CHAPTER 7

WASTEWATER TREATMENT FACILITY AND EFFLUENT DISPOSAL EVALUATION

INTRODUCTION

As described in Chapter 4, the City of Bingen wastewater treatment facility (WWTF) consists of a headworks equipped with an automated self-cleaning fine screen, two parallel gravity grit channels and a Parshall flume influent flow meter; an oxidation ditch treatment system with two ditches for biological treatment; and an ultraviolet (UV) disinfection system with horizontal UV lamps. Treated effluent is discharged by a gravity outfall into the Columbia River and an effluent pumping system is available when river levels are sufficiently high that gravity disposal is not possible. Waste solids from the biological treatment process are pumped to a two-cell aerobic digester, and digested solids are dewatered with a centrifuge. Dewatered biosolids are stored in a covered storage area and land applied on agricultural land owned and permitted by others.

Figure 7-1 shows the existing WWTF layout and Table 7-1 provides design criteria for the current treatment facility.

TABLE 7-1

Design Data for Bingen Wastewater Treatment Facility

Treatment Facility Capacity		
Average Design Flow(1)		800,000 gpd
Peak Design Flow(2)		2,000,000 gpd
BOD_5 Loading (maximum month average)		1,311 lb/d
TSS Loading (maximum month average)		1,311 lb/d
Design Population		4,100
Unit Process	**Qty. of Units**	**Description**
Headworks		
Gravity Grit Channel	2	1.5' W x 32' L
Influent Flow Meter	1	6" Parshall flume
Influent Screen	1	self-cleaning auger mechanical screen with 1/4" perforations; 2 hp
Oxidation Ditches		
Older Oxidation Ditch (1972 construction)	1	317,000 gal, trapezoidal channel 6' nominal water depth 7.5' D (total) x 15' W (bottom) x 30' wide (top) x 170' L
Older Oxidation Ditch Rotors	2	14' L cage rotors, 25 hp ea.
Older Oxidation Ditch Mixers	4	Submersible, 4 hp ea.

TABLE 7-1 – (continued)

Design Data for Bingen Wastewater Treatment Facility

Unit Process	Qty. of Units	Description
Newer Oxidation Ditch (1996 construction)	1	400,000 gal, rectangular channel, 10' nominal water depth 12.83' D (total) x 18' W x 156' L
Newer Oxidation Ditch Rotors	2	17' L brush rotors, 25 hp ea.
Newer Oxidation Ditch Mixers	1	Submersible, 5 hp
Newer Oxidation Ditch Floating Aspirating Aerators	2	10 hp
Return Activated Sludge Pumping System		
Pumps	3	non-clog centrifugal, 370–570 gpm, 7.5 hp ea.
Pump Control System	3	magnetic flow meter and VFD control
Waste Activated Sludge Pumping System		
Pumps	2	7.5 hp ea., VFD control, 40–110 gpm
Secondary Clarifiers		
Circular Center Feed	3	40' dia., 13' sidewall depth
Clarifier Drive	3	1 hp ea.
Ultraviolet (UV) Light Disinfection System		
Channels	2	18'-1" L x 6'-9" H x 2'-1/2" W
Modules	4	
UV Lamps	160	horizontal, low-intensity, low-pressure
Effluent Pumping System		
Pumps	3	submersible non-clog centrifugal, 750 gpm, 7.5 hp
Aerobic Digester		
Rectangular Concrete Tanks	2	18.2' D x 35' W x 42' L (ea. tank), 200,000 gal ea. (operating volume)
Aeration System	2	210 cfm blowers, 15 hp ea. fine-bubble diffusers
Mixers	4	submersible (2 per tank), 4 hp ea.
Sludge Dewatering and Storage Facilities		
Centrifuge	1	40 hp, 70 gpm feed rate
Covered Storage Beds	2	38.5' x 48' ea., 3,696 ft^2 total

(1) Average design flow is the term used in the NPDES permit for maximum month flow.
(2) Peak design flow is the term used in the NPDES permit for peak day flow.

BASIS FOR EVALUATION

Table 7-2 shows existing flow and loading criteria and projected year 2022 and 2032 flow and loading criteria developed in Chapter 5.

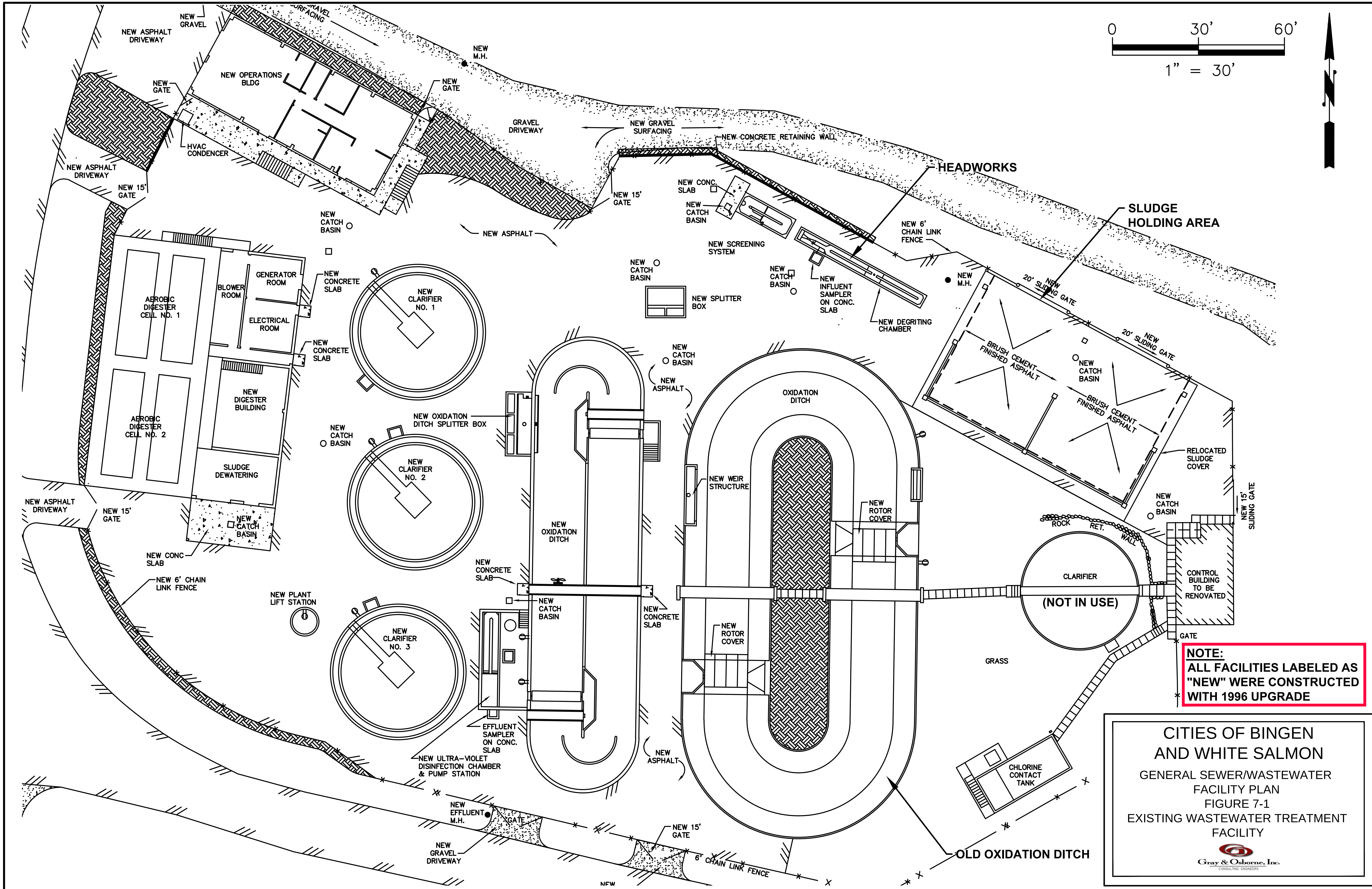

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TABLE 7-2

Current and Projected WWTF Flow and Loading Criteria

Parameter	Existing Design Criteria	Projected 2022	Projected 2032
Total Base Flow	none	0.30 mgd	0.33 mgd
Average Annual Flow	none	0.39 mgd	0.42 mgd
Maximum Month Flow	0.80 mgd	0.55 mgd	0.59 mgd
Peak Day Design Flow	2.0 mgd	1.07 mgd	1.10 mgd
Peak Hour Design Flow	none	2.08 mgd	2.14 mgd
BOD_5 Loading (average annual)	none	859 lb/d	947 lb/d
BOD_5 Loading (maximum month)	1,311 lb/d	1,233 lb/d	1,360 lb/d
TSS Loading (average annual)	none	917 lb/d	1,023 lb/d
TSS Loading (maximum month)	1,311 lb/d	1,394 lb/d	1,554 lb/d
TKN Loading[(1)] (maximum month)	none	247 lb/d	263 lb/d
Design Population	4,100	not applicable	not applicable
Equivalent Residential Units	none	2,649	2,953

(1) Total nitrogen is comprised of organic nitrogen, ammonia, nitrite, and nitrate. Organic nitrogen is determined by the Kjeldahl method. Total Kjeldahl nitrogen is the total of organic and ammonia nitrogen. TKN loadings are used to design and size nitrogen removal processes at the WWTF. The City does not presently measure TKN, however, design criteria needs to be established to determine the capacity of the oxidation ditch to nitrify and denitrify. Typical domestic wastewaters have a 5:1 BOD_5:TKN ratio. Dividing the maximum BOD_5 load of 1,360 lb/d by 5 yields a projected 2032 design TKN loading of 263 lb/d.

A new NPDES permit was issued on May 20, 2014. There were no changes in the effluent limits from the previous permit. These limits are shown in Table 7-3. The proposed flow and loading criteria in Table 7-2 and effluent limits in Table 7-3 will be the basis for the wastewater treatment facility evaluation.

TABLE 7-3

NPDES Permit Effluent Limits for Bingen WWTF(1)

Parameter	Average Month Limit	Weekly Limit
BOD_5	30 mg/L, 197 lb/d 85% removal (minimum)	45 mg/L, 296 lb/d
TSS	30 mg/L, 197 lb/d 85% removal (minimum)	45 mg/L, 296 lb/d
Fecal Coliform	200/100 ml	400/100 ml
pH (2)	≥6 and ≤9	
Parameter	**Average Monthly Limit**	**Maximum Day Limit (3)**
Temperature	23.8 °C	25.0 °C
Total Ammonia (as NH_3-N)	10.0 mg/L	10.2 mg/L

(1) The average monthly and weekly effluent limitations equal the arithmetic mean of the samples taken. The average monthly and weekly limitations for fecal coliform are equal to the geometric mean of the samples taken.

(2) Indicates the range of permitted values. The permittee must report the instantaneous maximum and minimum pH monthly, not average pH values.

(3) The maximum daily effluent limit is defined as the highest allowable daily discharge. The daily discharge means the discharge of a pollutant measured during a calendar day. For pollutants with limitations expressed in units of mass, the daily discharge is calculated as the total mass of the pollutant discharged over the day. For other units of measurement, the daily discharge is the average of the pollutant measured over the day. This does not apply to pH.

Each component of the WWTF and outfall is evaluated below, and estimated costs are presented for recommended improvements to address deficiencies identified in the evaluation. A summary of recommended improvements and their estimated costs is provided at the end of this chapter.

HEADWORKS EVALUATION

As shown in Table 4-3 and Figure 4-4, the existing headworks consists of dual gravity grit channels followed by a Parshall flume flow meter and a mechanical self-cleaning screen with 1/4-inch perforations and integral 2-hp auger screw screenings compactor. A parallel bar screen located in an adjacent channel is used when the automated screen is out of service or the hydraulic capacity of the screen is exceeded.

The grit channels are 18-inches wide and 35-feet long, and flow through each channel has a velocity range of 0.96 to 1.13 feet per second (fps) at the current design flows (0.8 to 2.0 mgd). For horizontal flow gravity grit channels, Metcalf and Eddy's Wastewater Engineering Treatment and Reuse, 4th Edition recommends a range of horizontal design velocity of 0.8 to 1.3 fps, with 1.0 fps considered an ideal velocity.

The projected peak hour flows will cause the velocity in the grit channel to slightly exceed the existing velocity range, but such high peak flows will be of short duration and will not exceed typical ranges for this type of grit removal system, hence no improvements to the grit removal system are recommended.

The 6-inch Parshall flume is equipped with an ultrasonic level measuring system that measures the level upstream of the throat of the flume. The relationship between upstream level, H_a, and the flow, Q, is expressed as follows:

$$Q \text{ (mgd)} = 2.06\ H_a^{1.58} \text{ where the units for } H_a \text{ are feet}$$

For a projected peak hour flow of 2.14 mgd, the H_a measurement will be 1.024 feet, which is within the operating range of the existing flume and level measurement system. Therefore, no improvements to the influent flow measuring system are recommended.

The self-cleaning automated screen has a rated hydraulic capacity of 2.0 mgd. The proposed peak hour flows could slightly exceed the screen's capacity by up to 7 percent, but the parallel bar screen is capable of handling these overflows.

