CITY OF PHILOMATH
Wastewater System Facilities Plan,
Philomath, Oregon

Section 7

Treatment System Evaluation and Recommendations

SECTION 7
TREATMENT SYSTEM EVALUATION AND RECOMMENDATIONS

7.1. Introduction & General Evaluation Criteria

This section develops and evaluates alternatives to adequately treat and dispose of projected flows and loads throughout the planning period. A wide range of alternatives were evaluated as part of the planning effort.

This section addresses the following key questions:

- What are the existing treatment system deficiencies?
- What treatment system components are likely to become deficient during the planning period?
- How shall the existing and projected deficiencies be corrected?

The existing and projected treatment system deficiencies are presented along with a set of basic alternatives, or tools, for addressing each of the individual deficiencies. The basic alternatives are assembled into sets of primary alternatives that each address all of the existing and projected treatment system deficiencies. A total of nine primary alternatives were evaluated for initial screening, seven of which were evaluated further and finally reduced to the five principle alternatives. For the sake of brevity, only the principal alternatives are described in this section. A present worth analysis for the five principle alternatives is presented as well as a final recommended treatment plan. Should the City choose not to implement the recommended plan, the remaining alternatives, or permutations thereof, may be reevaluated and implemented.

7.2. Identification of Treatment System Deficiencies

The purpose of this section is to determine the components of the treatment system that are or will become deficient during the planning period. A few minor treatment system deficiencies were identified in **Section 4**. This section is intended to supplement that listing. One of the primary goals of this section is to present an overall list of deficiencies that must be addressed by the City during the planning period. Treatment system deficiencies are typically the result of aging or outdated equipment and systems or systems that lack the capacity to accommodate increases in wastewater flows and organic loading due to growth in the community.

The capacity of the existing WWTP is governed by both the hydraulic loading and the organic loading. Once one or the other of these capacities has been reached, NPDES permit violations can be anticipated.

7.2.1 Hydraulic Storage Capacity

The existing treatment facilities operate under a summer-holding winter-discharge scheme. Wastewater that flows to the plant is stored in the lagoons throughout the summer holding period (May 1 to October 31) and released during the winter discharge season (November 1 to April 30). Under such a scheme, one of the key capacity criteria is the hydraulic storage available in the lagoons. Mass conservation methods may be used to determine the amount of storage required during the summer holding period. The volume of storage is determined by summing the plant inflows and outflows over the storage period. The storage requirements in order to maintain the summer-holding winter-discharge operational scheme throughout the planning period are listed in **Table 7-1**. The storage requirements are based on the following assumptions.

- ADWF * 184 days equals wastewater inflow.
- Zero wastewater outflow.
- 15 inches net summer evaporation (Evaporation – Rainfall).
- Zero lagoon leakage.
- 2 foot Minimum lagoon water depth.
- 8 foot Maximum lagoon water depth.
- 3 foot minimum lagoon freeboard.
- Additional lagoons constructed with same bottom, water surface, and top of dike elevations as existing lagoons.

TABLE 7-1
Summer Holding Storage Requirements

Year	ADWF (mgd)	Existing Storage Available (ac-ft)	Storage Required (1) (ac-ft)	Storage Deficit (ac-ft)	Additional Lagoon Area Required (ac)
2003	0.454	232	210	0	0
2005	0.477	232	223	0	0
2010	0.520	232	244	12	2.0
2015	0.567	232	266	34	5.7
2020	0.619	232	291	59	9.8
2025	0.678	232	318	86	14.4
2027	0.703	232	330	98	16.3

(1) Storage requirements include evaporative losses from additional lagoon area

As **Table 7-1** demonstrates, flows to the treatment plant will likely exceed the hydraulic storage capacity of the lagoons sometime between the years 2005 and 2010. Therefore the selected alternative must provide additional storage volume or provide a summertime discharge alternative.

7.2.2 Hydraulic Loading Capacity

The hydraulic structures, pipelines and unit processes must have the hydraulic capacity to convey anticipated peak flows throughout the design period. The facilities of concern include:

- Headworks
- Influent Measurement Equipment
- Distribution Piping
- Transfer Structure
- Outlet Structure and Piping
- Disinfection System
- Dechlorination System
- Effluent Measurement Equipment
- Outfall

Wastewater is pumped to the headworks where it is normally distributed to the southern lagoon. Therefore, the headworks, influent measurement equipment, and distribution piping must be capable of conveying and measuring the peak hourly flows delivered to the plant from the pump stations. The existing peak flows may be estimated by assuming all of the pumps at the three major pump stations are on. The projected peak hourly flow from the pump stations at the end of the planning period is difficult to estimate at this time. As recommended in **Section 6**, Pump Station A must be replaced and the capacity of the Newton Creek pump station may also need to be increased during the planning period. Therefore, the peak pumping rates cannot be known until a detailed design for these facilities is performed. Nonetheless, some general conclusions may be drawn from an analysis of the existing facilities.

Headworks and Influent Flow Measurement Equipment. Flow enters the existing headworks and passes through a 12-inch Parshall flume. The top of the headworks structure is approximately three feet above the bottom of the flume. Therefore, a flow depth of two feet provides for one foot of freeboard. Based on Parshall flume tables, two feet of head corresponds to a flow of 7.425 MGD. The existing peak hourly flow from the collection system is approximately 7.217 MGD (see **Section 4**). The projected peak hourly flow from the collection system at the end of the planning period is approximately 9.906 MGD. It is important to note that these peak hourly flows are from the gravity collection system to the pump stations. Common pump station design practice is to size the pumps such that the peak pump station discharge rate is higher than the inflow to the station. As such, the peak flow to the headworks will likely be higher than peak hourly flows from the gravity collection system. Therefore, the headworks very likely lacks hydraulic capacity to convey projected peak flows and maintain freeboard in the structure. It will need to be replaced sometime during the planning period. The anticipated peak flows also exceed the measurement range of the influent flow measurement equipment (see **Section 4**). Therefore, the influent flow measurement equipment will also need to be upgraded during the planning period. Installation of a new ultrasonic flow meter with electronic data collection capabilities is recommended.

Distribution Piping. Flow is directed to one of the two ponds through 20-inch ductile iron discharge piping. Manning's equation may be used to estimate the capacity of this pipeline. Since the pipe outlet is submerged below the lagoon surface, flow is outlet controlled and the slope of the hydraulic grade line rather than the pipe slope should be used in the calculation. The remaining details of the calculation are as discussed in **Section 6** for the gravity collection system capacity analysis. The bottom of the headworks structure is approximately 2.25 feet above the high water level in the lagoons. The length of pipe from the headworks to the discharge point is approximately 465 feet. Therefore, the slope of the hydraulic grade line is 0.484%. At this slope, the capacity of the pipeline is approximately 6.27 MGD. When compared to the peak inflows discussed above, the distribution piping appears to lack the required capacity when the lagoons are at their maximum water level. This piping will either need to be replaced or the elevation of the headworks will need to be increased during the planning period. If the selected alternative includes retaining the southerly pond as the first pond downstream of the headworks, then the piping must be replaced or relief line must be installed.

Maximum Discharge Rate. The first lagoon acts as a flow equalization basin. Therefore, downstream flows are controlled by the maximum discharge rate from the plant. The discharge rate is set by the operator. For all practical purposes the peak discharge rate is controlled by the regulatory mass load limits to the Marys River. Wastewater of a particular quality can only be discharged at a rate that does not exceed the mass load limits. Therefore, the discharge rate is limited by the effluent quality. Though the permitted effluent limits are 50 mg/l TSS and 30 mg/L BOD, values less than 10 mg/l TSS and 10 mg/L BOD are not uncommon during the winter months. Therefore, the maximum discharge rate is, to some extent, an operational convenience that is dependent upon the quality of the final effluent. The maximum discharge rate is listed as a function of the effluent quality in **Table 7-2.**

TABLE 7-2
Maximum Plant Discharge Rate
Based on Existing Monthly Mass Load Limits
(460 lb BOD/day, 760 lb TSS/day)

BOD (mg/L)	TSS (mg/L)	Allowable Maximum Discharge Rate (mgd)
5	8	11.0
10	17	5.51
15	25	3.68
20	33	2.75
25	42	2.20
30	50	1.82

Clearly, excessively oversized discharge facilities are costly and impractical. Therefore, choosing a reasonable discharge rate is prudent. The existing chlorination facilities are sized to provide 30 minutes of contact time at a peak discharge rate of 3.85 mgd. Since the second chlorine contact chamber is relatively new, all principle alternatives include utilizing these facilities. Therefore, a peak discharge rate of 3.85 mgd will be used to determine the adequacy of the facilities downstream of the first lagoon cell.

