

Secondary Clarifier

The existing WWTP has a single secondary clarifier basin. The basin is 32 feet in diameter and has a maximum side water depth of 12.1 feet. The surface loading rate for the secondary clarifier at the average wet weather design flowrate of 0.48 million gallons per day (MGD) is 600 gallons per day per square foot, (gpd/ft²). This is 86% of the accepted loading rate of 700 gpd/ft². The surface loading rate at the peak design flowrate of 1.2 MGD is 1,495 gpd/ft², this is 25% higher than the accepted loading rate of 1,200 gpd/ft². Weir overflow rates at 0.48 MGD and 1.2 MGD are about 4,800 gpd/ft and 11,935 gpd/ft respectively. These are well below accepted loading rates for secondary clarifiers of 15,000 gpd/ft and 30,000 gpd/ft respectively.

Sludge from the SBC/RBC units that settle to the bottom of the clarifier is removed by a 75 gpm Moyno progressing cavity pump located in the blower building and is discharged to the headworks where it mixes with the raw wastewater influent and settles in the primary clarifier prior to being pumped to the aerobic digester. The existing clarifier operates well and does not have serious problems associated with the tankage or the clarifier mechanism. No significant work is required on this unit as part of a WWTP capacity expansion project. The type of work that is recommended to be performed on this unit if the expanded WWTP continues to utilize secondary clarifiers is cleaning and re-coating the tank (both the interior and exterior), and the clarifier mechanism.

Chlorine Disinfection and Contact Basin

Flow from the secondary clarifier passes through the 10-inch effluent line with a Foxboro magnetic flow meter to a manhole containing a chlorine injector diffuser. Following mixing with the chlorine solution, the flow enters the chlorine contact basins which can be operated either independently or in parallel. The contact basins allow the chlorine disinfectant to remain in contact with the treated effluent for an adequate period of time to ensure the number of fecal coliform is below the NPDES effluent limitation. Typically this is achieved if a chlorine concentration of 1 milligram per liter

(mg/l) is maintained for 60 minutes. Both basins are currently utilized at all times to ensure that a 60 minute detention time is provided. The basins are sized to provide 60 minutes of detention time at the AWWF of 0.48 MGD, and approximately 24 minutes of detention time at the PDF of 1.2 MGD. Standard design criteria requires 60 minutes of detention time at the AWWF and 20 minutes at the peak daily design flow.

Because there is a potential to exceed water quality standards for chlorine (see Section III), and because of the extreme danger a chlorine leak would cause to the operators and the community the alternatives will evaluate changing the disinfection method from chlorination to ultraviolet (UV) light. Discussion on these alternatives are included later in this section.

Effluent and Storm Drain System

Disinfected effluent from the WWTP normally flows by gravity to the Lewis River through a 1,050 foot long pipeline. The pipeline consists of approximately 110 feet of 15-inch pipe and 940 feet of 16-inch pipe. When river stage exceeds 18 feet, the disinfected effluent must be pumped. Effluent is pumped through an 8-inch force main to the first downstream manhole on the gravity line at the southeast corner of the dike which surrounds the plant. There is a flap valve on the gravity line entering this manhole. When the pump station is in operation, the flap valve is closed and no flow enters the manhole from the gravity line. The effluent pump station is located at the south end of the chlorine contact basins. The pump station has two 650 gpm pumps which are controlled by a float switch.

If the WWTP remains at its present location, an effluent pump station will continue to be required for those times when the river level exceeds 18 feet. Larger pumps will be required to provide adequate pumping capacity to remove all of the effluent from the upgraded WWTP facility. If an SBR treatment process is utilized the gravity discharge pipeline will need to be upsized to a 24-inch pipe to handle the flow rate of the SBR decant phase.

Aerobic Digester

Primary sludge and secondary sludge are currently blended in the primary clarifier and periodically wasted to the existing aerobic digester for stabilization. Sludge is transferred from the clarifier to the digester by a 75 gpm Moyno progressing cavity pump located in the blower building. The existing digester was converted from a Clarigester package WWTP to an aerobic digester when the secondary treatment processes were constructed in 1974.

