CHAPTER 4

EXISTING WASTEWATER FACILITIES

This chapter describes the current condition and operating performance, and identifies known deficiencies, of the City of Canyonville's wastewater conveyance and treatment systems.

WASTEWATER CONVEYANCE SYSTEM

The City of Canyonville is responsible for operating and maintaining the collection system that conveys wastewater from residential and commercial users to the municipal wastewater treatment plant. This section describes the collection system infrastructure and assesses existing sewerline capacities. The collection system was evaluated in further detail in the Draft *1998 Wastewater Facilities Plan* (Appendix C).

Canyonville's wastewater collection system was largely designed and constructed between 1959 and 1961. Although this infrastructure development did incorporate a small portion of an older sewer system in the downtown area, the vast majority of the system was newly constructed at that time. Over the years, the City has extended sewer service to various new developments on the periphery of the core system. In addition to incremental system expansions, the City constructed a low pressure line in 1989 to serve customers in the North Canyonville area.

Pump Stations and Force Main

A small pressurized collection system serves North Canyonville and Stanton Park. The system is designed such that each system user in this service area operates and maintains their own small-scale pump station that discharges septic tank effluent into the City's low-pressure main. The pressure main is constructed of PVC pipe ranging in diameter from 3 to 6 inches.

Flows from North Canyonville combine with larger flows from Canyonville proper at an influent manhole immediately upstream of the Wastewater Treatment Plant (WWTP) raw sewage pump station. Dissolved sulfides present in the septic tank effluent are released as hydrogen sulfide, causing corrosion of concrete surfaces in the manhole. No other collection system corrosion issues are known.

Gravity Sewers

Most of Canyonville's collection system is composed of gravity sewers. The prevalence of sloping terrain in the area creates the necessary topographic conditions for an effective gravity system.

The main trunk of the gravity sewer parallels the course of Canyon Creek, conveying wastewater to the treatment plant which is sited at the confluence of Canyon Creek and the South Umpqua River. Sewer pipe diameters range from 6 inches in the furthest upstream reaches of the collection system to 15 inches in the main trunk as it approaches the WWTP. The majority of the sewers are 8 inches in diameter. Slopes vary considerably in the system, ranging from 0.16 percent along Canyon Creek to an extreme of nearly 30 percent for a hillside development. Topographic conditions control the invert elevations of the sewers, which are as shallow as 3 feet on the main trunk. Pipe materials are generally asbestos cement and PVC, although there are some concrete and clay pipes dispersed throughout the system. The majority of new sewers are PVC pipe. The size and length of the gravity sewers are summarized in Table 4-1.

TABLE 4-1: Collection System Inventory

Pipe Diameter, inches	Pipe length, feet												
	Depth, feet												
	4	5	6	7	8	9	10	11	12	13	14	Unknown	Total
6												400	400
8	1465	2084	8647	6160	2720	2264	1436	450	1165	100	90	4750	31331
10		327	480	275		600		315					1997
12	110	542	742	164				127	175	106			1966
15		646	206	567	627	509	144						2699
Total	1575	3599	10075	7166	3347	3373	1580	892	1340	206	90	5150	38393

Source: Draft 1998 Wastewater Facilities Plan

The collection system does not contain any known designed overflow points for bypassing wastewater during high flow conditions. However, peak flows associated with the high infiltration and inflow (I/I) from large storm events could cause unintentional overflows at certain manholes along Canyon Creek just upstream of the treatment plant during extreme high flow conditions. There are also two locations on the plant site where designed bypasses can occur: at the influent manhole and the secondary effluent collection box.

Past raw sewage bypasses have been the result of raw sewage pump failures during storm events. In the past, one or more of the vacuum-primed pumps would clog or fail to prime, reducing the overall capacity of the pump station. Since replacement of the vacuum-primed pumps with one submersible raw sewage pump in 1997, and another in 2000, only one raw sewage bypass has been reported. This overflow was associated with clogging of the main trunk line at its crossing of Canyon Creek in December 2009, under relatively low flow conditions.

Condition, Deficiencies, and Status of Conveyance System

The current condition and capacity of the wastewater conveyance system is described in this section.

General System Condition. The entire collection system was recently smoke tested. The results were generally inconclusive; however, the following deficiencies were identified:

- Several lateral cleanout caps were missing.
- Two broken laterals were discovered. The homeowners were required to perform repairs.

The City owns closed-circuit television equipment jointly with the City of Myrtle Creek and has been inspecting the collection system section-by-section. An 800-ft section of 8-inch diameter sewer in Leland Avenue was recently sliplined as a result of these inspections. In addition, several manholes were inspected during the flow testing performed as part of the *Draft 1998 Wastewater Facilities Plan* in March 1998. All the manholes inspected were in good condition, with no signs of excessive erosion or corrosion.

Capacity Assessment. The capacity of a gravity sewer is determined by its diameter, slope, and, to a lesser extent, its material of construction.

The flow analysis in Chapter 5 shows that the current peak hour flow (PHF) at the Canyonville WWTP is approximately 1.6 MGD. The *Draft 1998 Wastewater Facilities Plan* (Appendix C) indicated that the controlling capacity for the main trunk is 1.7 MGD; therefore, the sewer system should not become surcharged during current peak wet weather conditions. Surcharging causes water to back-up in manholes, which in turn raises the water pressure in the system and allows for higher flow rates in the sewer pipes. The surcharging capacity of the system is determined by the maximum height water is able to back-up in the critical manhole without overflowing, given an appropriate safety factor.

Chapter 5 develops flow projections for the wastewater conveyance and treatment system for design year 2035. The design PHF is 2.2 MGD, which exceeds the surcharged capacity of the main trunk. A significant portion of this flow is caused by infiltration and inflow.

Infiltration and Inflow. Infiltration and inflow (I/I) refers to water that enters sewer lines due to defects in the system or illicit storm drain connections. Infiltration is groundwater that enters the system from the surrounding soil through defective pipes, joints, or manholes. Inflow is stormwater that directly enters the system from sources such as illicit drainage connections and flooded manhole covers.

The City recognizes that I/I is a significant concern and that its removal can make for a more cost effective wastewater program. However, relatively limited information on I/I is available at this time. For example, collection system flow monitoring has not been conducted in recent years.

As part of the Draft *1998 Wastewater Facilities Plan* (Appendix C), an analysis of plant flow records and field inspections during a winter storm event was conducted to determine I/I flows associated with Canyonville's collection system. The storm event investigation resulted in evidence of a possible single I/I source that caused an increase in flow along the main trunk that passes beneath Canyon Creek. In order to further investigate the possibility of defects or deterioration in the creek crossing pipeline, it was recommended that an inspection of this reach of the system be conducted with closed-circuit television equipment. The inspection occurred shortly thereafter and a hole in the trunk line was found. The hole was repaired and flows have since decreased.

I/I Sources. Extraneous water can enter the collection system in a number of different ways. It is important to note that infiltration increases during and immediately after storm events. As rain water saturates the soil, it can leak into pipes through defects that are normally well above the groundwater table. This type of infiltration is referred to as rainfall-dependent infiltration. Rainfall dependent infiltration is often difficult to distinguish from inflow and even more difficult to pinpoint. Even sewer television inspections sometimes cannot locate sources of rainfall dependent infiltration unless the inspection is performed during a storm event.

I/I Removal. I/I is removed from a wastewater collection system through rehabilitation or replacement of pipes and manholes. The first step in an I/I removal program is to identify sources of I/I and prioritize rehabilitation projects. Typically, those projects which have the potential to remove the most I/I at the lowest cost are initiated first. While I/I sources have not been identified as part of a comprehensive identification and rehabilitation program, the City regularly conducts television inspections of portions of the system and corrects deficiencies when possible. The City's ongoing inspection and repair efforts are summarized in Appendix D.

Deficiencies. Areas of concern recently identified by City staff include:

- Low spots near MH C7-1 and MH C-5, under an existing culvert crossing, contribute to the capture of gravel in Lateral C-7.
- Gravel observed in the main at the Canyon Creek Crossing from MH-A-25 to MH-A-26.
- Joint leakage observed at various manholes in the system.

WASTEWATER TREATMENT PLANT

The City of Canyonville WWTP is located on the south bank of the South Umpqua River, just upstream of the river's confluence with Canyon Creek. This section describes the existing WWTP.

Plant History

The City of Canyonville has operated a WWTP at the site of the existing plant since 1955. The original plant consisted of an influent pump station, headworks, combination primary and secondary clarifier, trickling filter, and anaerobic sludge digester. A major plant upgrade was initiated in 1979. These improvements included:

- Modifying the influent pump station. The pump station was altered so that the pumps rested at a higher elevation. The purpose of this modification was to keep the pumps' motors above the 100-year flood elevation. New return activated sludge (RAS) pumps were installed inside the influent pump station building.
- Constructing a new package treatment plant. The new package plant consisted of a grit removal system, comminutor, activated sludge aeration basins designed to operate in contact/stabilization mode, and a secondary clarifier.
- Constructing a new filter and secondary effluent pump station.
- Converting the trickling filter into a chlorine contact basin.
- Converting the existing dual clarifier into a second secondary clarifier.

Shortly after construction of these improvements was completed, it was discovered that the RAS pumps would not operate reliably. Therefore, air-lift pumps were installed on both clarifiers. The air-lift pump on the older clarifier (Secondary Clarifier 2) was unable to develop adequate head to return sludge to the aeration basin; consequently, Secondary Clarifier 2 is not operated as a clarifier and is instead used for sludge storage during the wet weather season.

In response to periodic equipment failures and high maintenance requirements, plant operators installed one new raw sewage pump and one new secondary effluent pump in 1996.
To enhance the capacity, performance, and reliability of the existing plant, a number of upgrades were constructed in 1998, including improvements to the RAS system; a diffused aeration system; piping changes to allow operation in plug flow mode; and associated electrical and controls systems.

Plant Design

The Canyonville WWTP is an activated sludge facility which can operate in contact/stabilization and plug flow process modes. A site plan and process schematic of the existing system are shown in Figures 4-1 and 4-2, respectively. Plant design data is included in Table 4-2.

A 15-inch diameter gravity sewer conveys raw sewage to the plant. The plant also receives septic tank effluent through the plant influent manhole via a 6-inch diameter pressure line.

The wastewater flows from the influent manhole to the influent pump station wet well. The raw sewage pump station houses three raw sewage pumps: two are submersible variable speed pumps, and the third is a constant speed, vacuum-primed pump. The pumps discharge into an 8-inch diameter force main; a 6-inch magnetic flow meter measures plant influent flow.

The force main conveys the raw sewage to the treatment unit. The treatment unit consists of a headworks, aeration basin, and secondary clarifier. Wastewater first flows through a 4-foot diameter Pista grit removal system. An air-lift pump lifts grit slurry to a grit classifier for dewatering. The de-gritted wastewater then flows through a 12-inch comminutor, which grinds rags and large solids. A manually raked bar screen serves as a backup to the comminutor.

The main body of the treatment unit is circular in shape, with a central secondary clarifier and aeration basins occupying the annular space between the clarifier and the outer tank wall. The aeration basins were designed to operate as two parallel contact/stabilization processes; however, the RAS piping has been modified to allow for operation in plug flow mode as well. Mixed liquor (the combination of raw sewage and RAS) flows from the aeration basins to an integral flow distribution structure. Only Secondary Clarifier 1 is normally in service, therefore, it receives all of the mixed liquor. Secondary effluent flows to the secondary effluent pump station wet well via a 10-inch diameter gravity line, while RAS is returned to the treatment process.

The secondary effluent pump station is equipped with three pumps: a variable-speed submersible pump and two constant-speed vertical turbine pumps. The pumps lift the secondary effluent to a fabricated flow distribution structure located above the effluent filter. During normal operation, all flow is routed through the filter. However, during wet weather periods, operators occasionally bypass a portion of the secondary effluent around the filter when flow rates exceed the unit’s hydraulic capacity.

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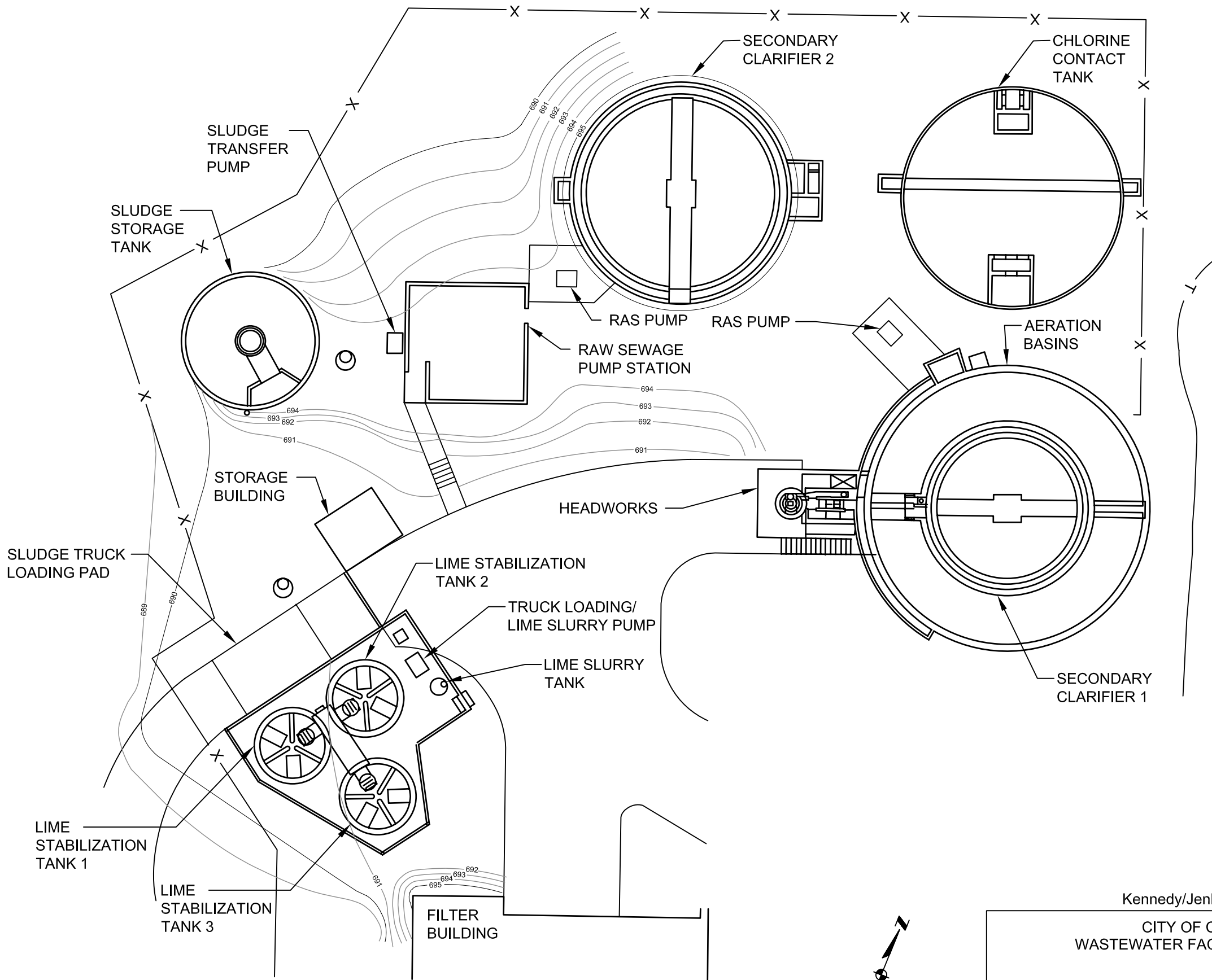

Kennedy/Jenks Consultants

CITY OF CANYONVILLE
WASTEWATER FACILITIES PLAN

SITE PLAN

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MAY 2010

FIGURE 4-1

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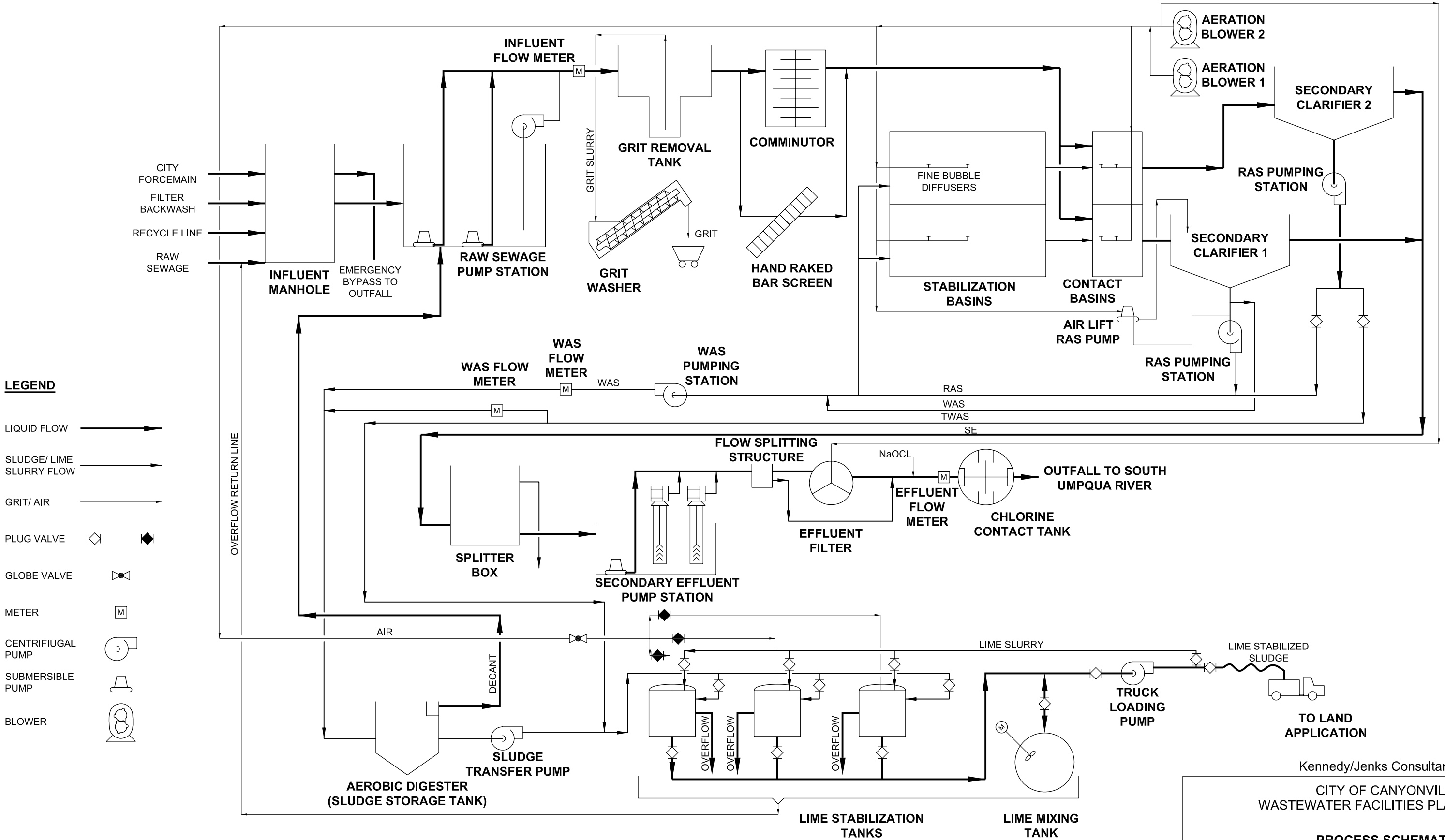

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Kennedy/Jenks Consultants

CITY OF CANYONVILLE
WASTEWATER FACILITIES PLAN

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MAY 2010

FIGURE 4-2

Table 4-2: Plant Design Data

Item	Value
Raw sewage pumps	
Number	3
Capacity, each, gpm	2@380, 1@440
Motor horsepower, each	2@7.5, 1@5
Influent flowmeter	
Type	Magnetic
Diameter, inches	6
Headworks	
Grit removal tank	
Number	1
Diameter, feet	4
Capacity, mgd	1.0
Comminutor	
Number	1
Capacity, mgd	1.1
Hand raked bar screen	
Number	1
Width, feet	1.5
Activated sludge secondary process	
Stailization basins	
Number	2
Volume, each, gallons	48,000
Contact basins	
Number	2
Volume, each, gallons	21,500
Aeration blower	
Number	2
Capacity, scfm	1@700, 1@1,000
Motor horsepower	1@30, 1@50
Aeration diffusers	
Type	Fine bubble membrane
Number	400
Diameter, inches	9
Secondary clarifiers	
Secondary Clarifier 1	
Diameter, feet	26
Sidewater depth, feet	10
Secondary Clarifier 2	
Diameter, feet	34
Sidewater depth, feet	8
Return Activated Sludge Pumps	
RAS pump 1	
Type	Self-priming centrifugal
Number	1
Capacity, gpm	100-290
Motor horsepower	5
RAS pump 2	
Type	Self-priming centrifugal
Number	1
Capacity, gpm	100-350
Motor horsepower	7.5
Air-lift RAS pump	
Number	1

Item	Value
Waste activated sludge pumps	
Type	Centrifugal
Number	2
Capacity, each, gpm	300
Motor horsepower, each	5
Effluent filter system	
Secondary effluent pumps	
Number	3
Capacity, each	1@450, 2@350
Motor horsepower, each	1@10, 2@7.5
Effluent filter	
Type	Granular
Number of cells	3
Diameter, feet	14
Media depth, feet	3
Effluent flowmeter	
Type	Ultrasonic
Disinfection	
Type	Sodium hypochlorite
Number of storage tanks	2
Capacity, gal	500
Number of metering pumps	3
Capacity, gph	3.2
Chlorine contact tank	
Number	2
Sidwater depth, feet	3.3
Total volume, gallons	31,000
Sludge storage tank	
Number	1
Diameter, feet	24
Sidewater depth, feet	18
Volume, gallons	60,900
Sludge lime stabilization system	
Sludge transfer pump	
Type	Self-priming centrifugal
Number	1
Capacity, gpm	275
Motor horsepower	5
Sludge truck/lime slurry pump	
Type	Self-priming centrifugal
Number	1
Capacity, gpm	200
Motor horsepower	3
Lime stabilization tanks	
Type	Upright storage tank
Number	3
Volume, each, gal	6,000
Mixing air flow, scfm	40
Lime slurry mixing tank	
Type	Vertical PE storage tank
Number	1
Volume, gal	120
Standby generator	
Type	Diesel
Number	1
Capacity, kw	200

Filtered secondary effluent flows by gravity through a 10-inch diameter line to the chlorine contact tank. An ultrasonic flowmeter measures flow as the effluent enters the tank. Sodium hypochlorite solution is injected at the upstream end of the tank. The tank is divided in half by a hydraulic wall so that one side can be taken out of service for cleaning. Final effluent is discharged into the South Umpqua River through a shoreline outfall structure. The outfall was damaged in the mid-1990s – the outlet pipe was washed downstream.

Sludge is currently pumped from the aerobic digester (sludge holding tank) to Secondary Clarifier 2 for thickening by gravity. Thickened sludge is conveyed to one of three lime stabilization tanks. Lime is added to the sludge to raise the pH and comply with EPA sludge stabilization requirements. After stabilization, the sludge is pumped into a tanker truck and hauled to agricultural reuse sites.

Plant Operations

The operation and condition of the WWTP's various treatment processes was assessed through evaluation of the plant operating reports, past construction documents, and discussion with plant operators. Unit processes at the Canyonville WWTP consist of:

- Raw sewage pumping
- Influent flow measurement
- Grit removal
- Sewage grinding
- Activated sludge secondary treatment
- Secondary clarification
- Return activated sludge pumping
- Secondary effluent pumping
- Filtration
- Effluent flow measurement
- Disinfection
- Outfall
- Sludge stabilization
- Biosolids reuse

Raw Sewage Pumping. The raw sewage pump station houses two Flygt non-clog submersible pumps and one Cornell Model 4NNT non-clog centrifugal pump. The 7.5-horsepower Flygt pumps are equipped with variable frequency drives and are rated for 380 gallons per minute (gpm) at 23 feet of head. The Cornell pump has a constant-speed 5-horsepower motor and is rated for 440 gpm at 23 feet of head. The Cornell pump is mounted above the water surface elevation in the wet well; therefore, it is fitted with a vacuum priming system. Plant operators report that the vacuum priming system requires a great deal of maintenance. The pumps are controlled with float switches. The variable speed submersible pumps serve as the lead pumps. The older, constant speed pump operates during peak flows and serves as a backup.

The raw sewage pump wet well is 14 feet in diameter. The large wet well causes maintenance problems because it traps grease and grit. Plant operators report that grease buildup is particularly troublesome despite the fact that the City has a grease ordinance in place. Due to its depth and poor access, removing grease and grit from the wet well is difficult and time

consuming. DEQ defines the capacity of a pump station as the capacity of the station with the largest pump out of service. Observing this criterion, the capacity of the raw sewage pump station is 1.2 MGD.

Influent Flow Measurement. A 6-inch magnetic flow meter provides influent flow measurement. The meter is on the discharge line from the raw sewage pumps, and is located inside the blower vault. While the capacity of the meter is unknown, it is reasonable to expect that the meter is capable of accurately sensing velocities of 10 feet per second. For a 6-inch-diameter pipe, a velocity of 10 feet per second corresponds to a flow rate of 1.3 MGD. City staff indicate that the strip chart recorder that receives a signal from the flow meter functions poorly.

Grit Removal. The grit removal tank is a 4-foot diameter Pista vortex chamber manufactured by Smith and Loveless. Rotating paddles keep lighter organic material in suspension while grit settles to the bottom of a hopper. An air-lift pump conveys grit slurry to a grit washer; dewatered grit drops into a dumpster. The gear reducer on the Pista system has failed. Parts are no longer available for the unit, so no automatic grit removal occurs at the plant. Grit accumulates in the aeration basins and must be periodically removed. The grit washer has also failed and is no longer operational.

Sewage Grinding. Sewage grinding was popular in the 1960s and 1970s as an alternative to screens. Comminutors grind rags and other large solids in the raw sewage to reduce their impact on downstream processes. However, the shredded rags tend to reform into large strands resembling ropes, which can clog pumps and pipelines. Screening of raw sewage is now generally recognized as the preferred method of removing rags. Canyonville's comminutor is a 12-inch Worthington; its rated capacity is 1.1 MGD. However, the comminutor is no longer operational. All flow is bypassed to the manual bar screen.

Aeration Basins. The WWTP's aeration basins occupy the annular space around Clarifier 1. The aeration basins have a total volume of 139,000 gallons. They were originally designed to operate as two parallel contact/stabilization processes. However, in the late 1990s, the RAS piping was modified to allow the basins to operate as a single plug flow tank.

In contact stabilization mode, there are two distinct treatment zones: the stabilization zone and the contact zone. The stabilization zone contains RAS, which is primarily bacteria. The contact zone contains a mixture of RAS and raw sewage (mixed liquor). RAS and raw sewage mix at the upstream end of the contact zone. Because the RAS has been aerated in the stabilization zone without a source of food for a significant amount of time, the bacteria are "starved" and uptake soluble organic material in the raw sewage faster than they can oxidize it. The excess organic material is stored within the bacteria for later oxidation. The mixed liquor flows to the secondary clarifier where the bacteria are separated from the water and then returned to the stabilization zone. In the stabilization zone, the bacteria are aerated so that they can oxidize the stored organic material. By the time the bacteria leave the stabilization zone, all stored organic material has been oxidized and the process can be repeated. Relative to plug flow, contact stabilization offers increased treatment capacity for a given basin volume.

Plug flow mode is more straightforward than contact stabilization mode. RAS is mixed with raw sewage to form mixed liquor at the upstream end of the aeration basin. After aeration, the mixed liquor flows to the secondary clarifier where the bacteria are separated from the water. The RAS is pumped back to the upstream end of the aeration basin and the process is repeated.

The capacity of aeration basins is often expressed in terms of maximum month flow and BOD load at minimum month temperature. The aeration basins are estimated to adequately treat a maximum month flow of 0.6 mgd and a maximum month BOD load of about 1,000 pounds per day (ppd) at a wastewater temperature of 12 degrees C. A discussion of current loading is included in Chapter 5.

Plant operators have noted problems with filaments in the aeration basins. They have also indicated a continuing problem with controlling nitrification. Grit buildup is common since the grit removal system has failed.

Activated Sludge Secondary Treatment. The activated sludge secondary treatment process uses aerobic microorganisms to consume soluble organic material in the wastewater. Given an adequate supply of oxygen, the microorganisms consume the organic material in the wastewater, producing carbon dioxide and more microorganisms. Excess microorganisms are wasted from the process as sludge. The activated sludge system consists of the aeration basins and aeration system.

Aeration System. The activated sludge treatment process requires a source of oxygen to allow for biological oxidation of organic material. The Canyonville WWTP supplies oxygen to the bacteria by diffusing compressed air into the aeration basins. Aeration equipment consists of blowers and aeration diffusers. A new blower rated for 1000 standard-cubic-foot-per-minute (scfm) at 9 pounds per square inch (psi) was added to the system in 1998 to supplement the plant's existing 700 scfm blower. The original 700 scfm blower was rated for only 7 psi; it is therefore retained as a backup to the new unit.

Secondary Clarification. The WWTP has two secondary clarifiers: Secondary Clarifier 1 is 26 feet in diameter and has a sidewater depth of 10 feet, while Secondary Clarifier 2 has a diameter of 34 feet and a sidewater depth of 8 feet. Both clarifiers are equipped with radial launders. Secondary Clarifier 1 was constructed as part of the 1978 plant upgrade and is located at the center of the treatment unit. From a hydraulic standpoint, Secondary Clarifier 2 is nearly 4 feet lower than Clarifier 1. Adjustable weirs can be used to split flow between the two units; however, operators report problems accurately controlling flow with this method. Secondary Clarifier 2 has not been used in recent years because sludge cannot be pumped from the tank. A new RAS pump was installed for Secondary Clarifier 2 as part of the 1998 Interim Improvements project; this allows Secondary Clarifier 2 to be placed in service during periods of high flow and to serve as a backup to Secondary Clarifier 1.

The capacity of a clarifier is expressed in terms of its maximum overflow rate. Overflow rate is defined as the flow leaving through the launders divided by the surface area of the clarifier and is normally expressed in gallons per day per square foot of surface area (gpd/sf). The maximum overflow rate is the point at which an excessive amount of solids are lost over the weirs. For a new, well-designed, deep (16 to 18 feet) clarifier with peripheral launders, a reasonable maximum overflow rate based on PHF is 1,300 to 1,400 gpd/sf. Canyonville's clarifiers are relatively shallow, and the radial launders may promote some short circuiting and solids carryover. Therefore, the maximum overflow rate of the existing clarifiers is estimated to be significantly lower than the 1,300 gpd/sf value used for new clarifiers.

Plant operators report that grease builds up in Secondary Clarifier 1; its scum removal mechanism does not remove scum from the water surface effectively. In addition, scum cannot be easily removed from the clarifier's scum pit.

Secondary effluent from the two clarifiers combines in an outlet structure and flows by gravity to the secondary effluent pump station wet well.

Return Activated Sludge Pumping. Two self-priming centrifugal pumps are used for pumping RAS from the secondary clarifiers to the aeration basins. The smaller capacity pump is rated for 290 gpm, while the larger capacity pump is rated for 350 gpm. The RAS pumps are equipped with variable frequency drives so that operators can adjust RAS flow rate from 100 gpm to their upper limit to optimize process operation.

Secondary Effluent Pumping. The secondary effluent pump station is equipped with two single-stage vertical turbine pumps and a submersible sewage pump. The pumps lift the secondary effluent to a fabricated flow distribution structure above the effluent filter.

Filtration. The effluent filter is a 3-cell, 14-foot diameter unit with sand and anthracite media. The filter is used year-round; however, it is only needed during the dry weather season when the plant is required to comply with 10 milligram per liter (mg/L) effluent TSS and BOD concentration limits. The filter is backwashed with both air and water. Air for backwash is generated by the process air blowers. An automatic valve diverts process air to the filter during the backwash cycle. An air compressor provides air to pneumatic control levels.

Effluent Flow Measurement. The effluent flow meter is an ultrasonic level detector located at the upstream end of the chlorine contact tank above an existing v-notch weir. It replaced the previous flow metering device, which was a float-style meter, as part of the 1998 upgrade.

Disinfection. The disinfection system at the plant was recently upgraded from a chlorine gas system to liquid sodium hypochlorite. A 12 percent concentration of sodium hypochlorite is dosed using one of three diaphragm metering pumps. Sodium hypochlorite solution is stored in two polyethylene chemical storage tanks with a combined capacity of 500 gallons. The 40-foot-diameter, 31,000 gallon chlorine contact tank is a converted trickling filter. A hydraulic wall divides the tank so that one half can be taken out of service for cleaning while the other is still operating.

Outfall. As discussed previously, the existing outfall was damaged during a flood in the mid-1990s. Currently, during low river flows, the treated effluent runs across the bank and into the river.

Sludge Management Program. The Federal Part 503 Regulations list two separate types of sludge treatment requirements: pathogen reduction and vector attraction reduction. With regards to treatment to reduce pathogens, treated WWTP sludge is classified into two categories: Class A biosolids and Class B biosolids. The Canyonville WWTP produces Class B biosolids. The treatment requirements for Class B are less stringent than for Class A; however, there are more restrictions for final disposal of Class B biosolids. The Federal Part 503 Regulations list five possible processes for producing a Class B biosolids.

An aerobic digester was constructed as part of the 1979 upgrade to meet vector attraction and Class B pathogen reduction requirements; however, the tank is too small to provide the detention times necessary to meet the required volatile solids destruction.

Because the aerobic digester cannot comply with Class B treatment standards, plant operators use lime to stabilize the sludge. The vector attraction reduction and Class B pathogen reduction requirements for lime stabilization are:

- Raise the pH above 12. The pH must remain above 12 for 2 hours without the addition of more lime.
- Maintain the pH above 11.5 for 22 hours without the addition of more lime.

Waste activated sludge is pumped to the existing aerobic digester for storage. The liquid sludge is then transferred to Secondary Clarifier 2, which is currently being used for sludge thickening and storage. Supernatant is returned to the influent pump station wet well. Thickened sludge removed from the clarifier has a solids concentration of 2.5 to 3.0 percent; it is pumped to one of three 6,000-gallon high-density polyethylene (HDPE) lime stabilization tanks. Lime is added to the sludge in the dedicated compartment and treated to Class B standards. The HDPE tanks replace the former method of using a compartment in the aeration basins for lime stabilization.

A sludge truck/slurry pump is used to load liquid sludge into a tanker truck for transport. There is no wet weather sludge storage facility.

Biosolids Reuse. Application of biosolids onto agricultural land must be done in accordance with accepted agronomic rates; nitrogen is usually the key nutrient. In addition, the application of heavy metals must be monitored and kept within approved limits.

The Part 503 Sludge Regulations and the Oregon Administrative Rules state that animals cannot graze on pastureland for 30 days after Class B biosolids are applied. Public access to the land also must be restricted for 30 days.

The City currently has agreements with local farmers for spreading up to 12,000 gallons (4 loads of 3,000 gallons per load) of biosolids per day. Plant staff indicate that one particular farmer has recently installed an irrigation system and is no longer interested in accepting “liquid” biosolids from the WWTP. Biosolids quantities are summarized in Table 4-3:

TABLE 4-3: Biosolids Quantities

Parameter	2008 Peak Month Value
Waste Activated Sludge Solids (lbs/day)	800
Average Solids Concentration of WAS (mg/L)	5,200
Sludge Volume (gal/day)	18,700
Lime Stabilization Capacity (gal/day)	18,000
Stabilized Solids Concentration (percent)	2.5
Stabilized Solids (lbs/day)	980
Stabilized Solids Volume (gal/day)	5,000
Sludge Storage Capacity (gal)	60,900
Sludge Storage HRT (days)	12
Annual Biosolids Production (dry tons)	40

Plant Hydraulic Capacity

As part of the *City of Canyonville WWTP Capacity Analysis* (May, 1997), a hydraulic computer model was used to estimate the hydraulic capacity of the Canyonville WWTP. Wet weather operations (filter bypassed) were evaluated and the capacities of the pump stations were not considered.

The hydraulic profile for the plant was evaluated under the following conditions:

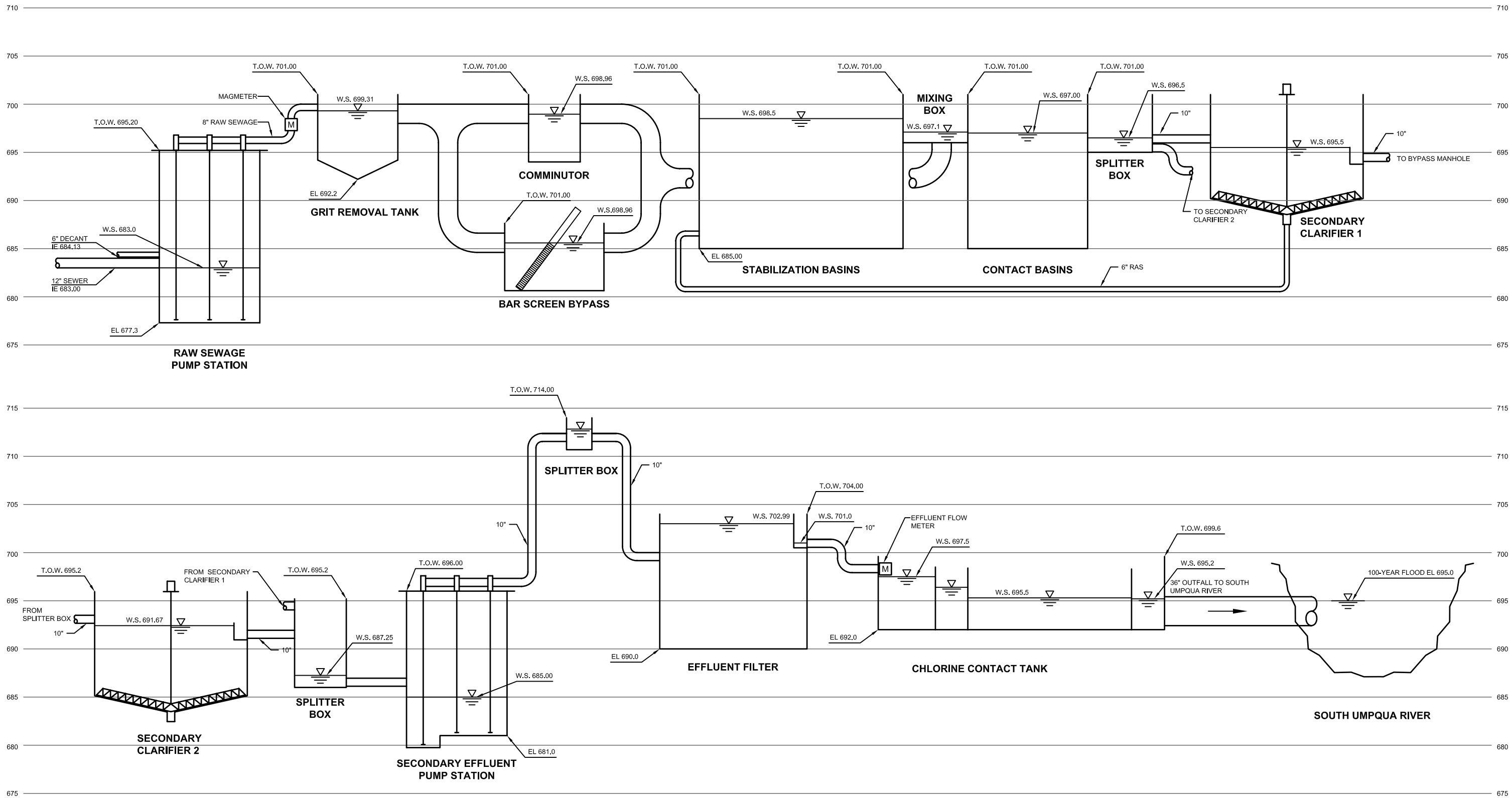
- Plant influent flow of 1.2 MGD.
- RAS flow at 40 percent of influent flow.
- River at 100-year flood level.

- Flow split evenly between the two clarifiers.

The model indicated that no structure walls would be overtopped under these conditions. However, the calculations indicated that the following would occur:

- The baffle walls in the chlorine tank would be overtopped—some short circuiting would occur.
- All weirs in the chlorine contact tank would be submerged. Effluent flow measurement would not be possible.

A hydraulic profile of the WWTP at an influent flowrate of 1.2 MGD is included as Figure 4-3.



NOTE:
STRUCTURE AND WATER SURFACE ELEVATIONS TAKEN FROM "CITY OF CANYONVILLE WASTEWATER TREATMENT FACILITY" DRAWINGS DATED SEPTEMBER 1978 AND DEVELOPED BY HGE INC. 100-YEAR FLOOD ELEVATION FROM NATIONAL FLOOD INSURANCE PROGRAM FLOOD INSURANCE RATE MAP NO. 410059-1145A FOR DOUGLAS COUNTY, OREGON, DATED DECEMBER 1978.

HYDRAULIC PROFILE

HORIZ NO SCALE
0 1 2 3 4 5
VERT 1"=10'

Kennedy/Jenks Consultants

CITY OF CANYONVILLE
WASTEWATER FACILITIES PLAN

HYDRAULIC PROFILE

0976004.00
MAY 2010

FIGURE 4-3

F:\CAD\2009\09004\01_Canyonville_WW_Facilities_Plan\FIGURE_HYDPROF.dwg BECKY WILTSE 5/3/2010 1:24 PM

Unit Performance and Deficiencies

The performance and condition of the WWTP's various treatment processes was determined through discussions with plant operators and a review of waste discharge reports. Annual average performance data for removal of influent wastewater constituents is shown in Table 4-4.

TABLE 4-4: Plant Performance - 2008

Month	Influent Flow		Effluent Concentrations					
	Average Day, MGD	Maximum Day, MGD	Average BOD_5, mg/L	Average TSS, mg/L	Total P, mg/L	TKN, mg/L	Ammonia, mg/L	Nitrate, mg/L
Dry Weather	0.22	0.40	4	5	4	19	17	3
Wet Weather	0.29	0.99	8	9	No Limit	No Limit	11	No Limit
January	0.42	0.99	8	9			7	
February	0.30	0.48	12	10			12	
March	0.27	0.37	8	7			13	
April	0.24	0.27	10	8			15	
May	0.23	0.28	6	5	4	19	14	4
June	0.23	0.40	6	6	3	4	12	4
July	0.22	0.25	3	6	4	24	20	2
August	0.21	0.23	4	3	3	23	20	2
September	0.20	0.23	3	4	4	22	20	4
October	0.20	0.23	3	6	4	22	18	1
November	0.23	0.29	3	14			8	
December	0.29	0.61	5	6			10	

The WWTP's unit processes are able to effectively reduce BOD and TSS to below the waste discharge limits of 10 mg/L BOD and TSS at average dry weather flows.

Existing process unit deficiencies are listed as follows, with current capacities shown in Table 4-5.

Raw Sewage Pumping. The raw sewage pumps have occasionally become clogged with debris. Previous studies have concluded that the station's capacity cannot be increased cost effectively, as the influent pump room does not have sufficient space to install additional pumps. In addition, City staff indicate the existing vacuum prime pump loses prime and is unreliable.

Grit Removal and Sewage Grinding. The existing headworks consists of a comminutor and grit removal system. Both of these pieces of equipment have failed. The gearbox on the Pista grit removal system is broken. The grit screw in the washing mechanism is also broken and currently out of service. The comminutor has been removed altogether, requiring all debris to be removed by manual screening.

Aeration Basins. Structural cracks are evident on the concrete walls of the basins. A corrosion leak has been found on the air header between Aeration Basins 3 and 4.

Secondary Clarification. The existing secondary clarifiers were constructed at different elevations, making it difficult for plant personnel to operate the two clarifiers simultaneously. Nuisance filamentous foam has been observed throughout the activated sludge system.

Waste Activated Sludge. Plant staff would like to have the ability to add polymer to the sludge wasting system to enhance sludge thickening.

Disinfection. The chlorine contact tank, converted from the original trickling filter, shows signs of structural deterioration and problematic leaks. In addition, there is currently no means for dechlorinating plant effluent.

Outfall. As mentioned earlier, a portion of the plant outfall was washed away during high river flows. The plant has been discharging through the damaged outfall since this incident.

Sludge Management. The plant is currently using the aerobic digester and Secondary Clarifier 2 for onsite solids storage. Both the digester and the lime stabilization process do not have adequate capacity for anticipated design year solids quantities. The digester no longer has the equipment in place to operate as an aerobic digester.

