Capacity at Max MLSS	lbs BOD/day	4,500	6,200	3,100
PE $CBOD_5$ Load	lbs BOD/day	9,500	6,400	6,400
PE $CBOD_5$ Concentration	mg/L	87	127	127
Corresponding Design Flow	MGD	6.2	5.8	2.9
Corresponding AAF at Capacity	MGD	2.8(1)	5.8	2.9
If Capacity ≥ Design Load?		No	Maybe	No
Key Outputs from Process Model(2)				
MLSS Concentration	mg/L	2,500	2,500	2,500
Simulated Total SRT	days	2.4	4.0	2.0
Is Modeled SRT ≥ Design SRT?		No	Yes	No
Secondary Process HRT	hours	1.5	3.2	1.6
Total Airflow Demand	scfm	2,000	1,700	1,400
Secondary Effluent CBOD	mg/L	14	12	17
Secondary Effluent Soluble CBOD	mg/L	4	2	4
Secondary Effluent TSS	mg/L	20	24	24
Secondary Effluent NH_3-N	mg/L	18	29	29
Secondary Effluent pH	-	7.0	7.1	7.1
Additional Scenarios	**Unit**	**S1b Design MML**	**S2b Design AAL**	**S3b Maintenance**
Max MLSS Concentration	mg/L	1,300	1,300	1,300
PE $CBOD_5$ Load Capacity at Max MLSS	lbs BOD/day	2,300	3,200	1,600
Corresponding Design Flow	MGD	3.2	3.0	1.5
Corresponding AAF at Capacity	MGD	1.5(1)	3.0	1.5
If Load Capacity at Design Load or Larger?		No	No	No

Notes:

(1) Corresponding AAF = Design MMF/(PF of MMF/AAF)

(2) Simulated for the maximum MLSS allowed.

The following summarizes the aeration basin capacity assessment:

- The existing two aeration basins will be capable of handling the design AAL if the MLSS in the basins can be increased to 2,500 mg/L, but not if MLSS is limited to 1,500 mg/L.

- The existing aeration basins can't handle the design MML or AAL during a maintenance period, even if the MLSS in the process is increased to 2,500 mg/L. The limited $CBOD_5$ treatment capacity was estimated to be 4,500 ppd at the lowest monthly average temperature and an influent MMF of 6.2 MGD, which can occur when the AAF reaches 2.8 MGD.

The assessment shows the typical SRT can't be met under AAL conditions if the MLSS is limited to 1,500 mg/L. In recent years, the aeration basins have been operated around 4 to 5 days SRT under typical load conditions and at 3 days SRT for a month or two at about 1,300 mg/L of MLSS. Therefore, they can be considered already running at treatment capacity, particularly during months with higher monthly average loads. With the plant operation team's efforts, there have been no violations in terms of $CBOD_5$, but it is becoming more challenging to adjust operational parameters to consistently meet permit requirements. Therefore, some earlier action is required to address the capacity shortage.

7.4.4 Aeration Capacity Assessment

To assess the required upgrades for the aeration system, the maximum aeration demands in the existing basins were calculated for both the design MML and PDL conditions. Aeration demand escalates with increasing water temperature, as the dissolved oxygen (DO) saturation concentration diminishes with rising temperatures. Additionally, the demand varies depending on the minimum DO concentration maintained in the aeration basins, which is dictated by the targeted SRT and water temperature. Table 7-22 summarizes the estimated aeration demand for the three selected scenarios summarized in Table 7-21 taking these factors into account.

Table 7-22: Summary of Aeration Demand Assessment

Parameter	Unit	S4 Design MML	S5 Design PDL	S6 Design PDL
Temperature	°C	22	22	17
DO in Aeration Basins	mg/L	2.0	1.0	1.0
Air Demand(1)	scfm	3,690	4,750	2,690
Aeration Capacity of Turbo Blower	cfm/each	2,655 at 7.1 psig, 2 each		
Aeration Capacity of Centrifugal Blower	cfm/each	2,500 at 8.5 psig, 1 each		
Is Existing Aeration Capacity Sufficient?		Maybe if no increase in back pressure		
Diffuser Aeration Capacity(1)	scfm/sq ft	1.43 to 7		
Airflow per Basin(2)	scfm/each	964 to 4,700		

Notes:
(1) Calculations assume a 13 ft submergence with a 2% SOTE per foot.
(2) Estimates from the Aeration System Evaluation, based on the diffuser specifications, assume a 13.5 ft submergence and a total SOTE of 35.6%.