Transfer Structure and Piping. Flow from the first to second lagoons is controlled by the transfer structure. The structure includes three canal gates at various elevations to allow flexibility in the level at which water is withdrawn. The three circular gates are 18, 14, and 12 inches in diameter from the bottom to the top respectively. If flow through the gates is modeled as an orifice, only 0.49 feet of head is required to drive 3.85 MGD through the structure if only the 18-inch gate is fully open. If the other gates are open, the head requirement is even less. This head differential is relatively small. Therefore, the hydraulic capacity of the transfer structures is adequate. The transfer pipe between the two lagoons is 18-inch diameter and 85 feet long. An analysis similar to the analysis for the distribution piping shows that only 0.27 feet of head is required to drive 3.85 MGD through the 18-inch transfer pipe. Again, this head differential is relatively small. Thus, the transfer piping should be adequate through the planning period.

Outlet Structure and Piping. Flow from the second lagoons is controlled by the outlet structure. The outlet structure is identical to the transfer structure. Therefore, based on the hydraulic analysis described above, the hydraulic capacity of the outlet structure is adequate. Under normal flow conditions, lagoon effluent flows through approximately 80 feet of 18-inch diameter piping before discharging into the chlorine contact chamber. The high water level in the chamber is approximately 2.5 feet below the high water level in the lagoons. This head differential is more than enough to drive 3.85 MGD through the outlet piping. As such, the hydraulic capacity of the outlet piping should be adequate through the planning period.

Disinfection System. As described above the contact chamber is designed to provide 30 minutes of contact time at a discharge rate of 3.85 MGD. Therefore, the contact chamber volume is adequate. The chlorine disinfection equipment is sized to deliver chlorine at a maximum feed rate of 200 pounds per day. At 3.85 MGD, this equates to a chlorine dosage of 6.2 mg/L. Based on DMR data for the plant, this chlorine dosage is significantly higher than the required dosage to provide disinfection. Therefore, the hydraulic capacity of the chlorine disinfection equipment should be adequate through the planning period. The disinfection equipment may need to be repaired periodically due to aging or outdated parts. However, the cost for these types of repairs is relatively small and should not place a significant burden on the City's OM&R budget.

Dechlorination System. The dechlorination system is sized to deliver sulfur dioxide at a maximum feed rate of 50 pounds per day. At 3.85 MGD, this equates to a dosage of 1.6 mg/L. Based on plant records, the chlorine residual rarely approaches 1 mg/L. Since 1 part of sulfur dioxide is require to neutralize 1 part of chlorine residual, the dechlorination system should be adequate through the planning period. However, should the City choose to feed sulfur dioxide at a greater rate, the dechlorination system has been provided with the necessary parts to upsize to 100 pounds per day.

Effluent Flow Measurement Equipment. Effluent flows are measured with a 4-foot rectangular suppressed weir and a float actuated mechanical meter. The configuration of the weir within the contact chamber allows approximately 6.27 MGD to pass over the weir at the design high water level in the tank. Therefore, the hydraulic capacity of the weir is sufficient. The existing effluent flow meter is nearly 20 year old and though it has served the City well, the technology is outdated when compared to newer equipment now available. As such, it is recommended that the effluent flow meter be replaced with a new ultrasonic flow meter that has data storage and instrumentation control features. This improvement will allow the City to better manage treatment plant operation.

Outfall. After passing through the chlorine contact chamber, plant effluent is routed through approximately 1760 feet of 24-inch diameter pipe to an outlet structure. The top of the riverbank elevation near the outfall is approximately 250 feet. This elevation roughly corresponds to the 5-year flood event. The top elevation of the outfall structure is approximately 253 feet. The required head to drive 3.85 MGD through the 16-inch pipeline is approximately 0.6 feet. The head requirement to drive 3.85 MGD through 1760 feet of 24-inch diameter pipe is approximately 1.2 feet. Therefore the total head requirement from the chlorine contact chamber to the outfall is approximately 1.8 feet at the bank full condition. At these conditions, the water surface elevation in the chlorine contact chamber would be approximately 251.8 feet. The elevation of the top of the chlorine contact chamber walls is approximately 253 feet. Therefore, the capacity of the outfall pipelines is sufficient to discharge 3.85 MGD when the river is at a bank full condition and no modifications to these facilities are anticipated during the planning period.

7.2.3 Organic Loading Capacity

The facultative lagoons provide primary and secondary treatment of the waste stream. The organic loading capacity of the lagoons is finite. If this capacity is exceeded compliance problems will result. The lagoons were designed for an overall organic loading rate of 35 pounds BOD per acre per day with a maximum of 50 pounds BOD per acre per day to the first cell. An analysis of the existing plant shows that the BOD loading to the first cell controls the plant capacity. The first cell is approximately 20 acres. Therefore, the overall capacity of the plant is approximately 1000 pounds BOD per day (50 lbs/ac/day * 20 ac). Based upon the information presented in **Section 5,** the projected loading rates are listed in **Table 7-3.**

TABLE 7-3 Organic Loading Requirements			
Year	BOD Loading (ppd)	Existing Organic Capacity of the Primary Cell (ppd)	Additional Primary Cell Capacity Required (ppd)
2003	820	1000	0
2005	962	1000	0
2010	1072	1000	72
2015	1194	1000	194
2020	1331	1000	331
2025	1482	1000	482
2027	1548	1000	548

As **Table 7-3** demonstrates, the organic loading to the treatment plant will likely exceed the capacity of the lagoons sometime between the years 2005 and 2010. Therefore the selected alternative must provide additional organic treatment capacity.

7.2.4 Summary of Treatment System Deficiencies

Based on the discussions in **Section 4** and the information presented above the existing treatment system deficiencies are summarized in **Table 7-4**.

TABLE 7-4 Summary of Treatment System Deficiencies	
Location	Description of Deficiency
Headworks	May lack capacity to convey projected peak flows at the end of the planning period.
Influent Flow Measurement and Sampling Equipment	May lack capacity to measure projected peak flows at the end of the planning period. End of useful life.
Distribution Piping	May lack capacity to convey projected peak flows at the end of the planning period.
Lagoon Dike Roadways	Aging in need new gravel surfacing.
Lagoon Dikes	Erosion due to lack of rip rap protection.
Lagoons	Lack hydraulic storage capacity required through the planning period.
Lagoons	Lack organic loading capacity required through the planning period.
Effluent Flow Measurement and Sampling Equipment	End of useful life.

7.3. General Treatment System Alternatives

A broad range of alternatives must be considered as part of the planning for major improvements to wastewater treatment systems. These alternatives generally include no action, expansion of the existing wastewater treatment plant (WWTP), or construction of a new WWTP. The alternative of regional treatment via pumping to Corvallis is also considered for discussion purposes. Discussions of each of these general approaches is presented below.

7.3.1 No Action

The No Action alternative must be considered in the facilities planning process to help establish the need for action. Under this alternative, no significant changes would be made to the existing treatment facilities, and the City would continue to operate the existing WWTP as well as possible.

While this is an alternative, it is not considered feasible for the planning period considering the status of the current treatment facility, and the projected increases in flows and loadings. If the existing system deficiencies are not addressed, the plant will eventually reach a point where it is no longer possible to meet the discharge permit requirements and violations will occur, with attendant risks to residents and the environment. The No Action alternative is therefore not recommended and will not be considered further.

7.3.2 Regional Treatment

The only city or service district close to Philomath that it would even be conceptually feasible to approach about regional treatment is Corvallis. Even though regional treatment typically has the benefits of reducing capital and O&M costs in some cases, this alternative is not economically feasible based on the following. The total cost for the new pump stations and force main required to convey wastewater from Philomath to Corvallis will likely far exceed the cost to expand the existing WWTP.

- The force main length would be approximately 7.5 miles from Newton Creek Pump Station to the Corvallis Wastewater Reclamation Plant (WWRP), and would include two ridge crossings and two bridge crossings (Oak Creek and Dixon Creek). In addition some work within state highway right-of-ways would be required as well as a substantial amount of work in high traffic commercial areas within the City of Corvallis. There would also be substantial operation and maintenance costs associated with the new pump stations due to the extremely high head conditions. In concept, Pump Station A would have to oversized, and the Timber Estates and the Newton Creek Pump Stations would have to be immediately upgraded.

