CITY OF DAYTON
Wastewater System Facilities Plan
Dayton, Oregon

CHAPTER 7

TREATMENT SYSTEM EVALUATION & RECOMMENDATIONS

Chapter Outline

CHAPTER 7 TREATMENT SYSTEM EVALUATION AND RECOMMENDATIONS

7.1. INTRODUCTION & GENERAL EVALUATION CRITERIA

This section includes the development and evaluation of 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 should 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 ten primary alternatives were evaluated for initial screening, and reduced to the three principal alternatives. For the sake of brevity, only the principal alternatives are described in this section. A cost estimate is included for the three principal alternatives 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 of the 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 during the planning period. Treatment system deficiencies are typically the result of aging or outdated equipment or systems that lack the capacity to accommodate increases in wastewater flows and organic loading due to growth in the community.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. Wastewater mass balance was used to determine the amount of storage required during the summer holding period. The volume of storage is determined by summing the plant inflows (effluent inflow and rainfall) and outflows (treated effluent and evaporation) over the storage period. The storage volumes required 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.
- Existing lagoon area = 11.71 acres, average storage depth =5.5 ft
- 2 foot minimum lagoon water depth, for new lagoons.
- 8.5 foot Maximum lagoon water depth, for new lagoons.
- 3 foot minimum lagoon freeboard.

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 (2) (ac)	Total Lagoon Area Required (ac)
2011	0.295	64	142	78	12	22
2015	0.316	64	152	88	14	23
2020	0.346	64	167	102	16	26
2025	0.379	64	182	118	18	28
2030	0.416	64	200	136	21	31
2035	0.459	64	221	157	24	34

(1) Storage requirements include evaporative losses from additional lagoon area
(2) Based on an active storage depth of 6.5 feet.

As Table 7-1 demonstrates, flows to the treatment plant are currently exceeding the hydraulic storage capacity of the lagoons. Field evidence supports this calculated result. Over the last few discharge seasons the City has had to discharge additional days beyond the permitted window to dispose of the effluent stored during the summer and the winter time inflows. The lagoons have also been fill to a level significantly above the design water level (thereby reducing freeboard). This observation, combined with the calculations presented in Table 7-1 demonstrates that the existing plant lacks adequate hydraulic storage capacity. 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 Flow Measurement Equipment
- Lagoon Distribution Piping
- Lagoon Transfer Structures
- Disinfection System
- Dechlorination System
- Effluent Flow Measurement Equipment
- Outfall

Wastewater is pumped from the Main Pump Station to the headworks, where it is normally split and distributed evenly to Lagoons 1, 2 and 3. Therefore, the headworks, influent flow measurement equipment, and distribution piping must be capable of conveying and measuring the peak hourly flows delivered to the plant. The existing peak flows may be estimated by assuming all of the pumps at the Main Pump Station are on. The projected peak hourly flow from the pump station at the end of the planning period is difficult to estimate at this time, since the upstream gravity sewer are commonly surcharged. As recommended in **Section 6**, the Main Pump Station must be replaced to increase the pumping capacity 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 6-inch Parshall flume. The top of the headworks structure is approximately two feet above the bottom of the flume. Therefore, a flow depth of one foot provides for one foot of freeboard. Based on Parshall flume tables for a 6-inch flume, one foot of head corresponds to a flow of 1.331 MGD. The existing peak hourly flow from the collection system is approximately 3.282 MGD (see **Section 4**). The projected peak hourly flow from the collection system at the end of the planning period is approximately 4.596 MGD. Since the peak hourly flows from the collection system to the main pump station are greater than the capacity of the 6-inch Parshall flume, the headworks lacks the needed capacity. This flume overflows today if all three lagoon pipes are not open. 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. This exacerbates the capacity problem. Since the headworks lack adequate capacity, it will need to be replaced during the planning period before the pump stations are upgraded. The new headworks should include a new 12" Parshall flume. 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.

Lagoon Distribution and Transfer Piping. Flow is directed to any or all of the three lagoon cells through three 6-inch ductile iron pipes. Manning's equation can be used to estimate the capacity of these pipelines. 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 2.5 feet above the high water level in Cell 1, approximately 8-feet above the high water level in Cell 2, and approximately 11-feet above the high water level in Cell 3. The length of pipe from the headworks to the discharge point ranges from approximately 200 feet for Cell 1, approximately 110 feet for Cell 2 and approximately 320 feet for Cell 3. Therefore, the slope of the hydraulic grade line for Lagoon Cells 1, 2 and 3 is 1.25%, 7.30% and 3.44%, respectively. At this slope, the capacity of the pipelines to Cells 1 through 3 are approximately 0.407 MGD, 0.981 MGD and 0.674 MGD, respectively. Summing these values, the total flow capacity of the distribution piping from the headworks to the primary lagoon cells is approximately 2.0 MGD. When compared to the peak inflows discussed above, the distribution piping lacks the required capacity when the lagoon cells are at their maximum water level. Therefore, this piping should be replaced during the planning period.

Maximum Discharge Rate

The three primary lagoon cells act as flow equalization basins. Therefore, downstream flows are controlled by the maximum discharge rate from the plant. The maximum discharge rate may be determined by performing water balance calculations for the plant. This is done by accounting for the volume of water flowing into the plant during the winter discharge season, rainfall on the lagoon surface, and the release of stored water in the lagoons. These three volumes of water are summed and a discharge rate may be calculated by dividing this sum by the number of days over which discharge occurs. As described in greater detail below, three principal alternatives were analyzed in detail. The average discharge rate for each of these alternatives is approximately 1.6 MGD. In order to provide the operators with flexibility and redundancy, it is recommended that all facilities be designed for a slightly higher discharge rate. For the sake of this study a peak discharge rate of 2 MGD was selected for all three principal alternatives. Therefore, all existing facilities downstream of cells 1 through 3 must be capable of conveying 2 MGD.

Transfer Piping

Flow can be directed from cell 1 to cell 2, cell 2 to cell 3, and cell 3 to cell 4, or from cells 1 through 3 to cell 4 via 10-inch transfer piping. All of the transfer piping has sufficient capacity to transfer 2 MGD of flow at low water levels except for the bypass piping between cell l and cell 4, which has a capacity of 1.8 MGD at lagoon low water levels. The difference between 1.8 MGD and the desired 2.0 MGD is not large enough to justify replacing this pipe since the series piping can be used to convey the full 2.0 MGD. However, with the proposed lagoon configuration in the recommended alternative, the existing location of the transfer structure between existing Cell 2 and Cell 3 would cause the wastewater to short circuit. To prevent short circuiting the existing transfer structure should be abandoned and a new transfer structure should be constructed on the west side of the common dike between Cell 2 and Cell 3.