In all three evaluated scenarios, the air demand was less than the capacity of two turbo blowers, assuming a back pressure of 7.1 psig. The plant also has an old centrifugal blower from the 1986 design. The city's aeration system evaluation indicates that supplying 964 scfm of air to one basin results in an air pressure of 6.7 psig at the top of the dropleg. Based on information from the operation team, one turbo blower is typically run at a reduced speed to maintain the

DO demand in the aeration basin, switching to the centrifugal blower during storm periods or for cleaning the diffusers on the aeration tank floor. Generally, the turbo blower will trip if back pressure rises above its capacity, and the centrifugal blower can provide the same amount of air at a higher back pressure.

At the plant, a back pressure rise is expected in the following situations:

- The buried air pipe from the blower building to the aeration basin is experiencing significant air leakage, as evidenced by bubbles observed rising from water puddles during the site visit. This air loss can increase air demand, leading to a rise in back pressure.

- During storm periods, the service blower is often switched from the turbo blower to the centrifugal blower to meet the increased air demand. The rise in water levels in the aeration basin due to increased flow or reduced primary $CBOD_5$ removal can lead to an increase in air demand, resulting in higher back pressure.

- Maintenance for inspecting and cleaning the diffusers is typically recommended once a year. Instead of taking one aeration basin offline, the operation team has used the centrifugal blower to increase airflow to the aeration basin to clean the diffusers occasionally. Operators have reported that using the centrifugal blower to clean the diffusers causes the MLSS in the aeration basin to increase by about 300 mg/L, suggesting that fine grit may be causing diffuser fouling, which becomes another factor increasing back pressure in the air pipes.

- Over time, the oxygen transfer efficiency of the diffusers decreases, leading to an increased airflow requirement to meet the oxygen demand.

Resolving these issues may reduce the back pressure demand and allow the two turbo blowers to run together and meet the high demand. The final recommendations for improvement will depend on the selected upgrades for the aeration basins. Since the basins are already running at or above the estimated capacity, if new basins are added to address the capacity issues, that should be considered in determining the final improvements for air distribution and blower selection.

7.4.5 Secondary Clarifier Capacity Assessment

The capacity of the secondary clarifiers was assessed for a MMF, a maintenance scenario at AAF and a peak flow scenario. The capacity of the secondary clarifier was evaluated by comparing the SOR and SLR with the typical design limits used for an activated sludge system in secondary treatment. Additionally, a state point analysis was performed to assess how the capacity changes, especially when considering the sludge volume index (SVI).

SVI affects the treatment capacity of a secondary clarifier by indicating the settleability of the sludge. A higher SVI suggests poor settling, leading to slower sludge settling rates and potentially overloading the clarifier, reducing its capacity. Conversely, a lower SVI indicates better settling, which can improve the clarifier's capacity to handle higher flow rates or solids

loading without compromising performance. State point analysis helps assess the clarifier capacity by graphically comparing the hydraulic and solids loading rates with the settling characteristics of the sludge, which are influenced by the SVI. A higher SVI indicates poorer sludge settling, shifting the settling curve down and potentially limiting the clarifier's capacity, as it may struggle to separate solids efficiently.

In the last two years, the annual average SVI and the maximum monthly average SVI at the plant were 250 and 440 mL/g, respectively, which are much higher than 175 mL/g assumed in 2008 Rerating Study to determine the capacity of the two secondary clarifiers and therefore the recommended maximum primary effluent pumped flow to the secondary process. The annual average SVI of 250 mL/g was assumed to assess the secondary clarifier capacity, but calculations were developed to confirm the rerated capacity from 2008 rerating study for validation of the calculations. The assessment results for the design conditions and validation run for 2008 Rerating Study are summarized in Table 7-23.