TABLE 4-5: Current Wastewater Treatment Plant Deficiencies

Process	Current Capacity	Description of Deficiency
Structural repairs	NA	Differential settlement and structural deficiencies observed on site.
Screening	NA	Currently there is no automatic screening at the plant, and no screening whatsoever upstream of the raw sewage pump station.
Raw Sewage Pump Station	1.2 MGD	Existing pump station has insufficient capacity, yet inadequate space for expansion. Vacuum primed pump is unreliable. Influent flowmeter and strip chart function poorly.
Grit Removal	1.1 MGD	Existing grit removal equipment has deteriorated beyond repair.
Aeration Basins	140,000 gal	Existing aeration basins have insufficient capacity. Structural cracking and corrosion-based leaks have been observed.
Secondary Clarifiers	1.2 MGD	Cannot be operated simultaneously. Foaming has been observed.
WAS Pumping	600 gpm	No current means for adding polymer in order to enhance thickening.
Secondary Effluent Pumps	1.7 MGD	Existing submersible pump has insufficient capacity to handle increased flows. The pump station has insufficient space for expansion.
Disinfection	3.2 gph	Chlorine contact tank shows signs of deterioration. No means for dechlorination of plant effluent.
Plant Outfall	NA	The outfall is damaged and there is no diffuser in place.
Biosolids Storage	12 days	Insufficient sludge storage and inoperable aerobic digester.
Lime Stabilization	18,000 gpd	Insufficient lime storage and treatment capacity for current peak month sludge flow.

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CHAPTER 5

WASTEWATER FLOWS AND LOADS

CURRENT FLOWS

Operations personnel at the Wastewater Treatment Plant (WWTP) regularly monitor influent and effluent parameters and report these data to the Oregon Department of Environmental Quality (DEQ) on a monthly basis as required by their National Pollutant Discharge Elimination System (NPDES) permit. This chapter summarizes data from the discharge monitoring reports (DMRs) and analyzes recent data to estimate current wastewater flows and loads. Unit flow and loading rates were then developed and used along with population projections presented in Chapter 3 to prepare flow and load projections for future conditions. The flow and load projections serve as the basis for assessing the adequacy of the existing treatment systems and sizing new treatment facilities.

Flow Rates – Definitions of Terms

There are several current plant flow rates that must be determined in order to develop accurate flow rate projections for the design year (2035). These flow rates are defined as follows:

- Average Dry Weather Flow (ADWF) – the average flow at the plant during the dry weather season of May-October.
- Base Sanitary Flow – the average daily sanitary wastewater flow rate. It is equivalent to the ADWF less the dry weather Inflow and Infiltration (I/I).
- Average Wet Weather Flow (AWWF) – the average flow at the plant during the wet weather season of November-April during a year with average rainfall.
- Average Annual Flow (AAF) – the arithmetic mean of all flows.
- Maximum Month Dry Weather Flow (MMDWF) – the anticipated flow when rainfall quantities are at the 1-in-10 year probability level for the month of May.
- Maximum Month Wet Weather Flow (MMWWF) – the plant flow when rainfall quantities are at the 1-in-5 year probability level for the month of January.
- Maximum Day Dry Weather Flow (MDDWF) – the maximum daily plant flow during the dry weather season of May-October.
- Maximum Day Wet Weather Flow (MDWWF) – the flow rate that corresponds to a 24-hour storm event with a 1-in-5 year recurrence interval that occurs during a period of high groundwater and saturated soils.
- Maximum Week Wet Weather Flow (MWWWF) – The maximum weekly average plant flow occurring during the wet weather season of November through April.
- Peak Hour Flow (PHF) – the peak flow rate sustained for one hour that is expected to occur during the MDWWF. This flow is also referred to as the peak instantaneous flow.

Flow Measurement and Rainfall Records

Prior to beginning the flow analysis, it is important to identify any inconsistencies in the data or limitations of flow measurement or pumping equipment. Any atypical conditions that may impact historical flow records should be understood. Rainfall and flow data from November 2002 through December 2008 were used for this analysis.

Table 5-1 summarizes plant flow records for November 2002 through April 2009. As expected, wastewater flows are higher during the wet weather season. Precipitation saturates the soil and raises the groundwater table, causing infiltration of water into the sewers and increasing plant flow. The difference between monthly average influent and effluent flow rates in Table 5-1 is likely due to filter backwash flows being included in the influent measurement. Based on experience with other granular media filters, the portion of the measured influent flow stemming from filter backwash is expected to be between 10 and 15 percent of the total measured flow. A comparison of influent and effluent flows is presented on Figure 5-1. While there is a difference in average influent and effluent flows, there is little difference between influent and effluent peak flows. Because the filter backwash flow appears to impact the average monthly flows, but not the peak flows, the plant effluent flowrate will be used to determine the ADWF, AWWF, and AAF. The plant influent flow rate will be used to determine the MMDWF, MMWWF, MWWWF, and MDWWF.

Strip chart records for the WWTP containing influent and effluent flow data were compared to DMR data and also indicated that the average influent flow rate is higher than the average effluent flow rate.

Table 5-1: Summary of Plant Flow Records

Month	Influent, Flow, MGD	Effluent, Flow, MGD	Max Daily Influent Flow, MGD	Month	Influent, Flow, MGD	Effluent, Flow, MGD	Max Daily Influent Flow, MGD
Nov-02	0.22	0.16	0.45	Jan-06	0.57	0.52	0.91
Dec	0.44	0.39	0.97	Feb	0.44	0.38	0.60
Jan-03	0.36	0.32	0.64	Mar	0.39	0.35	0.49
Feb	0.28	0.24	0.43	Apr	0.32	0.26	0.43
Mar	0.27	0.24	0.39	May	0.26	0.20	0.32
Apr	0.30	0.27	0.40	Jun	0.27	0.20	0.32
May	0.24	0.21	0.29	Jul	0.27	0.19	0.32
Jun	0.23	0.19	0.34	Aug	0.26	0.18	0.30
Jul	0.24	0.19	0.34	Sep	0.25	0.18	0.30
Aug	0.23	0.19	0.25	Oct	0.23	0.16	0.27
Sep	0.22	0.18	0.27	Nov	0.27	0.22	0.41
Oct	0.21	0.18	0.25	Dec	0.36	0.32	0.90
Nov	0.21	0.19	0.28	Jan-07	0.33	0.28	0.53
Dec	0.33	0.32	0.61	Feb	0.32	0.29	0.61
Jan-04	0.34	0.33	0.65	Mar	0.27	0.25	0.39
Feb	0.33	0.32	0.58	Apr	0.24	0.22	0.27
Mar	0.26	0.24	0.47	May	0.20	0.18	0.32
Apr	0.24	0.22	0.29	Jun	0.19	0.16	0.22
May	0.22	0.20	0.25	Jul	0.19	0.15	0.23
Jun	0.22	0.19	0.32	Aug	0.19	0.15	0.21
Jul	0.22	0.19	0.25	Sep	0.19	0.15	0.22
Aug	0.23	0.20	0.25	Oct	0.20	0.16	0.26
Sep	0.23	0.20	0.27	Nov	0.23	0.19	0.45
Oct	0.24	0.21	0.28	Dec	0.36	0.33	0.65
Nov	0.25	0.22	0.55	Jan-08	0.42	0.42	0.99
Dec	0.29	0.28	0.66	Feb	0.30	0.31	0.48
Jan-05	0.31	0.28	0.48	Mar	0.27	0.24	0.37
Feb	0.27	0.23	0.42	Apr	0.24	0.22	0.27
Mar	0.27	0.25	0.50	May	0.23	0.20	0.28
Apr	0.29	0.28	0.38	Jun	0.23	0.18	0.40
May	0.30	0.27	0.56	Jul	0.22	0.16	0.25
Jun	0.22	0.18	0.27	Aug	0.21	0.15	0.23
Jul	0.23	0.18	0.27	Sep	0.20	0.16	0.23
Aug	0.23	0.18	0.28	Oct	0.20	0.15	0.23
Sep	0.33	0.26	0.44	Nov	0.23	0.18	0.29
Oct	0.30	0.25	0.35	Dec	0.29	0.23	0.61
Nov	0.34	0.29	0.48	Jan-09	0.25	0.21	0.35
Dec	0.48	0.45	1.10	Feb	0.25	0.21	0.47
				Mar	0.30	0.27	0.46
				Apr	0.24	0.18	0.28
				Maximum	0.57	0.52	1.10
				Minimum	0.19	0.15	0.21
				Average	0.27	0.24	0.41

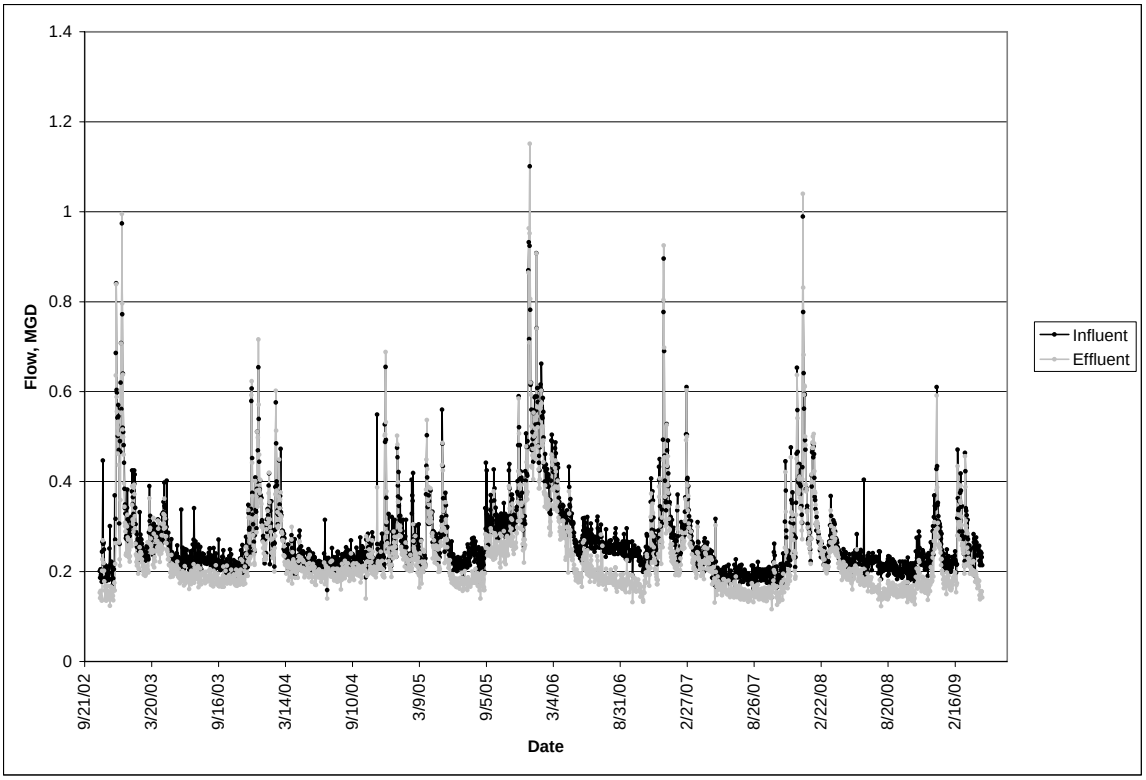


Figure 5-1: Influent VS Effluent Flows

As Canyonville does not have a local weather station, rainfall data was taken from the National Weather Service Riddle Station # 357169. An existing statistical analysis of rainfall data provided by the National Oceanic and Atmospheric Administration (NOAA) was used to determine statistical rainfall quantities, and a correlation between rainfall and plant influent and effluent flows was developed. Daily plant flow and rainfall quantities for the November 2002 through April 2009 are shown on Figure 5-2.

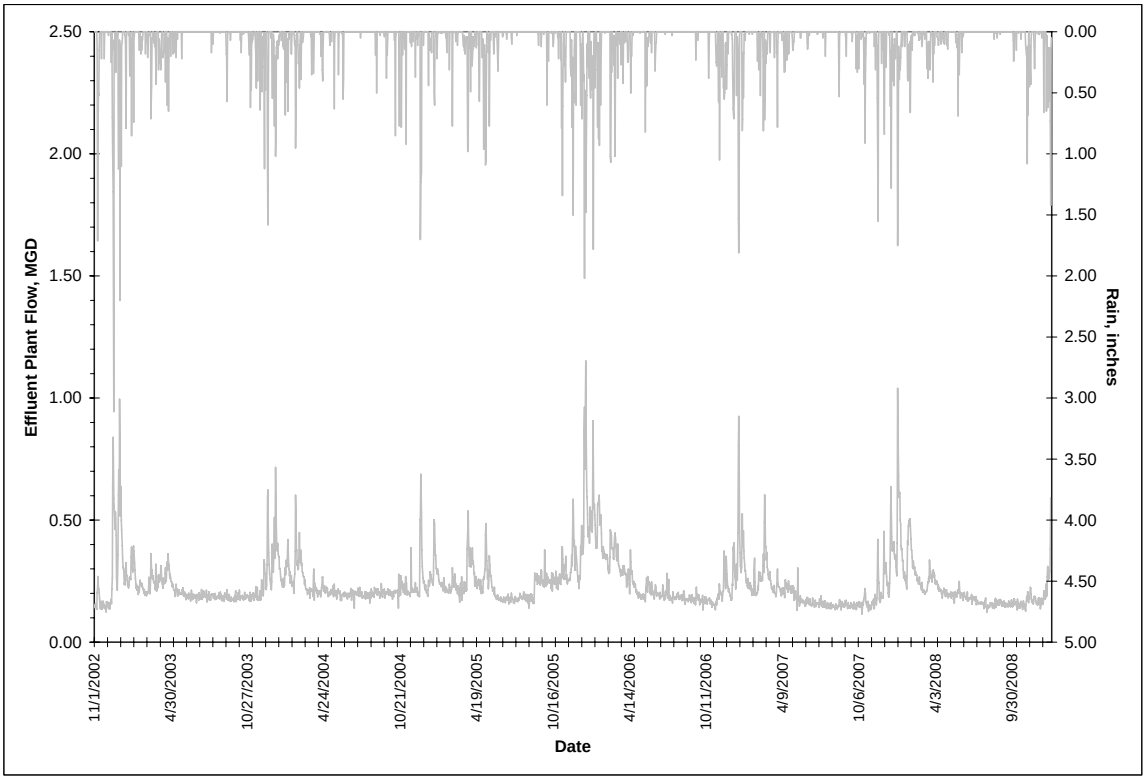


Figure 5-2: Historical Daily Plant Flow and Rainfall

Dry Weather Flow

Average Dry Weather Flow (ADWF). The dry weather season is defined in the City's NPDES permit as May through October. Flow data from May 2008 through October 2008 were used to determine ADWF. The ADWF in 2008 was 0.17 MGD, based on the effluent flow rate. For dry weather seasons 2003 through 2008, flows range from 0.15 to 0.27 MGD. It should be noted that the ADWF of 0.17 MGD has decreased from the value of 0.21 MGD given in the *1998 Facilities Plan*. The decrease is attributable repairs made to a section of sewer crossing under Canyon Creek that had been identified as a major source of I/I.

Base Sanitary Flow. The Base Sanitary Flow was determined by examining water use records for January 2008. Because the number of water and sewer system users are comparable (567 water accounts and 549 sewer accounts), winter water use during the month of January is a good measure of non-consumptive water usage. Consequently, the total Base Sanitary Flow was estimated to be 0.15 MGD.

Dry weather I/I. The dry weather I/I is determined by subtracting the Base Sanitary Flow from the ADWF. Dry weather I/I can also be approximated as the minimum night time flows during dry weather, and is estimated to be 0.02 MGD. This was confirmed by reviewing influent flow strip charts.

Maximum Month Dry Weather Flow (MMDWF). MMDWF is largely influenced by rainfall. To determine the MMDWF, total monthly rainfalls for the months of January to May for the most recent year on record (2008) are plotted against their respective monthly average influent flows. The MMDWF corresponds to the estimated 1-in-10 year May rainfall (90th percentile on the NOAA Clim 20 Study) of 3.04 inches as published by the National Oceanic and Atmospheric Administration (NOAA), and is presented in Figure 5-3. By comparison, the highest monthly average dry weather flow reported for the years 2003 through 2008 was 0.33 MGD in September 2005 (see Table 5-1).

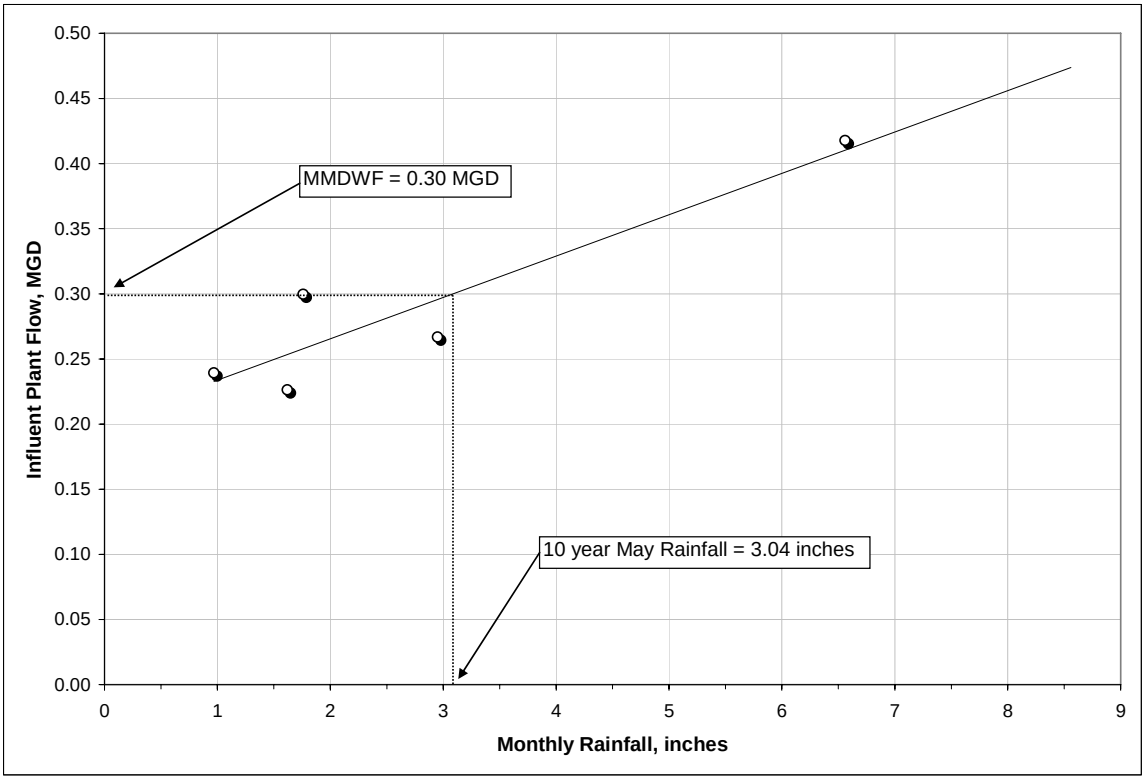


Figure 5-3: Maximum Month Dry Weather Flow (MMDWF)

Maximum Day Dry Weather Flow (MDDWF). To determine the MDDWF, maximum daily flows for the dry weather season, May through October, for the years 2003 through 2008 were reviewed. As indicated in Table 5-1, the MDDWF occurring between 2003 through 2008 is 0.56 MGD, occurring in May 2005.

Wet Weather Flow

Average Wet Weather Flow (AWWF). The wet weather permit season extends from November through April. To determine the AWWF, wet weather season rainfalls for November 2002 through April 2008 are plotted against their respective monthly average effluent flows. The AWWF corresponds to the average total wet weather season rainfall. The average November through April rainfall for the years 1971 through 2000 as published by NOAA is 24.97 inches, and the AWWF is estimated on Figure 5-4.

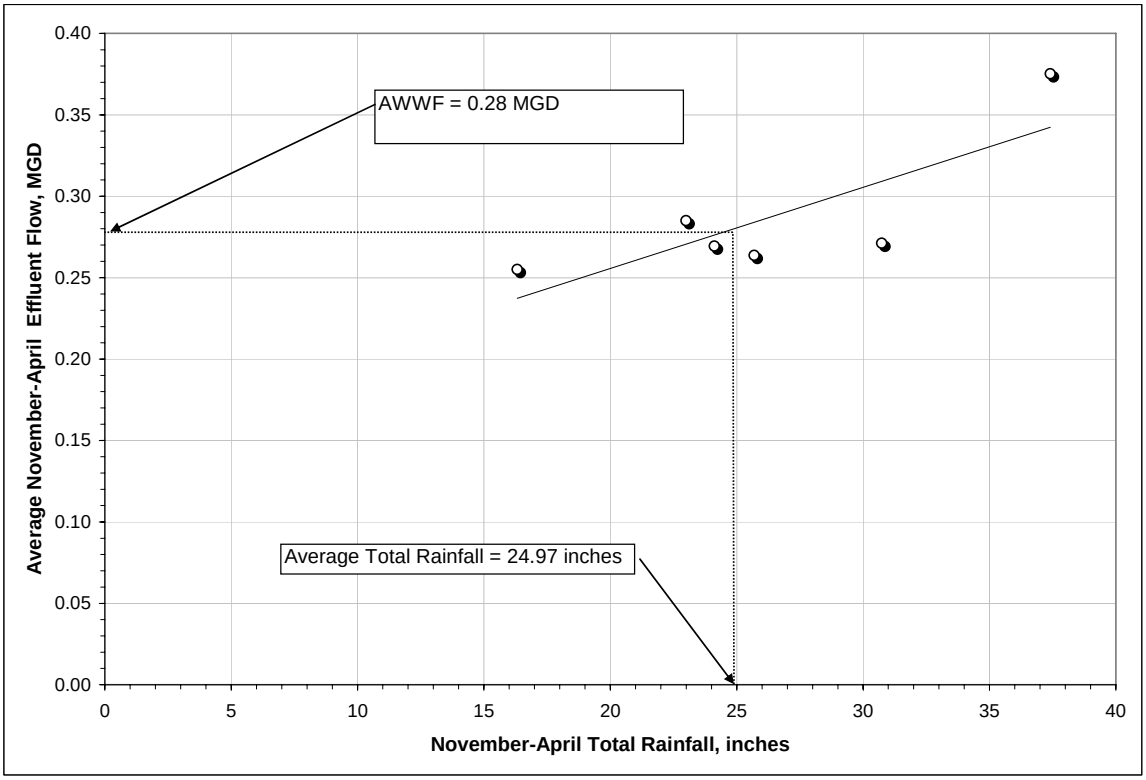


Figure 5-4: Average Wet Weather Flow (AWWF)

Average Annual Flow (AAF). Average Annual Flow (AAF) is the average of all flows for the period of record and is calculated to be 0.23 MGD.

The ADWF, AWWF, and AAF flows for years 2002 through 2008 are compared to yearly total rainfall amounts on Figures 5-5, 5-6, and 5-7, respectively.

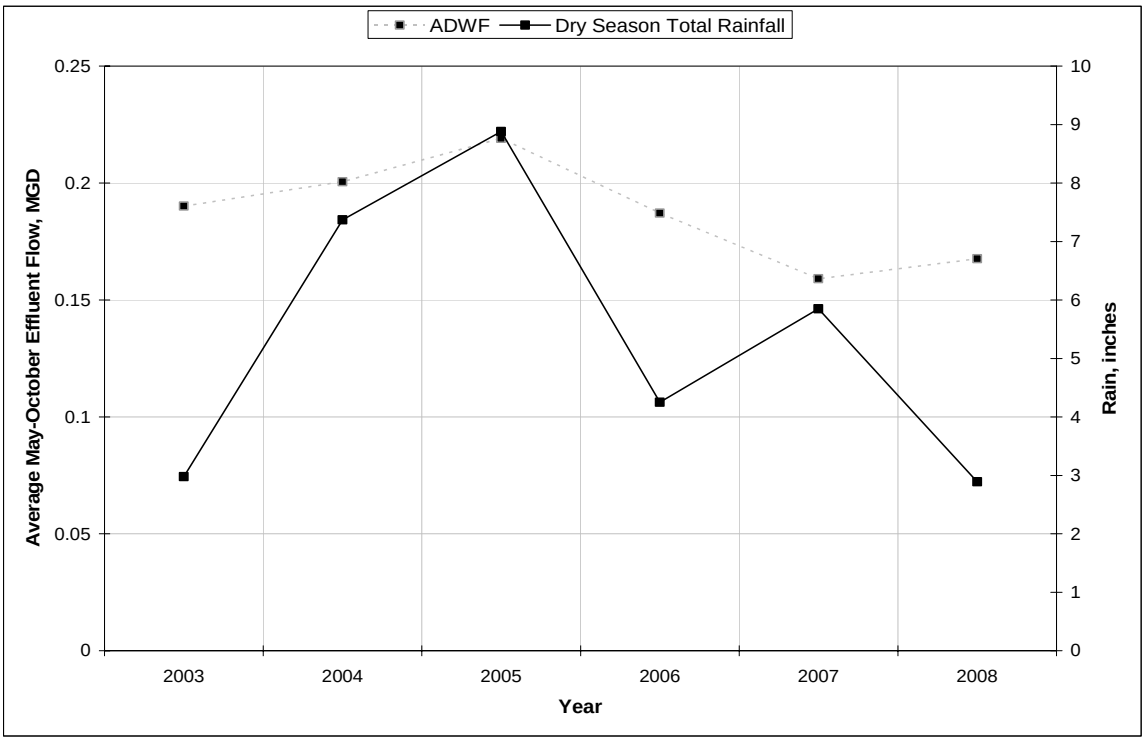


Figure 5-5: ADWF and Rainfall Comparison

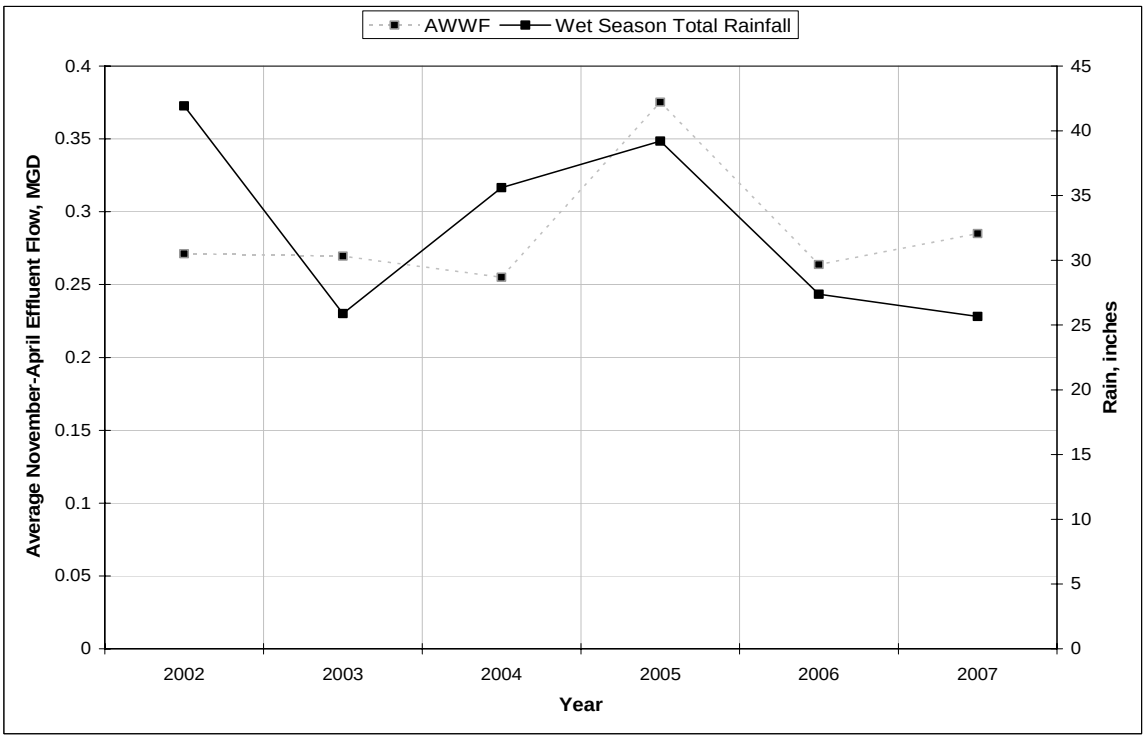


Figure 5-6: AWWF and Rainfall Comparison

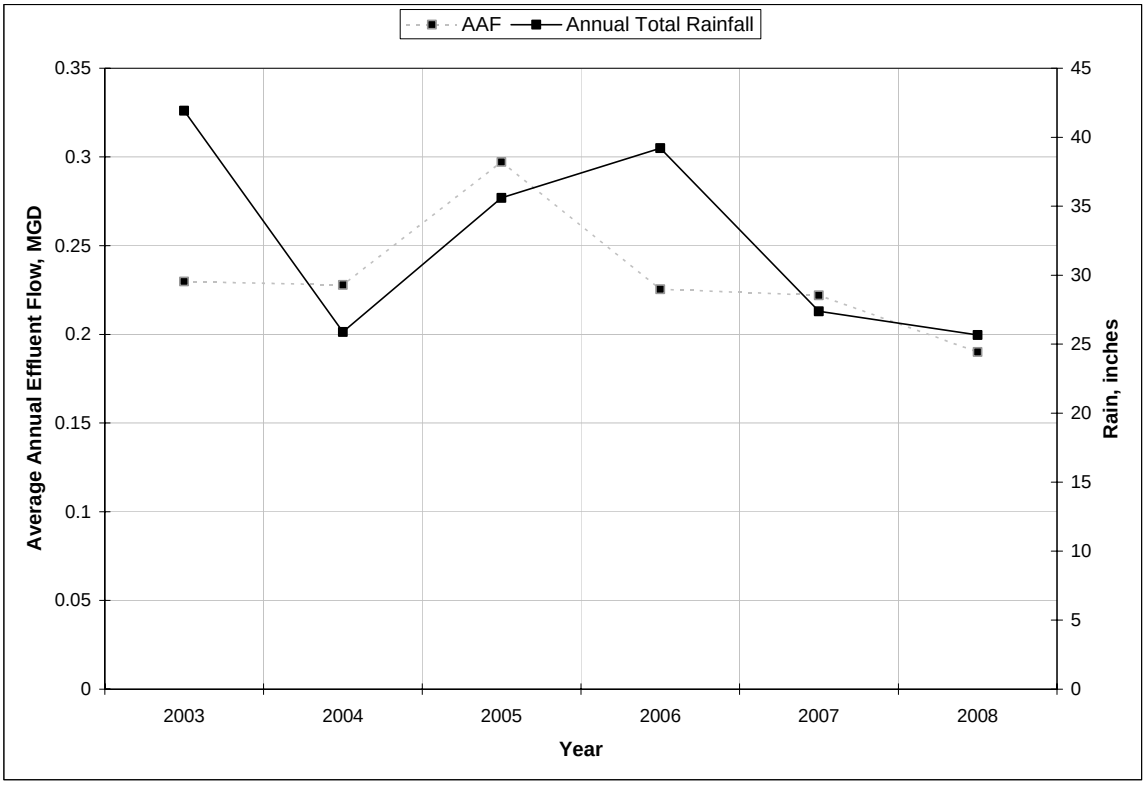


Figure 5-7: AAF and Rainfall Comparison

Maximum Month Wet Weather Flow (MMWWF). To determine the MMWWF, total monthly rainfalls for the months of January through May for the most recent year on record (2008) are plotted against their respective monthly average influent flows in Figure 5-8. Per DEQ guidelines, MMWWF is determined as the flow that corresponds to the 1-in-5 year January rainfall. However, for this analysis, the 1-in-5 year (80th percentile from the NOAA Clim 20 Study) December rainfall total was used to determine the MMWWF because it exceeds the 1-in-5 year January rainfall. The highest monthly average wet weather influent flow reported occurring between 2002 through 2009 was 0.57 MGD in January 2006 (see Table 5-1).

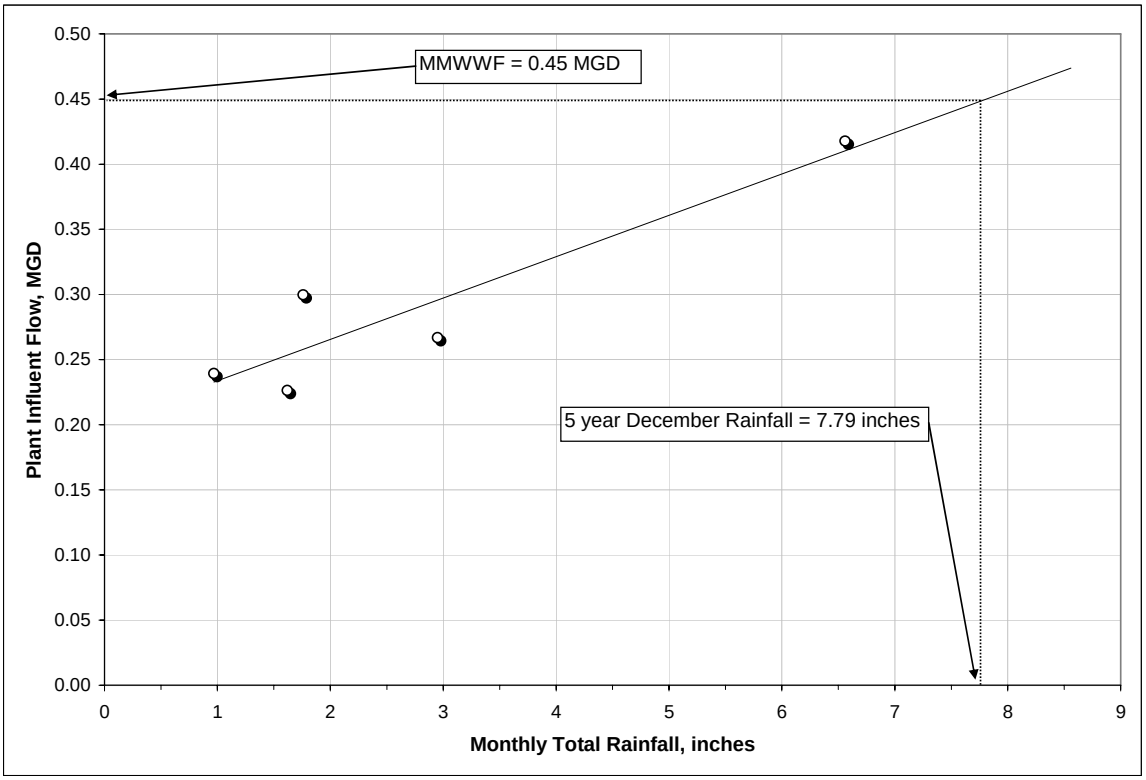


Figure 5-8: Maximum Month Wet Weather Flow (MMWWF)

Maximum Day Wet Weather Flow (MDWWF). To determine the MDWWF, daily rainfall amounts during storm events are plotted against corresponding influent flows.
Storm events with daily rainfall amounts of 1 inch or greater occurring during periods of high groundwater (December through February) for years 2002 through 2009 were identified and plotted against their respective influent flows. Events for which there was little or no rainfall on the previous day were excluded from the analysis. The plot is presented as Figure 5-9. The MDWWF corresponds to the 1-in-5 year, 24-hour storm event. The numeric value of this event is uncertain. Several options are available:

- The City of Riddle used a value 2.84 inches
- A review of the 50 largest events on record suggests a value of approximately 2.5 inches
- NOAA isopluvial maps (Atlas 2, Volume X, Figure 26) show a value of 3.0 inches

- DEQ's December 2009 *SSO Enforcement Internal Management Directive* lists a value of 3.5 inches

It was decided to use NOAA isopluvial map value of 3.0 inches in 24 hours for the purposes of estimating MDWWF.

As shown on Figure 5-9, there is no definitive relationship between daily rainfall and daily flow for the given storm event criteria. However, the MDWWF as determined by this approach is 1.14 MGD. The MDWWF recorded between November 2002 and April 2009 was 1.10 MGD (December 31, 2005).

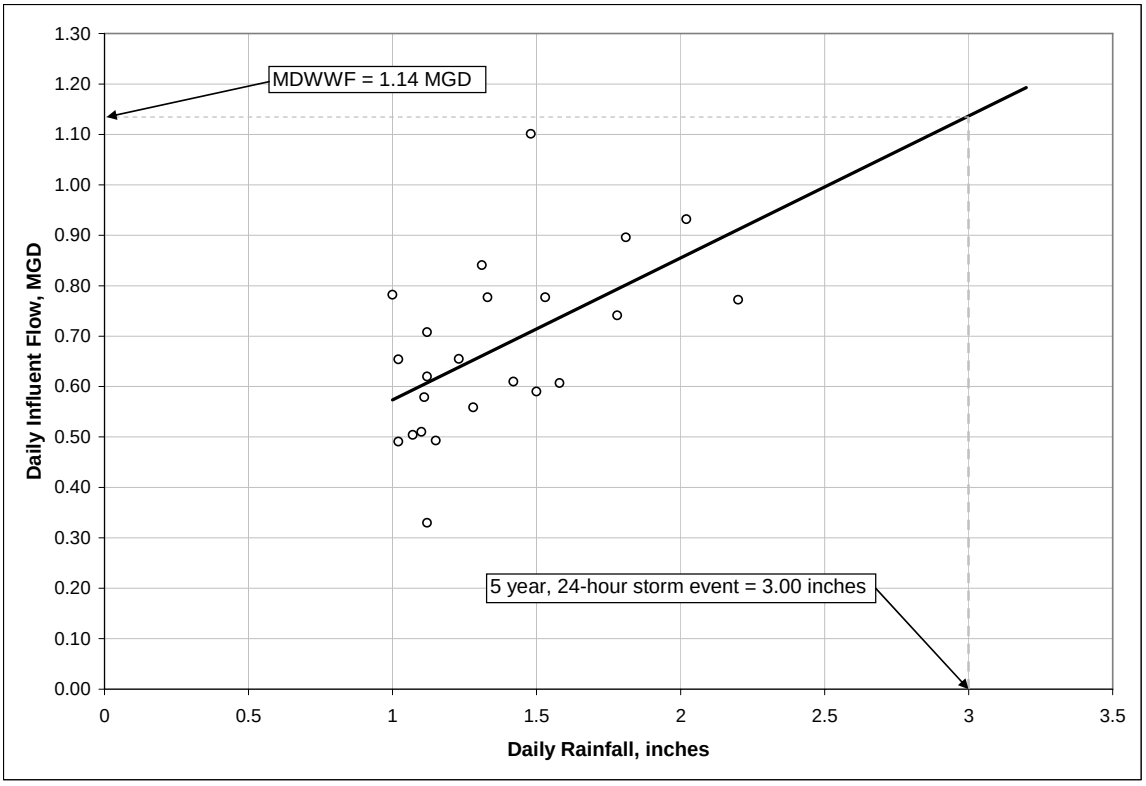


Figure 5-9: Maximum Day Wet Weather Flow (MDWWF)

To better predict flow during the 1-in-5 year, 24-hour storm event, a regression model was created and calibrated with WWTP flow data. The model predicts a simulated plant effluent flow by adjusting the coefficients and powers of a multi-variable equation. The model takes the form of the following equation:

$$Q = A(X_{90})^a + B(X_{60})^b + C(X_{30})^C + D(X_7)^d + E(X_3)^e + F(X_2)^f + G(X_1)^g + H(X_{-1})^h + Y$$

Where: Q = Simulated plant influent flow (MGD)

X_{90} = Average previous 90 days rainfall, inches

X_{60} = Average previous 60 days rainfall, inches

X_{30} = Average previous 30 days rainfall, inches

X_7 = Average previous 7 days rainfall, inches

X_3 = Average previous 3 days rainfall, inches

X_2 = Average previous 2 days rainfall, inches

X_1 = Rainfall occurring the day flow measurement was taken, inches

X_{-1} = Rainfall occurring the previous day flow measurement was taken, inches

Y= Constant representing base flow

Rainfall data from 2002 through 2009 and initial values for coefficients and powers were entered into the equation, and a simulated flow is produced. The simulated *Q* is then subtracted from its respective actual flow and the result is squared. The sum of the squares for each day represents the overall error of the model. To calibrate the model, the error is reduced to its minimum value by adjusting the coefficients and powers. Figure 5-10 presents the results of the model calibration. Actual flows on the X axis are plotted against the simulated Qs on the y axis. A model with zero error would show all data points on the line simulated flow = actual flow. The values of the coefficients and powers are presented in Table 5-2.

Table 5-2: Flow Simulation Model Coefficients

A =	1.07767
a =	1.892011
B =	0.539591
b =	2.253808
C =	0.785883
c =	1.998394
D =	0.158556
d =	1.352203
E =	0.061917
e =	1.007917
F =	0.000416
f =	10.70839
G =	0.023478
g =	0.923312
H =	0.113549
h =	1.652981
Y =	0.219223

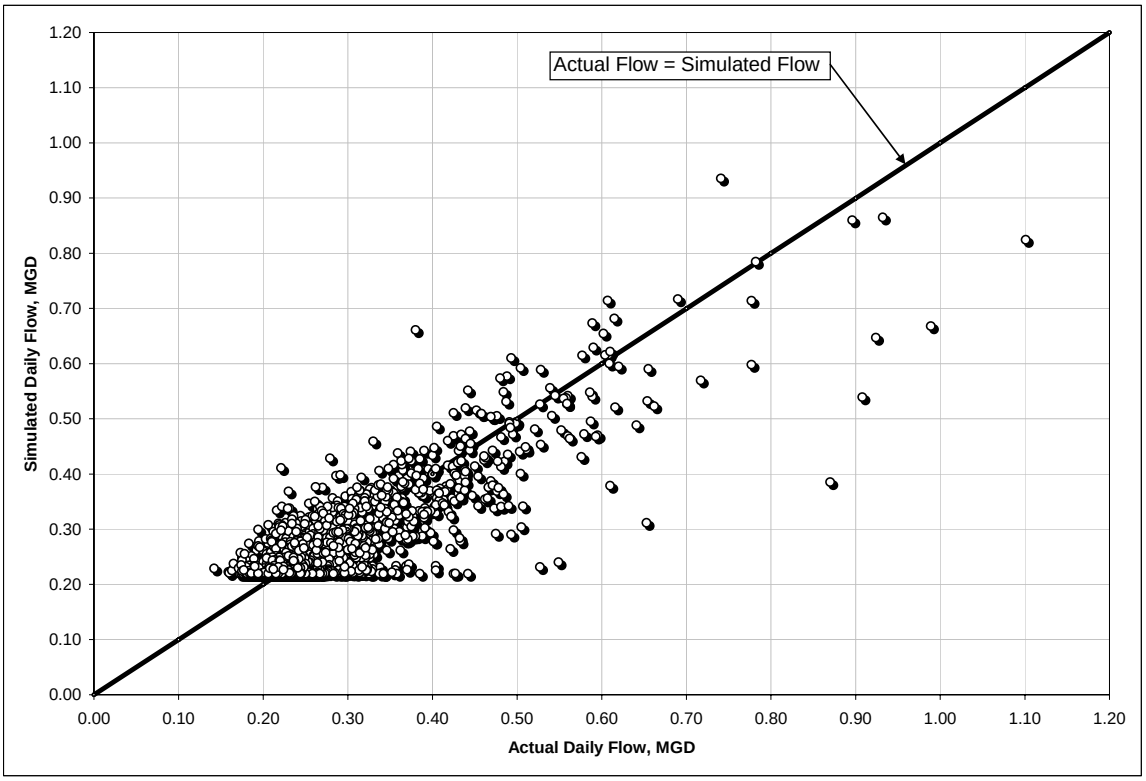


Figure 5-10: Regression Model Calibration for Daily Flows

Once calibrated, the model was then applied to historical rainfall data from the Riddle station. The results are in the form of simulated WWTP flows corresponding to the historical rain data, and are presented on Figure 5-11. Because the model is used to predict peak flows, days receiving less than 1-inch of rainfall are not plotted, although they were included in the development of the simulation.

