Sizing Headworks

A new grit removal system is recommended for the upgraded WWTP. The existing grit removal system is nearly 25 years old and the operator reports that a significant amount of time is spent manually removing grit from the grit chamber. The existing grit chamber is approximately 8.5 feet wide x 11 feet long x 10 feet deep and has a usable volume of approximately 5,240 gallons. At the Phase I peak daily flowrate of 1.6 MGD, the unit will provide 4.72 minutes of detention. This value is in conformance with standard design criteria which indicates that 2-5 minutes of detention time should be provided for aerated grit chambers at the peak hourly flow rate. Based on this, the grit removal chamber is adequately sized to provide grit removal for the Woodland WWTP until Phase II improvements are required if new mechanical equipment consisting of a new grit pump, cyclonic separator and grit classifier are supplied and installed as part of the Phase I improvements. Since the existing aerated grit removal facility was constructed utilizing Dorr Oliver mechanical equipment, the new pump, cyclonic separator and grit classifier should be specified as Dorr Oliver. The upgraded aerated grit removal system will remove 65 mesh or larger grit.

Another type of grit removal system which has been reported to work well utilizes a vortex flow pattern. The Pista grit removal system manufactured by Smith & Loveless was reviewed for possible application at the Woodland WWTP. Wastewater enters and exits the grit removal unit tangentially. A rotating turbine maintains a constant flow velocity, and adjustable pitch blades promote separation of organics from the grit. The action from the rotating turbine produces a toroidal-flow path for grit particles. Grit settles by gravity into the hopper in one revolution of the basin's contents. Solids are removed from the hopper by a vacuum primed turbo grit pump. Grit removed by the pump can be discharged to a hydroclone for removal of the remaining organic material. This type of grit removal system can remove 100 mesh and larger grit. For Phase I, a Pista Model 2.5 would be required to remove grit from a peak daily flow of 1.6 MGD. The peak design flow for the unit is 2.5 MGD.

It is recommended that for the Phase I improvements that the existing aerated grit removal system be upgraded with new mechanical equipment if the existing headworks continues to be utilized. This will be significantly cheaper than installing an entirely new system, such as the Pista system. If a new headworks facility is constructed as part of Phase I improvements the City should strongly consider installing a new grit removal system such as the Pista system. Capacity can be increased in the future by installing a second unit as part of the Phase II improvements.

Primary Clarifier Sizing

A second primary clarifier providing primary treatment capacity of 1.0 MGD needs to be constructed as part of the Phase I improvements if the existing treatment process continues to be utilized. Due to the excellent performance of the existing clarifier in removing settleable solids from the influent flow the new clarifier would basically be a replica of the existing primary clarifier. The new primary clarifier will be a 28 foot diameter basin, with a side water depth of 10 feet. At the MMF to each clarifier of 0.5 MGD, the surface loading rate will be 812 gpd/ft^2 and the weir loading rate would be 5,682 gpd/lf. Detention time in the new clarifier tank at the MMF would be 2.2 hours. A second primary clarifier unit would provide the WWTP with Class I Reliability by providing 50% of the total design flow if the other clarifier is out of service. At the PDF to each clarifier of 0.8 MGD, the surface loading rate would be 1,300 gpd/ft^2 and the weir loading rate would be 9,091 gpd/ft. The new clarifier should be a center feed unit and should be designed in accordance with or nationally recognized design references.

When the Phase II upgrade is required, two additional primary clarifiers would also need to be constructed. These units would be mirror images of the clarifier constructed as part of Phase I in size, volume, and operation. Four primary clarifiers would provide the WWTP with 2.0 MGD of primary treatment capacity, and the required redundancy of 50% capacity with two units out of service.

A flow splitter box would also need to be constructed as part of the Phase I improvements to split the total raw wastewater flow between the existing and the new clarifier unit. The splitter box should be a cast-in-place concrete structure with slide gates and weirs. In addition the box should be constructed so that when the Phase II clarifier units are constructed flow can be split evenly to the four units.