Chlorine Contact Chamber. The contact chamber is designed to provide 30 minutes of contact time at a peak discharge rate of 0.933 MGD. As noted above, the design year peak discharge rate from the facility is estimated to be approximately 2.0 MGD. Therefore, the existing contact chamber does not have the needed capacity and must be upsized during the planning period. The new chlorine contact chamber would need to be 42,000 gallons to disinfect the effluent flow of 2.0 MGD. In addition to capacity issues, the elevation of the top of wall at the chlorine contact chamber is approximately 103.59. The 100-year flood plain elevation is approximately 106.5 per the 2010 FEMA Flood maps. Therefore, the existing chlorine contact chamber will flood during a 100-year flood event. The new contact chamber should be constructed with a wall height above the 100-year flood plain elevation.

Effluent Flow Measurement Equipment. Effluent flows are measured with a 60° V-notch weir and a float actuated mechanical meter. The configuration of the weir within the contact chamber allows approximately 0.933 MGD to pass over the weir at the design high water level in chlorine contact chamber. As noted above, the design year peak discharge rate is 2.0 MGD. Therefore, the hydraulic capacity of the weir is insufficient. The existing effluent flow meter is nearly 20 years old and though it has served the City well, the technology is outdated when compared to 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, as well as a larger weir with sufficient capacity.

Outlet Piping & Outfall. After passing through the chlorine contact chamber, plant effluent is routed through the compliance manhole and then approximately 325 feet of 10-inch diameter pipe to a single port outfall that discharges in the Yamhill River above the ordinary high water level. The outfall piping has a slope of approximately 7% based on the outfall elevation of 67.52-feet and the compliance manhole invert elevation of 91.10-feet. As noted above, the top of the chlorine contact chamber is below the 100-yr flood plain. Therefore, sufficient head is not available to pass any flow during the 100-yr flood event. However, since the existing chlorine contact chamber lacks adequate volume, a new chlorine contact chamber will be required and can be placed above the 100-yr flood plain. The proposed location of the new chlorine contact chamber will result in less than the required 10.2 feet to drive the 2 MGD effluent through the outfall piping. Therefore, the existing outlet pipe to the outfall does not have sufficient capacity and should be upsized during the planning period. Replacing the outfall will require work below the ordinary high water level in the stream. This requires a lengthy permitting process. It is recommended that a new multiport outfall diffuser be installed at the same time the outfall piping is being replaced. This will increase mixing and dilution and can take advantage of the same permitting process required for the outfall improvements.

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. In Western Oregon, facultative lagoons can treat an overall organic loading rate of 35 pounds BOD per acre per day. Further design practice is to limit the organic loading rate of the first cell in a series of cells to 50 pounds of BOD per acre per day. With the first three lagoon cells operated in parallel, the maximum loading rate of 50 pounds BOD per acre per day controls the loading rate of the facility. The total size of the first three cells is approximately 6.07 acres, and the total plant size is 11.71 acres. Therefore, the overall capacity of the plant as designed is approximately 410 pounds BOD per day (35 lbs/ac/day * 11.71 ac). However, based on the maximum 50 pounds BOD per acre per day applied to the first three cells, the plant capacity is 304 pounds BOD per day (50 lbs/ac/day * 6.07 ac). Based upon the information presented in **Section 5**, the projected loading rates are listed in Table 7-2.

Table 7-2 | Organic Loading Requirements

Year	BOD Loading (ppd) (1)	Existing Organic Capacity of the Three Primary Cells (2) (ppd)	Existing Organic Capacity of the Entire Treatment Plant (3) (ppd)	Additional Primary Cell Organic Capacity Required (ppd)	Additional Required Organic Loading Capacity of the Entire Treatment Plant (ppd)
2011	655	304	410	351	245
2015	703	304	410	399	293
2020	768	304	410	464	358
2025	842	304	410	538	432
2030	925	304	410	621	515
2035	1021	304	410	717	611

(1) Includes residential load and 111 pounds BOD from future industrial load.
(2) Based on an aerial loading rate of 50 pounds BOD per acre per day.
(3) Based on an aerial loading rate of 35 pounds BOD per acre per day.

As Table 7-2 demonstrates, the existing lagoons lack the needed organic treatment capacity to treat organic loads through the remainder of the planning period and improvements are needed.

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 7-3.

Table 7-3 | Summary of Treatment System Deficiencies

Location	Description of Deficiency
Headworks	Inadequate capacity to convey existing and projected peak flows.
Influent Flow Measurement and Sampling Equipment	Inadequate capacity to measure existing and projected peak flows. End of useful life.
Lagoon Distribution (Headworks to existing cells 1 through 3)	Distribution piping lacks capacity to convey existing and projected peak flows.
Lagoon Dike Roadways	Aging in need new gravel surfacing.
Lagoons	Inadequate hydraulic storage capacity required through the planning period. Inadequate organic treatment capacity required through the planning period.
Chlorine Contact Chamber	Inadequate capacity for the projected peak flows.
Effluent Flow Measurement and Sampling Equipment	End of useful life.
Effluent Flow Measurement and Sampling Equipment	Inadequate capacity for the projected peak flows.
Outfall	Inadequate capacity to convey projected peak flows

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, or construction of a new treatment plant. The alternative of regional treatment via pumping to Dundee 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 even less able to meet the discharge permit requirements and violations will occur. The No Action alternative is therefore not recommended and will not be considered further.

7.3.2 Regional Treatment

The only municipalities close to Dayton that it would even be conceptually feasible to approach about regional treatment are Lafayette, Dundee, or McMinnville. Even though regional treatment typically has the benefits of reducing capital and O&M costs in some cases, this alternative is not economically feasible for Dayton. Regional treatment will not only require a pump station and forcemain to convey wastewater to the regional treatment facility, it will also require expansion of the regional treatment plant to provide the capacity needed to treat Dayton's waste stream. The total cost for these facilities will far exceed the cost to expand the existing WWTP or construct a new WWTP in Dayton. The regional treatment alternative is therefore not recommended and will not be considered further. The following bullet points include some further discussion supporting this decision.

- The force main length would be several miles from the existing WWTP to the Lafayette, Dundee, or McMinnville Wastewater Treatment Plants (WWTP). 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 Cities under consideration. There would also be increased operation and maintenance costs associated with the new pump station (Main Pump Station) due to the higher head conditions needed to convey wastewater a further distance.

- Dayton would have to buy capacity in the Lafayette, Dundee, or McMinnville WWTP and pay a portion of the operation and maintenance costs. In essence, Dayton would have to pay one of the nearby Cities the avoided cost for that City having to upsize their WWTP and/or collection system to accommodate the flows from Dayton. Although there might be some incremental savings of scale based on the larger size of the joint treatment facility, the capital and O&M costs required to pump wastewater from Dayton to a the proposed regional facility would exceed any cost savings.