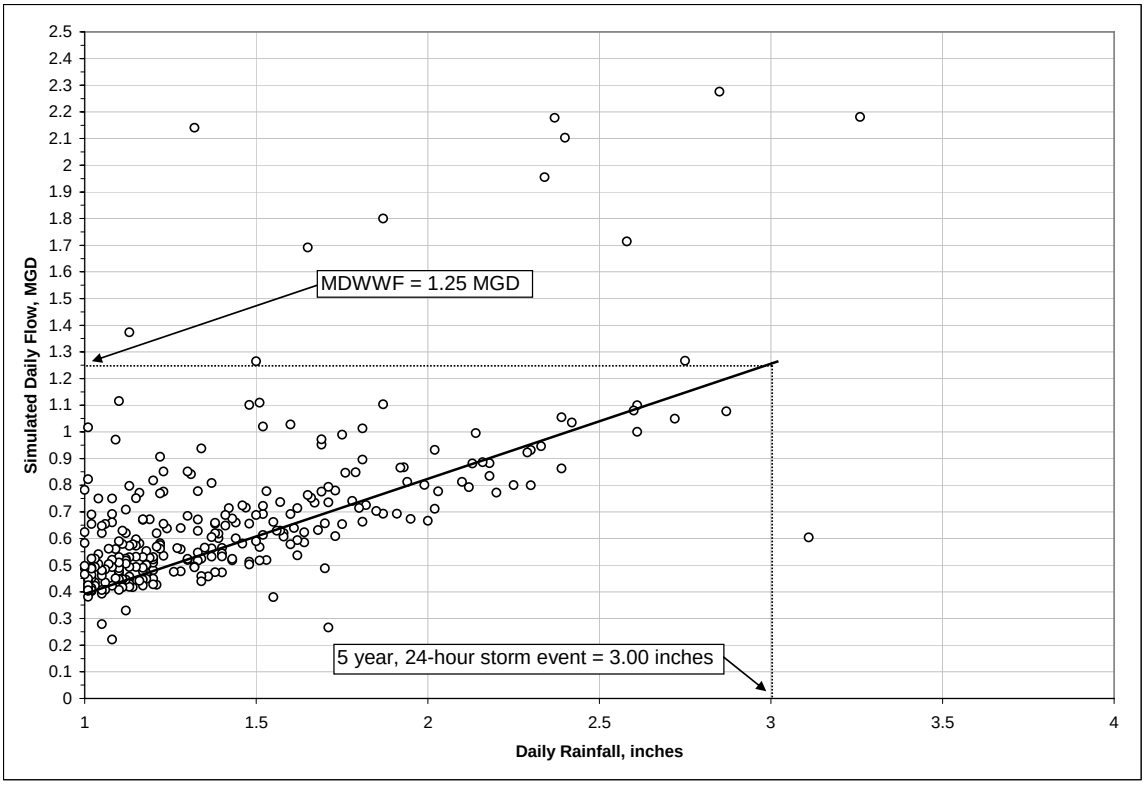


Figure 5-11: Simulated Daily Flows

The simulated daily flow corresponding to the 1-in-5 year, 24-hour storm event of 3 inches of rain is presented on Figure 5-11.

MWWWF and PHF

By plotting the AAF, MMWWF and MDWWF on a log probability plot, it is possible to estimate the MWWWF and PHF. The flows are plotted according to their respective exceedance probability on log-probability scales. AAF has a 50 percent exceedance probability, while MMWWF has an exceedance probability of one month in 12, or 8.3 percent. Similarly, the MDWWF has an exceedance probability of one day per year, or 0.27 percent. MWWWF and PHF are taken from this plot according to their respective exceedance probabilities 1.9 percent and 0.011 percent, respectively. The log-probability plot is provided in Appendix E.

Summary of Current Flows

A flow summary including ADWF peaking factors is shown in Table 5-3.

Table 5-3: Summary of Current Flows

Parameter	Year 2008 flow, MGD	Existing Sewers Peaking Factors
Service Area Population	1,730	
Base Sanitary Flow	0.15	
Dry Weather I/I	0.02	
ADWF	0.17	1.00
AAF	0.23	1.35
AWWF	0.28	1.65
MMDWF	0.30	1.76
MMWWF	0.45	2.65
MDDWF	0.56	3.29
MWWWF	0.68	4.00
MDWWF	1.25	7.35
PHF	1.60	9.41

CURRENT LOADS

The wastewater loading components of principal interest are the five-day biochemical oxygen demand (BOD_5) and total suspended solids (TSS) of the raw sewage. BOD_5 is a measure of the amount of oxygen required to biologically oxidize the organic material in the wastewater over a specific time period. A 5-day BOD test is conventionally used for domestic wastewater testing. TSS is a measure of the particulate material suspended in the wastewater. The loading parameters of interest are the annual average loading, maximum month loading, maximum week loading, and maximum day loading.

Current and projected BOD and TSS loadings at treatment facilities must be estimated in order to determine:

- Secondary treatment process size
- Aeration system design
- Solids treatment and handling system design
- Solids production rates

The primary nutrients of interest at a wastewater treatment facility are nitrogen and phosphorus. In domestic wastewater, nitrogen is primarily in the form of ammonia, while the majority of the phosphorus is in the form of soluble phosphate. Nutrients are necessary for the growth of microorganisms and aquatic plant life. However, many receiving waters have excessive algal growth that is caused in part by high nitrogen and phosphorus levels. Nutrient concentrations in the raw wastewater must be sufficient to support the growth of microorganisms in the biological treatment process.

However, most wastewaters contain more of these constituents than needed to support the process and the excess passes through to the effluent unless specific nutrient reduction measures are taken in the design and operation of the facilities. Therefore, many treatment facilities incorporate treatment processes which remove nutrients prior to effluent discharge.

Analysis of Plant Records

DMRs were analyzed to develop average and maximum month, week, and day BOD and TSS loadings. By examining monthly BOD and TSS records, it is possible to detect seasonal variations in loadings. For the 2008 population of 1,730 residents, the average BOD load of 440 pounds per day (ppd) corresponds to a unit load of 0.25 pounds per capita per day (pcd), slightly higher than the textbook value of 0.20 pcd. The apparent high unit loading rates are in part due to the fact that the filter backwash water and aerobic digester supernatant recycle streams drain to the influent manhole or raw sewage pumping station, and therefore are included in the influent samples. In terms of plant operations, knowing the total load to the secondary treatment process is more important than knowing the load associated with the raw sewage alone. However, in terms of accurately defining the characteristics of the raw sewage, it is necessary to sample the raw sewage upstream of the recycle streams. Depending on the design and operation of the plant, 5 to 15 percent of the process load could be associated with recycle streams. For the purposes of this study, we will estimate the recycle stream load at 10 percent of the measured total load. This reduces the average unit BOD load to 0.22 pcd. Another source of increased BOD loadings may be due to the restaurants which primarily serve freeway travelers and casino patrons. Restaurants can contribute a significant load to the treatment plant.

Table 5-4 lists monthly averages for BOD and TSS loadings and concentrations. Average wet weather BOD loads are essentially equivalent to average dry weather BOD loads. Average wet weather TSS loads are approximately 10 percent greater than dry weather TSS loads.

Table 5-4: Monthly BOD and TSS Loadings

	Flow		Influent				Effluent			
Month	Influent	Effluent	BOD, mg/L	BOD, ppd	TSS, mg/L	TSS, ppd	BOD, mg/L	BOD, ppd	TSS, mg/L	TSS, ppd
Nov-02	0.22	0.16	226	415	214	394			28	21
Dec	0.44	0.39	136	495	114	415			203	63
Jan-03	0.36	0.32	137	407	127	379			122	46
Feb	0.28	0.24	210	491	186	435			41	21
Mar	0.27	0.24	196	441	188	423			84	42
Apr	0.30	0.27	204	519	186	472			60	26
May	0.24	0.21	260	515	217	429			28	16
Jun	0.23	0.19	251	482	194	373			19	12
Jul	0.24	0.19	243	487	174	347			7	5
Aug	0.23	0.19	230	437	164	312			3	2
Sep	0.22	0.18	250	461	160	295			7	5
Oct	0.21	0.18	246	433	160	282			12	8
Nov	0.21	0.19	295	528	160	287			12	8
Dec	0.33	0.32	164	456	108	299			53	20
Jan-04	0.34	0.33	149	418	127	354			81	30
Feb	0.33	0.32	173	477	131	360			127	48
Mar	0.26	0.24	208	455	162	354			34	17
Apr	0.24	0.22	250	497	170	336			29	16
May	0.22	0.20	286	535	238	445			31	18
Jun	0.22	0.19	251	456	177	321			16	10
Jul	0.22	0.19	245	446	186	338	5	8	3	4
Aug	0.23	0.20	222	420	180	340	6	10	6	9
Sep	0.23	0.20	229	439	166	319	7	12	6	10
Oct	0.24	0.21	268	536	217	432	8	14	5	9
Nov	0.25	0.22	223	460	180	372	8	15	4	7
Dec	0.29	0.28	186	457	159	389	10	23	6	13
Jan-05	0.31	0.28	232	592	167	427	10	23	7	17
Feb	0.27	0.23	258	581	267	601	10	19	15	28
Mar	0.27	0.25	199	443	189	420	11	23	15	31
Apr	0.29	0.28	98	239	168	409	11	25	19	44
May	0.30	0.27	186	472	155	394	5	12	5	11
Jun	0.22	0.18	282	522	235	433	4	6	9	13
Jul	0.23	0.18	206	395	193	369	4	6	7	10
Aug	0.23	0.18	214	410	219	421	7	10	9	14
Sep	0.33	0.26	273	747	244	667	6	12	9	19
Oct	0.30	0.25	206	523	172	436	6	12	10	21
Nov	0.34	0.29	178	511	119	341	18	44	17	41
Dec	0.48	0.45	150	605	121	485	16	59	23	85
Jan-06	0.57	0.52	134	635	135	640	22	96	17	74
Feb	0.44	0.38	179	655	143	523	13	40	14	43
Mar	0.39	0.35	167	544	128	417	15	45	15	44
Apr	0.32	0.26	197	526	205	545	15	31	16	34
May	0.26	0.20	200	620	245	507	4	6	7	11
Jun	0.27	0.20	219	500	196	447	3	5	6	10
Jul	0.27	0.19	373	826	281	621	2	2	3	4
Aug	0.26	0.18	255	546	203	435	5	7	6	9
Sep	0.25	0.18	181	381	173	364	4	6	5	8
Oct	0.23	0.16	197	384	249	485	5	7	7	9
Nov	0.27	0.22	220	487	182	402	14	26	13	23
Dec	0.36	0.32	126	383	153	463	8	20	4	11
Jan-07	0.33	0.28	175	480	217	595	12	28	12	28
Feb	0.32	0.29	143	378	172	454	9	22	10	25
Mar	0.27	0.25	182	414	206	469	13	26	12	26
Apr	0.24	0.22	213	425	270	538	10	19	10	19
May	0.20	0.18	287	490	220	375	6	9	4	6
Jun	0.19	0.16	204	322	239	378	3	3	3	4
Jul	0.19	0.15	221	359	203	329	3	3	2	2
Aug	0.19	0.15	246	386	231	363	3	4	3	4
Sep	0.19	0.15	273	438	256	410	3	4	2	3
Oct	0.20	0.16	212	348	228	375	3	4	5	6
Nov	0.23	0.19	212	409	260	501	6	10	6	9
Dec	0.36	0.33	116	351	103	312	7	18	8	22
Jan-08	0.42	0.42	89	311	102	355	8	28	9	30
Feb	0.30	0.31	134	335	161	402	12	29	9	24
Mar	0.27	0.24	188	418	171	381	8	17	7	14
Apr	0.24	0.22	225	449	312	623	10	19	9	15
May	0.23	0.20	261	492	285	537	6	10	5	8
Jun	0.23	0.18	238	456	194	372	6	9	6	10
Jul	0.22	0.16	354	664	234	439	3	4	6	8
Aug	0.21	0.15	253	445	204	359	4	5	3	4
Sep	0.20	0.16	255	435	228	389	3	4	4	6
Oct	0.20	0.15	256	432	220	371	3	4	6	8
Nov	0.23	0.18	155	300	233	452	3	5	13	19
Dec	0.29	0.23	229	546	277	661	5	9	8	15
Jan-09	0.25	0.21	225	474	199	418	9	16	12	21
Feb	0.25	0.21	251	527	254	534	13	23	13	24
Mar	0.30	0.27	169	417	162	398	9	20	12	27
Apr	0.24	0.18	228	456	388	775	7	10	5	7
Maximum	0.57	0.52	373	826	388	775	22	96	203	85
Minimum	0.19	0.15	89	239	102	282	2	2	2	2
Average	0.27	0.24	215	470	195	428	8	17	19	19
Wet Weather Maximum	0.57	0.52	295	655	388	775	22	96	203	85
Wet Weather Minimum	0.21	0.16	89	239	102	287	3	5	4	7
Wet Weather Average	0.31	0.28	186	462	183	448	11	26	29	28
Dry Weather Maximum	0.33	0.27	373	826	285	667	8	14	31	21
Dry Weather Minimum	0.19	0.15	181	322	155	282	2	2	2	2
Dry Weather Average	0.23	0.19	248	479	209	404	4	7	8	9

Daily BOD and TSS loadings and concentrations are presented on Figures 5-12 through 5-15.

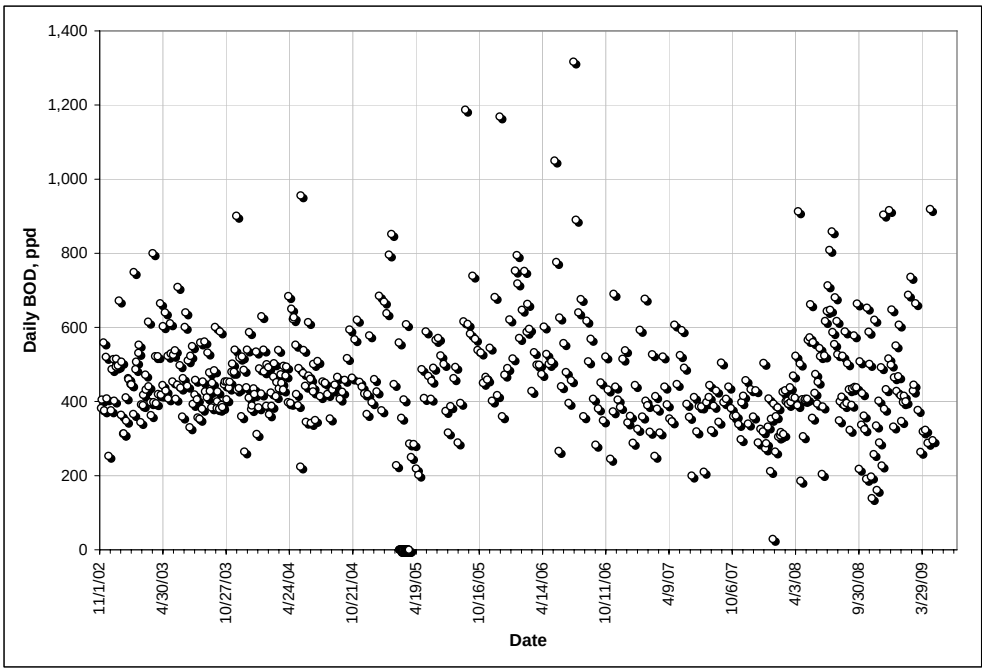


Figure 5-12: Daily BOD Loadings

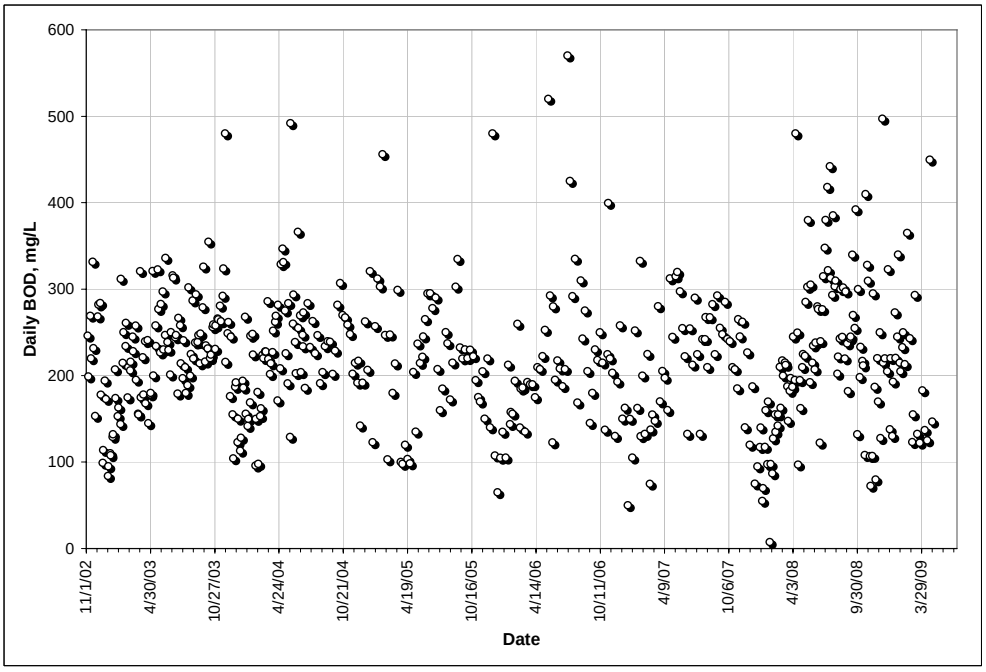


Figure 5-13: Daily BOD Concentrations

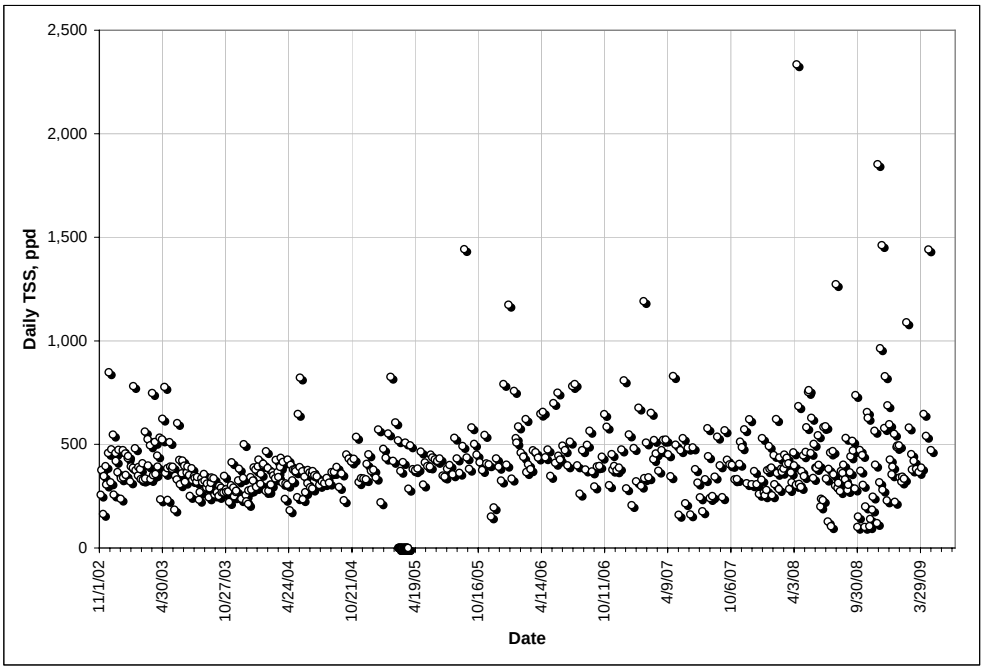


Figure 5-14: Daily TSS Loadings

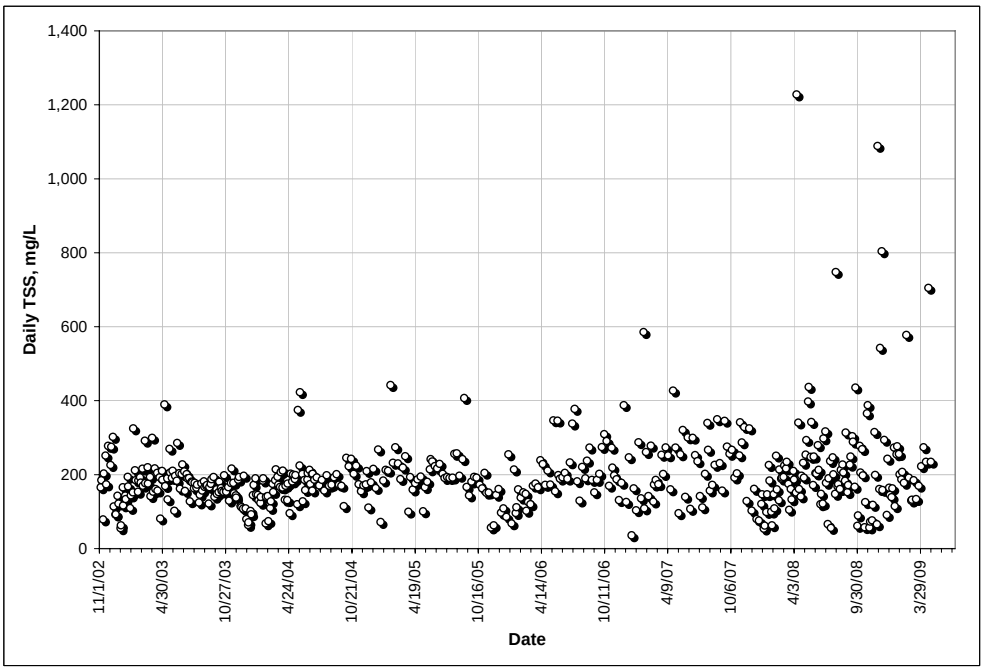


Figure 5-15: Daily TSS Concentrations

Table 5-5 presents maximum month, week, and day BOD and TSS loadings for November 2002 through April 2009. Maximum week loadings for BOD and TSS were determined by percent probabilities of exceedance. The analysis is provided in Appendix E.

Table 5-5: 2002 - 2008 Plant Loadings

Parameter	Load based on plant records, ppd	Estimated recycle stream load, ppd[(a)]	Estimated sewage load (rounded), ppd
Maximum month BOD	830	80	750
Maximum week BOD	1,000	100	900
Maximum day BOD	1,190	120	1,100
Maximum month TSS	780	80	700
Maximum week TSS	1,050	110	940
Maximum day TSS	1,460	150	1,300

Note:

(a) Recycle stream load in estimated to be 10 percent of plant loading records.

Figure 5-12 depicts measured daily plant BOD loading for November 2002 through April 2009. The highest daily BOD load recorded was 1,317 ppd on 11 July 2006; however, this load is regarded as an outlier. The second, third and fourth highest samples are 1,187, 1,169, and 1,050 ppd, respectively. Because these reported loads are in relatively close agreement they will be used to establish the peak day load. A value of 1,190 ppd is used as the peak day BOD loading. Assuming that the recycle streams account for 10 percent of the load, the raw sewage peak day BOD loading is approximately 1,100 ppd.

Figure 5-14 depicts measured daily plant TSS loading for November 2002 through April 2009. The highest daily TSS load recorded was 2,335 ppd on 10 April 2008. The second highest sample is 1,853 on 27 November 2008. However, these loadings are considered to be outliers. The third, fourth, and fifth highest samples are 1,462, 1,443, and 1,441 ppd, respectively. Because these reported loads are in relatively close agreement they will be used to establish the peak day load. A value of 1,460 ppd is used as the peak day TSS loading. Assuming that the recycle streams account for 10 percent of the load, the raw sewage peak day TSS loading is approximately 1,300 ppd.

Average annual BOD and TSS loads for 2008 are presented in Table 5-6.

Table 5-6: 2008 Plant Loadings

Parameter	Load based on plant records, ppd	Estimated recycle stream load, ppd[(a)]	Estimated raw sewage load (rounded), ppd
Annual average BOD	440	40	400
Annual average TSS	450	50	400

Note:

(a) Recycle stream load is estimated to be 10 percent of plant loading records.

Other Parameters. There are a number of other constituents in wastewater that influence plant operations, including grit, grease, and nutrients.

Grit. Grit production at a WWTP can vary dramatically, and is a function of the following:

- Sewer condition – sewers with numerous large cracks and holes will allow soil and gravel into the collection system, increasing grit quantities at the plant.
- Soil type – sand soils can cause increased grit loads at the plant.
- Grit removal system – effective grit removal systems will remove more grit

The WWTP's grit removal system is currently non-functional. For a plant the size of Canyonville's, textbook grit production values range from 0.1 to 7 cubic feet per day, with 0.5 cubic feet per day considered a typical value. Canyonville WWTP operators report a grit accumulation rate of 2 to 3 cubic feet per day (approximately 0.5 to 0.75 cubic yards per week) prior to the failure of the system.

Nutrients. Nutrients of primary concern at a WWTP are nitrogen and phosphorus. The Canyonville WWTP's discharge permit has not required sampling of the raw sewage for nitrogen and phosphorus; therefore, no nutrient data is available. Typically, the majority of the nitrogen in raw sewage is in the form of ammonia; average concentrations range from 15 to 30 mg/L. Raw sewage phosphorus concentrations are usually between 4 and 8 mg/L, with the majority of the phosphorus in a soluble form, such as phosphate.

UNIT DESIGN FLOWS AND LOADS

Table 5-7 presents unit design values for the year 2008. Included are flow values in gallons per capita per day (gcd) and loading values in pounds per capita per day (pcd).

Table 5-7: Unit Design Values

Parameter	Year 2008
Service area population	1,730
ADWF, gcd	100
AWWF, gcd	160
AAF, gcd	130
Annual average BOD, pcd	0.23
Maximum month BOD, pcd	0.43
Maximum week BOD, pcd	0.52
Maximum day BOD, pcd	0.64
Annual average TSS, pcd	0.23
Maximum month TSS, pcd	0.40
Maximum week TSS, pcd	0.54
Maximum day TSS, pcd	0.75

FLOW AND LOAD PROJECTIONS

Canyonville's service area includes residential, commercial and public users. Several motels are also within the City's service area. The City currently has no industrial users, and because there is no industrial land available, no future industrial users are anticipated. Businesses and schools are expected to grow at the same rate as overall population. The ADWF, AWWF and AAF from Table 5-3 are increased as a function of population.

Projected Wastewater Flows

Projecting peak flows is not as straightforward because peaks are largely determined by I/I. If peak flows are estimated purely as a function of population growth, the estimates will be overly conservative. With such an approach, the projections would assume the same infiltration rates for new sewers as for existing sewers. It is generally accepted that modern sewer construction techniques and materials are better at preventing groundwater infiltration relative to older sewers. To account for reduced I/I rates associated with new sewers, a reduced peaking factor will be used. A MDWWF: ADWF peaking factor of 4 to 1 will be used for new development. Furthermore, it is assumed that deterioration of existing sewers will be offset by ongoing rehabilitation and so peak flows from existing sewers will remain unchanged.

To estimate the full set of future flows, the projected AAF and MDWWF are plotted on a log probability plot. Other future flows are taken according to their respective exceedance probabilities. The log probability plot is presented in Appendix E. Table 5-8 presents the flow projections and compares peaking factors for new development and existing sewers.

Table 5-8: Summary of Projected Flows

Parameter	Year 2008 flow, MGD	New Development flow, MGD	Design Year (2035) flow (rounded), MGD	Existing Sewers Peaking Factors[(a)]	New Development Peaking Factors[(a)]
Service Area Population	1,730	1,034	2,764		
Base Sanitary Flow, MGD	0.15	0.09	0.24		
Dry Weather I/I	0.02	0.01	0.03		
ADWF	0.17	0.10	0.27	1.00	1.00
AAF	0.23	0.14	0.37	1.35	1.35
AWWF	0.28	0.17	0.45	1.65	1.65
MMDWF	0.30	0.18	0.48	1.76	1.76
MMWWF	0.45	0.24	0.7	2.65	2.40
MDDWF	0.56	0.33	0.9	3.29	3.29
MWWWF	0.68	0.31	1.0	4.00	3.10
MDWWF	1.25	0.41	1.7	7.35	4.00
PHF	1.60	0.58	2.2	9.41	5.80

Note

(a) Relative to ADWF

Projected Wastewater Characteristics

Load projections are based on unit design values and are proportional to the population served.

To determine design year (2035) loadings, the unit design values are multiplied by the projected population of 2,764. Projected loadings are presented in Table 5-9.

Table 5-9: Projected Loadings

Parameter	Year 2035 (rounded), ppd
Service Area Population	2,764
Annual average BOD	640
Maximum month BOD	1,200
Maximum week BOD	1,400
Maximum day BOD	1,800
Annual average TSS	640
Maximum month TSS	1,100
Maximum week TSS	1,500
Maximum day TSS	2,100

WASTEWATER CHARACTERISTICS SUMMARY

Table 5-10: Wastewater Characteristics Summary

Parameter	Year 2008	New Development	Design Total (rounded) (Year 2035)
Service Area Population	1,730	1,034	2,764
Flows			
Base Sanitary Flow, MGD	0.15	0.09	0.24
Dry Weather I/I, MGD	0.02	0.01	0.03
ADWF, MGD	0.17	0.10	0.27
AAF, MGD	0.23	0.14	0.37
AWWF, MGD	0.28	0.17	0.45
MMDWF, MGD	0.30	0.18	0.48
MMWWF, MGD	0.45	0.24	0.7
MWWWF, MGD	0.56	0.33	0.9
MDDWF, MGD	0.68	0.31	1.0
MDWWF, MGD	1.25	0.41	1.7
PHF, MGD	1.60	0.58	2.2
Loads			
Annual average BOD, ppd	400	240	640
Maximum month BOD, ppd	750	450	1,200
Maximum week BOD, ppd	900	500	1,400
Maximum day BOD, ppd	1,100	700	1,800
Annual average TSS, ppd	400	240	640
Maximum month TSS, ppd	700	400	1,100
Maximum week TSS, ppd	940	560	1,500
Maximum day TSS, ppd	1,300	800	2,100

CHAPTER 6

BASIS OF PLANNING

Evaluation criteria to be used for planning are established in this chapter. These criteria include current and anticipated future regulatory requirements, economic evaluation factors, and non-economic evaluation standards. The planning period used for the *Wastewater Facilities Plan* extends through year 2035. A 26-year planning horizon was selected to provide time for the City to acquire funding and complete design and construction of the needed improvements.

REGULATORY FRAMEWORK

The regulatory environment surrounding water quality protection in Oregon is relatively complex, requiring interaction and cooperation between a number of federal, state, and local agencies. The first step in the process is to assign beneficial uses to the water body receiving treated effluent. This task is the responsibility of the Oregon Water Resources Department (OWRD). A water body's beneficial uses depend on characteristics such as its size and location. The following are the designated beneficial uses for the South Umpqua River (Oregon Administrative Rules—OAR 340-041-0320, Table 320A):

- Public Domestic Water Supply
- Private Domestic Water Supply
- Industrial Water Supply
- Irrigation
- Livestock Watering
- Fish & Aquatic Life
- Wildlife and Hunting
- Fishing
- Boating
- Water Contact Recreation
- Aesthetic Quality
- Hydro Power

Designated fish uses for the South Umpqua River at Canyonville include (OAR 340-041-0320, Figures 320A and 320B): (1) Salmon and trout rearing and migration (2) Salmon and steelhead spawning from October 15 through May 15.

It is the responsibility of the Oregon Department of Environmental Quality (DEQ) to establish and enforce water quality and waste treatment standards that ensure the river's beneficial uses are preserved. The DEQ's general policy is one of antidegradation of surface water quality. Effluent discharges from wastewater treatment plants (WWTPs) are regulated through the National Pollutant Discharge Elimination System (NPDES). All discharges of treated effluent to a receiving stream must comply with the conditions of an NPDES permit. The Environmental Protection Agency (EPA) oversees state regulatory agencies and can intervene if the state agencies do not adequately protect water quality.

This section summarizes the regulatory requirements pertinent to wastewater facilities planning for the WWTP.

Oregon Administrative Rules for Wastewater Treatment

The state surface water quality and waste treatment standards for the South Umpqua River are detailed in the following sections of the Oregon Administrative Rules (OARs):

OAR 340-041-0004 lists policies and guidelines applicable to all basins. DEQ's policy of antidegradation of surface waters is set forth in this section.

OAR 340-041-0007 through 340-041-0036 describe the standards that are applicable to all basins.

OAR 340-041-0320 through 340-041-0326 contain requirements specific to the Umpqua basin including beneficial uses, approved Total Maximum Daily Loads (TMDLs) in the basin, water quality standards, and the minimum design criteria for waste treatment.

The surface water quality and waste treatment standards in the OARs are viewed as minimum requirements. Additional, more stringent limits developed though the TMDL process supersede the basin standards.

Current NPDES Permit Requirements

The WWTP's effluent is subject to the requirements of its NPDES permit (Appendix F). While the permit's expiration date is listed as 31 May 2009, the City has completed a renewal application, so the permit remains in effect (OAR 340-045-0040). Current treatment requirements are summarized in Table 6-1.

Table 6-1: Current NPDES Permit Requirements

	Average effluent concentrations, mg/L				
Parameter	**Monthly**	**Weekly**	**Monthly average, lb/day**	**Weekly average, lb/day**	**Daily maximum, lb**
May 1 through October 31					
$CBOD_5$	10	15	42	63	84
TSS	10	15	42	63	84
Total chlorine residual	0.04	0.1			
Ammonia	5.4	11			
November 1 through April 30					
$CBOD_5$	25	40	100	160	210
TSS	30	45	130	190	250
Total chlorine residual	0.18	0.48			

Other parameters	
E.coli bacteria	Shall not exceed 126 organisms per 100 mL monthly geometric mean. No single sample shall exceed 406 organisms per 100 mL.
pH	Shall be within the range of 6.0 – 8.5
BOD_5 and TSS removal efficiency	Shall not be less than 85 percent monthly average for $CBOD_5$ and 85 percent monthly average for TSS.
Excess Thermal Load (May 15 through October 15)	Shall not exceed a weekly average of 6.8 million Kcals per day.

Clean Water Act Section 303(d) List

The 2004/2006 Integrated Report water quality assessment list categorizes constituents into one of five base classifications, as follows:

- Category 1: All standards are met. (This category is not used in the assessment).
- Category 2: Attaining - Some of the pollutant standards are met.
- Category 3: Insufficient data to determine whether a standard is met.
  - 3B: Potential concern - Some data indicate non-attainment of a criterion, but data are insufficient to assign another category.
- Category 4: Water is water quality limited but a TMDL is not needed. This includes:
  - 4A: TMDL approved - TMDLs needed to attain applicable water quality standards have been approved.
  - 4B: Other pollution control requirements are expected to address all pollutants and will attain water quality standards.
  - 4C: Impairment is not caused by a pollutant (e.g., flow or lack of flow is not considered a pollutant).
- Category 5: Water body is water quality limited and a TMDL is needed.

A summary of the 2004/06 303(d) list for the South Umpqua River at Canyonville is shown in Table 6-2.

Table 6-2: 2004/06 303(d) List for South Umpqua River at Canyonville

Parameter	Season	Criteria[(a)]	Beneficial Uses	Year	Assessment Action
Category 5 – TMDL Required					
Temperature	Year Round	18 degrees C 7-day Average	Salmon & trout rearing and migration	2004	Added
Dissolved Oxygen	Year round	8.0 mg/l or 90 percent saturation	Aquatic life	2004	Added
Chlorophyll-a	Summer	0.015 mg/L	Aesthetics, fishing, livestock watering, water contact recreation, water supply	2004	No status change

Parameter	Season	Criteria[a]	Beneficial Uses	Year	Assessment Action
pH	Summer	6.5 – 8.5	Aquatic life, salmonid rearing and spawning, Water Contact Recreation	2004	Added
E. Coli	Summer	126 org/100 ml	Water Contact Recreation	2004	Added
Category 3 – Insufficient Data					
Biological criteria	Undefined	Waters must support aquatic species	Aquatic Life	1998	Added
Alkalinity	Year Round	Toxic Substances (Table 20)	Aquatic Life	2004	Added
Zinc	Year Round	Toxic Substances (Table 20)	Human Health & Aquatic Life	2004	Added
Sedimentation	Undefined	Turbidity	Aquatic Life, salmonid rearing and spawning	1998	Added
Antimony	Year Round	Toxic Substances (Table 20)	Human Health	2004	Added
Arsenic	Year Round	Toxic Substances (Table 20)	Human Health & Aquatic Life	2004	Added
Barium	Year Round	Toxic Substances (Table 20)	Human Health	2004	Added
Beryllium	Year Round	Toxic Substances (Table 20)	Human Health	2004	Added
Cadmium	Year Round	Toxic Substances (Table 20)	Human Health & Aquatic Life	2004	Added
Chloride	Year Round	Toxic Substances (Table 20)	Aquatic Life	2004	Added
Aquatic weeds	Summer	Deleterious growths	Aesthetics, fish and aquatic life, water contact recreation	1998	Added
Lead	Year Round	Toxic Substances (Table 20)	Human Health & Aquatic Life	2004	Added
Thallium	Year Round	Toxic Substances (Table 20)	Human Health & Aquatic Life	2004	Added
Chromium	Year Round	Toxic Substances (Table 20)	Human Health & Aquatic Life	2004	Added
Nickel	Year Round	Toxic Substances (Table 20)	Human Health & Aquatic Life	2004	Added
Manganese	Year Round	Toxic Substances (Table 20)	Human Health & Aquatic Life	2004	Added
Copper	Year Round	Toxic Substances (Table 20)	Human Health & Aquatic Life	2004	Added
Iron	Year Round	Toxic Substances (Table 20)	Human Health & Aquatic Life	2004	Added

Parameter	Season	Criteria[(a)]	Beneficial Uses	Year	Assessment Action
Selenium	Year Round	Toxic Substances (Table 20)	Human Health & Aquatic Life	2004	Added
Chlorophyll-a	Fall/Winter/ Spring	0.015 mg/L	Aesthetics, fishing, livestock watering, water contact recreation, water supply	2004	Added
Silver	Year Round	Toxic Substances (Table 20)	Human Health & Aquatic Life	2004	Added
Category 2 – Attaining Some Criteria					
Ammonia	Year Round	Toxic Substances (Table 20)	Aquatic Life, domestic water supply	2004	Attaining criteria/uses
pH	Year Round	6.5 – 8.5	Aquatic Life & Water Contact Recreation	2004	No Status Change
Dissolved Oxygen	October 15 – May 15	Not less than 11 mg/L or 95 percent saturation	Salmon and steelhead spawning	2004	Added
Phosphate/ phosphorus	Summer	50 ug/L	Aquatic life	2004	Added
E. Coli	Fall/Winter/ Spring	126 Org./100 ml	Water Contact Recreation	2004	Added
pH	Fall/Winter/ Spring	6.5 to 8.5	Fish passage, salmonid spawning and rearing, Water Contact Recreation	2004	No status change

Note:

(a) "Table 20" refers to Table 20 in OAR 340-41.

Umpqua Basin Total Maximum Daily Load

The Umpqua Basin TMDL was approved by EPA in April, 2007. For the South Umpqua River, the TMDL addresses:

- Bacteria
- Temperature
- Algae
- Dissolved oxygen
- pH
- Total and inorganic phosphorus
- Biological criteria

The TMDL prescribes Waste Load Allocations (WLAs) for the Canyonville WWTP for total phosphorus, inorganic phosphorus, temperature, and bacteria. These parameters are discussed in the following subsections.

TMDL Requirements for Phosphorus. The TMDL lists specific WLA requirements for phosphorus and targets in-stream phosphorus concentrations of less than 0.10 milligrams of phosphorus per liter (mgP/L) to protect aquatic life. Total and inorganic phosphorus WLAs for the WWTP are river flow dependent and the DEQ has identified river flow regimes and associated phosphorus WLAs for the South Umpqua River. The total phosphorus WLAs for the Canyonville WWTP were established in a 29 July 2009 draft DEQ memorandum (Appendix G) and are summarized in Table 6-3.

Table 6-3: Flow Regimes for Total and Inorganic Phosphorus WLAs

Parameter	Monthly Median River Flow, cfs[a, b]	Monthly Median Discharge, ppd[c, d]
Total phosphorus (May 1 – October 31)	<56	1.2
	56 to 867	River flow (in cfs) x 0.0147
	>867	20

Notes:

(a) cfs = cubic feet per second
(b) River flow data taken from the Tiller Gauging station (USGS 14308000).
(c) ppd = pounds per day
(d) Monthly median is the median for a month calculated from the observed mean daily values. In the case of phosphorus loading, the monthly median is calculated from the daily load, which is calculated from the daily average concentration and daily average effluent flow.

TMDL Requirements for Temperature. The Umpqua Basin TMDL establishes thermal wasteload allocations (WLAs) for point source discharges in the Umpqua River Basin to address the temperature listing during the salmon and trout rearing and migration season criterion (18°C). The thermal waste load allocations are calculated to ensure that the discharge will not increase stream temperatures beyond the applicable criterion more than 0.1°C (cumulatively) at the steam's point of maximum impact. The TMDL calculated waste load allocations during the critical case scenario using the following equation:

Thermal WLA = HUA * (Qps + Qr) * c
Where, HUA = Human Use Allowance (0.1°C)
Qps = Point Source Effluent Flow
Qr = upstream river flow
c = conversion factor (4.1868 MW/°C-cms)

In the critical case scenario for Canyonville, the TMDL uses a Qps of 0.022 cms (0.50 mgd) and a Qr of 2 cms (45.6 mgd). This results in a wasteload allocation of 0.847 MW. The TMDL also calculates an effluent temperature associated with this critical case wasteload allocation, which is 36.8 °C.

Five years of temperature and flow data for the South Umpqua River and the WWTP were analyzed for this *Facilities Plan* using the DEQ's WLA methodology. The results of the analysis confirmed that the WWTP will not exceed its temperature WLA to the South Umpqua River. Even when combining "worst case" conditions that do not happen concurrently (low river flow and high effluent flow), the analysis indicated that the resulting effluent temperature associated with the thermal WLA would be 28.5 °C. This compares to the highest recorded effluent temperature between July 2004 and April 2009 of only 25.4 °C.

The NPDES permit renewal process will use the WLA equations presented in the TMDL to calculate thermal effluent limits. These limits may be based on critical case scenarios and/or a variety of flows and river temperatures.

The TMDL does not address the spawning time period for the South Umpqua River near Canyonville (October 15 through May 15). The biologically-based numeric criterion for the spawning period is 13 °C. The limited information gathered on the stream temperature in the Umpqua River in the vicinity of Canyonville indicates that the stream is attaining the temperature criteria of 13°C. Therefore, OAR 340-041-0028(11)(b) would apply:

> "A point source that discharges into or above salmon & steelhead spawning waters that are colder than the spawning criterion, may not cause the water temperature in the spawning reach where the physical habitat for spawning exists during the time spawning through emergence use occurs, to increase more than the following amounts after complete mixing of the effluent with the river:
>
> A. If the rolling 60-day average maximum ambient water temperature, between the dates of spawning use as designated under subsection (4) (a) of this rule, is 10 to 12.8 °Celsius, the allowable increase is 0.5 °C above the 60 day average; or
>
> B. If the rolling 60 day average maximum ambient water temperature, between the dates of spawning use as designated under subsection (4) (a) of the rule, is less than 10 degrees Celsius, the allowable increase is 1.0 °C above the 60 day average, unless the source provides analysis showing that a greater increase will not significantly impact the survival of salmon or steelhead eggs or the timing of salmon or steelhead fry emergence from the gravels in downstream spawning reaches."

There is inadequate information available to determine the rolling 60 day average maximum ambient water temperature. However, a "worst case" scenario can be developed using conservative assumptions. The lowest 60 day average stream flow with a 10 year recurrence for the USGS gage in the South Umpqua River at Tiller for the period October 15 through May 15 is 301 cubic feet per second (cfs). The average effluent temperature during this period from the data available is 18.4°C and the projected maximum month dry weather flow is 0.48 MGD (0.74 cfs). Assuming that the stream temperature is approximately 10° C during this time, the resulting increase in stream temperature would be as follows:

$$\Delta T = Qps(Tps-Tr)/(Qr+Qps) = 0.75(18.4-10)/(301+0.75) = 0.02° C$$

Therefore, while additional data is needed for a definitive answer, it is very unlikely that the effluent would cause more that a 0.5 °C increase above the 60 day average during the spawning season.

Thermal Plume Limitations. Oregon Administrative Rules 340-041-0053(2)(d), requires evaluation of the following potential adverse impacts:

A. Impairment of an active salmonid spawning area where spawning redds are located or likely to be located.

B. Acute impairment or instantaneous lethality.

C. Thermal shock cause by a sudden increase in water temperature.

D. Migration blockage.

According to Pete Baki, Habitat Conservation Biologist at Oregon Department of Fish and Wildlife, Roseburg Office, the area in the immediate vicinity of the outfall is not an active spawning area and redds are not likely to be located in that area. However, there are active redds about 100 feet downstream and 150-200 upstream of the current outfall location. The existing mixing zone description is 15 feet upstream and 165 feet downstream. Therefore, Canyonville should locate the new outfall a minimum of 165 feet upstream of the redd area, which is about 65 feet upstream of the current location.

The WWTP's maximum recorded effluent temperature is 25.4° C, which is significantly below the incipient lethality temperature of 32.0° C. Consequently, the effluent will not cause a thermal shock during the critical low flow conditions because the natural thermal potential is higher than the effluent temperature. Therefore, while DEQ will evaluate the mixing zone through the permitting process, this analysis indicates that Canyonville's effluent will not cause unacceptable thermal conditions.

