CHAPTER 7

WASTEWATER TREATMENT PLANT ANALYSIS

INTRODUCTION

This chapter provides a description and analysis of the City of Woodland Wastewater Treatment Plant (WWTP). The WWTP's ability to meet current and potential future NPDES permit discharge limits are evaluated at future flows and loadings derived in Chapter 5. Other evaluations presented in this chapter include an energy efficiency analysis to identify possible energy saving improvements for the WWTP; an analysis of the aerobic digester for its long-term ability to stabilize waste solids; and the potential of adding a solids dewatering system to reduce sludge disposal costs.

Wastewater from the City of Woodland is treated at the City's WWTP, which was upgraded in 2002. Prior to the 2002 upgrade the WWTP consisted of a headworks with screening and grit removal; one primary clarifier; one submerged biological contactor (SBC) and two rotating biological contactor (RBC) units; one secondary clarifier; a chlorine contact tank for disinfection; and an aerobic digester and sludge drying beds for solids treatment.

The upgrades constructed in 2002 replaced the RBC/SBC treatment system with a new sequencing batch reactor (SBR) treatment system, along with a new headworks equipped with an automated self-cleaning fine screen and an automated grit removal system. An ultraviolet (UV) light disinfection system was added to replace the chlorine gas disinfection system. A package aerobic digester with gravity thickener was constructed to replace the aerobic digester and drying beds.

Treated effluent is discharged into the Lewis River through a submerged outfall pipe. Figure 7-1 shows the hydraulic profile and process schematic for the existing WWTP. Figure 7-2 shows the WWTP layout.

The WWTP NPDES Permit includes effluent limits for BOD_5, TSS, and fecal coliform for treated effluent discharged into the Lewis River. These limits are presented in Table 7-1. The WWTP NPDES Permit also establishes loading limits for the maximum month flow, peak instantaneous design flow and BOD_5 influent loading for maximum month and TSS influent loading for the maximum month, which are also included in Table 7-1.

TABLE 7-1

Woodland WWTP NPDES Permit Effluent and Loading Limits

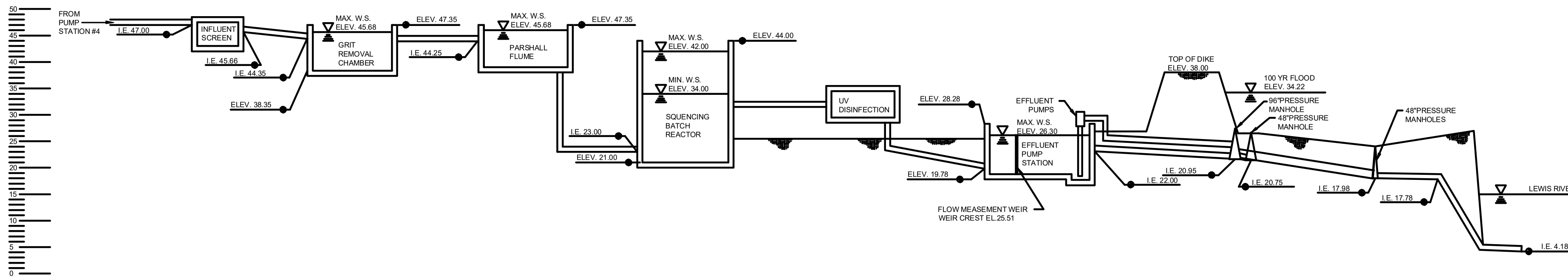
Parameter	Monthly Average(1)	Weekly Concentration
Effluent BOD_5	30 mg/L	30 mg/L
Effluent TSS	30 mg/L	30 mg/L
Effluent Fecal Coliform	200/100 mL	400/100 mL
Maximum Month Flow	2.0 mgd	
Peak Instantaneous Flow	3.2 mgd	
Maximum Month BOD_5 Loading	3,107 lb/day	
Maximum Month TSS Loading	3,160 lb/day	

(1) Monthly average effluent concentrations shall not exceed the limit listed in the table, or 15% of the respective influent concentrations, whichever is more stringent.

The existing WWTP utilizes a three basin sequencing batch reactor (SBR) treatment system for biological treatment. The normal flow pattern of the liquid treatment process includes a headworks with an automated fine screen, a manual bar screen, a grit removal chamber, three SBR basins, waste activated sludge pumping system, an ultraviolet light disinfection system, an effluent pumping system, a non-potable washwater pump and distribution system, and the outfall structure to the Lewis River.

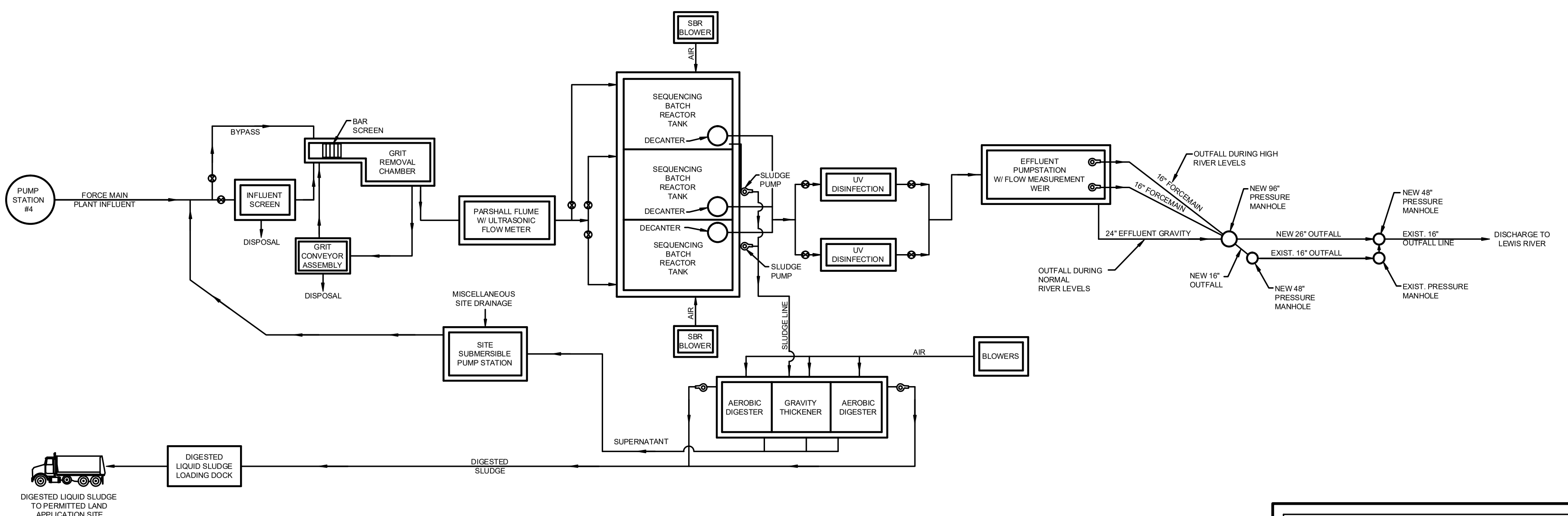
The solids treatment and handling system consists of a prethickened aerobic digester (PAD) system that consists of a premix tank, two aerobic digester tanks and gravity thickener. Liquid digested sludge is pumped from the PAD to a truck loading station with submersible centrifugal pumps. Airlift pumps, one for each digester cell, are used to transfer sludge from the digester to the thickening unit. Two airlift scum pumps are used to transfer scum from the thickener to the digesters.

Table 7-2 presents the existing design data for the WWTP.



WWTP HYDRAULIC PROFILE

NOT TO SCALE



WWTP FLOW SCHEMATIC

NOT TO SCALE

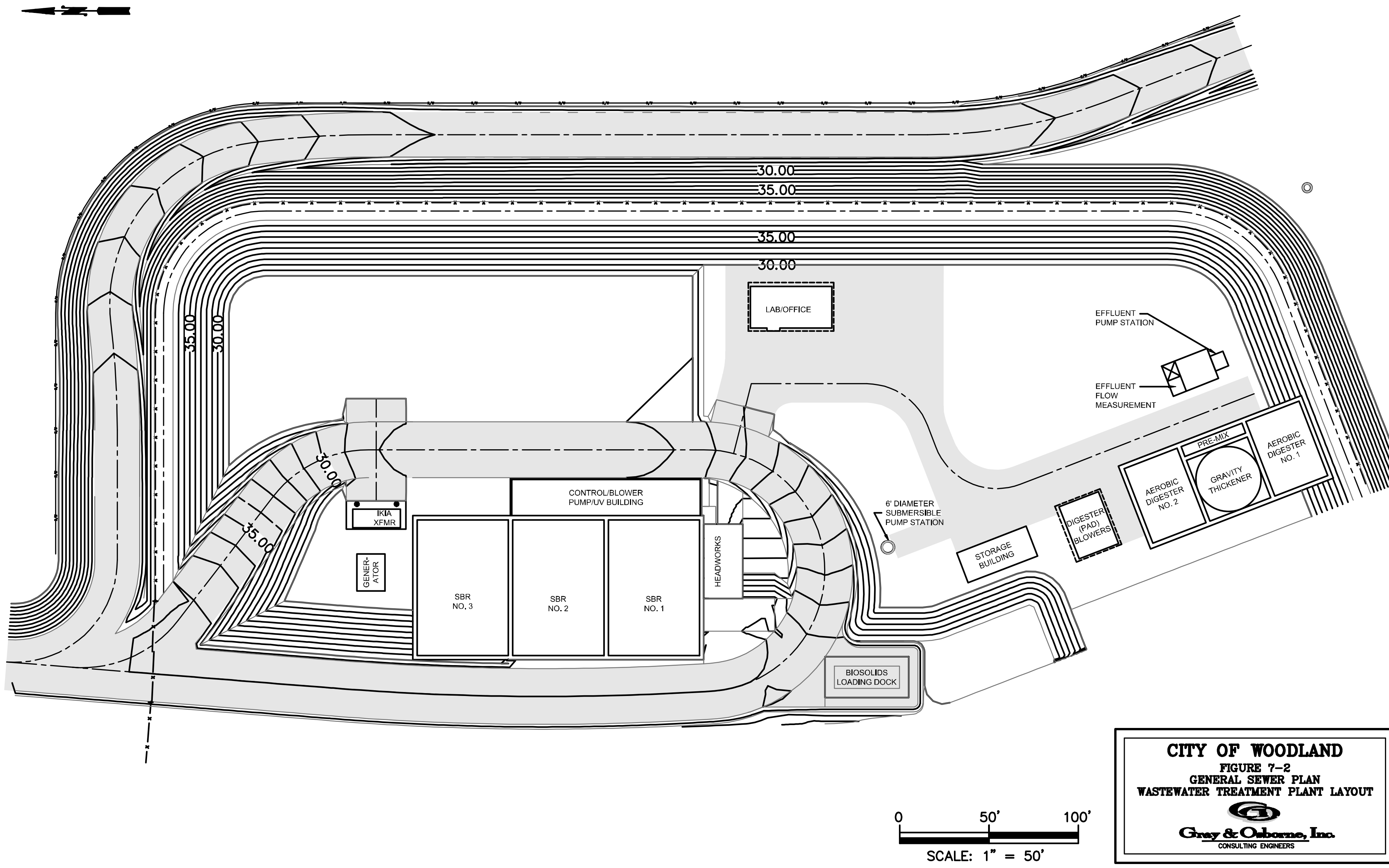
CITY OF WOODLAND

FIGURE 7-1

GENERAL SEWER PLAN

WASTEWATER TREATMENT PLANT HYDRAULIC PROFILE AND PROCESS FLOW SCHEMATIC

Gray & Osborne, Inc.
CONSULTING ENGINEERS



CITY OF WOODLAND

FIGURE 7-2

GENERAL SEWER PLAN

WASTEWATER TREATMENT PLANT LAYOUT

Gray & Osborne, Inc.