- None of the potential Cities under consideration have planned to receive flows from a user the size of Dayton. Therefore, theses cities are not likely to have the excess capacity needed to serve Dayton. Therefore, Dayton would have to fund a major expansion at the selected plant.

- To date, no discussions have occurred with Lafayette, or McMinnville. The City has informally discussed the possibility with the City of Dundee. Dundee is currently constructing a new $12 million WWTP. In order for Dundee to accept sewage from the City of Dayton, Dundee would need to almost double its treatment plant capacity to account for Dayton's sewage flows at the end of the planning period. This, coupled with the large expense of constructing a forcemain to Dundee, renders this alternative significantly more costly than the alternatives presented later in this section. The cost to double the size of Dundee's wastewater treatment plant is likely to be in the $8 million to $10 million range. Approximately 5.75 mile of forcemain piping is required to convey wastewater from Dayton to Dundee. The cost for the forcemain would be in the range of $5.5 million to $8 million ($180/ft to $260/ft). Therefore, the total cost for a regional treatment plant in Dundee would be in the range of $13.5 million to $18 million. This cost range doesn't include any river or ravine crossings which could add cost to the project. This cost range is significantly higher than the cost for the recommended alternative.

- In addition to capital cost, none of the potential Cities have excess disposal capacity built into their NPDES permits. This means that the DEQ must increase the permitted mass load allocations for the City that receives Dayton's waste stream. This is especially true during the dry weather months since Dayton does not discharge during the summer. Due to the water quality limitations that exist in the area, the process of obtaining a mass load allocation increase would be costly with an uncertain outcome.

7.3.3 Construct New WWTP

Under this alternative, consideration was given to constructing an entirely new plant. The new plant would utilize none of the existing treatment facilities. This alternative includes abandoning the existing facilities and constructing a new plant next to the existing WWTP. This was considered in an effort to explore the idea that it may be more cost effective to abandon the existing facilities altogether and construct entirely new facilities that more efficiently utilize the available land area and treatment processes.

Two operational configurations were considered for the new plant. The first includes a summer-hold winter-discharge operational scheme. The second includes winter-discharge with summer land application. These alternatives are discussed in greater detail later in this section.

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 handle projected flows through the planning period. Under this alternative, additions or modifications to the existing facilities were considered. For the most part, the only component of the existing facilities to remain in service would be the existing lagoons. The new facilities would provide the required additional hydraulic storage and organic treatment capacity.

7.4. 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 considered 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 a lesser concern. Therefore, hydraulic storage and effluent disposal are mutually inclusive issues. As such, they are considered together in the development of treatment alternatives.

Two broad categories of solutions were considered. The first 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. The second category consists of alternatives that eliminate the need for additional hydraulic storage, which include providing a summertime surface water discharge or a summer reuse alternative.

7.4.1 Polish Lagoon Effluent and Continue Exclusive Winter-Discharge to Yamhill River

Under this alternative, consideration was given to maintaining the existing operational practices at the WWTP. This alternative requires the addition of new lagoons to provide the 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 effluent 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 a function of the contaminant concentration and the discharge rate. Therefore, in order to discharge a higher volume of treated wastewater (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 the BOD and TSS values from the lagoon effluent prior to discharge.

Two technologies were identified as being capable of providing the required level of treatment. These include both filters and dissolved air flotation (DAF). Should the City choose this alternative, an exhaustive evaluation of proprietary manufactures should be performed at the preliminary design phase of the project. Bench and pilot studies are also critical to determine the effectiveness of the selected technology prior to implementing this alternative. Many manufactures offer bench and pilot test programs that could be utilized during the predesign to ensure success. That said the City of Mollala has used DAF clarifiers followed by media filters for many years to polish lagoon effluent. There are also many similar installations outside the State of Oregon. Therefore, we believe DAF and media filters will ultimately achieve the desired result.

In order to comply with the mass load limits in the NPDES permit, the DAF and media filters must reduce effluent BOD and TSS levels to below 11 mg/L and 18 mg/L respectively at the end of the planning period. This is demonstrated in Table 7-4. The average required discharge rate over the 181-day discharge period is listed in Table 7-4. 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 7-4.

Table 7-4 | Maximum Effluent Concentrations Based on Existing Mass Loads

Year	AAF (mgd)	Discharge Volume (1) (MG)	Average Discharge Rate (2) (mgd)	Maximum BOD Concentration (3) (mg/L)	Maximum TSS Concentration (4) (mg/L)
2011	0.391	161	0.89	20.3	32.4
2015	0.438	177	0.98	18.4	29.4
2020	0.501	201	1.11	16.2	26.0
2025	0.571	226	1.25	14.4	23.0
2030	0.652	256	1.41	12.7	20.4
2035	0.744	289	1.60	11.3	18.0

(1) Based on following assumptions: annual rainfall depth = 41.84 inches; annual lagoon evaporation = 24.12 inches; Total lagoon area = 33 acres
(2) Based on 181 day discharge period
(3) Based on average monthly BOD mass load limit = 150 pounds per day
(4) Based on average monthly TSS mass load limit = 240 pounds per day

7.4.2 Year-Around Discharge to the Yamhill River

Under this alternative, consideration was given to discharging to the Yamhill River during the summer months in order to reduce storage requirements. A summer time discharge permit would face some permitting challenges associated with the Yamhill River water quality limitations. The water quality limitations include a phosphorus TMDL, as well as temperature and dissolved oxygen 303(d) listing. Dayton does not have a mass load allocation in the Yamhill phosphorus TMDL. Therefore, the City would need to obtain a mass load allocation from DEQ and update the existing TMDL. This permitting process could be costly for the City without guaranteed success. In addition, temperature would also be an issue in the summer months for year around discharge. The temperature requirement could be addressed with additional infrastructure such as a deep lagoon. The deep lagoon would be used to store treated effluent. The effluent would be cooled by the ground and stratify as the warm water would rise to the top and the cool water would sink. The cool treated effluent could be drawn off the bottom and discharged to the Yamhill River. The temperature requirement could also be offset by planting riparian vegetation and trading the thermal benefit for the increased thermal load associated with the discharge.

In addition to phosphorus and temperature, the Yamhill River is listed for dissolved oxygen during May. Due to this listing it is not possible to obtain a mass load for the discharge of oxygen demanding waste during the month of May. DEQ must first complete a TMDL for dissolved oxygen. The completion of a TMDL for dissolved oxygen is not expected in the coming years. As an alternative to waiting for the TMDL process to be completed, the City could store effluent during the month of May.

If all of these issues can be addressed, it may be possible to obtain a year around discharge permit from the DEQ. However, the process of gaining regulatory approval for a year-around discharge alternative will take time, require significant consulting costs, and will have an uncertain outcome.