Submerged Biological Contactor Sizing

Additional SBC capacity is required as discussed earlier in this chapter to provide for both the Phase I design capacity of 1.0 MGD and the Phase II capacity of 2.0 MGD. Utilizing the Phase 1 BOD_5 influent load of 1,986 lbs/day of total BOD_5 from Table VII-23, and 33% total BOD_5 removal across the primary clarifiers, the total BOD_5 load to the SBC units is 1,324 lbs/day. Utilizing a soluble BOD_5:total BOD_5 ratio of 0.6 the soluble BOD_5 to the SBC units is 794 lbs/day.

Utilizing the projected Phase II BOD_5 influent load of 3,107 lbs/day of total BOD_5 from Table VII-22, and 33% total BOD_5 removal across the primary clarifiers, the total BOD_5 load to the SBC units is 2,071 lbs/day. Utilizing a soluble BOD_5:total BOD_5 ratio of 0.6, the soluble BOD_5 to the SBC units is 1,234 lbs/day. Table VII-24 shows how many SBC units are required to provide enough surface area to stay within accepted aerated biological contactor loading rates based on the existing and projected characteristics of the influent wastewater for both the Phase I and Phase II BOD_5 loads.

Table VII-24
Design Capacity of Existing & New SBC Units Operated in Parallel

Total First Stage Area (1,000 Ft^2)	Total System Surface Area (1,000 Ft^2)	Allowable First Stage Total BOD_5 Loading (lbs/day)	Allowable First Stage Soluble BOD_5 Loading (lbs/day)	Allowable Total System Total BOD_5 Loading (lbs/day)	Allowable Total System Soluble BOD_5 Loading (lbs/day)
Total and Soluble BOD_5 Capacity of Two SBC Units					
297	571.4	1,723	1,040	1,429	857
Total and Soluble BOD_5 Capacity of Three SBC Units					
445.5	857.1	2,514	1,559	2,143	1,286
Total and Soluble BOD_5 Capacity of Four SBC Units					
594	1,142.8	3,445	2,079	2,857	1,714

1: Loading is based on aerated unit loading rates from Table VII-1.
a) 5.8 pounds total BOD_5 per 1,000 ft^2 of first stage surface area
b) 3.5 pounds soluble BOD_5 per 1,000 ft^2 of first stage surface area
c) 2.5 pounds total BOD_5 per 1,000 ft^2 of total system surface area
d) 1.5 pounds soluble BOD_5 per 1,000 ft^2 of total system surface area

As shown in Table VII-24, two SBC units would be required to have enough media surface area to comply with recommended loading rates for projected Phase I soluble BOD_5 loading. Three SBC units would ultimately be required to provide adequate capacity for projected Phase II soluble BOD_5 loading.

For redundancy it is recommended that the City install two additional SBC units as part of Phase I and a fourth SBC unit as part of Phase II if the existing treatment process continues to be utilized. In addition to redundancy the extra unit constructed during each phase would provide capacity for limited nitrification during normal operations.

To assure that the projected Phase II loads are achieved the City should continue to aggressively enforce their sewer billing ordinance as it relates to large dischargers. This will ensure high strength dischargers pay for their impact on the City's WWTP capacity and will provide an incentive for these type of dischargers to implement effective pre-treatment systems. As further incentive to have high strength dischargers implement pre-treatment the City should perform an industrial/commercial wasteload survey in accordance with DOE recommendations, work with DOE to develop a pre-treatment ordinance and strongly enforce it.

Secondary Clarifier Sizing

For the Phase I upgrade, another secondary clarifier is needed to provide full treatment capacity to 1.0 MGD. The new clarifier would be a replica of the existing secondary clarifier which is a 32 foot diameter basin, with a sidewater depth of 12 feet. At a MMF to each clarifier of 0.5 MGD the surface loading rate will be 995 gpd/ft^2 and the weir loading rates will be 7,921 gpd/ft. Detention time provided by each tank at the average design flow is 2.15 hours. Class I Reliability will be provided by the second secondary clarifier and the second primary clarifier. If one of the secondary clarifiers is out of service, one of the primary clarifiers will operate as an emergency secondary clarifier. This will provide service for more than 75% of the total design flow. The new clarifier should be a center feed unit and be designed in accordance with nationally recognized design references. A flow splitter box would also need to be constructed to split the total raw wastewater flow between the two clarifier units. The splitter box should be a cast-in-place concrete structure with slide gates and weirs.