CONSULTING ENGINEERS

TABLE 7-2

Design Data for Woodland Wastewater Treatment Plant

Design Parameter	Two-Basin Operation	Three-Basin Operation
Liquid Treatment Process		
Influent Flow Measurement		
Type	Parshall Flume with Ultrasonic Level Sensor	
Quantity	1	
Throat Width	9"	
Influent Screen		
Type	1/4-inch Automated Self-Cleaning	
Quantity	1	
Grit Removal		
Type	cyclonic grit removal chamber with grit concentrator	
Quantity	1	
Grit Pump Type	vertical centrifugal turbo grit pump	
Grit Pump	250 gpm at 34 ft TDH; 10 hp	
Dewatering grit screw conveyor	250 gpm; 1 hp	
Sequencing Batch Reactor (SBR)		
Quantity of Basins	2	3
Length (each basin)	73'	
Width (each basin)	49'	
Wall Height	23'	
Minimum Water Level	13'	
Maximum Water Level	21'	
Minimum Volume (each basin)	347,827 gallons	
Variable Volume (each basin)	214,048 gallons	
Maximum Volume (each basin)	561,875 gallons	
Total Volume	1,123,750 gallons	1,685,625 gallons
Blowers	4 - 50 hp; 625 scfm at 10 psi each (1 standby) 2 - 75 hp; 937 scfm at 10 psi each	
Total Aeration Supply	4,374 scfm (3,749 scfm w/ 1 – 50 hp blower out of service)	
Blower Usage	4 - 50 hp units (1 unit for redundancy)	4 - 50 hp units and 2 - 75 hp units (1 unit for redundancy)
Aeration Diffuser System	medium bubble	
SBR Mixers	3 - 20 hp floating turbine mixers (1 per basin)	
Decanter Type	Floating decanter with electric control valve	
Average Decant Rate	3,333 gpm (4.8 mgd)	
Waste Activated Sludge Pump	2 - 3 hp centrifugal pumps (1 duty, 1 standby)	
Treatment Plant Process Control	programmable PLC	
Quantity of Treatment Cycles/Day	5	5
Length of Each Treatment Cycle	4.8 hours	
Ultraviolet Disinfection Units		
Quantity	2	
Type	in-vessel, high intensity, medium pressure lamps	
Quantity of Lamps (per unit)	6	
Maximum Energy Required (per unit)	27 kW	

TABLE 7-2 – (continued)

Design Data for Woodland Wastewater Treatment Plant

Design Parameter	Two-Basin Operation	Three-Basin Operation
Liquid Treatment Process – (continued)		
Effluent Flow Measurement	Flat Weir and Ultrasonic Level Sensor	
Effluent Pumps (Used During High River Conditions)		
Quantity	2 (1 duty, 1 standby)	
Type	vertical turbine	
Motor Size	50 hp	
Capacity	4.8 mgd @ 39' TDH	
Non-Potable Water Pumps		
Quantity	1	
Type	6-stage vertical turbine	
Capacity	30 gpm @ 160' TDH	
Solids Treatment Process (Pre-Thickened Aerobic Digestion (PAD))		
Pre-Mix Basin		
Quantity	1	
Dimensions	5.5' x 35' x 22' max depth	
Volume	32,660 gal	
Gravity Thickener Basin		
Quantity	1	
Size	35' diameter	
Sidewater Depth	16.5'	
Digester Basin Dimensions		
Quantity	2	
Dimensions	30.5' x 43' x 24' max depth each	
Volume	235,440 gal each	
Total Volume	470,880 gal	
Aeration Blowers	3 - 50 hp units (2 duty, 1 standby); 2 – 25 hp units	
Blower Capacity	3 - 650 scfm @ 8 psi each 2- 325 scfm @ 8 psi each	
Aeration Diffuser Type	Medium bubble with shear tubes	
Scum Airlift Pumps	4", Qty. 2 (1 per basin)	
Thickened Sludge Airlift Pumps	6", Qty. 2 (1 per basin)	
Digester Basin Decanter Type	Telescoping Valve, Qty. 2 (1 per basin)	
Digester Sludge Transfer Pumps Type	2 – 5 hp submersible centrifugal pumps (1 per basin)	
Digester Sludge Transfer Pumps Capacity	400 gpm @ 30 feet TDH	

INFLUENT PUMP STATION

The influent pump station, Pump Station 4, discharges raw wastewater into the headworks of the WWTP. The pump station includes one wet well and three raw sewage pumps that are designed to raise the sewage to a level which allows the wastewater to flow by gravity through the treatment system. The influent pump station has a design capacity of 1.22 million gallons per day (mgd) or 850 gpm with two of three pumps running. The pump operation is controlled by wet well liquid level.

WASTEWATER TREATMENT UNIT PROCESS DESCRIPTION

The following descriptions of the wastewater treatment unit processes represent the current plant operation based on a physical survey of the treatment plant, review of as-built drawings, equipment manuals and discussions with the operator.

PRELIMINARY TREATMENT

The headworks is designed for screening and degritting as well as influent flow measurement and sampling. Raw sewage first enters the WWTP through a steel box that houses an automated self-cleaning fine screen, manufactured by Hycor, to remove large debris. The influent screen has a screenings basket 1/4-inch-diameter perforated openings. The screen consists of a screw conveyor which is designed to collect the large debris from the screenings basket and move it to the compactor to be dewatered. The screen compactor dewaters the captured debris and places it in a dumpster for landfill disposal.

Screened influent then flows through a short pipe into a concrete channel, which contains a manual bar screen with 1/2-inch openings to capture any solids not captured by the automated screen. There is also a bypass pipe directly from the pump station force main to the manual bar screen channel, for use if the automatic screen is out of service.

The concrete channel next leads the influent into a grit removal chamber. The grit removal chamber uses a forced vortex to settle out dense debris by gravity, which is removed from the bottom of the chamber periodically by a pump. Organic matter is kept in suspension by an axial propeller in the grit chamber which is manually controlled.

Centered below the grit chamber is a 3-foot-diameter, 5-foot-deep, grit storage hopper. A constant-speed vertical centrifugal turbo grit pump periodically lifts grit from the hopper to be transferred to the grit concentrator and then the dewatering conveyer. The grit pump is designed to handle 250 gpm at 34 feet TDH (total dynamic head). The grit concentrator is rated for 250 gpm and removes heavier grit from the screened wastewater. This grit then is discharged to the dewatering grit conveyor and dewaters the concentrated grit and discharges it to a container for temporary storage.

Screened and degritted wastewater flows through a Parshall flume flow meter to measure flow before going to the secondary treatment process. A composite sampler collects influent samples.

SECONDARY TREATMENT

The secondary treatment process consists of a three-basin sequencing batch reactor (SBR). Screened and degritted wastewater enters one of the three 562,000-gallon rectangular concrete basins that utilize activated sludge for biological treatment.

The basins are connected by three overflow pipes in their common walls to prevent overflow during extreme circumstances. The basins were designed for mixed liquor concentrations ranging from 2,000 to 4,500 mg/L MLSS (mixed liquor suspended solids). At current flows and loadings, only two SBR basins are utilized and the operator maintains the MLSS concentration between 2,500 and 3,000 mg/L in the summer, and 2,700 to 2,900 mg/L in the winter.

In the two-SBR basin mode currently in use, the wastewater undergoes five phases within the SBR basin four times per day. First a Mixed Fill phase, which lasts 75 minutes, wherein the preliminary effluent enters the chambers while the basin is mixed without introducing oxygen to create an anoxic environment to encourage the activated sludge to consume the carbonaceous waste as well as denitrify. Next, the chamber undergoes 105 minutes of React Fill phase where effluent from the grit removal chamber continues to flow in and air is introduced through diffusers in the bottom of the basins to create aerobic conditions. 75 minutes of the React phase follow with continued aeration while the fill is directed to the other SBR tank. These phases are followed by 55 minutes of settling where both mixing and aeration is discontinued. During this stage activated sludge settles and thickens at the bottom of the basin and clarified water forms on the surface. Next comes a 50-minute decanting phase. In this phase, the clarified wastewater is removed from the basin through a decanter while waste sludge is removed with waste sludge pumps. The effluent from the SBRs then flows by gravity to the disinfection system.

EFFLUENT DISINFECTION

Two in-vessel medium-pressure UV-disinfection units, manufactured by Aquionics, disinfect the clarified effluent from the SBRs. Each unit was designed for a minimum UV transmittance of 60 percent. The disinfected effluent then flows to the effluent pump station through an effluent flow measuring system upstream of the pump station.

EFFLUENT FLOW MEASUREMENT AND EFFLUENT PUMP STATION

The effluent pump station includes one wet well, a wet well extension, and two effluent pumps (one duty, one standby). Effluent from the UV disinfection units flows through a wet well extension into the wet well and is then discharged to the Lewis River in one of two ways: by a gravity main when the river level is low, or through the effluent pump to force mains when the river level is high. The effluent pumps each have a design capacity of 3,333 gpm, which matches the maximum decant flow rate. Effluent flow is measured upstream of the pump station in the wet well extension by a 4-foot rectangular weir and ultrasonic flow meter. A composite sampler collects effluent samples from the wet well extension ahead of the weir.

NON-POTABLE WATER SYSTEM

The WWTP has a non-potable water system composed of a pump and a distribution system. The variable speed vertical turbine six stage pump is located in the sampling building adjacent to the effluent pump station wet well and is rated for 30 gpm at 160 feet of TDH. Effluent from the treatment plant is pumped through the distribution system to five hose bibs by the SBRs and several hydrants around the plant, as well as two spray washes (one by the gravity thickener and one by the influent screen at the headworks).

SOLIDS TREATMENT AND HANDLING

The purpose of the solids treatment and handling system is to thicken and stabilize waste activated sludge (WAS) from the SBR basins so it can be hauled away and beneficially used as a fertilizer via land application on agricultural land. The solids treatment and handling system consists of a premixing basin, a 35-feet-diameter gravity thickener, and two 235,440-gallon aerobic digesters.

Stabilization of the solid waste happens in two phases. In Phase 1, the pre-mixing basin, the gravity thickener and Digester 1 operate in a loop. The biosolids are constantly circulated through the three chambers, with the goal of increasing the solids concentration to 3 to 3.5 percent solids. During this time, Digester 2 is isolated and continuously aerated and mixed. Phase two is similar, with Digester 2 acting as the active digester and Digester 1 as the isolated digester. At capacity, each phase is intended to be a minimum of 20 days, giving the solids a total stabilization time of 40 days so that the digester operation can meet the criteria for Class B pathogen reduction using the "time-temperature" method of 40 days mean cell residence time (MCRT) at a minimum temperature of 20 degrees C.

The digester aeration system consists of five constant speed rotary lobe positive displacement digester blowers. Three of the blowers are 50-hp, rated for 650 scfm at 8 psig; the other two are 25-hp rated at 325 scfm at 8 psig. Each digester is also equipped with a constant speed submersible centrifugal digested sludge pump rated for 400 gpm at 30 feet of TDH. The pumps draw liquid sludge from the digester and discharge to the truck loading station from which they will be hauled to a permitted land application site.

CURRENT WWTP LOADINGS AND PERFORMANCE

Figures 5-2 and 5-3 showed the historical influent BOD_5 and TSS loadings to the WWTP from January 2009 to January 2014. As shown in Table 5-3 historical BOD_5 and TSS loadings to the WWTP have averaged 1,144 lb/day and 1,407 lb/day, respectively, between 2009 and 2013, with maximum month loadings averaging 1,378 for BOD_5 and 1,744 for TSS over that same period. The average BOD_5 and TSS loadings to the plant during this period represent 44 percent and 55 percent of the permitted BOD_5 and TSS maximum month loadings of 3,107 lb/day and 3,160 lb/day, respectively.

The highest maximum month BOD_5 loading was 1,703 lb/day in November 2009 and the highest TSS loading was 2,078 lb/day in November 2009. These represent 54 percent and 65 percent of the respective permitted BOD_5 and TSS loadings for the WWTP.

Figures 7-3 and 7-4 show historical BOD_5 and TSS effluent concentrations and removal efficiencies between 2009 and early 2014. As these figures indicate, the plant has not exceeded effluent limits for BOD_5 and TSS and has consistently met the 85 percent removal requirement for both BOD_5 and TSS.

Although there is no effluent limit for ammonia, the WWTP effluent is monitored for ammonia removal. Figure 7-5 shows monthly average ammonia concentrations for the period January 2009 to April 2014. With the exception of May 2012 when effluent ammonia was measured as 2.74 mg/L, effluent ammonia concentrations have been well under 1 mg/L indicating that complete nitrification is occurring in the SBRs.

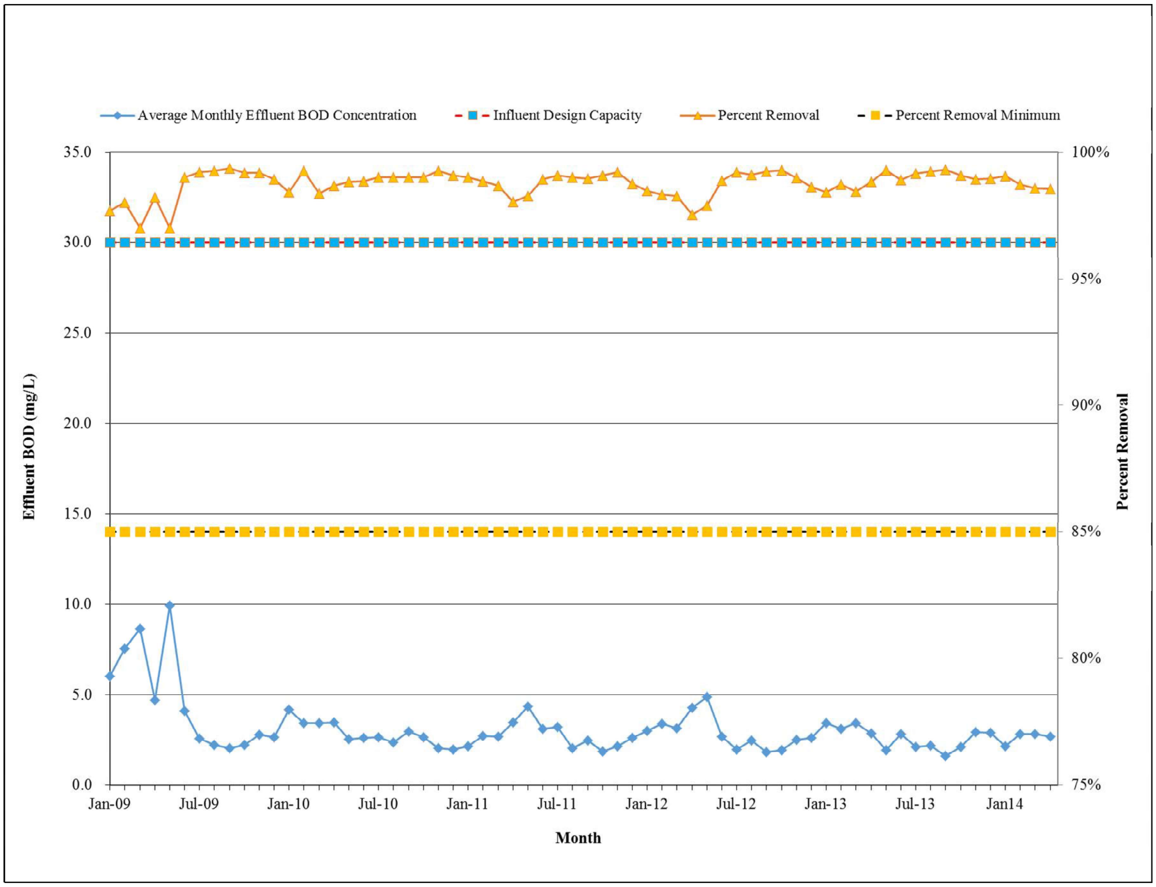


FIGURE 7-3

Effluent BOD Concentrations and BOD Removal Efficiency (January 2009 to April 2014)