Assuming the City was able to obtain a year-around discharge permit, the treatment plant improvements needed to produce the high quality effluent that would be required during the summer months may end up having higher capital and operational costs than the recommended alternative. At this time, it is not possible to evaluate the cost of a year-around treatment alternative since the permit requirements are not known. For example, stringent effluent requirements such as nutrient removal would have a significant impact on the overall treatment plant cost. Until the specific permit requirements associated with a summer time discharge are defined, it is not possible to evaluate summer time discharge alternatives.

In short, due to regulatory uncertainty of attaining a year around discharge permit and the potential for higher construction and operation costs this alternative was eliminated from further consideration.

7.4.3 Reclaimed Water Irrigation

The irrigation of reclaimed water during the summer months can reduce hydraulic storage requirements, and in some cases it can be more cost effective to irrigate reclaimed water than it is to store the water for discharge during the following winter discharge season. This is particularly the case where a City owns a significant amount of land near the treatment plant that can be used for irrigation. Unfortunately, the City does not own a suitable irrigation site near the treatment plant. As such, summer irrigation of treated effluent from Dayton would require land acquisition and pumping and conveyance facilities to convey effluent to the irrigation site. The cost for this is higher than the cost for the additional lagoon storage volume. Therefore, reclaimed water irrigation it is not considered cost effective when compared to constructing the additional lagoon volume needed to store effluent during the summer months. That said, as the City continues to grow beyond the current planning period, there is little doubt that the irrigation of reclaimed water will eventually become a cost effective solution to address treatment plant capacity issues. However, during the planning period, the irrigation of reclaimed water is not considered to be a cost effective disposal option for the City and is not considered further.

7.5. PRIMARY AND SECONDARY TREATMENT ALTERNATIVES

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

7.5.1 Facultative Lagoons

This alternative includes providing a new facultative lagoon cell or cells. 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 low. The drawback of this alternative is that it tends to require the greatest area of land.

Several lagoon configurations were considered. These included placing one new lagoon at the upstream end of the existing cell 1, and using cells 1 through 4 as polishing cells. The other configuration includes not using any of the existing lagoons in the new plant and constructing entirely new lagoons.

Based on a typical maximum aerial loading rate of 35 pounds per acre per day, the information in Table 7-2 suggests that the minimum facultative lagoon area required is approximately 30 acres. It is important to note that this does not include hydraulic 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 (131 acre-feet of additional storage or a 34 acre lagoon with 6.5 feet of active storage) as shown in Table 7-1.

7.5.2 Partially-Mixed and Completely-Mixed Aerated Lagoons

Partially mixed lagoons are typically deeper and more heavily organically loaded than facultative lagoons. Oxygen is supplied directly by floating mechanical aerators, submerged diffused aerators, or by floating mechanical mixers that enhance surface reaeration and inhibit 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.

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. Completely mixed aerated lagoons were considered as a method to reduce lagoon area in order to save land acquisition costs.

7.5.3 Constructed Wetlands

Constructed wetlands are generally defined as wetlands 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 prior to 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, which do not provide hydraulic storage. Therefore, they are only feasible in Dayton in conjunction with an additional storage lagoon, a summer discharge alternative, or as a polishing step. The area immediately surrounding the existing treatment plant is under the 100-yr flood plain. 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.

Due to the complications discussed above and the large land area requirements, none of the constructed wetland alternatives were feasible or cost effective. Therefore, constructed wetlands were removed from further consideration.

7.5.4 Mechanical Treatment

As used herein, mechanical treatment refers to some type of suspended or attached growth biological treatment process such as sequencing batch reactors, oxidation ditches, trickling filters, rotating biological contactors, etc. As discussed below, a year-round discharge to the Yamhill River is not considered feasible for Dayton. As such, any mechanical treatment alternative must include a storage lagoon to store treated or untreated wastewater during non-discharging periods. In Western Oregon wastewater collection systems that accumulate large amounts of I/I, hydraulic storage (rather than organic treatment requirements) typically control the size of lagoon facilities which are sized to provide dry weather storage. In other words, a wastewater lagoon sized to provide hydraulic storage will generally be large enough to provide sufficient organic treatment. Therefore, since a storage lagoon is required to store wastewater during non-discharging periods, mechanical treatment coupled with storage lagoons is not needed and is not cost effective. To illustrate this statement, a rough cost estimate for a plant that included both storage lagoons and a mechanical secondary process was prepared. The estimated cost for this option is in the range or $11 million to $14 million. This range is substantially higher than the cost of the recommended alternative. However, such a plant does not make a lot of sense to construct since the lagoons can be used to provide secondary treatment and the mechanical secondary process is redundant. For this reason, a mechanical treatment facility was eliminated from further consideration.

7.5.5 Membrane Bioreactor

Under this alternative the construction of a Membrane Bioreactor (MBR) was briefly considered. An MBR utilizes a bioreactor and microfiltration in one unit process. The advantages include a small footprint, nutrient removal, and a high quality effluent. The typical effluent from an MBR has low concentrations of BOD, TSS, turbidity, and bacteria. The disadvantages of an MBR include high capital costs, high energy cost, and potentially high membrane replacement cost. An MBR might potentially be cost effective if the City were able to discharge effluent year-around. However, year-around discharge is not currently allowed under the existing permit and will be difficult to permit in the future as discussed above.

The City of Dundee is currently constructing a $12 million dollar MBR treatment facility. Dundee also has an existing lagoon that is deep which allows the City to discharge the cool water from the bottom of the lagoon in order to meet summertime effluent temperature permit limits. Dayton does not have such a facility. Therefore, Dayton would need to address summer effluent temperature requirements is a different manner that would add significant cost to an MBR alternative. Even if the temperature issue could be addressed in a cost effective manner, we believe the capital and operational costs associated with an MBR are not competitive with the principal alternatives presented below. As such, an MBR plant was removed from further consideration.

7.5.6 Facultative Lagoon Effluent Polishing

Under this alternative hydraulic storage and treatment capacity are provided by constructing additional facultative lagoons. A polishing step is utilized to further increase lagoon effluent quality to meet the effluent permit mass load limits. Several treatment technologies were investigated to polish facultative lagoon effluent. Generally, polishing lagoon effluent refers to removing the algae and solids from the treated effluent. These treatment technologies include membrane filtration, micro strainers, and dissolved air flotation (DAF) with media filters.