When the Phase II upgrade is required, two additional secondary clarifiers would need to be constructed. These units would also be mirror images of the existing clarifier in size, volume, and operation. Four clarifier units would provide the WWTP with 2.0 MGD of secondary clarification capacity and meet Class I Reliability criteria. Based on the layout of the existing plant and the land constraints associated with expanding the WWTP at the present site, construction of three new secondary clarifiers is recommended as part of Phase II. This would allow for the demolition of the existing secondary clarifier and provide adequate space for construction of the fourth SBC unit.

In addition, the splitter box constructed during Phase I should be built so that when the Phase II clarifier units are constructed, the flow can be split evenly to the four units with minimum operator attention.

Sequencing Batch Reactor Sizing

Sizing sequencing batch reactor (SBR) treatment plant components was based on the design waste flow and loading values and the desired effluent concentrations of 30 mg/l or less for BOD_5 and TSS as discussed in this report. All components were sized based on completely mixed activated sludge design guidelines and from correspondence with SBR equipment manufacturers.

The Phase I SBR treatment process was sized utilizing dual SBR basins with each basin operating through five cycles per day for all flows up to the projected PDF. Each cycle duration would be 288 minutes (4.8 hours). For the first 144 minutes of each treatment cycle an SBR basin would be in a fill phase. Following the fill phase the SBR would continue to aerate, settle, and then decant. These three phases (which would also include sludge wasting) total 144 minutes. Since the SBR basin would not be filling during the aerate, settle and decant cycles the second SBR basin would be receiving all of the influent flow.

The Phase II SBR was sized utilizing the Phase I dual SBR basins and incorporating a third SBR basin into the treatment process for increased system flexibility. Each basin would operate through five cycles per day for all flows up to the projected PDF. Each cycle duration would be 288 minutes (4.8 hours). For the first 96 minutes of each treatment cycle an SBR basin would be in a fill phase. Following the fill phase the SBR would continue to aerate, settle, and then decant. These three phases (which would also include sludge wasting) total 192 minutes. Since an SBR basin does not receive any influent flow during the aerate, settle and decant cycles the other two SBR basins receive all influent flow.

The alternating of fill and non-fill phases between the SBR basins would allow the plant to continually receive influent flow without pre-equalization storage. The SBR basins are sized to provide 12. 5 hours of HRT at the Phase II MMA of 2.0 MGD. The design decant rate for the SBR system is 3,333 gpm (4.8 MGD) for both Phase I and Phase II. Water quality modeling has verified that year round discharge to the Lewis River at the identified decant rate will not violate water quality standards, therefore no post equalization is required for either Phase I or Phase II.

Table VII-25 presents a summary of the recommended SBR design criteria. The recommended design criteria is slightly higher in terms of influent BOD and TSS loading than the design criteria developed in Chapter V and summarized earlier in this section. This is due to the fact that the recommended design criteria is based on the actual capacity of the proposed treatment processes rather than the projected influent loadings. This will provide the City of Woodland capacity in the event that flows/loads increase quicker than anticipated. The other factor in utilizing the actual capacity of the treatment process is that there is no increase in capital cost over sizing the plant to adequately handle the projected influent flow/load conditions since the basin sizes and all of the mechanical equipment does not change.