Discussions with the operator indicate that although the automated screen has required periodic maintenance, it is still in good working order. Therefore, no improvements to screen are recommended.

BIOLOGICAL TREATMENT

The biological treatment system consists of two oxidation ditches and three secondary clarifiers. The original oxidation ditch was constructed in 1972 and has two 14-foot-long cage rotors (Lakeside cage rotors) with 25-horsepower motors. The rotors have been subject to several repair efforts. One of these rotors is currently inoperative and the other rotor is functional. The inoperative rotor has a worn shaft that makes long-term operation impossible without further repairs. The original oxidation ditch has a trapezoidal channel cross section and a 6-foot operating depth. It is equipped with four 4-horsepower submersible mixers which allow for the oxidation ditch rotors to be turned off for denitrification using an oxidation reduction potential (ORP) based control system. The ORP system requires new ORP meters to make it operational.

The second oxidation ditch was constructed in 1996 with a rectangular channel and a nominal 10-foot operating depth. It is equipped with two 17-foot-long brush rotors (Lakeside Magna rotors) with 25-horsepower motors. Gray & Osborne completed an evaluation of the oxidation ditch in February 2012 and considered improvements to the biological treatment system, including adding supplemental aeration equipment, to manage the fish processing plant loadings. Recently, in response to the potential for these organic loadings to the WWTF from the Tribal FishCo fish processing plant, the City installed two 10-hp floating aspirating aerators manufactured by Aire-O_2. These aerators are currently operating in the newer oxidation ditch.

In addition to the aeration equipment the newer ditch is equipped with one 5-horsepower submersible mixer and an ORP control system for denitrification and SVI control. The City operates the newer ditch almost exclusively and has only operated the older ditch as a backup when the newer ditch must be taken out of service for maintenance. The City operates the ORP system for denitrification and SVI control. While operation of the biological treatment process with denitrification has many benefits, this operation does potentially reduce the capacity of the treatment facility to remove ammonia (to achieve nitrification) and denitrification is not necessary to achieve the permit effluent limits. This situation will be further explained in the following analysis.

TREATMENT CAPACITY

The City's oxidation ditch capacity will be evaluated based on four different operating modes that could potentially be used:

1. Operating the newer ditch exclusively. The system will be analyzed based on full nitrification and utilizing the ORP control system for partial denitrification.

2. Operating the newer ditch exclusively for full nitrification only, without denitrification.

3. Operating both the newer and older ditch based for full nitrification only.

4. Operating both the newer and older ditch based on full nitrification and utilizing the ORP control system for partial denitrification.

The analyses will be performed at the 20-year projected loadings for BOD_5, TSS and TKN and the permitted flow rate of 0.8 mgd. The City does not want to derate the permitted flow of the facility, and, therefore, the permitted flow of 0.8 mgd will be used in lieu of the 20-year projected maximum month flow of 0.59 mgd.

Scenario 1

The first part of this analysis will consider Scenario 1, as described above, whereby the newer ditch only is operated to achieve treatment for the projected loadings and for the current permitted flow of 0.8 mgd. The newer oxidation ditch is operated with intermittent aeration to accomplish both nitrification and denitrification in a single tank. When the aeration is turned off the tank acts as an anoxic reactor as nitrate is used in lieu of oxygen for BOD_5 removal. The benefits to this operation with denitrification include the reduction in energy due to intermittent aeration, alkalinity recovery, some SVI control and the removal of nitrogen by conversion of nitrates to nitrogen gas.

The City is not required to meet a nitrogen limit, and, therefore, while operation of the ditch in alternating anoxic and oxic modes has benefits to the process, denitrification is not required and operation in this mode reduces the capacity of the biological system to fully nitrify and meet the effluent ammonia limits.

The first step in determining the required design solids retention time (SRT) and treatment capacity for this scenario is to calculate the maximum specific nitrifier growth rate ($u_{n,m}$), decay rate (k_{dn}), and ammonia half saturation coefficient (K_N) using the equations below. The winter design temperature of 10°C is based on historical low temperature data.

$$\mu_{n,m,10} = (\mu_{n,m}) \times (\theta^{t-20}) = (0.75/d) \times (1.072^{10-20}) = 0.374/d$$

$$k_{dn,10} = (k_{dn}) \times (\theta^{t-20}) = (0.08 mg/L) \times (1.029^{10-20}) = 0.060 \text{ mg/L}$$

$$K_{N,10} = (K_N) \times (\theta^{t-20}) = (0.74 \text{ mg/L}) \times (1.053^{10-20}) = 0.442 \text{ mg/L}$$

The numerical values for the various kinetic parameters above are typical for domestic wastewater.

The City must operate its WWTF to remove ammonia to at least an effluent concentration of 10.2 mg/L on a maximum daily basis to meet its WWTF discharge permit. It is difficult to control a process that only partially nitrifies, and, therefore, the assumption in this analysis is that complete nitrification will be achieved, producing an effluent ammonia concentration of 1.0 mg/L or less. This is a conservative approach and will ensure the maximum daily limit can be met. Other assumptions include a dissolved oxygen concentration of 2.0 mg/L and an oxygen half saturation coefficient (K_o) of 0.5 mg/L. The actual nitrifier growth rate is then calculated using the following equation.

$$\mu_n = (\mu_{n,m,10})\left(\frac{N}{K_N + N}\right)\left(\frac{DO}{K_O + DO}\right) - k_{dn,10} = (0.374/d)\left(\frac{1.0}{0.442 + 1.0}\right)\left(\frac{2.0}{0.5 + 2.0}\right) - 0.060/d$$

This equation yields a net specific nitrifier growth rate of 0.148/d, which is then used to calculate the required SRT using the following equation:

$$SRT = 1/\mu_n = 6.8 \text{ day}$$

Applying a safety/peaking factor of 1.5 to this value, to account for daily fluctuations in ammonia loading, produces a required oxidation ditch SRT of 11 days.

The newer oxidation ditch is equipped to employ intermittent aeration to accomplish both nitrification and denitrification in a single tank. At the current flows and loadings the operator partially denitrifies by operating a 5-hour anoxic cycle per day. The time for anoxic and aerobic periods is important in determining the system's treatment performance. The analysis below determines the required anoxic and oxic time required to meet a target nitrogen effluent concentration of 10 mg/L for full denitrification and an effluent ammonia concentration of 1.0 mg/L or less.

The first step in determining the anoxic time is the calculation of the specific denitrification rate (SDNR) with the following equations from Wastewater Engineering Treatment and Reuse, 4th Edition (Metcalf & Eddy, 2003):

$$SDNR = \frac{0.175 A_n}{Y_{net} SRT}$$

$$A_n = 1.0 - 1.42Y + \frac{1.42(k_d)(Y)(SRT)}{1 + (k_d)(SRT)}$$

$$Y_{net} = \frac{Y}{1 + (k_d)(SRT)}$$

Where:

SDNR	= specific denitrification rate, lb NO_3-N/lb biomass d
Y_{net}	= net yield for heterotrophic biomass, g VSS/g bCOD
A_n	= net oxygen utilization coefficient, lb O_2 / lb bCOD removed
SRT	= 11 days (from above)
$k_{d,t}$	= endogenous heterotrophic decay coefficient, d^{-1} = 0.081 / d (see below)
Y	= 0.40 lb/lb bCOD (typical for domestic wastewater)

The values for $k_{d,t}$ can be determined as follows.

$$k_{d,10^0} = (k_{n,\max})(q^{t-20}) = (0.12/d)(1.04^{10-20}) = 0.081/d \quad \text{(typical for domestic wastewater)}$$

Therefore:

$$A_n = 1.0 - 1.42Y + \frac{1.42(k_d)(Y)(SRT)}{1 + (k_d)(SRT)} = 1.0 - 1.42(0.4) + \frac{1.42(0.081)(0.4)(11)}{1 + (0.081)(11)} = 0.70 \text{ lb/lb}$$

$$Y_{net} = \frac{Y}{1 + (k_d)(SRT)} = \frac{0.4}{1 + (0.081)(11)} = 0.211 \text{ lb/lb}$$

$$SDNR = \frac{0.175 A_n}{Y_{net} SRT} = \frac{0.175(0.70)}{(0.211)(11)} = 0.053 \text{ lb } NO_3\text{-N/lb biomass/d}$$

To determine the amount of time the ditch must be operated in an anoxic cycle, the active biomass concentration must be calculated with the following equation:

$$X_b = \left(\frac{Q(SRT)}{V}\right)\left(\frac{Y(S - S_o)}{1 + k_{dt}(SRT)}\right)$$

Where:

- X_b = active biomass concentration, mg/L
- Q = design flow, 0.8 MGD
- V = newer oxidation ditch volume, 0.40 MG
- Y = 0.40 lb/lb bCOD (from above)
- SRT = 11 days (from above)
- $k_{d,t}$ = 0.081/d (from above)
- S = mass of influent bCOD, taken as 1.6 x influent BOD_5 = 2,174 lb/d (326 mg/L bCOD at 0.80 MGD)
- S_o = mass of effluent bsCOD, taken as 1.6 x effluent BOD_5 = 66 lb/d (10 mg/L bsCOD at 0.80 MGD)

Therefore:

$$X_b = \left(\frac{(0.8)(11)}{0.40}\right)\left(\frac{(0.4)(326 - 10)}{1 + (0.081)(11)}\right) = 1{,}470 \text{ mg/L}$$

Using the specific denitrification rate (SDNR) of 0.053 lb NO_3-N per lb·d from above, the biomass is capable of denitrifying the following mass of nitrates:

$$Mass = (SDNR)(X_b)(V)\left(\frac{1lb}{335{,}039mg}\right)\left(\frac{1L}{0.264gal}\right) = \frac{(0.053)(1{,}470)(400{,}000)}{(119{,}748)} = 258 \text{ lb/d}$$

The actual mass of nitrates removed by denitrification depends on the duration of the daily anoxic period in the ditch. Estimating this removal can be determined by calculating that the Bingen WWTF will discharge approximately 32 mg/L of nitrate without denitrification through cell uptake, or about 80 percent of the influent TKN. Assuming total nitrogen is reduced to 10 mg/L through the denitrification process, this effluent concentration results in a daily denitrification of:

$$(32 - 10)(0.80)(8.34) = 147 \text{ lb/d } NO_3\text{-N}$$

The amount of time that the oxidation ditch must be operated anoxically is therefore:

$$\frac{147}{258}\frac{lb/day}{lb/day} \times 24\frac{hr}{day} = 13.6\frac{hr}{day}$$

This anoxic period means that the oxidation ditch would therefore be operated aerobically 10.4 hours per day. Since the SRT of 11 days required for nitrification is based upon 24 hours of aerobic performance, a total SRT based upon 10.4 hours of aeration per day would be required.

$$\text{Total SRT} = \left(\frac{Aerobic\ \ SRT}{Aerobic\ \ hr/d/24}\right) = \frac{SRT_A}{\frac{10.4}{24}} = \frac{11}{\frac{10.4}{24}} = 24.3\ \text{days}$$

This new total SRT of 24.3 days affects the calculation of SDNR. Therefore, an iterative calculation process is performed to determine a new total SRT that is consistent with the design SRT_A of 11 days and the daily aerobic period duration. The iterative process results in a total required SRT of 21 days to achieve full nitrification and target partial denitrification. The design daily anoxic period is 12.1 hours and the aerobic period is 11.9 hours. (Note: If denitrification is not implemented, there would be no anoxic operation, this iterative process is unnecessary, and the SRT would remain 11 days.)

In order to calculate the aerobic mass required for the design SRT, the net sludge production for the treatment system must first be estimated. Assuming a cell yield of 0.4 lb VSS/lb biodegradable COD (bCOD), an influent wastewater and biomass VSS/TSS ratio of 0.85, and a design temperature of 10 degrees C, the total sludge production can be determined using the following equation, where the first term is the heterotrophic cell growth, the second term is the inert cell mass decay product, and the third term is the net nitrifier cell growth:

$$P_X = \left[\frac{(Y)(S - S_0)}{(1+(k_{d,t})(SRT))0.85}\right] + \left[\frac{(f_d)(k_d)(Y)(S - S_0)(SRT)}{(1+(k_{d,t})(SRT))0.85}\right] + \left[\frac{(Y_n)(NO_x)}{(1+(k_{dn,t})(SRT_A))0.85}\right] + X_{iVSS} + X_{iTSS}$$

Where:

P_X	= mass of waste activated sludge per day, lb/d (to be determined)
Y	= heterotrophic cell yield = 0.40 lb/lb bCOD (from above)
Y_n	= autotrophic cell yield = 0.12 lb/lb TKN(typical for domestic wastewater)
S	= 2,174 lb/d (from above)
S_0	= 66 lb/d (from above, assumes 10 mg/L $sBOD_5$ in the effluent)
f_d	= fraction of cell mass remaining as cell debris = 0.15 lb/lb (typical for domestic wastewater)
$k_{d,t}$	= 0.081/d (from above)
$k_{dn,t}$	= 0.060/d (from above)
SRT	= total solids retention time = 21 days (from above)
SRT_A	= aerobic solids retention time = 11 days (from above)

X_{iVSS} = volatile nonbiodegradable solids, volatile suspended solids (VSS) assumed to be 85 percent of influent TSS, volatile nonbiodegradeable solids assumed to be 40 percent of VSS
=0.4 x 0.85 x influent TSS = 528 lb/d

X_{iTSS} = influent nonvolatile suspended solids, assumed as 15 percent influent TSS = 233 lb/d

t = oxidation ditch temperature = 10 °C

NO_x = amount of influent TKN oxidized to nitrate, assumed as 80% of influent TKN = 0.8 x 263 = 210 lb/d

The sludge production can then be calculated as follows:

$$P_X = \left[\frac{(0.4)(2{,}174 - 66)}{(1+(0.081)(21))0.85}\right] + \left[\frac{(0.15)(0.081)(0.4)(2{,}174 - 66)(21)}{(1+(0.081)(21))0.85}\right] + \left[\frac{(0.12)(210)}{(1+(0.060)(11))0.85}\right] + 528 + 233 = 1{,}239 \text{ lb/d}$$

This equation yields a total estimated waste sludge production of 1,239 lb/d. At the design total SRT of 21 days, this waste sludge production results in a required total aerobic mass of 26,019 lbs. With a known oxidation ditch volume of 400,000 gallons, the required MLSS concentration is calculated to be 7,795 mg/L. It is recommended that the oxidation ditches be operated at a MLSS below 5,000 mg/L to prevent solids overload of the secondary clarifiers, sludge bulking (represented by high SVI) due to a low F/M, and lowered oxygen transfer by the aeration equipment at high solids concentrations.