Membrane filtration of facultative lagoon effluent is a relatively new approach to polishing effluent. Pilot testing has shown that it is feasible and produces a very clean effluent. However, the membrane loading rates tend to be relatively low. Therefore, large membrane areas are required. This large membrane area results in high power costs and membrane replacement costs. Due to relatively high capital costs, power costs, and membrane replacement costs this treatment technology was not cost effective over the planning period and was not carried through to the primary treatment alternatives. That said, this technology is rapidly advancing. Therefore, the City may wish to briefly reconsider this alternative during the pre-design phase of the wastewater treatment plant improvement project. If future experience with membrane filtration of lagoon effluent shows that increased flux rates are feasible, and/or replacement costs of the membranes have decreased, this alternative may become more cost effective.

Micro strainers or disc filters are a relatively new treatment approach to polishing facultative lagoon effluent. This approach uses a screen of a specified diameter to remove the algae and solids from the lagoon effluent. Generally, these screens have a pore size ranging from 10 to 40 μm. Algae has a particle size range from 1 to 100μm. Therefore, feasibility of micro straining depends on the predominate type and size of algae in the lagoon effluent and may require flocculation prior to the micro strainer or disc filter. At the time this facilities plan was written, the City of Amity in Oregon was in the process of installing micro strainers to filter lagoon effluent. We understand that the facilities in Amity will be operational in the spring of 2012. If Amity is able to successfully remove algae and solids from the lagoon effluent and produce a polished effluent with less than 10 mg/L TSS and BOD, Dayton may want to consider this technology during the preliminary design phase of the WWTP improvement project as a potential cost effective alternative to the polishing process recommended below. However, at this time this technology is largely unproven for removing algae and is not considered further in this plan.

In DAF systems, a pump is used to mix effluent with air under pressure. After mixing with air under pressure the water is super-saturated with gas. The supersaturated water is then released into tankage at atmospheric conditions. The dissolved air comes out of solution and floats upward coagulating the solids in the water. These solids are skimmed off the top while the treated effluent is transported to the next treatment step. Many times a coagulant is also added to the influent and mixed prior to the addition of dissolved air. Coagulant addition provides greater solids removal. DAF has been used successfully to remove a large fraction of TSS and BOD from facultative lagoon effluent. To ensure the City could meet the required effluent quality, gravity media filters may also be required after the DAF filter. DAF by itself can effectively polish lagoon effluent to meet the current effluent mass load limits. Therefore, the DAF and gravity filters could be installed in two phases. The DAF clarifiers could be installed first. Then, as the City grows and produces more sewage, the City could install the media filters after the DAF clarifiers to ensure that the City can meet the mass load limits at the end of the planning period.

Whichever lagoon polishing technology is chosen, bench and pilot studies should be performed to ensure that the technology will meet the required effluent limits at the end of the planning period.

7.6. ADVANCED TREATMENT ALTERNATIVES

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

7.7. EFFLUENT DISINFECTION ALTERNATIVES

Several effluent disinfection alternatives were considered including hypochlorite, ozone, ultraviolet light, and gas chlorine. The existing disinfection system is a hypochlorite system. The effluent is chlorinated as it leaves the fourth lagoon prior to entering the contact chamber. Calcium thiosulfate chemical solution is added to dechlorinate the effluent prior to being discharged to the Yamhill River. Due to age and capacity limitation, it is anticipated that the chemical feed equipment will need to be replaced as some time during the planning period. Therefore, it makes sense to consider alternative disinfection systems.

Converting to chlorine gas or ozone based systems were immediately eliminated from consideration based on the lack of operator experience, the City's desire and the fact that such changes would result in little or no cost savings.

Ultraviolet disinfection is attractive because it eliminates the need for chemical usage and eliminates problems with the formation of chlorination byproducts. However, ultraviolet disinfection systems typically have higher power costs than chlorine systems. 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 very similar to that of a chlorine disinfection system. 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. Since the principal treatment alternatives include lagoon polishing processes, ultraviolet disinfection is feasible. However, the principal treatment alternatives included the addition of the effluent polishing facilities in two phases. The first phase is the installation of the DAF clarifiers and the second phase is the installation of the media filters at a later date. After the installation of the first phase, only the DAF clarifiers will be active. Upsets in the DAF clarification process could lead to ineffective disinfection with an ultraviolet disinfection system. A chlorine disinfection system is not limited in this way.

All of the principal treatment plant alternatives under consideration include a hypochlorite fee system for disinfection and a calcium thiosulfate feed system for dechlorination. This decision was made because the costs for UV disinfection and chlorination/dechlorination are roughly the same. A UV disinfection system may not be compatible with the phased implementation of the treatment process, and the operators are already familiar with hypochlorite disinfection and calcium thiosulfate dechlorination.

7.8. BIOSOLIDS TREATMENT AND DISPOSAL ALTERNATIVES

Generally, the majority of the biosolids in lagoon systems collect in the primary cells. Therefore, we would assume that the first three lagoon cells contain the majority of the biosolids. As described in **Section 4.5.2**, a biosolids survey needs to be completed early in the planning period to determine the quantity of biosolids in the lagoons and refine the cost estimate for removal. It is anticipated that the biosolids in Lagoons 1 through 3 will need to be removed during the planning period. Furthermore, all of the alternatives evaluated are lagoon-based systems with no separate biosolids treatment facilities. As such, the biosolids treatment and disposal alternative will be removal of biosolids from the primary cells and beneficially land applied on adjacent agricultural lands during the next planning period.

7.9. SUMMARY OF INITIAL ALTERNATIVES SCREENING

A wide range of alternatives were considered as described in the previous sections. A summary of the screening process is provided in Table 7-5 for convenience of the reader.

Table 7-5 | Summary of Initial Treatment Plant Alternatives Screening

Alternative	Estimated Cost	Evaluation	Carry Forward for Principal Alternatives Development?
General Treatment Alternatives			
No Action	N/A	• Will lead to permit violations	No
Regional Treatment	• Varies depending on partner City • \$13.5 - \$18 Million for partnership with Dundee (1)	• Not cost effective • Permitting challenges for receiving plant	No
Construct New Treatment Plant	TBD	• Feasible alternative	Yes
Upgrade Existing Treatment Plant	TBD	• Feasible alternative	Yes
Hydraulic Storage/Effluent Disposal Alternatives			
Polish Lagoon Effluent and Continue Exclusive Winter Discharge	TBD	• Feasible alternative	Yes
Year-Around Discharge	NA	• Significant permitting challenges • May not be cost effective	No
Reclaimed Water Irrigation	NA	• No suitable nearby sites • Not cost effective	No
Primary and Secondary Treatment Alternatives			
Facultative Lagoons	TBD	• Feasible alternative	Yes
Partially Mixed Lagoons	TBD	• Feasible alternative	Yes
Constructed Wetlands	NA	• Not cost effective • Large land area • Recycling requirements	No
Mechanical Treatment	\$11 - \$14 Million	• Since lagoons required for summer storage, primary and secondary treatment can be provided in the lagoons and additional mechanical treatment is not required	No
Membrane Bioreactor	NA	• Not cost effective • High operation costs	No
Facultative Lagoon Effluent Polishing	TBD	• Feasible alternative	Yes
Advanced Treatment Alternatives			
Not required for Dayton			
Effluent Disinfection Alternatives			
Chorine disinfection	TBD	• Feasible Alternative	Yes
Ultraviolet disinfection	NA	• Not compatible with lagoon effluent	No

(1) Refer to Section 7.3.2 for a breakdown of cost estimates.