Table 7-23: Summary of Secondary Clarifier Capacity Assessment

Parameter	Unit	S7 Design MM WWF	S8 Design AAF	S9 2008 Rerating Study
Secondary Clarifiers, in Service	each	2	1	2
Total Surface Area	sf	19,007	9,503	19,007
Secondary Effluent Flow	MGD	13.5	6.2	22.8
RAS Flow	MGD	5.7	4.6	5.7
RAS TSS Concentration	mg/L	8,400	5,800	7,500
MLSS in Mixed Liquor	mg/L	2,500	2,500	1,500
SOR	gal/d/sf	710	650	1,200
SLR	ppd/sf	21	24	19
Is SOR within typical?		No	No	Yes
Is SLR within typical?		Yes	Yes	Yes
State point position relative to gravity flux line		Above	Touching	Below

Parameter	Unit	S10 MLSS at 2,500 mg/L		S11 MLSS at 1,300 mg/L	
Secondary Clarifiers, in Service	each	2		1	
Total Surface Area	sf	19,007		9,503	
RAS Flow	MGD	5.7		5.7	
Flow Condition	MGD	MMF	PHF	MMF	PHF
Max Capacity	MGD	10.4	10.4	13.5	24.0
RAS TSS Concentration	mg/L	7,100	7,100	4,400	6,800
SOR	gal/d/sf	550	550	710	1.260
SLR	ppd/sf	18	18	11	17
Is SOR within typical?		Yes	Yes	No	No
Is SLR within typical?		Yes	Yes	Yes	Yes
State point position relative to gravity flux line		Touching	Touching	Below	Touching

The existing two secondary clarifiers are unable to handle the design MM WWF conditions. They can only manage AAF conditions under a maintenance scenario, assuming an MLSS of 2,500 mg/L from the aeration basins and an SVI of 250 mL/g. Lowering the MLSS to 1,300 mg/L allows the clarifiers to handle 13 MGD of MMF and 24 MGD of PHF, respectively. However, reducing MLSS to increase clarifier capacity decreases the treatment capacity of the aeration basins.

The treatment capacity of the secondary clarifiers can be improved through three main strategies: adding new clarifiers, increasing RAS capacity (which has limited impact), or enhancing sludge settleability. Among these, improving sludge settleability is the most viable solution. Despite the 2009 construction of an anaerobic selector intended to improve sludge settleability, recent data indicate that settleability has deteriorated.

The anaerobic selector was designed by adding a divider wall in the aeration basin, directing wastewater from the anaerobic to the aerated zone via a weir wall. A surface-floating mixer circulates DO to the bottom, preventing true anaerobic conditions and promoting bacteria associated with sludge bulking. Additionally, high loading and reduced SRT in the aeration basin hinder the maintenance of anaerobic conditions, further impairing sludge settleability. Research shows that properly functioning anaerobic selectors can reduce SVI to 60–90 mL/g.

The key findings and recommendations are summarized as follows:

- The current secondary clarifiers are insufficient for design MM WWF conditions and depend on MLSS and SVI adjustments for AAF conditions.
- Enhancing sludge settleability is the most practical solution for increasing clarifier capacity.
- Modifications to the existing anaerobic selector should focus on achieving true anaerobic conditions by addressing DO recirculation issues.
- Future aeration basins should incorporate properly functioning anaerobic selectors to enhance sludge settleability and overall performance.

7.5 WWTP Unit Process Capacity

The capacity of the remaining major process units was evaluated through desktop calculations, comparing key design and operational parameters with typical values recommended and ranges from Wastewater Engineering: Treatment and Resource Recovery (5th Edition, 2014, by Metcalf & Eddy), as well as the requirements outlined in the WA Orange Book. The results of this assessment are provided in the following sections.

7.5.1 Headworks Screening

The WWTP currently utilizes three mechanical bar screens and two grit chambers at its headworks facility, thereby meeting the requirements for Reliability Class I according to the Orange Book. The total capacity of the three screens was designed for 48 MGD. However, this capacity must be increased to handle the 2044 PHF of 58.7 MGD to maintain the desired

freeboard at the headworks and prevent overtopping. While increasing capacity is the immediate priority, addressing I/I issues in the service areas could help reduce peak flow impacts over time. However, reducing I/I will require significant time and cost, making capacity improvements the main focus at this stage.

The City is currently evaluating the replacement of two screens that are approaching 30 years of service, having been installed in 1995. The replacement project should also consider increasing the screening capacity or maximizing the capacity within the existing channel size to meet future flow demands effectively.