- Philomath would have to buy capacity in the Corvallis WWRP and pay a portion of the operation and maintenance costs. In essence, Philomath would have to pay Corvallis the avoided cost for them having to upsize their WWRP to accommodate the flows from Philomath. Although there might be some incremental savings of scale based on the larger size of the Corvallis WWRP, the capital and O&M costs required to pump wastewater to the Corvallis WWRP would exceed any cost savings.

The regional treatment alternative is therefore not recommended and will not be considered further.

7.3.3 Construct New WWTP

Under this alternative, consideration was given to constructing an entirely new plant with new liquid and solids streams. The new plant would utilize none of the existing treatment facilities. This alternative includes abandoning the existing facilities and constructing a new mechanical plant either adjacent to the existing facilities or at an alternative location.

As described in **Section 4**, the existing plant has and continues to serve the City well. The plant is relatively simple and inexpensive to operate and has proven to be very reliable. Though the facilities will reach capacity during the planning period, they are well suited for expansion, and with a few minor modifications will likely serve the City well through the next planning period. The bottom line is that the construction costs for a completely new plant far exceed the costs for expanding the existing plant. Therefore, construction of a new plant is not recommended and will not be considered further.

7.3.4 Upgrade Existing Treatment Plant

Under this alternative, consideration was given to upgrading the existing plant. As described above, the primary deficiency with the existing facilities is a lack of hydraulic and organic capacity to treat projected flows through the planning period. Under this alternative, additions or modifications to the existing facilities were considered. For the most part, the existing facilities would remain in service and the new facilities would provide the required additional hydraulic and organic capacity. The existing facilities are in relatively good condition and are well suited to incremental expansion. The entire subset of alternatives evaluated under this general alternative are less costly than a regional treatment scheme or a new plant. Therefore the recommended general approach is to evaluate alternatives for upgrading the existing facilities.

7.4. Primary and Secondary Treatment Alternatives

As described above, one of the primary deficiencies in the existing treatment facilities is the inability to provide the required organic treatment capacity throughout the planning period. This section, therefore, presents the alternatives that were evaluated to increase the organic treatment capacity of the plant.

7.4.1 Facultative Lagoons

This alternative includes providing an additional facultative lagoon to supplement the existing lagoons. The proposed improvements under this alternative would create a three-cell facultative lagoon system. As the City's experience with the existing facultative lagoon system demonstrates, this treatment technology is relatively simple and inexpensive to maintain and operate. The power requirements are minimal, and essentially no rotating machinery is required. Therefore, power and maintenance costs are very low. The drawback of this alternative is that it tends to require the greatest amount of land area.

Several lagoon configurations were considered. These included placing the new lagoon at the upstream or downstream ends of the existing lagoons. Series and parallel flow streams were also considered. Another variable considered is lined versus unlined lagoons. In order to meet current seepage requirements, synthetic liners are often required. The existing lagoons are unlined. As described in **Section 4**, the seepage rate from the lagoons is below the current DEQ maximum. As such, a reasonable assumption is that the new lagoon could be constructed using only the native onsite clays. A significant amount of land adjacent to the existing WWTP is available where relatively tight soils could be "mined". Without a detailed geotechnical report, it is difficult to determine if unlined lagoons are feasible at this time. Nonetheless, eliminating the synthetic liner would present significant cost savings. As such, both lined and unlined lagoons were carried through the analysis as separate alternatives. Should the City choose the unlined lagoon alternative, a geotechnical investigation should be conducted early in the design process to verify the adequacy of the onsite materials.

Based on a typical maximum aerial loading rate of 35 pounds per acre per day, the information in **Table 7-3** suggests that the minimum facultative lagoon area required is approximately 16 acres. It is important to note that this does not include storage requirements. If the summer-holding winter-discharge operational scheme is to be maintained throughout the planning period a larger lagoon area will need to be provided in order to meet hydraulic storage capacity requirements. The disposal alternatives are considered later in this section.

7.4.2 Partially-Mix Lagoons

Partially mixed lagoons are typically deeper and more heavily loaded organically than facultative lagoons. Oxygen is supplied directly by floating mechanical aerators, submerged diffused aerators, or by floating mechanical mixers that enhance surface reaeration and algae growth. Key design parameters include the amount of aeration and mixing, total horsepower requirements, and aerator or mixer spacing. The aeration is designed to meet the oxygen requirements for BOD removal and in some cases, nitrification. Only a moderate degree of mixing is provided so that solids are not maintained in suspension as in the activated sludge process.

Several permutations of the partially mixed lagoon alternative were considered. These included converting one or both of the existing facultative lagoons to partially mixed lagoons or constructing a new partially mixed lagoon at the upstream end of the treatment plant. Two alternatives for supplying oxygen were also considered. These included floating mechanical aerators and floating solar powered mixers. In order to ensure mixing during periods of heavy cloud cover and at night, the solar powered mixers must be provided with backup power to be considered a viable alternative. The power costs for floating mechanical aerators is substantially higher than for solar powered mixers. Therefore, aerators were dropped from further consideration.

A preliminary screening of the partially mixed lagoon alternatives showed that the organic treatment capacity of the existing lagoons could be sufficiently increased by adding mechanical mixers. Therefore, the primary advantage of this alternative is that no additional lagoons would be required for treatment purposes. However, in order to address the hydraulic capacity issues, a summertime discharge alternative must be provided as part of this alternative.

7.4.3 Completely-Mixed Aerated Lagoons

Completely mixed aerated lagoons are an extension of the partially mixed aerated lagoons. The level of aeration and mixing is increased to provide enough mixing to maintain the solids in suspension. Completely mixed aerated lagoons provide BOD removal in much the same way as the activated sludge process. Higher aeration rates permit shorter detention times and thus, smaller lagoon areas.

Consideration was given to constructing a completely mixed aerated lagoon at the upstream end of the existing lagoons. The aerated cell would be deeper and the water level would remain relatively constant. A large portion of the BOD would be removed in this cell before effluent was delivered to the existing facultative lagoons. To provide the required storage capacity either an additional storage lagoon or a summer discharge alternative would be required. Both of these options were considered. However, preliminary screening showed that the added power costs required for complete mixing were not cost effective when compared to the other alternatives. Therefore, this alternative was eliminated from further consideration.

7.4.4 Constructed Wetlands

Constructed wetlands are generally defined as systems designed for wastewater treatment in an area where natural wetlands do not exist. There are two different types of constructed wetlands. Free water surface wetlands consist of a relatively shallow channel along which the wastewater flows. Subsurface flow wetlands consisting of a layer of permeable media through which the wastewater flows. Both systems utilize emergent aquatic vegetation that promote microbial growth. Both systems also include some type of barrier beneath the wetland bed to prevent groundwater contamination. Primary treatment is required for constructed wetland systems.

Similar to the lagoon alternatives, constructed wetlands require a large area, especially for communities that experience high levels of precipitation. Recent case studies have shown that constructed wetlands can produce high quality effluent. Constructed wetlands can also be used for polishing in conjunction with facultative lagoons or other secondary treatment processes, further reducing BOD and TSS concentrations.

Constructed wetlands are designed as flow through systems that do not provide hydraulic storage. Therefore, they are only feasible in Philomath in conjunction with an additional storage lagoon or a summer discharge alternative. The area immediately surrounding the treatment plant is subject to flooding. Therefore, large dikes would need to be constructed around to the perimeter of the wetland to prevent inundation during high water events. In addition, since flow through constructed wetlands must be maintained to promote the health of the aquatic vegetation, effluent from the lagoon must be recycled during periods when the plant is not discharging to either the river or a land application facility.

A number of constructed wetland configurations were considered. However, due to the complications discussed above and the large land area requirements, none of the configurations were costs effective. Therefore, constructed wetlands were removed from further consideration.

7.5. Advanced Treatment Alternatives

Advanced treatment is not expected to be required to meet discharge requirements to the Marys River during the permitted winter discharge period and will not be considered further.

7.6. Effluent Disinfection Alternatives

Several effluent disinfection alternatives were considered including hypochlorite, ozone, ultraviolet light, and gas chlorine. The existing disinfection system is a gas chlorine system. The effluent is chlorinated as it leaves the second lagoon. Contact time is provided in two chlorine contact chambers. The sulfur dioxide solution is added to dechlorinate the chlorinated effluent prior to being discharged to the river. Both the chlorination and dechlorination systems have the required capacity to serve the City through the planning period. In addition, the first chlorine contact chamber and the dechlorination system are relatively new. Therefore, any alternative disinfection system must provide substantial cost savings or offer another significant advantage in order to justify changes at this time.