TMDL Requirements for Bacteria. The South Umpqua River is designated water quality limited for bacteria. The Umpqua Basin TMDL established overall bacteria load allocations and set point source load allocations at the in-stream standard as follows:

- Log mean E.coli count of 126 organisms per 100 milliliters (mL).
- Not-to-exceed E.coli count of 406 organisms per 100 mL.

These allocations are consistent with the Umpqua Basin standards.

Ammonia Toxicity

Ammonia is a substance normally found in wastewater. Biological treatment processes can convert a large portion to nitrate (NO_3^-) and nitrite (NO_2^-), but the treated effluent will still contain some ammonia. The draft *1998 Wastewater Facilities* acknowledged that the existing treatment plant was not designed to fully nitrify, and identified inadequate effluent dilution in the South Umpqua River due to the lack of outfall diffuser as a potential contributor to dry weather ammonia toxicity. The 2004 NPDES permit renewal also recognized the potential for ammonia toxicity, the inadequate dilution caused by the broken outfall, and that the treated effluent runs down the bank and into the South Umpqua River. The draft *1998 Facilities Plan* proposed installation of a new multiport diffuser. In 2005, the City submitted a letter to DEQ reaffirming their intention to construct a new diffuser, pending the outcome of the TMDL.

The ammonia toxicity analysis presented in this section should be considered preliminary, as DEQ will perform a reasonable potential analysis (RPA) and establish water quality based effluent limits as appropriate during the next permit renewal.

Ammonia toxicity is based on pH, alkalinity, and temperature. Water quality data was downloaded from DEQ's Laboratory Analytical Storage and Retrieval (LASAR) database for the South Umpqua River in the vicinity of Canyonville. Consistent with the DEQ's December 2007

Regulatory Mixing Zone Internal Management Directive (IMD), Table 6-4 provides the upper 90th percentile pH, upper 90th percentile temperature, and 50th percentile alkalinity for the period of concern (May through October).

Table 6-4: South Umpqua River Water Quality

Month	pH	Temperature (deg. C)	Alkalinity (mg CaCO3/L)	Acute Criterion (mg/L)	Chronic Criterion (mg/L)
May	8.2	20.0	30	4.6	0.5
Jun	8.5	23.7	38	2.2	0.4
Jul	8.8	28.0	43	1.6	0.3
Aug	9.1	25.0	47	2.0	0.4
Sep	8.9	22.4	50	2.7	0.5
Oct	8.2	18.0	48	5.3	0.6
Nov	8.1	10.6	40	6.0	0.8

DEQ requires that the water outside the boundary of the mixing zone be free of materials in concentrations that will cause chronic (sublethal) toxicity and that the water outside the zone of immediate dilution (ZID) be free of pollutants that will cause acute toxicity. The Mixing Zone IMD requires DEQ to use critical case flows in determining dilution. The 1Q10 flow ((lowest 1-day flow with a 10-year recurrence) is the critical case for acute toxicity, while the 7Q10 flow (lowest 7-day-average flow with a 10-year recurrence) is the critical case for chronic toxicity. South Umpqua River flows, applicable to toxicity analysis are shown in Table 6-5.

Table 6-5: South Umpqua River Flow

Month	7Q10	1Q10	Harmonic Mean	30Q5
May	294	256	170	621
Jun	125	113	170	262
Jul	62.1	57.2	169	103
Aug	39.6	37.7	170	54.5
Sep	36.2	32.6	170	49.4
Oct	37.8	36.8	169	62.7
Nov	58	46.1	170	283
May - Oct	34	32.1	169	43.5

In determining the conditions at the boundary of the mixing zone and ZID, the flow, pH, temperature, and alkalinity of the effluent must also be considered. The City does not currently analyze for alkalinity. For the purposes of this evaluation, the following assumptions are made, which may be conservative:

- Effluent alkalinity = 80 mg CaCO3/L
- Effluent pH = 7.5
- Effluent flow = 0.7 mgd

Also, because the outfall has not been constructed, the dilution available is currently unknown. However, assuming that 25 percent of the river flow is available for dilution, Table 6-6 provides the resulting effluent ammonia limits using these assumptions and typical effluent temperatures. Calculations are provided in Appendix H.

Table 6-6: South Umpqua River Water Quality

Month	Effluent Temperature (deg. C)	Monthly NH3 (mg/L)	Daily NH3 (mg/L)
May	20.0	42.8	85.9
Jun	23.7	14.1	28.3
Jul	28.0	5.3	10.7
Aug	25.0	4.2	8.4
Sep	22.4	4.5	9.1
Oct	18.0	6.5	13.0
Nov	18.0	12.6	25.2

Figure 6-1 provides a summary (maximum, minimum, and average) of current effluent ammonia concentrations.

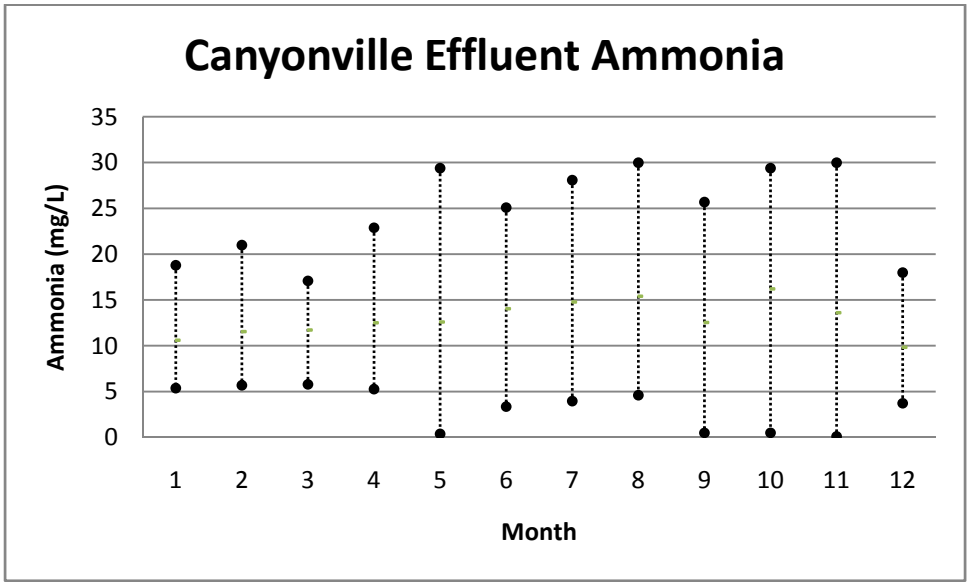


Figure 6-1: Current Effluent Ammonia Concentrations

Since nitrified activated sludge can create a cloudy secondary effluent, and increase effluent total suspended solids concentrations, operations staff generally try to avoid nitrification. However, as can be seen from this chart, nitrification does occur on occasion during most months.

From the above information, the following preliminary conclusions can be reached:

- Ammonia toxicity is not an issue in May because of relatively high river flows
- It would be advantageous to the City to request a permit modification for ammonia limits based on monthly river conditions.
- Ammonia toxicity may not be an issue in June due to relatively high river flows
- Ammonia toxicity may be an issue during November because river flows can still be relatively low—a 12 mg/L effluent ammonia limit appears to be appropriate.

In recent discussions, DEQ representatives have indicated that the new NPDES permit will not include wet weather ammonia limits. Therefore, while the preceding analysis suggests that ammonia toxicity could be an issue in November, planning will be based on dry weather ammonia limits only. In addition, DEQ representatives are open to river flow based ammonia limits, so ammonia removal would not be necessary during the relatively high flow month of May.

Future Regulatory Issues

While accurately predicting specific future regulatory requirements is difficult at best, recent legislation, directives, environmental group focus, and litigation can all be analyzed and used to develop reasonable estimates of future regulatory trends. Future regulatory requirements can be grouped into two categories: (1) those for which regulations have been established but not yet applied to the City's NPDES permit, and (2) those for which regulations have not yet been developed. These future regulatory requirements are summarized in Tables 6-7 and 6-8, respectively.

Table 6-7: Known Future Regulatory Issues

Item	Description	Issues
NPDES permit	Permit has expired and renewal application has been submitted.	Compliance with effluent chlorine limits will require dechlorination. Compliance will necessitate construction of new outfall diffuser.
Mutual agreement and order	Establishes compliance schedule.	Compliance schedule lists deadlines for facilities plan completion and construction of WWTP improvements. Failure to meet schedule results in fines/penalties.
South Umpqua River TMDL	TMDL addresses bacteria, temperature, algae, dissolved oxygen, pH, total and inorganic phosphorus, and biological criteria.	TMDL includes WLAs for total phosphorus, inorganic phosphorus, and temperature. Permit limits for these constituents will likely result.
Umpqua Basin standards (OAR 340-041-0326)	Base standards for wastewater treatment and receiving water quality.	Basic policy of anti-degradation of receiving water quality.
Toxicity Reasonable Potential Analysis (RPA) Internal Management Directive (IMD)	Numeric criteria for over 100 toxic substances.	RPA required. Action required if reasonable potential for toxicity.

Item	Description	Issues
Mixing zone IMD	New regulations for evaluating mixing zones.	Increases complexity of mixing zone evaluations. Requires assessment of numerous toxic substances. Mixing zone will likely decrease in size, reducing available dilution
EPA/DEQ reliability/redundancy requirements	Sets forth requirements for number and capacity of treatment units.	Potential cost impacts associated with construction of redundant treatment units.

Table 6-8: Potential Future Regulatory Issues

Item	Description	Issues
New Ammonia Criteria	In 2009, EPA published draft revisions to the ammonia criteria based on the presence of freshwater mussels.	These are draft rules and it is not certain if they will be finalized. The draft criteria rules are similar to current criteria if freshwater mussels are not present. If freshwater mussels are present, the acute criteria is about half and the chronic about ¼ of the current criteria.
303 (d) listings and potential future TMDLs	See Table 6-2.	Constituents on 303 (d) list could trigger future TMDLs and associated additional treatment requirements.
Mixing zones	Periodic legislative attempts to eliminate mixing zones.	Elimination of mixing zones would require compliance with all water quality criteria at end of pipe.
Contaminants of emerging concern (CEC)	Personal care products and hormones creating water quality concerns.	CECs receiving increased publicity and triggering more regulatory and public concern.
Fish consumption	Certain toxins bioaccumulate in fish tissue.	EPA/DEQ may increase the assumed quantity of fish consumed by a factor of 10, reducing effluent limits for bioaccumulative compounds accordingly. Could require higher level of treatment.
Wet weather policy	DEQ currently allows sanitary sewer overflows (SSOs) during storm events greater than 5-year, 24-hour. All SSOs may be prohibited.	Design peak flows could increase.
Elimination of compliance schedules	Inclusion of compliance schedules in permits has been challenged.	Issue has delayed permit renewals. Unclear how time for making improvements would be granted.

Anticipated Future Treatment Requirements

Future treatment requirements for CBOD and TSS are not expected to differ significantly from the current permit requirements. Limits for other parameters are likely to become more restrictive as they are made consistent with the *Umpqua Basin TMDL* and recent regulatory initiatives. Potential permit changes conditions include:

- Phosphorus: Dry weather season effluent phosphorus limits consistent with the WLA will likely be included in the next NPDES permit. Limits will probably be linked to river flow rate.
- Thermal Load: The thermal load limit in the existing permit will be updated to reflect the WLA.
- Mixing zone: The dimensions of the regulatory mixing zone (RMZ) and available mixing will likely decrease compared to the current permit.
- Ammonia: Ammonia limits are anticipated to become more restrictive to reflect the toxicity and mixing zone IMDs, and proposed EPA toxicity criteria.

Anticipated future treatment requirements are summarized in Table 6-9.

Table 6-9: Anticipated Future NPDES Permit Requirements

	Average effluent concentrations, mg/L				
Parameter	**Monthly**	**Weekly**	**Monthly average, lb/day**	**Weekly average, lb/day**	**Daily maximum, lb**
May 1 through October 31					
$CBOD_5$	10	15	42	63	84
TSS	10	15	42	63	84
Total chlorine residual	0.04	0.1			
Phosphorus	--	--	In accordance with Table 6-3	--	
Ammonia[(a)]	4	8			
November 1 through April 30					
$CBOD_5$	25	40	100	160	210
TSS	30	45	130	190	250
Total chlorine residual	0.18	0.48			
Other parameters					
E.coli bacteria	Shall not exceed 126 organisms per 100 mL monthly geometric mean. No single sample shall exceed 406 organisms per 100 mL.				
pH	Shall be within the range of 6 - 9				
BOD_5 and TSS removal efficiency	Shall not be less than 85 percent monthly average for $CBOD_5$ and 85 percent monthly average for TSS.				
Excess Thermal Load (May 15 through October 15)	In accordance with thermal WLA listed in Umpqua Basin TMDL.				

Notes:

(a) Estimated. River flow based limits anticipated to apply June through October. See Table 6-6.

As discussed previously, the current Clean Water Act 303 (d) list includes a number of constituents of concern that could ultimately trigger associated limits. However, in order to establish limits, a TMDL would have to be developed and WLAs assigned. It is unlikely that this process would be completed within the next one or two 5-year permit cycles. Because the content of future TMDLs cannot be accurately predicted, these potential issues should not drive the initial decision-making process.

BIOSOLIDS

Biosolids are regulated by both the U.S. Environmental Protection Agency (EPA) and DEQ to ensure quality standards are met. Regulations address trace metal concentrations, pathogen content, odor potential and basic operational practices. Beneficial reuse of biosolids has long been preferred over historical disposal practices such as incineration or landfilling. Land application practices and marketable biosolids products are encouraged as an alternative to disposal by state and federal regulatory authorities.

Biosolids are generally categorized into one of two classifications: Class A and Class B. Class A biosolids contain very low levels of pathogens. To achieve Class A criteria biosolids must undergo heating, composting, digestion or lime addition that reduces pathogens to very low levels. Once this is achieved, Class A biosolids can be land applied without any pathogen-related restrictions at a site. If Class A biosolids meet the low metals concentration standards under EPA's Part 503 regulations they are considered to be Exceptional Quality (EQ). Class A EQ biosolids can be bagged and marketed to the public for application to lawns, gardens and multiple other landscape uses. A description of the testing requirements and metals limits for biosolids is included as Appendix I.

Class B biosolids have less stringent standards for treatment and contain small amounts of pathogens. Class B requirements ensure that pathogens in biosolids have been reduced to levels that protect public health and the environment and include certain restrictions for crop harvesting, grazing animals and public contact. As is true of their Class A counterpart, Class B biosolids must undergo heating, digestion or increased pH processes before leaving the wastewater treatment plant. Both Class A and B biosolids must conform to vector attraction reduction requirements. Unlike Class A, Class B biosolids cannot be bagged or marketed to the public for application to gardens, landscapes or other public uses. Rather, Class B biosolids are typically land applied to agricultural sites under permits and with restricted public access. When managed in this way, biosolids land application provides beneficial reuse or recycling of nutrients and organic matter.

Class A Biosolids

For Class A biosolids, microbial monitoring for fecal coliform or *Salmonella* sp. is conducted to verify pathogen reduction. Because Class A biosolids may be used without site restrictions, all Class A material must be tested to show that the microbial requirements are met at the time when it is ready to be used, sold or given away. In addition to meeting process requirements, Class A biosolids must meet one of the following indicator organism requirements:

- The fecal coliform bacteria count must be less than 1,000 MPN per gram total solids (dry gram weight);
- The *Salmonella* sp. bacteria count must be less than 3 MPN per 4 grams of total solids (dry weight basis).

Pathogen reduction requirements for Class A biosolids make them suitable for public handling and use. The Federal biosolids regulations (40 CFR Part 503) list six alternatives for Class A pathogen reduction:

1. Thermal treatment
2. High pH and high temperature
3. Treatment in a process that reduces viruses
4. Treatment in a process that reduces pathogenic bacteria
5. Treatment in a Process to Further Reduce Pathogens (PFRP):
    a. Composting
    b. Heat drying
    c. Heat treatment
    d. Thermophilic aerobic digestion
    e. Beta ray irradiation
    f. Gamma ray irradiation
    g. Pasteurization
6. Treatment in a process equivalent to a PFRP

Class B Biosolids

Compared to Class A biosolids, Class B biosolids have less stringent treatment requirements but more use restrictions. Class B pathogen reduction alternatives prescribed in Part 503.32 are as follows:

1. Monitoring of indicator organisms and maintaining prescribed limits
2. Treatment in a Process to Significantly Reduce Pathogens (PSRP):
    a. Aerobic digestion
    b. Air drying
    c. Anaerobic digestion
    d. Composting
    e. Lime stabilization
3. Treatment in a process equivalent to a PSRP

Use restrictions for Class B biosolids include:

- Food crops with harvested parts that touch biosolids shall not be harvested for 14 months after application of biosolids.
- Food crops with harvested parts below the ground surface shall not be harvested for 38 months after application of biosolids.
- Food crops with harvested parts that do not contact biosolids shall not be harvested for 30 days after application of biosolids.
- Animals shall not be grazed on land for 30 days after application of biosolids.
- Turf that is to be used in areas with potential public exposure shall not be harvested for 1 year after application of biosolids.
- Public access to land with high potential for public exposure shall be restricted for 1 year after application of biosolids.
- Public access to land with low potential for public exposure shall be restricted for 30 days after application of biosolids.

Vector Attraction Reduction

Vector attraction reduction requirements apply to both Class A and Class B biosolids. The vector attraction reduction requirement alternatives prescribed in 40 CFR Part 503.33 include:

1. Provide 38 percent reduction in volatile solids content.
2. Demonstrate vector attraction reduction with additional bench-scale anaerobic digestion.
3. Demonstrate vector attraction reduction with additional bench-scale aerobic digestion.
4. Meet specific oxygen uptake criteria with aerobically digested biosolids.
5. Provide aerobic treatment at greater than 40 degrees C for 14 days or longer.
6. Provide alkali addition under specified conditions.
7. Dry biosolids with no unstabilized biosolids 75 percent solids concentration.
8. Dry biosolids with unstabilized biosolids 90 percent solids concentration.
9. Inject biosolids beneath the soil surface.
10. Incorporate biosolids into the soil within 6 hours of land application.
11. Cover land applied biosolids with soil.

The City currently produces a Class B biosolids product. To meet pathogen reduction and vector attraction reduction requirements the City implements a lime stabilization system. Pathogen reduction requirements in accordance with 40 CFR Part 503.32 (3)(b) Processes to Significantly Reduce Pathogens, Alternative 5 is implemented by the City. This alternative requires that sufficient lime is added to the sewage sludge to raise the pH to 12 after two hours of contact. Vector Attraction Reduction requirements in accordance with 40 CFR Part 503.33 (b)(6) is implemented by the City. This alternative requires that the pH of sewage sludge be raised to 12 or higher by alkali addition and, without the addition of more alkali, shall remain at 12 or higher for two hours and then at 11.5 or higher for an additional 22 hours. The current biosolids treatment system is described in Chapter 4.

The City generates biosolids that meet the “low” metals standards in accordance with Table 3 of 40 CFR Part 503.13 for Pollutant Concentrations. However, if pollutant concentrations ever exceeded the ceiling concentration limits or “high metals” standards set forth in Table 1 of Part 503.13 the City would not be able to beneficially reuse biosolids until the pollutant concentrations fell below these limits. Additional information on the existing sludge management system is included in Chapter 4.

The City’s frequency of monitoring for the pollutants listed in 40 CFR Part 503.13; the pathogen density requirements in 503.32; and the vector attraction reduction requirements in 503.33 are set forth in Table 1 of 503.16. Based on current biosolids loading, the City monitors their biosolids using sampling and analytical methods established under 40 CFR 503.8 once per year, prior to seasonal land application. The City maintains a log indicating the quantity, quality, and location of biosolids applied to local agricultural sites. A copy of the City’s 2009 biosolids report is included as Appendix J.

RECYCLED WATER

Oregon adopted revised recycled water standards in 2008 (Oregon Administrative Rules—OAR—340-55). Generally speaking, the revised rulemaking increased and clarified allowed uses, and streamlined recycled water program development. In addition, quality criteria for five classes of recycled water were established. For the WWTP, three classes of recycled water may come under consideration:

- Class D recycled water is essentially disinfected secondary effluent. Class D recycled water must not exceed a 30-day log mean of 126 E. coli organisms per 100 milliliters and 406 E. coli organisms per 100 milliliters in any single sample. The City of Canyonville's WWTP currently produces effluent suitable for Class D recycled water use.
- Class C recycled water is secondary effluent disinfected to a median of 23 total coliform organisms per 100 mL and less than 240 total coliform organisms per 100 mL in any two consecutive samples.
- Class A recycled water is filtered secondary effluent that conforms to the bacteria and turbidity limits listed in Table 6-10.

Table 6-10: Class A Recycled Water Quality Criteria

Parameter	Criteria
Turbidity, NTU	
24-hr average	< 2
< 5 percent of time	5-10
Maximum	10
Total coliform, #/100 mL	
7 day median	2.2
Maximum single sample	23

RELIABILITY/REDUNDANCY CRITERIA

Wastewater treatment reliability and redundancy criteria are established by the EPA and are described in the EPA's Technical Bulletin "Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability" (EPA 430-99-74-001). These criteria require WWTPs to maintain a minimum level of treatment if there is a failure of a process component. For the purposes of reliability and redundancy, the WWTP is considered a Class I Facility during the dry weather season and a Class II Facility during the wet weather season.

The EPA criteria for reliability/redundancy applicable to the WWTP are summarized in Table 6-11. DEQ has developed reliability criteria which supplement the EPA criteria. These requirements are listed in Table 6-12.

Table 6-11: EPA Process Reliability/Redundancy Criteria

Process	EPA Requirements[a]
PRELIMINARY TREATMENT	
Raw Sewage Pump Station	Multiple units and backup pumps must be included in the design so that peak flows can be handled if any pump is out of service.
Screening System	At least two channels must be provided, each equipped with a bar screen. Provisions should be made to isolate flow from any screening unit and to dewater each unit. Works with only two bar screens must have one bar screen designed to permit manual cleaning.
Grit Removal System	Where a single grit removal unit is utilized, a bypass must be provided.
Grit Pumps	A backup pump shall be provided for each set of pumps which performs the same function.
Grit Cyclones & Classifiers	No requirement listed.
Unit Operations Bypassing	The bypassing system shall be designed to provide control of the diverted flow such that only that portion of the flow in excess of the hydraulic capacity of the units in service need by bypassed.
SECONDARY TREATMENT	
Aeration basins	At least two (2) equal volume basins must be provided.
Aeration blowers	There shall be a sufficient number of blowers to enable the design oxygen transfer to be maintained with the largest capacity unit out of service. At least two units shall be installed.
Air diffusers	The air diffusion system shall be designed such that the largest section of diffusers can be isolated without measurably impairing oxygen transfer capability
Secondary Clarifiers	There must be at least two units designed so that, with the largest capacity unit out of service, the remaining unit(s) can handle at least 75 percent of the design flow during dry weather (Class I) and 50 percent of the design flow during wet weather (Class II).
RAS Pumps	A backup pump shall be provided for each set of pumps which performs the same function.
WAS Pumps	A backup pump shall be provided for each set of pumps which performs the same function.
TERTIARY TREATMENT	
Filters	Filters shall have provisions for bypassing if the peak wastewater flow cannot be handled hydraulically with the largest unit out of service.

Process	EPA Requirements[(a)]
DISINFECTION	
	A sufficient number of units shall be provided and sized, such that the capacity with the largest unit out of service is at least 50 percent of the total design flow to that unit operation.
SOLIDS TREATMENT	
Lime stabilization tanks	No requirement listed
Biosolids Storage	No requirement listed.
STANDBY POWER	
Power source	Two separate and independent power sources each with sufficient capacity to operate all vital treatment components, lighting, and ventilation during peak flow conditions shall be provided.

Notes:

(a) "Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability" EPA Technical Bulletin No. 430-99-74-001.

Table 6-12: DEQ Process Reliability/Redundancy Criteria

Process	DEQ Requirements
Treatment plant overall	There shall be no overflows at design peak hour flow.
Raw Sewage Pump Station	The station must be capable of conveying the design peak hour flow with the largest pump out of service.
Screening System	The screening system must be capable of treating the design peak hour flow. Provisions for bypassing in the event of equipment failure must be provided.
Grit Removal System	The grit system must be capable of treating the design maximum day wet weather flow. Provisions for bypassing in the event of equipment failure must be provided.
SECONDARY TREATMENT	
Aeration basins	The aeration basins must provide full treatment capacity to design maximum month wet weather flow.
Secondary Clarifiers	The secondary clarifiers must treat the design maximum month wet weather flow with all units in service at a surface overflow rate of 1200 gallons per day per square foot (gpd/sf), and must treat the design maximum month dry weather flow with one unit out of service at a surface overflow rate of 800 gallons per day per square foot (gpd/sf).

Process	DEQ Requirements
TERTIARY TREATMENT	
Filters	Filters shall have capacity to treat the maximum day dry weather flow with all cells in service and the average dry weather flow with one cell out of service.
All facilities necessary for the production of recycled water	Recycled water facilities must be capable of treating design flows with the largest treatment unit out of service.
Standby power for recycled water facilities	Adequate capacity to operate all recycled water facilities during a power outage

FACTORS FOR CONDUCTING ECONOMIC EVALUATIONS

Capital Cost Estimates

All cost estimates are "order-of-magnitude" estimates as defined by the American Association of Cost Engineers International (AACEI). An order of magnitude estimate is one that is made without detailed engineering data and uses techniques such as cost curves and scaling factors applied to estimates developed for similar projects. The overall expected level of accuracy of the cost estimates presented is -30 percent to +50 percent. This means that bids can be expected to fall within a range of 30 percent under to 50 percent over the estimate for each project. This is consistent with the guidelines established by the AACEI for planning level studies.

The capital cost estimates were prepared using the 20-Cities Engineering News Record (ENR) Construction Cost Index average of 3.8 percent, which is anticipated to be reached by the time the initial project is bid. The estimates reflect a professional opinion of costs at this time and are subject to change as the design of each project component develops. The markups that were applied are summarized in Table 6-13.

Table 6-13: Mark Up Factors Used in Developing Preliminary Construction Cost Estimates

Item	Markup
Contingencies	30 percent
Contractor General Conditions	15 percent
Contractor Overhead and Profit	15 percent
Engineering, Legal and Administration	25 percent

The preliminary construction cost estimates do not include the following:

- Potential cost increases due to unknown historical or cultural impacts to construction
- Potential costs associated with identification and mitigation of hazardous waste
- Easement or land acquisition
- Contingencies
- Engineering, legal and administrative (ELA) costs

Total project capital costs for planning alternatives will be calculated by multiplying the sum of the estimated construction costs (with general conditions and overhead and profit) by factors to account for contingencies, and engineering, legal and administrative costs.

The engineering, legal and administrative cost factors will be applied to the construction cost following the application of the contingencies factor.

Operation and Maintenance (O&M) Costs

Operation and maintenance costs will be based on annual average flow and load conditions. Unit costs for labor, materials and power were developed based on current City costs and will be used to develop operation and maintenance costs. O&M unit costs are summarized in Table 6-14.

Table 6-14: Operation and Maintenance Unit Costs

Item	Unit Cost
O&M Labor (including benefits), $/hour	40.00
Electrical Energy, $/kwH	0.06
Hydrated Lime (Ca(OH)2) $/delivered ton	180.00
Liquid Alum (Al2(SO4)3), $/lb	0.55

Present Worth

Consistent with Natural Resource Conservation Services guidelines for 2009 water resources projects, an annual discount rate of 4.625 percent will be used for present worth analyses. Alternatives will be compared on a present worth basis. The present worth cost of the alternatives will include:

- Capital costs
- O&M costs including labor, materials, chemicals and energy.

CRITERIA FOR NON-ECONOMIC EVALUATIONS

Alternatives will be evaluated using criteria that will assess their relative appropriateness and feasibility. These criteria include:

- Performance: The alternative should be able to consistently meet treatment requirements.
- Expandability: The alternative should be expandable in the future to accommodate future population growth.
- Ease of Operation: The alternative should be straightforward in its operation, requiring a reasonable amount of operator attention. Further, the alternative should not create uncomfortable or unsafe working conditions for operators.
- Reliability: The alternative should provide consistent results with a reasonable amount of maintenance. The alternative should provide redundant equipment or systems.

- O&M Considerations: This criterion includes considerations such as the amount of equipment that must be operated and maintained, and the ability to handle upset conditions.
- Constructability: The alternative should be realistically constructible with minimal disruption to treatment plant operation.
- Odor: Alternatives that contain or do not produce odors are favored.
- Flexibility: This criterion considers the operational options that would be available to WRF personnel to optimize efficiency and performance and accommodate future changed conditions.
- Complexity: Alternatives that require extensive use of equipment and sophisticated controls are considered to be more complex.
- Energy use: Alternatives that are energy efficient, or use less energy, are favored.
- Track record: Alternatives that have been used successfully in similar applications are favored.
- Control: This criteria relates the amount of control the City has over an alternative relative to other stakeholders.
- Regulatory Compliance: Alternatives that provide compliance with likely future regulatory requirements are favored.
- Impacts to other processes: Some alternatives can increase the cost and complexity of other treatment processes.
- Implementation: Alternatives that have few implementation hurdles are favored.

CHAPTER 7

DEVELOPMENT AND EVALUATION OF ALTERNATIVES

The Canyonville Wastewater Treatment Plant (WWTP) has a history of compliance with the treatment requirements set forth in its National Pollutant Discharge Elimination System (NPDES) permit. However, as the City faces issues such as aging equipment and structures; increased flows and loads from a growing population; and more restrictive permit requirements triggered by the South Umpqua River Total Maximum Daily Load (TMDL); it will be necessary to upgrade many of the WWTP's treatment processes. Careful planning and implementation of these improvements will ensure that the WWTP continues to satisfy its permit requirements in the years to come. The primary objective of this chapter is to evaluate long-term wastewater management program alternatives that address anticipated environmental, regulatory, growth, and community issues.

Due to the wide array of alternatives available to the City, a deliberate, systematic approach to alternative development and evaluation was utilized. This approach essentially started with "big picture" wastewater program alternatives, and progressively refined and added detail and definition. By periodically applying evaluation criteria, inadequate alternatives were identified and eliminated along the way. This approach, outlined on Figure 7-1, allowed for efficient consideration of a wide range of options.

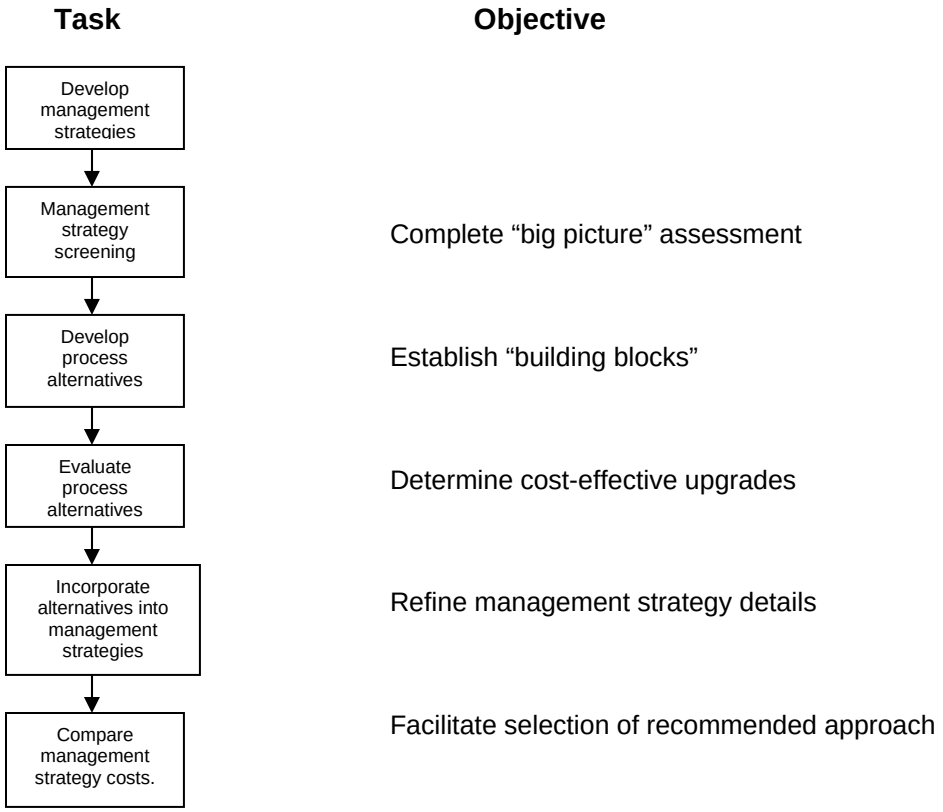


Figure 7-1: Alternative Development and Evaluation Approach

WASTEWATER MANAGEMENT STRATEGIES

Prior to evaluating potential improvements for each unit treatment process at the WWTP, it is useful to consider potential long-term wastewater management strategies. These strategies represent “big-picture” approaches to addressing wastewater issues long term. This section presents evaluation criteria, the potential management strategies, strategy screening, and potential approaches. The final use of the treated effluent typically defines the elements of the management strategy.

Management Strategy Evaluation Criteria

The criteria used to evaluate alternative management strategies include regulatory, relative costs, operation and maintenance (O&M), and implementation. An effective management strategy includes a suite of treatment technologies and controls that provide the flexibility and reliability needed to meet the anticipated requirements.

Regulatory Criteria. In order to establish design criteria for future treatment facilities, it is necessary to consider anticipated future treatment requirements. Chapter 6 presented an analysis of the anticipated treatment requirements. This section reviews key criteria.

- *Thermal Limits.* The South Umpqua TMDL included a thermal waste load allocation (WLA) for the WWTP.
- *Ammonia Limits.* Dry weather ammonia concentration limitations will be included in the next NPDES permit. Based on discussion with DEQ, it is anticipated that wet weather ammonia limits will not be imposed.
- *Phosphorus Limits.* The TMDL establishes dry weather phosphorus waste load allocations for the WWTP. Estimated effluent limits were presented in Chapter 6.
- *Mixing Zone*. The Regulatory Mixing Zone Internal Management Directive (DEQ, December 2007) established new procedures for estimating available mixing between receiving streams and WWTP effluent. Estimated available dilution was presented in Chapter 6.
- *Toxics.* The toxic substances discharged by the WWTP that impact the South Umpqua River are chlorine and ammonia. Dechlorination will be a requirement of the next NPDES permit. Ammonia toxicity is discussed above. Future permit renewal cycles will likely require that other toxic substances be monitored to assess their impact to the River.
- *Metals.* Due to toxicity concerns, metals limits could conceivably become part of future permit requirements.
- *Turbidity.* The WWTP complies with the current turbidity standards set in the NPDES permit. It is probable that a new standard may be approved in the future that could affect the turbidity limits.
- *PBTs.* Persistent bioaccumulative and toxic (PBT) pollutants are chemicals that are toxic, persist in the environment, bioaccumulate in food chains, and pose risks to human health and ecosystems. PBTs are currently unregulated; however, concern over these constituents is increasing and they could be regulated in the future.

Cost Criteria. The initial screening of wastewater management strategies will include a qualitative cost comparison. Cost considerations can be generally placed into one of three categories:

- *Capital costs.* Capital costs include planning, engineering, administration, permitting, land acquisition, and construction.
- *Operation and maintenance costs.* Major O&M cost elements include labor, energy, chemicals, equipment replacement, and facility maintenance.
- *Construction phasing.* Constructing treatment facilities on a "just-in-time" basis often provides an economic benefit by (a) distributing costs over time, and (b) deferring expenses as long as possible.

O&M Considerations. The operation and maintenance of a wastewater treatment facility covers a broad spectrum of tasks required to assure reliable performance. The following factors were considered:

- *Increased Loading.* The ability to expand in order to accommodate growth needs to be considered in the planning process.
- *Performance.* The recommended facility improvements should meet or exceed the anticipated treatment requirements.
- *Operational Flexibility.* The facility improvements should maintain the ability to adapt to regulatory changes as needed.
- *Reliability.* Each unit process should provide quality, proven performance, the ability to take units out of service for maintenance, and bypassing capability.
- *Maintainability.* The design should consider non-proprietary components, parts availability.
- *Odors.* Recommended facilities should create minimal odor or be suitable for addition of odor containment and treatment.
- *Environmental.* The recommended management strategy should minimize impacts to river water quality, and minimize energy and chemical usage.

Implementation. Implementation considerations include:

- *Cash flow.* Are cash flow demands achievable?
- *Land availability.* Is adequate land available for construction of new facilities?
- *Operation during construction.* The City is responsible for maintaining full compliance with the NPDES permit limits at all times during construction.
- *Acceptability.* Are stakeholders and the public likely to support the project?

Development of Management Strategies

Potential wastewater management strategies were discussed with City representatives at a 13 August 2009 meeting. This section presents a summary of the following management strategies:

- MS1. Year-round discharge
- MS2. Membrane treatment with year-round discharge
- MS3. Indirect discharge
- MS4. Dry weather reuse, wet weather discharge

- MS5. Membrane treatment with dry weather reuse, wet weather discharge
- MS6. Zero discharge
- MS7. Dry weather reuse with Tribe, wet weather discharge

MS1. Continued Year-Round Direct Discharge. With this strategy, the WWTP would be expanded and improved with a combination of upgraded and new treatment facilities. Dry weather phosphorus and ammonia removal would be required, and a new outfall would be constructed. Because of the low effluent phosphorus limits, both biological and chemical treatment facilities are anticipated. A schematic for this strategy is presented as Figure 7-2.

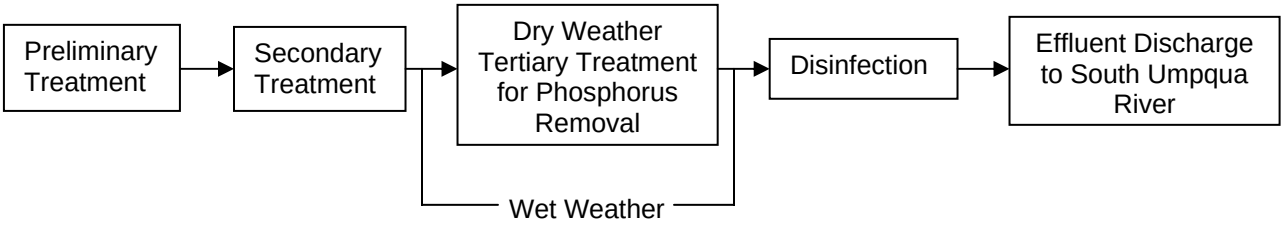


Figure 7-2: Management Strategy MS1. Continued Year-Round Direct Discharge

MS2. Membrane Treatment with Continued Year-Round Direct Discharge. Strategy MS2 consists of demolishing many of the existing treatment facilities and replacing them with a membrane bioreactor (MBR). MBRs combine the functions of aeration basins, secondary clarifiers, and filters into a single treatment process. While MBRs consistently produce high quality effluent and have relatively low space requirements, their limited peak flow treatment capacity generally results in high costs and much of the equipment being out-of-service during normal flow conditions. Phosphorus removal would be attained through both biological and chemical means. A new outfall would be constructed. This strategy is shown schematically on Figure 7-3.

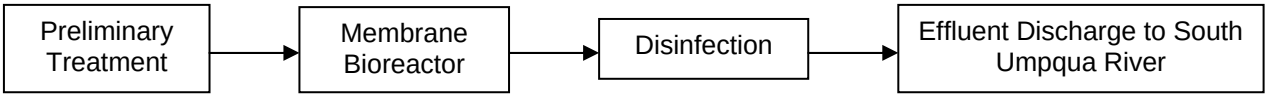


Figure 7-3: Management Strategy MS2. Membrane Treatment with Continued Year-Round Direct Discharge

MS3. Indirect Discharge. DEQ developed guidance for indirect discharge systems in September 2007. In essence, indirect discharge is infiltration of treated effluent into a shallow groundwater aquifer that flows into a nearby receiving stream. The direction of groundwater flow must always be toward the river, and subsurface effluent must not impact any water supply wells. Consequently, the feasibility of indirect discharge is site specific, and is affected by factors such as groundwater gradient, soil type, presence of water supply wells, geology, and topography. Comprehensive hydrogeologic investigations are needed to ascertain feasibility and design criteria. The potential benefits of indirect discharge include:

- Enhanced dilution
- Effluent cooling

- Additional biological and physical treatment by the soil
- Potential phosphorus removal (depending on soil type)
- Potential ammonia removal (depending on site conditions)

Indirect discharge systems can take many forms, including infiltration ponds, “leaky” wetlands, piped exfiltration galleries, and irrigation above agronomic rates. An indirect discharge system designed to accommodate effluent flows year-round would eliminate the need for an outfall. Figure 7-4 presents a schematic for a year-round indirect discharge strategy.

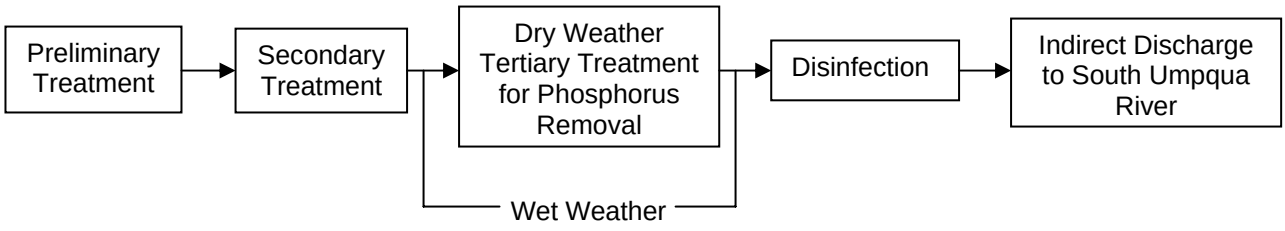


Figure 7-4: Management Strategy MS3. Indirect Discharge

MS4. Dry Weather Reuse, Wet Weather Direct Discharge. Many Oregon communities that discharge to receiving streams with low summertime flows have implemented recycled water programs. In most instances, the recycled water is used for agricultural irrigation.
While removal of effluent from a receiving stream decreases its flow, it is nevertheless generally accepted that effluent reuse provides a net water quality benefit. As discussed in Chapter 6, DEQ’s recycled water regulations identify five classes of recycled water (A through E). Because the WWTP is equipped with a filtration system, it may be capable of producing Class A (the highest quality) recycled water. Lower classes of recycled water require lesser levels of treatment, but at the expense of additional use restrictions.

Because effluent would no longer be discharged to the river during the dry weather season, ammonia and phosphorus removal requirements would be largely avoided. Construction of a new outfall would be required for use during the wet weather season.

When land for effluent storage facilities is available, it is often beneficial to determine the balance between construction of storage facilities and development of irrigation land.
This determination can be used to minimize either land area or costs. A schematic of this strategy is provided on Figure 7-5.

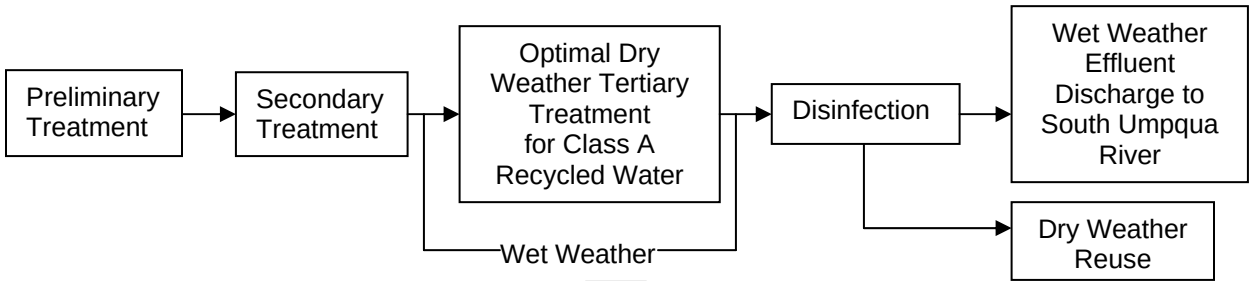


Figure 7-5: Management Strategy MS4. Dry Weather Reuse, Wet Weather Direct Discharge

MS5. Membrane Treatment with Dry Weather Reuse, Wet Weather Direct Discharge. This strategy combines the MBR treatment system described in Strategy MS2 with the effluent reuse system of Strategy MS4. The MBR would produce Class A recycled water. Figure 7-6 shows a schematic diagram of this strategy.