The digester tank is 26 feet in diameter with a sidewater depth of 17.5 feet. The floor of the tank is sloped to the center at a 1V:3H slope. Total volume of the tank including the bottom cone is approximately 75,000 gallons. The design hydraulic retention time (HRT) of the digester is 25 days, with a volatile solids (VS) loading rate of 0.074 lbs VS/day/ft³ (which equates to a design loading of 740 pounds of VS per day). Aeration is provided by two positive displacement, rotary lobe blowers each with a capacity of 420 standard cubic feet per minute (scfm) at 8 psi. 385 scfm of air is required, based on an aeration rate of 38.5 scfm/1,000 ft³ of digester volume, to adequately aerate the sludge during digestion. Air is diffused through the digester contents providing both oxygen and mixing. The air is introduced at the bottom of the tank through diffusers, with the rate of air usage controlled by a valve on the north side of the digester tank.

The digester operates on a fill and draw basis meaning that the supernatant and sludge withdrawal pipelines are normally closed except when intentionally opened for withdrawing either of the two components. The feed sludge pumping cycle is operated by a timer and supernatant/sludge withdrawal is done intermittently. Supernatant is returned to the plant headworks. Stabilized biosolids are periodically removed from the digester and land applied in liquid form at a private site located in Cowlitz County.

The monthly averages of the daily volume of sludge transferred to the aerobic digester from July 1997 through June 1998 ranged from 3,156 gpd to 4,283 gpd, with an overall monthly average of 3,495 gpd. Table VII-12 summarizes the solids mass balance information for the past year of operation at the Woodland WWTP.

Volatile solids loading has ranged from 0.055 – 0.085 lbs/day-ft³ during the past year with an average volatile solids loading rate of 0.069 lbs/day-ft³. The loading range is close to the stated design loading rate, and is well below currently accepted design criteria for aerobic digesters of 0.1-0.3 lbs VS/day-ft³. Volatile solids reduction across the digester is below the desired range of 40-50 percent for most of the year. The 503 Regulations require a minimum of 38 percent of VS reduction across the digester to demonstrate vector attraction reduction is satisfactorily met.

**Table VII-12
Woodland WWTP Solids Data from July 1997 through June 1998**

Month	Sludge Wasted from Primary Clarifier					Biosolids Wasted from Digester				
	Digester Feed Sludge (gpd)	% Total Solids (by wt.)	Wt. of Solids (lbs ¹)	VSS:TSS Ratio	Wt. of VSS (lbs ¹)	Sludge Wasted (gpd)	% Total Solids (by wt.)	VSS:TSS Ratio	VSS Reduction Across Digester (%)	Average Digester Temp. (°C)
Jul-97	3,156	3.19	840	0.784	659	6,690	1.39	0.755	15.1	23
Aug-97	3,232	3.41	919	0.788	724	4,471	1.22	0.731	26.9	25
Sep-97	3,621	2.77	837	0.777	650	7,307	1.13	0.739	18.7	25
Oct-97	3,206	2.85	762	0.786	599	7,168	1.17	0.725	28.2	23
Nov-97	3,289	2.83	776	0.794	616	5,400	1.19	0.746	23.8	19
Dec-97	3,263	2.48	675	0.819	553	6,000	1.07	0.746	35.1	17
Jan-98	3,273	2.89	789	0.805	635	5,419	1.09	0.772	18.0	16
Feb-98	3,439	2.61	749	0.813	609	5,271	1.18	0.726	39.1	18
Mar-98	4,283	2.88	1,029	0.814	838	7,471	1.00	0.750	31.4	19
Apr-98	3,981	3.16	1,049	0.814	854	5,760	1.16	0.746	32.9	21
May-98	3,600	3.37	1,012	0.808	818	4,703	1.43	0.708	42.4	25
Jun-98	3,600	2.87	862	0.818	705	4,227	1.44	0.658	57.2	27
Average	3,495	2.94	858	0.802	688	5,824	1.21	0.734	30.7	22