Therefore, operating the newer oxidation ditch exclusively (Scenario 1) does not provide enough capacity to both fully nitrify and denitrify at the 20-year projected loading for BOD_5, TSS and TKN and the permitted flow of 0.8 mgd.

Scenario 2

Scenario 2 involves the use of the newer ditch only, to fully nitrify at the 20-year projected BOD_5, TSS and TKN loading and the permitted flow of 0.8 mgd, without denitrification. In this scenario, aeration is provided continuously, and there is no anoxic period. The recommended design aerobic SRT of 11 days as calculated above would be utilized to achieve full nitrification. This SRT results in a P_x, or waste sludge production of 1,375 lb/d, and a required aerobic mass of 15,125 lbs. This mass results in a required MLSS of 4,324 mg/L, which is within the range of recommended operation concentration.

The City's newer oxidation ditch appears to have the volume to meet the treatment requirements for the projected loadings and rated flow when not operating a denitrification cycle. While this analysis shows that the City's newer ditch has the volume to meet the treatment requirements, the aeration capacity of the newer ditch is yet to be determined and will be estimated below in the aeration section of this analysis.

Scenario 3

Scenario 3 involves operation of both the newer and older ditch while providing full nitrification, but without denitrification. As previously mentioned, denitrification is not required for permit compliance and operation in this mode potentially reduces the capacity of the system to full nitrify.

The calculations used to determine operating characteristics for full nitrification only with two ditches are the same with the exception that the ditch volume is a total of 0.717 mgd in lieu of 0.4 mgd. These calculations show a required MLSS of 2,412 mg/L. This MLSS concentration is well within the typical ranges for oxidation ditch system.

In conclusion, when operating both oxidation ditches without a denitrification cycle the City can treat the projected flows and loadings and meet the required permitted effluent limits.

Scenario 4

Scenario 4 involves operation of both oxidation ditches to achieve full nitrification and partial denitrification. This scenario would allow the City to treat to the permitted effluent requirements and achieve the benefits of operating a denitrification cycle.

In this scenario the same SRT is utilized as calculated in Scenario 1, 21 days. This analysis shows that the MLSS required is 4,379 mg/L, which is within the typical ranges of oxidation ditch operation.

This analysis concludes that when operating both ditches the City appears to have enough volume to provide full nitrification and partial denitrification (to 10 mg/L total N) and achieve the required permit effluent limits. As with Scenario 2, the volume has been proven to be adequate for this mode of operation, but the aeration capacity will be determined below in the aeration section of this analysis.

AERATION REQUIREMENTS

The calculations above only present the portion of the analysis that determines if the oxidation ditches have adequate volume to treat the projected loadings and permitted flow. The system analysis below determines if the existing rotors and supplemental aeration will be adequate for the 20-year planning period for each scenario of operation that was determined as viable. Therefore, the calculations below only show the aeration analysis for Scenarios 2, 3 and 4 from above. As determined above, Scenario 1 did not have enough treatment capacity to both nitrify and denitrify in a single tank, and, therefore, aeration calculations were not performed for this scenario.

Scenario 2

The following analysis is based on Scenario 2, which is the use of only the newer oxidation ditch for full nitrification at the projected loadings and the rated flows, this scenario does not include denitrification.

To biologically oxidize the BOD_5 in the wastewater, oxygen must be continuously added to the oxidation ditch. The required amount of oxygen consists of a carbonaceous oxygen demand and a nitrogenous oxygen demand.

The carbonaceous oxygen demand is calculated as follows:

$$Carbonaceous\,O_2 Demand = S - S_o - 1.42(P_{xbio})$$

Where:

- S = mass influent bCOD, 2,174 lb/d (from above)
- S_o = mass effluent bsCOD, 66 lb/d (from above)
- P_{xbio} = biodegradable biological mass, 0.85((P_X)- X_{iVSS} -X_{iTSS})
  = 0.85((1,375 lb/d - 528 lb/d -233 lb/d) (P_X, X_{iVSS}, X_{iTSS} from above) = 521 lb/d

Therefore, the carbonaceous oxygen demand is 1,369 lb/d. The nitrogenous oxygen demand is calculated by first calculating the amount of nitrogen oxidized to nitrate:

$$Nitrogenous\,O_2 Demand = 4.33(TKN - NH_4 - 0.12(P_{xbio}))$$

Where:

- TKN = influent TKN, 263 lb/d (from above)
- NH_4 = effluent ammonia, 7 lb/d (assumed 1 mg/L concentration)
- P_{xbio} = 521 lb/d (from above)

Therefore, the nitrogenous oxygen demand is 839 lb/d.

Therefore, the total oxygen demand is 2,208 lb/day, as determined below.

Total O_2 demand = Carbonaceous O_2 demand + Nitrogenous O_2 demand
= 1,369 lb/d + 839 lb/d = 2,208 lb/d

Applying a safety factor of 1.3 to account for fluctuations in the diurnal loads results in a design oxygen demand of 2,871 lb/day.

Oxygenation equipment is specified based upon standard oxygen transfer rate (SOTR), the oxygen transfer rate in clean 20 degrees C water with no suspended solids. The SOTR is calculated as follows:

$$AOTR = SOTR\left(\frac{\beta C_{STH} - C_O}{C_{S20}}\right)\left(1.024^{T-20}\right)\alpha$$

Where:

a	= oxygen transfer correction factor, 0.9 (typical for this treatment process)
b	= salinity surface tension factor, 0.95
C_{STH}	= oxygen saturation concentration in clean water, 7.9 mg/L
C_{S20}	= dissolved oxygen concentration at 20 degrees C and 1 atm, 9.08 mg/L
C_O	= operating dissolved oxygen concentration, 2 mg/L
T	= 23 degrees C

The resulting SOTR is, therefore, 3,690 lb/d delivered over 24 hours, or 154 lb/hr.

Based upon manufacturer's information for the two new rotors, the oxygen transfer is 1,877 lb/day (together) or 78 lb/hr. The two aspirating aerators can provide 768 lb/day or 32 lb/hr. The total capacity of the aeration system in the newer oxidation ditch is 110 lbs/hr. The City does not have enough aeration capacity in the newer oxidation ditch alone to achieve full nitrification and meet the permitted effluent limits for both BOD_5 and ammonia.

Scenario 3

For Scenario 3 where both ditches were in operation and only nitrification is provided without denitrification, the oxygen demand is the same as for Scenario 2, 154 lb/hr, but the additional aeration provided by the older ditch rotors meets the aeration demands. The rotors in the older ditch have a capacity of 1,416 lb/day (together) or 59 lb/hr. The additional 59 lb/hr results in a total aeration capacity of 169 lb/day which is adequate to meet the 154 lb/hr oxygen demand calculated above.

Scenario 4

The Scenario 4 was performed to determine if the City has enough aeration capacity to provide full nitrification and operate a denitrification cycle with both ditches in service. This scenario would allow the City to treat to the permitted effluent requirements and achieve the benefits of operating a denitrification cycle.

One of the benefits of denitrification is the use of the oxygen included in the nitrates for BOD_5 removal in the place of free oxygen provided by the aeration system. For each pound of nitrate nitrogen removed, 2.86 lbs of oxygen is produced. This results in an oxygen credit of 420 lb/d (2.86 lb O_2/d*147 lb/d nitrates denitrified = 420 lb/d). In this scenario the total oxygen demand is 2,011 lb/d, and the resulting SOTR is 3,360 lb/d. This oxygen must be delivered in 11.9 hours in lieu of 24 hours due to the operation of the denitrification cycle (12.1 hours anoxic per day) . As a result of the anoxic operation the aeration system must be designed to deliver 282 lb/hr. The existing aeration system does not have enough capacity to meet the City's needs when trying to operate both ditches and utilize the ORP system for a denitrification cycle, consequently the denitrification cycle decreases the treatment capacity of the biological system.

SUMMARY

The City currently uses an ORP control system to operate a denitrification cycle. While there are many benefits to this operation, including some SVI control, the denitrification cycle reduces the capacity of the oxidation ditch system to fully nitrify and meet the permitted effluent limits.

In summary, in order for the City to treat their 20-year projected loadings and the permitted flow of 0.8 mgd to the required permit effluent limits it is necessary to have both oxidation ditches in service. As stated at the beginning of this section the rotors in the older oxidation ditch have been subject to numerous repairs and one of the rotors is inoperable at this time. If the City were to decide not to repair the older ditch, the WWTF could potentially be derated by Ecology because of its inability to fully nitrify and meet the effluent limits. The derating of the facility may result in a sewer moratorium to prevent future growth in both cities. It is recommended that the City address the deficiencies in the older ditch and maintain it in service to enable the City to maintain its rated capacity and support future development in the Cities of Bingen and White Salmon.

The calculations above conclude that the City will only be able to meet the effluent permit limits at the 20-year projected flows and the permitted flow if both oxidation ditches are in service and denitrification is not employed. There may be concerns about some of the benefits that will be lost by not operating a denitrification cycle. One of those benefits is SVI control. SVI control can be better achieved by the construction of bioselectors upstream of the oxidation ditch. This is explained later in this chapter. Another benefit that may be lost is pH control by the recovery of alkalinity during the denitrification cycle, this is further discussed below.

Alkalinity Evaluation

Conversations with the WWTF operator indicate that an alkalinity deficiency was an issue prior to the last WWTF upgrade. The low alkalinity resulted in problems meeting the effluent pH lower limit of 6.

Early in the last decade (2000 – 2002), White Salmon drilled two wells and for a time groundwater, which typically has higher alkalinity than surface water, was the exclusive source for the City of White Salmon's potable water. The City of White Salmon returned to near exclusive use of surface water with the construction of its new slow sand filtration plant on Buck Creek in 2010; however, comparing WWTF discharge monitoring reports (DMRs) from the 2007 – 2008 period to the current period (2010 – 2014), no significant differences in effluent pH have become apparent since the City of White Salmon's new surface water filtration plant went into operation. Following is an evaluation to determine if the influent alkalinity is sufficient for the current operation of nitrification/denitrification.

The stoichiometric reaction for the oxidation of ammonia nitrogen to nitrate shows that two miles of hydrogen are produced for every mole of ammonia nitrogen oxidized. In a wastewater treatment system, these hydrogen ions are neutralized by the wastewater's natural alkalinity (buffering capacity), preventing this acid conditions from significantly reducing the pH within the treatment system. However, if the alkalinity present in the influent wastewater is not sufficient to neutralize the hydrogen ions released during nitrification, the pH within the system will begin to drop. This, in turn, can lead to low mixed liquor and effluent pH and a significant reduction in nitrification efficiency. An effluent pH value below 6 is a permit violation. Mixed liquor with pH readings outside the range of 7.2 to 8.0 can have an inhibitory effect on the nitrifying organisms.

To determine whether the alkalinity in the wastewater is sufficient a nitrogen mass balance must be performed. The first step is to determine how much nitrogen is in the waste cell tissue. The biodegradable biological mass of the was activated sludge was calculated to be 521 lb/d above, assuming that 0.12 lb N/lb of biomass is present results in 62 lb/d of nitrogen present in the waste cell tissue.

The mass of TKN oxidized (nitrification) and the mass of nitrates denitrified must be determined in order to calculate how much alkalinity is consumed and how much alkalinity is produced in the process. Following is the equation to determine the quantity of nitrates denitrified.