7.10. DEVELOPMENT OF PRINCIPAL TREATMENT SYSTEM ALTERNATIVES

The existing treatment system deficiencies are listed in Table 7-3. 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.
Facultative lagoons hydraulic Storage	Upsize the hydraulic storage capacity Remove sludge from existing lagoons
Hypochlorite feed system lacks capacity	Upsize equipment.
Calcium thiosulfate feed system lacks capacity	Upsize equipment.
Chlorine contact chamber lacks capacity	Replace contact chamber with larger chlorine contact chamber.
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.
Outfall is above the Yamhill River ordinary high water lever and lacks capacity	Replace effluent piping to pass peak flows and install multi-port diffuser.

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. Expand Existing Plant/ Continue Summer-Hold Winter Discharge & Polish Lagoon Effluent	
Deficiency	Method of Addressing Deficiency
Hydraulic Capacity	Additional lagoons for combined storage.
Organic Capacity	Additional facultative lagoon with synthetic liner. Polish effluent to meet mass load limits.
Distribution Piping Capacity	Utilize existing transfer piping between lagoons. New lagoon constructed upstream of existing primary lagoon will require the installation of new discharge piping and new transfer structure and piping. A transfer pump station will also be required to transfer water from lagoon 4 to the DAF and filters.
Disposal Method	Polish lagoon effluent to continue summer-holding winter-discharge and retain existing mass load limits. A new outfall would be required.
2. New Facultative Lagoons/Continue Summer-Hold Winter Discharge & Polish Lagoon Effluent	
Deficiency	Method of Addressing Deficiency
Hydraulic Capacity	New lagoons for storage (controls lagoon size).
Organic Capacity	New facultative lagoons with synthetic liner & polished effluent to meet mass load limits.
Distribution Piping Capacity	N/A. New lagoons will require the installation of new distribution piping discharge piping and new transfer structures. Will require additional effluent pump station.
Disposal Method	Polish lagoon effluent to continue summer-holding winter-discharge and retain existing mass load limits. A new outfall would be required.
3. New Aerated Lagoons & Facultative Lagoon Expansion/Continue Summer-Hold Winter Discharge & Polish Lagoon Effluent	
Deficiency	Method of Addressing Deficiency
Hydraulic Capacity	Additional lagoons for combined storage.
Organic Capacity	Additional Aerated primary lagoon followed by deep facultative lagoons with synthetic liner. Polish effluent to meet mass load limits.
Distribution Piping Capacity	Utilize existing distribution piping between lagoons. . New lagoon constructed upstream of existing primary lagoon will require the installation of new discharge piping and new transfer structure and piping. Will require additional effluent pump station.
Disposal Method	Polish lagoon effluent to continue summer-holding winter-discharge and retain existing mass load limits. A new outfall would be required.

7.10.1 Alternative 1 – Expand Existing Plant / Summer-Hold Winter Discharge & Polish Lagoon Effluent

Under this alternative the City would expand the existing WWTP. Please refer to Figure 7-1. This alternative includes constructing one new facultative lagoon with a synthetic liner. In combination with the existing lagoons, the additional lagoon will provide the hydraulic storage capacity through the planning period. In addition DAF clarifiers and media filters would be constructed downstream of lagoon 4 to polish the wastewater to meet the winter time discharge permit limits. Therefore, this alternative does not include summer land application facilities. This alternative also includes the construction of a new headworks, distribution piping to the new lagoon, a new transfer structure between the new lagoon cell and cell 1, a new structure between cell 2 and 3, and a new transfer pump station to pump effluent from existing cell 4 to the DAF clarifiers and media filters. This alternative also includes a new operations building, chlorine contact chamber, disinfection equipment, dechlorination equipment, and outfall piping, and outfall diffuser. Since the existing lagoon cells are to remain in service, this alternative includes repairing the leak through the north dike of cell 4. This alternative can be construction within the existing UGB.

Wastewater enters the new headworks that consists of a new 12-inch Parshall flume, sampling equipment, and flow monitoring equipment. From the headworks wastewater is then directed by gravity via transfer piping to the new 24-acre facultative lagoon with a synthetic liner. As shown in Table 7-1 and Table 7-2 the lagoon size is driven by hydraulics rather than BOD loading rate. The new lagoon is proposed with a synthetic geomembrane liner rather than a native clay liner to form a seepage barrier.

From the new lagoon wastewater would be transferred by gravity to the existing lagoon cell 1 and transferred in series to cells 2, 3 and 4. To avoid short circuiting, a new transfer structure is recommended between existing cells 2 and 3 at the west end of the common dike. Due to topographic constraints a pump station is required to lift wastewater from cell 4 to the lagoon effluent polishing process. Therefore, this alternative includes the construction of a transfer pump station. The pump station would draw suction directly from the existing cell 4 and discharge to the polishing unit(s).

As described above, the City would install a lagoon effluent polishing process to further reduce effluent BOD and TSS concentrations prior to discharge to meet the effluent mass load limits. The proposed polishing process consists of DAF clarifiers followed by media filters. The DAF clarifiers and media filter process equipment would be located under a new above grade steel cover sharing a common wall with the new operations building. The media filter modules can be located outside of the covered area to reduce costs. The operations building would include an office, a lab, a water closet, electrical room, chemical storage room, and compressed air room. From the DAF clarifiers, effluent would enter the media filters. From the media filters effluent would pass through the new chlorine contact chamber. As part of the filter backwash process a backwash pump would be required to pump water from the chlorine contact chamber to backwash the filters. Sludge and backwash removed from the DAF and filters process would be routed to a plant pump station and pumped to the new lagoon cell 1. From the chlorine contact chamber disinfected wastewater would be dechlorinated and monitored at a compliance manhole. Treated effluent would then be routed to a new multi port outfall and discharged into the Yamhill River during the winter discharge period.