1: Weights are by dry weight

The current HRT in the digester is approximately 21.5 days based on the average daily feed sludge rate. Volatile solids reduction in aerobic digesters has been demonstrated to be primarily a direct function of the digester sludge temperature and the HRT of the digester. EPA's 1979 "Process Design Manual – Sludge Treatment and Disposal" indicates that to achieve 40% VS reduction requires approximately 475 deg. C-days. At

an HRT of 21.4 days the temperature of the sludge in the digester required to achieve 40% VS reduction is 22 degrees C (72 degrees F). The sludge temperature in Woodland's digester ranges from 22-32 deg. C from about May through October, however, it is between 15-20 deg. C (59-68 deg. F) for the rest of the year.

Based on the existing digester's failure to consistently meet sludge treatment regulations and because of the projected waste flows and loads for the planning period covered by this report, Woodland needs to increase solids stabilization capacity. Improvements are required to ensure the City can comply with the *503 Regulations* and the new state biosolids rule adopted in March 1998 by DOE entitled *WAC Chapter 173-308 Biosolids Recycling*. Solids stabilization facilities need to include more than one digestion tank to provide operational and maintenance flexibility. Solids stabilization alternatives that are generally suitable for Woodland are aerobic digestion, autothermal thermophilic digestion, anaerobic digestion, composting, and lime stabilization. Discussion of these alternatives is included later in this chapter.

TREATMENT PLANT IMPROVEMENT ALTERNATIVES

The current characteristics of the influent wastewater to be treated by the City of Woodland's wastewater treatment plant is described in Section V of this report. It consists of municipal wastewater which has an average yearly BOD₅ concentration of 307 mg/l. This is approximately 1.53 times higher than typical municipal wastewater and the original design concentration of 200 mg/l. The wastewater is required by WAC 173-221 and the City's NPDES permit to be treated to technology based effluent limitations as discussed in Section III of this report. Treated effluent from the WWTP is discharged to the North Fork of the Lewis River (Lewis River) at rivermile 6.5. At this location, the discharge must meet water quality standards for several toxicants at the edge of a defined mixing zone as required by WAC 173-201A and discussed in Section III of this report. The discharge also cannot violate Class A water quality standards for pH, temperature, dissolved oxygen or coliform bacteria.

The existing plant's major facility and operational areas which are in need of improvement or increased capacity, to comply with DOE reliability requirements, to ensure current water quality standards are met and that treatment requirements are consistently met in the future, are described below:

1. Additional primary and secondary clarifier capacity is required by DOE to ensure the plant continues to meet requirements even with one unit out of service.
2. Even though the existing plant functions well and does an excellent job of removing both BOD₅ and TSS, DOE has expressed concern that the plant is essentially overloaded at the current flows and loads it is receiving. DOE has specifically stated that they believe the WWTP chronically receives more flow and BOD₅ than the plant was designed to handle. As discussed in Section IV and earlier in this section, the WWTP is not chronically overloaded in terms of its ability to meet its NPDES permit requirements for flow and BOD₅. However, the plant is approaching its actual treatment capacity. For this reason, the community needs to begin designing the recommended WWTP improvements upon approval and adoption of this report by both DOE and the City. The City should also construct the recommended improvements as soon as the design is completed so that the community is assured of having adequate wastewater treatment facilities for the next 10-25 years.
3. Another area where the existing WWTP needs to be improved is in the solids treatment, handling and disposal facilities. The existing aerobic digester is a converted Clarigester package treatment plant and does not have adequate capacity to consistently treat the solids removed from the wastewater as required by: 1) the *40 CFR - Part 503 - Standards For the Use or Disposal of Sewage Sludge, (503 Regulations)*, implemented by EPA in 1993; and 2) the new state *Biosolids Management Rule, (WAC 173-308)*, implemented by DOE in March of 1998. For this reason the existing solids treatment, handling, and disposal facilities are no longer acceptable.
4. A growing area of general concern is regarding the harmful effects of the current method of disinfecting treated effluent, chlorine prior to discharge, utilized by the plant. With greater attention being focused on effluent toxicity, particularly as it relates to salmon and steelhead, traditional disinfection by chlorine is not as acceptable as it has been in the past. Chlorination is still a viable disinfection technique if the residual chlorine is removed from the effluent prior to discharge into the receiving water. It is anticipated that the City's next NPDES Permit will address residual chlorine concentration limits for the effluent if disinfection continues to be accomplished utilizing chlorine.