TKN Oxidized = $TKN-NH_4-(0.12P_{xbio})$
Nitrate Denitrified = $TKN-NH_4-(0.12P_{xbio})-NO_3-N$

Where

TKN = influent TKN, 263 lb/d, (from above)
NH_4 = effluent ammonia, 7 lb/d, (from above)
P_{xbio} = biodegradable biomass wasted, 521 lb/d, (from above)
NO_3-N= effluent nitrate mass 66 lb/d, (from above based on 10 mg/L in the effluent)

These equations result in 194 lb/d of TKN oxidized to nitrates (nitrification) and 128 lb/d of nitrate denitrified. The amount of alkalinity consumed in the biological processes is calculated as follows:

Consumption = Nitrification (7.14 mg $CaCO_3$) –

(Denitrification)(3.57 mg $CaCO_3$)

= (194 lb/d TKN Oxidized (7.14 mg $CaCO_3$) –
(128 lbs/d NO_3-N Denitrified)(3.57 mg $CaCO_3$)

The total alkalinity consumed is calculated to be 929 lb/d or 139 mg/L at a maximum month flow of 0.8 mgd. An alkalinity of 80 mg/L is requires in the oxidation ditch to maintain a pH of 7.2 the total alkalinity required is 219 mg/L (139 mg/L + 80 mg/ =219 mg/L)

Sampling performed at the WWTF in February of 2015 indicated that the influent alkalinity was above 200 mg/L. This data would indicate that there is sufficient influent alkalinity for the current operation. Historically, the WWTF effluent pH often drops below 7, yet it is rarely below 6.7. This maintenance of a near neutral pH is probably helped by the denitrification provided in the oxidation ditch.

If low effluent pH is a problem in the future due to lack of alkalinity in the raw wastewater entering the plant and the need exists to curtail denitrification as loadings to the WWTF increase, the City could add a small chemical addition system during the times when the biological treatment process may be alkaline deficient, and thereby prevent the effluent pH from dropping below the lower effluent limit of 6.

The estimated cost of a chemical addition system installed by a contractor is $30,000. However, based a review of pH reported in the monthly DMRs for the last four years, it is not anticipated that low pH will be a significant problem in the foreseeable future and any capital investment should be deferred until the need for a chemical feed system is confirmed.

RECOMMENDED AERATION IMPROVEMENTS

The existing aeration system is sufficient in capacity for the 20-year planning period if denitrification is not employed; however the condition of the rotors in the older ditch is very poor. On September 20, 2011, Lakeside Equipment Corporation had a field technician on site to evaluate the older ditch's rotors. The conclusions of the Lakeside field representative were as follows:

> *The Foote Jones reduce on the stub shaft wobbles considerably. A new stub shaft is recommended at a minimum. The covers couldn't be raised so torque tube could be the culprit that causes the wobble. Due to the wobble, the bearings won't last long.*
>
> *When this type of reducer is clamped on the shaft for a long period of time, they seldom come off without damage to the reducer and shaft they are mounted on. On the torque tubes and brackets that hold the paddles, if yearly maintenance of*

> *sand blasting, inspecting and repainting the assembly is not taken, the tube assembly may break if reused. On the old ditch I recommend two new units.*

Lakeside provided budget-level cost estimates for replacing the two 25-horsepower cage rotors on the older ditch with new rotors. As shown in Table 7-4, the estimate for replacing the rotors with two new 25-horsepower Mini Magna rotors is $270,000 including design and construction administration costs.

TABLE 7-4

Preliminary Cost Estimate for Aeration System Replacement on Older Oxidation Ditch

No.	Item	Quantity	Unit Price	Amount
1	Mini-Magna Rotors (25 hp) Installed	2 EA	$85,000	$170,000
2	Electrical and Control Modifications	1 LS	$12,000	$12,000

Subtotal	$182,000
Sales Tax (7.5%)	$ 13,650
Subtotal	$195,650
Contingency (15%)	$ 29,347
Total Estimated Construction Cost	$224,997
Engineering and Administration (20%)	$ 44,999
Total Estimated Project Cost (rounded up to nearest $1,000)	**$270,000**

Repairing the existing oxidation ditch rotors without full replacement is expected to provide a few more years of operating life to the rotors. The City recently replaced the gear reducer on the south rotor of the newer oxidation ditch at a cost of $4,900 (material cost). The operator has previously stated that the gear reducer on the north rotor of the older ditch is near the end of its life. The operator also believes that the inboard bearing on the older ditch south rotor probably also needs replacing. A summary of estimated repair costs for the existing rotors on the older ditch are provided in Table 7-5. Costs shown include materials and labor for a complete installation as well as engineering and construction administration costs.

TABLE 7-5

Preliminary Cost Estimate for Rotor Repairs on Older Oxidation Ditch

No.	Item	Quantity	Unit Price	Amount
1	Replace Stub Shafts on Both Rotors	1 LS	$21,000	$21,000
2	Replace Inboard Bearing on South Rotor	1 LS	$5,000	$5,000
3	Replace Gear Reducer on North Rotor	1 LS	$9,000	$9,000

Subtotal.. $35,000
Sales Tax (7.5%).. $ 2,625
Subtotal.. $37,625
Contingency (25%).. $ 9,406
Total Estimated Construction Cost $47,031
Engineering and Administration (20%).. $ 9,406
Total Estimated Project Cost (rounded up to nearest $1,000).......................... $57,000

The older oxidation ditch has cracks and spalls in the concrete structure that should be repaired to prevent any leakage or further deterioration of the structure. The preliminary estimated cost to repair the concrete structure in the older ditch is $25,000 which includes a 25 percent contingency and 25 percent for engineering and construction administration.

Although the cost of replacing the rotors on the older ditch is more than the repair of the existing rotors, rotor replacement is expected to last far longer than rotor repairs for bringing the older ditch into full service. New rotors will last at least 20 years and the repair work could only provide as little as one to two more years of service before additional repairs are needed. It should also be noted that the cost of replacing the rotors and repairing the spalled concrete on the older ditch is substantially less than constructing a completely new oxidation ditch, which is estimated to be approximately $1.4 million based on actual construction costs for a similar sized oxidation ditch that was recently constructed in the Town of Cathlamet and completed in 2013.

SECONDARY CLARIFIERS

Following biological treatment, mixed liquor from the oxidation ditch flows by gravity into one or more of the three circular secondary clarifiers. The secondary clarifiers provide a quiescent environment where settable solids in the mixed liquor are separated from the flow by gravity sedimentation. Settled sludge is transported by mechanically operated rotating rake arms along the floor of the clarifier to a central hopper. Solids are removed from the hopper for return to the oxidation ditch by means of the return activated sludge (RAS) pump located in the digester building. Scum is pumped from the system by a scum pump located in the digester building. Effluent exits the clarifiers by passing over a weir launder located around the perimeter.

Wastewater Engineering Treatment and Reuse (Metcalf & Eddy, 2003) recommends a maximum surface loading rate of 400 to 700 gpd/ft^2 at a maximum month flow and 1,000 to 1,600 gpd/ft^2 at peak hour flow for properly designed and operated clarifiers. Also, as noted in the Criteria for Sewer Works Design, in order to meet Ecology's reliability standards for reliability class II facility, the facility must have a minimum of two secondary clarifiers, and the secondary clarifiers must be able to be capable of treating 50 percent of the design flow when the largest clarifier is out of service. The Bingen WWTF has three clarifiers of equal size, therefore, in order to meet the redundancy requirements two clarifiers should be able to treat 50 percent of the design flow in the event that a third clarifier is taken from service for inspection, maintenance and repair.

In addition to its recommendation for surface loading rates, Wastewater Engineering recommends solids loading rates of 24 to 36 lb/ft^2/d at a maximum month flow and 43 lb/ft^2/d at peak hour flow. As shown in Table 7-6 below, the secondary clarifiers are capable of meeting these criteria at the projected flows and MLSS concentration of both 2,500 mg/L and 4,400 mg/L (these are calculated in the section regarding biological treatment).

TABLE 7-6

Bingen WWTF Secondary Clarifier Design Analysis

Secondary Clarifiers	
Type	Circular, Center Feed, Center Withdrawal
Diameter	40 ft
Side Water Depth	13 ft
Surface Area, each	1,256 ft^2
50% of Flow (assumes two clarifiers in service)	
MMF Surface Overflow Rate @0.40 mgd	159 gpd/ft^2
PHF Surface Overflow Rate @ 1.0 mgd	796 gpd/ft$^{2`}$
MMF Solids Loading Rate at 0.8 mgd, MLSS 2,400 mg/L	7 lbs/d/ft^2
PHF Solids Loading Rate @ 1.4 mgd, MLSS 2,400 mg/L	11 lbs/d/ft^2

There are no recommended improvements to the clarifiers related to performance or capacity. However, the three clarifiers are equipped with hydrostatic relief valves that diminish the effect of static pressure from high groundwater around the clarifiers and thereby prevent uplift of the clarifier structure when groundwater levels are high and the clarifiers are empty or only partially full. Each clarifier is equipped with four hydrostatic relief valves and the operator has observed that most of the valves are either plugged or inoperable. It is recommended that the City replace all of the valves. The estimated cost to replace the valves is $12,000, which includes 20 percent for engineering and construction administration.

BIOLOGICAL SELECTORS

As BOD_5 and solids loadings to the treatment facility increase above the current design capacity, it will be necessary to maintain higher mixed liquor suspended solids (MLSS) concentrations in the activated sludge process to maintain the required level of treatment, particularly biological ammonia removal. This higher MLSS concentration will have an impact on solids settling and performance of the clarifiers.

Solids settling is measured using a parameter called sludge volume index (SVI). Presently, the operator reports SVIs well in excess of 200 mL/g, indicating solids with poor settling characteristics. An ideal SVI would be under 150 mL/g. To prevent poorly settling mixed liquor solids and provide adequate secondary clarifier capacity, it is recommended that a biological selector (bioselector) be installed. Bioselectors will improve solids settling in the clarifiers as clarifier solids loadings increase and will effectively replace the existing ORP control system for SVI control. The existing ORP control system does not provide SVI control as effectively and continuously as a well-designed bioselector. Temporary measures for reducing SVI include chlorination of the RAS to hinder growth of filamentous bacteria; however, this measure can result in degradation of effluent quality due to high turbidity. Chlorination for SVI control can also contribute to higher effluent ammonia because chlorine will damage the nitrifying bacteria population in the mixed liquor.

Bioselectors are reactors that are designed to create conditions that will favor the growth of floc-forming bacteria over filamentous bacteria, thereby producing sludge with good settling characteristics. The theory behind the use of biological selectors to control the growth of filamentous organisms is linked to bacterial kinetics. Due to their high surface-to-mass ratio, filamentous bacteria are able to more efficiently utilize soluble substrate at lower concentrations than are the non-filamentous bacteria. Therefore, if the food-to-microorganism (F/M) ratio (BOD_5 to active biomass) is low, which is generally the case with oxidation ditches, filamentous bacteria be will favored over floc-forming bacteria, leading to the development of poorly settling sludge. Conversely, although floc-forming bacteria are less efficient under low F/M conditions, they have a higher maximum specific growth rate than filamentous bacteria at high F/M conditions. Consequently, if the available substrate (BOD_5) concentration is relatively high, conditions will favor floc-forming bacteria and filamentous bulking is less likely to occur. Therefore, selector zones will be utilized to create conditions that favor floc-forming bacteria.