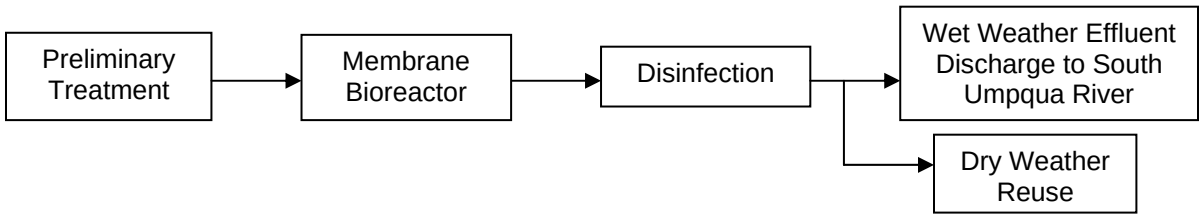


Figure 7-6: Management Strategy MS5. Membrane Treatment with Dry Weather Reuse, Wet Weather Direct Discharge

MS6. Zero Discharge. DEQ requires that Facilities Plans evaluate a "zero discharge" alternative. For the purposes of this plan, the zero discharge strategy would consist of a large scale recycled water program. Effluent would be stored in a reservoir during the wet weather season, and applied to agricultural land during dry weather. Because no effluent would be discharged to the river, phosphorus and ammonia removal would not be required. Figure 7-7 presents a schematic of this management strategy.

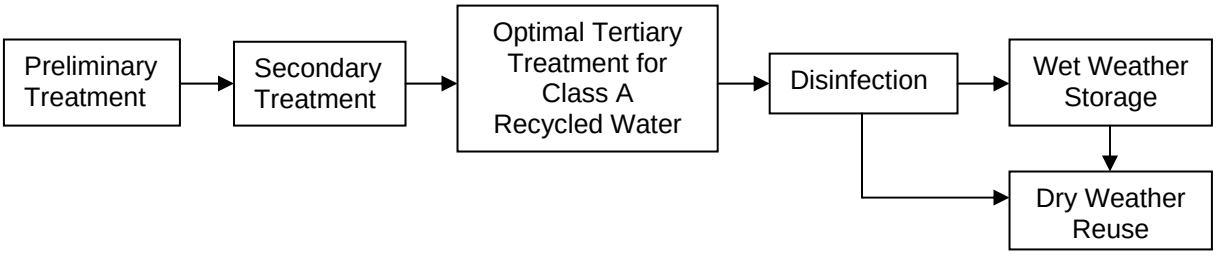


Figure 7-7: Management Strategy MS6. Zero Discharge

MS7. Dry Weather Reuse with Tribe, Wet Weather Discharge. The Cow Creek Band of Umpqua Tribe of Indians' (Tribe) has developments to the northwest of the WWTP that require irrigation. The City and Tribe have been in contact regarding the potential for the City to supply recycled water for use by the Tribe. The Tribe is regulated by the Environmental Protection Agency (EPA) rather than DEQ, so different recycled water quality requirements apply. Figure 7-8 presents a schematic of this management strategy.

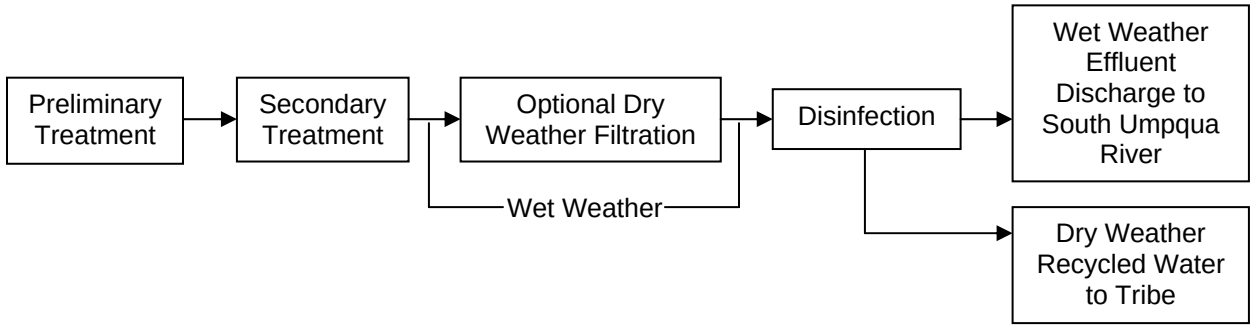


Figure 7-8: Management Strategy MS7. Dry Weather Reuse with Tribe, Wet Weather Discharge

Management Strategy Initial Assessment and Screening

Table 7-1 applies the evaluation criteria to the six wastewater management strategies under consideration. Key considerations in this initial assessment follow:

Table 7-1: Initial Assessment of Wastewater Management Strategies

	Regulatory							Cost			O&M considerations							Implementation			
Wastewater Management Strategy	Thermal	Ammonia	Phosphorus	Mixing zone	Toxics	Metals	Turbidity	Capital	O&M	Phasing	Increased loading	Performance	Flexibility	Reliability	Maintainability	Odors	Environmental	Cash flow	Land availability	Construction operations	Acceptability
MS1. Continued Year-Round Direct Discharge	-	0	0	0	-	-	-	+	0	0	0	0	-	0	+	+	-	+	0	-	+
MS2. Membrane Treatment with Continued Year-Round Direct Discharge	-	0	0	0	-	-	+	-	-	-	+	+	0	0	0	+	-	-	+	+	+
MS3. Indirect Discharge	+	0	?	+	0	0	+	0	+	0	0	0	0	0	0	+	0	0	-	-	0
MS4. Dry Weather Reuse, Wet Weather Direct Discharge	+	+	+	0	+	+	+	0	0	+	+	0	+	0	+	0	+	0	-	-	-
MS5. Membrane Treatment with Dry Weather Reuse, Wet Weather Direct Discharge	+	+	+	0	+	+	+	-	-	-	+	+	+	0	0	0	-	-	-	+	-
MS6. Zero Discharge	+	+	+	+	+	+	+	-	-	-	+	+	-	+	+	-	+	-	-	0	-
MS7. Dry Weather Reuse with Tribe, Wet Weather Discharge	+	+	+	0	+	+	+	+	0	0	0	0	-	?	+	+	+	-	+	+	?

Legend:

+	positive
0	neutral
-	negative
N/A	not applicable
?	Unknown

MS1. Continued Year-Round Direct Discharge. The main advantages of Strategy MS1 are:

- Cost. Relative to other strategies, capital costs are likely to be lower as the need for all-new treatment, conveyance, and reuse systems is minimized.
- Implementation. Because this strategy is essentially a continuation of the City's existing wastewater management practices, stakeholder concerns are likely to be minimal.

The primary disadvantage of this approach is the potential impact of future regulatory requirements. Because the WWTP's effluent would be discharged to the South Umpqua River year-round, the City would be affected by future changes to water quality regulations, which generally become increasingly stringent over time. This strategy will be retained for further evaluation.

MS2. Membrane Treatment with Continued Year-Round Direct Discharge. Compared to Strategy MS1, the MBR included in Strategy MS2 produces higher quality effluent. However, this enhanced effluent quality is not needed for compliance with near-term water quality standards. Furthermore, the cost of a new MBR treatment system is significant, particularly given the flow peaking factors experienced at the WWTP and the limited hydraulic loading capability of existing membrane technologies. For these reasons, Strategy MS2 will not be evaluated further. For reference, cost information for a membrane treatment system sized to treat the maximum month wet weather flow is included in Appendix K.

MS3. Indirect Discharge. Preliminary investigations indicate that adequate land for an indirect discharge system may be available within reasonable proximity to the WWTP. Indirect discharge offers the opportunity to offset treatment facility upgrade and outfall costs. While a great deal of uncertainty remains, indirect discharge will be evaluated further.

MS4. Dry Weather Reuse, Wet Weather Direct Discharge. Because the effluent would no longer be discharged to the river during critical low flow conditions, this strategy would provide considerable water quality and regulatory benefits. Wet weather discharge would be continued to eliminate the need for a large storage reservoir. While the cost to implement a recycled water program can be significant, eliminating the need for phosphorus and ammonia removal facilities offsets these costs to an extent. Strategy MS4 will be evaluated further.

MS5. Membrane Treatment with Dry Weather Reuse, Wet Weather Direct Discharge. This management strategy provides all the same benefits as Strategy MS4; however, the addition of the MBR increases effluent quality and cost. The enhanced effluent quality provided by membrane treatment is not required by current regulatory standards. Consequently, the additional cost incurred would not provide a significant benefit at this time. Therefore, Strategy MS5 will not be evaluated further.

MS6. Zero Discharge. This strategy provides significant regulatory advantages. By eliminating effluent discharge to the river, the City would not longer be subject to current or future water quality standards. However, the amount of land area and storage volume needed to eliminate effluent discharge through year-round reuse is significant. We estimate irrigation land area and storage requirements at 800 acres and 115 million gallons, respectively. At 10 feet deep, the storage reservoir would have a surface area of over 35 acres. The amount of agricultural land within reasonable proximity of the WWTP is limited. Consequently, it is highly unlikely that the City would be able to acquire over 800 acres through purchase or long-term lease. In addition, assuming an average cost of $5,000 per acre, the cost of land alone would exceed $4 million. Adding to this the cost of a storage reservoir, transmission pipelines, irrigation equipment, and treatment facility upgrades, and this strategy becomes unreasonably expensive. Consequently, Strategy MS6 will not be evaluated further.

MS7. Dry Weather Reuse with Tribe, Wet Weather Discharge. Similar to Strategy MS4, this approach provides substantial regulatory and water quality benefits. However, it offers the potential for significantly lower costs as well, as effluent storage and reuse facilities would largely be provided by the Tribe. Concerns include having a consistent, reliable water demand by the Tribe and acceptability to DEQ. This strategy will be evaluated further.

Retained Wastewater Management Strategies

The following wastewater management strategies are retained for further evaluation:

- MS1. Continued Year-Round Direct Discharge
- MS3. Indirect Discharge
- MS4. Dry Weather Reuse, Wet Weather Direct Discharge
- MS7. Dry Weather Reuse with Tribe, Wet Weather Discharge

These strategies differ primarily in the means of final effluent reuse or discharge. Other differences are summarized in Table 7-2.

Table 7-2: Comparison of Wastewater Management Strategy Elements

	Wastewater Management Strategy			
Element	**MS1. Year-round discharge**	**MS3. Indirect discharge**	**MS4. Dry weather reuse, wet weather discharge**	**MS7. Dry Weather Reuse with Tribe, Wet Weather Discharge**
Headworks	Required	Required	Required	Required
Influent pumping	Required	Required	Required	Required
Secondary treatment (aeration basins and clarifiers)	Required	Required	Required	Required
Dry weather phosphorus removal	Required	May not be required	Not required	Not required

Element	Wastewater Management Strategy			
	MS1. Year-round discharge	MS3. Indirect discharge	MS4. Dry weather reuse, wet weather discharge	MS7. Dry Weather Reuse with Tribe, Wet Weather Discharge
		depending on soil characteristics		
Dry weather ammonia removal	Required	May not be required depending on soil characteristics	Not required	Not required
Secondary effluent pumping	Required	Required	Required	Required
Filtration for turbidity removal (reuse)	Not required	Not required	Required for production of Class A recycled water	Not required
Filtration for TSS removal (river discharge)	Required	Not required	Not required	Not required
Disinfection	Chlorination or UV	Chlorination or UV	Chlorination	Chlorination
Dechlorination	Required if chlorination used for disinfection	Not required	Required for wet weather discharge. Not required for recycled water (dry weather)	Required for wet weather discharge. Not required for recycled water (dry weather)
Effluent pumping	Not required	Required	Required	Required
Outfall diffuser	Required	Not required	Required for wet weather	Required for wet weather
Effluent/recycled water transmission pipeline	Not required	Required	Required	Required
Recycled water storage	Not required	Not required	Required	May be required (some storage provided by Tribe)
Indirect discharge system	Not required	Required	Not required	Not required
Irrigation system	Not required	Not required	Required	Not required (provided by Tribe)

Many of the elements listed in Table 7-2 can be implemented in multiple ways. The following section evaluates liquid stream treatment process alternatives.

LIQUID STREAM TREATMENT PROCESS ALTERNATIVES

The previous section identified four wastewater management strategies that will be evaluated in further detail. These strategies represent “big picture” approaches for management of the City’s wastewater program. Each of these strategies are comprised of multiple components, such as treatment facility upgrades, new treatment facilities, new conveyance systems, new discharge facilities, and effluent reuse systems. Each of these components represents a “building block” of the wastewater management strategy. Some building blocks, such as headworks and influent pumping upgrades, are required regardless of the direction of the City’s wastewater program, and therefore apply to all management strategies. Other building blocks, such as an indirect discharge system, apply only to a single management strategy. This section evaluates specific alternatives for preliminary treatment and influent pumping; secondary treatment; tertiary treatment; disinfection; and effluent discharge and reuse.

Preliminary Treatment and Influent Pumping Alternatives

As previously discussed in Chapter 4, the existing preliminary treatment and influent pumping facilities at the WWTP are either in poor condition or inoperable. They also have insufficient capacity to accommodate future design flows and loads. As a result, two preliminary and influent pumping alternatives are developed and evaluated in this section. As indicated in Table 7-2, preliminary treatment and influent pumping are required for each wastewater management strategy. Because of this, selection of an alternative can be made independently of the overall wastewater management strategy.

Alternative H1. Screening Upstream of Pumping. This alternative consists of constructing a new plant headworks that includes screening upstream of influent pumping. Mechanical fine screening equipment would be installed in a channel constructed below the elevation of the existing influent sewer point. Flows would enter the screening channel by gravity from the City’s collection system and pass through a fine screen with 1/8-inch or 1/4-inch openings sized to accommodate the design peak hour flow (PHF). Wastewater would then exit the channel downstream of the screen into a self-cleaning, trench style wet well containing four equally sized submersible, non-clog pumps.

The influent pumping station wet well would share a common-wall with the influent screen channels. The pumps would be controlled by adjustable frequency drives, and their operation adjusted to maintain distinct set-point elevations within the wet well. The capacity of the pumping station would be such that the design PHF could be conveyed with one pump out of service.

The influent pumping station would discharge flows to a Headcell treatment unit for grit removal. The Headcell and grit removal equipment would be located adjacent to the influent pumping station. The top of the Headcell unit would be approximately 10 feet above grade. Screened and de-gritted wastewater would flow by gravity from the Headcell unit to the secondary treatment process. Grit collected in the Headcell would be washed and concentrated. The Headcell unit would be designed to handle a flow through rate equally to the design PHF of 2.2 MGD. Design data for this alternative are shown in Table 7-3, while a schematic diagram is provided on Figure 7-9.

Table 7-3: Alternative H1 Screening Upstream of Pumping Design Data

Item	Value
Influent Screening	
Screen type	Mechanical, perforated plate with separate washer and compactor unit
Screen number	1
Capacity, each, MGD	2.2
Opening size, inches	1/8 or 1/4
Screen type	Manual Bar Screen (by-pass)
Screen number	1
Capacity, each, MGD	2.2
Opening size, inches	1
Influent Pumping	
Pump type	Submersible
Pump number	4
Capacity, each, gpm	510
Grit Removal	
Grit removal system	Eutek Headcell, Eutek Grit Snail
Number of units	1
Capacity, each, MGD	2.2

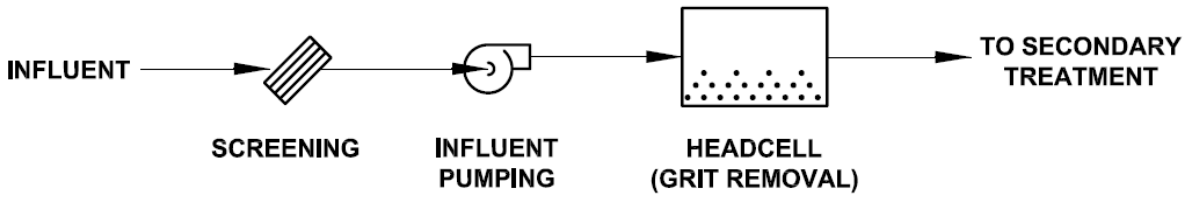


Figure 7-9: Screening Upstream of Pumping Schematic Diagram

Alternative H2. Pumping Upstream of Screening. Under this alternative, a new self cleaning, trench style wet well would be constructed below the elevation of the existing influent sewer. General design of the pumping station would be as described in Alternative H1. The station would convey raw wastewater to a new screen channel constructed approximately 10 feet above grade. The channel would contain mechanical screening equipment designed to accommodate the design PHF. Wastewater would flow by gravity through the screen channel and exit into a vortex grit chamber. The vortex chamber would cause grit to settle while retaining organics in the waste stream. Grit would accumulate in the hopper at the bottom of the vortex chamber and be removed by a grit pump, washed by a cyclone, and dewatered by a grit classifier.

De-gritted wastewater would flow to the secondary treatment process. Design data and a schematic diagram for this alternative are provided in Table 7-4 and Figure 7-10, respectively.

Table 7-4: Pumping Upstream of Screening Design Data

Item	Value
Influent Pumping	
Pump type	Submersible
Pump number	4
Capacity, each, gpm	510
Influent Screening	
Screen type	Mechanical, basket type with integral washer and compactor
Screen number	1
Capacity, each, MGD	2.2
Opening size, inches	1/8 or 1/4
Screen type	Manual Bar Screen (by-pass)
Screen number	1
Capacity, each, MGD	2.2
Opening size, inches	1
Grit Removal	
Grit removal system	Vortex Grit Chamber, Grit Pump, Grit Cyclone, Grit Classifier
Number of units	1
Capacity, each, MGD	2.2

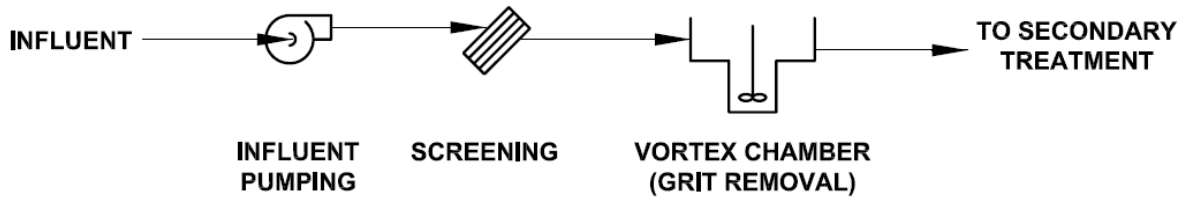


Figure 7-10: Pumping Upstream of Screening Schematic Diagram

Comparison of Preliminary Treatment and Influent Pumping Alternatives. Estimated capital costs for the two preliminary treatment and influent pumping alternatives are summarized in Table 7-5. A breakdown of capital costs is included in Appendix K. A comparison of non-economic factors is provided in Table 7-6.

Table 7-5: Capital Cost Comparison of Primary Treatment and Influent Pumping Alternatives

	Screening Upstream of Pumping H1	Pumping Upstream of Screening H2
Capital Costs[(a)]	$2,700,000	$2,100,000

Notes:

(a) Capital costs include the following multipliers: general conditions, 15 percent; contractor overhead and profit, 15 percent; contingency, 30 percent; engineering, legal, and administration, 25 percent; escalation to mid-point of construction, 3.8 percent.

Table 7-6: Non-economic Comparison of Preliminary Treatment and Influent Pumping Alternatives

	Alternative	
Evaluation Criteria	**H1. Screening upstream of pumping**	**H2. Pumping upstream of screening**
Performance	With screening upstream of influent pumping, the pumps would be less prone to clogging.	With influent pumping upstream of screening, the pumps would be more prone to clogging.
Expandability	Grit removal would be easily expandable because it sits above grade. It would be relatively difficult to expand screening because of the excavation required.	Influent screening and grit removal are both easily expandable, because they are above grade.
Ease of Operation	This alternative would involve less time and effort dealing with clogging influent pumps. Safety is comparable with other alternative.	This alternative would involve more time and effort dealing with clogging influent pumps. Safety is comparable with other alternative.
Reliability	This alternative would be expected to be more reliable due to screening being upstream of the influent pumps.	This alternative would be expected to be less reliable due to pumping being upstream of screening.
O&M Considerations	O&M requirements for both alternatives are essentially equivalent.	O&M requirements for both alternatives are essentially equivalent.
Constructability	Construction of this alternative would involve more excavation, making construction of this alternative more difficult.	Construction of this alternative would involve less and shallower excavation, making construction of this alternative less difficult.
Odor	Both alternatives would have comparable odor issues.	Both alternatives would have comparable odor issues.
Flexibility	Both alternatives offer comparable flexibility. Screening can be	Both alternatives offer comparable flexibility. Screening can be bypassed

Evaluation Criteria	Alternative	
	H1. Screening upstream of pumping	H2. Pumping upstream of screening
	bypassed to a manually cleaned bar screen, lead/lag configuration of pumps can be selected, and there is no standby grit removal system.	to a manually cleaned bar screen, lead/lag configuration of pumps can be selected, and there is no standby grit removal system.
Complexity	Both alternatives have comparable complexity.	Both alternatives have comparable complexity.
Energy Use	Total connected horsepower for this alternative is estimated at 27.33HP.	Total connected horsepower for this alternative is estimated at 31.5HP.
Regulatory Compliance	Complies with redundancy requirements.	Complies with redundancy requirements.
Track Record	Dozens of similar systems in operation.	Hundreds of similar systems in operation.
Impacts to Other Processes	Selection of either alternative will not impact downstream processes.	Selection of either alternative will not impact downstream processes.
Implementation	Implementation of either alternative is relatively simple.	Implementation of either alternative is relatively simple.

Secondary Treatment Alternatives

In general terms, secondary treatment includes biological oxidation of organic material by micro-organisms and gravity separation of micro-organisms from the water through settling. In some instances, secondary treatment can also include the oxidation of ammonia (nitrification) and nutrient (phosphorus and/or nitrogen) removal. Table 7-7 summarizes their applicability to the wastewater management strategies developed earlier in this chapter.

Table 7-7: Secondary Treatment Alternative and Management Strategy Applicability

Secondary Treatment Alternative	Wastewater Management Strategy			
	MS1. Year-round discharge	MS3. Indirect discharge	MS4. Dry weather reuse, wet weather discharge	MS7. Dry Weather Reuse with Tribe, Wet Weather Discharge
S1. Upgrade existing treatment unit			X	X
S2. New conventional activated sludge	X	X	X	X
S3. Sequencing batch reactor	X	X	X	X
S4. Oxidation ditch	X	X	X	X

Prior to beginning a discussion on secondary treatment systems, it is useful to review some of the applicable terminology.

- Aerobic: Dissolved oxygen (DO) present.
- Anoxic: DO absent, nitrates present.
- Anaerobic: DO and nitrates absent.
- Alkalinity: A measurement of the pH buffering capacity of water, typically expressed in milligrams per liter of calcium carbonate.
- Denitrification: The biological conversion of nitrate to nitrogen gas under anoxic conditions. Alkalinity is recovered.
- Nitrification: The biological oxidation of ammonia to nitrate under aerobic conditions. Alkalinity is consumed.
- Anoxic selector: A biological selector that uses anoxic conditions to discourage the growth of unwanted filamentous microorganisms. Desirable organisms are favored due to their ability to thrive under anoxic conditions.
- Anaerobic selector: A biological selector that uses anaerobic conditions to discourage the growth of unwanted filamentous microorganisms. Anaerobic selectors encourage the growth of organisms that provide biological phosphorus removal
- Biological phosphorus removal (BPR): A process in which organisms that uptake excess phosphorus are encouraged to grow. An anaerobic selector is part of such a system.
- Biological nutrient removal (BNR): A process in which combinations of organisms that uptake excess phosphorus, oxidize ammonia, and convert nitrate to nitrogen gas are encouraged to grow.
- Return activated sludge (RAS): The biological solids that settle in clarifier and are returned to the basin as a source of micro-organisms that provide treatment.
- Mixed liquor: The mixture of RAS and raw sewage in the aeration basin.

Alternative S1. Upgrade Existing Treatment Unit. While the existing treatment unit can be modified to provide long-term treatment of biochemical oxygen demand (BOD) and total suspended solids (TSS), there is insufficient tank volume available for reliable ammonia removal (nitrification) during the low-temperature, low-river-flow months of June and October. Therefore, this alternative applies only to wastewater management strategies that eliminate river discharge during the dry weather season. Major elements of this alternative include:

- Conversion of the treatment unit's center clarifier into additional aeration basin tankage
- Piping and hydraulic modifications to the upgraded treatment unit
- Two new secondary clarifiers with return activated sludge (RAS) pump stations
- New blowers and blower room

Design data for this alternative are shown in Table 7-8, while a simplified schematic is provided as Figure 7-11.

Table 7-8: Upgrade Existing Treatment Unit Design Data

Item	Value
Aeration Basins	
Number	1
Diameter, FT	50
Sidewater Depth, FT	13.5
Volume, total, gallons	179,000
Sludge age, days, at max month load and MLSS of 3,000 mg/L	4
Blowers	
Type	Variable speed, rotary lobe
Number	3
Capacity, each, scfm	718
Discharge pressure, max, psig	10
Diffusers	
Type	Fine bubble
Number	500
Mixer	
Type	submersible
Number	2
Horsepower, each	3
ML Recycle Pumps	
Type	Submersible propeller
Number	2
Horsepower, each	3
Secondary Clarifiers	
Number	2
Diameter, FT	32
Sidewater Depth, FT	14
Capacity at PHF, total , MGD	2.2
Overflow rate at PHF, gpd/sqft	1,370
RAS pumps	
Type:	Variable speed, submersible pumps
Number, each clarifier	2
Capacity, each pump, gpm	385
Horsepower, each	5
WAS/scum pumps	
Type:	Variable speed, submersible pumps

Item	Value
Number	2
Capacity, each pump, gpm	100
Horsepower, each	3

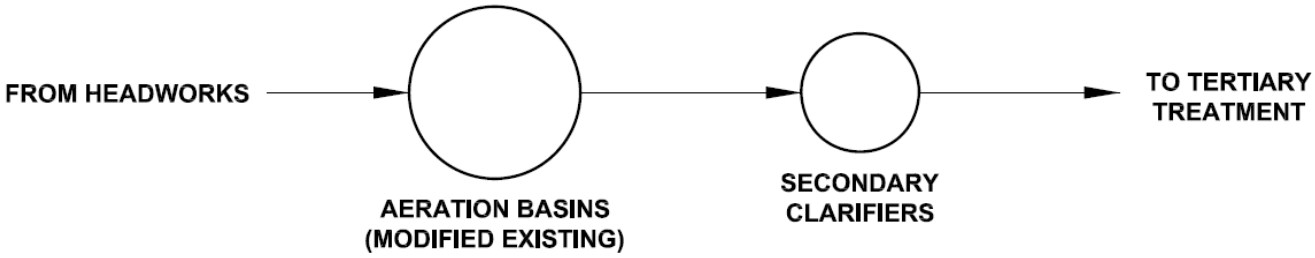


Figure 7-11: Alternative S1 Upgrade Existing Treatment Unit Schematic Diagram

Integrated fixed film activated sludge (IFAS) is a refinement to this alternative that would provide dry weather nitrification and allow for year-round river discharge. With IFAS, media is installed in the aeration basins which promotes a higher biological solids inventory and increased nitrification. However, preliminary estimates showed that this process would not be cost competitive with other alternatives under consideration (Appendix K).

Alternative S2. New Conventional Activated Sludge. This alternative would provide BOD, TSS, and ammonia removal, as well as biological phosphorus removal (BPR). When combined with tertiary treatment and disinfection, Alternative S2 would produce effluent suitable for year-round discharge to the South Umpqua River.
Major components of this alternative include:

- New aeration basin capable of operating in multiple process modes, including biological nutrient removal (BNR).
- Two new secondary clarifiers with RAS pump stations
- New blowers and blower room

The new aeration basin would be configured to operate in one of the many available BNR processes. Nitrification (the conversion of ammonia to nitrate) is achieved through long solids retention times (SRTs). Denitrification (the conversion of nitrate to nitrogen gas) occurs in anoxic zones. Phosphorus removal is facilitated by anaerobic zones. Aeration, recycle pumping, and mechanical mixing are required to create conditions necessary for successful performance. BPR would reduce the chemical demands of the tertiary phosphorus removal system. In addition, to meet reliability requirements, the aeration basins would be designed so that half can be taken out of service for maintenance. Design data for this alternative is provided in Table 7-9. A flow schematic is shown as Figure 7-12.

Table 7-9: New Conventional Activated Sludge Design Data

Item	Value
AERATION BASINS	
Number	1
Length, FT	100
Width, FT	34
Sidewater Depth, FT	18
Volume, gallons	483,000
Sludge age, days, at max month load and MLSS of 3,000 mg/L	10
Blowers	
Type	Variable speed, rotary lobe
Number	3
Capacity, each, scfm	523
Discharge pressure, max, psig	10
Diffusers	
Type	Fine bubble
Number	400
Mixer	
Type	submersible
Number	2
Horsepower, each	3
ML Recycle Pumps	
Type	Submersible propeller
Number	4
Horsepower, each	3
Secondary Clarifiers	
Number	2
Diameter, FT	32
Sidewater Depth, FT	14
Capacity at PHF, total , MGD	2.2
Overflow rate at PHF, gpd/sqft	1,370
RAS pumps	
Type:	Variable speed, submersible pumps
Number, each clarifier	2
Capacity, each pump, gpm	385
Horsepower, each	5
WAS/scum pumps	
Type:	Variable speed, submersible pumps
Number	2
Capacity, each pump, gpm	100
Horsepower, each	3

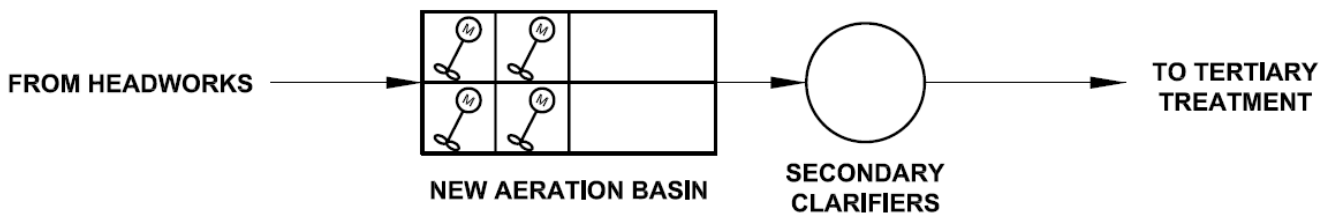


Figure 7-12: Alternative S2 New Conventional Activated Sludge Schematic Diagram

Alternative S3. Sequencing Batch Reactor. Similar to Alternative S2, this alternative would provide BOD, TSS, and nutrient removal through the construction of a new sequencing batch reactor (SBR). A variation of the activated sludge process, SBRs are widely recognized as one of the most economical secondary treatment systems for small communities where all-new treatment plants are required. Elements of an SBR secondary treatment process include:

- A minimum of two SBR tanks equipped with mixers, aeration system, and decanters
- New blowers and blower room
- Flow equalization tank

As its name implies, SBRs treat wastewater in batches. A typical SBR consist of two tanks; one tank is filling with wastewater while the other is performing treatment. The treatment process consists of the following basic steps:

1. Fill: The wastewater is pumped into the tank. The tank is already partially full from the previous treatment cycle and contains a large quantity of biological solids.
2. Mix: Once the tank is full, mixers activate to create anaerobic and anoxic conditions to promote denitrification and BPR.
3. Aerate: Once the mixing cycle is completed, the aeration system is activated to provide aerobic BOD and ammonia treatment.
4. Settle: The aeration system is shut off and the solids are allowed to settle to the bottom of the tank. This step eliminates the need for clarifiers.
5. Decant: Clear water is decanted from the upper portion of the tank. The effluent is directed to the flow attenuation system.
6. Sludge Wasting: Sludge pumps are activated and excess sludge is conveyed to the sludge treatment system.

The SBR's complex treatment sequencing is controlled by a computer. While computerized controls simplify normal operations, manual operation during a control system outage is difficult.

To economically comply with Class I reliability standards, a two-tank SBR system that can operate for short periods using a single tank would be constructed. Because it is a batch process, an SBR must discharge effluent at a rate higher than the influent flow rate. Consequently, downstream facilities, such as filtration and disinfection systems,

must be significantly larger than for a continuous-flow process. This issue can be addressed by adding an equalization basin between the SBR and the downstream processes to provide flow attenuation.

SBRs can be equipped with a variety of different aeration systems, including coarse bubble diffusers, fine bubble diffusers, jet aeration, and surface aerators. Fine bubble diffusers are often the most cost effective from a life-cycle cost perspective. Design data and a simplified flow schematic are provided in Table 7-10 and Figure 7-13, respectively.

Table 7-10: Sequencing Batch Reactor Design Data

Item	Value
Sequencing Batch Reactor	
Number	2
Length, each, FT	43
Width, each, FT	43
Sidewater Depth, FT	20
Volume, each, 1,000 cubic FT	37
Volume, each, gallons	277,000
Sludge age, days, at max month load and MLSS of 3,000 mg/L	13
Equalization Tank	
Number	1
Length, FT	25
Width, FT	25
Sidewater Depth, FT	20
Blowers	
Type	Variable speed, rotary lobe
Number	3
Capacity, each, scfm	390
Discharge pressure, max, psig	9
Aeration Headers	
Number	2
Number of Aerators per header	40

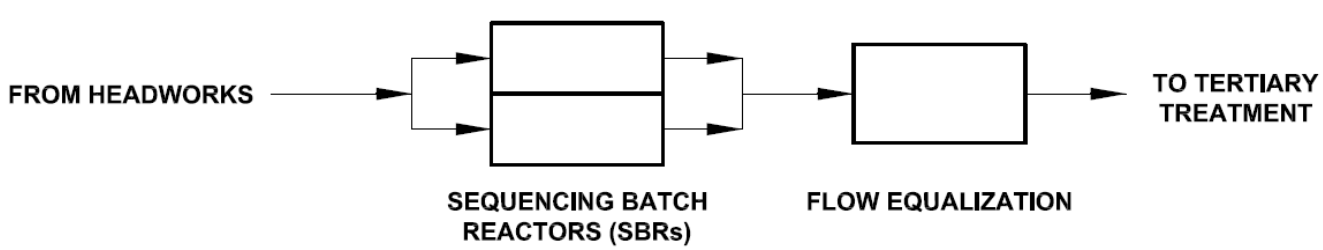


Figure 7-13: Alternative S3 Sequencing Batch Reactor Schematic Diagram

Alternative S4. Oxidation Ditch. Like Alternatives S2 and S3, oxidation ditches can produce effluent which is suitable for year-round discharge. Oxidation ditches have been used at small treatment facilities for decades. Originally configured as economical earthen basins with sloped sides, oxidation ditches have evolved to the point where they employ sophisticated equipment and control systems, and can comply with highly stringent effluent standards. The principal elements of an oxidation ditch system consist of:

- Oxidation ditch with multiple channels
- Aeration system
- Two new secondary clarifiers with RAS pump stations

From a process design perspective, oxidation ditches vary from a conventional activated sludge process mainly in that the mixed liquor flows in a circular pattern around the ditch during operation. This seemingly minor distinction can in fact make a significant difference in treatment performance. The circular flow pattern functions as a high-flow internal recycle stream that inherently promotes denitrification.

Oxidation ditches are renowned for their forgiving nature and simple operation. However, a side effect of simplicity is often minimal flexibility, and this is the case with oxidation ditches. Operators are limited in the adjustments they can make to accommodate changing conditions.

Compliance with reliability standards would necessitate construction of an oxidation ditch with multiple channels, so that one can be taken out of service for maintenance or repairs. While many different types of aeration systems can be installed in oxidation ditches, experience shows that surface aerators are normally the most cost effective. Surface aerators provide both aeration and the horizontal energy needed to move the tank contents in a circular path. Design data for the oxidation ditch system are provided in Table 7-11, while a schematic diagram is shown on Figure 7-14.

Table 7-11: Oxidation Ditch Design Data

Item	Value
Oxidation Ditch	
Number	1
Overall length, FT	94
Overall width, FT	70
Sidewater depth, FT	15
Volume, gallons	535,700
Sludge age, days, at average load and MLSS of 2,800 mg/L	20
Anaerobic Tank	
Number	1
Overall length, FT	15
Overall width, FT	15
Sidewater depth, FT	12
Volume, gallons	20,200

Item	Value
Aerators	
Type	Rotary disc
Number	2
Horsepower, each	25
Anoxic Mixer	
Type	Floating
Number	1
Horsepower, each	3
Anaerobic Mixer	
Type	Floating
Number	1
Horsepower, each	1
ML Recycle Pumps	
Type	Submersible propeller
Number	1
Horsepower, each	3
Secondary Clarifiers	
Number	2
Diameter, FT	32
Sidewater Depth, FT	14
Capacity at PHF, total , MGD	2.2
Overflow rate at PHF, gpd/sqft	1,370
RAS pumps	
Type:	Variable speed, submersible pumps
Number, each clarifier	2
Capacity, each pump, gpm	385
Horsepower, each	5
WAS/scum pumps	
Type:	Variable speed, submersible pumps
Number	2
Capacity, each pump, gpm	100
Horsepower, each	3

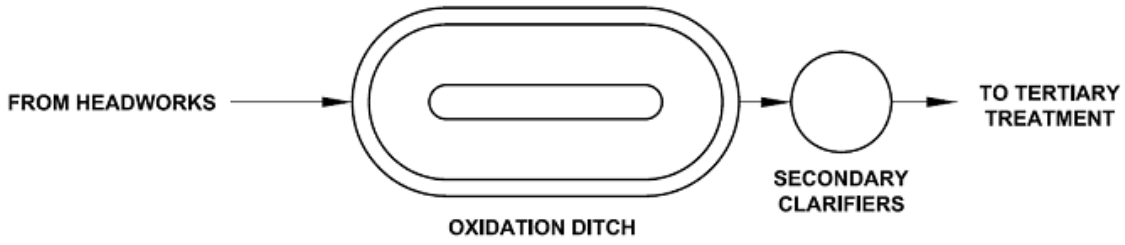


Figure 7-14: Alternative S4 Oxidation Ditch Schematic Diagram

Comparison of Secondary Treatment Alternatives. Estimated capital costs for the four secondary treatment alternatives are summarized in Table 7-12. A comparison of non-economic factors is provided in Table 7-13.

Table 7-12: Capital Cost Comparison of Secondary Treatment Alternatives

	Modify Existing Aeration Basins	Construct New Aeration Basins	Sequencing Batch Reactor	Oxidation Ditch
	S1	S2	S3	S4
Capital Costs[(a)]	$3,400,000	$5,400,000	$4,200,000	$4,600,000

Note:

(a) a Capital costs include the following multipliers: general conditions, 15 percent;contractor overhead and profit, 15 percent; contingency, 30 percent; engineering, legal, and administration, 25 percent,escalation to mid-point of construction, 3.8 percent

Table 7-13: Non-economic Comparison of Secondary Treatment Alternatives

Evaluation Criteria	Alternative			
	S1. Upgrade Existing Treatment Unit	S2. New Conventional Activated Sludge	S3. Sequencing Batch Reactor	S4. Oxidation Ditch
Performance	Provides BOD removal only	Provides BOD, ammonia, and some phosphorus removal	Provides BOD, ammonia, and some phosphorus removal	Provides BOD, ammonia, and some phosphorus removal
Expandability	Difficult to expand existing circular tank	Rectangular tank allows for economical future common-wall construction	Rectangular tank allows for economical future common-wall construction	Future expansion requires construction of additional tanks
Ease of Operation	WWTP personnel familiar with operation of existing system	Added complexity of biological nutrient removal and recycle pumping	Significantly different than existing system. Computerized controls	Relatively straightforward operations for biological nutrient removal system
Reliability	Relatively high rate system is more susceptible to upsets	Extended aeration resistant to upsets	Extended aeration resistant to upsets. Generally stable process. Reliance on computerized controls	Extended aeration resistant to upsets. Generally stable process
O&M Considerations	No complex nutrient removal processes. Reliance on 30-year-old tankage	More equipment than other alternatives	No separate clarifiers	Surface aeration eliminates need for blowers
Constructability	Conversion of center clarifier to additional aeration tankage requires careful	Large space requirements may necessitate land acquisition to	Large space requirements may necessitate land acquisition to permit	Large space requirements may necessitate land acquisition to

Evaluation Criteria	Alternative			
	coordination	permit operation during construction	operation during construction	permit operation during construction
Odor	Higher odor potential than other alternatives	Extended aeration should minimize odor potential	Extended aeration should minimize odor potential	Extended aeration should minimize odor potential
Flexibility	Contact/stabilization, plug flow, and anaerobic selector operating modes	Significant operational flexibility can be incorporated into design	Operational flexibility accomplished through adjustment of treatment times	Minimal operational flexibility
Complexity	Similar to existing system	Highly complex nutrient removal process requires significant operator attention	Complex automated control system, but process is relatively simple when control system is functional	Relatively simple system typically requires few operational adjustments
Energy Use	Lower than other alternatives due to lack of ammonia removal	Efficient aeration equipment. Low head loss	Fill/empty batch system increases required pumping energy. Efficient aeration equipment	Relatively inefficient surface aeration equipment. Low head loss
Track record	The City has operated this type of system for decades	Full biological nutrient removal activated sludge systems are typically constructed at larger WWTPs	Common at small WWTPs since the mid 1980s	Common at small WWTPs for many decades
Regulatory Compliance	Should reliably comply with effluent cBOD and TSS limits. Cannot comply with dry weather ammonia and phosphorus limits	Should reliably comply with effluent cBOD, TSS, and ammonia limits. Tertiary treatment needed to comply with phosphorus limits	Should reliably comply with effluent cBOD, TSS, and ammonia limits. Tertiary treatment needed to comply with phosphorus limits	Should reliably comply with effluent cBOD, TSS, and ammonia limits. Tertiary treatment needed to comply with phosphorus limits
Impacts to Other Processes	Similar to existing system	Biological phosphorus removal would reduce tertiary chemical use. Extended aeration may reduce sludge production	Biological phosphorus removal would reduce tertiary chemical use. Extended aeration may reduce sludge production. Fill/empty operation greatly increases flow rates experienced by downstream processed, necessitating flow attenuation	Biological phosphorus removal would reduce tertiary chemical use. Extended aeration may reduce sludge production

Evaluation Criteria	Alternative			
Implementation	Space required for new clarifiers	Space required for new aeration tank and clarifiers. May be necessary to acquire additional land.	Space required for SBR tank. No clarifiers needed. May be necessary to acquire additional land	Space required for new aeration tank and clarifiers. May be necessary to acquire additional land

Tertiary Treatment Alternatives

As discussed in Chapter 6, the WWTP's current NPDES permit does not include a limit for total and inorganic phosphorus discharged to the South Umpqua River; however, dry weather season effluent phosphorus limits consistent with the WLA will be included in the next NPDES permit. Effluent phosphorus limits will be linked to river flow rate. The current downflow sand filtration system is effective at capturing suspended solids and trace amounts of phosphorus, but is not configured to remove phosphorus in the amounts necessary to meet future requirements.

The removal of phosphorus from wastewater involves the incorporation of phosphate into a solid form and the subsequent removal of those solids. This can sometimes be done through biological removal alone, with a reduction in effluent phosphorus concentrations to 0.5 mgP/L possible. However, reliable BNR systems are generally larger facilities with significant resources available for process control control and monitoring. In addition, BNR facilities are generally more susceptible to process upset than conventional biological treatment systems. Consequently, reliance solely on biological processes for reliable compliance has not been pursued as an option for the City.

Chemical precipitation is a common form of phosphorus removal and is brought about by the addition of salts and multivalent metal ions that form precipitates of soluble phosphates. The chemicals most commonly used are lime, alum, and ferric chloride. Since the WWTP is already using lime for biosolids stabilization, it could be assumed that lime would also be the chemical used for phosphorus removal. However, lime in the quantities required for adequate phosphorus removal would raise the pH of the final effluent to above 9, which would necessitate additional chemical addition for pH adjustment prior to discharge. Therefore, alum or ferric chloride are anticipated to be used in the tertiary treatment alternatives.

Chemical phosphorus removal can occur anywhere in the liquid treatment process downstream of the headworks, but the most effective location for dosing alum or ferric chloride is after secondary treatment, prior to filtration. Phosphorus levels less than 0.02 mgP/L have been achieved at some WWTPs that use advanced filtration techniques.

The existing sand filter is sized to handle approximately 0.7 MGD with all three cells in service. For planning purposes, one cell is assumed to be out of service when sizing equipment for the maximum month dry weather flow rate (0.48 MGD for the year 2035). As a result, the existing filtration system would only have a firm capacity of 0.46 MGD.

Design criteria for the phosphorus removal system are shown in Table 7-14. Although alum is indicated as the precipitant chemical in the table, ferric chloride quantities would be similar.