The items described above represent the major areas which need to be improved to ensure that the City can continue to consistently meet or exceed current and future permit limits. Many alternatives were reviewed to treat the City's wastewater. Alternatives considered include optimizing operation of the existing plant, upgrading the existing plant, and construction of a new WWTP either at the existing plant site or at a new location that would have the capacity and flexibility of assuring the required level of treatment is consistently achieved. These alternatives are discussed below.

Optimize Operation of Existing Plant

Currently the treatment plant operator does an excellent job of operating the plant. More operator time could be spent at the treatment plant, but it is doubtful that any measurable improvement in effluent quality will result simply because there is nothing additional the operator needs to do when the plant is operating within its design loading. During high flow conditions the operator modifies his sludge pumping process to capture his solids and prevent them from washing out of the plant. It may be possible to improve this operation if more resources were available. More work can always be done around the plant area but this does not improve effluent quality. The treatment plant operator could certify himself at a higher level but, again, it is doubtful that any improvement in the quality of effluent would be seen because the current operator is very knowledgeable in all aspects of wastewater treatment. In short, there is little more the operator can do to optimize operation of the plant and produce a higher quality effluent. Certainly optimizing operation of the plant will not avoid the need for major plant modifications.

Upgrading the Existing Plant

The concept of upgrading the existing plant to adequately handle the anticipated design flows for the next 10-25 years was examined. Advantages of this approach include the generally good condition of the existing treatment units, the high quality of treated effluent consistently provided by the existing treatment processes and operator

familiarity with existing treatment processes, lower energy consumption, and ease of operation.

Due to the advantages of this approach, an evaluation of increasing WWTP capacity through the addition of more primary clarifier units, additional SBC units, and additional secondary clarifier units was performed to determine the cost effectiveness of expanding the existing treatment processes.

Drawbacks to this approach include the existing plant location, lack of available land at the existing plant site, and the challenge of incorporating additional treatment units and associated flow splitter structures into the existing plant's hydraulic grade line.

Due to the drawbacks mentioned, the alternative of constructing a new treatment plant at the existing plant site was also examined to determine if a new treatment process could provide the required capacity within the existing plant site and thereby minimize the amount of additional land which may be required. At the City Council's request the alternative of constructing a new treatment plant at a completely new location was also examined as well as evaluating the cost to move the treatment plant so that it could discharge treated effluent directly into the Columbia River.

New Treatment Plant Alternatives

For this report, a sequencing batch reactor (SBR) treatment process was examined to determine if it is cost effective to construct an entirely new secondary treatment WWTP at either the existing plant site or at a completely new site. The SBR process was selected for evaluation for the following reasons: 1) it has been found to be a cost effective treatment process for flows in the range of Woodland's projected flows; 2) it is flexible enough to handle variable flows; 3) it can provide treatment for nitrogen removal (if required in future NDPES permits); and 4) it is operator friendly.

The SBR process is one of many variations of suspended growth biological treatment processes. Suspended growth systems are the most common of the secondary treatment processes in use in the United States today and are expected to remain so as new facilities are constructed to comply with the Clean Water Act requirements. The general process is characterized by wastewater and sludge solids being combined, mixed and aerated in a reactor basin. The process operates using either continuous or batch (plug) influent flow depending on the specific alternative. Contents of the reactor basin are referred to as mixed liquor, and consist of wastewater, microorganisms (living and dead) and inert, biodegradable and non-biodegradable suspended and colloidal matter. The particulate fraction of the mixed liquor is termed mixed liquor suspended solids (MLSS).