7.5.2 Grit Removal

The current design capacity of the grit removal system is reported to be 30 MGD under PHF conditions, as outlined in the 2008 Rerating Study. Due to grit accumulation in the anaerobic digesters and discrepancies observed in the plant's hydraulic analysis that may be attributed to grit, the facility should consider either adding additional capacity or replacing the current system with an alternative. For wastewater mixed with CSO, grit production can range from 0.5 to 27 cf/MG, which can be up to five times higher than that of separated sewer systems. Therefore, replacing the existing system with one that offers higher grit removal efficiency for a wider range of particle sizes, even at high flow rates, could be a viable option. The estimated retention time in the existing grit chambers will be reduced further in the design PDF and PHF as summarized in Table 7-24. The cells highlighted indicate processes that surpass the existing design capacity or fall short of the typical design standards as outlined in Metcalf & Eddy or Orange Book.

Table 7-24: Grit Removal Capacity Assessment

Grit Removal Criteria	Capacity		
Type and Number	2 Aerated grit tanks		
Total Volume, cf	10,528		
Flow Condition	30 MGD(1)	2044 PD WWF	2044 PH WWF
Detention Time, minutes	3.8	3.4	1.9
Typical Range in Orange Book	3-5 at PDF		

Notes:
(1) Peak day flow capacity rated in 2008 Study.

7.5.3 Primary Clarifier

According to the requirements of the Orange Book, the remaining primary clarifier must be able to handle 50% of the total design flow when the largest primary clarifier is out of service. Both the SOR under this maintenance scenario at the design MMF and at the design PD WWF will be increased above the acceptable operation range provided in the Orange Book. Therefore, additional primary removal capacity needs to be added. Table 7-25 the primary removal capacity assessment.

Table 7-25: Primary Removal Capacity Assessment

Primary Removal Criteria	Capacity		
Flow Condition	20 MGD(1)	2044 MM WWF	2044 PD WWF
Number in Service, each		2	
Total Surface Area in Service, sf		10,053	
SOR, gal/d/sf	1,990	1,300	3,300
Typical Range in Orange Book		800-1,200 at Average 2,000-3,000 at Peak	

Notes:
(1) Maximum month flow capacity rated in 2008 Study.

7.5.4 Disinfection

According to the requirements of the Orange Book, the remaining chlorine contact chamber must be capable of handling 50% of the total design flow when the other chamber is out of service. Additionally, the contact chamber must be sized to provide a minimum detention time of 1 hour at the average design flow or 20 minutes at the peak daily flow, whichever is greater. Redundancy is also required for chlorine feed equipment.

The WWTP disinfection system meets the redundancy requirements. However, additional capacity is necessary to achieve the required detention time under peak flow conditions. Furthermore, the chemical feed pumping capacity must be expanded to meet the maximum daily demand estimated for the design PD WWF. The storage duration will be maintained at more than 2 weeks under AAF conditions but reduced to about 2 weeks under MM WWF conditions. Table 7-26 the disinfection system capacity assessment.

Table 7-26: Disinfection System Capacity Assessment

Disinfection Criteria	Capacity		
Chlorine Contact Chamber Assessment			
Flow Condition	30 MGD(1)	2044 AAF	2044 PD WWF
CCC Detention Time, min	20	101	18
Min Required in Orange Book	20	60	20
Chemical Feed System Assessment			
Flow Condition	2044 AAF	2044 MM WWF	2044 PD WWF
Chlorine Dose Assumed, mg/L	4(2)	4(2)	8.6(2,3)
Chlorine Demand, ppd	210	450	2,380
Chlorine Demand, gpd	200	430	2,290
Current Max Injection Pump Rate, gpd		760	
Demand Assumed for Odor Control System, gpd		350(4)	
Active Chlorine Storage Tank Volume Assumed, gallons		9,920(5)	
Storage Duration, days	18	13	NA
$NaHSO_3$:Cl_2 Assumed		1.6(6)	
$NaHSO_3$ Demand, ppd	330	720	3,830
$NaHSO_3$ Demand, gpd	80	180	930

Disinfection Criteria	Capacity		
Current Max Injection Pump Rate, gpd		204	
Active $NaHSO_3$ Storage Tank Volume Assumed, gallons		2,400(5)	
Storage Duration, days	30	14	NA

Notes:

(1) Peak day flow capacity rated in 2018 Study

(2) The typical chlorine dosage for secondary effluent, as stated in the Orange Book, ranges from 2 to 8 mg/L. Therefore, a dosage of 4 mg/L is assumed for estimating chlorine requirements under AAF and MMF conditions.

(3) Under the design PD WWF, only 22.8 MGD is assumed to flow to the aeration basins, while the remaining 11.1 MGD bypasses the secondary process. According to the Orange Book, the typical chlorine dosage for primary effluent ranges from 5 to 10 mg/L. The chlorine dosage for the combined flow of 22.8 MGD of secondary effluent and 11.1 MGD of primary effluent is estimated by assuming a dosage of 8 mg/L for the primary effluent.