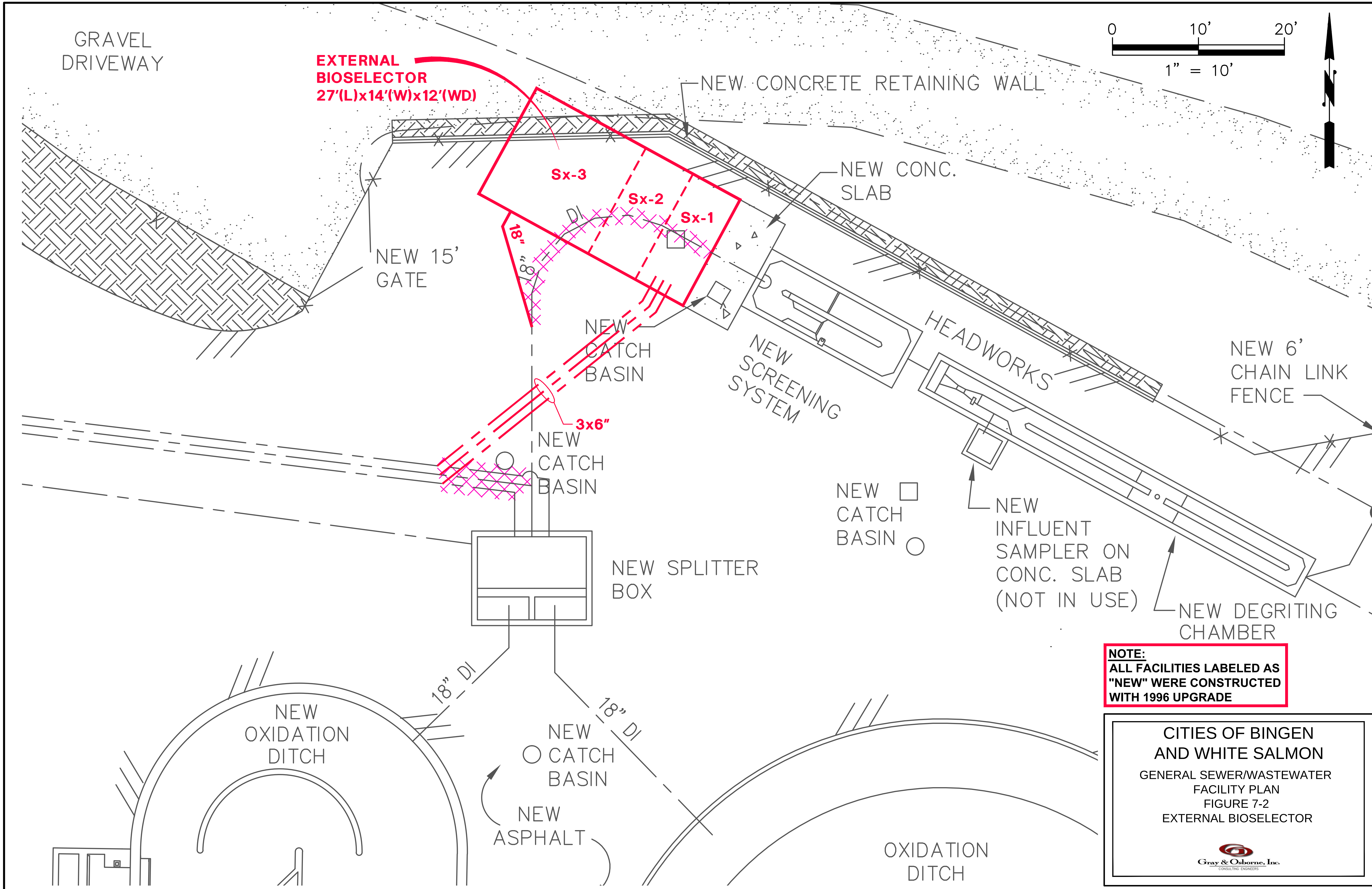
When sizing selector zones, care must be taken to ensure that the F/M ratio is not too high. If the initial contact loadings are too high, viscous or non-filamentous bulking may occur, rendering the selector zones ineffective. Based on past experience, the recommended selector zones would be sized to produce an F/M ratio of 8 lb BOD_5/lb MLSS-d in the first selector compartment.

Two means of achieving biological selection were considered. One would be an external selector that would be installed if both oxidation ditches remained in service. A second alternative considers installing an internal bioselector in the newer ditch if the newer ditch is used exclusively for treatment. Three selector zones (Sx-1, Sx-2 and Sx-3) are recommended for greater effectiveness.

Assumptions for sizing the external bioselector are as follows:

> BOD loading to two oxidation ditches: 1,360 lb/ day
> Design MLSS: 2,400 mg/L (see biological treatment calculations above)
> F/M = 8 lb BOD/ lb MLSS/ day
> lb MLSS = 1360 lb/ 8 = 170 lb MLSS
>
> Volume @ 2,400 mg/L = 170 lb / (2,400) (1×10^{-6})(8.34) = 8,500 gallons
>
> Tank Dimensions:
> Sx-1 & Sx-2: 14 ft x 6.50 ft x 12 ft (WD)
> (based on minimum required volume of 1,135 ft^3 for each selector);
> Sx-3: 14 ft x 14 ft x 12 ft (WD)
> (based on a minimum required volume of 2,271 ft^3 for this selector);
> Mixing: Mechanical mixers or submersible mixers

Figure 7-2 shows the proposed layout for the external bioselector. This facility would be a cast-in-place concrete structure, consisting of three compartments: 6.5' x 14' x 12' (WD) for selector compartments Sx-1 and Sx-2, and 14' x 14' x 12' (WD) for Sx-3; with a design MLSS concentration of 2,400 mg/L and a design F/M of 8 lb BOD/lb MLSS-d for Sx-1. Each compartment would be equipped with a 2.5-horsepower vertical mixer. The estimated cost for a new external selector is presented in Table 7-7.



CITIES OF BINGEN AND WHITE SALMON

GENERAL SEWER/WASTEWATER FACILITY PLAN

FIGURE 7-2

EXTERNAL BIOSELECTOR

L:\Bingen\12226.00 General Sewer Plan Wastewater Facility Plan\Task 001 - WWF-GSP Wastewater Treatment Facility\WWTF Evaluation\Figure 7-2.dwg, 3/12/2015 2:32:22 PM, mnagel

TABLE 7-7

External Bioselector

No.	Item	Quantity	Unit Price	Amount
1	Mobilization and Demobilization	1 LS	$15,000	$16,000
2	Temporary Pumping	1 LS	$5,000	$5,000
3	Trench Safety System	1 LS	$2,000	$2,000
4	Site Excavation, Shoring & Bracing	1 LS	$35,000	$35,000
5	Concrete	56 CY	$1,000	$56,000
6	Piping	1 LS	$11,000	$11,000
7	Submersible Mixer	3 EA	$16,000	$48,000
8	Electrical	1 LS	$27,000	$27,000

Subtotal	$200,000
Sales Tax (7.5%)	$ 15,000
Subtotal	$215,000
Contingency (25%)	$ 53,750
Total Estimated Construction Cost	$268,750
Engineering and Administration (25%)	$ 67,188
Total Estimated Project Cost (rounded up to nearest $1,000)	**$336,000**

If the City were to use the newer oxidation ditch exclusively for treatment a bioselector constructed internal to the existing oxidation ditch tank could be used. Assumptions for sizing the internal bioselector in the newer oxidation ditch:

> BOD loading to newer oxidation ditch: 759 lb/ day
> MLSS: 2400 mg/L (see biological treatment calculations above)
> F/M in SX-1: 8 lb BOD/ lb MLSS/ day
> Tank Volume:
> Sx-1 & Sx-2: 634 ft^3 (4,700 gallons)
> Sx-3: 1267 ft^3 (9,416 gallons)
> Mixing: Coarse bubble air

Figure 7-3 shows a possible configuration for the internal bioselector installed in the newer oxidation ditch. The volume of the first two selector compartments, Sx-1 and Sx-2, would be 634 ft^3 each and the third selector compartment, Sx-3, volume would be 1,267 ft^3. The assumed design MLSS would be 2,400 mg/L and the assumed design F/M would be 8 lb BOD/lb MLSS-d for Sx-1. Mixing for the internal selector would be provided by one 80 scfm, 5-horsepower blower using coarse-bubble diffusers. The cost for installing an internal selector in the newer oxidation ditch is presented in Table 7-8.

TABLE 7-8

Internal Bioselector in Newer Oxidation Ditch

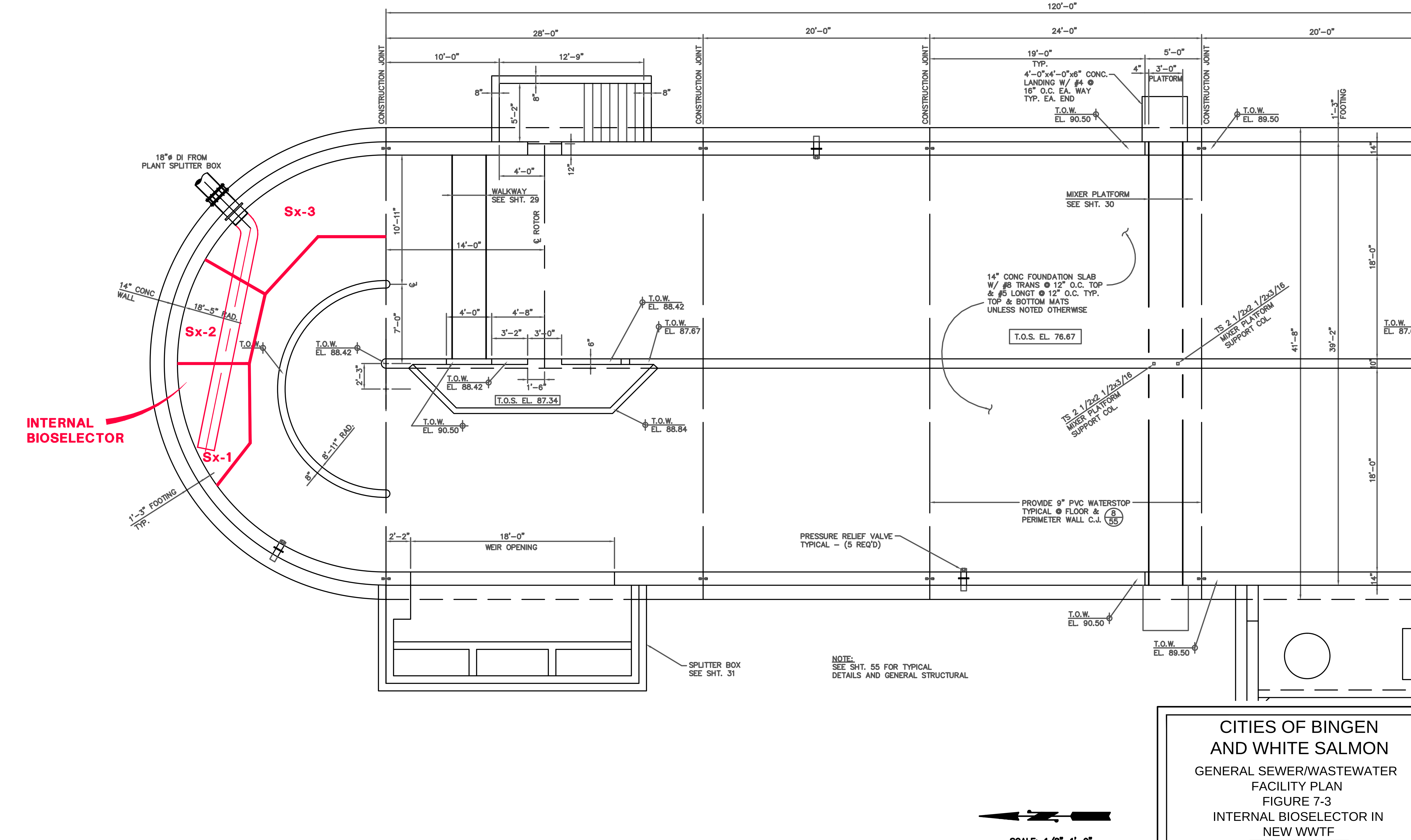
No.	Item	Quantity	Unit Price	Amount
1	Mobilization and Demobilization	1 LS	$7,000	$7,000
2	1-inch thick HDPE Panel	650 SF	$15	$9,750
3	Structural Steel	2,800 LB	$8	$22,400
4	Mixing Blower	1 EA	$12,000	$12,000
5	Mixing Diffusion Equipment	1 LS	$10,000	$10,000
6	Piping	1 LS	$4,000	$4,000
8	Electrical	1 LS	$12,000	$12,000

Subtotal.. $ 77,150
Sales Tax (7.5%)... $.. 5,786
Subtotal.. $ 82,936
Contingency (25%) .... $ 20,734
Total Estimated Construction Cost $103,670
Engineering and Administration (20%).. $ 20,734
Total Estimated Project Cost (rounded up to nearest $1,000)....................... $125,000

Although the external bioselector is more expensive than an internal bioselector in the newer oxidation ditch, it provides bioselection for the entire flow to the treatment facility in the event two oxidation ditches are operational. Therefore, the external bioselector is the recommended alternative for adding bioselection to the treatment process.

ULTRAVIOLET LIGHT (UV) DISINFECTION SYSTEM

Secondary effluent from the clarifiers flows by gravity to the UV disinfection system located adjacent to the oxidation ditch. The system was installed in 1996 and consists of two channels containing two banks of lamps in each channel. Each bank consists of five modules and each module contains eight lamps. The system has a total of 160 lamps. The channels are isolated with stop gates upstream of the lamps. Level is controlled by weighted gates downstream of the lamps. The system control panel is located in the digester building. The control panel indicates the status of each lamp, the run time, and the intensity. The UV modules are removed from the channel for manual cleaning as required. Design criteria for the WWTF disinfection system are provided in Table 7-9.



CITIES OF BINGEN AND WHITE SALMON

GENERAL SEWER/WASTEWATER FACILITY PLAN

FIGURE 7-3

INTERNAL BIOSELECTOR IN NEW WWTF

Gray & Osborne, Inc.
CONSULTING ENGINEERS

TABLE 7-9

UV Disinfection System Design Criteria

UV Disinfection System	
Quantity of Channels	2
Quantity of banks	4 (2 in each channel)
Quantity of Modules per Bank	5
Quantity of Lamps per Module	8
Total Quantity of Lamps	160
Peak Disinfection Flow	2.0 mgd
UV Transmittance	65%
Design Monthly Average Fecal Coliform	200/100 mL
UV Lamp Type	Low Pressure, Low Intensity
Lamp Orientation	Horizontal
Channel Width @ Modules	15 in
Channel Depth	48 in (max ws el. 24 in)

The UV disinfection system was designed to adequately treat 2.0 mgd with one channel out of service. The City's NPDES permit discharge limits for fecal coliform are the same today as they were when the UV system first into operation, 200/100 mL on a monthly average basis and 400/100 mL on a weekly average basis. The UV system has adequate capacity for the 20-year planning period. The operator has not reported any issues with the UV system operation and it has consistently met permit limits for effluent fecal coliform. Therefore, no improvements for the UV disinfection system are recommended at this time.