Converting to hypochlorite or ozone based systems were immediately eliminated from consideration based on the increased chemical or chemical generation costs. Ultraviolet disinfection is attractive because it eliminates the need for chemical usage, and eliminates problems with the formation of chlorination byproducts. Ultraviolet disinfection systems typically have higher power costs than chlorine gas systems. However, the cost savings of eliminating chemical usage typically offset the additional power costs. Therefore, for general comparison purposes, the operating costs for ultraviolet disinfection systems tend to be lower than for chlorine disinfection systems. The primary limitation of ultraviolet disinfection is that it is only effective for relatively clean effluents. Lagoon systems are subject to algae blooms that tend to shield pathogens from the ultraviolet light. Therefore, ultraviolet disinfection is only viable for non-lagoon treatment facilities or lagoon facilities that include a polishing step prior to disinfection. Ultraviolet disinfection was ultimately removed from consideration due to higher capital costs. As mentioned, the City has recently upgraded the chlorine gas disinfection facilities. These facilities have the required capacity to serve the

City through the planning period. As such, converting to an alternative disinfection system is not cost effective when compared to utilizing the existing system.

7.7. Hydraulic Storage/Effluent Disposal Alternatives

As described in **Section 7.2** the existing treatment facilities lack the hydraulic storage capacity required throughout the planning period. This section presents the alternatives that were evaluated to increase the hydraulic storage capacity of the plant. The need for hydraulic storage is driven by effluent disposal practices. For example, effluent from the existing plant is only discharged during the winter months. Consequently, all flow to the plant during the summer months must be stored for discharge during the next winter discharge period. The required storage volume is provided in the lagoons. If a summertime discharge was available, the need to store plant inflows during the summer months would be reduced and the hydraulic storage capacity of the plant would be lesser concern. Therefore, hydraulic storage and effluent disposal are not mutually exclusive issues. As such, they are considered together in the development of treatment alternatives.

Two broad categories of solutions were considered. The first category consists of alternatives that eliminate the need for additional hydraulic storage. These alternatives include providing a summertime surface water discharge or a summer reuse alternative. The second category simply includes adding additional storage volume and providing a higher level of treatment to discharge a higher volume of wastewater under the existing mass load limits.

7.7.1 Year-round Discharge to Marys River

Under this alternative, consideration was given to reducing the need for summertime hydraulic storage by discharging to the Marys River during the summer months. In order to discharge during the summer current regulations require an effluent with BOD and TSS concentrations below 10 mg/L. This would require an effluent polishing step at the downstream end of the existing lagoons. In addition, the City would have to demonstrate to the DEQ that the summertime discharge met all other effluent quality standards (i.e., temperature, DO, toxic substances, etc.). This alternative could potentially help cool the Marys River, which is water quality limited for temperature during the summer months. The plant effluent passes through approximately 1760 feet of 24-inch diameter buried pipe before being discharged. This pipeline could potentially cool the lagoon effluent below river temperatures. During low mid and late summer flows, the plant effluent would be a significant proportion of the overall flow in the stream. If the plant effluent was sufficiently cooled in the 24-inch outfall, it may reduce the temperature of the river.

The primary drawback of this alternative is that it would add significant operational complexity and operational cost to the WWTP. The polishing step would require significantly more operator attention and expertise than the existing facility. Therefore, operating costs would increase. In addition, the process of gaining regulatory approval for this alternative is likely to be long and expensive and have only a marginal chance of success. Discussions to date with DEQ personnel suggest this alternative is not desired or considered feasible. Therefore, this alternative was eliminated from further consideration.

7.7.2 Summer Effluent Reuse

Under this alternative consideration was given to eliminating or reducing the need for hydraulic storage by discharging to a land application system during the summer holding period. A large number of reuse alternatives were evaluated. Key factors that were considered in developing each of the alternatives are listed as follows.

Overall Project Approach – One of the fundamental decisions that must be made when evaluating an effluent reclamation project is whether the project will be approached from a "wastewater disposal" philosophy or a "resource optimization" philosophy. With the "wastewater disposal" approach, the dominating priority is to dispose of excess wastewater. Using this approach, a municipality will generally desire to discharge the maximum amount of water that can be agronomically utilized on the lowest cost reuse site without much consideration to profitability or crop health. Crop selection criteria are developed to optimize water consumptive rates with little value placed on the agricultural crop. With the "resource optimization" approach, the dominating priority is to maximize the profitability of the agricultural crop by optimizing the use of the wastewater resource. In other words, wastewater is applied at rates that will produce the greatest profitability. The recommended approach is a resource optimization approach. The primary advantage of this approach is the greater potential for profitability. This approach also tends to be more environmentally friendly and is more compatible with the exclusive farm use zoning.

Ownership Alternatives - A number of ownership alternatives were considered. These include public versus private land ownership, public versus private operation, and land lease. The ownership alternatives are critical when evaluating the capital costs, operation and maintenance costs, and when evaluating revenue from potential crop sales. For example, capital costs for a publicly owned reuse facility are much higher than for land that is simply leased to a agricultural user. Likewise, O&M costs are higher for a publicly maintained facility than for a privately maintained facility. Finally, the division of crop revenue between the private owner/operator and the municipality must also be considered. Based circumstances in Philomath, two ownership alternatives were evaluated for further consideration. The first alternative includes public ownership of the land, crops, and all reuse facilities with operation contracted to a private farm operator. This alternative involves significant capital resource and risk but has the greatest potential for profitability. The second alternative includes a typical lease arrangement of irrigated land to a farm operator. The City would provide water at pressure to the lessee. The lessee would be responsible for all crop and irrigation costs, all operation costs, and would have full ownership of all proceeds from the sale of the harvest crops.

Site Selection Considerations – Key site selection criteria that were evaluated include proximity to the treatment plant, proximity to existing or future residential development, topography, climate, groundwater, soil characteristics, and land use. The land immediately west and north of the existing treatment plant was evaluated for a potential reuse site and was determined to be well suited for a reuse facility.

Wastewater Quantity and Quality Requirements – Each of the reuse alternatives were evaluated based on the volume of treated effluent required as well as the effluent quality requirements. Based on the preliminary evaluation, the effluent from the plant was determined to be suitable for effluent reuse. Depending on the type of irrigation system, the effluent from a lagoon system may require filtering to prevent clogging the distribution system.

Crop Selection Alternatives – Key crop selection considerations include, agronomic application rates, harvesting schedule, history of use for land application facilities, replanting requirements, and market conditions. Several different crops were considered including grass pasture, alfalfa, grass silage, hybrid poplar, Christmas trees, peppermint, and ornamental trees. Of these, short rotation hybrid poplar trees grown for lumber and veneer markets and Frazier fir trees grown for the Christmas tree markets had the most promising economics. Therefore, these crops were further evaluated to estimate the profit potential. It was determined that hybrid poplars had a moderate potential for profitability while the Frazier firs had a much higher profit potential. However, Frazier firs required approximately twice the capital investment of hybrid poplars.

Irrigation System Alternatives - A number of irrigation alternatives were evaluated. These included center pivot, travelling gun, wheel line, hand set lines, aluminum solid set lines, polyethylene solid set lines with micro-spray heads, and polyethylene solid set lines with drip emitters. Based on the two crops identified as having potential for profitability, the appropriate types of irrigation systems are solid set type systems using either low cost polyethylene laterals with micro-spray heads, or aluminum laterals with bronze sprinkler heads.

The reuse alternatives were screened separately from the overall wastewater treatment facility. A total of three reuse alternatives were selected for further evaluation. These are listed as follows.

1) Land lease of City-owned land with a summer irrigation source to a private agricultural operator.
2) City-owned hybrid poplar agricultural facility with a contract operator.
3) City-owned Frazier fir agricultural facility with a contract operator.

Based on discussions with City personnel, the preference is to lease the land to a private agricultural user that will have full control of crop selection and may keep all proceeds from crop sales (reuse alternative 1). This option requires less initial capital investment, and poses the least amount of risk to the City. The drawback is that the City surrenders all potential profits from the sale of the crop. City personnel based this decision on the belief that the City should remain in the business of supplying a wastewater utility to its customers only. The capital investment required to establish an agricultural enterprise would likely require higher user fees. City personnel do not believe the modest potential profitability justifies the increased cost and risk to the users.

7.7.3 Polish Lagoon Effluent and Continue Exclusive Winter-Discharge to Marys River

Under this alternative, consideration was given to maintaining the existing operational practices at the WWTP. This includes constructing an additional 20-acre lagoon to provide the summer storage capacity required through the planning period. Plant inflows would be stored through the summer and discharged in the winter as per the current operational scheme.