(4) The plant typically orders 5,200 gallons of sodium hypochlorite solution every 10 days. The chlorine demand for the odor control system is estimated by subtracting the chlorine demand required for the disinfection of the historical AAF of 5.16 MGD.

(5) 80% of the total tank volume is assumed to be utilized as active storage volume.

(6) Assumed to be 10% greater than the theoretical does of 1.46 mg/L required to neutralize 1 mg/L of chlorine.

7.5.5 Solids Stream Handing Processes

The Orange Book does not prescribe specific redundancy requirements for solids stream processes, but all pumps that perform separate functions must meet the pumping redundancy requirements. The WWTP utilizes two pieces of equipment for the primary sludge pumps, WAS pumps, RDTs, and TWAS pumps, which meet the redundancy requirements. The current capacity of this equipment is sufficient to handle the anticipated production under the design loads, as summarized in Table 7-27.

Table 7-27: Solids Stream Handling System Capacity Assessment

Solids Stream Criteria	Capacity	
Load Conditions	2044 MML	2044 AAL
Primary TSS Removal Assumed, %	46%(1)	59%(1)
Primary Sludge TSS, ppd	9,400	8,800
Primary Sludge Flow, gpd	22,400(2)	21,000(2)
Primary Sludge Pump Capacity, gpd/each	50,400	
WAS TSS Production Estimated, ppd	7,500(3)	4,600(3)
WAS Flow, gpd	128,000(4)	78,000(4)
WAS Pump Capacity, gpd/each	324,000	
RDT Flow Capacity, gpd/each	324,000	
TWAS TSS Production Estimated, ppd	7,000(5)	4,200(5)
TWAS Flow, gpd	16,700(6)	10,200(6)
TWAS Pump Capacity, gpd/each	53,200	

Notes:

(1) Historical primary TSS removal rates were analyzed in relation to hydraulic retention time, and the assumed TSS removal rates were determined using the regression line derived from this correlation.

(2) The primary TSS concentration is assumed to be 5%.

(3) The estimate is calculated as the product of PE $CBOD_5$ load and the observed yield from Table 7-21.
(4) The WAS TSS concentration is assumed to be 7,000 mg/L.
(5) The TSS capture rated is assumed to be 93%.
(6) The TWAS TSS concentration is assumed to be 5%.

7.5.6 Anaerobic Digestion

The anaerobic digestion capacity is evaluated using typical design parameters from Metcalf & Eddy, as the Orange Book does not specify requirements for this process. The anaerobic digesters appear capable of handling sludge production under 2044 loading conditions, with the HRT remaining above 15 days and the volatile solids (VS) loading rate staying below the typical limits recommended for mesophilic digestion, as summarized in Table 7-28. While the existing centrifuge capacity is adequate to process the digested sludge, adding a redundant unit is recommended to enhance process reliability.

Table 7-28: Anaerobic Digestion Process Capacity Assessment

Digestion Process Criteria	Capacity	
Load Conditions	2044 MML	2044 AAL
Primary Sludge + TWAS Flow, gpd	39,100	31,200
HRT, days	20	25
Typical HRT, days	15	
Primary Sludge VS Load, ppd	6,600(1)	6,100(1)
TWAS VS Load, ppd	5,200(2)	3,400(2)
VS Loading Rate, ppd/cu.ft	0.11	0.09
Typical VS Loading Rate, ppd/cu.ft	0.15 @ 5% TS and 15 days HRT	
VS Destruction Rate	56%(3)	
Digested Sludge TS, ppd	9,700	7,700
Centrifuge Feed Flow, gpd	39,100	31,200
Centrifuge & Feed Pump Capacity, gpd	288,800	
Dewatered Biosolids TS, ppd	8,000(4)	6,300(4)

Notes:
(1) The VS to TS ratio in the primary sludge is assumed to be 0.7.
(2) The VS to TS ratio in the TWAS is assumed to match the values estimated for WAS sludge from the process modeling: 0.82 at MML and 0.79 at AAL.
(3) Typical VS destruction rate at the typical VS roading rate at 15 days of SRT, according to Metcalf & Eddy.
(4) The current TS capture rate is assumed to be 82% per process calibration modeling.