Table VII-25 Design Criteria for Proposed SBR WWTP		
	Phase I- 2 SBR Basin Sizing	*Phase II- 3 SBR Basin Sizing*
Dimensions of each SBR Basin[1]	73' x 49'	73'x 49'
Minimum Volume/SBR Basin	347,852 gallons (13' SWD)	347,852 gallons (13' SWD)
Variable Volume/SBR Basin	214,063 gallons (8' SWD)	214,063 gallons (8' SWD)
Total Volume/SBR Basin	561,915 gallons (21' SWD)	561,915 gallons (21' SWD)
HRT for MMA[2]	16.7 hours	12.5 hours
Design Low Water MLSS Concentration	4,500 mg/l	4,500 mg/l
Biological Mass in SBR Basins	26,113 lbs	39,169 lbs
Allowable Influent BOD Loading	2,475 lbs/day	3,720 lbs/day
Allowable Influent BOD Concentration for ADWF	464 mg/l	348 mg/l
Food:Microorganism Ratio	0.095	0.095
Solids Retention Time	13.3 days	13.3 days
MLVSS:MLSS Ratio	0.73	0.73
MLVSS Concentration	3,285 mg/l	3,285 mg/l
WAS solids generated	1,634 lbs/day	2,455 lbs/day
WAS solids concentration	0.5-1.0%	0.5-1.0%
WAS volume to digesters	19,590-39,180 gal/day	29,433-58,866 gal/day
Lbs WAS / lb BOD_5 applied	0.66	0.66
Aeration required oxygen transfer	300 lbs/aeration hour/basin	300 lbs/aeration hour/basin
Blower Size Required for aeration	2-75 Hp units[3]	4-75 Hp units[3]
Aeration Diffuser System	Coarse Bubble	Coarse Bubble
Mixing Power Required/Basin[4]	1-20 Hp turbine mixer	1-20 Hp turbine mixer
Decant Rate	3,333 gpm (4.8 MGD)	3,333 gpm (4.8 MGD)
Sludge Pump Equipment/Basin	1-3Hp submersible pump	1-3Hp submersible pump
Treatment Plant Process Control	Programmable PLC	Programmable PLC

1: Basin size based on 5 cycles/basin/day each cycle lasting 4.8 hours for all flows up to PDF. If flows exceed design PDF cycle times would be shortened by float switch control.
2: HRT based on total minimum volume of all SBR basins divided by MMA of 1.0 MGD for Phase I and 2.0 MGD for Phase II.
3: Phase I requires a 3rd blower unit, and Phase II requires a 5th blower unit for redundant capacity.
4: Mixing power estimated based on 40 Hp/MGD at minimum volume and 30 Hp/MGD at maximum volume.

Since the various SBR equipment manufacturers generally have different optimum basin configurations, the equipment supplier is often pre-selected or determined prior to detailed design of the basins and yard piping. Alternatives which are commonly employed for competitive selection of the SBR process equipment include the following:

1. Pre-purchase of equipment utilizing competitive bids and performance based specifications.
2. Evaluated bids based on overall life cycle costs.
3. Common tank construction.

It is recommended that either procurement alternative 1 or alternative 2 be utilized by the City of Woodland to select the SBR process equipment prior to final design of the WWTP. SBR basin and equipment sizing and cost estimates for the SBR alternative evaluated in this section are based on utilizing equipment manufactured by Aqua-Aerobic Systems, Inc.

Disinfection Equipment Sizing

There are two distinct types of UV disinfection systems that are suitable for disinfecting treated wastewater. The first is the open channel type which uses low pressure, low intensity bulbs. The bulbs are arranged in a rack formation suspended from the side walls of an open channel. The bulbs are installed parallel to the flow. Most of the open channel systems (with capacities of 1 MGD or less) in the wastewater industry are prefabricated turnkey systems complete with lamp rack, stainless steel channel and control panel. It is also possible to make use of existing chlorine contact channels by installing racks of lamps within the channel. This usually requires additional concrete work to construct a channel that the selected rack assembly would fit into. The prominent manufacturer of the open channel type systems is Trojan Technologies, Inc. (Trojan) of London, Ontario, Canada.