Table 7-14: Summary of Phosphorus Removal Requirements

Parameter	Design Year 2035
Sec. Effluent Flowrate – MMDWF (MGD)	0.48
Average Total-P Concentration in Sec. Effluent (mg/L)[(a)]	1
Target Total-P Concentration in Final Effluent (mg/L)	0.3
Molar Ratio of Alum-to-Phosphorus (Al:P)	2.6:1
Alum Required (lbs/day)	9
Alum Solution Required (gal/day)	20
Solids from Phosphorus Removal (lbs/day)	65
Phosphorus Removal Sludge Volume (gal/day)	1,600

Note:

(a) Based on upstream biological phosphorus removal.

Two tertiary treatment alternatives involving chemical dosing and filtration are developed and evaluated in this section. Each of the tertiary treatment alternatives was based on an assumption that the phosphorus concentration in the secondary effluent would be 1mg/L, consistent with upstream BPR. The two methods of tertiary treatment evaluated include (1) chemical coagulation and filtration, and (2) reactive filtration. Other filtration technologies, including cloth disk filters, compressible medium filters, magnetic filtration, and continuous backwash filters have been used for phosphorus removal and should be evaluated during preliminary design. The two alternatives included in this section have the smallest footprint and would allow for reuse of the existing filtration system.

Table 7-15 summarizes the applicability of the tertiary treatment alternatives to the wastewater management strategies developed earlier in this chapter.

Table 7-15: Effluent Discharge and Reuse Alternative and Management Strategy Applicability

	Wastewater Management Strategy			
Alternative	**MS1. Year-round discharge**	**MS3. Indirect discharge**	**MS4. Dry weather reuse, wet weather discharge**	**MS7. Dry Weather Reuse with Tribe, Wet Weather Discharge**
T1. Chemical Coagulation and Filtration.	X	X		
T2. Reactive Filtration	X	X		

Alternative T1. Chemical Coagulation and Filtration. This alternative involves the addition of a chemical coagulant, either alum or ferric chloride, prior to filtration. The chemical would be injected into the secondary effluent stream, gently mixed in a flocculation tank to allow the chemical to adhere to the soluble phosphorus to form a precipitant, and filtered through a downflow sand filter. Precipitant is typically retained on the surface of the filter bed. The precipitant would be removed in the form of sludge during the backwash cycle of the sand filter. Bed depth of the sand filter would be similar to the existing filter, on the order of 3 feet, with flow-through constant and backwash operation occurring in a batch process.

Alternative T1 would require expansion of the existing administration/filtration building to allow for the addition of another set of filter cells. The filter would be identical in size to the existing 14-ft diameter filter, with three compartments that could be operated independently. A 2,500 gallon chemical storage tank would be included in the expanded building to provide a 30-day minimum supply of chemical, along with an 8-ft diameter flocculation tank with propeller mixer. Effluent and reject water pumps would be upgraded to allow for the additional design flowrate. Design data for Alternative T1 is included in Table 7-16.

Table 7-16: Chemical Coagulation and Filtration Design Data

Item	Value
Secondary Effluent Pumps	
Pump type	Submersible
Pump number	3
Capacity, each, gpm	800
Filters	
Media	Granular (sand)
Number of cells	6 (3 exist.)
Diameter, feet	14
Capacity per cell, gpm	160
Flocculation Tank	
Number	1
Diameter, feet	8
Volume, gallons	3,000
Chemical Storage Tanks	
Type	Vertical PE Storage Tank
Number	1
Diameter, feet	8
Volume, each, gal	2,500
Reject Water Pumps	
Pump type	Submersible
Pump number	2
Capacity, each, gpm	100

A schematic diagram of the alternative is included as Figure 7-15.

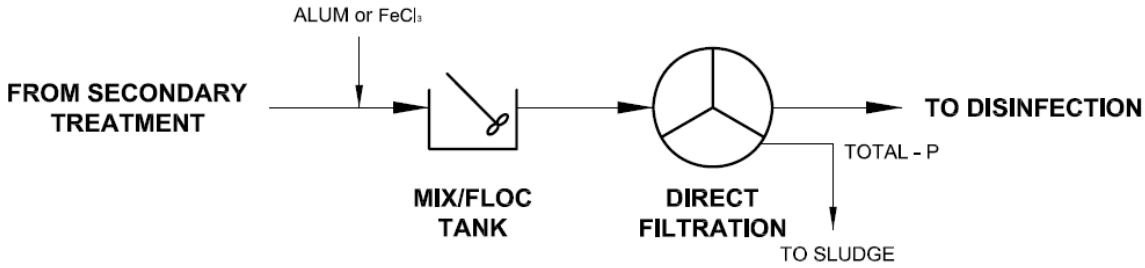


Figure 7-15: Alternative T1 Chemical Coagulation and Filtration Schematic Diagram

Alternative T2. Reactive Filtration. Reactive filtration involves the use of adsorptive media in combination with co-precipitation. Adsorptive media is sand that is factory pre-coated with an iron oxide coating. Continuously regenerating the reactive filter media is accomplished using a moving bed filter to constantly grind the surface of the media, creating new sites for adsorption. A hydrous ferric oxide (HFO) coating is formed on the media surface, abraded away within the filter, and continuously regenerated. HFO allows adsorption of phosphorus. Single-pass systems can meet Total-P limits as low as 0.10 mgP/L, with multi-pass systems capable of 0.010 mgP/L.

The filters used are modular free-standing fiberglass units. Precipitate, including the iron and phosphorus, is continuously separated from the process flow using an air-lift system without the need for backwashing. Bed depth of the moving bed filter would be approximately 5 feet, with flow-through constant. Alternative T2 would require the addition of a separate open framed steel-structure to house three fiberglass filter tanks. Each filter tank would be 9-ft in diameter and 14-ft tall, and provide phosphorus removal during dry-weather months and standard filtration for wet-weather, if necessary. The cells would operate in parallel. Chemical totes would be used in lieu of a storage tank.

The manufacturer Blue Water provides the filtration and chemical dosing equipment as a package system. The media is non-proprietary, but the chemical addition and adsorption technology is proprietary. Design data for Alternative T2 is included in Table 7-17, while a schematic diagram of this alternative is included as Figure 7-16.

Table 7-17: Reactive Filtration Design Data

Item	Value
Secondary Effluent Pumps	
Pump type	Submersible
Pump number	3
Capacity, each, gpm	800
Phosphorus Removal Filters	
Media	Granular (HFO sand)
Number of filters	3

Item	Value
Diameter, feet	9
Capacity per Cell, gpm	110
Existing Filters	
Media	Sand
Number of cells	3
Diameter, feet	14
Capacity per cell, gpm	160
Reject Water Pumps	
Pump type	Self-priming centrifugal
Pump number	2
Capacity, each, gpm	100

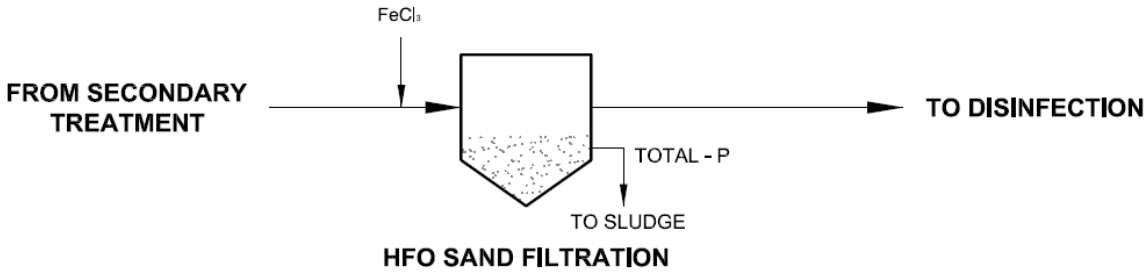


Figure 7-16: Alternative T2 Reactive Filtration Schematic Diagram

Comparison of Tertiary Treatment Alternatives. Estimated capital costs for the two tertiary treatment alternatives are summarized in Table 7-18. Noneconomic criteria are compared in Table 7-19.

Table 7-18: Capital Cost Comparison of Tertiary Treatment Alternatives

	T1 Chemical Coagulation and Filtration	T2 Reactive Filtration
Capital Costs[(a)]	$1,900,000	$2,100,000

Notes:

(a) Capital costs include the following multipliers: general conditions, 15 percent; contractor overhead and profit, 15 percent; engineering, legal, and administration, 25 percent, contingency, 30 percent; escalation to mid-point of construction, 3.8 percent

Table 7-19: Non-economic Comparison of Tertiary Treatment Alternatives

Evaluation Criteria	T1. Chemical Coagulation and Filtration	T2. Reactive Filtration
Performance	Provides phosphorus removal to < 0.3 mgP/L	Provides phosphorus removal to < 0.3 mgP/L
Expandability	Future expansion requires additional filter cells; potentially limited by building	New building provides space for future fourth fiberglass tank
Ease of Operation	Similar to existing filtration system, but with chemical addition	Slightly more involved than current operations
Reliability	Alum use means potential for filter clogging or more frequent backwash	Ferric chloride use means potential for filter clogging or more frequent backwash
O&M Considerations	More equipment to maintain than other alternative	Less equipment to maintain than other alternative
Constructability	May require existing filter shutdown for building expansion, which could be difficult in winter months	May require existing filter shutdown for piping connections, but new building and filters could be constructed any time of year
Odor	Limited and would be contained in building expansion	Reduced due to iron addition, which binds with sulfide compounds
Flexibility	Surface loading and chemical dosing rates fully flexible	Surface loading and chemical dosing rates fully flexible
Complexity	Chemical addition and flocculation are the only challenge beyond existing system	More complex technology than existing equipment
Energy Use	Similar to existing system	Lower than other alternative due to elimination of backwash system
Track Record	Most common form of chemical phosphorus removal at WWTPs	Fairly new technology with less than 10 installations nationwide
Regulatory compliance	Should reliably comply with effluent phosphorus limits with or without upstream BPR	Should reliably comply with effluent phosphorus limits with or without upstream BPR
Impacts to other processes	Backwash system increases loading on secondary treatment system	Will increase sludge production. Cannot use ferric chloride if downstream UV disinfection is constructed
Implementation	Filter building addition should fit within site footprint	Space required for new building

Disinfection Alternatives

Disinfection is the inactivation of pathogenic microorganisms to protect public health. The existing disinfection system utilizes sodium hypochlorite solution. Contact time is provided by a circular chlorine contact basin (CCB) that is a converted trickling filter. Continued use of the existing CCB for disinfection is a concern for four reasons: 1) the existing structure is known to leak, (2) the tank does not have sufficient volume to treat

design flows, (3) there is minimal initial mixing between the secondary effluent and disinfectant, and (4) the CCB has an inadequate length-to-width ratio. Because of these deficiencies, this section presents and evaluates two disinfection alternatives sized to accommodate future flows. Disinfection is required for each wastewater management strategy. The year round discharge and indirect discharge strategies (MS1 and MS3) may employ either chlorination or UV disinfection alternatives. The reuse alternatives (MS4 and MS7) would benefit from a chlorine residual being maintained in the recycled water, and therefore incorporate chlorine disinfection.

Alternative D1. Chlorination/Dechlorination. This alternative consists of constructing a new chlorine contact basin. After application of a chlorine solution, secondary effluent would flow through a rectangular tank with serpentine channels. The tank would be divided into two halves so that one half could be used while the other is out of service for maintenance. The tank would also include the flexibility to operate both halves in series.

Disinfected effluent would flow through an effluent channel where plant utility water and effluent pumps would be located. Dechlorination would be accomplished through sodium bisulfite addition in the effluent channel. Table 7-20 presents design data for Alternative D1, while a schematic diagram is shown in Figure 7-17.

Table 7-20: Alternative D1 Chlorination/Dechlorination Design Data

Item	Value
Chlorine Contact Tank	
Tank number	1
Volume, total, gal	61,000
Tank length, total, feet	50
Tank width, total, feet	25
Total length of channels, LF	216
Channel width, FT	3
Channel sidewater depth, FT	12.5
Disinfection Limits	
Discharge, number of e. Coli/ 100mL	126
Reuse, number of e. Coli/ 100mL	23
Sodium Hypochlorite	
Storage Tanks, number	2
Volume, each, gallons	300
Concentration, each, by weight	12.5 percent
Feed Pumps, number	2
Capacity, each, gph	2
Sodium Bisulfite	
Storage Tanks, number	2
Volume, each, gallons	55
Concentration, each, by weight	35 percent
Feed Pumps, number	2
Capacity, each, gph	0.5

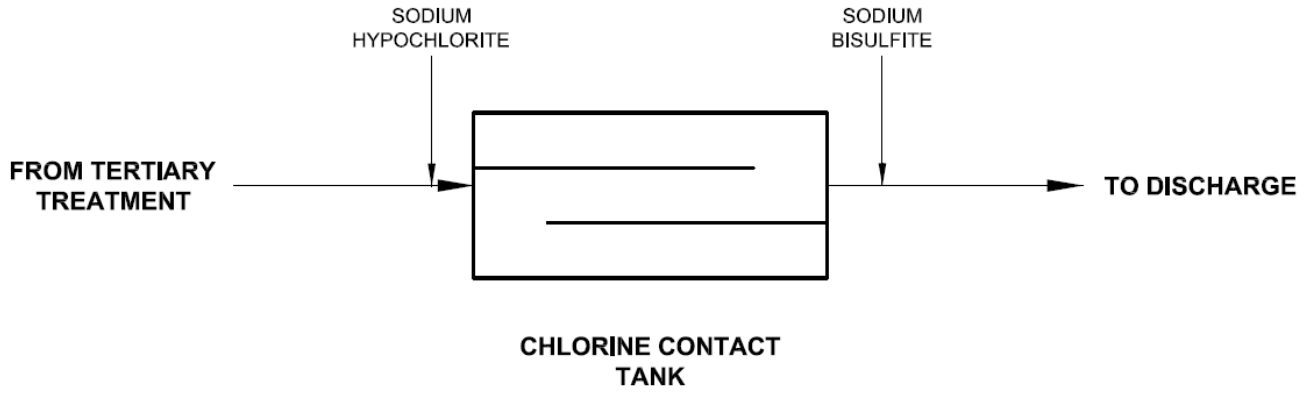


FIGURE 7-17: Alternative D1 Chlorination/Dechlorination Schematic Diagram

Alternative D2. UV Disinfection. This alternative involves construction of a low-pressure, open channel UV disinfection system. Secondary effluent would flow by gravity into the upstream end of a concrete channel equipped with a bank of UV lamps. The bank would consist of 3 modules, and each module would include 8 lamps, for a total of 24 lamps. The system would be designed to allow for future construction of an additional UV channel. An effluent channel would be constructed at the downstream end of the UV channel where utility water and effluent sample pumps would be located. Design data for this alternative are shown in Table 7-21, while a schematic diagram is provided as Figure 7-18. The system would be designed to disinfect the design PHF of 2.2 MGD.

Table 7-21: UV Disinfection Design Data

Item	Value
UV Channel	
Channel length, FT	18
Channel sidewater depth, inches	62
UV Lamps	
Banks, number	1
Modules, number	3
Lamps, number	24
Lamps, type	Low-pressure, high output

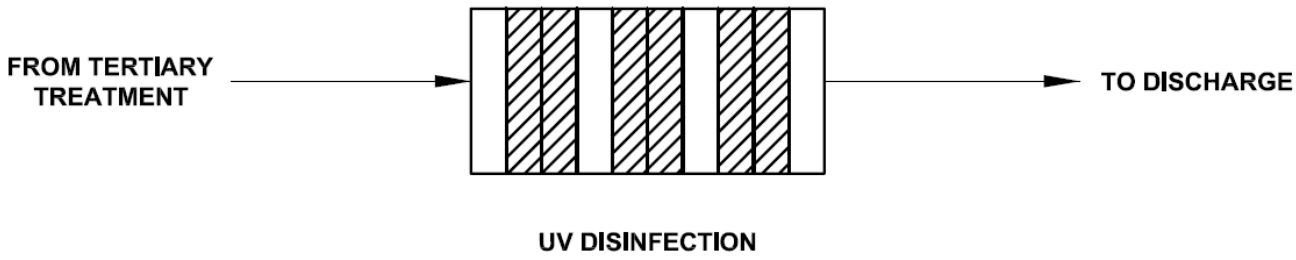


FIGURE 7-18: Alternative D2 UV Disinfection Schematic Diagram

Comparison of Disinfection Alternatives. Estimated capital costs for the two disinfection alternatives are summarized in Table 7-22. A comparison of non-economic factors is provided in Table 7-23.

Table 7-22: Capital Cost Comparison of Disinfection Alternatives.

	Chlorine Contact Tank, D1	UV Disinfection, D2
Capital Costs[(a)]	$1,900,000	$900,000

Note:

(a) Capital costs include the following multipliers: general conditions, 15 percent; contractor overhead and profit, 15 percent; contingency, 30 percent; engineering, legal, and administration, 25 percent; escalation to mid-point of construction, 3.8 percent

Table 7-23: Non-economic Comparison of Disinfection Alternatives

Evaluation Criteria	Alternative	
	D1. Chlorination/Dechlorination	D2. UV Disinfection
Performance	Can meet disinfection requirements for both discharge and re-use.	Generally considered to be suitable for discharge only.
Expandability	Expansion of the chlorine contact basin would be through cost effective common wall construction.	Expansion of the UV disinfection system would involve construction of an adjacent UV channel and the purchase of additional UV lamps.
Ease of Operation	Hypochlorite and bisulfite are hazardous chemicals. Hypochlorite often causes leaks at pipe joints.	Requires monitoring of lamp functionality.
Reliability	Proven reliable.	Impacted by high effluent solids such as during plant upsets.
O&M Considerations	Maintenance of hypochlorite and bisulfite mixers, metering pumps, and chemical storage tanks.	Maintenance and periodic replacement of UV lamps. Lamps are self-cleaning.
Constructability	Large footprint and greater amount of excavation	Small footprint.
Odor	No odors.	No odors.

Evaluation Criteria	Alternative	
	D1. Chlorination/Dechlorination	**D2. UV Disinfection**
Flexibility	The chlorine contact tank would be designed as two halves, and would allow one half to be taken out of service. Adjustments in chemical doses allow for addition disinfection.	The UV open channel system would offer little flexibility.
Complexity	More equipment.	Equipment is more sophisticated.
Energy Use	The system would require a relatively small amount of energy to operate.	High energy demand.
Regulatory Compliance	Reliable permit compliance.	Reliable permit compliance.
Track Record	Hundreds of systems in service.	Hundreds of systems in service.
Impacts to Other Processes	Suitable for reuse or discharge.	Suitable for discharge only.
Implementation	Greater site requirement.	Small footprint.

Effluent Discharge and Reuse Alternatives

Four effluent discharge and reuse alternatives are developed and evaluated in this section. They essentially mirror the wastewater management strategies developed earlier in this chapter as shown in Table 7-24.

Table 7-24: Effluent Discharge and Reuse Alternative and Management Strategy Applicability

Alternative	Wastewater Management Strategy			
	MS1. Year-round discharge	**MS3. Indirect discharge**	**MS4. Dry weather reuse, wet weather discharge**	**MS7. Dry Weather Reuse with Tribe, Wet Weather Discharge**
E1. Year-round river discharge	X			
E2. Indirect discharge		X		
E3. Dry weather reuse, wet weather discharge			X	
E4. Dry weather recycled water to Tribe, wet weather river discharge				X

Alternative E1. Year-Round River Discharge. This alternative consists of constructing a new outfall diffuser in the South Umpqua River. The diffuser would be designed to provide complete mixing with 25 percent of the river flow at the edge of the regulatory

mixing zone (RMZ) per DEQ's RMZ Internal Management Directive. In addition, the outfall must be capable of conveying the projected year 2035 peak hour flow (PHF) of 2.2 MGD. Dilution modeling would be completed as part of diffuser preliminary design. Based on the evaluation in Chapter 6, it is anticipated that ammonia toxicity will control mixing requirements. Design data for this alternative is provided in Table 7-25, while a simplified schematic diagram is shown as Figure 7-19.

Table 7-25: Year-Round Discharge Design Data

Item	Value[(a)]
Outfall diameter, inches	18
Outfall length, feet	100
Diffuser length, feet	30
Diffuser ports, number	3

Note:

(a) Preliminary values to be verified during preliminary design of the outfall diffuser.

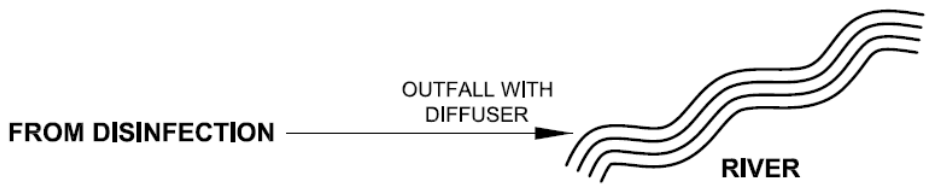


Figure 7-19: Alternative E1 Year-Round Discharge Schematic Diagram

Alternative E2. Indirect Discharge. Indirect discharge of municipal wastewater treatment plant effluent is an acceptable and increasingly common alternative to discharging effluent directly to surface water bodies. Indirect discharge methods are typically employed in situations when the direct discharge of effluent results in, or is expected to result in, an exceedence of water quality criteria.

Indirect, or hyporheic, discharge systems are designed such that effluent is distributed to the subsurface, mixes with groundwater, and then discharges below the ground surface to a receiving water as a baseflow contribution to the stream. During subsurface migration, certain effluent parameters such as temperature, ammonia, phosphorus, and turbidity can be reduced through adsorption or filtration by soil particles or by mixing with cooler native groundwater. Thus, an indirect discharge system is used to effectively condition the effluent to improve its quality prior to discharging to a stream in the hyporheic zone, while still providing the benefits of flow augmentation to that stream. The degree of additional treatment attained through indirect discharge is site specific, and dependent on soil and groundwater characteristics.

The 2007 DEQ Internal Management Directive (IMD) outlines the procedures for DEQ to approve proposed indirect discharge projects in Oregon. A key concept for indirect discharge systems discussed in the IMD is that of a "waste management area." A "waste management area" is defined in the Oregon groundwater quality rules as "any area

where waste, or any material that could become waste if released to the environment, is located, or has been located." For indirect discharge systems, the boundaries of a waste management area correspond to locations in the subsurface where the discharged effluent has undergone sufficient conditioning, or attenuation, such that its quality does not violate applicable surface water or groundwater quality standards. In essence, a waste management area is analogous to a mixing zone for surface water discharges. Discharge permit compliance points for the indirect discharge system are typically set at the boundaries of the waste management area.

Establishing a waste management area for an indirect discharge system requires the completion of a thorough hydrogeologic characterization at a potential site. The hydrogeologic characterization typically requires the installation of site monitoring wells used to establish groundwater flow conditions over at least one annual cycle; collection and analysis of groundwater, soil, and receiving water (stream) samples for key constituents; field pilot tests of the proposed system to quantify infiltration estimates and effluent attenuation; and sometimes numerical modeling to evaluate long-term effects of the system on groundwater and surface water quality. DEQ uses the results of the hydrogeologic characterization to determine if the proposed indirect discharge system is technically feasible, is appropriately located, and is cost effective and likely to succeed relative to other discharge options outlined in a facilities plan.

Indirect discharge systems typically consist of the following effluent delivery methods, constructed in areas with a favorable subsurface hydraulic connection to a surface water body:

- Exfiltration Galleries
- Constructed Wetlands
- Rapid Infiltration Basins

A brief discussion of additional siting requirements and advantages and disadvantages for each of the three indirect discharge methods are discussed below.

***Exfiltration Galleries*.** With this method, WWTP effluent is distributed to the subsurface through a series of buried exfiltration piping galleries. Although the actual design will vary depending on site conditions, a typical exfiltration gallery consists of a network of perforated pipes installed in sand-filled trenches, with the bottom of the trenches completed at least several feet above the groundwater surface, or water table. The separation from the water table is required so that the system does not constitute a direct discharge of WWTP effluent to groundwater, a condition prohibited by state indirect discharge regulations. Furthermore, cyclical exposure to oxidized (unsaturated zone) and anoxic (saturated zone) subsurface conditions can promote attenuation of certain effluent parameters, particularly ammonia.

Additional site requirements and advantages and disadvantages of using exfiltration galleries as part of an indirect discharge system include the following:

- Requires suitably coarse (permeable) soils that are not overlain by a significant thickness of low permeability soils (silt, clay).
- Requires sufficient land area for the pipe gallery network, the amount of which depends on the rate of effluent to be discharged and soil properties;

however, footprint of required land area typically is much less than for other discharge delivery methods.

- Discharge system needs to be close to the stream, with a favorable subsurface hydraulic connection to the stream, but not within flood-prone portions of the stream system.
- For cooling purposes, the preferred depth of penetration of discharged effluent should be at least 6 feet.
- Maintenance or repairs to the discharge system, including unclogging of pipe perforations, can be problematic because most infrastructure is below ground.
- Conditioning of effluent might not be as effective as with rapid infiltration (RI) basins because exposure to oxidized conditions is not as extensive; however, the ability to cool effluent may be greater than that of RI basins.

Constructed (Treatment) Wetlands. With this method of indirect discharge, effluent is distributed to shallow constructed (or treatment) wetlands. “Leaky” constructed wetlands can be designed in which the effluent percolates downward to mix with native soils and groundwater, with eventual subsurface discharge to the stream in the hyporheic zone. The soil and vegetative characteristics associated with the wetland components serve to enhance reduction of key effluent constituents prior to the effluent’s introduction to a receiving water body (stream).

Additional site requirements and advantages and disadvantages of using constructed wetlands as part of an indirect discharge system include the following:

- Can be implemented in areas with low soil permeability.
- Requires sufficient land area, the amount of which depends on the rate and volume of effluent to be discharged, the type and degree of treatment required, and, for a “leaky” wetland system, the infiltration capacity of the subsurface.
- For temperature treatment, densely-spaced emergent vegetation can be included to provide effective shading and thus additional cooling capacity.
- For nitrogen treatment, shallow water depths (less than one foot) are necessary to maintain high dissolved oxygen levels necessary for processing of the dissolved nitrogen species within the effluent.
- Phosphorus treatment using constructed wetlands depends largely on the pH of the water and the adsorption potential of wetland soils.

Rapid Infiltration Basins. With this method, effluent percolates into the subsurface through a series of shallow rapid infiltration (RI) basins excavated at or near the existing ground surface. Rapid infiltration basins have been used to reduce nutrient loads in WWTP effluent. Through suitable cycling of aerobic and anaerobic conditions within a series of RI basins, nitrification/denitrification reactions can be promoted to provide total nitrogen removal. In addition adsorption of phosphorus to soil particles can reduce phosphorus loading to streams. Effluent cooling using RI basins is accomplished primarily by heat exchange between the effluent and native soils and groundwater, the latter of which in summer months is typically much cooler than the effluent temperature.

Additional site requirements and advantages and disadvantages of using rapid infiltration basins as part of an indirect discharge system include the following:

- Requires suitably coarse (permeable) soils that are not overlain by a significant thickness of low permeability soils (silt, clay).
- Requires sufficient land area for basins, which depends on the rate of effluent to be discharged, degree and type of treatment desired, and soil properties.
- RI basins need to be close enough to stream and with suitable site conditions (e.g., topography, slope, soil permeability) to ensure hydraulic connection between RI basins and stream, and that predominant groundwater flow direction is from the basins towards the stream.
- RI basins require periodic maintenance (for example, removal of accumulated sediment).
- Provides partial nitrogen removal of effluent because of exposure to both oxidized (while in basin) and anoxic conditions (in subsurface).

Preliminary Feasibility Assessment of Potential Indirect Discharge Areas. This preliminary indirect discharge feasibility assessment is intended to provide information suitable for deciding whether or not to conduct an additional, more comprehensive feasibility study. Potential site suitability was evaluated using only remote data sources such as soil maps, aerial photographs, topographic maps, and information from nearby well logs. To more thoroughly evaluate indirect discharge feasibility at a specific site would, at a minimum, require information gained from site subsurface investigations (temporary borings, test pits and/or monitoring well installation) and pilot-level infiltration tests.

Furthermore, potential sites were evaluated only on the basis of estimated infiltration capacity, or the volumetric rate at which effluent could be expected to be sustainably discharged at a particular site. Projected year 2035 design flows were used to estimate the required infiltration capacity for an indirect discharge system.

Attenuation potential, or “conditioning”, of effluent to reduce constituents of concern (for example, phosphorus and ammonia) was not evaluated because such processes are highly dependent on site-specific soil and groundwater physical and chemical properties. These properties cannot be determined without a site-specific hydrogeologic characterization.

Using the general siting criteria discussed previously, three potential indirect discharge areas were identified close to the WWTP site.

Area 1 – Immediately Northwest of WWTP. This roughly triangular-shaped area is immediately northwest of and adjacent to the WWTP, bounded by Canyon Creek to the west and the South Umpqua River to the north. There are two reported soil types within this area. The area along both sides of Canyon Creek is underlain by soils described as either gravelly sandy loam or fine sandy loam, both of which are reported to possess moderately rapid permeability, with a saturated hydraulic conductivity of approximately 4 inches per hour. The strip of land adjacent to the South Umpqua River is reportedly underlain by a gravelly loam or gravelly clay loam with moderately slow permeability, with a saturated hydraulic conductivity of approximately 1 inch per hour.

Based on information obtained from the Canyonville, Oregon 15-minute topographic quadrangle map, it appears that the average ground surface elevation in this area varies from about 5 to 15 feet above the base of the adjacent streams. Although depth to bedrock in the immediate area is not known, it is assumed to be near or perhaps a few feet below the base of the streams, so adequate soil thickness is believed to exist in this area. The depth to groundwater is not known, but is assumed to be approximately the same as stream surface elevations at any given time. The ground surface height above both streams suggests that flooding potential of an indirect discharge system would be of moderate concern.

For this preliminary evaluation, it was assumed that a series of exfiltration galleries or pipes would be most appropriate for this area. It was assumed that exfiltration pipes could be installed in trenches approximately 5 feet deep and 5 feet wide along the south bank of the South Umpqua River and both banks (east and west) of Canyon Creek. The estimated available frontage along the three stream banks would be approximately 1,250 feet.

Using the conceptualized exfiltration pipe system described above, along with the reported soil characteristics and available topographic information, it is estimated that approximately 0.30 MGD of effluent could be sustainably discharged in Area 1. Approximately 85 percent of this estimated capacity, or 0.26 MGD, would be accommodated in the reportedly more permeable soils along Canyon Creek.

Aside from uncertainties regarding site-specific soil and groundwater conditions, the following potential limitations were identified for an indirect discharge system located in Area 1:

- Land availability, particularly on the west side of Canyon Creek.
- Stability of stream banks.
- Might require infringement into riparian buffer (trees).
- Need for conveyance pipe to cross Canyon Creek.

The indirect system described for Area 1 would require the following general components:

- 1,250 feet of perforated 8-inch diameter pipe installed in trenches approximately five feet deep by five feet wide; trench lined with geotextile fabric and filled with imported clean medium sand.
- Flow control valves.
- Pump station to convey effluent to exfiltration pipe locations (note: only head sufficient to convey effluent to pipe systems would be required; discharge system would not require additional pressurization).

Area 2 – East of WWTP. This is a heavily wooded area to the east of the WWTP, bounded by the South Umpqua River to the north and a small residential area to the south. There is one principal reported soil type within this area, described as gravelly loam with very high permeability (reported saturated hydraulic conductivity of approximately 53 inches per hour).

Based on information obtained from the Canyonville, Oregon 15-minute topographic quadrangle map, it appears that the average ground surface elevation in this area varies from about 40 to 50 feet above the base of the South Umpqua River. This is the highest elevation of all the sites evaluated, and is most favorable from both a system performance and low flood-potential standpoint. Depth to bedrock in the immediate area is not known, and is assumed to be at least several feet below the base of the streams. This would mean that adequate soil thickness should exist. The ground surface height above the stream suggests that flooding potential of an indirect discharge system would be of little to no concern.

For this preliminary evaluation, it was assumed that a single line of exfiltration pipe would be most appropriate for this area. It was assumed that the exfiltration pipe could be installed in a trench approximately five feet deep and five feet wide along the top of the bank of the South Umpqua River. The estimated available frontage along this area was estimated to be approximately 300 feet.

Using the conceptualized exfiltration pipe system described above, along with the reported soil characteristics and available topographic information, it is estimated that approximately 1.2 MGD of WWTP effluent could be sustainably discharged in Area 2.

Aside from uncertainties regarding site-specific soil and groundwater conditions, the following potential limitations were identified for an indirect discharge system located in Area 2:

- Land availability.
- Possible conflict with existing residences.
- Stability of stream bank.
- Might require infringement into riparian buffer (trees).

The indirect direct system described for Area 2 would require the following general components:

- 300 feet of perforated 8-inch diameter pipe installed in a trench approximately five feet deep by five feet wide; trench lined with geotextile fabric and filled with imported clean medium sand.
- Flow control valves.
- Pump station to convey effluent to exfiltration pipe location.

Area 3 –North Side of River. Area 3 is a low-lying open area in the riparian zone of the South Umpqua River on the north side of the river. The estimated available area totals approximately 10 acres. In general, this area is bounded to the north by Gazley Road, to the east by a wooded area, to the south by the South Umpqua River, and to the west by Gazley Bridge Road.

There is one principal reported soil type within this area, described as fine sandy loam with a reported moderately rapid permeability (saturated hydraulic conductivity of approximately 4 inches per hour).

It appears that the average ground surface elevation in this area varies from about 5 to 8 feet above the base of the South Umpqua River. As with the other areas, depth to bedrock is not known. It is assumed to be near or perhaps a few feet below the base of

the streams. The ground surface height above the stream suggests that flooding potential of an indirect discharge system would be of moderate to high concern.

It appears that either rapid infiltration basins or a leaky constructed wetland would be most appropriate for this area. It is likely that the depth to groundwater is quite shallow in Area 3, which would preclude consideration of a subsurface system using exfiltration pipes. It was assumed that either RI basins or a constructed wetland would encompass approximately 10 acres of open land in Area 3. The average depth of five separate 2-acre basins (or the sides of wetland cells) was estimated to be 3 feet. A constructed wetland might be a more viable option, from both a regulatory and technical perspective, given the area's location within the riparian zone and floodplain of the South Umpqua River. Using reported soil hydraulic characteristics and assuming typical RI basin operational considerations, a 10 acre area would be expected to accommodate approximately 1.0 mgd of effluent on a sustained basis.

Aside from uncertainties regarding site-specific soil and groundwater conditions, the following potential limitations were identified for an indirect discharge system located in Area 3:

- Land availability.
- Flooding potential.
- Infringement into riparian buffer zone (could be mitigated with use of constructed wetland instead of RI basins).
- Requires major conveyance pipe crossing of South Umpqua River.

The RI basin system described for Area 3 would require the following general components:

- A 16-inch conveyance pipe extending from the WWTP to the indirect discharge system, including a river crossing.
- Cut / fill and finish grading of approximately 25,000 cubic yards of existing soils to configure five separate 2-acre RI basins, to a depth of three feet.
- Flow control valves.
- Pump station to convey effluent from WWTP to RI basins.

In addition to the general components listed above for an RI basin system, a constructed wetland system would also require the installation of wetland plant species throughout the 10-acre area.

Table 7-26 provides a summary of the results of the preliminary feasibility assessment of potential indirect discharge sites.

Table 7-26: Indirect Discharge System Capacity Summary

Area – Location	Type of Indirect Discharge System	Estimated Infiltration Capacity
Area 1 – Immediately northwest of WWTP	Exfiltration pipes along banks of South Umpqua River and Canyon Creek	0.30 MGD
Area 2 – East of WWTP	Exfiltration pipes along bank of South Umpqua River	1.2 MGD
Area 3 – North Side of South Umpqua River	Rapid infiltration basins or constructed wetland	1.0 MGD

Indirect Discharge Design Criteria. The comparison of effluent discharge and reuse alternatives will be based on a system with the following major components:

- Effluent pump station
- Exfiltration gallery indirect discharge system constructed in Areas 1 and 2 as described above, with a capacity of 1.5 MGD
- Monitoring well system
- Inclusion of phosphorus and ammonia removal facilities at the WWTP may not be necessary depending on soil characteristics
- Bankside river outfall for use during high flow events (effluent flows greater than 1.5 MGD)

Area 3 is not included due to concerns about the cost of a river crossing, potential impacts from flooding, and uncertain land availability. Design data for the indirect discharge system are included in Table 7-27, while a simplified schematic diagrams is shown on Figure 7-20.

Table 7-27: Indirect Discharge Design Data

Item	Value
Effluent pump station	
Pump type	Submersible
Pump number	3
Capacity, each, gpm	750
Exfiltration system	
Trench and pipe length, feet	1,550
Pipe diameter, inches	8
Monitoring wells	
Number	10
Bankside outfall	
Number	1
Diameter, inches	12

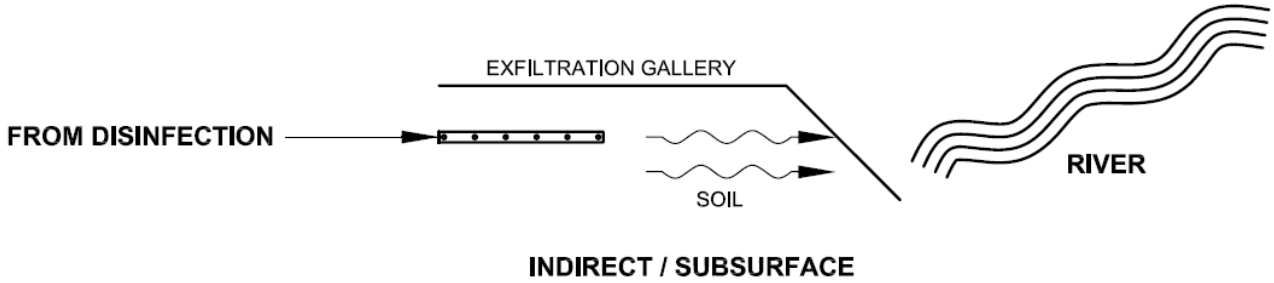


Figure 7-20: Alternative E2 Indirect Discharge Schematic Diagram

Alternative E3. Dry Weather Reuse, Wet Weather River Discharge. Under Alternative E3, a recycled water program would be developed and implemented. This alternative consists of dry weather agricultural reuse along with continued wet weather river discharge. The following major components would be required:

- Outfall and diffuser for wet weather river discharge
- Recycled water pump station
- Recycled water transmission line
- River pipeline crossing to agricultural land north of WWTP
- Off-site, lined, recycled water storage reservoir
- Irrigation equipment
- Agricultural land

Because the WWTP is equipped with a filtration system, the City could produce either Class A or Class C recycled water. However, it should be noted that if the two classes of water are mixed together (such as in the storage reservoir) the resultant mixture would be Class C recycled water. Producing Class A recycled water would increase the number of usage options available.

A water balance was developed to estimate land and storage requirements for projected 2035 design flows. Three different scenarios were evaluated to obtain estimates of land required for agricultural crop irrigation and recycled water storage:

1. Dry weather reuse (June through September) with no recycled water storage.

2. Dry weather reuse (June through August) with storage capacity for 2 months (September and October).

3. Dry weather reuse (June through September) with storage capacity for 1 month (October).

The following environmental information and assumptions were used in the development of the water balance:

- Average precipitation and temperature data over 30 years (1971-2000) for the Riddle 2 NNE weather station.
- The Blaney-Criddle evapotranspiration methodology, which incorporates precipitation, temperature, crop coefficient, site latitude and percent daylight.

- The crop coefficient for pasture grass developed by the U.S. Department of Agriculture, Soil Conservation Service – Part 623 National Engineering Handbook, Chapter 2: Irrigation Water Requirements.
- Assumptions used in the water balance included an irrigation efficiency factor of 90 percent.
- A land application area safety factor of 44 percent based on plant ADWF conditions and pasture grass irrigation.
- A storage volume safety factor of 15 percent based on plant ADWF conditions.

Conformance with both water quality standards and irrigation demands must be considered simultaneously in order to develop an effective, efficient recycled water program. Based on average precipitation and evapotranspiration data, irrigation is not feasible during the month of October (precipitation exceeds evapotranspiration). Because river flows are typically low in October (7Q10 flow is 38 cfs), it will not be possible to comply with anticipated ammonia and phosphorus limits without the treatment facility upgrades needed for year-round discharge. However, despite the fact that it is classified as a dry weather month, continued river discharge should be possible in May (7Q10 flow of 294 cfs) as effluent phosphorus limits would be approximately 2 mg/L and the available dilution would preclude the need for ammonia removal.

Consequently, it is anticipated that effluent would continue to be discharged to the river in May, while effluent would be retained in a lined storage reservoir in October. Table 7-28 summarizes the results of the water balance calculation for the three reuse scenarios.

Table 7-28: Water Balance Summary

Scenario	2035 Flow (MGD)	Irrigation Season	Pasture land, acres [(a)]	Storage volume, MG[(a)]
1) Dry weather reuse, no storage	ADWF = 0.27	June-September	190	None
2) Dry weather reuse, 2 months storage	ADWF = 0.27	September through October storage, June-August reuse	130	19
3) Dry weather reuse, 1 month storage	ADWF = 0.27	October storage, June-September reuse	190	10

Note:

(a) Based on ADWF conditions and includes 44 percent land area safety factor to account for buffer land and higher-than-normal plant flow and rainfall conditions and a 15 percent storage volume safety factor.

Scenario 1 is presented for comparison purposes only as it would not provide compliance with water quality standards in October and November unless phosphorus and ammonia removal facilities were constructed at the WWTP. Both Scenario 2 and 3 would result in compliance. The tradeoff between these scenarios is essentially irrigation

land area versus storage volume. Costs will be based on Scenario 3. Design data for this alternative are provided in Table 7-29. A simplified schematic diagram is shown on Figure 7-21.

Table 7-29: Dry Weather Reuse, Wet Weather Discharge Design Data

Item	Value[(a)]
Outfall	
Diameter, inches	18
Length, feet	100
Diffuser length, feet	30
Diffuser ports, number	3
Recycled water pump station	
Pump type	Submersible
Pump number	3
Capacity, each, gpm	400
Recycled water transmission main	
Diameter, inches	12
Length, feet	2,500 (a)
Recycled water storage reservoir	
Construction	Earthen
Volume, Mgal	19
Liner type	Membrane
Depth, feet	10
Surface area, acres	7
Irrigation pump station	
Pump type	Vertical turbine
Pump number	2
Capacity, each, gpm	1,000
Irrigation water transmission main	
Diameter, inches	12
Length, feet	5,000 (a)
Agricultural land area, acres	190
Irrigation system type	Site dependent

Note:

(a) Key facility locations are unknown. Assumed value used for the purposes of cost estimating.

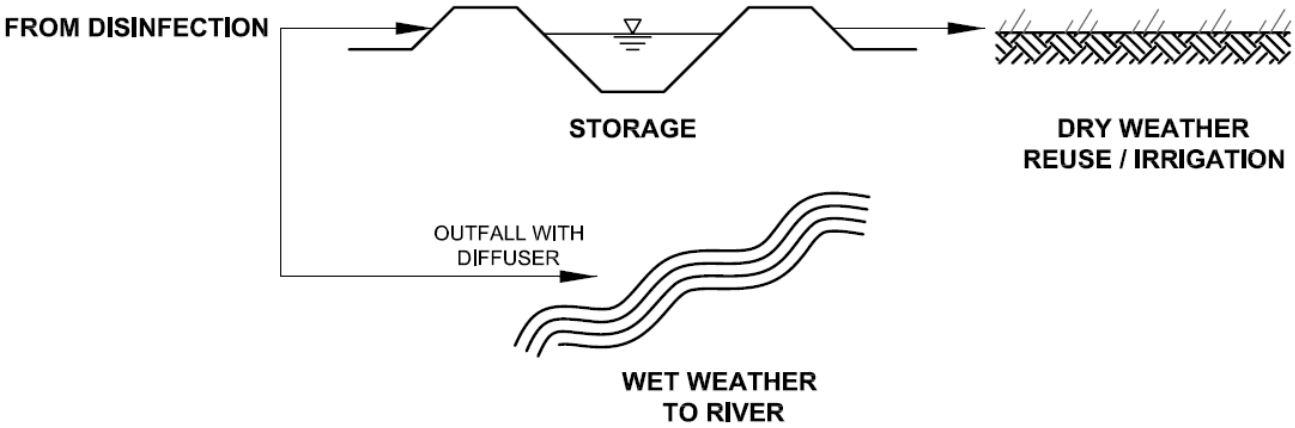


Figure 7-21: Alternative E3 Dry Weather Reuse, Wet Weather Discharge Schematic Diagram

Alternative E4. Dry Weather Recycled Water to Tribe, Wet Weather River Discharge. The City may have an opportunity to collaborate with the Cow Creek Band of Umpqua Tribe of Indians (Tribe) on the development of a joint effluent reuse project. The City would provide recycled water to the Tribe during the dry weather season, and continue to discharge its effluent to the river during the wet weather season. The major aspects of the City's portion of the project would include:

- Upgrades to the WWTP necessary to accommodate growth
- Upgrades to the WWTP necessary to continue to produce effluent that complies with current NPDES permit requirements
- A new outfall for continued discharge during the wet weather season
- A pump station and pipeline to convey recycled water to the Tribe during the dry weather season
- Facilities to meet DEQ reliability and redundancy requirements. A storage tank capable of storing one week's worth of effluent is assumed for the purposes of this evaluation.