SOLIDS TREATMENT AND MANAGEMENT

RETURN ACTIVATED SLUDGE PUMPS

Return active sludge to the oxidation ditches from the three secondary clarifiers is accomplished by three return activated sludge (RAS) pumps located in the digester building. Sludge flows from the hopper at the bottom of each clarifier through an 8-inch pipe to the RAS pumps. A piping manifold allows sludge to be withdrawn from any clarifier by any of the pumps.

Each RAS pump is a vertical centrifugal pump. The volume of the sludge removed from each clarifier can be controlled by varying the speed of the RAS pumps, and each RAS pump is equipped with a 6-inch magnetic flow meter on the pump discharge to monitor the rate of return. All three pumps discharge to the oxidation ditch splitter box. Pump speed is manually adjustable at the pump VFD or at the plant control system. The operator has the option of inputting a pump output that is automatically maintained via feedback from the flow meter and setting the RAS flow to be maintained at a percentage of influent flow.

Typical RAS flows for an extended aeration activated sludge process are approximately 50 to 100 percent of the MMF. For the 20-year planning period, 100 percent of the MMF is 0.59 mgd, or 409 gpm, and 100 percent of the rated flow is 0.8 mgd, or 556 gpm. Since each RAS pump has a maximum capacity of 570 gpm, the capacity is more than adequate for the 20-year planning period. No deficiencies have been identified at the RAS pump station.

SLUDGE PUMPS

Each RAS pipe from the clarifier is equipped with a three-way plug valve. During normal operation the valve is positioned so that sludge can flow to the RAS pump. However, during a wasting cycle the valve is positioned so that sludge may flow to one of two waste activated sludge pumps.

The waste activated sludge pumps are located adjacent to the RAS pumps in the digester building. Each of the sludge pumps is a 4-inch double disc, positive displacement type pump with a belt drive. These pumps are used to pump waste sludge from the RAS system, pump digested sludge to the dewatering equipment or the truck fill station, or pump thickened sludge back to the digester.

Each pump is equipped with a variable frequency drive and has a maximum flow rate of 110 gpm. The pumps appear to have adequate capacity for the 20-year planning period and no deficiencies have been identified at this time.

AEROBIC DIGESTER

The waste solids from the oxidation ditch treatment process are pumped to a two-cell aerobic digester for further stabilization to meet Class B requirements for land application of biosolids. The aerobic digester consists of rectangular concrete tanks with 200,000 gallons of operating volume in each cell. Each digester cell has two 4-horsepower submersible mixers to keep solids in suspension during periods when the digester aeration system is turned off. The digester aeration system consists of two 210 scfm, 15-horsepower positive displacement blowers installed in the equipment building and submerged fine bubble diffusers in the digesters. The original fine-bubble diffusers were replaced approximately 8 years ago. One of the digester mixers is inoperable at this time.

The operator has concerns that the digesters may not be receiving sufficient air to properly treat the waste solids due to low dissolved oxygen measurements. The digester was sized to meet the criteria for Class B pathogen reduction using the “time-temperature” method of 40 days mean cell residence time (MCRT, equivalent to Solids Retention Time, SRT) at a minimum temperature of 20 degrees C. This is the method used to meet pathogen reduction requirements for aerobic digestion under WAC 173-308-170(5)(a). The City verifies that the vector attraction reduction standard in WAC 173-308-180 is met by achieving 38 percent volatile solids (VS) reduction in bench tests comparing undigested sludge VS entering the aerobic digester with digested sludge VS leaving the digester.

Digested solids from the aerobic digester are pumped to a centrifuge for dewatering. The dewatered sludge is placed in a covered storage area before hauling away for land application. Bingen currently contracts with Natural Selections Farms to haul its biosolids to wheat and hay fields near Goldendale, Washington for land application. The quantities of biosolids hauled away between 2008 and 2013 are presented in Table 7-10.

TABLE 7-10

Biosolids Production – City of Bingen Wastewater Treatment Facility

Year	Wet Tons	Percent Solids	Dry Tons
2008	177	17.9	31.7
2009	190	15.2	28.8
2010	195	18.2	35.5
2011	172	21.7	37.3
2012	89	21.2	19.39
2013	143.06	21.4	30.61

The City of Bingen is in the process of acquiring a permit from the Washington State Department of Ecology to land apply its biosolids on a farm in Centerville, Washington, that primarily grows wheat and hay. The City plans to start applying biosolids there in late spring or summer of 2014.

The objective of the operation of the digesters is to meet the Class B pathogen reduction requirements using the time and temperature requirements of 40 days at 20 degrees C, in accordance with WAC 173-308-170(6)(a). The operator has expressed concern with the City's ability to meet the time and temperature requirements of 40 days at 20 degrees C.

Per WAC 173-308-170(6)(a), the City can alternatively meet the Class B pathogen reduction method by demonstrating that the digester can hold a temperature of 15 degrees C for 60 days. In the winter the City would have to demonstrate the ability to hold 60 days of sludge in lieu of 40 days. The following calculations indicate that the 200,000-gallon digesters have sufficient capacity for an SRT of 60 days at design loading.

$$\text{Volume} = \frac{Q * X_i}{X\left(k_d * P_v + \frac{1}{SRT}\right)}$$

Where:

Q_i = Influent flow rate, ft3/day
X_i = influent suspended solids, assumed to be 9,000 mg/L
X = digester suspended solids, mg/L, assumed to be 20,0000 mg/L
K_d = Reaction rate, 1/day, assumed to be 0.12/day
P_v = Volatile fraction of digester suspended solids, assuming that 80% WAS is volatile solids and 42% of the volatile solids are destroyed in the digester, 0.70
SRT = solids retention time, days, 60 days

The equation results in a calculated volume required of 157,500 gallons. Since a volume of 157,500 is needed to provide a 60-day solids retention time, the 200,000-gallon digesters have sufficient volume to meet the time and temperature requirement at 60 days, 15 degrees C if operated at 20,000 mg/L.

The operator has indicated that the existing blowers are undersized and the diffusers are not efficient for oxygen transfer due to the low dissolved oxygen measurements. The original predesign report (Gray & Osborne, June 1994) assumed a summer digester solids concentration of 10,000 mg/L and a winter solids concentration of 20,000 mg/L. According to the operator, the digester is operated between 9,000 mg/L to 20,000 mg/L. The following calculations assume that the digester will be operated at 20,000 mg/L in both the summer and the winter, which is the conservative design approach. If the digesters are operated at a lower concentration, the oxygen transfer will be more efficient and provide more treatment, but inadequate SRT may be provided.

Similar to calculations for the oxidation ditch, the digester oxygenation equipment is specified based upon standard oxygen transfer rate (SOTR), the oxygen transfer rate in clean 20 degrees C water with no suspended solids. The SOTR is calculated as follows:

$$AOTR = SOTR\left(\frac{a b C_{STH} - C_O}{C_{S20}}\right)\left(1.024^{T-20}\right)a$$

Where:

a = oxygen transfer correction factor, 0.2 for 20,000 mg/L
b = salinity surface tension factor, 0.95
C_{STH} = oxygen saturation concentration in clean water, 7.9 mg/L
C_{S20} = dissolved oxygen concentration at 20 degrees C and 1 atm, 9.08 mg/L
C_O = operating dissolved oxygen concentration, 2 mg/L
T = 23 degrees C

The resulting SOTR is therefore, 3,518 lb/d, or 185 lb/hr.

The oxygen transfer efficiency (OTE) of new fine bubble diffusers is estimated to be 2 percent per foot. Therefore, at a water depth of 17 feet, this rate provides an OTE of 34 percent.

The air flow required is calculated as follows:

> SCFM = SOR/(60 min/hr)/OTE/(0.0173 lb O_2/SCFM air)
> SCFM required = 1,049 SCFM

The existing 15 hp 420 scfm (210 scfm each) scfm blowers are undersized to supply the required air flow (1,049 scfm) for VSS destruction if the digesters operate in parallel and the solids concentration is 20,000 mg/L. In theory the digesters could be operated at 10,000 mg/L, which would increase the oxygen transfer and reduce the required air flow to 525 scfm. This difference is due to the increase in the alpha factor, which is the oxygen transfer ratio of wastewater to water, at lower solids concentration. Alpha is estimated to be 0.2 for a solids concentration of 20,000 mg/L and 0.4 for a solids concentration of 10,000 mg/L.

If the digesters were operated between 10,000 mg/L and 15,000 mg/L solids concentration, and at design loading, the existing blowers could provide air for one digester and one new blower sized at 420 scfm could supply air for the second digester. The total required air supply would be 840 scfm. However, operating the digesters at a lower concentration will result in additional dewatering required by the centrifuge and will increase the polymer usage. Also, operating at 20,000 mg/L solids will maximize the storage requirements in the winter and provide a 60 day SRT.

Operation of the digesters at 20,000 mg/L results in a total air requirement that exceeds the capacity of the existing blowers, and they would have to be replaced with two new blowers, each designed for 550 scfm, for a total air supply of 1,100 scfm. By providing two new blowers the City will have the flexibility to operate the digesters at a higher concentration, have more than adequate storage in the winter, and maximize the efficiency of their dewatering operation. In addition the existing blowers are 20 years old and they will soon be reaching the end of their useful life.

It is recommended that the City install two new 30-hp 550 scfm blowers equipped with variable frequency drives to replace the existing blowers. The variable frequency drives will allow the blower output to be adjusted to meet the actual need. The recommended approach will provide the City with sufficient aeration to operate the digester at 20,000 mg/L and provide for sufficient treatment to meet the biosolids regulations for Class B pathogen reduction.

It is also recommended that the City replace the existing diffuser system with a 9-inch disc EPDM membrane diffuser system. Preliminary layouts provided by manufacturers for a full floor coverage result in 126 diffusers per tank. The existing diffusers are fine bubble, but old. It may be possible to reuse some of the air piping delivery piping, which can be determined during the design phase.

The recommended air flow for mixing must also be calculated to ensure that there is enough mixing energy to keep solids in suspension. Air flow for mixing is calculated at 0.2 cfm per ft^2, as follows:

$$\text{Air Flow for Mixing} = (0.2\ \text{cfm/ft}^2)(35\ \text{ft} \times 42\ \text{ft}) = 294\ \text{cfm}$$

Since the air flow requirement for sufficient oxygen supply at design level is 1,049 scfm, there is adequate air flow for mixing with both the existing blowers and the recommended larger blowers.

The digester is intended to operate in oxic and anoxic modes, by starting and stopping the blower for controlled periods of time. The digester has controls to operate the blower for a set amount of time and then stop the blower to shut the air off and operate the mixers for an anoxic cycle. The advantages to this operation are not only the energy savings achieved, but operating an anoxic cycle will allow for some alkalinity recovery and a more stable pH. The existing 4-hp mixers provide adequate mixing energy during the anoxic cycle. To maintain the oxic and anoxic operation, it is recommended that the City replace the inoperable mixer.

The WWTF operator reports that the swing joints used in the decanting operation are worn and require replacement. The swing joints allow the operator to raise and lower the decant pipe. It is recommended that the swing joints be replaced when the digesters are out of service.

The dissolved oxygen (DO) concentrations of the digesters should be regularly monitored to help with control of the air supply and to provide the desired DO concentration. Dissolved oxygen concentration is the preferred parameter to measure for digester operation since it is a direct indication of the amount of constituent (oxygen) needed by the aerobic microbes to oxidize the organic material in the waste sludge.

In the past, measurement of dissolved oxygen in aerobic digesters was difficult because of the high solids concentrations and the frequent low DO concentrations, which were difficult to accurately measure using DO probes equipped with diffusion membranes and electrolytes. The mechanism used by these DO probes (diffusion of oxygen through the membrane into the electrolyte and to the electrical anode) results in improved measurement accuracy as the DO concentration increased; however, accuracy at DO concentrations below 0.5 mg/L was poor. Consequently, DO measurement in digesters is not very successful with use of these probes. Recently, newer DO measurement technology using fluorescence radiation has greatly increased DO accuracy at very low concentrations. This technology provides greater accuracy as DO concentration decreases, since the amount of radiation increases as DO decreases. Probes manufactured by Hach (Model LDO) and Insight use this new technology.

It is recommended that this type of DO probe be used to monitor the dissolved oxygen concentration in the aerobic digesters. It is not recommended for blower control at this time. It is also recommended that the blowers be equipped with VFDs for speed control. This will allow the City some opportunity to save energy costs by reducing the speed of the blowers when the basins are operated at lower levels or at lower solids concentrations. Presently when the digesters are at a low level the City does not operate the blowers, or operates them at full speed. This is not a desirable way to operate the digester and the VFDs would give the City maximum flexibility and operational control.

The cost for two new blowers, VFDs, new diffusers, one new mixer, swing joint and dissolved oxygen probes are presented in Table 7-11.