The other type of UV system for wastewater applications is the closed conduit type. Closed conduit type systems utilize medium pressure, high-intensity bulbs which are positioned within a piece of stainless steel pipe which the flow passes through. Lamps are horizontal and are positioned perpendicular to the flow. Closed conduit type systems use about 25% as many bulbs as open channel type systems, however, medium pressure bulbs are not as energy efficient as low pressure bulbs. The prominent manufacturer of the closed conduit type system is Aquionics of Erlanger, Kentucky.

Both types of UV systems would work at the Woodland WWTP. Preliminary pricing information was obtained from both manufacturers referenced above to develop planning level costs. Closed conduit systems are available with capacities of 1.2 MGD 2.1 MGD, and 2.6 MGD. Budget prices on units of these capacities are estimated at $70,000 each for the 1.2 MGD units, $80,000 each for the 2.1 MGD units, and between $90,000-95,000 each for the 2.6 MGD units. The closed conduit system would be installed in the effluent piping. The pipeline would be split into two pipeline branches each passing 50% of the total flow. Each pipeline branch would incorporate a closed conduit UV unit. This would provide for redundancy and would allow one unit to be removed from operation during low flow periods. The pipeline branches could be passed through the existing chlorine contact chamber with access provided for operators to perform routine maintenance on the equipment. A sump pump would be required to transfer washdown and rainwater entering the basin back to either the plant headworks or the SBR basins. The empty weight of the larger UV units is 264 pounds therefore a lifting hoist would need to be provided to allow the removal of the equipment from the basin if necessary. Estimated installed costs for the sump pump and hoist system is $10,000.

Open channel equipment is available with capacities from 0.5-20 MGD. Open channel equipment with a 2.0 MGD capacity for effluent from an SBC/RBC process was estimated at $150,000 for planning level cost estimates. Open channel equipment with a 4.8 MGD capacity for batch effluent from an SBR process was estimated at $120,000 for planning level cost estimates. The difference in equipment cost is due to differences in the size of solids particles contained in typical SBC/RBC effluent compared to typical activated sludge effluent. SBC/RBC effluent typically has a significant percentage of solids particles that are larger than 30 microns and can often have a lower UV transmissivity (clarity) than activated sludge effluent. When larger solid particles are present or the effluent has a lower transmissivity UV equipment manufacturers generally increase the amount of light recommended due to concerns that typical ultraviolet light levels may not adequately penetrate the solids. If the ultraviolet light is blocked or shielded due to the presence of solid particles the effectiveness of disinfection can be dramatically reduced. For this reason 10 samples of existing secondary clarifier effluent were obtained from Woodland's WWTP between January 11, 1999 and January 15, 1999 and submitted to Trojan for preliminary UV transmissivity testing and determination of the mean particle size. Transmissitivy was determined on both unfiltered and filtered specimens of secondary clarifier effluent. As shown in Table VII-26 the existing SBC/RBC treatment process is producing a good quality effluent with a high percentage of transmissivity. Due to the mean particle size being about 50 microns and approximately 50% of the particles being larger than 30 microns in all the samples Trojan continued to be cautious in preliminary sizing of UV equipment for the SBC treatment process. It is believed that this approach would prove to be overly conservative in sizing UV equipment for the SBC treatment process since larger size particles should settle in the secondary clarifier if their composition is that of a tight dense particle. If the composition of the larger particles is more like a loose stranded particle (which it would need to be if neutrally buoyant) the ultraviolet light would be able to penetrate the particle and effectively disinfect the effluent. Trojan recommends that a collimated beam test be performed on the effluent prior to finalizing the required design UV dose if the SBC process continues to be utilized. This report recommends that if the SBC process continues to be utilized that a pilot study be performed on the effluent at the WWTP so that the actual UV dosage required can be determined based on actual field conditions.