From the City's perspective, the key to the success of such an arrangement is that the Tribe must reliably take the City's effluent during times when the WWTP cannot comply with water quality standards. Given the requirements of the TMDL and anticipated ammonia toxicity issues, the Tribe would need to take all of the City's effluent from June 1 through October 31 to assure compliance without major plant performance upgrades. Likewise, from the Tribe's perspective, the recycled water must reliably meet all quality standards required by the EPA for the varied uses of the water. Design data for this alternative are provided in Table 7-30. A simplified schematic diagram is shown on Figure 7-22.

Table 7-30: Dry Weather Recycled Water to Tribe, Wet Weather River Discharge

Item	Value[(a)]
Outfall	
Diameter, inches	18
Length, feet	100
Diffuser length, feet	30
Diffuser ports, number	3
Recycled water pump station	
Pump type	Split case
Pump number	2
Capacity, each, gpm	300
Recycled water transmission main	
Diameter, inches	12
Length, feet	180
Recycled water storage tank[(a)]	
Volume, Mgal	1.9
Depth, feet	18
Diameter, ft	135

Note:

(a) Assumes 1 week of storage at design year ADWF

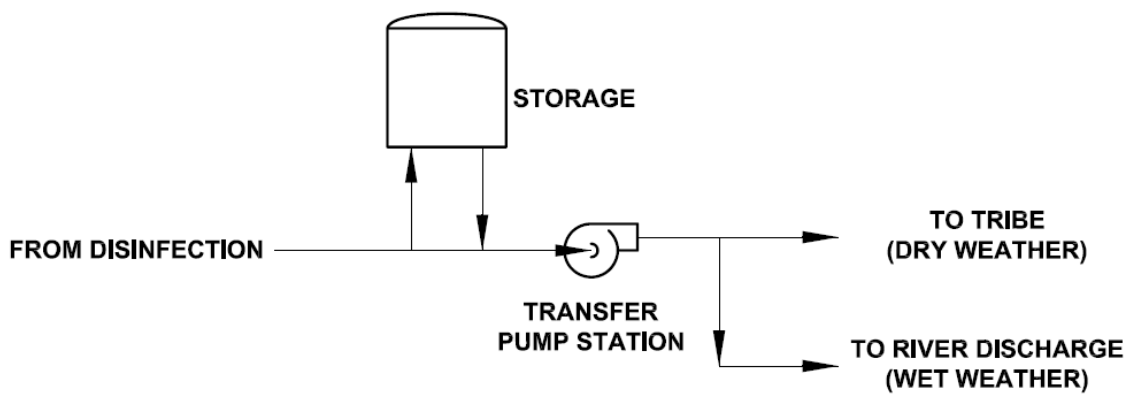


Figure 7-22: Alternative E4 Dry Weather Recycled Water to Tribe, Wet Weather River Discharge Schematic Diagram

Comparison of Effluent Reuse and Discharge Alternatives. Estimated capital costs for the four effluent discharge and reuse alternatives are summarized in Table 7-31. A comparison of non-economic factors is provided in Table 7-32.

Table 7-31: Capital Cost Comparison of Effluent Discharge and Reuse Alternatives

	E1 - Year Round River Discharge	E2 - Indirect Discharge	E3 - Dry Weather Reuse, Wet Weather River Discharge	E4 – Dry Weather Recycled Water to Tribe, Wet Weather River Discharge
Capital Costs[(a)]	$500,000	$3,000,000	$4,900,000	$4,400,000

Note:

(a) a Capital costs include the following multipliers: general conditions, 15 percent; contractor overhead and profit, 15 percent; contingency, 30 percent; engineering, legal, and administration, 25 percent,escalation to mid-point of construction, 3.8 percent

Table 7-32: Non-economic Comparison of Effluent Discharge and Reuse Alternatives

Evaluation Criteria	Alternative			
	E1. Year-round river Discharge	**E2. Indirect Discharge**	**E3. Dry Weather Reuse, Wet Weather Discharge**	**E4. Dry Weather Recycled Water to Tribe, Wet Weather River Discharge**
Performance	Diffusers provide consistent mixing with river water	Enhanced mixing and potential effluent polishing compared to outfall diffuser. Performance depends on soil characteristics	WWTP can comply with Class C recycled water standards. Storage reservoir and reuse prevent river discharge during critical conditions	Existing WWTP complies with current permit limits, which would govern during dry weather
Expandability	Outfall can be oversized and design to accommodate larger discharge ports	System would be difficult to expand without constructing a river crossing	Expandability potentially limited by land availability	Expandability dictated by Tribe water needs
Ease of Operation	Essentially no operator attention required	Groundwater quality monitoring and effluent pumping would be required	Replaces advanced treatment with pumping and irrigation systems	Tribe assumes responsibility for operating portions of system
Reliability	No additional equipment is required. A properly designed diffuser can have a service life of several decades.	Potential exfiltration pipe clogging concerns	Large storage reservoir provides excellent reliability as it can back up treatment facilities during wet weather	Reliability of Tribe's water demand unknown. Termination or expiration of Reuse Agreement would require City to construct additional facilities.
O&M	Outfall diffuser has	Effluent pump	Relatively high labor	Tribe assumes

Evaluation Criteria	Alternative			
	E1. Year-round river Discharge	**E2. Indirect Discharge**	**E3. Dry Weather Reuse, Wet Weather Discharge**	**E4. Dry Weather Recycled Water to Tribe, Wet Weather River Discharge**
Considerations	minimal O&M requirements	station increases O&M requirements. Periodic cleaning of exfiltration pipe may be required	associated with irrigation system operation. However, potential for contract operations of agricultural portion	responsibility for operating irrigation system
Constructability	Obtaining permits for in-river work is difficult and time consuming	Numerous current unknowns could affect constructability: bedrock, groundwater elevation, \soils	Permits for in-river outfall installation and pipeline river crossing required	Pipeline creek crossing required.
Odor	No odors	Minimal odors	Potential odors from storage reservoir	No odors from City facilities
Flexibility	Reuse could be implemented in the future	Reuse could be implemented in the future	Reservoir provides significant discharge management flexibility	Minimal flexibility. Essentially no short-term options available if Tribe discontinues accepting recycled water
Complexity	Least complex alternative	Relatively simple	Requires management of storage reservoir and agricultural site	Relatively simple facilities; however, Reuse Agreement could complicate wastewater management
Energy use	No additional energy use	Energy required for pumping to exfiltration galleries	Energy required to pump to reservoir and to pressurize irrigation piping network	Energy required to pump to Tribe system
Track Record	Thousands of WWTPs have outfall diffusers	Relatively new technology with only a handful in service	Many cities irrigate recycled water during the dry weather season	Many cities have reuse agreements with other entities
Control	City retains control over effluent. All facilities are on City-owned land	City retains control over effluent.	City would retain control over effluent if City purchases agricultural land. Control reduced	Tribe has control over the use of recycled water. Effluent reuse contract would place additional restrictions on

Evaluation Criteria	Alternative			
	E1. Year-round river Discharge	**E2. Indirect Discharge**	**E3. Dry Weather Reuse, Wet Weather Discharge**	**E4. Dry Weather Recycled Water to Tribe, Wet Weather River Discharge**
				effluent quality.
Regulatory Compliance	Would be subject to all future, more stringent water quality standards	Could make the City exempt from some future water quality regulations	The City would be exempt from future dry weather water quality regulations	If DEQ doesn't approve Reuse Agreement conditions, they would require City to construct facilities that can comply with water quality standards
Impacts to Other Processes	Would necessitate construction of phosphorus and ammonia removal facilities	Depending on site and soil characteristics, in-plant phosphorus and ammonia removal facilities may not be required.	Eliminates the need for dry weather ammonia and phosphorus removal	Eliminates the need for dry weather ammonia and phosphorus removal
Implementation	Permitting and in-river work are biggest implementation issues	Detailed hydrogeological study required to determine whether indirect discharge is feasible	Land availability and public acceptability concerns	Minimum land requirements for WWTP expansion

BIOSOLIDS MANAGEMENT ALTERNATIVES

As discussed in Chapter 4, the existing biosolids treatment system at the WWTP is undersized for current and future flows, including an inadequate capacity for sludge storage. As a result, three biosolids management alternatives are developed and evaluated in this section. All three alternatives are compatible with the liquid treatment alternatives and wastewater management strategies developed earlier in this chapter.

As discussed in Chapter 6, management of biosolids that are produced in the wastewater treatment process are regulated by DEQ regulations OAR 340-050. DEQ's biosolids regulations were developed in 1995 and reference the Federal U.S. Environmental Protection Agency's Title 40 of the Code of Regulation, Part 503 (commonly referred to as 40 CFR Part 503). The numerical limits, management practices, recordkeeping, and monitoring requirements established in the state and federal regulations determine the requirements at the WWTP.

A biosolids product is characterized by its quality (Class A, EQ or B) which is the result of the solids processing technology used for its production. The WWTP currently

produces Class B biosolids, which is suitable for restricted land application or landfill disposal. Transitioning to Class A biosolids is typically expensive for smaller treatment plants, and may be unnecessary unless a market for high-quality Class A biosolids has been identified. The City already has customers for its Class B biosolids, therefore management strategies included in this chapter are based on continued treatment to Class B standards.

Biosolids are hauled to local farmers' fields when weather conditions permit application. The ability to find and maintain suitable land application sites is dependent on public acceptance of the final product. All biosolids intended for land application must meet three basic requirements:

Pollutant Limits. Pollutant limits for 9 metals cannot be exceeded. Table 3 of Part 503.13 specifies the low pollutant concentration limits for these 9 metals. If the City's biosolids meet these concentration limits as well as the requirements for Class A biosolids they are considered to be Exceptional Quality (EQ), and pollutant tracking is done in accordance with Tables 2 and 4 in Part 503.13. Pollutant concentration ceiling limits are established in Table 1 of Part 503.13. This table represents the maximum concentration limits allowed for biosolids. If biosolids exceed these limits they are deemed unsuitable for beneficial use.

Pathogen Limits. Pathogens are disease-causing organisms that are present in sewage sludge. Pathogen reduction requirements set forth in the Part 503 regulations must be met along with corresponding access restrictions to the biosolids application site.

Vector Attraction Reduction Requirements. Vectors are organisms such as rodents or insects that can spread disease by carrying and transferring pathogens. Sludge treatment processes must utilize one of the 10 vector attraction reduction methods that are required by OAR 340-050 and the Part 503 regulations. This requirement may also be met through appropriate testing.

Biosolids at the WWTP must be treated to reduce the density of pathogenic organisms. Class A treatment processes essentially eliminate pathogens. Class B treatment processes reduce, but do not entirely eliminate pathogens. Processes that meet Class B requirements include alkaline stabilization, anaerobic digestion, aerobic digestion, and composting. Alkaline stabilization is currently used for sludge treatment at the WWTP and will be discussed later in this chapter. Anaerobic digestion would require that new concrete tanks, sludge mixing equipment, and sludge heating equipment be constructed, which is expensive. The WWTP was originally designed to stabilize sludge using aerobic digestion; however, the original aerobic digester is significantly undersized, providing a hydraulic detention time of only 4 days at current solids loads, much less than the 60 days required by Part 503 rules. Composting would require a footprint exceeding that of the existing WWTP site.

Future sludge quantities will primarily be a function of population growth; however, the additional sludge produced by chemical precipitation of phosphorus (as discussed in the "Tertiary Treatment" section above) will also increase sludge quantities. Current and future sludge quantities are shown in Table 7-33.

Table 7-33: Summary of Sludge Quantities

Parameter	Year 2008	Design Year 2035
Waste Activated Sludge Solids (lbs/day)	800	1280
Solids from Phosphorus Removal (lbs/day)	0	260
Total Solids (lbs/day)	800	1560
Average Solids Concentration of WAS (mg/L)	5200	5200
Sludge Volume (gal/day)	18,700	35,800

Alternatives

Three biosolids treatment alternatives, all associated with alkaline stabilization using lime, are developed and evaluated in this section. Each of the biosolids alternatives were based on the City continuing the current process of biosolids beneficial reuse by land application. This method is appropriate for a rural community such as Canyonville, which is in close proximity to agricultural land.

The methods of lime stabilization commonly used include (1) addition of lime to sludge prior to dewatering, termed “lime pretreatment,” (2) the addition of lime to sludge after dewatering, or “lime posttreatment,” and (3) lime stabilization without dewatering. In the lime stabilization process, lime is added to untreated sludge in sufficient quantity to raise the pH to 12 or higher for a minimum of 2 hours, and a pH of 11.5 or higher for the following 22 hours.

Alternative B1. Lime Pretreatment. Pretreatment involves treating liquid sludge with lime prior to dewatering. The sludge is mixed with an acceptable amount of liquid lime slurry, which is then stored for a period of 24 hours to allow for pathogen deactivation. The deactivated sludge is then pumped to equipment for dewatering to 15 percent solids concentration.

Lime pretreatment of liquid waste activated sludge requires approximately 1.5 times more lime than lime posttreatment. The higher lime dose is needed to attain the required pH because of the high chemical demand of the liquid. Overdosing of lime provides enough residual alkalinity to ensure that the pH does not drop below 11 for several days. Lime pretreatment dosage varies with type of sludge and solids concentration, but is generally in the range of 750 to 1050 pounds of lime per dry ton of dry solids (lb $Ca(OH)_2$/ton dry solids).

Dewatering associated with lime pretreatment is typically accomplished using a pressure-type dewatering device, such as a screw press. Centrifuges and belt filter presses are typically not used in this situation due to abrasive wear and scaling issues. In this alternative, the existing lime stabilization system would be supplemented with additional lime stabilization tanks, lime slurry tanks, dosing equipment, sludge transfer pumps, and an expanded containment slab. Dewatering equipment is in the form of a screw press, which would be installed in a separate reinforced concrete building to protect from inclement weather and contain odors. Odors would be treated with an external odor scrubber. Design data for Alternative B-1 is included in Table 7-34.

Table 7-34: Lime Pretreatment Design Data

Item	Value
Lime Quantity Required (design), ppd	690
Lime Stabilization Tanks	
Type	Vertical HDPE Storage Tank
Number	5 (3 exist./2 new)
Diameter, feet	12
Volume, each, gal	6,000
Lime Slurry Mixing Tanks	
Type	Vertical HDPE Storage Tank
Number	2 (1 exist./1 new)
Diameter, feet	4
Volume, each, gal	120/200
Sludge Transfer Pumps	
Pump type	Self-priming centrifugal
Pump number	2 (1 exist/1 new)
Capacity, each, gpm	275
Sludge Truck/Lime Slurry Pump	
Pump type	Progressive cavity
Pump number	2
Capacity, each, gpm	25
Dewatering System	
Type	Screw press
Number	1
Capacity, gpm	25

During dry weather periods in which land is available for application of liquid stabilized biosolids, the dewatering process can be bypassed. During wet weather, this alternative allows for storage or landfill disposal of dewatered biosolids. It should be noted that landfill disposal does not require lime addition to a compliance with Class B standards. A schematic diagram of the alternative is included as Figure 7-23.

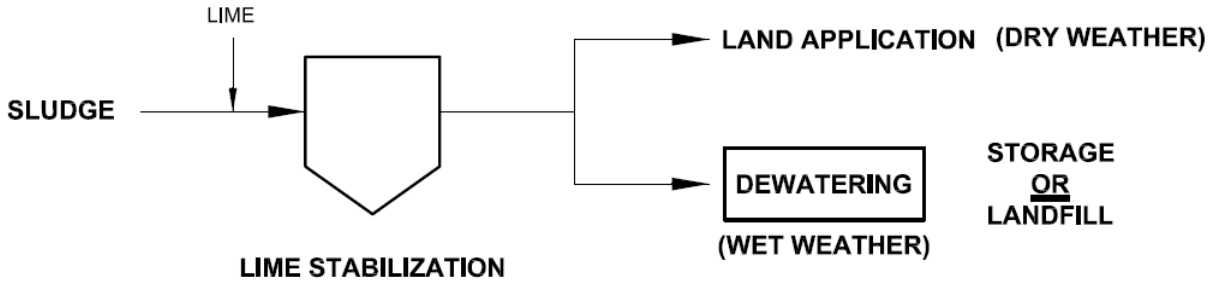


Figure 7-23: Alternative B1 Lime Pretreatment Schematic Diagram

Alternative B2. Lime Posttreatment. Post-treatment involves dewatering liquid sludge prior to treatment with lime. The waste activated sludge would be stored temporarily in the existing sludge holding tank before being pumped to equipment for dewatering to approximately 15 percent solids content. Dry lime would then be mixed with the dewatered sludge in a screw conveyor to raise the pH of the mixture to 12 or higher. The use of dry lime results in an exothermic reaction when mixed with sludge, which improves pathogen deactivation. The screw conveyor provides mixing of the lime with the dewatered sludge. Pathogen deactivation occurs rapidly, resulting solids concentrations are typically in the range of 15 to 20 percent solids. Dewatered, stabilized solids are transferred via a piston pump to storage/disposal.

Lime posttreatment dosage varies with type of sludge and solids concentration, but is generally in the range of 500 to 700 lb $Ca(OH)_2$/ton dry solids. The resulting conditioned sludge has a dry, crumbly texture which allows for long-term storage or land application with a conventional spreader. Dewatering equipment would be installed in a new reinforced concrete building to protect from inclement weather and contain odors. Odors would be treated with an external odor scrubber. Design data for Alternative B2 is included in Table 7-35.

Table 7-35: Lime Posttreatment Design Data

Item	Value
Sludge Transfer Pumps	
Pump type	Self-priming centrifugal
Pump number	2
Capacity, each, gpm	75
Dewatering System	
Type	Belt Filter Press
Number	1
Capacity, gpm	75
Lime Quantity Required (design), ppd	460
Dry Lime Storage Tank	
Type	Storage Tank
Number	1
Diameter, feet	3
Volume, each, gal	120
Screw Conveyor	
Quantity	1
Diameter, inches	12
Horsepower	5
Sludge Truck/Lime Slurry Pump	
Pump type	Piston pump
Pump number	2
Capacity, each, gpm	25

This alternative provides sufficient flexibility to allow for land application, storage, or landfill disposal of the stabilized biosolids, depending upon seasonal demand from local land owners. Dewatered biosolids can typically be distributed onto agricultural land using standard disking equipment. Lime addition is not required for landfill disposal.
A schematic diagram of the alternative is included as Figure 7-24:

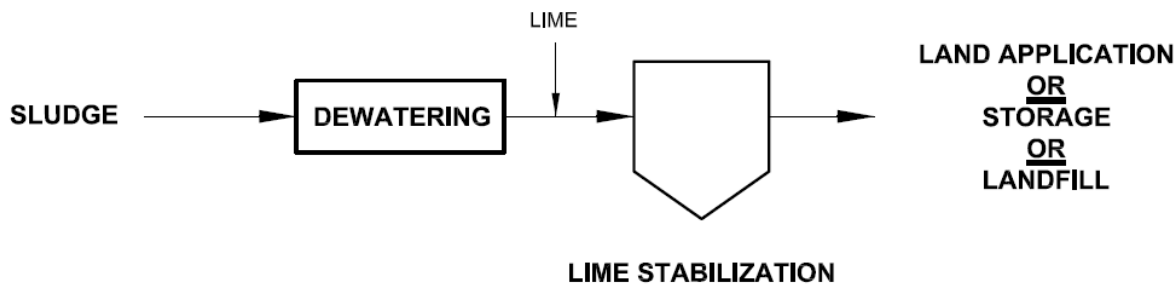


Figure 7-24: Alternative B2 Lime Posttreatment Schematic Diagram

Alternative B3. Lime Stabilization Only. This alternative is consistent with the method currently practiced by the WWTP for sludge stabilization. As opposed to the other two alternatives, Alternative B3 does not include provisions for dewatering lime stabilized biosolids. Waste activated sludge is mixed with an acceptable amount of liquid lime slurry and then transferred to one of the existing sludge storage tanks for storage and decanting. After decanting, the deactivated sludge typically has a solids concentration of 2.3 percent solids.

Similar to pretreatment, lime stabilization requires approximately 1.5 times more lime than lime posttreatment (750 to 1050 lb $Ca(OH)_2$/ton dry solids).

This alternative consists of expanding the existing lime stabilization system, including adding two additional lime stabilization tanks, an additional lime slurry tank, a large sludge storage tank, and redundant sludge transfer and slurry pumps. New tanks and equipment would be installed on an expanded concrete containment slab. During wet weather months when the City is historically unable to access approved application sites, the stabilized biosolids would need to be stored onsite before applying to land in the spring. A 2.0 million gallon reinforced concrete storage tank would provide 7-months of storage, which may be needed during wet years. Design data for Alternative B3 is included in Table 7-36.

Table 7-36: Lime Stabilization Only Design Data

Item	Value
Lime Quantity Required (design), ppd	690
Lime Stabilization Tanks	
Type	Vertical HDPE Storage Tank
Number	5 (3 exist./2 new)
Diameter, feet	12
Volume, each, gal	6,000
Lime Slurry Mixing Tanks	
Type	Vertical HDPE Storage Tank
Number	2 (1 exist./1 new)

Item	Value
Diameter, feet	4
Volume, each, gal	120/200
Sludge Transfer Pumps	
Pump type	Self-priming centrifugal
Pump number	2 (1 exist/1 new)
Capacity, each, gpm	275
Sludge Storage Tank	
Type	Reinforced Concrete
Capacity	2.0 MG
Sludge Truck/Lime Slurry Pump	
Pump type	Self-priming centrifugal
Pump number	1 (exist.)
Capacity, each, gpm	200

Since dewatering is not included in this alternative, biosolids reuse and disposal options must adhere to current practices. Biosolids would be land applied during dry weather and stored in liquid form during wet weather. A schematic diagram of the alternative is included as Figure 7-25.

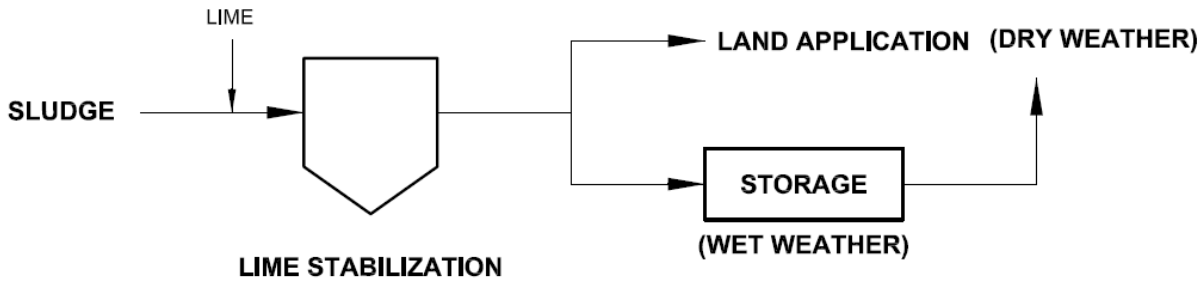


Figure 7-25 Alternative B3 Lime Stabilization Only Schematic Diagram

Comparison of Biosolids Management Alternatives. Estimated capital costs for the three biosolids management alternatives are summarized in Table 7-37. Anticipated O&M costs and present worth are compared in Tables 7-38 and 7-39, respectively.

Table 7-37: Capital Cost Comparison of Biosolids Management Alternatives

	B1 Lime Pretreatment	B2 Lime Posttreatment	B3 Lime Stabilization Only
Capital Costs[(a)]	$2,400,000	$2,000,000	$3,900,000

Notes:

(a) Capital costs include the following multipliers: general conditions, 15 percent; contractor overhead and profit, 15 percent; engineering, legal, and administration, 25 percent, contingency, 30 percent; escalation to mid-point of construction, 3.8 percent

Table 7-38: O&M Cost Comparison of Biosolids Management Alternatives

	B1 Lime Pretreatment	B2 Lime Posttreatment	B3 Lime Stabilization Only
Annual O&M costs	$100,000	$100,000	$80,000

Table 7-39: Present Worth Comparison of Biosolids Management Alternatives

	B1 Lime Pretreatment	B2 Lime Posttreatment	B3 Lime Stabilization Only
Capital Costs	$2,400,000	$2,000,000	$3,900,000
20 year O&M Costs	$1,900,000	$2,000,000	$1,600,000
Present Worth	$4,300,000	$4,000,000	$5,500,000

Lime pretreatment allows the WWTP to continue providing liquid biosolids to local land owners (if preferred) during the dry weather period, so continuous dewatering is not necessary as in the posttreatment alternative. Lime posttreatment has several advantages when compared to pretreatment, including (1) dry lime can be used without need for a wet slurry, so no additional water is added to the sludge; and (2) there are no special requirements for dewatering (scaling of equipment is not an issue). A potential disadvantage of posttreatment is the release of odorous gases, particularly amines, due to the exposed dewatering action of the belt filter press. Lime postreatment is also not a common practice in Oregon, which could be a concern from a regulatory perspective. DEQ has indicated that that are uncomfortable with a recommendation to proceed with Lime posttreatment. Alternative B3 Lime stabilization only would require the least amount of additional equipment; however, reuse and disposal options are limited due to the liquid state of the biosolids. A non-economic comparison is summarized in Table 7-40.

Table 7-40: Non-economic Comparison of Biosolids Management Alternatives

Evaluation criteria	B1. Lime Pretreatment	B2. Lime Posttreatment	B3. Lime Stabilization Only
Performance	Produces a consistent product, but with higher lime requirement than posttreatment	Requires least amount of lime of all alternatives	Produces a consistent product, but lower solids concentration
Expandability	Lime system is modular and screw press operating hours can be extended	Lime system is modular and belt filter press operating hours can be extended	Lime system expandable with additional tanks and piping
Ease of Operation	Requires monitoring of screw press, but less adjustments than typical of belt filter press	Requires monitoring of belt filter press and periodic adjustments	Plant staff familiar with technology
Reliability	Screw press typically	Belt filter press and	Fewest moving parts of

Evaluation criteria	B1. Lime Pretreatment	B2. Lime Posttreatment	B3. Lime Stabilization Only
	more reliable than belt filter press	screw conveyor can have bearing issues	alternatives
O&M Considerations	Parts for screw press may be difficult to obtain	Least number of pumps required for system operation	Requires storage and conveyance of liquid sludge, which creates higher potential for pipe and pump clogging
Constructability	Straightforward addition to existing system – shutdowns limited	More involved construction due to larger equipment and conveyance of drier sludge	Straightforward addition to existing system – shutdowns limited
Odor	Odors anticipated to be minimal	Highest potential for odors due to exposed sludge at press	Odors anticipated to be minimal
Flexibility	Year-round options for disposal	Year-round options for land application, storage, or landfill	Seasonally limited disposal options
Complexity	More complex than existing system, requiring additional operator attention	More complex than existing, requiring additional operator attention	Similar to existing
Energy use	Similar to B2	Similar to B1	Lowest energy use
Track record	Screw presses have been in operation for 10+ years	Not proven in Oregon	Common technology for other small WWTPs
Regulatory compliance	Should regularly comply with EPA Class B biosolids requirements	Should regularly comply with EPA Class B biosolids requirements. Unfamiliar process may not be acceptable to DEQ	Should regularly comply with EPA Class B biosolids requirements
Impacts to other processes	High pH filtrate from screw press may impact biological system	Sludge dewatering prior to lime results in a more-neutral pH waste stream	Similar to existing system
Implementation	Space required for dewatering building; may be able to fit on existing plant site	Space required for dewatering building; may be able to fit on existing plant site	Space near existing lime stabilization system for expansion. Space required for large storage tank

PLANT REPAIRS AND SUPPORT FACILITIES

Often overlooked in the planning process, support facilities are crucial to the successful operation of any treatment plant. In addition, many older treatment plants are in need of repairs to ensure that the useful life of structures and treatment facilities is reached. These improvements are described in this section.

- Structural repairs. There are signs of differential settling at several of the WWTP's existing structures. Structural repairs and foundation rehabilitation is recommended.

- Electrical service. The WWTP's existing electrical service may not have adequate capacity to supply new and future demands. In addition, there are often code compliance issues associated with older services. A new transformer, main breaker, and switchboard are included in the cost estimate.
- Standby generator. The existing standby generator does not have adequate capacity to power all new and existing facilities. A new engine-generator will be provided.
- Non-potable water pumps. Significant long-term savings can be realized by using plant effluent for sprays, hose stations, and other non-potable needs rather than City potable water. The non-potable water pumps would be installed downstream of disinfection facilities.
- Distributed control system (DCS). Most newer WWTPs are equipped with computerized monitoring and control systems which enhance reliability and reduce labor requirements. Some of the existing equipment at the WWTP is currently controlled by a programmable logic controller (PLC). A DCS would build upon this system.
- Site improvements. This category of improvements includes paving, landscaping, and outside lighting.

ECONOMIC COMPARISON OF WASTEWATER MANAGEMENT STRATEGIES

A non-economic comparison of wastewater management strategies was presented as part of the initial screening process earlier in this chapter. This section presents an economic comparison generally using the lowest cost alternatives, identified above, as building blocks. The one exception is the biosolids management. Due to questions regarding DEQ approval of Alternative B2 Lime Posttreatment, costs are based on Alternative B1. Tables 7-41 through 7-43 summarize the economic comparison.

Table 7-41: Capital Cost Comparison of Wastewater Management Strategies

Item	Wastewater Management Strategy			
	MS1. Year-Round Discharge Cost, $1000	**MS3. Indirect Discharge Cost, $1000**	**MS4. Dry Weather Reuse, Wet Weather Discharge Cost, $1000**	**MS7. Dry Weather Recycled Water to Tribe, Wet Weather River Discharge, $1000**
Headworks	$2,100	$2,100	$2,100	$2,100
Secondary Treatment	$4,200	$4,200	$3,400	$3,400
Secondary Effluent Pumping	$80	$80	$80	$80
Tertiary Filtration	$1,900	$1,900	$0	$0
Disinfection	$900	$900	$1,900	$1,900
Effluent Reuse and Discharge	$500	$3,000	$4,900	$4,400
Biosolids Management	$2,400	$2,400	$2,400	$2,400
Structural Repairs	$335	$335	$335	$335

Item	Wastewater Management Strategy			
	MS1. Year-Round Discharge Cost, $1000	MS3. Indirect Discharge Cost, $1000	MS4. Dry Weather Reuse, Wet Weather Discharge Cost, $1000	MS7. Dry Weather Recycled Water to Tribe, Wet Weather River Discharge, $1000
Electrical Service	$223	$223	$223	$223
Standby Generator	$335	$335	$335	$335
Distributed Control System	$223	$223	$223	$223
Site Improvements	$800	$800	$800	$800
Operations Building	$669	$669	$669	$669
Land Acquisition	$200	$200	$1,450	$200
Total	$14,900	$17,400	$18,800	$17,100

Table 7-42: O&M Cost Comparison of Wastewater Management Strategies

Item	MS1. Year-Round Discharge, $/year	MS3. Indirect Discharge, $/year	MS4. Dry Weather Reuse, Wet Weather Discharge, $/year	MS7. Dry Weather Recycled Water to Tribe, Wet Weather River Discharge, $/year
Energy	$32,000	$33,000	$33,000	$35,000
Labor	$297,000	$304,000	$326,000	$301,000
Chemicals, services, and materials	$156,000	$156,000	$169,000	$169,000
Total O&M cost	$485,000	$493,000	$528,000	$505,000

Table 7-43: Present Worth Comparison of Wastewater Management Strategies

Item	Wastewater Management Strategy			
	MS1. Year-Round Discharge Cost, $1000	MS3. Indirect Discharge Cost, $1000	MS4. Dry Weather Reuse, Wet Weather Discharge Cost, $1000	MS7. Dry Weather Recycled Water to Tribe, Wet Weather River Discharge, $1000
Total Capital Costs (rounded)	$14,900	$17,400	$18,800	$17,100
Present worth of O&M costs	$6,100	$6,200	$6,700	$6,400
Present Worth	$21,000	$23,600	$25,500	$23,500

CHAPTER 8

RATE STUDY

This chapter estimates user rates based on the preliminary capital improvement plan (CIP) in Chapter 9, the population growth projections in Chapter 3, and the City's existing rate structure.

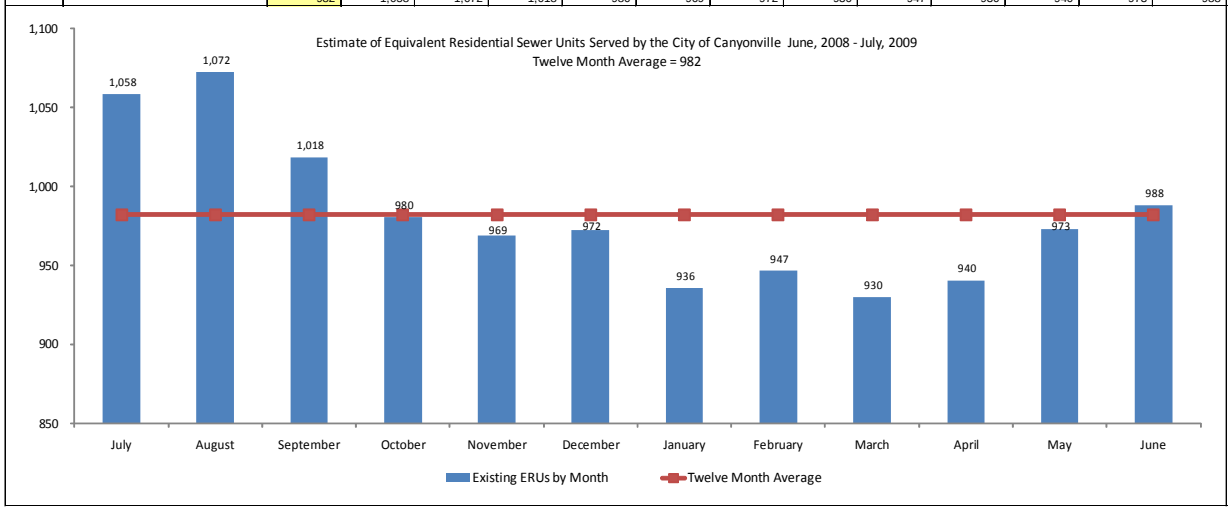
DEMOGRAPHICS AND USER PROFILE

City utility billing records for the period July, 2008 through June, 2009 show an average of 556 active sewer accounts. Over this one year time period, these active accounts generated billed revenues of $350,784. For modeling and forecasting purposes, the project team normalized this observed billed data into industry standard measures called Equivalent Residential Units (ERUs). The current estimate of total ERUs served by the City's sewer utility, by customer class, is shown in Table 8-1.

Table 8-1: Estimate of Current ERUs by Customer Class

City of Canyonville
Derivation of Sewer System Equivalent Residential Units

Account Number	Account Description	FY 08-09 Average	2008 July	August	September	October	November	December	2009 January	February	March	April	May	June
200	Sewer residential - In City	485	493	491	489	487	491	487	483	478	475	481	481	487
2001	2 units 1 sewer	4	4	3	4	4	3	4	4	4	4	4	4	4
201	Sewer 2 units in city	14	14	14	14	14	14	14	14	14	14	14	14	14
202	Sewer 3 units in city	6	8	6	6	6	6	6	6	6	6	6	6	6
203	Sewer 4 units in city	12	12	12	12	12	12	12	12	12	12	12	12	12
204	Sewer 5 units in city	5	5	5	5	5	5	5	5	5	5	5	5	5
206	Sewer 8 units in city	12	11	11	11	12	12	12	12	12	11	11	11	12
207	Sewer residential outside city	2	2	2	2	2	2	2	2	2	2	2	2	2
212	Seer 4 units outside city	7	7	7	7	7	7	7	7	7	7	7	7	7
213	Sewer outside city/no water	30	30	30	30	30	30	30	30	30	30	30	30	30
217	Sewer 11 units outside city	10	17	20	15	8	7	7	7	7	7	7	7	8
218	Sewer 12 units outside city	9	9	10	13	8	8	8	8	8	8	8	8	8
219	Sewer 37 units in city	24	24	24	24	24	24	24	24	24	24	24	24	24
220	Sewer school/church	75	113	120	81	72	69	72	50	65	55	54	83	67
225b	Sewer MHP 36 units inside city	34	33	33	33	33	33	33	33	33	33	33	33	36
226	Sewer 28 units in city	18	18	18	18	18	18	18	18	18	18	18	18	18
227	Sewer restaurant - grease trap	22	26	28	29	26	22	20	20	18	15	21	21	17
229	Sewer carwash/laundry	10	10	11	10	9	9	10	9	11	9	9	9	8
230	Sewer grease trap-outside city	10	23	20	17	10	5	5	5	5	5	6	6	10
2345	Sewer 40 units in city	33	40	45	36	34	28	33	25	25	25	25	28	47
240	Sewer 87 units in city	43	43	43	43	42	43	42	42	42	43	43	42	43
245115	Knoll terrace 115 units	114	111	112	112	112	113	116	115	115	116	116	116	116
250	Sewer restaurant approved	3	3	4	3	3	3	3	3	3	3	3	4	3
291	Sewer 2 units in city/4	2	2	2	2	2	2	2	2	2	2	2	2	2
		982	1,058	1,072	1,018	980	969	972	936	947	930	940	973	988



As Table 8-1 shows, for the fiscal year ended June, 2009, the City served an average of 982 ERUs. For purposes of the rate forecast, ERUs are escalated at 1.75 percent per year over the forecast horizon, consistent with the population growth rate presented in Chapter 3.

CURRENT SEWER SYSTEM COSTS AND REVENUES

The municipal sewer system is operated as an enterprise within the City's budget structure. Accounting and budgeting are done within three dedicated funds.

Wastewater Fund (07)

Referred to as the sewer operating fund, all operating revenues including sewer service charge rate revenues are budgeted and accounted for within this fund. All costs to operate and maintain the sewer system are also budgeted and accounted for in the Wastewater Fund. The principal categories of operating costs are personal services (labor), materials and services, transfers to other funds, and debt service. General operating contingencies and long term debt reserve requirements are also housed within the Wastewater Fund. A snapshot of historical, estimated, and current year budgeted cash flows for the Wastewater Fund is shown in Table 8-2

Table 8-2: Historical, Estimated, and Budgeted Wastewater Fund Cash Flows

City of Canyonville
Analysis of Wastewater Operating Fund (07) Cash Flow

Category/Line Item Description	Actual 2007	Actual 2008	Estimated 2009	Budget 2010
Resources:				
Beginning Fund Balance	360,276	387,247	349,651	308,454
Transfers from Other Funds	-	-	-	-
Revenues	367,610	359,524	332,487	524,400
Total Resources	$ 727,886	$ 746,770	$ 682,138	$ 832,854
Requirements:				
Personal Services:	107,530	147,299	137,117	154,900
Materials & Services	73,561	129,365	110,556	366,850
Capital outlays	100	-	-	-
Transfers to other funds - OUT	110,132	71,290	77,000	81,040
Debt service	49,316	49,166	49,011	56,514
Contingencies/Designations:	46,564	46,564	46,564	173,550
Unappropriated ending fund balance	340,683	303,087	261,890	(0)
Total Requirements	$ 727,886	$ 746,770	$ 682,138	$ 832,854

Wastewater System Development Charge Fund (21)

This is a dedicated fund to account and budget for wastewater System Development Charges (SDCs). Oregon revised statute (ORS) 223 requires the City to account for SDCs separately and to only use SDC receipts on capital costs related to its sewer system. A snapshot of the Wastewater SDC fund is shown in Table 8-3.

Table 8-3: Historical, Estimated, and Budgeted Wastewater SDC Fund Cash Flow

City of Canyonville
Analysis of Wastewater SDC Fund (21) Cash Flow

	Actual		Estimated	Budget
	2007	2008	2009	2010
Resources:				
Beginning Fund Balance	740,080	822,756	878,219	895,312
Sales and Services:				
System Development Charges:				
Reimbursement	18,236	10,290	-	28,000
Improvement	15,801	8,911	-	26,000
Interest Income	38,640	36,262	17,093	20,000
Transfer IN - Sewer Fund	10,000	-	-	-
Total Resources	$ 822,756	$ 878,219	$ 895,312	$ 969,312
Requirements:				
Administration	-	-	-	-
Materials and Services				
Legal Services	-	-	-	-
Engineering	-	-	-	-
Wastewater System Improvements	-	-	-	-
Subtotal Expenditures	-	-	-	-
Transfers OUT:				
Wastewater Capital Reserve Fund	-	-	-	-
Wastewater Fund	-	-	-	-
Subtotal Transfers OUT	-	-	-	-
Contingency	-	-	-	-
Ending Fund Balance	822,756	878,219	895,312	969,312
Total Requirements	$ 822,756	$ 878,219	$ 895,312	$ 969,312

A key planning assumption concerning the Wastewater SDC Fund relates to the Fiscal Year 2010 budget. In that year, the City budgeted expenditures of $100,954 for engineering costs and $867,046 for wastewater system improvements. At the same time, a significant SDC Fund balance was available for these projects estimated at $969,000 for Fiscal Year 2010. This balance was to be used for funding the two projects identified above which would eliminate the SDC Fund balance shown in the City's budget. However, these project costs (engineering & wastewater system improvements) have been included in CIP presented in Chapter 9. Therefore, the project team, after discussions with City staff, has assumed that SDC Fund resources will carry over to the next fiscal year and be available to fund the overall CIP. It has also been assumed for modeling purposes that future expenditures within the Wastewater SDC Fund will come in the form of transfers out. Specifically, it has been assumed the City will transfer $100,000 per year from the SDC Fund to support future debt service payments. Modeling indicates this level of transfers can be sustained until Fiscal Year 2010. This approach reduces the debt service costs that would have to be paid through rates alone.

Wastewater Capital Reserve Fund (27)

This fund accounts for sewer system capital expenditures. The principal resources for this fund come from transfers out of other funds, specifically, cash from the Wastewater Fund (and long term debt proceeds) and SDCs from the Wastewater SDC Fund. Current and future capital expenditures are a result of the capital improvements developed through the City's Facilities Plan. A snapshot cash flow of the Wastewater Capital Reserve Fund is shown in Table 8-4.

Table 8-4: Historical, Estimated, and Budgeted Wastewater Capital Reserve Fund Cash Flows

City of Canyonville
Analysis of Wastewater Capital Reserve Fund (27) Cash Flow

	Actual		Estimated	Budget
	2007	2008	2009	2010
Resources:				
Beginning Fund Balance	65,256	88,680	86,778	138,207
Revenues:				
Late Fees	-	-	-	-
Investment income	3,424	3,138	1,429	400
Subtotal revenues	3,424	3,138	1,429	400
Transfers from other funds - IN:				
Wastewater Fund	20,000	44,200	50,000	48,600
Wastewater SDC Fund	-	-	-	-
Subtotal transfers IN	20,000	44,200	50,000	48,600
Total Resources	$ 88,680	$ 136,018	$ 138,207	$ 187,207
Requirements:				
Materials & Services				
Sewer Rehabilitation	-	49,240	-	50,000
Capital Outlays				
Sewer System Improvements	-	-	-	-
Transfers to other funds - OUT:				
General Fund	-	-	-	-
Wastewater Fund	-	-	-	-
Subtotal transfers to other funds - OUT	-	-	-	-
Reserves and Contingencies:				
Contingency	-	-	-	127,100
Unappropriated ending fund balance	88,680	86,778	138,207	10,107
Total Requirements	$ 88,680	$ 136,018	$ 138,207	$ 187,207

Existing Debt Profile

The City has two existing loans related to its wastewater system:

- DEQ State Revolving Fund (SRF) Construction Loan (1999). The original loan amount of $649,108 was used for interim improvements at the wastewater treatment plant. Outstanding principal as of July 1, 2009 is $373,615. The annual payment is $46,546 and the loan will be retired in 2018.
- DEQ SRF Planning Loan (2009). The City borrowed $200,000 for facilities planning. This 5-year loan has an annual payment of $45,844.