TABLE 7-11

Additional Blower and New Diffusers for Aerobic Digester

No.	Item	Quantity	Unit Price	Amount
1	Mobilization and Demobilization	1 LS	$20,000	$20,000
2	Aeration Blower next to Digester	1 LS	$68,000	$68,000
3	Diffusion Aeration Equipment	1 LS	$45,000	$45,000
4	Piping	1 LS	$13,000	$13,000
5	Swing Joint	1 LS	$10,000	$10,000
6	Dissolved Oxygen Probe	1 LS	$19,000	$19,000
7	Submersible Mixer	1 LS	$35,000	$35,000
8	Electrical (including VFDs)	1 LS	$58,000	$58,000

Subtotal (Rounded) $268,000
Sales Tax (7.5%) .... $ 20,100
Subtotal $288,100
Contingency (25%) .... $ 72,025
Total Estimated Construction Cost $360,125
Engineering and Administration (20%) .... $ 72,025
Total Estimated Project Cost (rounded up to nearest $1,000) $433,000

OTHER POTENTIAL TREATMENT FACILITY IMPROVEMENTS

A review of the WWTF operation and discussions with the operator identified several other issues at the WWTF, including the following:

- Arc Flash Protection on Electrical Equipment
- Automatic Transfer Switch (ATS) for Auxiliary Power Generator

- Circuit Breaker Testing
- Effluent Pumping System (Reliability during a Flood)
- Heat Pump for Operations Building (Efficiency of the Heat Pump)
- RAS/WAS Magnetic Flow Meters
- Stormwater Management
- WWTF SCADA System
- Weir Gate Modifications at Older Oxidation Ditch

Each of these items is discussed below with recommendations for addressing the issue.

ARC FLASH PROTECTION

An arc flash is part of an arc fault, a type of electrical explosion resulting from a low-impedance connection to ground or another voltage phase in an electrical system. A common cause of arc flash injuries happens when switching-on electrical circuits and, in particular, tripped circuit-breakers. A tripped circuit-breaker often indicates a fault has occurred somewhere down the line from the panel. The fault must usually be isolated before switching the power on, or an arc flash can easily be generated. Small arcs usually form in switches when the contacts first touch, and can provide a place for an arc flash to develop. If the voltage is high enough, and the wires leading to the fault are large enough to allow a substantial amount of current, an arc flash can form within the panel when the switch is turned on.

Arc flash protection is required by OSHA and soon will be an NEC requirement (as is selective coordination of overcurrent protection). The City needs an arc flash study, preferably before any improvements are done, both to guide the design engineer and to provide safe working conditions for the City personnel and Contractor construction personnel. With an arc flash study new equipment can be selected to work with the short circuit, selective overcurrent protection and arc flash constraints rather than trying to force the Contractor into making modifications during construction. It is recommended that the short circuit, coordination and arc flash study be done as a part of design. A final version should be done either by the Contractor or the Engineer during construction to incorporate any changes which may occur during construction.

Estimated Cost: $10,000 including sales tax (to be completed during design and construction)

AUTOMATIC TRANSFER SWITCH (ATS) FOR AUXILIARY POWER GENERATOR

The existing automatic transfer switch (ATS) is an older style switch that does not have the features of newer models, including the “neutral delay” feature which is used in most wastewater treatment facilities to allow rotating equipment to come to a stop before transferring from generator to utility power (also is used when transferring from a “hot” utility to generator during “test” mode). The current ATS at the WWTF can create a "bump" that has affected equipment during transfer but, we understand the operator has made modifications to address this issue. As a result (assuming the "neutral delay" is not needed), the existing transfer switch likely has many years of service remaining if it is well maintained and exercised regularly. The ATS could be tested it to verify it functions correctly, but unless the City is not exercising it regularly and it has not had any problems, testing is probably not warranted.

Estimated Cost: $1,000 (for testing prior to design if needed)

CIRCUIT BREAKER TESTING

The critical breakers should be trip tested by current injection if they have not been tested since the WWTF startup in 1996. This will help identify any issues with defective trip mechanisms or "stuck" contacts. This can be done before or during construction and any problems revealed dealt with by City maintenance or change order. The critical breakers should include the mains, feeders, and any "critical" process equipment items.

Estimated Cost: $10,000 including sales tax (based on hiring an independent testing agent during the construction phase)

EFFLUENT PUMPING SYSTEM

The effluent pumping system control panel is located in the UV disinfection and effluent pump system chamber below the 100-year flood plain elevation of 89 feet. The operator is concerned that the pumps would not operate in the event of a flood that exceeds this elevation, which is the elevation of the grade around the effluent pump station and UV system.

According to the Department of Ecology's *Criteria for Sewage Works Design* (Orange Book), the plant unit processes shall be located above the 100-year flood plain or be otherwise protected from a 100-year flood event. Relevant citations from Ecology's Orange Book are provided below.

> ***G2-1.5.2 Flood Protection***
>
> *The plant unit processes shall be located at an elevation which is the 100-year flood/wave action, or shall otherwise be adequately protected against 100-year flood/wave action damage. Newly constructed plants should remain fully operational during a 100-year flood/wave action.*
>
> ***G2-6.3.5 Reliability and Maintenance Considerations/G. Flooding/1. Equipment***
>
> *Wherever possible, electrical equipment should be installed above the maximum flood level. Flooding from any source should be considered, including the*

possibility of piping or structural failure within the facility (such as a piping failure that could flood the dry pit of a pump station).

T3-6 Reliability/T3-6.1 General

In accordance with the requirements of the appropriate reliability class, capabilities shall be provided for satisfactory operation during power failures, flooding, peak loads, equipment failure, and maintenance shutdown. As defined in EPA's publication, "Design Criteria for Mechanical, Electrical, and Fluid System Component Reliability," reliability is "a measurement of the ability of a component or system to perform its designated function without failure... Reliability pertains to mechanical, electrical, and fluid systems and components. Reliability of biological processes, operator training, process design, or structural design is not addressed here."

The cost of moving the pump control panel is estimated to be $30,000 (included 20 percent for design and construction administration costs). Since the top of the existing effluent pump and UV chamber is at 89 feet, which is the 100-year flood elevation, the current design meets the intent of the Orange Book. Therefore, it is not recommended that the City make any changes to the location of the existing electrical components within the UV and effluent pumping chamber.

HEAT PUMP FOR OPERATIONS BUILDING (EFFICIENCY OF THE HEAT PUMP)

The existing heat pump for the operations building is nearly 20 years old and is not as efficient as heat pumps currently on the market.

The current heat pump was installed in 1996, and is a 1995 model with a capacity of 42,000 BTU/hr. It is a Carrier Model 38YKC (outdoor unit) 042300 2-stage heat, 1-stage cool, 15 Kwatt backup strip heat.

A more efficient Carrier heat pump with a 13 SEER efficiency rating (the minimum now allowed) would be a 3.5 ton, model #FB4 CNF042 (indoor unit) / 25HCD342 (outdoor unit) installed with lineset, thermostat and filters. Estimated cost for this unit is $7,000 including tax, if installed by the local Carrier vendor (this cost does not include any electrical improvements, engineering or construction administration costs).
The next more efficient unit is a 15 SEER 3.5 ton, model #FV4 CNF006 (indoor unit)/25HNH542 (outdoor unit). This unit would cost about $10,000 including tax, if installed by the local Carrier vendor (no engineering or construction administration costs are included).

If the City chooses to replace the heat pump at the Operations Building, the 13 SEER unit is recommended since the payback period (20 to 25 years) is likely more than the life of the machine. Per discussions with the local vendor (Oregon Equipment Company, 1505 West First Street, The Dalles, OR 97058) energy savings would be on the order of $125 to $150 per year. The payback does not include any potential rebates from the PUD for installing a more efficient unit.

Estimated Cost: $7,000 including sales tax (does not include engineering or electrical improvements – this cost assumes the City contracts directly with the local vendor to replace the heat pump).

RAS/WAS MAGNETIC FLOW METERS

The recycled activated sludge (RAS) and waste activated sludge (WAS) pumping systems have magnetic flow meters associated with their operation. Four of the five RAS/WAS flow meters are inoperative. It is recommended that all five flow meters be replaced since they are all nearly 20 years old and in deteriorated condition.

Estimated Cost: $34,000 ($6,800 each, includes 20 percent for engineering and construction administration costs).

STORMWATER SYSTEM

Stormwater runoff within the WWTF is currently directed to catch basins that drain to the plant drain pump station, which sends the runoff back to the headworks for treatment in the WWTF. The operator is concerned that these storm flows add unnecessary hydraulic loads to the WWTF. Because the WWTF is not hydraulically overloaded at the present time, nor is it expected to be during the planning period, this arrangement does not impact the WWTF's ability to operate effectively. Therefore, no changes are recommended to the existing arrangement of collecting stormwater runoff within the WWTF and treating it and disposing with the WWTF effluent.

WWTF SCADA SYSTEM IMPROVEMENTS

The City intends to confer with its integrator to develop a plan and budget for improving the existing SCADA system.

WEIR GATE MODIFICATIONS ON OLDER OXIDATION DITCH

This work would involve fabricating a weir plate and welding it to the existing gate on the outlet of the older oxidation ditch to allow the liquid level in the ditch to be raised and allow greater immersion of the rotors. Greater immersion would allow the rotor aerators to achieve a higher oxygen transfer rate and this work is a recommended improvement.

Estimated Cost: $4,000 (includes sales tax, 25 percent contingency and 20 percent for engineering and construction administration costs).

OUTFALL EVALUATION

OUTFALL HYDRAULIC ANALYSIS

The effluent from the UV disinfection system normally flows by gravity to the City's WWTF outfall located in the Columbia River at river mile 170.2. The wastewater outfall is 18-inch ductile iron (DI) between the effluent pump chamber and the effluent manhole, 16-inch concrete for approximately 1,000-feet downstream from the outfall manhole and 16-inch HDPE for 850 from the end of the concrete pipe to the terminus in the river. The effluent pumping system is designed to pump effluent to the Columbia River at river elevations up to 89 feet (100-year flood elevation).

Record drawings indicate the outfall terminates approximately 1,000 feet from the shore of a small inlet near the western edge of Bingen Lake. The outfall terminus is not equipped with a diffuser and the terminus is located approximately 450 feet from the nearest shoreline under normal river conditions.

The current design flow of the WWTF is 0.80 mgd and the peak design flow is 2.00 mgd, which correspond to the maximum monthly average and peak day flows, respectively. As shown in Table 7-2, the year 2032 average annual flow and peak day flow are projected to be 0.42 mgd and 1.10 mgd, respectfully. The year 2032 peak hour flow is projected to be 2.14 mgd.

Results of the hydraulic analysis are summarized below:

Assumptions:

Maximum water surface elevation in pump chamber, all conditions:	81.33 ft
Average River Elevation	76.5 ft
100-Year Flood Elevation	89.0 ft
Available Static Head, Gravity Flow:	81.33 - 76.5 = 4.78 ft
Static Head, Pumped Flow:	89.0 – 81.33 = 7.67 ft
8-in DI pressure pipe Cl 53, Inside Diameter:	8.20"
Ball Check Valve Losses: From Flygt literature	
18-in DI Cl 52, Inside Diameter:	18.55"
1,000 LF 16-Inch Reinforced Concrete Pipe, Inside Diameter:	16.00"
850 LF HDPE, Inside Diameter:	DR 17: 14.00"
	DR 11: 12.92"
Gravity Entrance Head Loss Coefficient, K (at wet well):	0.78
Tideflex TF-2 Valve Losses: From Red Valve literature	
Effluent MH Head Loss Coefficient, K (In+Out):	1.0/0.5
Outfall Exit Head Loss Coefficient, K:	1.0
Friction Factor:	C=120

Maximum gravity and pumped flow rates were calculated with different assumptions for the HDPE pipe dimension ratio (DR) since the actual DR of the HDPE pipe segment is unknown:

Gravity Flow, assuming 850 LF of IPS DR17 HDPE: 1,275 gpm; 1.84 mgd
Gravity Flow, assuming 850 LF of IPS DR11 HDPE: 1,150 gpm; 1.76 mgd
Duplex Pump Flow, assuming 850 LF of IPS DR17 HDPE: 1,640 gpm; 2.36 mgd
Duplex Pump Flow, assuming 850 LF of IPS DR11 HDPE: 1,600 gpm; 2.30 mgd

With the most conservative DR (DR11), the pumped flow rate (assuming two pumps operating) exceeds the year 2032 projected peak hour flow. The maximum gravity flow rate for an assumed DR of 11 exceeds the projected peak day flow for year 2032. Therefore, the existing effluent pumping system and outfall is adequate for projected flows within the planning period.

OUTFALL DILUTION ANALYSIS

An evaluation of the existing outfall used to discharge treated effluent into the Columbia River was performed to determine if the projected flows and loadings would have any impact on existing effluent limits for regulated constituents. A detailed evaluation of the analysis is included in Appendix K and is summarized here.

Gray & Osborne consulted with Ecology's Central Regional Office and Ecology recommended modeling the outfall discharge using WWTF design flows of 0.8 mgd average annual flow and 2.0 mgd peak day rather than the predicted effluent flow rates to calculate critical condition dilution factors. In each case, the design flows for annual average and peak day exceed the effluent flow rates predicted through the 20-year planning period. These slightly design higher flows would result in lower dilution factors and are, therefore, more conservative for purposes of modeling and predicting future effluent limits.