Table VII-26
EXISTING WWTP EFFLUENT UV TRANSMISSIVITY AND PARTICLE SIZE DATA

Sample Date	*Sample Description*	*Flow (gpm)*	*% Transmissivity Unfiltered*	*% Transmissivity Filtered*	*Mean Particle Size (microns)*	*% Particles > 31 microns*	*TSS (mg/l)*
1-11-99	Low Flow	202	64	69	47.5	48.1	16
1-11-99	Peak Flow	630	67	71	46.2	48.9	23
1-12-99	Low Flow	295	67	70	43.8	49.0	13
1-12-99	Peak Flow	590	67	71	43.7	48.0	17
1-13-99	Low Flow	230	67	70	49.3	50.6	19
1-13-99	Peak Flow	618	67	71	47.2	51.7	22
1-14-99	Low Flow	290	67	71	45.0	49.1	20
1-14-99	Peak Flow	600	69	72	46.8	52.1	21
1-15-99	Low Flow	295	65	68	48.5	53.5	22
1-15-99	Peak Flow	625	65	68	50.7	55.5	25
90th Percentile			67	71	49		23

Trojan's preliminary recommendation for the Woodland WWTP is an open channel UV system with 216 lamps if the SBC process continues to be utilized or an open channel system with approximately 120 lamps if the SBR treatment process is implemented.

For open channel systems larger than 1.0 MGD a separate concrete structure must be constructed at the plant site in which the UV light rack units are installed. The structure recommended by Trojan for the SBC effluent disinfection system consists of a concrete channel 36' long x 4.25' wide x 4' deep in which the lamp units would be located. The outlet to the channel would be approximately 10' long x 12' wide x 4' deep with a serpentine weir for flow control. The structure recommended by Trojan for the SBR effluent disinfection system consists of a concrete channel 26' long x 4' wide x 4' deep in which the lamp units would be located. The outlet to the channel would be similar to the one described above for the SBC effluent disinfection system. The interior of the existing chlorine contact basin could possibly be modified so that the rack units could be installed without constructing a new open channel structure. This would require removing existing interior baffle walls and constructing new baffle walls to achieve the proper channel geometry for the specific system constructed. Estimated cost for the required concrete work is $15,000 if the chlorine contact basin can be utilized. If the contact tank cannot be utilized the estimated cost for a new UV disinfection open channel structure is estimated to be $25,000.

The PDF for Phase I is 1.6 MGD. Applying DOE's Class I Reliability standards for disinfectant contact basins to the UV system requires units in sufficient number and size so that with the largest-flow-capacity unit out of service, the remaining units should have a design flow capacity of at least 50% of the peak day flow. If effluent is continuously disinfected and discharged through the day (SBC treatment process) at least two units will be needed so that at least 0.8 MGD of disinfection capacity is provided with one unit out of service. Therefore if a closed type system is utilized two units should be supplied as part of the Phase I WWTP upgrade. If effluent is discharged in batches (SBR treatment process) required capacity of the UV equipment would equal the decant (discharge) flowrate of the batch system. As shown in Table VII-25, the SBR decant rate for both Phases I and II would be 3,333 gpm (4.8 MGD). This will require UV disinfection equipment with a capacity of at least 4.8 MGD. Again if a closed conduit system is installed two of the 2.6 MGD capacity units should be supplied as part of the Phase I WWTP upgrade for redundancy. No additional UV units would be required as part of the Phase II upgrade since the decant rate from the SBR basins will remain constant.

An advantage offered by the closed conduit type system is that the lights are automatically cleaned with a mechanized wiper thereby reducing the amount of operator time required to periodically clean the equipment. Clean quartz lamp sleeves is critical to ensure a properly functioning UV system. It could be necessary to periodically remove the lamps from the closed type system if dissolved solids or organic matter bond to the sleeves. Lamps in the open channel type system require monthly cleaning by the plant operators. Cleaning these lamp units consists of removing a lamp rack (module) from the open channel, placing it on the work stand and manually cleaning the lamp sleeves. A similar operation is also utilized when lamps burn out and require changing.

Either type of UV disinfection can be successfully incorporated into the Woodland WWTP Phase I improvements. It is recommended that the closed conduit system be utilized as the basis of design due at this point. Planning around the closed conduit system results in a slightly higher cost which provides a conservative cost figure. The closed conduit system also provides additional flexibility in that the units can be installed in the effluent piping at any convenient location. Based on this recommendation a base planning level estimated cost to supply and install a closed conduit UV disinfection system consisting of two 2.6 MGD units for Phase I is $200,000.