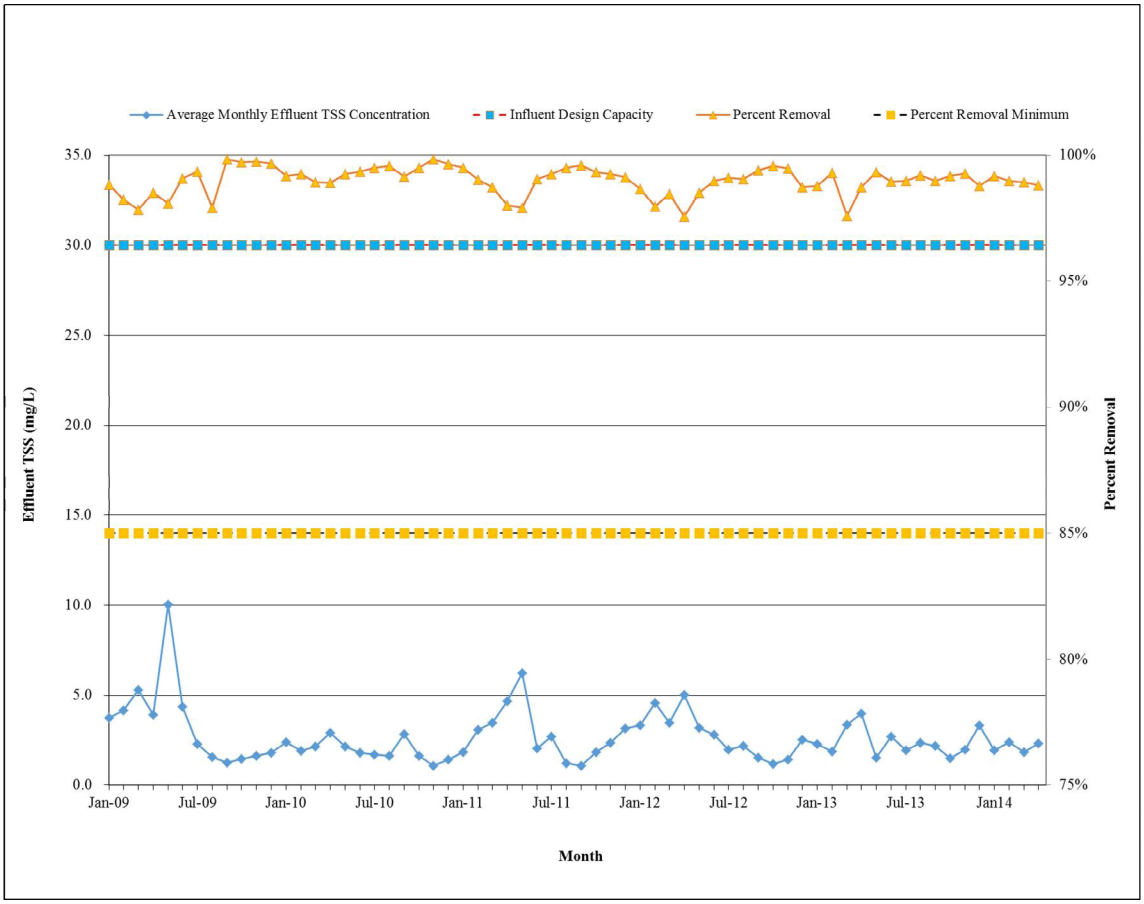


FIGURE 7-4

Effluent TSS Concentrations and TSS Removal Efficiency (January 2009 to April 2014)

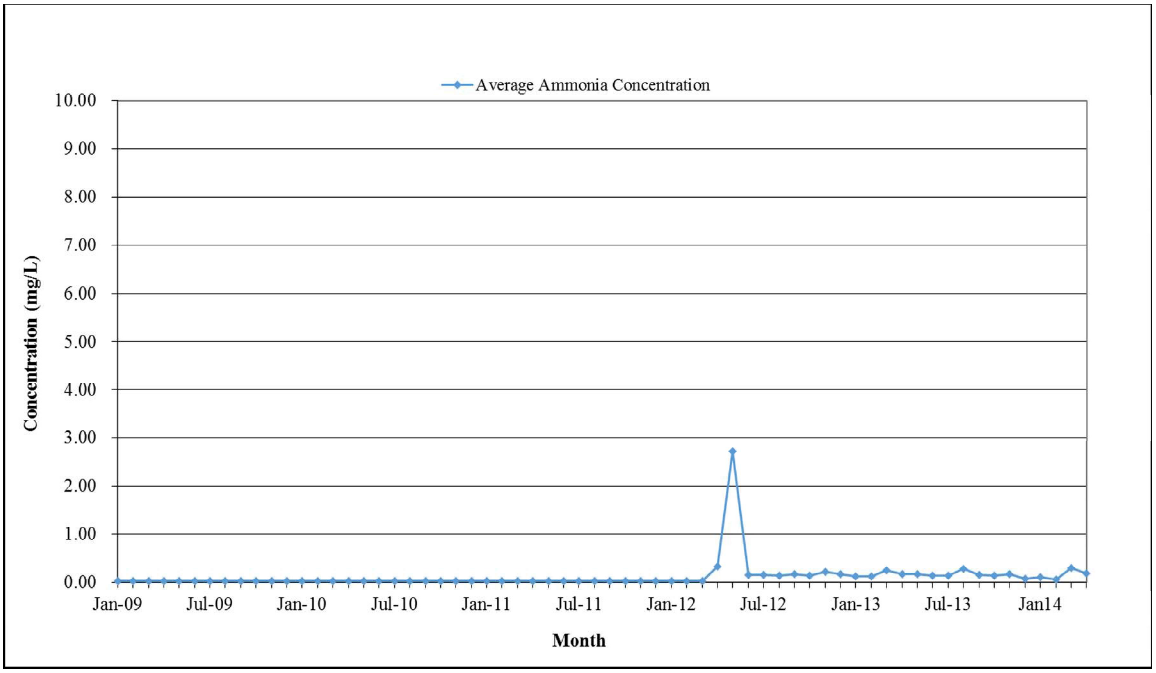


FIGURE 7-5

Effluent Ammonia Concentrations (January 2009 to April 2014)

EVALUATION OF POTENTIAL FUTURE PERMIT LIMITS

As noted previously in the Plan, the quality of effluent discharged by the City's WWTP is regulated by the City's National Pollutant Discharge Elimination System (NPDES) permit, issued by Ecology.

The NPDES permit includes technology-based and water quality-based numeric effluent limitations, implemented to meet current standards and protect the receiving water. One of the steps to updating the NPDES permit is mixing zone modeling that provides a factor of dilution of the effluent from the WWTP into a receiving water body (Lewis River). Based on this dilution factor and a number of other variables including effluent and receiving water conditions, one can predict whether or not pollutants within the effluent will have a reasonable potential to cause an exceedance of water quality standards (for aquatic biota and human health) in the receiving water.

A mixing zone study was completed for the Woodland WWTP in 1999 (and updated in 2005 and 2011) that calculated dilution factors based on effluent flows that were projected for the WWTP.

The Washington Department of Ecology (Ecology) has required re-evaluation of this reasonable potential to exceed water quality standards as a condition of approval of the General Sewer Plan. A copy of this Effluent Permit Limit Evaluation (Evaluation) is provided in Appendix K. The Evaluation examined if a reasonable potential exists for the City WWTP effluent to cause an exceedance of water quality standards, and trigger a need for new permit limits or to implement plant capital or operating improvements.

The Evaluation concluded that no reasonable potential to cause an exceedance of water quality standards exists for the pollutants considered (ammonia and common toxic metals). A sensitivity analysis was conducted to determine at what (95^{th} percentile) effluent concentration reasonable potentials exist and permit limits would be triggered. These concentrations are 27 µg/L for copper, 214 µg/L for zinc, and 24,500 µg/L (24.5 mg/L) for ammonia-nitrogen. For ammonia, the WWTP should continue to be operated with a sufficient SRT for complete nitrification, in order to avoid a reasonable potential for exceedance. The capacity of the SBRs for nitrification is evaluated later in this chapter. As there are no known industrial dischargers of elevated (non-domestic) levels of copper and zinc, it is expected that the copper is coming from routine corrosion in household plumbing, similar to what was observed in studies in Puyallup and Sumner (see Appendix K). As noted in Appendix K, if effluent metals concentrations increase, the City could evaluate alternatives including: (1) chemical precipitation at the WWTP; (2) drinking water pH adjustment; and (3) site-specific metals criteria development through a Water Effects Ratio/Biotic Ligand Model evaluation.

Discussions with the Department of Ecology during review of the *General Sewer Plan* indicate that Ecology will require that the mixing zone be updated prior to renewal of the NPDES permit, which expired March 31, 2017. It is recommended that the City budget for a mixing zone study that includes in-river dye testing to verify dilution modeling assumptions and results. The cost of such a study is estimated to be $75,000.

PROJECTED WWTP FLOWS AND LOADINGS AND EXPECTED PERFORMANCE

Recent WWTP BOD_5 and TSS removal performance has been consistently excellent, as the BOD_5 and TSS loadings have averaged 43.6 percent and 55.2 percent of the permitted BOD_5 and TSS loadings respectively.

In following sections the performance of the WWTP will be evaluated at the projected 2033 flow and loading rates developed in Chapter 5, and summarized in Table 7-3. The maximum capacities of each WWTP process for flow and/or loading are also determined. For the hydraulic analysis, the intermittent in-plant recycle flow of 350 gpm has been added to the influent peak hour flow. The projected BOD_5 and TSS loadings for year 2033 are 72 percent and 82 percent, respectively, of the current permitted capacity for the WWTP. However, that rated capacity did not account for nitrification, which will be evaluated in this section.

TABLE 7-3

Projected 2033 Flow and Loading Rates for Woodland Wastewater Treatment Plant

Parameter	2033
Influent Flow Rates	
Average Annual Flow	0.91
Maximum Month Flow	0.96
Peak Day Flow	1.06
Peak Hour Flow	1.72
Influent Loading Rates	
Annual Average BOD_5, (lb/d)	2,016
Maximum Month BOD_5, (lb/d)	2,237
Annual Average TSS, (lb/d)	2,135
Maximum Month TSS, (lb/d)	2,602
Annual Average TKN, (lb/d)	373
Maximum Month TKN, (lb/d)	405

The 2033 loading rates, where applicable, are compared to equipment data and accepted design criteria, such as published in the Washington State Department of Ecology (Ecology) "Orange Book," WEF *Manual of Practice No. 8* (MOP 8, 2010) and Metcalf & Eddy *Wastewater Engineering* (5th Edition, 2014), to determine if capacity is sufficient for the 2033 loading rates and define the maximum capacity. This evaluation is summarized in Table 7-4. The following text provides a more detailed analysis of each component and the applicable criteria.