The outfall analysis indicates that there is not a reasonable potential to exceed water quality criteria for the following regulated parameters:

- Ammonia
- BOD5
- pH
- Dissolved Oxygen
- Temperature

Therefore, no changes to the existing discharge permit limits are expected.

The analysis also concluded that the current effluent limit for ammonia in the NPDES permit for the WWTF is protective of water quality and may be more conservative than is actually required. However, the WWTF analysis assumes the necessary solids retention time (SRT) in the oxidation ditch treatment system needed to achieve complete nitrification and this conclusion regarding the effluent ammonia limit would not impact the manner in which the WWTF is operated.

Table 7-12 summarizes key conclusions of the outfall analysis.

TABLE 7-12

Results of Effluent Outfall Analysis

Parameter	Effluent Flow Rate (mgd)	Effluent Flow Rate (cfs)	Chronic Dilution Factor w/25 Percent of Critical Flow	Chronic Dilution Factor Calculated from CORMIX	Chronic Dilution Factor from Current NPDES Permit
Maximum Month Design Flow	0.80	1.24	16,473	143.3	59.1
Peak Day Design Flow	2.00	3.09	6,611	**92.1**	**--**[(1)]
Parameter	**Effluent Flow Rate (mgd)**	**Effluent Flow Rate (cfs)**	**Acute Dilution Factor w/2.5 Percent of Critical Flow**	**Acute Dilution Factor Calculated from CORMIX**	**Acute Dilution Factor from Current NPDES Permit**
Annual Average Design Flow	0.80	1.24	1,648	**11.1**	**--**[(2)]
Peak Day Design Flow	2.00	3.09	664	11.4	4.0

Parameter	Projected Limits		
	Maximum Daily	**Weekly Average**	**Monthly Average**
Total Ammonia[)]	29.9 mg/L	N/A	14.9 mg/L

(1) The current NPDES Permit calculated the chronic dilution factor with the maximum average monthly effluent flow of 0.80 mgd.

(2) The current NPDES Permit calculated the acute dilution factor with the maximum daily effluent flow of 2.00 mgd.

The effluent outfall analysis indicates that ammonia does not show a reasonable potential to exceed water quality standards at current effluent concentrations. Additionally, this analysis indicates that an ammonia limit for the Bingen WWTF is not needed to meet water quality criteria in the receiving water.

WATER REUSE EVALUATION

INTRODUCTION

This section presents a brief evaluation of the feasibility of reusing effluent from the City of Bingen WWTF. Chapter 2, in part, covered regulations concerning water reuse.

The Washington State *Water Reclamation and Reuse Standards* define four classes of reclaimed water (Classes A, B, C, and D) distinguished by treatment technologies and the final bacterial concentration. Class A reclaimed water, the highest classification, is generally required for uses with potential for public contact, such as would be encountered in the two Cities. Under RCW 90.46, Class A reclaimed water means reclaimed water that, at a minimum, is at all times an oxidized, coagulated, filtered, disinfected wastewater with total coliform bacteria density as the disinfection standard.

To meet Class A reclaimed water standards, the facility effluent must be coagulated and filtered in order to meet a continuous turbidity standard. Reclaimed water must be disinfected to meet a coliform bacteria standard that is much stricter than the standard for secondary effluent. In addition, reclaimed water processes must meet the reliability and redundancy requirements in the state standards and this requires added monitoring and controls that are not provided with a conventional wastewater treatment facility. The City of Bingen currently does not coagulate or filter its effluent prior to disposal and is not equipped with the redundancy features and alarms to meet Class A reclaimed water criteria.

The City of Bingen WWTF is permitted for discharge to the Columbia River based on limits established for BOD_5, TSS, ammonia, and fecal coliform. The City can meet its NPDES discharge limits with its existing treatment system technology, yet it will not meet requirements for reclaimed water since it neither coagulates or filters its effluent and does not disinfect to the State's reclaimed water standards which require 5-log virus inactivation when UV disinfection is used as the disinfection process.

POTENTIAL FOR WATER REUSE

Potential uses of reclaimed water for the two Cities are limited and would be challenging and costly to implement, but issues associated with wastewater reuse and possible beneficial uses are discussed below.

Water Rights Issues

The two Cities currently have sufficient water rights to meet their joint needs within the current 20-year planning period. Because existing water rights are sufficient, use of reclaimed water would have a minimum impact in offsetting water rights limitations.

Additionally, the City of Bingen would need to perform a water rights impairment analysis to discontinue discharging any or all of its existing treated effluent into the Columbia River. This analysis would need to demonstrate that withdrawing any portion of the effluent currently discharged into the River would not have a detrimental impact on the intended uses for the river. Current uses of the Columbia River include recreation, fishing and navigation. The analysis would have to demonstrate that loss of the effluent discharged from the WWTF does not impact these uses.

Irrigation/Landscaping Use

Potential uses of reclaimed water include irrigation of school grounds, parks, and golf courses. In every case, due to the potential for public contact, the reclaimed water would need to be treated to Class A standards, and the WWTF would need to be upgraded to provide the treatment and operational reliability levels necessary to meet the Class A standard.

Husum Hills Golf Course is a nine-hole course and the only golf course in the immediate vicinity of the Bingen WWTF. It is located approximately 9 miles, via SR 14 and SR 141, from the Bingen WWTF. There are several parks in the area that could be irrigated with reclaimed water. The school complex in north White Salmon has athletic fields that could utilize reclaimed water for irrigation as well. Table 7-13 lists estimated potential reclaimed water usage for irrigation in the area.

TABLE 7-13

Potential Reclaimed Water Usage

Irrigation Area	Distance from WWTF (miles)	Irrigated Area (acres)(1)	Annual Usage (gal/year)(2)	Peak Day (gpd)(3)
Husum Hills Golf Course	8.7	37.7	22,520,201	8,189,164
White Salmon Parks/Athletic Fields				
White Salmon Athletic Fields	3.4	13.2	7,885,057	2,867,293
Rhinegarten Park	2.1	2.6	1,553,117	564,770
Jewett Creek Park	1.8	1.6	955,764	347,551
Bingen Parks				
Daubenspeck Park	0.7	3.8	2,269,941	825,433
Marina Park	0.3	1.5	1,075,235	390,995
Totals	n/a	**60.4**	**36,259,315**	**13,185,206**

(1) Estimated from aerial photos.
(2) Annual irrigation usage based on 22 inches per year over a 4-month irrigation season, per Washington State Irrigation Guide, Goldendale location.
(3) Peak day irrigation usage based on an irrigation demand of 8 inches in June, per Washington State Irrigation Guide, Goldendale location.

According to the Water Reclamation and Reuse Standards, Class A reclaimed water is required for irrigation of public areas. The estimated peak day demand is 13.18 mgd, about forty times the current annual average flow through the WWTF and sixteen times more than the rated capacity of the WWTF. Therefore, the irrigation demands would exceed the amount the WWTF could produce on a daily basis, unless the Cities constructed reclaimed water storage systems.

Commercial/Industrial Uses of Reclaimed Water

Winter water use represents non-irrigation water requirements. Winter water use by industries and commercial water customers averaged 26,200 gpd in White Salmon between 2009 and 2012. During the same period commercial water use in Bingen averaged 16,100 gpd and industrial water use averaged 51,000 gpd. A significant percentage of the commercial water usage is for food preparation and food processing, which would not be suitable for reclaimed water applications. SDS Lumber uses water for its boilers and reclaimed water could potentially offset some of their water use, however, additional treatment would be required for boiler uses because reclaimed water is high in TDS, which creates scale build up in boilers. SDS Lumber also uses water for spraying unprocessed wood and this practice could potentially use reclaimed water. However, SDS Lumber supplements potable water purchased from the City of Bingen with its own alluvial wells, and it is unclear how much of the water that used for non-potable uses could be offset by reclaimed water when SDS Lumber does not have to purchase water that is used in its industrial applications.

Cost of Developing a Reclaimed Water System

The components of the reclaimed water system that produces water suitable for public contact (Class A) would include the following at the Bingen WWTF:

- Secondary Effluent Pump, Valves, Piping and Controls (to transfer water from the clarifiers to a new coagulation/filtration system).

- Coagulation and Filtration System Valves, Piping and Controls (would need to be housed in a building for weather protection, filtered effluent would return to the inlet of the UV reactor).

- Upgraded UV System including New Control System (to meet Class A disinfection standards for 5-log virus inactivation and redundancy requirements).

- New Reclaimed Water Pump System (size and pump head would need to be determined after laying out the reclaimed water distribution system).

- Reclaimed Water Distribution System (piping to distribute to all reuse sites; the golf course was not included since reclaimed water demands are expected to exceed availability).

Estimated costs for each of these components are provided in Table 7-14. Engineering costs are expected to be higher as a percentage compared to more conventional treatment and pipeline projects and include the cost of a water rights impairment analysis. The treatment components are based on a 2.0 mgd capacity to manage the peak day flow.

TABLE 7-14

Reclaimed Water Treatment and Distribution System[(1)]

No.	Item	Quantity	Unit Price	Amount
1	New Secondary Effluent Pumping System (includes new piping, valves and electrical/controls)	1 LS	$150,000	$150,000
2	Coagulation and Filtration System in a Climate Controlled Weather Tight Building (includes all piping, valves and electrical/controls and HVAC)	1 LS	$1,500,000	$1,500,000
3	Reclaimed Water Pump Station (includes new piping, valves and electrical/controls)	1 LS	$150,000	$150,000
4	Upgrade UV System to Class A Standards (includes electrical and new controls)	I LS	$500,000	$500,000
5	RW Distribution System for Bingen	1 LS	$750,000	$750,000
6	RW Distribution System for White Salmon	1 LS	$2,500,000	$2,500,000
7	Additional Instrumentation, PLC and HMI Programming for Monitoring and Alarms	1 LS	$150,000	$150,000

Subtotal..........$5,700,000
Sales Tax (7.5%)..........$ 427,500
Subtotal..........$6,127,500
Contingency (25%)..........$1,531,875
Total Estimated Construction Cost..........$7,659,375
Engineering and Administration (25%) [(2)]..........$1,914,845
Total Estimated Project Cost (rounded up to nearest $1,000)..........$9,575,000

(1) Mobilization/demobilization costs are built into the individual line item costs.
(2) Engineering and construction administration costs reflect the complexity of the project.

As Table 7-14 indicates the investment in a reclaimed water treatment, pumping and distribution system is substantial. Not included are the additional operations and maintenance expenses associate with the two new pump stations, treatment system, chemical coagulant costs and maintenance of the reclaimed water distribution system.

Given the estimated magnitude of the costs and lack of any real demand for reclaimed water to offset non-potable water needs in Bingen and White Salmon, a reclaimed water project is not recommended for the two cities at this time.

CONCLUSIONS AND RECOMMENDATIONS

The existing WWTF requires improvements to provide treatment capacity and operational reliability necessary to meet current effluent standards for both existing and future flows and loadings.

Recommended WWTF improvements listed by priority are as follows:

1. Older Oxidation Ditch - Replace the aeration system with new rotors, construct limited structural repairs, and modify the weir gate

2. Clarifiers and RAS/WAS Pumping System – Replace hydrostatic valves and replace RAS/WAS meters

3. Aerobic Digester – Install new blowers, new diffusers, new dissolved oxygen probe and replace the worn decanter swing joints and one inoperative mixer.

4. Install an external bioselector.

5. Safety Improvements to the WWTF – Provide arc flash protection, circuit breaker testing and ATS testing

6. Operations Building – Replace existing heat pump for improved energy efficiency

Table 7-15 presents preliminary estimated costs for the improvements. Unless otherwise stated these costs include 7.5 percent sales tax, 25 percent contingency and 20 percent for engineering and construction administration. Estimated engineering and construction administration costs included 25 percent for the older oxidation ditch structural repairs and the external bioselector. It should be noted that these preliminary estimates do not include any costs for providing additional hardware, new software or any modifications to or reprogramming of the existing WWTF SCADA system.

TABLE 7-15

Preliminary Cost Estimates for Recommended Wastewater Treatment Facility Improvements[1]

No.	Item	Estimated Cost
1A	Replace Rotors on Older Oxidation Ditch	$270,000[2]
1B	Structural Repairs to Older Oxidation Ditch	$25,000[3]
1C	Outlet Weir on Older Oxidation Ditch	$4,000
2A	Clarifier Hydrostatic Valves	$12,000
2B	Replace RAS/WAS Flow Meters	$34,000
3	Aerobic Digester Improvements	$433,000
4	External Bioselector	$336,000[3]
5A	Arc Flash Protection	$10,000[4]
5B	Automatic Transfer Switch Testing	$1,000[4]
5C	Circuit Breaker Testing	$10,000[4]
6	Replace Heat Pump by Local Vendor	$7,000[4]
Total		**$1,142,000**

(1) All costs include 7.5 percent sales tax, 25 percent contingency and 20 percent for engineering and construction administration unless otherwise noted.
(2) Includes 15 percent contingency.
(3) Includes 25 percent for engineering and construction administration.
(4) Include 7.5 percent sales tax, does not include engineering or construction administration costs.