Reuse of Treated Effluent

The reuse of treated effluent alternative evaluated in this report consists of land application of treated effluent, during the dry season, to irrigate poplar (hybrid cottonwood) trees as a means to provide a marketable and renewable resource to the City of Woodland. As stated earlier in this section this alternative was selected for analysis due to the amount of agricultural land in close proximity to the City of Woodland and to the successful poplar tree planting/harvesting/re-planting recently completed by the Fort James Paper Company just north of the City's industrial park.

The effluent reuse alternative evaluation utilized an agronomic nitrogen application rate of 300 lbs of nitrogen (N) per acre per year. Average dry weather flows of 0.375 MGD for existing conditions, and 1.28 MGD for year 2023 conditions (refer to Section V) were used as hydraulic design criteria. The period of land application was assumed to occur from May through October (184 days). A total nitrogen effluent concentration of 30 mg/L was assumed as being representative of fixed-film biological treatment effluent for medium to high strength domestic wastewater. A total nitrogen effluent concentration of 15 mg/l was assumed as being representative of SBR treated effluent for medium to high-strength domestic wastewater.

Setback requirements for Class D Reclaimed Waters for spray irrigation are from Water Reclamation and Reuse Standards (DOE pub. no. 97-23) and are provided as the following:

1. There shall be a minimum of 300 feet between any reclaimed water pipeline and potable water supply.
2. Where reclaimed water is used for spray irrigation, there shall be a minimum of 100 feet between the area subject to irrigation and areas accessible to the public and the use area property line.
3. Where reclaimed water is used for surface irrigation, there shall be a minimum of 300 feet between the area subject to irrigation and any potable water supply well.

Table VII-27 shows the minimum land requirements for the proposed effluent reuse alternative based on the design criteria above and which treatment process is utilized at the WWTP.

Table VII-27
Woodland WWTP Treated Effluent Reuse Alternative Design Summary

	Reuse of Effluent From SBC Treatment Process	***Reuse of Effluent from SBR Treatment Process***
Agronomic Nitrogen Application Rate (lbs/acre/year)	300	300
Assumed Effluent Nitrogen Concentration (mg/l)	30	15
Year 2023 Dry Weather Flow (MGD)	1.28	1.28
Nitrogen Content of Effluent (lbs/day)	320	160
Annual Effluent Application Period (May-October)	184	184
Total Annual Nitrogen Content of Effluent (lbs)	58,800	29,440
Total Land Required for Effluent Application (acres)	196	98
Recommended Number of Application Plots	3	3
Required Area of Each Plot (acres)	66	33
Recommended Application Plots (1 spare plot)	1	1
Total Recommended Application Area (acres)	264	132
Land Area Allowance for Setback, Roads, etc (acres)	31	18
Total Area Recommended for Alternative	300	150

Due to the low nitrogen uptake of younger poplar trees, the poplars would originally be over-planted to increase the total nitrogen uptake from the young stand. As the trees mature, they will be thinned and the harvested trees will be chipped and spread on the ground as a future source of carbon. The average time for poplar trees to mature is 6-7 years and the stand should be harvested then. The spare plot provided for redundancy should be kept mostly fallow to allow it to be incorporated into the plot rotation each time a mature stand of poplars is harvested. A schedule for rotating the plots should be kept at all times, and a professional forest resources consultant should be hired to provide management services.

Land application of treated effluent also requires additional sampling and analysis of the treated effluent. The additional testing requirements which the City would need to implement are: 1) total fecal coliform organisms would be measured daily utilizing grab samples; and 2) turbidity would need to be measured using a continuous recording turbidimeter and read manually every four (4) hours.