TABLE 7-4

Comparison of Component Design Capacity/Criteria and Projected Condition

Component (Parameter)	Recommended Criteria/Capacity	Reference[1]	2033 Operating Condition[2] (Criteria Met?)	Maximum Capacity
Mechanical Fine Screen (Capacity)	Minimum 12-inch freeboard in front of fine screen	Manufacturer head loss curves	1.72 mgd peak hour (yes)	2.7 mgd influent peak hour
Grit Removal System	4.0 mgd peak hour design	Manufacturer	1.72 mgd peak hour (yes)	3.5 mgd influent peak hour
Parshall Flume	Minimum 12-inch freeboard in front of flume	Manufacturer head curves	1.72 mgd peak hour (yes)	3.2 mgd influent peak hour
SBR[4] (Min. Aerobic Solids Retention Time (SRT) for nitrification)	8.4 days	Design calculations per M&E, 2014	10.8 d (yes)	2,738 lb/day BOD_5 at 8.4 d
SBR[4] (F/M Ratio)	≤0.10 lb BOD_5/lb MLVSS/day	Ecology, 2008	0.08 (yes)	2,738 lb/day BOD_5 at 0.10
SBR[4] (BOD_5 Mass Loading Rate)	≤15 lb/1,000 ft^3/day	Ecology, 2008	11 (yes)	2,650 lb/day BOD_5 at 15

TABLE 7-4 – (continued)

Comparison of Component Design Capacity/Criteria and Projected Condition

Component (Parameter)	Recommended Criteria/Capacity	Reference(1)	2033 Operating Condition(2) (Criteria Met?)	Maximum Capacity
SBR(4) Peak Day Flow Capacity	Without changing cycle times or exceeding 33 percent decant depth, pass 75 percent of peak day flow with one basin out of service.	Ecology, 2008	14% decant depth (yes)	2.47 mgd peak day flow
SBR(4) (Aeration Capacity)	187 lb O_2/hr./basin Peak Actual Oxygen Requirement (AOR)	Blower manufacturer/design calculations	162 lb O_2/hr. (yes)	180 lb O_2/hr. @ 2,650 lb/day BOD_5 loading
UV Disinfection System (Hydraulic Capacity)	6.3 mgd/reactor (peak hour)	Manufacturer	4.8 mgd (yes)	6.3 mgd
UV Disinfection System (Disinfection Capacity)	4.8 mgd (decant rate, with one reactor out of service)	Manufacturer	4.8 mgd (yes)	4.8 mgd

(1) References: Washington State Department of Ecology Criteria for Sewage Works Design ("Orange Book"), 2008; Metcalf & Eddy *Wastewater Engineering* (5th Edition, 2014.

(2) Evaluated at projected 2033 flow and loading rates (Table 7-3).

(3) Hydraulic influent capacity accounts for intermittent in-plant recycle flow of 350 gpm (0.5 mgd).

(4) Refer to Table 7-7 for SBR evaluation.

HEADWORKS CAPACITY EVALUATION

The existing headworks processes were evaluated for peak hydraulic capacity based on manufacturer-provided head loss information, with a criteria of maintaining at least 12 inches of freeboard between the maximum water level and the top of wall. The headworks has capacity for the projected 2033 peak hour flow of 1.72 mgd in addition to the intermittent in-plant recycle flow of 0.5 mgd.

The mechanical fine screen is the capacity limiting process, with an influent peak hour flow capacity of 2.70 mgd, which is less than the current design peak flow of 3.20 mgd. A peak hour flow capacity of 3.20 mgd could be provided by rerouting the in-plant recycle to downstream of the mechanical fine screen. The in-plant recycle pumps could be rerouted to discharge directly to the secondary manual bar screen downstream of the fine screen. The remainder of the headworks facilities can handle an influent peak hour flow of 3.20 mgd plus the in-plant recycle.

SEQUENCING BATCH REACTOR CAPACITY EVALUATION

The WWTP utilizes three sequencing batch reactors (SBR) in a batch activated sludge system for biological oxidation of the wastewater. The SBR system was constructed in 2002, and each reactor has a maximum volume of 562,000 gallons. Currently the City uses two basins at a time, but the third basin can be brought into service when loading rates increase.

The SBR system was not initially designed for nitrogen removal, and the NPDES permit does not currently place any limitations on ammonia (NH_3) in the effluent. However, at recent loadings the system has sufficient capacity to provide consistent nitrification (conversion of ammonia to nitrate). As described in Appendix K, the WWTP should continue to nitrify in order to avoid having a reasonable potential to exceed ammonia water quality limits in the receiving water.

The capacity of the SBR system was evaluated on the basis of nitrification capacity and in accordance with Ecology criteria (Orange Book, 2008). Performance of the activated sludge system was evaluated at the current and projected flow and loading conditions, and maximum capacity, using a spreadsheet program (Appendix N).

In Table 7-5, various SBR cycle time programs are provided, as used in the spreadsheet model.

TABLE 7-5

SBR Cycle Times

Operational Settings	2002 Aqua-Aerobics Design	Recent Operations	Recommended for Design Loadings and Max. Capacity
Qty. of Basins in Service	3	2	3
Qty. of Cycles/day/basin	5	4	5
Cycle Time, Tc, hr.	4.8	6.0	4.8
Fill/Denit Time, Tf, hr.	0.58	1.25	0.45
Fill/Aerate Time, Tfa, hr.	1.02	1.75	1.15
React/Aerate Time, Ta, hr.	1.38	1.25	1.25
Settle Time, Ts, hr.	0.75	0.92	0.95
Decant Time, Tw, hr.	1.07	0.83	1.00
Total Fill time ratio (%)	33%	50%	33%
Total Aeration ratio (%)	50%	50%	50%

The minimum aerobic SRT that is required to provide nitrification can be calculated using the following process. The first step is to calculate the maximum specific nitrifier growth rate ($\mu_{n,m}$), average decay rate (k_{dn}), and ammonia half saturation coefficient (K_N) at the winter design temperature of 10ºC using the following equations.

$$\mu_{n,m,10} = (\mu_{n,m}) \text{ x } (\theta^{t-20}) = (0.9/\text{d}) \text{ x } (1.072^{10-20}) = 0.449/\text{d}$$

$$\text{Aerated } k_{dn,10} = (k_{dn}) \text{ x } (\theta^{t-20}) = (0.17 \text{ mg/L}) \text{ x } (1.029^{10-20}) = 0.128 \text{ mg/L}$$

$$\text{Anoxic } k_{dn,10} = (k_{dn}) \text{ x } (\theta^{t-20}) = (0.07 \text{ mg/L}) \text{ x } (1.029^{10-20}) = 0.053 \text{ mg/L}$$

The nitrifier decay rate $k_{dn,10}$ is averaged over the SBR cycle with 50 percent of each cycle having aerated conditions ($k_{dn,10}$ of 0.128 mg/L) and the remainder having anoxic conditions ($k_{dn,10}$ of 0.053 mg/L), for an average nitrifier decay rate $k_{dn,10}$ of 0.090 mg/L.

The numerical values for the various parameters above are conservative values typical for domestic wastewater, per *Wastewater Engineering* (Metcalf & Eddy, 5th Edition).

Assuming typical values for effluent ammonia concentration of 1 mg/L, a dissolved oxygen concentration (DO) of 2.0 mg/L, an ammonia half saturation coefficient (K_N) of 0.5 mg/L and an oxygen half saturation coefficient (K_O) of 0.5 mg/L, the actual nitrifier growth rate is calculated as follows:

$$\mu_n = (\mu_{n,m,10})\left(\frac{N}{K_N + N}\right)\left(\frac{DO}{K_O + DO}\right) - k_{dn,10} = (0.449/\text{d})\left(\frac{1.0}{0.50 + 1.0}\right)\left(\frac{2.0}{0.5 + 2.0}\right) - 0.090 \text{ /d}$$

This yields a net specific nitrifier growth rate of 0.15/d, which is then used to calculate the required SRT using the following equation:

$$SRT = 1/\mu_n = 6.70 \text{ days}$$

Applying a safety/peaking factor of 1.25 to this value, to account for daily fluctuations in ammonia loading, produces a required aerobic SRT for reliable nitrification of 8.4 days.

Table 7-6 provides a summary of the model input values.

TABLE 7-6

Activated Sludge Evaluation Input Values(1)

Parameter	Input Value	Source
Minimum Aerobic SRT	8.4 days	Calculations per M&E (2014)
Sludge Volume Index (SVI), Design Max.	155	WWTP Operational Data(2)
Maximum MLSS Concentration (at Max. Water Level)	3,000 mg/L	Operational history and Ecology (2008)
Minimum Temperature	10 degrees C	DMR data

(1) Input values used in activated sludge spreadsheet evaluation (Appendix N).
(2) 90th percentile maximum SVI recorded at the Woodland WWTP during 2016 was 153.

Table 7-7 provides the operational design criteria from the results of the spreadsheet evaluation at current and projected conditions.

TABLE 7-7

SBR Evaluation Results – Operational Characteristics(1)

Criteria	Recent (2009-13) Annual Avg.	Projected 2033 Maximum Month Flow and BOD_5	Maximum Capacity
SBR Basins in Service	2	3	3
Influent Flow Rate (mgd)	0.504	0.960	2.000
Influent BOD_5 (lb/d)	1,141	2,237	2,650
Influent TSS (lb/d)	1,407	2,602	3,160
Influent TKN (lb/d)	209	405	500
Max. Decant Depth Based on SVI	33%	33%	33%
Design Decant Depth	11%	11%	24%
Total HRT at High Water Level (hr.)	54	42	20
Aerobic SRT (d)	14.4	10.8	8.6
MLVSS:MLSS Ratio	0.65	0.67	0.67
F/M Ratio (lb BOD_5/lb MLVSS/d)	0.06	0.08	0.09
BOD_5 Mass Loading Rate (lb BOD_5/1000 ft^3/d)	9	11	15
Sludge Production (lb/d)	981	1,961	2,440
Max. AOR/Basin (lb O_2/hr.)	142	162	180
Peak Air Rqmt./Basin (scfm)	1,420	1,616	1,797

(1) Results from activated sludge spreadsheet evaluation (Appendix N).

The activated sludge spreadsheet evaluation determined that the existing SBR system has sufficient capacity, including nitrification, in excess of the projected 2033 flow and loading rates. However, the SBR capacity for influent BOD_5 in terms of mass loading criteria and nitrification is less than the current permitted capacity in the NPDES permit, and should be derated.

The Sequencing Batch Reactor system was installed in 2002. The concrete tanks and buildings have an expected life of 75 years and should not require extensive maintenance or replacement during the planning period. The mechanical equipment, however, has an expected life of 20 to 30 years, and the City should plan to replace the following major equipment in kind during the 2020 – 2030 timeframe:

- Floating mixers (3)
- Floating decanters (3)
- Waste activated sludge pumps (2)
- Aeration blowers (6)
- Control panel

The existing medium-bubble Transmax aeration diffusers consist only of piping and an orifice, and have an expected life of at least 40 years, which is beyond the planning period.

The automated valves and instrumentation should also be replaced during the planning period as needed under the maintenance budget.

SEQUENCING BATCH REACTOR AERATION EFFICIENCY EVALUATION

The City is interested in determining whether improvements to the current plant could be made to improve energy efficiency and reduce biosolids management costs. Based on a review of plant performance records, SBR aeration consumed nearly 50 percent of the total WWTP electricity usage. The SBR blower and diffuser system is the primary focus of the energy efficiency evaluation. The evaluation has three components: (1) optimization of existing operations; (2) diffuser technologies; and (3) blower technologies. 20-year lifecycle costs are calculated for potential capital improvements and compared to the 20-year lifecycle cost of the existing system.

A spreadsheet model was developed to evaluate the aeration demand and blower air flow required under a variety of loading rates and equipment configurations, and was calibrated with plant performance records.

Existing Operations

The SBR system is equipped with six positive displacement (PD) blowers: four with 50-hp motors and two with 75-hp motors. One of the 50-hp blowers is a redundant standby blower, which can back up either set of blowers. The pipe manifold allows flexibility, but the typical arrangement is for the 50-hp blowers to be paired with Basins 1 and 2, and the 75-hp blowers with Basin 3 when all three basins are in service.

The WWTP currently uses two-basin operation based on the current loading rates. The aerated phases in the SBR cycle are 50 percent of the cycle time. With two basins in service, the aerated phases do not overlap and one set of blowers runs continuously while the automated valves switch the air supply between the basins according to the SBR cycle program. When three basins are in service, as when the loading rates are near the three-basin design criteria, at certain points in the SBR cycle two basins will be aerated simultaneously. The second set of blowers, i.e., the 75-hp blowers, would provide air to the second basin in aeration.

The SBR control system targets a dissolved oxygen concentration setpoint during the aerated phases of the cycle, by turning the in-service blowers on and off. Currently three 50-hp blowers run approximately 70 percent of the time to meet an average BOD_5 loading of 1,144 lb/day (the 2009-2013 annual average).

The energy efficiency evaluation found that different blower configurations would reduce power. Table 7-8 provides information on the current and two proposed blower configurations. The air demand could be met while operating fewer blowers. The most energy-efficient option would be to run one 75-hp blower continuously. This operational mode will require changes to valve, control panel and electrical settings.

TABLE 7-8

Current SBR Aeration Electricity Consumption

Parameter	Baseline (Current Operations)	Current Loading w/Two – 50-hp Blowers	Current Loading w/One – 75-hp Blower
Influent BOD_5 Loading (lb/day)	1,144		
Actual Oxygen Demand (lb/day)	2,002		
Required Air Flow Rate (scfm)	833		
Quantity of SBR Basins in Service	2		
Aerated Cycle Time Ratio, per Basin	50%		
Quantity of Blowers in Service	3 – 50 hp	2 – 50 hp	1 – 75 hp
Blower Output Air Flow Rate (scfm)	1,875	1,250	937
Average Blower Motor Power (hp)	120	80	62
Blower on Time Ratio During Aerated Cycles	70%	100%	100%
Average Daily Power Required (kWh/day)	1,504	1,432	1,110
Annual Avg. Power Consumption (kWh)	549,000	523,000	405,000
Average Annual Power Cost(1)	$41,700	$39,700	$30,800

(1) Power costs based on 7.7 cents per kilowatt-hour (kWh).