7.10.2 Alternative 2 – New Facultative Lagoons/Summer-Hold Winter Discharge & Polish Lagoon Effluent

This alternative is similar to Alternative 1 with the exception that none of the existing lagoons or treatment plant will be used. Please refer to Figure 7-2. The new lined facultative lagoons will be placed at a higher elevation in order to eliminate the need for a transfer pump station to lift the wastewater from the lagoons to the polishing process. All lagoon effluent is routed through a polishing process consisting of DAF clarifiers followed by media filters 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. Under this alternative two new lagoons cells are proposed. These consist of a new 24-acre cell 1 followed by a 10-acre cell 2. The lagoons would also include ancillary improvements such as, a headworks, discharge piping, and transfer structures. Due to size constraints this alternative would be constructed outside the City UGB which would require land use approval from the County prior to construction.

7.10.3 Alternative 3 – New Aerated & Facultative Lagoon Expansion/ Summer-Hold Winter Discharge & Polish Lagoon Effluent

This alternative is essentially the same as Alternative 1 with the addition of two, 0.65-acre aerated lagoons and a deep facultative lagoon (10-ft active storage) installed between the new headworks and the existing lagoon cell 1. Please refer to Figure 7-3 for an illustration of this alternative. The aerated lagoons remove a substantial amount of BOD allowing the new facultative lagoons to be constructed deeper which reduces the overall footprint of the new facultative lagoon. Effluent leaving the aerated lagoons would pass into the new 12-ft deep (10-ft of active storage), 16-acre lagoon. From the new 16-acre lagoon effluent is then transferred by gravity into the existing lagoon cell 1. From this point on, this alternative is identical to Alternative 1 as it includes utilizing the existing lagoons, a new transfer pump station, new DAF and media filter polishing processes, a new chlorine contact chamber, new disinfection and dechlorination equipment, a new operations building, new outfall piping, and a new outfall diffuser.

7.11. EVALUATION OF PRINCIPLE TREATMENT SYSTEM ALTERNATIVES

As described above, three principal alternatives have been identified to address treatment system deficiencies. In this subsection, each alternative is compared to arrive at the best treatment plan. A cost estimate was completed to compare the capital 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 8. 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 G. Since these alternatives have nearly the same operational scheme, operation and maintenance costs were not included in the cost estimate. However, a qualitative discussion of the differences in operation and maintenance costs is included in the following section. The results of the capital cost analysis are presented in Table 7-8. The advantages and disadvantages of each alternative are listed in Table 7-9.

Table 7-8 | Cost Comparison of Principal Treatment Alternatives

Alternative	Total Estimated Project Cost
1. Expand Existing Plant/Polish Effluent/Continue Summer-Hold Winter Discharge	$9,473,000
2. New Facultative Lagoons/Polish Effluent/Continue Summer-Hold Winter Discharge	$10,834,000
3. New Aerated & Facultative Lagoon Expansion/Polish Effluent/Continue Summer-Hold Winter Discharge	$9,387,000

(1) Costs are in 2011 dollars and assume dry weather construction, publicly bid project, ENR 20 cities index = 9,103. See Section 3.6 for basis of project cost estimates (i.e., 10% construction contingency, 20% engineering, 5% legal, permits, easements and administration).

Table 7-9 | Comparison of Principal Treatment Alternatives

Alternatives	
Advantages	**Disadvantages**
1. Expand Existing Plant/ Polish Effluent/Continue Summer - Hold Winter Discharge	
• Filtering step could be deferred to later in the planning period • Lower power cost (no aerated lagoons) • Utilizes existing lagoons	• Operational complexity • Higher risk of seepage problems in the existing lagoons
2. New Facultative Lagoons/Polish Effluent/Continue Summer - Hold Winter Discharge	
• Filtering step could be deferred to later in the planning period • Low risk of lagoon seepage problems from existing lagoons • Lowest power costs (no aerated lagoons or transfer pump station)	• Higher capital cost • Higher land costs
3. New Aerated & Facultative Lagoon Expansion/Polish Effluent/Continue Summer-Hold Winter Discharge	
• Filtering step could be deferred to later in the planning period • Smaller footprint • Less land acquisition	• Higher operational complexity • Higher risk of seepage problems in the existing lagoons • Higher power costs • Higher maintenance costs

Alternative 2 is the highest capital cost alternative, but the lowest operation costs due to the lack of aeration lagoons and the transfer pump station. Another major advantage of this alternative is that it does not rely on the existing lagoons. Seepage from the existing lagoons may be viewed as a liability for the City. The new lagoons wound be constructed with synthetic liners which would have very low seepage rates. The existing lagoons could be decommissioned and removed from service thereby eliminating the seepage problem. Aside from capital cost, another major drawback of Alternative 2 is that it has the largest footprint and land acquisition costs. Alternative 2 was eliminated due to this higher overall cost.

Alternatives 1 and 3 operate in a similar manner. They are summer hold winter discharge facilities that use DAF clarifiers and media filters to polish the lagoon effluent. The main difference between these alternatives is that Alternative 3 includes aerated lagoons and a smaller overall lagoon footprint. Alternative 1 includes only facultative lagoons with no aeration equipment. Alternatives 1 and 3 are similar in overall costs with Alternative 3 being slightly less. However, the difference in cost is relatively insignificant and would be offset by the higher Operation and Maintenance costs associated with the aerated lagoons included in Alternative 3. Based on today's power costs, the aerated lagoons in Alternative 3 will consume approximately $14,000 of power each year, an annual cost that Alternative 1 does not require. Therefore, over the planning period Alternative 1 is considered to be the more cost effective option.

As demonstrated in Table 7-8, Alternative 3 has the lowest capital costs. However, as previously discussed this alternative also has the highest operation and maintenance costs due to the aerated lagoons. Alternative 3 has the smallest footprint and therefore, the lowest land cost. The cost estimates are based land acquisition costs of $5,000 per acre. If the land costs are greater than the this assumed purchase price, Alternative 3 may potentially be more cost effective over the planning period.

As shown in Figure 7-1 (Alternative 1) and Figure 7-3 (Alternative 3) there is an alternative location for the new WWTP building, DAF equipment cover, filters and chlorine contact chamber. Therefore, these alternatives allow for some flexibility in the final location of these facilities. Also, in both Alternatives 1 and 3 there is a ±5-acre gravel area that is currently used for storage along the east side of the property. If the acquisition costs of this area are substantial the WWTP improvements could be expanded to the north in both Alternative 1 and 3.

7.12. RECOMMENDED TREATMENT PLAN

Due to lower life cycle costs and ease of operation, Alternative 1 is the recommended treatment plan. A schematic layout for the recommended improvements is included in Figure 7-1. It is recommended that improvements be constructed in two phases. A detailed cost estimate that lists each of components included in each phase is included in Table 7-11. Phase I includes all the treatment plant components except for the construction of the media filters. Phase II includes the construction of media filters. The purpose of phase I is to increase the organic treatment and hydraulic storage capacity of the plant in order to meet the new NPDES permit. The existing treatment facilities lack organic treatment and hydraulic storage capacity. Therefore, the phase I improvements are required early in the planning period. Based on the population growth and flow projections described in Section 5, the Phase I improvements should enable the City to comply with the NPDES discharge permit limits until approximately 2020. Beyond 2020, the WWTP may have trouble producing effluent of the needed quality at certain times of the year due to the limitations of the DAF clarifiers. If this proves to be the case, the media filters can be added to enable the City meet the permit limits for the remainder of the planning period. This phased approach provides the City with some flexibility in the event that growth is slower than anticipated.