PROJECTED SEWER SYSTEM COSTS – OPERATIONS AND MAINTENANCE

The forecast of sewer system operations and maintenance (O&M) costs starts with the adopted Fiscal Year 2010 budget within the Wastewater Fund. Under the headings of personal services and materials & services, operating expenditures are broken down into numerous line items of expenditure. For the rate forecast, each of these line items is independently adjusted for inflation. Generally, inflation is assumed to be 3 percent per year with the following key exceptions:

Employer contributions to PERS….4 percent per year

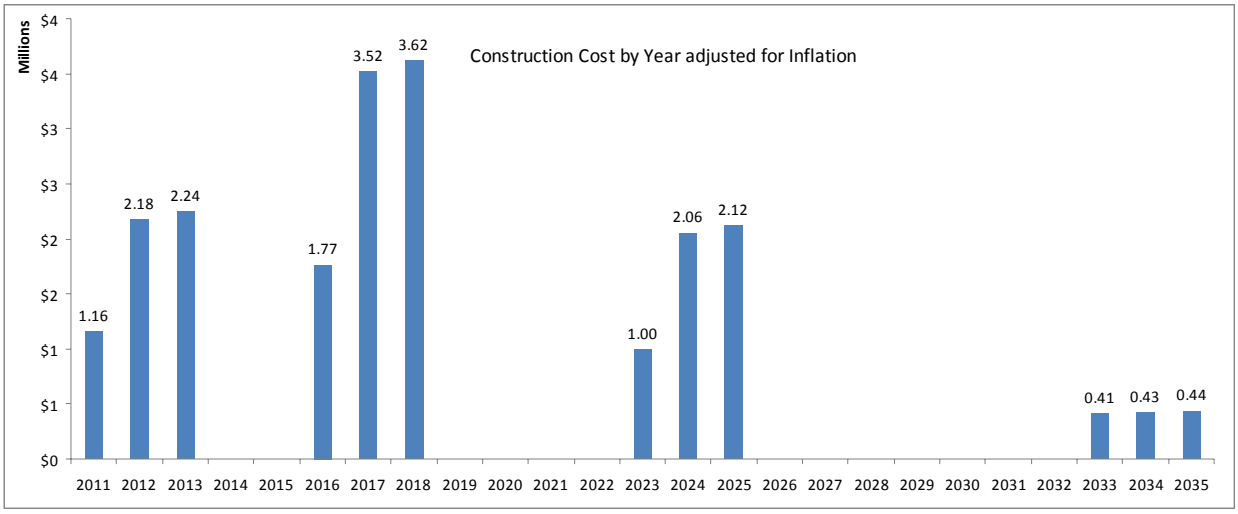
Employer contributions to employee health insurance premiums….6 percent per year

In Fiscal Year 2014, operations and maintenance costs are recalibrated to coincide with the assumed completion of the first phase of the wastewater treatment plant expansion/reconstruction project. As estimated in Chapter 7, the City will incur additional costs for labor, power, and chemicals/materials and these costs have been incorporated into the financial forecast models. The estimated total O&M expense for the wastewater program in year 2014 is $485,000 annually. After this fiscal year O&M cost adjustment, only inflationary changes are made to the Wastewater Fund budget line items.

PROJECTED SEWER SYSTEM COSTS – CAPITAL IMPROVEMENTS

As discussed above, future wastewater system capital expenditures are based on the CIP in Chapter 9. The cash flow requirements for this CIP are shown in Table 8-5.

Table 8-5: Projected Wastewater CIP Cash Flow FY11-FY20



Source: Kennedy/Jenks Engineers

The total CIP in 2009 dollars is $16.6 million. Adjusted for likely inflation by year at 3 percent for planned construction (through 2035) these costs are $20.7 million. As Table 8-5 shows, significant capital outlays are envisioned over the next ten years. Between fiscal years 2011 and

2013, the inflation adjusted total capital outlays are $5.58 million. Then, between fiscal years 2016 and 2018, a second phase of improvements is scheduled amounting to an inflation adjusted total of $8.91 million.

The City does not have cash in reserve to fund these projected capital expenditures and will therefore have to issue debt or obtain grants to fund the CIP. For this analysis, two capital funding scenarios have been developed. The first, called the “base case”, assumes the City will qualify for loans from the United States Department of Agriculture’s Rural Utility Service (USDA-RUS). Under this base case, it has been assumed the City will obtain 20 year loans under this program at the current RUS interest rate of 3.375 percent per year. However, it should be noted that payment terms of as long as 40 years are available to reduce annual costs. In addition to the stated interest rate (assuming level debt service), the City will be required as a loan condition to pledge sewer system revenues such that net revenue of the system will at all times be equal to or greater than 125 percent annual debt service on all long term debt outstanding. Finally, a reserve will be required for each future debt issuance under the USDA-RUS program equal to one year’s debt service on all outstanding bonds. For modeling purposes, it has been assumed the City will upsize future borrowings to fund the reserve requirements and for the payment of issuance costs.

The second funding scenario assumes the City receives grant support for its capital improvement program, although no grant funding has been identified or committed at this time. This funding scenario assumes the City will qualify for the program maximum grant support of 45 percent of total project costs. For modeling purposes, it has been assumed the grant support occurs in each year the capital costs are incurred over the forecast horizon.

PROJECTED SEWER SYSTEM RATES

The financial forecast models solve for revenues required from monthly sewer rates. Both CIP funding scenarios take into account all of the key planning assumptions that have been discussed above. The resulting rate forecast is then expressed in dollars per ERU per month using the ERU forecast discussed earlier. It should be noted this analysis is not a cost of service study, and does not attempt to redefine customer class use characteristics or flow contributions to the system. For purposes of these projections, we have applied a consistent rate per ERU and not allocated future costs among the cities’ numerous customer classes and specific user group charges.

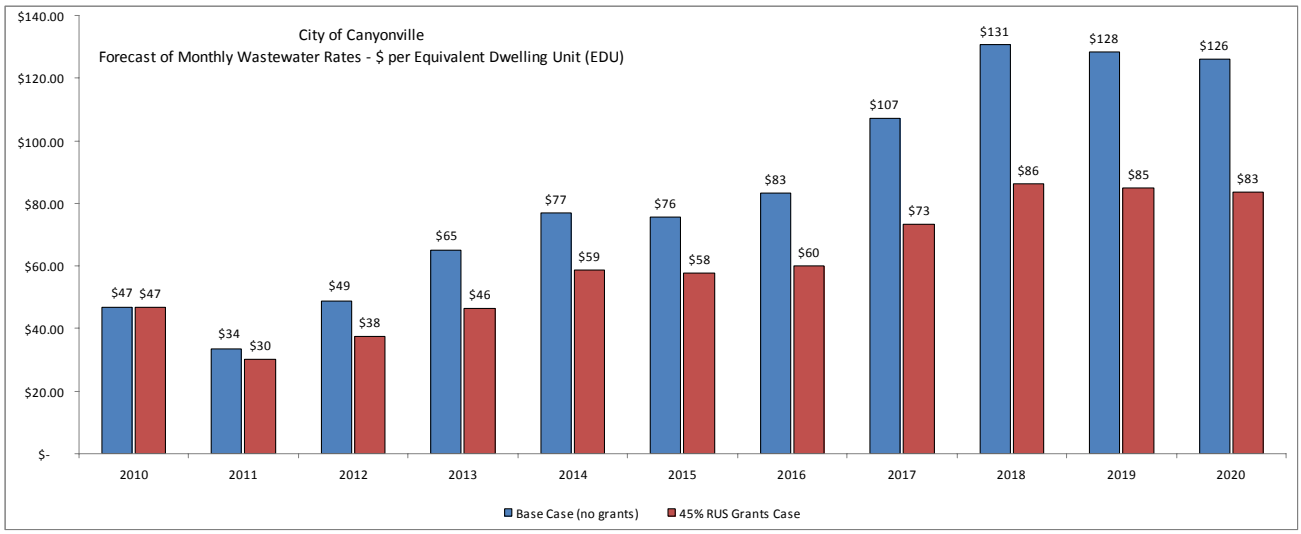
In the preparation of this analysis, the project team made certain assumptions with respect to conditions that may occur in the future. It is believed that for purposes of estimating rates, the assumptions are reasonable for the purposes of this forecast. However, some assumptions may vary due to changes in funding conditions or CIP costs/schedule. The principal forecast assumptions that will affect forecast outcomes are:

- The size and timing of future sewer system capital improvement costs
- The size and timing of future sewer system operations and maintenance costs
- The ability of the City to qualify for the USDA-RUS low interest wastewater loans
- The ability of the City to qualify for grant funding from the USDA-RUS or other granting agencies
- Cash transfers of $100,000 per year from the Wastewater SDC Fund to the Wastewater Fund to support anticipated future debt service payments
- Growth in ERUs occurring in the City at a rate of 1.75 percent per year

- SDC revenues are recognized based on currently adopted unit SDC rates (adjusted for inflation each forecast year) for reimbursement and improvement fee components

With the caveats noted above, our modeling has created two rate forecasts, one for each CIP funding scenario that has been discussed above. The results of these two forecasts are shown below in Table 8-6.

Table 8-6: Projected Rates Under 2 Scenarios – Base Case (no grants) & 45 percent Grant Funding



RATE STRUCTURE AND SYSTEM DEVELOPMENT CHARGES

The current wastewater rate for single family residences within the City is based on a flat fee of $30 per month. Multiple-Family properties pay a base rate of $30 per month per ERU and an "overage" rate of $.02393 per cubic foot of water exceeding their base allowance which varies based on the number of units. Other specific users groups within the City's wastewater rate structure include:

- Restaurants
- Car washes
- Laundromats
- Schools
- Churches
- RV parks
- Motels

Each group is charged $57.10 per month per account with a base allowance of 1,200 cubic feet of water plus a $.02499 per cubic foot charge over the base allowance (this overage charge varies depending on the user group). The City also has rates specific to other users groups such as schools. A 100 percent premium is applied to "Outside City" residential customers and

variable premium levels for the other outside city customer groups. These rates are contained in the City's Ordinance No. 593 dated April 22, 2008. Background information related to how these cost allocations and resulting rates were derived was not available.

The City's wastewater SDCs are codified in the document "Water and Sewer System Development Charges, Hook-Up and Inspection Fees" which does not contain the methodology but identifies 9 distinct user groups ranging (for example) from residential (1 ERU) to fast food (.06 ERU x number of employees) to hospitals (.89 ERU x number of beds). This funding analysis was not intended to review the City's existing SDC structure; however, "typical" wastewater SDCs in Oregon use approaches ranging from site specific discharge estimates, water meter size or actual fixture counts.

For purposes of this rate analysis, the City's existing SDCs were used to forecast revenues from this source through the planning period using the same annual growth of 1.75 percent as established in Chapter 3. Currently, the City's SDC has both the reimbursement (buy-in to existing capacity) and improvement (project costs related to providing capacity for future growth) elements of the fee and for a single family home these are $1,470 and $1,273 respectively, with a total SDC of $2,743. As part of the planning process, each of the proposed projects was evaluated in terms of what was being built to accommodate existing wastewater customers versus what was being constructed to provide capacity for future growth. This analysis concluded that of the total $16.6 million CIP, $7.5 million of these facility costs were being constructed to meet future capacity needs. Under ORS 223, this cost could be recovered from future connections to the City's wastewater system through the SDC. However, since debt and grants will likely be the primary funding mechanisms for these facility costs, a significant portion of this $7.5 million may not be eligible for SDCs (grant funded facilities for example are considered to be contributed capital and not SDC eligible). Accordingly, the rate forecast in this report is based on SDC revenues generated under the City's current SDC structure. Once a clearer picture develops on the actual funding approach by the City, SDCs should be revisited (both in terms of methodology and calculation) in order to assure that the SDC is set at a level to assure that growth is paying for its share of future wastewater capacity costs.

POTENTIAL FUNDING SOURCES

This analysis has focused on rate impacts based on funding through RUS loans and grants. The analysis did not research other applicable grant or loan sources which may be available to the City. However, the following is a list of potential funding sources that the City should consider in developing its final financial plan:

Federal Programs

- Rural Utilities Service (RUS) under the U.S. Department of Agriculture
- Community Development Block Grants (CDBG) under the U.S. Department of Housing and Urban Development
- Build America Bonds (BAB) under the Federal Stimulus Program

State Programs

- Special Public Works Fund (SPWF) under Oregon Business Development Department (OBDD)
- Water/Wastewater Financing Program under OBDD

- Clean Water State Revolving Fund Loan Program under the Department of Environmental Quality

IMPLEMENTATION

In order to move forward with the CIP shown in Chapter 9, the City would need to implement the revised rates described in this chapter beginning in 2011. Clearly, a $16.6 million CIP paid from a customer base of only 982 ERUs results in significant rate increases either under the grant or non-grant funding assumption. Infrastructure improvements of this magnitude are typically found in larger cities or districts where there are some economies of scale to offset rate requirements. This is not the case in Canyonville, which has only 557 sewer hook-ups. Overlaying this magnitude of cost on such a small revenue base results in a severe financial impact on City residents and businesses, producing rates that would be among the highest in Oregon. Additional financial plan refinement during preliminary design is recommended to establish a plan that is financially feasible for the residents and businesses of Canyonville. A “three pronged” approach that incorporates work on the following should be considered:

- Investigate and identify potential funding sources. Maximizing grants and low interest loans has the potential to measurably reduce user rates. All of the potential funding agencies identified in this chapter should be contacted.
- Identify potential cost saving measures. Some potential cost saving measures, such as collaboration with the Tribe, have been discussed within this facilities plan. All opportunities to reduce and/or defer costs while still meeting effluent quality requirements and capacity needs should be explored.
- Optimize rate structure and identify ability to pay. The potential cost saving measures and grant/loan packages should be incorporated into the financial model developed as part of this Facilities Plan to quantify their benefit in terms of rate reductions.

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CHAPTER 9

RECOMMENDED PLAN

The recommended plan presented in this chapter is based on the evaluation of alternatives and assessment of existing facilities. The recommended facilities are sized to serve the City's wastewater treatment needs through anticipated year 2035 conditions. Recognizing the high cost of treatment facility upgrades and the City's financial limitations, emphasis was placed on developing a phasing strategy that defers capital improvements to the extent possible. Deferring costs will give the City time to build financial reserves and acquire favorable funding packages.

WASTEWATER MANAGEMENT STRATEGY

Prior to the detailed evaluation of treatment process alternatives, potential wastewater management strategies were identified, screened, and evaluated. Seven potential wastewater management strategies were identified, and the following four were evaluated in detail:

- MS1. Year-Round Discharge
- MS3. Indirect Discharge
- MS4. Dry Weather Reuse, Wet Weather Discharge
- MS7. Dry Weather Reuse with Tribe, Wet Weather Discharge

Management Strategy MS1 Year-Round Discharge was found to have the lowest capital and present worth costs, followed by Management Strategy MS7. However, it is recognized that at this time, there are a number of uncertainties associated with management strategy MS7 that could impact its cost, including:

- Specific language in the final reuse agreement with the Tribe
- DEQ requirements for redundancy and reliability
- Longevity of agreement
- Reliability of water demand

Because of these uncertainties, Management Strategy MS1 Year-Round Discharge is recommended at this time. However, the City may want to consider conducting additional discussions with the Tribe. Depending on the results of these discussions and other factors, Management Strategy MS7 could potentially become more cost-effective.

DESCRIPTION OF RECOMMENDED FACILITIES AND IMPROVEMENTS

The recommended approach includes both new facilities to increase capacity and enhance performance, and improvements to existing facilities to correct deficiencies and reduce operation and maintenance (O&M) demands. This section describes recommended treatment process and support facility upgrades.

Liquid Stream Treatment Facilities

Liquid stream treatment alternatives were presented and compared in Chapter 7. Recommendations for each liquid stream process follow:

Preliminary Treatment and Influent Pumping. Preliminary treatment removes large solids and grit from the waste stream to protect downstream treatment processes and equipment. Influent pumping lifts the raw wastewater from the influent sewer up to the treatment plant. Two preliminary treatment and influent pumping alternatives were evaluated:

- H1. Screening Upstream of Pumping
- H2. Pumping Upstream of Screening

Both alternatives are compatible with Management Strategy MS1. While Alternative H2 was somewhat less expensive, it represents the continuation of a treatment approach that has caused significant operation and maintenance (O&M) problems at the WWTP for many years. City personnel report that the influent pumps clog periodically due to the presence of rags and other large solids in the raw wastewater. Historically, this periodic clogging has even caused raw sewage overflows. For this reason, Alternative H1 Screening Upstream of Pumping is recommended despite its higher cost.

Secondary Treatment. Secondary treatment removes organic material and solids from the screened and degritted wastewater. Secondary treatment can also provide ammonia removal and some phosphorus removal. Four secondary treatment alternatives were evaluated:

- S1. Upgrade Existing Treatment Unit
- S2. New Conventional Activated Sludge
- S3. Sequencing Batch Reactor (SBR)
- S4. Oxidation Ditch

Costs for two other alternatives were developed to verify that the lowest cost alternatives were being considered. These were:

- Partial membrane treatment (using a membrane bioreactor—MBR—to treat up to the maximum month wet weather flow)
- Integrated fixed film activated sludge (IFAS)

Costs for these were found to exceed those of one or more of the other alternatives being evaluated (Appendix K). Therefore, detailed analyses were not completed.

Of the four original alternative listed above, it is recognized that Alternative S1 would not provide reliable ammonia removal. Therefore, it is not compatible with Management Strategy MS1. Of the remaining alternatives, S2 has substantially higher cost and will not be considered further. The SBR and oxidation ditch are projected to have comparable costs.

A primary difference between the SBR and oxidation ditch is that the SBR does not require clarifiers. While this initially appears to be an advantage, it significantly limits construction phasing opportunities. Alternative S3 would require that the entire SBR system be constructed at one time. In comparison, with Alternative S4, construction of one of the clarifiers could potentially be deferred, as the combined capacity of one new clarifier and one of the WWTP's existing clarifiers would accommodate near-term needs.

It should be noted that changes in ammonia limits could affect the relative costs of Alternatives S3 and S4. Therefore, it is recommended that costs be re-evaluated if final ammonia limits developed by DEQ differ appreciably from those listed in Chapter 6. For the purposes of this report, costs will be based on Alternative S4 Oxidation Ditch. This approach provides the following advantages:

- Potential for construction phasing
- Adequate funds will be budgeted for both SBR and oxidation ditch

Constructing temporary improvements to the existing aeration basins will allow the City to defer construction of a major secondary treatment process upgrade. These temporary improvements are presented later in this chapter.

Tertiary Treatment. Tertiary treatment provides phosphorus removal as well as additional removal of solids and organic material. Two tertiary treatment alternatives were evaluated:

- T1. Chemical Coagulation and Filtration
- T2. Reactive Filtration

Both alternatives are compatible with Management Strategy MS1. The two alternatives had comparable costs, with Alternative T1 projected to be slightly less expensive. As an expansion of the City's existing filtration system, Alternative T1 offers several significant advantages:

- Technology is familiar to WWTP personnel
- Construction can be phased as the existing filter has adequate capacity for current needs
- Non-proprietary technology

For these reasons, Alternative T1 Chemical Coagulation and Filtration is recommended.

Disinfection. Disinfection inactivates pathogenic organisms in the effluent prior to discharge or reuse. Two disinfection alternatives, both of which are applicable to Management Strategy MS1, were evaluated in Chapter 7:

- D1. Chlorination/Dechlorination
- D2. UV Disinfection

Because of its significant cost, safety, and footprint advantages, Alternative D2 is recommended. However, the City's existing hypochlorite disinfection system is functional and can continue to serve the City's needs for some time. Its major shortcoming is a lack of a dechlorination system, which would be required in the upcoming NPDES permit. The inclusion of a temporary dechlorination system is discussed later in this chapter.

Effluent Discharge and Reuse. Consistent with Management Strategy MS1, Alternative E1 Year-Round River Discharge is recommended. However, as noted previously, the City may want to continue discussions with the Tribe to investigate the potential for a collaborative reuse program (Alternative E4 Dry Weather Recycled Water to Tribe, Wet Weather River Discharge), as there may be a potential for cost savings.

Biosolids Management

The City's biosolids are treated to meet Federal regulations for pathogen reduction and vector attraction. Three biosolids treatment alternatives, all associated with alkaline stabilization using lime, were evaluated in Chapter 7:

- B1. Lime Pretreatment
- B2. Lime Posttreatment
- B3. Lime Stabilization Only

The biosolids alternatives were based on the City continuing the current process of land application during dry weather. All three alternatives are compatible with Management Strategy MS1. As opposed to the other two alternatives, Alternative B3 does not include provisions for dewatering, requiring construction of a large storage tank, which results in the highest overall cost. Alternative B1 involves treating liquid sludge with lime prior to dewatering, which results in a higher amount of chemical use than with Alternative B2.

Alternative B2 Lime Posttreatment has the most operational flexibility and phasing options, and the lowest capital cost; however, DEQ is concerned about the ability of this technology to meet performance requirements based on the limited number of installations in Oregon. Alternative B1 is therefore the recommended alternative. With Alternative B1, the City may be able to construct the dewatering facilities as part of an early phase and delay upgrading the lime stabilization equipment until a significant increase in solids loading is observed.

Plant Repairs and Support Facilities

As noted in Chapter 7, the following repairs and support facilities are recommended:

- **Structural Repairs.** There are signs of differential settling at several of the WWTP's existing structures. Structural and foundation rehabilitation is recommended for those structures that are to be retained.
- **Electrical Service.** To assure adequate capacity and code compliance, a new transformer, main breaker, and switchboard are recommended.
- **Standby Generator.** A new engine-generator capable of powering the entire WWTP is recommended.

- **Non-Potable Water Pumps.** Non-potable water pumps to supply pressurized plant effluent for sprays, hose stations, and other non-potable needs are recommended.
- **Distributed Control System (DCS).** A new DCS is recommended to reduce operational labor demands and improve monitoring and control.
- **Site Improvements.** Paving, landscaping, and outside lighting is recommended to be included with plant upgrade projects.

Summary of Recommended Facilities

Design data for the recommended facilities is included in Table 9-1. A process schematic and preliminary site plan are shown on Figures 9-1 and 9-2, respectively. The preliminary site plan assumes that the City is not able to acquire additional land for facility expansion. Without additional land, the following concerns arise:

- Potentially higher construction costs associated with staging and temporary facilities to keep the treatment plant operational during construction
- Inability to accommodate future expansion (beyond year 2035) on the existing site
- Higher cost future expansion (beyond year 2035) due to inefficient placement of facilities

Table 9-1: Design Data for Year 2035 Recommended Facilities

Item	Value
Service Area Design Population	2,764
Flows	
ADWF, MGD	0.27
AWWF, MGD	0.45
MMDWF, MGD	0.48
MMWWF, MGD	0.7
MDDWF, MGD	1.0
MDWWF, MGD	1.7
PHF, MGD	2.2
Loads	
Annual average BOD, ppd	640
Maximum month BOD, ppd	1,200
Annual average TSS, ppd	640
Maximum month TSS, ppd	1,100
LIQUID STREAM	
Preliminary Treatment and Influent Pumping	
Influent Screening	
Type	Mechanical, perforated plate with separate

Item	Value
	washer and compactor unit
Number	1
Capacity, each, MGD	2.2
Opening size, inches	1/8 or 1/4
Type	Manual Bar Screen (by-pass)
Number	1
Capacity, each, MGD	2.2
Opening size, inches	1
Influent Pumping	
Pumps	
Type	Submersible, non-clog
Number	4
Drive	Variable speed
Capacity, gpm, each	510
Total dynamic head, feet (estimated)	50
Motor HP, each	15
Level control type	Ultrasonic
Overflow point	Wet well
Overflow discharge	South Umpqua River
Auxiliary power	
Type	Diesel-fired engine generator
Location	WWTP
Transfer switch	Automatic
Output, kW	200
Grit Removal	
Type	Centrifugal
Number	1
Capacity, each, MGD	2.2
Secondary Treatment	
Oxidation Ditch	
Number	1
Overall length, FT	94
Overall width, FT	70
Sidewater depth, FT	15
Volume, gallons	536,000
Sludge age, days, at average load and MLSS of 2,800 mg/L	20
Anaerobic Tank	
Overall length, FT	15
Overall width, FT	15
Sidewater depth, FT	12

Item	Value
Volume, gallons	20,000
Anaerobic Mixer	
Number	1
Horsepower, each	1
Aerators	
Type	Rotary disc
Number of drives	2
Horsepower, each	25
Mixer	
Type	Floating
Number	1
Horsepower, each	3
ML Recycle Pumps	
Type	Submersible propeller
Number	1
Horsepower, each	3
Secondary Clarifiers	
Number	2
Diameter, FT	32
Sidewater depth, FT	16
Capacity at PHF, total , MGD	2.2
Overflow rate at PHF, gpd/sq ft	1,370
RAS pumps	
Type	Submersible
Number, each clarifier	2
Capacity, each pump, gpm	385
Horsepower, each	5
WAS/scum pumps	
Type	Submersible
Number	2
Capacity, each pump, gpm	100
Horsepower, each	3
Secondary Effluent Pumps	
Type	Submersible
Number	3
Capacity, each, gpm	800
Tertiary Treatment	
Filters	
Type	Granular media
Number	1 existing, 1 new
Number of cells, total	6
Diameter, each, feet	14
Capacity per cell, gpm	160
Flocculation Tank	
Number	1
Diameter, feet	8

Item	Value
Volume, gallons	3,000
Chemical Storage Tank	
Coagulant	Alum or Ferric Chloride
Tank type	Vertical Polyethylene
Number	1
Diameter, feet	8
Volume, gal	2,500
Reject Water Pumps	
Type	Submersible
Number	2
Capacity, each, gpm	100
Disinfection	
UV Channels	
Number of channels	1
Channel length, FT	18
Channel sidewater depth, inches	62
UV Lamps	
Banks, number	1
Modules, number	3
Lamps, number	24
Lamps, type	Low-pressure, high output
Outfall	
Outfall diameter, inches	18
Outfall length, feet	100
Diffuser length, feet	30
Diffuser ports, number	3
BIOSOLIDS MANAGEMENT	
Lime Stabilization Tanks	
Type	Vertical HDPE Storage Tank
Number	5 (3 exist./2 new)
Diameter, feet	12
Volume, each, gal	6,000
Lime Slurry Mixing Tanks	
Type	Vertical HDPE Storage Tank
Number	2 (1 exist./1 new)
Diameter, feet	4
Volume, each, gal	120/200
Sludge Transfer Pumps	
Pump type	Self-priming centrifugal
Pump number	2 (1 exist/1 new)
Capacity, each, gpm	275
Dewatering System	
Type	Screw press
Number	1
Capacity, gpm	25
Sludge Truck/Lime Slurry Pump	
Pump type	Piston pump
Pump number	2

Item	Value
Capacity, each, gpm	25
SUPPORT SYSTEMS	
Standby Generator	
Number	1
Type	Diesel
Capacity, KVA	200
Utility Water System Pumps	
Type	Submersible
Number	2

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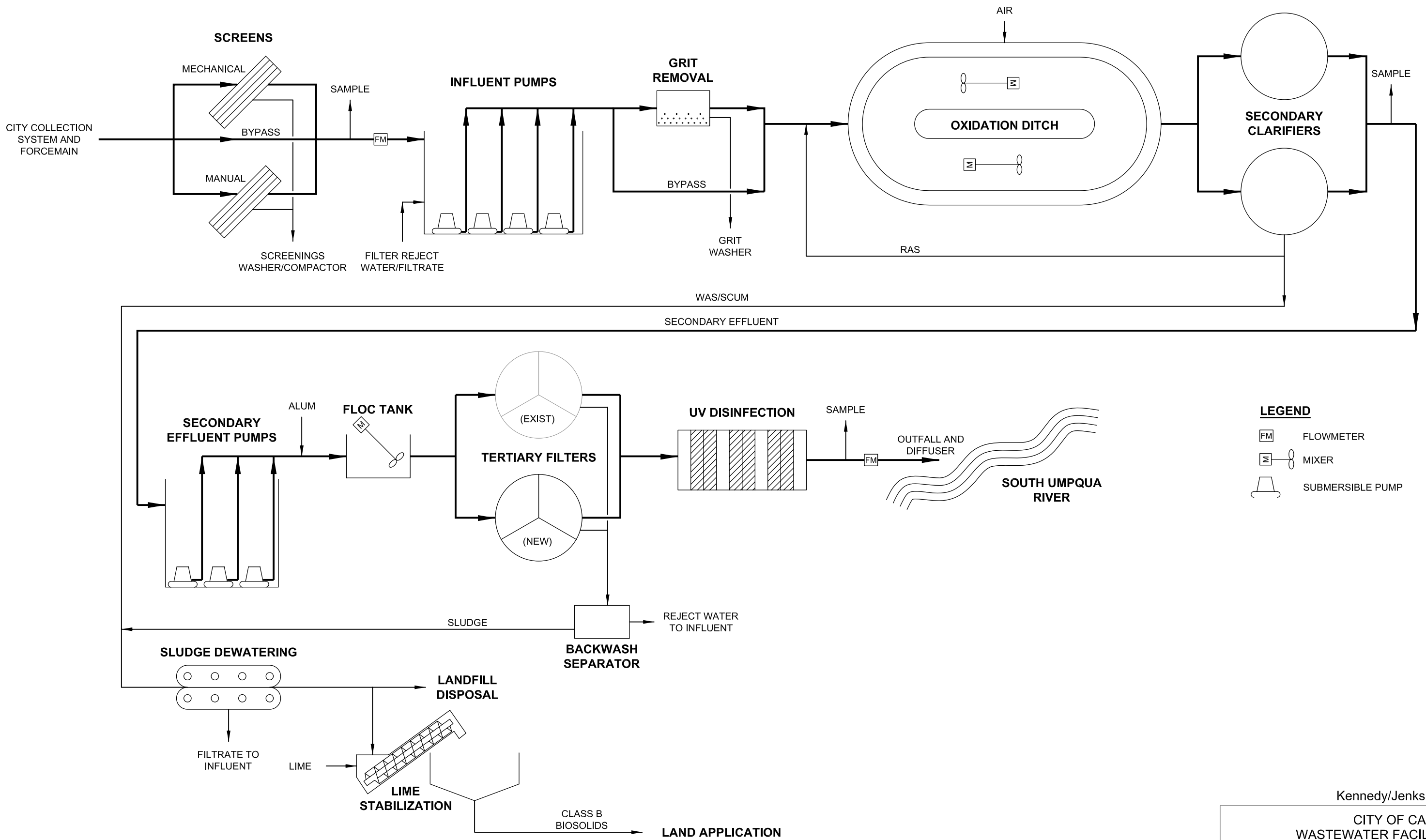

PROCESS SCHEMATIC
NOT TO SCALE

Kennedy/Jenks Consultants

CITY OF CANYONVILLE
WASTEWATER FACILITIES PLAN

SIMPLIFIED YEAR 2035
PROCESS SCHEMATIC DIAGRAM

0976004.00
MAY 2010

FIGURE 9-1

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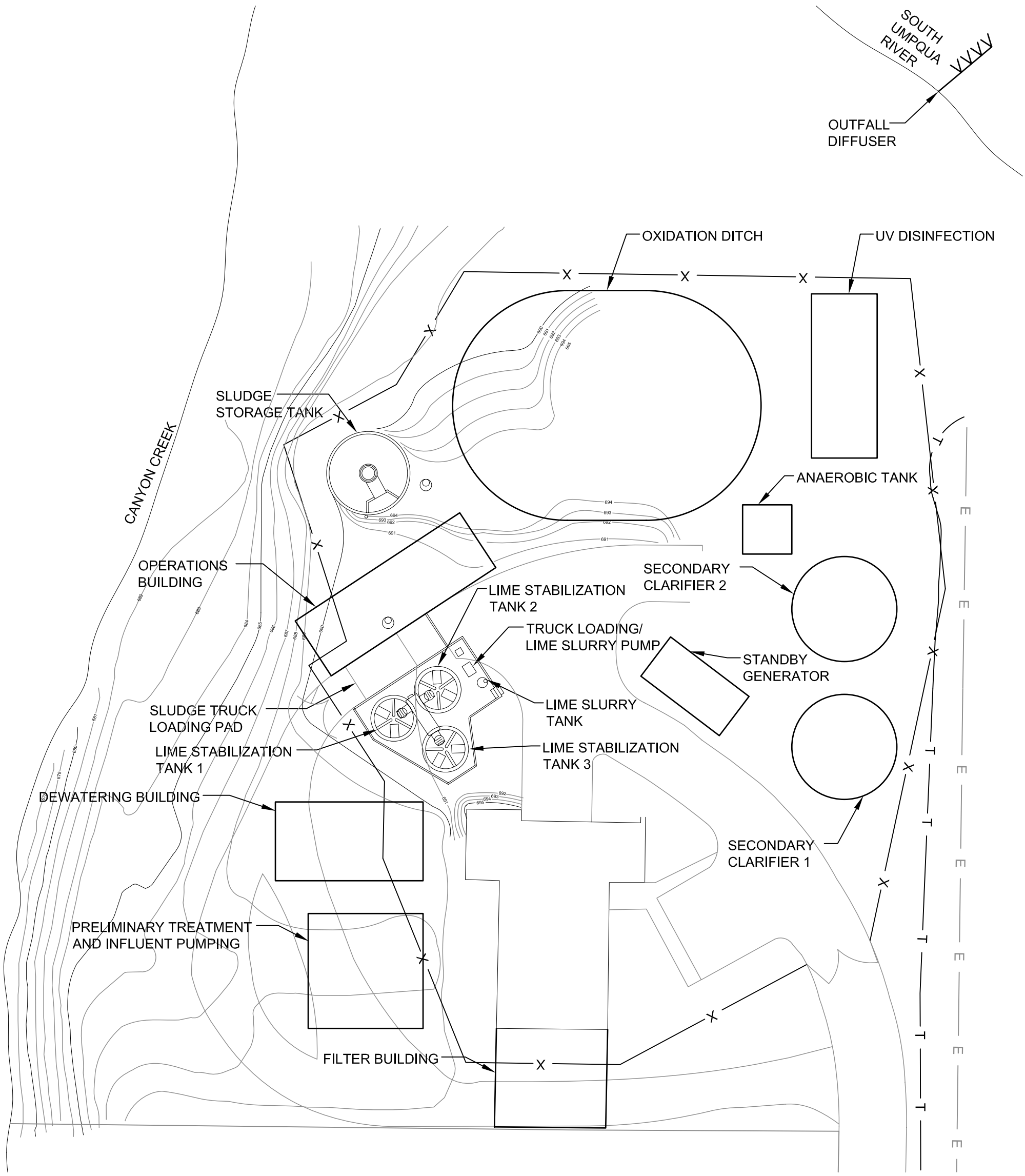

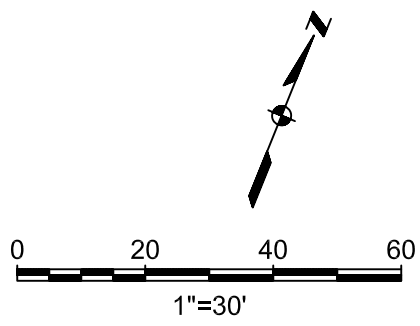

CITY OF CANYONVILLE
WASTEWATER FACILITIES PLAN

PRELIMINARY SITE PLAN

0976004.00
MAY 2010

FIGURE 9-2

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ESTIMATED COSTS

Capital costs for the facilities recommended at the design year of 2035 are summarized in Table 9-2.

Table 9-2: Estimated Capital Costs for Recommended Plan

Item	Cost[(a)], $1000
Preliminary Treatment and Influent Pumping	$ 2,700
Temporary Aeration Basin Improvements	$ 350
Secondary Treatment	$ 4,600
Secondary Effluent Pumping	$ 80
Tertiary Filtration	$ 1,900
Temporary Dechlorination	$ 150
UV Disinfection	$ 900
Effluent Reuse and Discharge	$ 500
Biosolids Management	$ 2,400
Structural Repairs	$ 335
Electrical Service	$ 223
Standby Generator	$ 335
Distributed Control System	$ 223
Site improvements	$ 800
Operations Building	$ 669
Land Acquisition	$ 200
Total (rounded)	$16,400

Note:

(a) Capital costs include the following multipliers: general conditions, 15 percent; contractor overhead and profit, 15 percent; engineering, legal, and administration, 25 percent, contingency, 30 percent; escalation to mid-point of construction, 3.8 percent

IMPLEMENTATION PLAN

The capital improvement plan (CIP) was developed by observing of the following principles:

- **Compliance.** The WWTP should continuously be in compliance with its NPDES permit and/or Mutual Agreement and Order (MAO) limits, even as the limits become increasingly stringent over time.
- **Capacity.** The WWTP should have adequate capacity to accommodate the current population as well as anticipated population growth.
- **Reliability.** The WWTP should comply with EPA and DEQ reliability and redundancy requirements, and should replace existing equipment at the end of its useful life.
- **Just-in-Time Construction.** Recognizing the substantial costs involved, the CIP should distribute the financial burden over time to the extent possible by constructing projects in phases.

Project Phasing Approach

Complying with all regulatory requirements associated with water quality standards and the TMDL represents an enormous capital expense. DEQ and funding agency representatives recognize that the City cannot financially accommodate an initial capital project that provides full regulatory compliance and meets long-term capacity needs. Consequently, the phasing plan presented in this section considers opportunities to extend the time period available to the City to bring the WWTP into full compliance.

Compliance with anticipated effluent ammonia limits represents the single greatest capital expense. Construction of the secondary treatment facilities needed to comply with ammonia limits, along with other crucial plant improvements, increases the cost of the first construction phase to levels that are not achievable. Consequently, DEQ representatives have agreed to issue an amendment to the City's MAO which would extend the time available to comply with ammonia limits. As part of this MAO amendment, DEQ will require the City to remove ammonia to the extent possible using the existing facilities. Because removing ammonia can have detrimental effects on plant operations, temporary aeration basin improvements have been added to the first phase of construction. These temporary improvements would consist of a mixer, baffle walls, and mixed liquor recycle pumping so that the secondary treatment process could operate with an anoxic selector. Another significant benefit of deferring the secondary treatment process upgrade is that effluent ammonia limits could become more restrictive in the coming years as EPA evaluates toxicity criteria. Knowing effluent limits prior to design plant upgrades results in more cost effective facilities.

In Chapter 7, UV disinfection was identified as the lowest cost disinfection alternative. While the City's existing disinfection system has provided reliable compliance with bacteria standards, it has limited capacity and no dechlorination capabilities. With the addition of temporary dechlorination facilities, the existing disinfection system could provide continued reliable service and compliance for a number of years. These relatively low cost temporary facilities would allow the City to defer construction of the new UV disinfection system.

The WWTP has inadequate biosolids storage capacity to accommodate land application restrictions during wet weather conditions. Construction of dewatering facilities will allow the City to dispose of sludge at a landfill during wet weather. However, due to financial constraints, dewatering facilities cannot be included in the first phase of construction. Consequently, it is anticipated that the City will hire a contractor to haul and dispose of residual solids during the wet weather season until a future construction phase.

Given the opportunities presented above, the recommended projects were grouped into four construction phases.

Phase 1—TMDL Compliance. This phase replaces the WWTP's failed headworks and completes plant improvements necessary to comply with the requirements of the TMDL. As discussed previously, DEQ representatives have agreed to amend the City's MAO to allow the deferral of the secondary treatment process upgrade. Construction of temporary aeration basin improvements will support this approach. Construction of a temporary dechlorination system will allow new UV disinfection facilities to be deferred. Phase 1 will consist of the following:

- **Preliminary Treatment and Influent Pumping.** These facilities will replace failed and high maintenance systems, and protect downstream treatment processes from grit and rags.
- **Temporary Aeration Basin Improvements.** The MAO amendment will require the City to remove ammonia to the extent possible with the existing secondary treatment system. These relatively minor upgrades will reduce the operational issues associated with ammonia removal.
- **Chemical Feed System.** Combining the new chemical feed system with the existing filter will provide compliance with effluent phosphorus limits.
- **Temporary Dechlorination Facilities.** Elimination of residual chlorine from the effluent is required by the NPDES permit.
- **Outfall.** A new outfall diffuser is necessary to meet water quality standards.
- **Support Facilities and Repairs.** Structural repairs, an upgraded electrical service, and a distributed control system will be needed to support the above.

Phase 2—Capacity Expansion and Ammonia Compliance. In this phase, the WWTP's overall treatment capacity will be increased to accommodate anticipated growth, and the facility will be brought into compliance with ammonia limits. Elements of Phase 2 consist of:

- Oxidation ditch (or SBR)
- Secondary clarifier No. 1
- Secondary effluent pump station expansion
- UV disinfection system
- Sludge dewatering facilities
- Operations building expansion

Phase 3—Capacity Expansion and Process Reliability. This phase of work will further expand the capacity of the WWTP to accommodate population growth. Elements of this phase include Clarifier 2, Filter 2, and replacement of the existing standby power generator.

Phase 4—Biosolids Reuse. Construction of a new, higher capacity lime stabilization system will allow the City to renew its program of biosolids application onto agricultural land.

Capital Improvement Plan

The CIP is presented in Table 9-3. A 3-year project duration was used to allow for preliminary design, design, bidding, and construction, but may not provide adequate time for funding acquisition. The timing of Phases 2, 3, and 4 is approximate and could be affected by population growth rate, MAO requirements, and other factors.

A number of uncertainties exist which could substantially affect the City's approach to long-term wastewater and biosolids management. Therefore, as uncertainties associated with regulatory issues, growth, and reuse opportunities become resolved

over time, the City should revisit the potential management strategies and alternatives presented in Chapter 7 to see if significant plan modifications are warranted. Examples of conditions that could trigger plan review and modifications include:

- **Ammonia Limits.** If the new NPDES permit includes ammonia limits that are substantially different that those listed in Chapter 6, the relative costs of management strategies could be affected.
- **Cost-Effective Reuse Opportunities.** A cost-effective reuse opportunity could reduce wastewater program costs.
- **Toxics and Metals.** Toxics and metals are receiving increased regulatory scrutiny, which could ultimately lead to more restrictive effluent limits. Treatment performance enhancements and effluent reuse are potential responses.
- **Persistent Bio-Accumulative and Toxic Pollutants (PBTs).** While currently unregulated, increasing scientific understanding and public awareness of PBTs could eventually lead to the establishment of effluent limits. With the exception of reverse osmosis (RO), few treatment processes are known to be effective at removing a wide range of PBTs. Effluent reuse is a potential response to the imposition of stringent PBT effluent limits.
- **Future South Umpqua River TMDLs.** Future TMDLs could result in more stringent effluent limits which could make effluent reuse more cost effective.

Table 9-3. Capital Improvement Plan

Item	Year																									Totals ($1,000)
	2009-10	2010-11	2011-12	2012-13	2013-14	2014-15	2015-16	2016-17	2017-18	2018-19	2019-20	2020-21	2021-22	2022-23	2023-24	2024-25	2025-26	2026-27	2027-28	2028-29	2029-30	2030-31	2031-32	2032-33	2033-34	2034-35	
	ADWF, mgd																										
	0.18	0.18	0.18	0.19	0.19	0.19	0.20	0.20	0.20	0.21	0.21	0.21	0.22	0.22	0.22	0.23	0.23	0.24	0.24	0.24	0.25	0.25	0.26	0.26	0.27	0.27	
	BOD load, ppd																										
	414	421	429	436	444	452	460	468	476	484	493	501	510	519	528	537	547	556	566	576	586	596	607	617	628	640	
	Cost, $1000																										
PLANNING		Phase 1 TMDL Compliance					Phase 2 Capacity and Ammonia							Phase 3 Capacity and reliability										Phase 4 Biosolids reuse			
Facilities planning	150																										150
Environmental report		75																									75
Funding assistance	50																										50
Permit negotiations		20																									20
LIQUID STREAM FACILITIES																											
Influent pumping and headworks		540	1,080	1,080																							2,700
Secondary treatment																											
Temporary aeration basin upgrades		70	140	140																							350
Oxidation ditch							440	880	880																		2,200
Clarifier 1							240	480	480																		1,200
Clarifier 2														240	480	480											1,200
Secondary effluent pumping							16	32	32																		80
Tertiary filtration																											
Chemical feed system		40	80	80																							200
Filter														340	680	680											1,700
Disinfection																											
Temporary dechlorination		30	60	60																							150
UV system							180	360	360																		900
Outfall		100	200	200																							500
BIOSOLIDS MANAGEMENT FACILITIES																											
Dewatering							400	700	700																		1,800
Lime stabilization																								200	200	200	600
REPAIRS AND SUPPORT FACILITIES																											
Structural repairs		67	134	134																							335
Electrical servce		45	89	89																							223
Standby generator														67	134	134											335
Distributed control system		45	89	89																							223
Site improvements		55	100	100			72	139	139					33	66	66								10	10	10	800
Operations building							134	268	268																		669
Land acquisition		40	80	80																							200
Totals ($1,000)	200	1,126	2,052	2,052			1,482	2,859	2,859					680	1,360	1,360								210	210	210	16,660

Capital costs include the following multipliers: general conditions, 15%; contractor overhead and profit, 15%; engineering, legal, and administration, 25%, contingency, 30%; escalation to mid-point of construction, 3.8%

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