Evaluation of Diffuser Technologies

The existing medium-bubble Transmax aeration diffusers have a lower oxygen transfer efficiency than fine-bubble diffusers typically used in activated sludge processes. However, they require no maintenance and have an expected life of 40 years. Switching to fine-bubble diffusers would result in lower blower energy costs, but also higher maintenance costs, particularly in an SBR where the solids settle on the tank floor during the settle and decant phases. The diffusers should be cleaned annually, and the expected diffuser membrane life of 7 to 10 years may be reduced to 5 years for SBR service.

Traditionally, fine-bubble diffusers have been permanently mounted on the tank floor, and draining the tank was required to clean or replace the diffuser membranes. Retrievable diffuser grids are now available, in which air is supplied through flexible hoses and an electric winch is provided to lift an entire diffuser grid section, composed of manifold pipes and fine-bubble diffuser tubes, out of the basin contents. With a retrievable fine-bubble diffuser system, the diffusers could be lifted out of the SBR basin for cleaning and membrane replacement. Ultra-high efficiency fine-bubble aeration plates are also available, which provide a higher oxygen transfer efficiency than standard fine-bubble diffusers, although they are not available in retrievable grids.

Table 7-9 summarizes the design criteria of the diffuser technologies that were evaluated for the SBR. Fine-bubble diffusers have oxygen transfer efficiency rates more than twice as high as the existing medium-bubble diffusers, which results in an air flow demand of less than half of the current demand.

TABLE 7-9

Design Data for Diffuser Technologies

Parameter	Medium-Bubble Diffusers (Transmax)	Standard Fine-Bubble Diffusers	Retrievable Fine-Bubble Diffusers	Ultra-High Efficiency Fine-Bubble Diffusers (Aerostrip)
Oxygen Transfer Efficiency (per foot submergence)	0.87%	1.80%	1.80%	2.40%
Expected Membrane Life (in SBR service)	N/A	5	5	10
Annual Labor Hours for Maintenance	0	120	48	120
Estimated Project Cost for Installation (2014 dollars)	N/A	$193,000	$455,000	$451,000

Electricity consumption was calculated for the evaluated diffuser technologies at current WWTP loadings and compared to the current diffusers, as shown in Table 7-10. Either standard or ultra-high efficiency fine-bubble diffusers have sufficient oxygen transfer efficiencies to allow one of the current 50-hp blowers to meet the aeration demand.

TABLE 7-10

Comparison of Aeration Electricity Consumption for Diffuser Technologies

Parameter	Baseline (Current Operations)	Current Loading w/75-hp Blower	Fine-Bubble Diffusers	Ultra-High Efficiency Fine-Bubble Diffusers (Aerostrip)
Current Influent BOD_5 Loading (lb/day)	1,144			
Current Actual Oxygen Demand (lb BOD_5/day)	2,002			
Quantity of SBR Basins in Service	2			
Aerated Cycle Time Ratio, per Basin	50%			
Required Air Flow Rate (scfm)	833		403	302
Quantity of Blowers in Service	3 – 50 hp	1 – 75 hp	1 – 50 hp	1 – 50 hp
Blower Output Air Flow Rate (scfm)	1,875	937	625	620
Blower Discharge Pressure (psi)	9.7	9.7	10.0	10.3
Average Blower Motor Power (hp)	120	62	41.2	42.1
Blower On Time Ratio During Aerated Cycles	70%	100%	100%	77%
Average Daily Power Required (kWh/day)	1,504	1,110	737	580
Annual Avg. Power Consumption (kWh)	549,000	405,000	269,000	212,000
Average Annual Power Cost(1)	$41,700	$30,800	$20,400	$16,100

(1) Power costs based on 7.7 cents per kWh.

In Table 7-11, capital, operation and maintenance (O&M) and 20-year lifecycle costs are presented for each diffuser technology option and compared to the costs of continuing current operations. The capital costs are total project costs, including construction and engineering, in 2014 dollars. The annual operations and maintenance costs include electricity at Woodland's current average rate of 7.7¢ per kilowatt-hour (kWh), equipment maintenance at an annual rate of 1.5 percent of the equipment cost, and additional maintenance labor at $75 per hour. The 20-year lifecycle cost is a net present value (NPV) for the years 2014 through 2033 (the planning period), assuming an annual inflation rate of 3.0 percent and an annual discount rate of 2.25 percent.

The lifecycle cost calculations assume that the BOD_5 loading to the plant increases linearly to match the projected loading rates for 2023 and 2033. Capital costs are assumed to be spent from reserves in the expenditure year; interest costs from loans are not accounted for.

With the current operations, the existing blowers will reach the end of their service life in 2027 and the City should budget for their replacement at this time. If fine-bubble diffusers are installed by that point, the design air demand is less and the blowers can be replaced with three 75-hp blowers (two duty, one standby).

TABLE 7-11

Lifecycle Cost Evaluation for Diffuser Technologies

Alternative		Total Project Cost ($2014)	Annual O&M Cost Estimate ($2014)	20-year Lifecycle Cost(1)
1	Current Operations(2)	$548,000	$44,400	$1,842,000
2	Current Loading with one – 75-hp Blower(2)	$548,000	$33,500	$1,530,000
3	Standard Fine-Bubble Diffusers(3)	$705,850	$32,100	$1,612,000
4	Retrievable Fine-Bubble Diffusers(3)	$1,017,500	$26,700	$1,828,000
5	Ultra High-Efficiency Fine-Bubble Diffusers (Aerostrip)(4)	$944,650	$27,800	$1,737,000

(1) Lifecycle cost assumes an annual inflation rate of 3.0 percent and an annual discount rate of 2.25 percent and linear growth loadings up to projected 2033 loadings.

(2) Project cost includes replacing six blowers in kind in 2027; O&M cost includes electricity and blower maintenance.

(3) Project cost includes diffusers in 2016, replacing diffuser membranes in 2021, 2026 and 2031, and replacing blowers with three – 75-hp blowers in 2027; O&M cost includes electricity, blower maintenance and diffuser cleaning labor.

(4) Project cost includes diffusers in 2016, replacing diffuser membranes in 2026, and replacing blowers with three – 75-hp blowers in 2027; O&M cost includes electricity, blower maintenance and diffuser cleaning labor.

As presented in Table 7-11, the fine-bubble diffuser options do not result in 20-year lifecycle costs lower than continued use of the Transmax diffusers with one 75-hp blower running continuously. Each of the fine-bubble diffuser alternatives have lower lifecycle cost than the current blower operations with three 50-hp blowers cycling on and off. The lower electricity costs with fine-bubble diffusers are offset by the capital costs of installing the diffusers and higher diffuser maintenance and replacement costs.

Evaluation of Blower Technologies

Alternatives to the existing positive displacement (PD) blowers could result in energy savings. Over the past decade, turbo centrifugal blowers have been demonstrated in the marketplace and offer potential power savings, particularly in larger blower sizes. Turbo blowers use high-speed permanent magnet motors to directly rotate the centrifugal blower shaft at up to 36,000 rpm.

The efficiency gains become larger as the blower size becomes larger; from our experience, turbo blowers with motor sizes under 100 hp are often not cost effective relative to positive displacement blowers because the increase in capital cost outweighs the reduced operations and maintenance costs.

In Table 7-12 design data for turbo centrifugal blowers is presented. With the existing medium-bubble diffusers, five 50-hp turbo blowers (two for each SBR basin in simultaneous aeration, and one common standby blower) would be required to meet the airflow requirements for the 3-basin design criteria. The airflow requirement with fine-bubble diffusers is much lower, and three 50-hp turbo blowers would be sufficient (one for each SBR basin in simultaneous aeration and one common standby blower).

TABLE 7-12

Design Data for Turbo Centrifugal Blower Alternatives

Parameter	Current PD Blowers (50 hp)	Current PD Blowers (75 hp)	Turbo Blowers w/Medium-Bubble Diffusers	Turbo Blowers w/Fine-Bubble Diffusers
Blower Quantity	4 (one standby)	2	5 (one standby)	3 (one standby)
Blower Design Capacity	625 scfm @ 9.7 psi	937 scfm @ 9.7 psi	920 scfm @ 9.7 psi	920 scfm @ 10 psi
Blower Motor Size	50 hp	75 hp	50 hp w/VFDs	50 hp w/VFDs
Annual Maintenance Cost (2014 dollars)(1)	$2,700		$4,200	$2,500
Estimated Project Cost for Installation (2014 dollars)	N/A		$840,000	$500,000

(1) Annual equipment maintenance cost is estimated as 1.5 percent of the equipment cost.

Electricity consumption was calculated for the turbo centrifugal blowers at current WWTP loading and compared to the current blowers, as shown in Table 7-13.

TABLE 7-13

Comparison of Aeration Electricity Consumption for Blower Technologies

Parameter	Baseline (Current Operations)	Current Operations w/75-hp Blower	Turbo Blowers w/Medium-Bubble Diffusers	Turbo Blowers w/Fine-Bubble Diffusers	Turbo Blowers w/Aerostrip Diffusers
Current Influent BOD_5 Loading (lb/day)	1,144				
Current Actual Oxygen Demand (lb/day)	2,002				
Quantity of SBR Basins in Service	2				
Aerated Cycle Time Ratio, per Basin	50%				
Required Air Flow Rate (scfm)	833		833	403	302
Quantity of Blowers in Service	3 – 50 hp	1 – 75 hp	1 – 50 hp	1 – 50 hp	1 – 50 hp
Blower Output Air Flow Rate (scfm)	1,875	937	833	550	560
Blower Discharge Pressure (psi)	9.7	9.7	9.7	10.0	10.3
Average Blower Motor Power (hp)	120	62	46.5	33.7	35.7
Blower on Time Ratio During Aerated Cycles	70%	100%	100%	100%	86%
Average Daily Power Required (kWh/day)	1,504	1,110	833	603	550
Annual Avg. Power Consumption (kWh)	549,000	405,000	304,000	220,000	201,000
Average Annual Power Cost(1)	$41,700	$30,800	$23,100	$16,700	$15,300

(1) Power costs based on 7.7 cents per kWh.

In Table 7-14, the capital, operation and maintenance (O&M) and 20-year lifecycle costs are provided for installing turbo blowers with each diffuser technology described in the previous section. One alternative has a marginally lower 20-year lifecycle than current operations using one 75-hp blower: installing turbo blower and standard fine-bubble diffusers in 2016. With the greater aeration efficiency of fine-bubble diffusers, only three 50-hp turbo blowers are required to meet the 3-basin aeration demand.

The estimated project cost for installation of the turbo blowers ($500,000) is slightly less than the replacement cost for the existing six PD blowers ($548,000). The electricity savings due to pairing fine-bubble diffusers and turbo blowers are offset by the additional capital and O&M costs associated with the fine-bubble diffusers. As presented in Table 7-14, none of the turbo blower or fine-bubble diffuser combinations result in 20-year lifecycle costs lower than continued use of the Transmax diffusers with one 75-hp blower running continuously. Continuing current operations with one 75-hp blower is recommended.

TABLE 7-14

Lifecycle Cost Evaluation for Turbo Blower Technology

	Alternative	Total Project Cost ($2014)	Annual O&M Cost Estimate ($2014)	20-year Lifecycle Cost(1)
1	Current Operations(2)	$548,000	$44,400	$1,842,000
2	Current Operations with 75-hp Blower(2)	$548,000	$33,500	$1,530,000
3	Turbo Blowers with Existing Transmax Diffusers(3)	$840,000	$27,300	$1,707,000
4	Turbo Blowers with Standard Fine-Bubble Diffusers(4)	$779,850	$28,200	$1,551,000
5	Turbo Blowers with Retrievable Fine-Bubble Diffusers(4)	$1,091,500	$22,800	$1,767,000
6	Turbo Blowers with Ultra High-efficiency Fine-Bubble Diffusers (Aerostrip)(5)	$1,018,650	$26,800	$1,751,000

(1) Lifecycle cost assumes an annual inflation rate of 3.0 percent and an annual discount rate of 2.25 percent and linear growth loadings up to projected 2033 loadings.
(2) Project cost includes replacing six blowers in kind in 2027; O&M cost includes electricity and blower maintenance.
(3) Project cost includes installing five 50-hp turbo blowers in 2016; O&M cost includes electricity and blower maintenance; power costs based on 7.7¢/kWh.
(4) Project cost includes installing three 50-hp turbo blowers and diffusers in 2016, and replacing diffuser membranes in 2021, 2026 and 2031; O&M cost includes electricity, blower maintenance and diffuser cleaning labor.
(5) Project cost includes installing three 50-hp turbo blowers and diffusers in 2016, and replacing diffuser membranes in 2026; O&M cost includes electricity, blower maintenance and diffuser cleaning labor.

SBR Recommendations

The mechanical equipment in the SBR system will reach the end of the expected life within the planning period and the City should budget to replace this equipment between 2020 and 2030. Energy efficiency improvements were evaluated, and the recommended alternative is to modify current blower operations. One 75-hp blower running continuously should provide sufficient aeration at the current loading rate, which will result in significantly lower electrical costs and 20-year lifecycle costs than current blower operations. This operational mode will require changes to valve and hand-off-auto switch settings, and may require control panel or programming modifications. Capital projects involving the installation of fine-bubble diffusers or turbo centrifugal blowers did not result in significantly lower lifecycle costs.