After sufficient time for biological processing, the mixed liquor is typically transferred to a separate settling basin (clarifier) to allow gravity separation of the MLSS from the treated wastewater. The settled MLSS is then recycled to the reactor basin to maintain a concentrated microbial population for degradation of influent wastewater constituents. Since microorganisms are continually synthesized in the process, a means must be provided for wasting some of the MLSS from the system. Wasting is generally from the clarifier to a sludge digester, where stabilization or treatment of the solids occurs. A basic suspended growth system consists of several interrelated components:

- A. Aeration basin(s) designed for the type of flow pattern (continuous, plug, or intermediate), and sized to provide a hydraulic retention time (HRT) in the range of 0.5 to 24 hours or more.
- B. An oxygen source and equipment to disperse atmospheric or pressurized air into the aeration basin at a rate sufficient to keep the system aerobic.
- C. A means of mixing the aeration basin contents to keep the MLSS in suspension.
- D. A clarifier to separate the MLSS from the treated wastewater. In an SBR system, mixing and aeration are stopped for a time interval to permit MLSS settling and treated wastewater decanting, thereby eliminating the need for a separate clarifier.

- E. A method of collecting the settled MLSS in the clarifier and recycling them to the aeration basin; however, this is not required in an SBR system.
- F. A means of wasting excess MLSS from the system.

The SBR is a fill and draw system that has gained acceptance in the United States in the past 10-15 years, but which has been used in Europe since the early 1900's. The SBR process utilizes a single, complete-mix reactor in which all steps of treatment occurs. Discrete cycles are used during prescribed time intervals. MLSS remains in the reactor during all cycles, thereby eliminating the need for a separate clarifier. Specific treatment cycles are:

1. Fill (raw or settled wastewater fed to the reactor).
2. React (aeration/mixing of the reactor contents).
3. Settle (quiescent settling and separation of MLSS from the treated wastewater).
4. Draw (withdrawal of treated wastewater from the reactor).
5. Idle (removal of waste sludge from the reactor bottom).

The idle cycle may be omitted by wasting sludge near the end of the react or draw cycles. Due to the batch nature of the process, flow equalization or multiple reactors are required to accommodate the continual inflow of wastewater to the facility.

Advantages of the SBR system include:

1. Elimination of primary and secondary clarifier and return activated sludge (RAS) pumping.
2. High tolerance for peak flows and shock loadings.
3. Avoidance of MLSS washout during peak flow events.
4. Clarification under ideal quiescent conditions.
5. Process flexibility to control filamentous bulking.
6. Minimal operator attention is required.

Another advantage of SBR plants reported by plant operators is the relative ease with which nitrogen removal can be achieved by simple changes to the periodicity and duration of aeration to allow for the necessary nitrification-denitrification steps to occur within the reactor. Some operators report effluent ammonia concentrations of less than 1 mg/l once the correct timing and length of aeration is achieved. Another advantage of SBR plants pointed out by EPA literature is that they can be operated to achieve

nitrification, denitrification, and phosphorous removal without chemical addition simply by controlling the duration and frequency of aeration.

The major disadvantage is the relative lack of long-term operational data due to the short operating history (10-15 years) of the process in the United States. An SBR plant must utilize a minimum of two SBR tanks or a storage (pre-equalization) tank and an SBR tank to accommodate continuous influent flow. Larger SBR plants often need to include post-equalization basins to allow the discharge of treated effluent to a receiving stream to be at a lower flowrate if water quality criteria cannot consistently be met at the SBR decant rate during low flow periods.

Several SBR systems have been constructed in Washington State. Examples of these include the Naselle Youth Camp WWTP operated by DSHS, the Boston Harbor WWTP operated by Thurston County, the McNeil Island Penitentiary WWTP, the Peshastin WWTP operated by Chelan County PUD, and the WWTP upgrade recently completed by the City of Arlington. Inquiries to plant operators have provided positive feedback indicating that if properly designed, the plants are operator friendly and produce a higher quality effluent than required under specific permit limitations.