The permitted mass loads limit the volume of wastewater that may be discharged to the river. The existing NPDES permit includes mass load limits for BOD and TSS. The mass load is directly proportional to the product of the contaminant concentration and the discharge rate. Therefore, in order to discharge a higher volume of waste (increase the discharge rate) under the current mass loads, the contaminant concentrations must be reduced. Therefore, this alternative includes the installation of an effluent polishing step to reduce high BOD and TSS values associated with algae blooms.

A review of the existing DMR data shows that during the front and back ends of the discharge period, the plant experiences algae blooms from time to time. These algae blooms increase the BOD and TSS values in the effluent. The purpose of the polishing step would be to reduce the BOD and TSS values associated with the algae blooms. During periods when algae concentrations are low, the plant routinely produces an effluent with BOD and TSS concentrations below 10 mg/L. As such, the polishing step would only be necessary during the warmer fall and spring months when climatic conditions support algae blooms.

Two technologies were identified as likely being capable of providing the required level of treatment. These include cloth media filters and dissolved air flotation (DAF). The specific proprietary products that were considered are the AquaDisk cloth media filter manufactured by Aqua-Aerobics Systems, Incorporated, and a packaged DAF unit manufactured by the F.B. Leopold Company Inc. Should the City choose this alternative, an exhaustive evaluation of treatment technologies should be performed. Polishing lagoon effluent in the manner considered here is relatively untested in the wastewater treatment industry. As such, bench and pilot studies are also critical to determine the effectiveness of the selected technology prior to implementing this alternative.

The two treatment technologies considered have, in large part, been developed for the unrestricted effluent reuse market where regulatory standards approaching water treatment are the driving force (e.g., California Title 22). As such, manufactures of these technologies typically approach applications from this perspective. For small low-density particles such as algae, it can be difficult to meet level IV reclaimed water standards with the above treatment technologies alone. As such, manufactures can be apprehensive to support the installation of these units to polish lagoon effluent. However, it should be noted that the proposed application does not require the production of an effluent capable of meeting unrestricted reuse standards. The lagoon effluent only needs to be marginally improved to meet the existing mass load limits through the planning period. This is demonstrated in **Table 7-5**. The average required discharge rate over the 181-day discharge period is listed in **Table 7-5**. This value was determined by performing an annual water balance on the proposed treatment plant. Based on the average discharge rate, the maximum BOD and TSS concentrations required to meet the permitted mass load limits were determined. These concentrations are listed by year in **Table 7-5**. As can be seen, the polishing step needs to reduce BOD and TSS levels below 20 mg/L and 35 mg/L respectively in order to meet the existing mass load limits through the planning period. Furthermore, during the majority of the winter discharge period, the BOD and TSS concentrations in the lagoons should be below these values prior to the polishing step. When this is the case, the polishing step may be bypassed.

TABLE 7-5
Maximum Effluent Concentrations Based on Existing Mass Loads

Year	AAF (mgd)	Discharge Volume (1) (million gallons)	Average Discharge Rate (2) (mgd)	Maximum BOD Concentration (3) (mg/L)	Maximum TSS Concentration (4) (mg/L)
2010	0.868	355	1.96	28.1	46.4
2015	0.935	380	2.10	26.3	43.4
2020	1.010	407	2.25	24.5	40.5
2025	1.094	438	2.42	22.8	37.7
2027	1.130	451	2.49	22.1	36.6

(1) Based on following assumptions: 5-year rainfall depth = 47.75 inches; 5-year annual pond evaporation = 23.05 inches; Total lagoon area = existing 37.5 acres + new 20 acres = 57.5 acres
(2) Based on 181 day discharge period
(3) Based on average monthly BOD mass load limit = 460 pounds per day
(4) Based on average monthly TSS mass load limit = 760 pounds per day

7.8. Biosolids Treatment and Disposal Alternatives

The majority of the biosolids are collected in the first lagoon cell. As described in **Section 4**, a biosolids survey was completed in November of 2001. Based on this data, it is not anticipated that the biosolids will need to be removed during the planning period. Furthermore, all of the alternatives evaluated are lagoon-based systems with no biosolids treatment facilities. As such, the biosolids treatment and disposal alternative which is most feasible will be removal of biosolids from the primary cell and beneficially land applied on adjacent agricultural lands during the next planning period.

7.9. Routine Maintenance for WWTPs

A routine maintenance program for the WWTP is just as important to the operation of the sewerage system as the systematic cleaning, inspection and rehabilitation program for the gravity collection system. In many respects, this program is even more critical, since major mechanical or control failures at the WWTP results in bypasses of untreated or partially treated wastewater.

The City currently has a file system with the status and maintenance history of all of the major components in the WWTP and influent pump stations. The City should continue their policy of preventative maintenance on system components.

Following the construction of the new WWTP improvements, the City should implement a policy to update and revise the O&M manuals as system components are replaced or upgraded.

7.10. Development of Principal Treatment System Alternatives

The existing treatment system deficiencies are listed in **Table 7-4**. The purpose of this subsection is to develop complete alternatives that address these deficiencies and that will provide reliable service through the planning period. For a number of the deficiencies listed, an evaluation of alternatives is not useful since the solutions are relatively obvious and straightforward. These solutions are common to all of the complete alternatives. As such, they are considered the as baseline improvements. The baseline improvements are listed in **Table 7-6.**

TABLE 7-6
Baseline Improvements Common to all Principal Alternatives

Deficiency	Recommended Solution
Headworks lack of capacity.	Replace headworks structure designed to pass peak flows.
Influent flow meter lack of capacity and end of useful life.	Replace with ultrasonic flow meter tied to tipping bucked rain gauge and influent sampler.
Influent Sampler end of useful life.	Replace with automatic refrigerated sampler.
Lagoon dike roadways aging.	Regrade and resurface.
Lagoon dike erosion.	Construct dike protection rip rap.
Effluent flow meter end of useful life.	Replace with ultrasonic flow meter tied to effluent sampler.
Effluent sampler end of useful life.	Replace with automatic refrigerated sampler.

The alternatives described above were compared against the deficiencies to develop the complete treatment system alternatives listed in **Table 7-7**. A brief description of each alternative follows.

TABLE 7-7
Principal Treatment System Alternatives

Alternatives	
1. Lined Facultative Lagoon/Summer Effluent Reuse	
Deficiency	Method of Addressing Deficiency
Hydraulic Capacity	Additional lagoon for storage combined with summer land application.
Organic Capacity	Additional facultative lagoon with synthetic liner.
Distribution Piping Capacity	N/A. New lagoon constructed upstream of existing primary lagoon will require the installation of new discharge piping and new transfer structure and piping.
Disposal Method	Continue winter discharge and add summer effluent reuse (land application) as required.
2. Unlined Facultative Lagoon/Summer Effluent Reuse	
Deficiency	Method of Addressing Deficiency
Hydraulic Capacity	Additional lagoon for storage combined with summer land application.
Organic Capacity	Additional facultative lagoon without synthetic liner.
Distribution Piping Capacity	N/A. New lagoon constructed upstream of existing primary lagoon will require the installation of new discharge piping and new transfer structure and piping.
Disposal Method	Continue winter discharge and add summer effluent reuse (land application) as required.
3. Lined Facultative Lagoon/Continue Summer-Holding Winter Discharge	
Deficiency	Method of Addressing Deficiency
Hydraulic Capacity	Additional lagoon for storage.
Organic Capacity	Additional facultative lagoon with synthetic liner.
Distribution Piping Capacity	N/A. New lagoon constructed upstream of existing primary lagoon will require the installation of new discharge piping and new transfer structure and piping.
Disposal Method	Polish lagoon effluent to continue summer-holding winter-discharge and retain existing mass load limits
4. Unlined Facultative Lagoon/Continue Summer-Holding Winter Discharge	
Deficiency	Method of Addressing Deficiency
Hydraulic Capacity	Additional lagoon for storage.
Organic Capacity	Additional facultative lagoon without synthetic liner.
Distribution Piping Capacity	N/A. New lagoon constructed upstream of existing primary lagoon will require the installation of new discharge piping and new transfer structure and piping.
Disposal Method	Polish lagoon effluent to continue summer-holding winter-discharge and retain existing mass load limits
5. Convert Existing Lagoons to Partially Mixed Lagoons/Summer Effluent Reuse	
Deficiency	Method of Addressing Deficiency
Hydraulic Capacity	Summer effluent reuse.
Organic Capacity	Increase capacity of existing lagoons by installing solar powered mechanical mixers.
Distribution Piping Capacity	Construct relief pipe
Disposal Method	Continue winter discharge and add summer effluent reuse (land application) as required.