A cost estimate for providing land application of effluent during the dry season is provided in Appendix I for both potential treatment processes. The estimated capital cost for this alternative is $12,500,000 if the existing SBC/RBC treatment process continues to be utilized. The estimated capital cost is $9,400,000 if the treatment process is changed to an SBR. Due to the high cost of implementing this alternative for both treatment processes it is recommended that the City continue to discharge all treated effluent to the Lewis River. Since all applicable water quality standards can be met by either treatment process, the discharge of treated effluent to the river results in stream flow augmentation and therefore is considered a beneficial reuse of the effluent.

Effluent Pump Station Upgrade

The existing effluent pumps (two pumps) each have a rated capacity of 650 gpm (0.94 MGD). The effluent pumps should be replaced as part of the Phase I upgrade with two new pumps, each rated for the PDF of 1.6 MGD if discharge is continuous and 4.8 MGD if the SBR treatment process is utilized. As mentioned previously if the SBR process is utilized the gravity effluent discharge line will need to be upsized to 24-inch to handle the decant flow rate. The existing chlorine contact basin can continue to serve as the pump well for the effluent pumps when pumping to the river is required. The new pumps, however, will discharge into a standpipe structure located within the treatment plant dike. This ensures the pumps pump against a constant head pressure and it provides the treatment plant with improved flood protection by eliminating a flap gate on the discharge pipeline.

Gravity Thickener Sizing for Anaerobic Digestion

Typical digester feed sludge solids concentration for anaerobic digestion is 4-6 percent. VS loading rates for high rate anaerobic digesters range between 0.10-0.50 lbs/d-ft^3 with hydraulic detention times of 10-20 days. Thickening a combination of primary and secondary sludge would be required to achieve the necessary solids concentration in the feed sludge. WAS would also need to be thickened, utilizing polymers, to achieve this range of solids concentration in the feed sludge.

The average solids concentration of blended primary/secondary sludge wasted from the primary clarifier to the existing aerobic digester from July 1997 through June 1998 is 2.75%. The average per capita solids wasting rate of blended sludge to the digester is

0.26 lbs/day from July 1997 through June 1998 based on a population of 3,570. Using these values and assuming the existing wastewater treatment process is utilized, the average daily volume of sludge that would be wasted from the primary clarifier(s) at an ADWF of 0.64 MGD is estimated to be 6,927 gpd (estimated population of 6,111). The amount of solids estimated to be wasted from the clarifier(s) is 1,589 lbs/day. The average daily volume of sludge that would be sent to the gravity thickener is estimated to be 13,703 gpd at an ultimate ADWF of 1.28 MGD (estimated population 12,089). The amount of solids estimated to be wasted from the clarifier(s) would be 3,143 lbs/day, under ultimate conditions.

A solids loading rate of 10-16 lbs/day-ft^2 (0.4-0.7 lbs/hr-ft^2) is recommended in *Metcalf & Eddy* for blended primary and biological contactor sludges. The loading rate will be discussed in this report in terms of lbs/hr-ft^2 since wasting to the thickener will not be continuous over a 24 hour period. The existing primary clarifier waste sludge pump has a capacity of 4,500 gallons per hour (75 gpm). At this pumping rate it will take approximately 1.54 hours a day to waste the required volume of sludge from the primary clarifier to the thickener (Phase I loading). Based on a loading rate of 0.7 lbs/hr-ft^2, the minimum thickener surface area required for a solids pumping rate of 1,031 lbs/hr (1,589/1.54) is 1,473 ft^2 (43.3 feet in diameter). The size of the gravity thickener would remain the same for the Phase II loadings since the solids pumping rate would remain constant.

If the waste sludge pumping rate is changed to 25 gpm, the solids loading rate is 344 lbs/hr. At this pumping rate and a loading rate of 0.7 lbs/hr-ft^2, the thickener surface area required is 491 ft^2 (25 feet in diameter). The existing aerobic digester is 26 feet in diameter and has a surface area of 531 ft^2 therefore it could be converted to a gravity thickener if the sludge pumping rate is changed to 25 gpm.

A common operating problem with gravity thickeners is the presence of undesirable odors particularly during the summer months. Odor is generated when septic conditions