The capital improvement program for the SBR system should include funding to replace the following major equipment in kind during the 2020 – 2030 timeframe, tentatively in the year 2027:

- Floating mixers (3)
- Floating decanters (3)
- Waste activated sludge pumps (2)
- Aeration blowers (6)
- Control panel

In Table 7-15, a preliminary project cost estimate is provided for these recommended replacements.

TABLE 7-15

SBR System Recommended Capital Plan for Year 2027 (in 2014 Dollars)

No.	Item	Quantity	Unit Price	Amount
1	Mobilization/Demobilization	1 LS	$70,000	$70,000
2	Aqua Aerobics Equipment Package w/Installation	1 LS	$420,000	$420,000
3	PD Blowers (50 hp) w/Installation	4 EA	$32,000	$128,000
4	PD Blowers (75 hp) w/Installation	2 EA	$64,000	$128,000
5	Sludge Pumps w/Installation	2 EA	$10,000	$20,000
6	Electrical	1 LS	$135,000	$135,000

Subtotal.. $901,000
Contingency (20%) $180,000
Subtotal..$1,081,000
Washington State Sales Tax (7.8%).. $84,000
Total Estimated Construction Cost $1,165,000
Engineering and Construction Administration (25%).. $291,000
TOTAL ESTIMATED PROJECT COST $1,456,000

EFFLUENT DISINFECTION SYSTEM EVALUATION

The existing effluent disinfection system consists of two in-vessel medium-pressure UV disinfection reactors, manufactured by Aquionics, which disinfect the clarified effluent from the SBRs. Each reactor is rated to handle a maximum decant flow of 4.8 mgd at a minimum UV transmission of 60 percent. The disinfected effluent then flows to the effluent pump station through an effluent flow measuring system upstream of the pump station.

Ultraviolet transmittance (the ability to transmit light) of the effluent is a key parameter in the design and operation of the system; however, it is not routinely measured at the Woodland WWTP. The transmittance of the daily effluent composite was spot-checked twice in February 2015; the values were 60.7 percent and 60.8 percent, similar to the design value of 60 percent.

Medium-pressure UV disinfection systems have significantly greater electricity consumption than low-pressure UV systems. In the years since the UV system was installed (2002), high-intensity low-pressure UV systems have been developed which reduce the lamp quantity requirement versus older low-pressure systems. A quote for a closed-vessel low-pressure UV disinfection system was obtained for an energy efficiency comparison. The low-pressure UV reactor units are larger than the current medium-pressure reactors, but it appears that sufficient space is available in the UV Room and the Electrical Room to accommodate a low-pressure UV system. Table 7-16 summarizes the lifecycle cost and payback information for replacing the UV system with a new low-pressure system in the near term.

The low-pressure UV disinfection system would use 53 percent less electrical power than the existing system. This equates to an electrical cost savings of about $2,000 per year. The capital cost of the new UV system is significantly higher than the electrical cost savings that would result during the 20-year life of the equipment, and the payback period is approximately 300 years. Therefore, replacement of the UV system with a low-pressure system is not recommended in the near term.

The current UV disinfection system will reach the end of the expected life (typically 15 to 20 years) within the planning period and the City should budget to replace this equipment between 2017 and 2022. When the UV disinfection system is replaced, installing a closed-vessel low-pressure high-output UV system could be considered. A quote was obtained for replacement of the current UV system in kind. In Table 7-17, the capital and lifecycle cost comparison for replacing the current UV system in kind or with a new low-pressure system are compared.

TABLE 7-16

Lifecycle Cost Evaluation for UV Disinfection System Replacement in 2016

Evaluation Parameter	Current Medium-Pressure System	Low-pressure High-Output System
Equipment Cost ($2014)	N/A	$265,000
Installation Project Cost ($2014)[(1)]	N/A	$662,500
Electrical Consumption (kW)	26.5	12.4
Annual Electrical Cost ($/yr)[(2)(3)]	$3,745	$1,720
Lamp Life (hour)	6,000	9,000
Lamp Life (years)[(4)]	6.6	9.9
Lamp Replacement Cost ($)	$650	$165
Total Lamp Quantity	16	80
Annual Lamp Replacement Cost ($)	$1,582	$1,338
Annual Operation Expense ($)	$5,306	$3,058
20-Year Life Cycle Cost[(5)]	$111,300	$712,600
Simple Payback Period (years)		300

(1) Installation project cost includes mobilization, overhead, mechanical and electrical installation, contingency, sales tax, engineering and construction administration. Estimated at 250 percent of equipment cost.
(2) Electrical power costs based on 7.7¢/kWh.
(3) Based on the design decant rate of 3,333 gpm with 5 decant cycles per day (60 minutes each). One reactor runs at full power during decant cycles.
(4) Assumes the duty reactor alternates for equal lamp use.
(5) Lifecycle cost assumes an annual inflation rate of 3.0 percent and an annual discount rate of 2.25 percent.

TABLE 7-17

Lifecycle Cost Evaluation for UV Disinfection System Replacement in 2020

Evaluation Parameter	New Medium-Pressure System	New Low-Pressure High-Output System
Equipment Cost ($2014)	$210,000	$265,000
Installation Project Cost ($2014)(1)	$483,000	$662,500
Electrical Consumption (kW)	26.5	12.4
Annual Electrical Cost ($/yr)(2)(3)	$3,745	$1,720
Lamp Life (hr)	6,000	9,000
Lamp Life (years)(4)	6.6	9.9
Lamp Replacement Cost ($)	$650	$165
Total Lamp Quantity	16	80
Annual Lamp Replacement Cost ($)	$1,582	$1,338
Annual Operation Expense ($)	$5,306	$3,058
20-Year Life Cycle Cost(5)	$584,000	$712,600

(1) Installation project cost includes mobilization, overhead, mechanical and electrical installation, contingency, sales tax, engineering and construction administration. Estimated at 250 percent of equipment cost for the low-pressure system, and 230 percent for the medium-pressure system due to fewer piping modifications.
(2) Electrical power costs based on 7.7¢/kWh.
(3) Based on the design decant rate of 3,333 gpm with 5 decant cycles per day (60 minutes each). One reactor runs at full power during decant cycles.
(4) Assumes the duty reactor alternates for equal lamp use.
(5) Lifecycle cost assumes an annual inflation rate of 3.0 percent and an annual discount rate of 2.25 percent.

As shown in Table 7-17, at the point of UV system replacement the low-pressure high-output system has a higher 20-year lifecycle cost than the medium-pressure system. Despite the annual electricity savings of about $2,000, the higher capital cost of the low-pressure system results in a higher lifecycle cost over the expected equipment lifetime. An updated energy efficiency analysis should be performed prior to replacing the UV system. However, based on the high capital costs and no lifecycle savings, replacement of the UV system with a new medium-pressure system will be included in the CIP for year 2020.

EFFLUENT SAMPLER

A composite sampler collects effluent samples from the wet well extension ahead of the weir. Ecology recommends that composite samplers are flow-paced, to ensure that the weighted composite samples accurately represent the effluent flow.

Effluent is removed from the SBRs on a batch basis during the decant cycles, and the water in the wet well extension is stagnant between decant cycles. The sampling point should be relocated from the wet well extension to a sample tap on the pipe between the UV disinfection reactors and the effluent pump station. Programming should be added so that the sampler is flow paced, such that no samples are taken when no SBR is in a decant cycle and there is no flow over the effluent weir.

AEROBIC DIGESTION SYSTEM EVALUATION

Sludge removed from the SBRs is treated in a pair of aerobic digesters and a gravity thickener tank in a process known as Prethickened Aerobic Digestion (PAD), designed by Enviroquip, currently known as Ovivo. The process was constructed in the 2002 WWTP upgrade. The digesters were sized to meet the criteria for Class B pathogen reduction using the "time-temperature" method of a 40-day mean solids residence time (SRT) at a minimum temperature of 20 degrees C. The digester tanks are covered to maintain the minimum temperature throughout the year. After the waste activated sludge (WAS) from the SBRs has been adequately stabilized in the aerobic digesters, it is pumped to a sludge truck for hauling in liquid form. The final biosolids, treated to Class B standards, are taken to a storage lagoon owned and operated by others and eventually to a nearby permitted biosolids land application site that is also owned and operated by others.

Recent performance of the aerobic digester was compared to the original 2002 design criteria (Table 7-18). The estimated WAS production from the SBR tanks is higher than assumed in the 2002 design criteria. This may be due to the higher ratio of TSS to BOD in the wastewater influent that has been observed compared to the influent design criteria (much influent TSS is inert, passing through the SBRs into the sludge). The higher sludge loading rate is coupled with a lower sludge concentration and lower volatile solids reduction observed in the aerobic digesters compared to the design criteria. Together, these factors result in higher biosolids production than would be expected based on the design criteria and a shorter SRT in the digester tanks. Because the WWTP loading rate is only about 30 percent of the original design loading and less than 45 percent of the permitted WWTP loading, the aerobic digester system has sufficient capacity at present.

In Table 7-18, observed performance values are extrapolated to the projected 2033 maximum month BOD_5 loading rate, resulting in 27-day SRT in the digester tanks. As WWTP loading rates increase, it appears that the digester will reach capacity (defined as providing a 40 day SRT) prior to 2033, and well before the original design capacity is reached. When increased loading rates cause the digester SRT to drop below 40 days, it is recommended that the City further evaluate options including a process review from Ovivo, the provider of this package digester process system, to determine if the design criteria can be achieved by further increasing percent solids in the digester to the design level of 3 – 3.5 percent. There may be a cost associated with acquiring Ovivo's services. If digester performance cannot be significantly improved through operational modifications, capital improvements may be needed. Two capital improvements would increase digestion capacity:

- Replace the gravity thickening tank with a mechanical sludge thickener, such as a rotary drum thickening equipment to achieve the solids concentration of 3 percent.

- Construct additional aerobic digester tank volume assuming 2 percent solids is the best the gravity thickener can achieve.

TABLE 7-18

Aerobic Digester System Design Criteria and Performance

Parameter	Original Design Criteria	Recent Performance Data	Recent Performance Data Extrapolated to Projected 2033 Loadings	Recent Performance Data Extrapolated to Permitted Loadings
Influent WWTP BOD_5 Loading (lb/day)	3,720	1,144 (1)	2,237(6)	3,107(7)
WAS production net yield (lb WAS/lb influent BOD_5)	0.66	0.87(2)	0.87	0.87
WAS production (lb/day)	2,455	1,000(3)	1,946	2,703
Volatile solids reduction in digester (%)	40 – 50%	21%(4)	21%	21%
Digester solids concentration (%)	3.0 – 3.5%	2.0%(5)	2.0%	2.0%
SRT (days)	40 days	52 days	27 days	19 days
Digested sludge (lb/day)	1,649	845(5)	1,648	2,290
Digested sludge rate (gal/day)	6,600 @ 3.0% solids	5,100 @ 2.0% solids	9,900 @ 2.0% solids	13,700 @ 2.0% solids

(1) From Table 5-2; 2009 to 2013 average.
(2) Net yield calculated from sludge wasting estimate.
(3) Sludge wasting estimated from WAS pump run time meter: (110 min/day) * (109 gal/min pump) * (10,000 mg/L) * (8.34/1,000,000 conversion factors) = 1,000 lb/day.
(4) Digester volatile solids reduction calculated from WAS production and digested sludge production, assuming WAS volatile content of 73 percent per design criteria.
(5) WWTP operating reports; 2009 to 2013 average digester concentration and digested solids production.
(6) Projected 2033 maximum month BOD_5 influent loading rate from Table 7-3.
(7) Permitted BOD_5 loading rate.

SOLIDS DEWATERING EVALUATION

The City of Woodland's approach to biosolids management until 2016 involved beneficial utilization by the application of liquid digested sludge on agricultural land near the City of Woodland. The liquid biosolids from the Woodland WWTP were hauled to a storage lagoon located in the City of Woodland, which is owned by others and provided year-round storage for the City's biosolids.

In 2016, the permit to operate the biosolids storage lagoon was terminated by the Washington State Department of Ecology and the City had to arrange hauling sludge to another wastewater treatment plant during the times of the year when the land application facility near Woodland is not available to take liquid biosolids.

Table 7-19 summarizes data for biosolids generated by the Woodland WWTP and the total costs for the biosolids management methods in use at the time.

TABLE 7-19

Woodland WWTP Annual Biosolids Generation Data and Costs

Year	Dry Tons (DT) Hauled (1)(2)	Annual WWTP BOD_5 Loading (Tons)	DT/ BOD_5 Loading Ratio	Average Solids Content Hauled Biosolids	Number of Hauling events per Year	Total Volume Hauled (10^6 gal)	Total Cost
2009	131	254	0.52	1.5%	9	2.106	$176,904
2010	164	195	0.84	1.9%	9	2.106	$176,904
2011	156	193	0.81	2.3%	7	1.638	$137,592
2012	156	195	0.80	2.0%	8	1.872	$157,248
2013	164	207	0.79	2.1%	10	2.310	$191,730
2014	176	191	0.92	1.03%	9	2.316	$194,544
2015	105	200	0.52	1.15%	9	2.386	$201,617
2016	97	273	0.35	1.59%	6	1.454	$129,745
Avg.	144	214	0.69	1.7%	8.4	1.919	$170,785

(1) As reported in the City's Annual Biosolids Report submitted to Department of Ecology for years 2009 – 2015.
(2) In 2016, the City began hauling some of its liquid biosolids to other wastewater treatment facilities.