7.10.1 Alternative 1 – Lined Facultative Lagoon/Summer Effluent Reuse

Under this alternative the City would construct a new facultative lagoon with a synthetic liner to provide the organic treatment capacity through the planning period. The lagoon would be constructed at the upstream end of the existing primary lagoon. Preliminary analysis shows that a minimum 24-acre lagoon is required. All of the baseline improvements identified above, including a new headworks, would be constructed as part of this alternative. In addition, this alternative includes new distribution piping to the new lagoon as well as a new transfer structure between the new lagoon and the existing first lagoon cell.

To prevent overloading the first lagoon, flow from the new headworks must be split between the first and second lagoons. In order to accomplish this, the headworks structure must include a splitter box downstream of the primary measuring device. As a general operating principle, flow to the new cell should be maximized without exceeding a loading rate of 50 pounds per acre per day (ppad). Initially 100% of the flow stream could be routed into the first lagoon cell. As the organic loading rate increases, flow will be gradually split. The new 24 acre lagoon may be loaded up to 1200 pounds of BOD per day (50 ppad * 24 acres = 1200 ppd. The projected BOD loading rate at 2010 is 1072 pounds per day (see **Section 5**). Therefore, the new lagoon will be capable of treating 100% of the flow stream at that time. However, at the end of the planning period, the BOD loading is projected at 1550 ppd. Based on this loading rate, the split would be approximately 80% to the first lagoon cell (50 ppad * 24 acres / 1550 ppd * 100% = 80%) and 20% to the second lagoon cell.

Initially, all of the plant effluent can be discharged to the river without exceeding the existing mass load limits. However, as flows increase due to population growth, summertime disposal will be required to prevent permit violations. As such, this alternative includes the construction of summer effluent reuse facilities. As discussed above, the preferred reuse alternative is to lease the adjacent agricultural land to a farm operator. The farm operator would be responsible for all farming operations including planting, fertilizer, irrigation system, management, and harvest according to the requirements of an approved effluent reuse plant. The City would provide disinfected lagoon effluent at pressures suitable for irrigation. The City must construct an irrigation pump station downstream of the chorine contact chamber. Flow from the chamber would be routed to a booster pumping station with a discharge riser located at the edge of the treatment plant site. The farm operator would be responsible for delivering the water to the crops.

7.10.2 Alternative 2 – Unlined Facultative Lagoon/Summer Effluent Reuse

This alternative is essentially the same as alternative 1 with the exception that the new lagoon will not be constructed with a synthetic liner. The existing lagoons were constructed by compacting the native materials to form a seepage barrier. These lagoons meet the DEQ's seepage requirements. As such, it is reasonable to assume that the new lagoon could be constructed in the same manner. By omitting the synthetic liner, the overall project costs can be reduced. However, this also increases the risk to the City. Should the seepage rate exceed DEQ standards in the future, the City could be forced to install a liner after the new lagoon has been placed into service. The installation of a synthetic liner after the lagoons have been placed into service will cost substantially more than if it is installed as part of the original construction effort (i.e., alternative 1). For this reason, a detailed geotechnical investigation at the initial design stage is recommended if this alternative is to be pursued.

7.10.3 Alternative 3 – Lined Facultative Lagoon/Summer-Holding Winter Discharge

This alternative is similar to alternative 1 with the exception that all lagoon effluent is routed through a polishing step prior to discharge. In this way, effluent BOD and TSS concentrations are sufficiently reduced such that all plant effluent can be discharged to the river. Therefore, this alternative does not include summer land application facilities. A new 24 acre lagoon would be constructed, as well as the ancillary lagoon improvements (i.e., discharge piping, transfer structure, etc.) described under alternative 1.

Under this alternative, the City would install an effluent polishing treatment unit to further reduce effluent BOD and TSS concentrations discharging to the river. Lagoon effluent would be intercepted downstream of the lagoons before passing into the chlorine contact chamber. It would then be routed through the polishing step and back into the chlorine contact chamber. The polishing equipment would be located in a new above grade building adjacent to the existing chlorination building. Due to topographic constraints, the lagoon effluent must be lifted to flow by gravity through the polishing step. Therefore, this alternative includes the construction of an effluent pump station along with the polishing process. The effluent pump station would draw suction directly from the lagoons and discharge to the polishing unit(s). From the polishing units, effluent would pass through the chlorine contact chamber before being discharged to the river.

7.10.4 Alternative 4 – Unlined Facultative Lagoon/Summer-Holding Winter Discharge

This alternative is the same as alternative 3 with the exception that the new lagoon will not be constructed with a synthetic liner. Therefore, the relationship between this alternative and alternative 3 is analogous to the relationship between alternative 2 and alternative 1. As such, this alternative is less expensive than alternative 3, but carries more risk.

7.10.5 Alternative 5 – Convert Existing Lagoons to Partially Mixed Lagoons/Summer Effluent Reuse

Under this alternative the treatment capacity of the existing lagoons is increased by installing floating solar powered mechanical mixers. Mixing the lagoons increases the oxygen transfer rate by continually exposing oxygen deficient water from the bottom of the lagoon to the surface layer. Increasing the oxygen transfer rate will enable to the City to increase the organic loading rate. Solar powered mixers are a relatively new technology with no installations at comparable facilities in the northwest. Therefore, though this alternative has the potential to offer substantial cost savings, it also increases the risk to the City. Should the mixers not live up to expectations, additional improvements will be required.

A preliminary analysis of this alternative showed that the installation of approximately six mixers in the first lagoon cell and three in the second would sufficiently increase the treatment capacity of the plant. The proposed mixers are Solar Bee Circulators, Model SB10000W with a total flow rate of 10,000 gpm as manufactured by Pump Systems, Incorporated. The six mixers in the first lagoon cell would be equipped with electric supplementary power kits. During the night and cloudy days when solar radiation is not sufficient to power the mixers, they will run off of electric power. The three mixers in the second cell will not be equipped with backup power and will be operated under solar power only. During the first 1-2 years of operation, the mixers will work to oxidize the sludge blanket in the first lagoon cell, and may actually reduce the volume of sludge. Due to the oxygen demand of the sludge, organic treatment capacity of the lagoons will not immediately be increased. Therefore, the mixers must be installed at least two years before any significant increase in treatment capacity is required/

Under this alternative, no additional lagoons would be constructed. As such, summer land application facilities similar to those described under alternative 1 must be constructed and will likely have to be constructed at an earlier date due to a reduced lagoon storage volume.

To address the distribution piping capacity issues, this alternative includes constructing a new relief pipe from the new headworks structure to the first lagoon cell. The existing distribution pipe from the existing headworks structure would remain in service. The relief pipe would be constructed such that flows in excess of the capacity of the main inlet piping would spill over into the relief pipe and flow by gravity to the first lagoon cell. The relief pipe would discharge at the lagoon edge rather than at the center of the lagoon. In this way, the relief pipe could be constructed without removing the lagoon from service.

7.11. Evaluation of Principle Treatment System Alternatives

As described above, five alternatives have been identified to address treatment system deficiencies. In this subsection, each alternative is compared to arrive at the best treatment plan. A present worth analysis was performed to compare the capital and annual costs for each alternative. The results of this analysis are presented in **Table 7-8**. The basis for the cost estimates is described in **Section 3**. Both capital and annual costs are estimated for each alternative. The capital costs are the total project costs including construction costs, engineering and surveying costs, administration costs, legal costs, permitting costs, and financing costs. A detailed breakdown of the capital costs is presented in **Appendix I**. The annual costs include power costs as well as O&M costs. The annual costs over the planning period are converted to present worth using a discount rate (i.e., expected rate of return – inflation) of 5% per year. All facilities are assumed to have no salvage value at the end of the planning period. The results of this analysis are presented in **Table 7-8**. The advantages and disadvantages of each alternative are listed in **Table 7-9**.