A modified version of the SBR process is known as the Intermittent Cycle Extended Aeration System (ICEAS). The major difference between the SBR process and the ICEAS process is that inflow and outflow are intermittent (batch) in the SBR system, while inflow into the treatment reactor is continuous in an ICEAS system. The drawback to having continuous inflow in a batch reactor is that partially treated wastewater can leave the tank during the outflow cycle period if any short circuiting occurs across the basin. For this reason most ICEAS systems incorporate a baffle wall to buffer the continuous inflow and minimize short circuiting.

Two important aspects of treating wastewater included in both the treatment alternatives described are disinfection of the treated effluent prior to discharge to the receiving waterbody, and the ability to treat and handle the solids (sludge) removed from the wastewater during treatment. Reuse of treated effluent is also becoming more acceptable as effluent quality continues to improve and more attention is focused on water quantity issues. These three items are discussed in the following sub-sections.

Disinfection

Disinfection is defined as the destruction of pathogenic microorganisms. It does not apply to nonpathogenic microorganisms or to pathogens that might be in the spore state. Historically in the United States, wastewater disinfection has been accomplished almost exclusively by chlorination. Recent studies relating to the generation of undesirable trihalomethanes and other chlorinated organic by-products of the chlorination process have focused more attention on effluent toxicity and water quality characteristics. This has led to traditional chlorine disinfection becoming more and more unacceptable. Chlorination is still a viable disinfection technique if the residual chlorine is removed from the effluent prior to discharge to the receiving water.

Chlorine has been the most widely used disinfectant because it is cheap, effective at a low concentration, and forms a residual if applied in sufficient dosage. It may be applied as a gas (the most commonly used method), or by hypochlorites. Chlorine gas is what is currently used at the existing WWTP.

The use of dechlorinating agents such as sulfur dioxide or sodium bisulfate are being incorporated into the chlorine disinfection process to lower or eliminate the residual chlorine and associated by-products in the effluent prior to discharge. Unlike chlorine disinfection, dechlorination with the above referenced agents is almost instantaneous. A well-blended mixture (before sampling and discharge) of the chlorinated effluent and dechlorinating agent is required for dechlorination to be effective. In the past, it has been almost impossible to reliably measure excess dechlorinating agent concentrations

or "zero" chlorine residual over long periods. Past designs have used various control systems to provide dechlorinating agent concentration control. Design of reliable dechlorination control processes is not a simple task, especially if zero residual is required. The other consideration which cannot be overlooked or overemphasized in either the design or operation of a chlorination-dechlorination disinfection system is safety, due to the hazardous nature of both chlorine gas and sulfur dioxide.

Another method of disinfecting treated wastewater is ultraviolet (UV) disinfection which is becoming more and more common in the United States. It is a physical process in which UV energy is absorbed into the DNA of microorganisms. The DNA then undergoes structural changes which prevent the microorganisms from reproducing. The principal advantage of UV disinfection is that it leaves no residual in the treated effluent to affect aquatic life in the receiving waters. Another advantage of UV disinfection is that contact times are typically one minute or less, compared to a typical contact time of 60 minutes for chlorine disinfection. UV disinfection in addition to being instantaneous and reliable is extremely safe when compared to chlorine gas disinfection.

Other methods of disinfection such as ozone, chlorine dioxide, and bromine chloride were preliminarily examined as part of this study. They were quickly ruled out for the City due to cost and complexity of operation. Chlorination-dechlorination was also eliminated from further consideration due to the less stringent safety requirements associated with UV disinfection. Therefore, UV disinfection is the recommended alternative which is examined in more detail for the City's WWTP facility.

Effluent Reuse

In order for this report to meet the requirements of a General Sewer Plan an analysis for reusing treated and disinfected effluent is required by the Washington State Department of Health (DOH). The reuse alternative evaluated in this report consists of land application of treated effluent, during the dry season, to irrigate poplar trees as a