The present method of biosolids management requires hauling liquid biosolids to the Aberdeen, Washington, wastewater treatment facility as much as 260 days per year (mid-October to June). During the remaining 105 days of the year (July to mid-October), the liquid biosolids can be hauled to the beneficial use facility in Woodland for immediate land application. The projected annual cost of the current operation is as follows:

Hauling to Aberdeen, Washington, WWTF:

Load (5,000 gal)/day * ($695/load to haul + $665/load to dispose) * 260 day/yr = $353,600/yr

Hauling and Land Applying in Woodland:

Load (5,000 gal)/day * ($435/load to haul) *105 day/yr +
5,000 gal/day * $0.04/gal for land application * 105 day/yr
= $45,675 + $21,000
= $66,675/yr

Total = $353.600 + $66,675 = $420,275/year

The previous biosolids management system allowed year-round off-site storage of liquid biosolids by others and the City was not required to store its biosolids at the WWTP during the non-growing season. This arrangement ended in 2016 and the City has had to haul its liquid digested biosolids to other wastewater treatment facilities for most of the year causing the cost of biosolids management to increase by more than a factor of two.

Per discussion with Tribeca Transport, the City's current contractor for managing its biosolids, if the City were to convert to managing its biosolids by land application of mechanically dewatered biosolids, the City could arrange with Tribeca to haul the dewatered biosolids to a beneficial use facility in Eastern Washington (Goldendale) for about 260 days out of the year and continue to bring the dewatered biosolids to Woodland about 105 days out of the year.

The advantage of converting to a biosolids management system involving mechanically dewatered biosolids is that annual costs would significantly decrease because, depending on the dewatering equipment used, the volume of biosolids will decrease by a factor of ten or more.

Table 7-20 shows potential costs for biosolids hauling and land application if sludge volumes are reduced by mechanical dewatering. Assumed percent solids are typical of those achieved with various mechanical dewatering systems. Note that the actual percent solids present in the cake from the dewatering operation will vary according to the dewatering system, raw sludge characteristics and other factors such as polymer selection.

TABLE 7-20

Potential Costs for Dewatered Biosolids Management

	lb/day (wet)	Wet Tons/yr	Loads/yr(1)	Goldendale Haul Cost(2)	Woodland Haul Cost(3)	Woodland Land Apply Cost(4)	Total Cost
Year 2017							
834 lb DS/d							
14%	5,957	1,087	72	$44,814	$4,483	$250	$49,547
16%	5,213	951	63	$39,212	$3,922	$219	$43,353
18%	4,633	846	56	$34,855	$3,487	$195	$38,536
Year 2033							
1,648 lb DS/d							
14%	11,771	2,148	143	$88,553	$8,858	$494	$97,905
16%	10,300	1,880	125	$77,483	$7,751	$433	$85,667
18%	9,156	1,671	111	$68,874	$6,890	$385	$76,148

(1) Each load assumed to be 15 wet tons.
(2) $868/load to haul to Goldendale BUF.
(3) $215/load to haul to Woodland BUF.
(4) $12/load to land apply at Woodland BUF.

Assuming sludge production of 1,648 lb/day (wet weight, based on the 2,237 lb/day BOD_5 projected 2033 loading) dewatering digested sludge to 14 percent, the annual cost of biosolids hauling and land application would be approximately $$98,000. This compares to a projected annual cost of more than$840,000 when WWTF loadings reach capacity with no dewatering process in place.

Current annual costs for biosolids management are expected to exceed $420,000. This is nearly ten times the projected cost if the current quantities of sludge were dewatered to 16 percent solids by weight. Accordingly, adding a dewatering process to the WWTF has significantly potential cost savings.

Because of the relatively short potential payback period, further evaluation of sludge dewatering systems is warranted. Three different mechanical dewatering systems will be evaluated: a screw press, a belt filter press and a centrifuge. Each system is described in greater detail below.

SCREW PRESS DEWATERING SYSTEM

A screw press is a mechanical device that achieves liquid/solid separation by gradually reducing the volume available for the solids as they are conveyed from the inlet to the outlet end of the screw press. The reduction in volume is achieved by a rotating tapered shaft that is larger in diameter at the discharge end than at the inlet end, as shown in Figure 7-6.

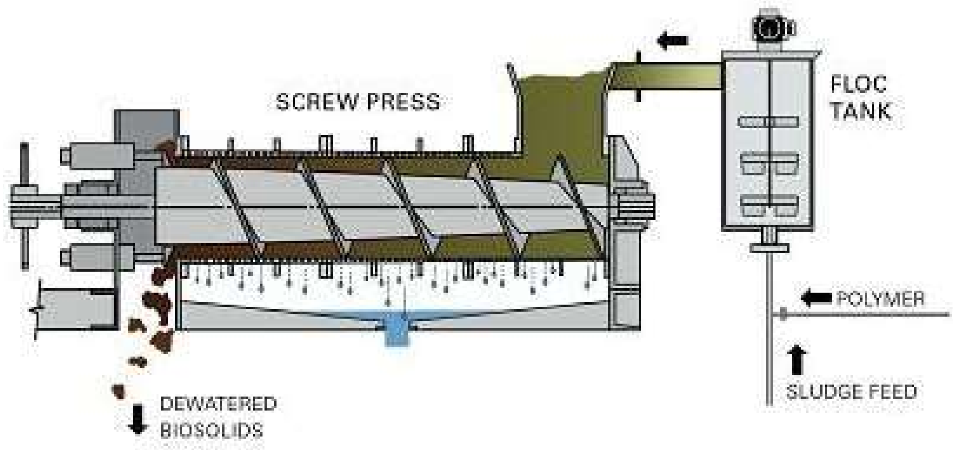


FIGURE 7-6

Dewatering Screw Press Schematic (from FKC)

Dewatering screw presses are becoming a more popular option for municipal sludge dewatering due to the relative ease and reliability of its operation (easy startup and shutdown), low mechanical complexity, low energy use, a smaller equipment footprint than a belt filter press and low noise levels. Within the last 15 years, new FKC screw presses have been installed at the Cities of Lynnwood, Sequim, Forks, Toppenish and Raymond and new systems are being installed in Puyallup and Lynnwood. The disadvantages of a screw press are lower throughput and lower percent solids in dewatered cake compared to other dewatering equipment.

BELT FILTER PRESS DEWATERING SYSTEM

A belt filter press dewaters by a combination of chemical conditioning, gravity drainage and shear and compression dewatering. As shown in Figure 7-7, a typical belt filter press first thickens sludge by polymer addition, followed by gravity drainage of water separated from the thickened solids. The thickened sludge is conveyed between two porous cloth belts which further remove water from the solids to produce a cake that is ejected from the end of the press.

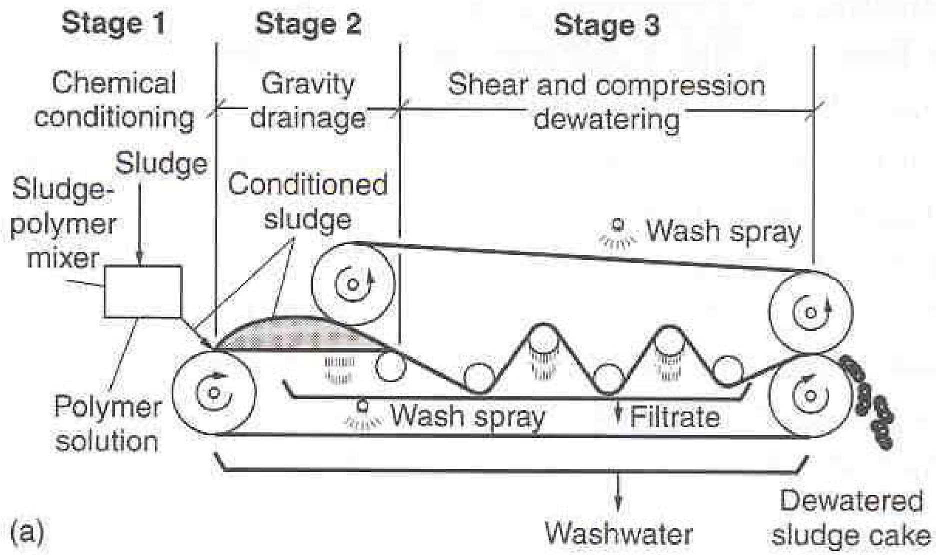


FIGURE 7-7

Belt Filter Press Dewatering System Schematic (from Metcalf and Eddy Wastewater Engineering Treatment and Reuse, 4th Edition)

The advantages of a belt filter press are higher cake concentrations compared to a screw press, low polymer doses and moderate power consumption. Disadvantages with this type of equipment include high water use, greater mechanical complexity, high labor requirements (not recommended to run unattended), high odor potential, a large equipment footprint and moderate noise levels. The process is also sensitive to the quality of the sludge being dewatered and high pressure spray water is required for belt cleaning. Odors and aerosols are difficult to contain with belt filter presses compared to other dewatering processes.

CENTRIFUGE DEWATERING SYSTEM

A centrifuge dewatering system operates by feeding sludge at a constant flow rate in a rotating bowl where it separates into a dense cake and a dilute waste stream called the centrate. Figure 7-8 shows a schematic for a decanter centrifuge

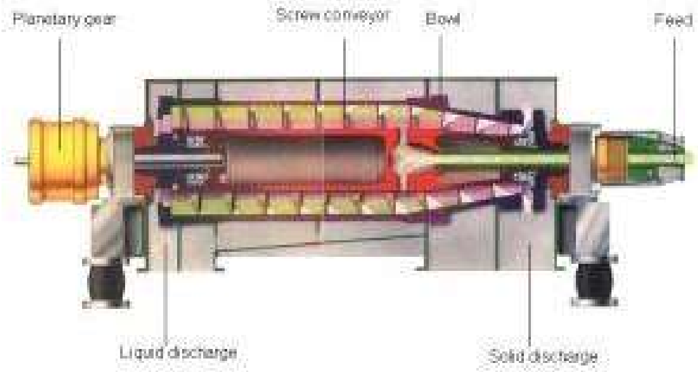


FIGURE 7-8

Centrifuge Dewatering System Schematic

A significant advantage of a centrifuge is a small equipment footprint compared to other dewatering equipment. Centrifuges can typically achieve a higher solids concentration in the cake sludge and handle higher sludge feed rates than with other mechanical dewatering systems, which means they are potentially less labor-intensive to operate. The disadvantages of a centrifuge are the higher power costs, high noise generation, and the relatively higher operator skill required compared to other dewatering systems. Centrifuge equipment components also tend to wear out more quickly due to the high rotational velocity and are much more sensitive to the presence of grit in the feed sludge than with other dewatering equipment.

CAPITAL COSTS FOR DEWATERING EQUIPMENT

Tables 7-21, 7-22, and 7-23 present preliminary cost estimates for purchasing and installing dewatering equipment based on the original rating of the WWTP by the design engineer: 3,720 lb/day BOD_5, which is about 17 percent higher than permitted capacity. This translates to a dewatering system capacity of 17,420 gal/day solids at a solids loading rate of 2,905 lb/day TS assuming 2 percent digested sludge as the feed. Costs also include a 60-foot x 120-foot covered storage area with 7,200 ft^2 of storage area that could be located in the area north of the existing lab/office building.

The difference between using the design rating versus the permitted rating is not expected to impact capital costs significantly because dewatering equipment that is sized to process 17,420 gal/day will also process 13,700 gal/day and simply be operated fewer hours during the week. This will be discussed in more detail in the operations and maintenance portion of the analysis.

It is also recommended that before equipment is sized and specified, pilot testing be performed with the various dewatering systems being evaluated to verify polymer dosage and dewatered cake concentrations. The costs presented below will be used to evaluate the potential financial benefits of implementing mechanical dewatering at the Woodland WWTP.

TABLE 7-21

Screw Press Dewatering System[1]

No.	Item	Quantity	Unit Price	Amount
1	Mobilization/Demobilization	1 LS	$140,000	$140,000
2	Screw Press Equipment Package with Control Panel and Polymer Feed w/Installation[2]	1 EA	$400,000	$400,000
3	Piping	1 LS	$75,000	$75,000
4	Building (25' x 40')[3]	1,000 SF	$350	$350,000
5	Shaftless Screw Conveyor w/Installation	1 EA	$55,000	$55,000
6	Electrical	1 LS	$160,000	$160,000
7	Covered Storage Area	7,200 SF	$100	$720,000

Subtotal $1,900,000
Contingency (20%) $380,000
Subtotal $2,280,000
Washington State Sales Tax (7.9%) $180,000
Total Estimated Construction Cost $2,460,000
Engineering, Construction Administration (25%) $615,000
TOTAL ESTIMATED PROJECT COST $3,075,000

(1) All costs in 2017 dollars and rounded to nearest $1,000.
(2) Based on FKC Model BHX-900x500L.
(3) Includes HVAC.

If the covered storage area were eliminated, the estimated cost of this option would be $1,908,000.