TABLE 7-8
Present Worth Cost Comparison of Principle Treatment Alternatives

Alternative / Item	Project Cost (Capital Cost)	Annual Cost	Present Worth
1. Lined Facultative Lagoon/Summer Effluent Reuse			
New headworks	$145,000	$500	$151,231
New influent flow and rainfall measurement equipment	$12,325	$250	$15,441
New refrigerated wastewater sampler	$17,400	$1,000	$29,862
New facultative lagoon with synthetic liner	$1,566,000	$5,000	$1,628,311
New distribution piping (from new headworks to new lagoon)	$234,900	$0	$234,900
New transfer piping (from new lagoon to existing primary lagoon)	$203,000	$0	$203,000
Lagoon dike rip-rap protection	$217,500	$0	$217,500
Lagoon dike roadway rehabilitation	$58,000	$2,500	$89,156
New 3-phase power service	$72,500	$0	$72,500
Irrigation booster pump station & forcemain	$290,000	$20,000	$539,244
Irrigation control building	$72,500	$1,000	$84,962
New effluent flow measurement equipment	$8,700	$250	$11,816
New effluent refrigerated wastewater sampler	$17,400	$1,000	$29,862
New SCADA system for wastewater utility	$100,000	$2,500	$131,156
Alternative 1 Totals	**$3,015,225**	**$34,000**	**$3,438,940**
2. Unlined Facultative Lagoon/Summer Effluent Reuse			
New headworks	$145,000	$500	$151,231
New influent flow and rainfall measurement equipment	$12,325	$250	$15,441
New refrigerated wastewater sampler	$17,400	$1,000	$29,862
New facultative lagoon with native clay liner	$1,044,000	$5,000	$1,106,311
New distribution piping (from new headworks to new lagoon)	$234,900	$0	$234,900
New transfer piping (from new lagoon to existing primary lagoon)	$203,000	$0	$203,000
New 3-phase power service	$72,500	$0	$72,500
Lagoon dike rip-rap protection	$217,500	$0	$217,500
Lagoon dike roadway rehabilitation	$58,000	$2,500	$89,156
Irrigation booster pump station & forcemain	$290,000	$20,000	$539,244
Irrigation control building	$72,500	$1,000	$84,962
New effluent flow measurement equipment	$8,700	$1,000	$21,162
New effluent refrigerated wastewater sampler	$17,400	$1,000	$29,862
New SCADA system for wastewater utility	$100,000	$2,500	$131,156
Alternative 2 Totals	**$2,493,225**	**$34,750**	**$2,926,287**

TABLE 7-8 (Continued)
Present Worth Cost Comparison of Principle Treatment Alternatives

Alternative			
Item	Project Cost (Capital Cost)	Annual Cost	Present Worth
3. Lined Facultative Lagoon/Continue Summer-Holding Winter Discharge			
New headworks	$145,000	$500	$151,231
New influent flow and rainfall measurement equipment	$12,325	$250	$15,441
New refrigerated wastewater sampler	$17,400	$1,000	$29,862
New facultative lagoon with synthetic liner	$1,566,000	$5,000	$1,628,311
New distribution piping (from new headworks to new lagoon)	$234,900	$0	$234,900
New transfer piping (from new lagoon to existing primary lagoon)	$203,000	$0	$203,000
Lagoon dike rip-rap protection	$217,500	$0	$217,500
Lagoon dike roadway rehabilitation	$58,000	$2,500	$89,156
Lagoon outlet piping modifications	$50,750	$0	$50,750
New 3-phase power service	$72,500	$0	$72,500
Effluent lift station & controls	$326,250	$15,000	$513,183
Effluent polishing equipment & controls	$1,305,000	$20,000	$1,554,244
Effluent polishing equipment building & sitework	$181,250	$1,000	$193,712
Yard piping modifications	$29,000	$0	$29,000
New Auxiliary power unit with automatic transfer switch	$84,100	$2,000	$109,024
New effluent flow measurement equipment	$8,700	$250	$11,816
New effluent refrigerated wastewater sampler	$17,400	$1,000	$29,862
New SCADA system for wastewater utility	$100,000	$2,500	$131,156
Alternative 3 Totals	**$4,629,075**	**$51,000**	**$5,264,648**
4. Unlined Facultative Lagoon/Continue Summer-Holding Winter Discharge			
New headworks	$145,000	$500	$151,231
New influent flow and rainfall measurement equipment	$12,325	$250	$15,441
New refrigerated wastewater sampler	$17,400	$1,000	$29,862
New facultative lagoon with native clay liner	$1,044,000	$5,000	$1,106,311
New distribution piping (from new headworks to new lagoon)	$234,900	$0	$234,900
New transfer piping (from new lagoon to existing primary lagoon)	$203,000	$0	$203,000
Lagoon dike rip-rap protection (existing lagoons)	$217,500	$0	$217,500
Existing dike roadway rehabilitation	$58,000	$2,500	$89,156
Lagoon outlet piping modifications (existing secondary lagoon)	$87,000	$0	$87,000
New 3-phase power service	$72,500	$0	$72,500
Effluent lift station & controls	$326,250	$15,000	$513,183
Effluent polishing equipment & controls	$870,000	$20,000	$1,119,244
Effluent polishing equipment building & sitework	$181,250	$1,000	$193,712
Yard piping modifications	$29,000	$0	$29,000
New Auxiliary power unit with automatic transfer switch	$84,100	$2,000	$109,024
New effluent flow measurement equipment	$8,700	$250	$11,816
New effluent refrigerated wastewater sampler	$17,400	$1,000	$29,862
New SCADA system for wastewater utility	$100,000	$2,500	$131,156
Alternative 4 Totals	**$3,708,325**	**$51,000**	**$4,343,898**

TABLE 7-8 (Continued) Present Worth Cost Comparison of Principle Treatment Alternatives			
Alternative			
Item	Project Cost (Capital Cost)	Annual Cost	Present Worth
5. Convert Existing Lagoons to Partially Mixed Lagoons/Summer Effluent Reuse			
New headworks	$145,000	$500	$151,231
Relief pipe from new headworks to first lagoon	$29,000	$0	$29,000
New influent flow and rainfall measurement equipment	$12,325	$250	$15,441
New influent refrigerated wastewater sampler	$17,400	$1,000	$29,862
Solar powered mechanical mixing equipment, installed	$464,000	$12,000	$613,547
Power distribution for backup power to mixers	$92,655	$0	$92,655
New Auxiliary power unit with automatic transfer switch	$29,000	$2,000	$53,924
New 3-phase power service	$72,500	$0	$72,500
Lagoon dike rip-rap protection	$72,500	$0	$72,500
Lagoon dike roadway rehabilitation	$58,000	$2,500	$89,156
Irrigation booster pump station & forcemain	$290,000	$25,000	$601,555
Irrigation control building	$72,500	$1,000	$84,962
New effluent flow measurement equipment	$8,700	$1,000	$21,162
New effluent refrigerated wastewater sampler	$17,400	$1,000	$29,862
New SCADA system for wastewater utility	$100,000	$2,500	$131,156
Alternative 5 Totals	**$1,480,980**	**$48,750**	**$2,088,513**

TABLE 7-9 Comparison of Principle Treatment Alternatives	
Alternatives	
1. Lined Facultative Lagoon/Summer Effluent Reuse	
Advantages	Disadvantages
• Very low power requirements	• High capital costs
• Same as existing treatment technology	• Reliance on land application
• Relatively simple	
• Guaranteed to work (i.e., time tested)	
• Very approvable from a regulatory standpoint	
• Construction of land application facilities can be deferred and possibly pushed into the next planning period depending on the effectiveness of I/I reduction efforts.	
• Low risk of permit violations	
2. Unlined Facultative Lagoon/Summer Effluent Reuse	
• Same as alternative 1	• Same as alternative 1
• Lower capital costs	• Higher risk of seepage problems
3. Lined Facultative Lagoon/Continue Summer-Holding Winter Discharge	
• No land application required	• High capital costs
• Construction of polishing step can be deferred and possibly pushed into the next planning period depending on the effectiveness of I/I reduction efforts.	• Operational complexity
	• More expensive to implement due to pilot testing requirements
	• High power costs
	• Not time tested
	• Higher risk of permit violations
	• Requires mass load allocation increase
4. Unlined Facultative Lagoon/Continue Summer-Holding Winter Discharge	
• Same as alternative 3	• Same as alternative 3
• Lower capital costs	• Higher risk of seepage problems
5. Convert Existing Lagoons to Partially Mixed Lagoons/Summer Effluent Reuse	
• Lowest capital cost	• Not time tested
	• Higher risk of failure
	• Land application required sooner
	• Requires the greatest summer discharge volume
	• Increased maintenance requirements

Based on the seepage characteristics of the existing lagoons, the construction of a synthetic liner may well not be necessary. As such, alternatives 1 and 3, are removed from further consideration. Alternative 4, is more costly and more complex than alternatives 2 and 5. As such alternative 4 is also removed from further consideration.

As demonstrated in **Table 7-8**, alternative 5 is the least costly. However, this is alternative also carries the highest risk. This is due to a number of factors. The solar powered mixers described under alternative 5 have not been used in western Oregon or Washington. As such, the application is not time-tested. There is also no practical way to test the effects of the mixers. The number, spacing, and size of the mixers would be determined based on the projected BOD loading at the end of the planning period. There is no practical way to simulate this loading scenario to verify that the mixers will work as designed. In other words, there is no practical way to run a pilot study. As a result, there is some risk of premature failure with this alternative. This alternative has the smallest hydraulic storage capacity. Therefore, land application facilities will be required sooner rather than later. In addition, the volume of water that must be land applied is highest with this alternative.

Alternative 2, is the next lowest cost alternative. Though more expensive, this alternative includes a number of advantages over alternative 5. Alternative 2, employs the same technology currently in use at the City's WWTP. This technology is relatively straightforward and inexpensive to operate. It has served the City well, and is almost guaranteed to provide reliable treatment through the planning period. This alternative also has a lower reliance on land application both from a quantity and timing standpoint. Due to the larger hydraulic storage volume, the construction of land application facilities may be delayed the longest. This has the added benefit of providing the most time for the implementation of the I/I correction program described in **Section 6**. As I/I reduction measures are implemented, the possibility exists that the I/I component of the wastewater flow may be reduced to such an extent that land application facilities may not be needed until very late in the current planning period. If this proves to be the case the cost difference between alternatives 2 and 5 becomes almost negligible.

The cost savings associated with alternative 5 are not believed to be significant enough to justify the increased risk. Therefore, alternative 2 is the recommended treatment alternative.

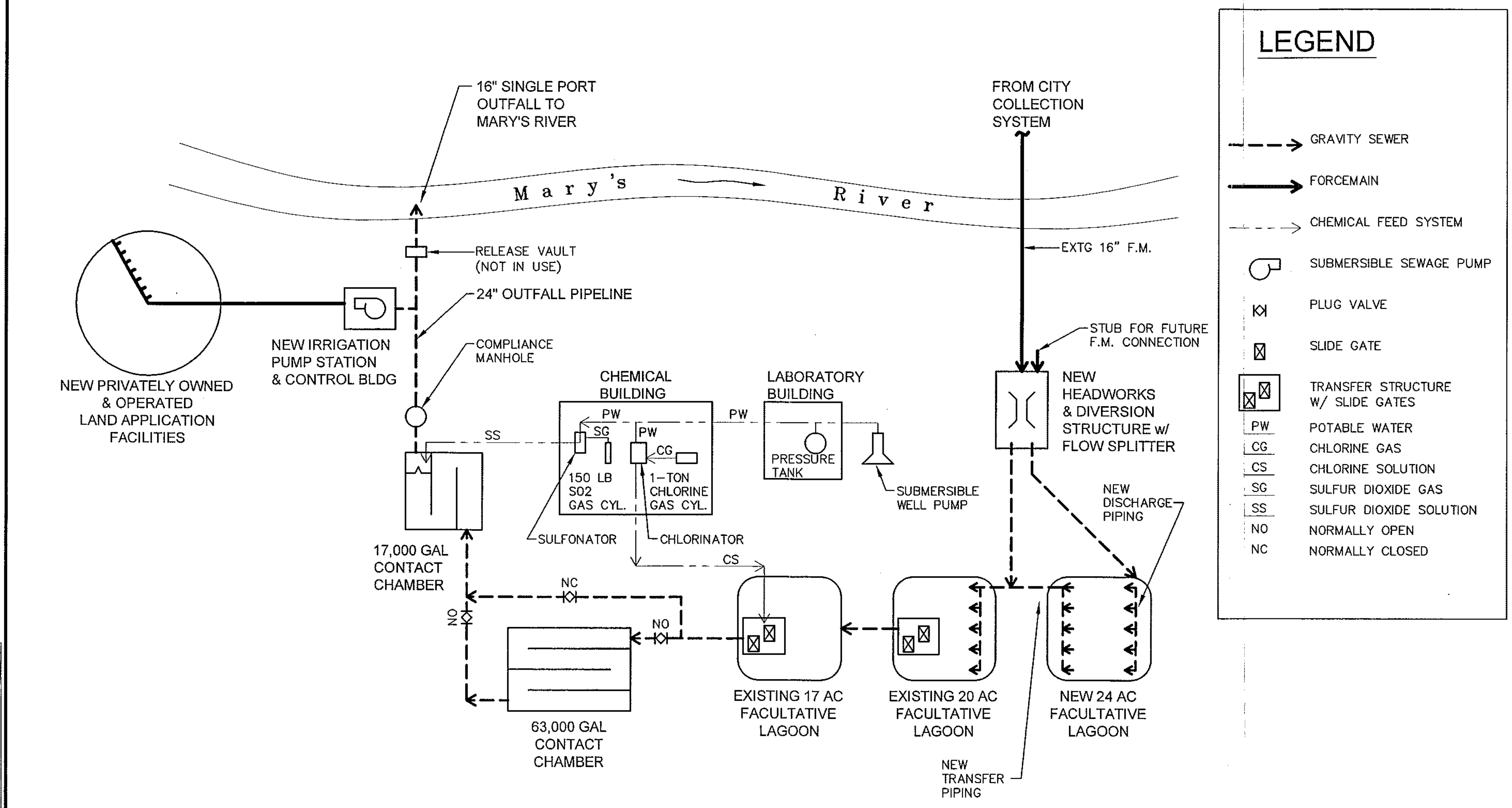
7.12. Recommended Treatment Plan

Alternative 2 is the recommended treatment plan. A process schematic for the recommended plan is included in **Figure 7-1**. The recommended improvements should be constructed in two phases. The components included in each phase are listed in **Table 7-10**. The purpose of phase I is to increase the organic treatment and hydraulic storage capacity of the plant. The existing treatment plant is sufficient for current flows and loadings. Therefore, the phase I treatment system improvements are not required immediately. They should be constructed when the flows and loadings to the existing plant exceed either the organic treatment or hydraulic storage capacity. Based on the analysis presented above (i.e., see **Tables 7-1** and **7-3**), the existing plant should reach capacity sometime between the years 2005 and 2010. The purpose of the phase II improvements is to increase the effluent disposal capacity of the plant. Phase II improvements will be necessary when either the summer storage capacity of the plant is exceeded, or when the discharge volume results in violation of the permitted mass load limits.

TABLE 7-10
Recommended Treatment System Improvements

Project **Unlined Facultative Lagoon/Summer Effluent Reuse**	Total Estimated Project Cost (1)	Oversize Cost Required for Future Growth
Phase I WWTP Improvements		
New headworks	$145,000	$145,000
New influent flow and rainfall measurement equipment	$12,325	$5,175(3)
New refrigerated wastewater sampler	$17,400	$7,300(3)
New facultative lagoon with native clay liner	$1,044,000	$1,044,000
New distribution piping (from new headworks to new lagoon)	$234,900	$234,900
New transfer piping (from new lagoon to existing primary lagoon)	$203,000	$203,000
Lagoon dike rip-rap protection	$217,500	$91,350(3)
Lagoon dike roadway rehabilitation	$58,000	$24,360(3)
New SCADA system for wastewater utility	$100,050	$42,000(3)
New effluent flow measurement equipment	$8,700	$3,650(3)
New effluent refrigerated wastewater sampler	$17,400	$7,300(3)
Total Phase I WWTP Improvements	**$2,058,275**	**$1,808,035**
Phase II WWTP Improvements		
New 3-phase power service	$72,500	$72,500
Irrigation booster pump station & forcemain	$290,000	$290,000
Irrigation control building	$72,500	$72,500
Total Phase II WWTP Improvements	**$435,000**	**$435,000**

(1) Total project cost includes construction costs, 15% contingency, 20% engineering, and 10% legal and administration costs. See Appendix I.
(2) Costs are in 2003 dollars and are based upon dry weather construction.
(3) Percentage of project required for future growth = (2027 population – current population)/2027 population = 42%



NO.	DATE	DESCRIPTION	BY
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REVISIONS

VERIFY SCALE
BAR IS ONE-HALF INCH ON ORIGINAL DRAWING
0 .5"
IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

DSN. CB
DRN. TMT
CKD. CB
DATE: JULY 03

WESTECH ENGINEERING, INC.
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City of Philomath
2003 Sanitary Sewer Facilities Plan

RECOMMENDED TREATMENT PLANT SCHEMATIC

FIGURE
7-1

JOB NUMBER
960.3130.0