TABLE 7-22

Belt Filter Press Dewatering System[1]

No.	Item	Quantity	Unit Price	Amount
1	Mobilization/Demobilization	1 LS	$154,000	$154,000
2	Belt Filter Press Equipment Package w/Control Panel w/Installation[2]	1 EA	$360,000	$360,000
3	Piping	1 LS	$75,000	$75,000
4	Building (30' x 50')[3]	1,500 SF	$350	$525,000
5	Shaftless Screw Conveyor w/Installation	1 EA	$55,000	$55,000
6	Electrical	1 LS	$185,000	$185,000
7	Covered Storage Area	7,200 SF	$100	$720,000

Subtotal......$2,074,000
Contingency (20%) $415,000
Subtotal......$2,489,000
Washington State Sales Tax (7.9%)...... $197,000
Total Estimated Construction Cost $2,686,000
Engineering, Construction Administration (25%) $672,000
TOTAL ESTIMATED PROJECT COST......$3,358,000

(1) All costs in 2017 dollars and rounded to nearest $1,000.
(2) Based on BDP Model 3DP 0.75-meter belt press.
(3) Includes HVAC.

Eliminating the covered storage for this option would reduce the total estimated cost to $2,2 million.

TABLE 7-23

Centrifuge Dewatering System(1)

No.	Item	Quantity	Unit Price	Amount
1	Mobilization/Demobilization	1 LS	$140,000	$140,000
2	Centrifuge Equipment Package (includes conveyor and sludge feed pump) w/Control Panel w/Installation(2)	1 EA	$540,000	$540,000
3	Piping	1 LS	$75,000	$75,000
4	Building (25' x 30')(3)	750 SF	$350	$263,000
6	Electrical	1 LS	$160,000	$160,000
7	Covered Storage Area	7,200 SF	$100	$720,000

Subtotal..$1,898,000
Contingency (20%) $380,000
Subtotal..$2,278,000
Washington State Sales Tax (7.9%)... $180,000
Total Estimated Construction Cost $2,458,000
Engineering and Construction Administration (25%)... $615,000
TOTAL ESTIMATED PROJECT COST...$3,073,000

(1) All costs in 2017 dollars and rounded to nearest $1,000.
(2) Based on Andritz D3LL decanter centrifuge.
(3) Includes HVAC.

Eliminating the covered storage for this alternative would reduce the cost to $1.9 million.

OPERATIONS AND MAINTENANCE COSTS OF DEWATERING EQUIPMENT

The evaluation below compares potential operation and maintenance (O&M) costs for three mechanical dewatering systems: a screw press, a belt filter press and a centrifuge. Expected operating data were obtained for three different dewatering systems based on dewatering aerobically digested waste activated sludge.

- FKC Model BHX-900x500L Screw Press
- BDP Model 3DP 0.75-meter Belt Filter Press
- Andritz Model 3DLL Decanter Centrifuge

A comparison of the operating data provided by each manufacturer is summarized in Table 7-24. Where applicable, for further comparison, typical performance levels reported in the literature for dewatering aerobically digested waste activated sludge are shown as well.

Because the hauling and land application cost analysis assumes biosolids production rates of 13,700 gal/day and 2,290 lb/day, the O&M cost evaluation uses the same assumptions. The O&M costs evaluation also assumes operating the equipment five days a week, so the projected weekly sludge production rate of 95,900 gal/week and 16,030 lb/week are translated to daily processing rates of 19,180 gal/day and 3,205 lb/day for a 5-day work week.

TABLE 7-24

Biosolids Dewatering Equipment Operating Data

Dewatering Equipment	Solids Loading Rate (lb/hr)	Hydraulic Loading Rate (gpm for 2% solids)	Discharge Cake Solids (% solids)	Solids Capture Efficiency (%)	Polymer Usage (lb/dry ton of solids)	Equipment Motor Horsepower
FKC Screw Press	388	39	14 – 18	>90%	15 - 20	• 3.0 hp – Screw Press • 1.5 hp – Floc Tank
Screw Press Literature Value	n/a	n/a	n/r	n/r	n/r	n/a
BDP Belt Filter Press	850	64	16 – 20	>90%	12 - 15	• 0.33-hp Paddle Wheel • 1-hp belt drive • 2-hp pressure suction • 2-hp hydraulic power unit • 10-hp wash water pump (75 gpm) • 5-hp feed pump • <1-hp polymer pump
Belt Filter Press Literature Values	n/a	n/a	12 – 25 (M&E)	>90%	4 - 16	n/a
Andritz Centrifuge	510	50	18 – 22	>95%	20 – 28	• 40-hp main drive • 10-hp back drive
Centrifuge Literature Values	n/a	n/a	18 – 25 (M&E)	>95%	20 - 30	n/a

n/a = not applicable because values vary based on equipment size
n/r = not reported
M&E = Metcalf and Eddy, *Wastewater Engineering Treatment and Reuse, 5th Ed.*

Table 7-25 summarizes operating costs based on the manufacturer's operating criteria for throughput rates and the most conservative estimate of polymer usage, along with the aforementioned assumptions for operating hours.

TABLE 7-25

Biosolids Dewatering Equipment Alternatives Preliminary Estimate of Operation and Maintenance Costs(1)

Dewatering Alternative	Equipment Operation (hr/day)	Average Operator Attendance (hr/week)	Annual Labor Cost(2)	Annual Polymer Cost(3)	Annual Power Cost(4)	Other Annual O&M Cost	Total Annual Cost
Basis: Current Average Digested Sludge Production of 834 lb/day (5,000 gal/day 2% solids)							
Screw Press	3.25	5	$9,100	$6,830	$201	$1,000	$17,131
Belt Press	2.00	10	$18,200	$5,123	$571	$3,000	$26,894
Centrifuge	2.50	10	$18,200	$9,563	$1,392	$2,000	$31,155
Basis: Projected BOD_5 Loading of 2,237 lb/day (10,000 gal/day 2% solids)							
Screw Press	8.75	8	$14,560	$9,589	$287	$2,500	$26,936
Belt Press	5.25	16	$29,120	$7,192	$816	$7,500	$44,628
Centrifuge	6.75	16	$29,120	$13,424	$1,989	$5,000	$49,533

(1) In 2017 dollars.
(2) Assumes labor cost of $35/hour.
(3) Assumes polymer cost of $2.25/lb.
(4) Assumes power cost of 7.7¢/kWh. Does not include building HVAC power costs.

The analysis reflected in Table 7-25 considers a mid-range cost assuming:

- 16 percent solids by weight for wet cake with a screw press dewatering system (this has been achieved with recent pilot testing at the Woodland WWTF)
- 7.7¢/kWh
- $2.25/lb polymer
- 20 lb polymer/DT solids processed
- 5 hour/week operator labor initially, 10 hour/week in year 2033 @ $35/hr
- 4.5 hp total motor horsepower to operate screw press

For the period 2017 – 2033, the cost for biosolids management with a dewatering screw press achieving 16 percent solids in the wet cake (including hauling and land application costs) is $2,700,000.

The projected BOD_5 loading for the 20-year planning period is 2,237 lb/day. Assuming the City continues with its present biosolids management arrangement the cost of the operation will exceed $8,000,000 through the end of the planning period.

The contractor that hauls biosolids from the Woodland WWTF has offered a price of $868 per load to haul dewatered biosolids to a beneficial use site in Goldendale, which would apply to the period of mid-October to July. For the summer months, the dewatered biosolids would be hauled to the nearby Woodland beneficial use facility at a cost of $215/load for the period July to mid-October. Under this arrangement, the City would not need a covered biosolids storage facility.

The initial cost of the dewatering system, estimated as $1,908,000 without a year-round storage facility at the WWTF would be recouped in 5.5 years based on the above assumptions.

Installing a mechanical dewatering system will also reduce the City's risk associated with its current biosolids management system, which requires daily hauling of liquid sludge to either a nearby land application site for a short time of the year or a treatment plant that is over 2 hours away for most of the year. If the current arrangement were to cease, the City would have to haul its liquid digested sludge much further at a greater cost. Because the potential long term savings with a mechanical dewatering system are significant and the risks are less relative to the hauling and land application of dewatered biosolids, the City should plan to design and construct a biosolids dewatering system. Initial pilot testing of a screw press indicates that it can achieve relatively high percent solids by weight and based on preliminary operations and maintenance costs, it would be less expensive to operate over the life of the equipment. The local contractor that takes the City's biosolids has offered an arrangement whereby the City does not need to construct an on-site storage facility and will pick up dewatered sludge as needed using a roll-off trailer. This will reduce the construction cost by as much as a third.

The City is currently planning to purchase a screw press that will meet current needs to process between 15 and 20 gallons per minute of digested sludge, which on an 8-hour per day basis can process approximately 7,000 to 10,000 gallons per day of digested sludge per day. The City has contracted with an engineer to do the design and will be submitting an engineering report to Department of Ecology for review and approval separate from this Plan. The current estimated cost for this dewatering system is $700,000.

WATER RECLAMATION AND REUSE EVALUATION

See Appendix L.

SUMMARY

The City of Woodland WWTP is presently operating successfully at loadings well below the permitted and design loading limits. The average BOD_5 and TSS loadings to the plant during the last 5 years represent 44 percent and 55 percent of the permitted BOD_5 and TSS maximum month loadings of 3,107 lb/day and 3,160 lb/day, respectively.

The capacity analysis associated with the evaluation of the liquid-stream components of the WWTP is summarized in Table 7-26. The flow and loading rates projected for the 20-year planning period, through 2033, are less than the currently permitted rates and less than the maximum capacity. It is recommended that the discharge point of the in-plant recycle pump be relocated from upstream of the influent fine screen to downstream of the screen. This change will allow the fine screen to handle the current peak flow of 3.2 mgd, removing a hydraulic bottleneck in the WWTP. There are no recommended improvements to increase the current permitted capacity of the WWTP. Based on nitrification capacity and Ecology criteria for mass loading rates, we recommend that the permitted influent BOD_5 loading rate be reduced from 3,107 to 2,650 lbs/day.

TABLE 7-26

Capacity Analysis Results for Woodland Wastewater Treatment Plant

Parameter	Current NPDES Permit	Projected 2033	Maximum Capacity	Recommended for New NPDES Permit
Influent Flow Rates				
Average Annual Flow	--	0.91	--	--
Maximum Month Flow	2.00	0.96	2.00	2.00
Peak Day Flow	--	1.06	2.47	2.47
Peak Hour Flow	3.20	1.72	3.20(1)	3.20(1)
Influent Loading Rates				
Annual Average BOD_5, (lb/d)	--	2,016	--	--
Maximum Month BOD_5, (lb/d)	3,107	2,237	2,650	2,650
Annual Average TSS, (lb/d)	--	2,135	--	--
Maximum Month TSS, (lb/d)	3,160	2,602	3,160	3,160
Annual Average TKN, (lb/d)	--	373	--	--
Maximum Month TKN, (lb/d)	--	405	500	--

(1) Peak hour capacity will be 3.20 mgd after the in-plant recycle pump discharge is relocated downstream of the influent fine screen; interim peak hour capacity of 2.70 mgd.

There is no compelling basis to recommend immediate improvements to the WWTP to improve energy efficiency. However, different blower configurations would save power. The air demand for the SBRs could be met by operating fewer blowers, with the most energy efficient option being the continuous operation of a single 75-hp blower. This operational mode will also require changes to valve, control panel and electrical settings.

When the existing SBR blowers reach the end of their useful life in approximately 12 to 13 years, it is recommended that the City reevaluate blower selection prior to replacing the SBR blowers. The energy efficiency evaluation found that different types of blower equipment and diffusers would have no substantial cost savings benefit and recommended replacing the SBR equipment with similar equipment at the end of the useful life of that equipment.

The City should also evaluate the performance of the prethickening aerobic digester to determine whether the thickening process can be improved to allow the digester to have sufficient capacity for the planning period. Currently, the digester can only achieve about 2 percent solids and is designed to operate at 3 to 3.5 percent solids. Unless these higher solids concentrations can be achieved in the digester, it will run out of capacity in less than ten years, which will necessitate either the addition of mechanical thickening or increasing the size of the digester basins.

The installation of a mechanical dewatering system to reduce digested sludge volumes was shown to reduce hauling and land application costs by as much as 75 percent. However, the payback period for the least cost dewatering alternative evaluated, a screw press, is on the order of 16 to 17 years. Pilot testing of a mechanical biosolids dewatering system is normally required to obtain accurate operating data from which design criteria can be derived. It is recommended that the City conduct pilot testing with a screw press to verify operating data, including polymer usage, percent solids achieved in the wet cake and the volumetric through-put rate. Once more accurate operating data are obtained from a pilot test, a more detailed evaluation of the potential payback for this alternative can be performed and the City can choose whether to proceed with design and installation for a dewatering screw press.

As noted in Appendix K, at projected future SBR decant flows, there is no reasonable potential for the WWTP effluent to cause an exceedance of water quality standards for ammonia or toxic metals in the Lewis River, which would necessitate new NPDES Permit limits. Measures to avoid future permit limits were discussed.

Per the evaluation presented in Appendix L, water reclamation and reuse is not recommended at this time due to the following factors:

1. Adequate supplies of potable water to meet current and projected non-potable water needs.

2. High capital, operation, and maintenance costs to upgrade the existing WWTP to meet water reclamation and reuse reliability and treatment standards.

3. Uncertainties about water rights impairment if the WWTP discharge is removed from the Lewis River.