Design Criteria for the recommended treatment plant improvements are listed in Table 7-10.

Table 7-10 | Design Data for Recommended Treatment Plant Improvements

Design Parameter	Design Criteria
Influent Flow Data (2035)	Refer to Table 5-5
Effluent Quality	
Required Effluent Quality (2035)	BOD5<11.3 mg/L, TSS<18.0 mg/L
Anticipated Effluent Quality (DAF & Filters)	BOD5<10 mg/L, TSS<10 mg/L
Headworks	
Flow Measurement	
Flume Size & Type	12" Parshall Flume
Peak Flow Capacity	10.43 MGD
Minimum Flow Capacity	0.0777 MGD
Transfer Pump Station	
Number of Pumps	3 with VFD's
Capacity	1400 gpm w/ all three pumps
Lagoons	
Cell 1 (Proposed)	
Surface Area	24 acres
Maximum Depth	8.5 feet
Working Depth	6 feet
Storage Volume	157 ac-ft
Lagoon Liner	HDPE
Cell 2, 3, & 4 (1980)	Refer to Table 4-6
Dissolved Air Flotation (DAF)	
Manufacturer	To Be Determined
Capacity	2 MGD
Number of units	2
Tank Diameter	25 feet
Hydraulic loading rate	1.9 gpm/ft^2
Chemical Feed	
Coagulant Type	To Be Determined
Feed Rate	To Be Determined
Operating Parameters	To Be Determined
Recycle Pumps	
Number & Horse Power	2, 15 HP
Gravity Filters	
Manufacturer	To Be Determined
Capacity	2 MGD
Number of Filters	4
Filtration Area (per tank)	95 ft^2
Design Filtration Rate (per tank)	4.7 gpm/ft^2
Media	
Depth & Type	4 inches gravel, 12 inches silica sand, 24 inches anthracite
Backwash	
Backwash Rate	15 gpm/ ft^2
Backwash Flow (1 filter)	1,425 gpm
Backwash Pump Number & Size	2, To Be Determined
Air Scour	
Air Scour Rate	3.0 scfm/ ft^2
Air Scour Flow (1 filter)	285 scfm
Disinfection	
Type	Sodium Hypochlorite
Feed Rate	To Be Determined
Chlorine Contact Chamber	
Volume	42,000 gallons
Contact Time	30 minutes
Maximum Flow Rate	2 MGD
Dechlorination	
Feed Solution	Calcium Thiosulfate
Feed Rate	To Be Determined

Table 7-11 | Recommended Treatment System Improvements

Lined Facultative Lagoon Expansion/ Polish Effluent/Continue Summer - Hold Winter Discharge	Total Estimated Project Cost (1)
Phase I WWTP Improvements	
Construction Costs	
Mobilization & Bond (8%)	$455,000
New headworks, flow measurement and sampling	$125,000
New facultative lagoon with synthetic liner and fencing	$1,680,000
New distribution piping (from new headworks to new lagoon)	$110,000
New lagoon transfer structures (2)	$130,000
New transfer piping (from lagoon to lagoon)	$44,000
Existing lagoon dike roadway rehabilitation	$60,000
Repair Leak in Lagoon 4	$100,000
Transfer pump station & controls	$350,000
New transfer piping (from exist. lagoon 4 to DAF & filters)	$60,000
New 3-phase power service	$50,000
Plant Office, DAF Equipment Cover & Site Work	$847,000
DAF Equipment & Piping	$781,000
Chemical Feed Equipment (Coagulant, Chlorine, disinfection)	$217,000
Plant pump station	$250,000
Plant pump station piping	$113,000
New Auxiliary power unit with automatic transfer switch	$150,000
New chlorine contact chamber	$339,000
New outfall piping	$117,000
New outfall and diffuser	$115,000
New SCADA system for Wastewater Utility	$50,000
Construction Cost Subtotal	**$6,143,000**
Soft Costs	
Purchase land for improvements	$180,000
Construction Contingency (10%)	$614,000
Engineering (20%)	$1,229,000
Legal, Administration & Permitting (5%)	$307,000
Soft Cost Subtotal	**$2,330,000**
Total Phase I WWTP Improvements	**$8,473,000**
Phase II WWTP Improvements	
Construction Costs	
Mobilization & Bond	$55,000
Filter Equipment & Piping	$686,000
Construction Cost Subtotal	**$741,000**
Soft costs	
Construction Contingency (10%)	$74,000
Engineering (20%)	$148,000
Legal, Administration & Permitting (5%)	$37,000
Soft Cost Subtotal	**$259,000**
Alternative 1 Phase II WWTP Improvements	**$1,000,000**
Grand Total Phase I & Phase II WWTP Improvements	**$9,473,000**

(1) Costs are in 2011 dollars and assume dry weather construction, publicly bid project, ENR 20 cities index = 9,103. See Section 8.2 for basis of project cost estimates (i.e., 10% construction contingency, 20% engineering, 5% legal, permits, easements and administration).

Table 7-7 | Principal Treatment System Alternatives

Alternatives

Deficiency	Method of Addressing Deficiency
1. Expand Existing Plant/ Continue Summer-Hold Winter Discharge & Polish Lagoon Effluent	
Hydraulic Capacity	Additional lagoons for combined storage.
Organic Capacity	Additional facultative lagoon with synthetic liner. Polish effluent to meet mass load limits.
Distribution Piping Capacity	Utilize existing transfer piping between lagoons. New lagoon constructed upstream of existing primary lagoon will require the installation of new discharge piping and new transfer structure and piping. A transfer pump station will also be required to transfer water from lagoon 4 to the DAF and filters.
Disposal Method	Polish lagoon effluent to continue summer-holding winter-discharge and retain existing mass load limits. A new outfall would be required.
2. New Facultative Lagoons/Continue Summer-Hold Winter Discharge & Polish Lagoon Effluent	
Deficiency	Method of Addressing Deficiency
Hydraulic Capacity	New lagoons for storage (controls lagoon size).
Organic Capacity	New facultative lagoons with synthetic liner & polished effluent to meet mass load limits.
Distribution Piping Capacity	N/A. New lagoons will require the installation of new distribution piping discharge piping and new transfer structures. Will require additional effluent pump station.
Disposal Method	Polish lagoon effluent to continue summer-holding winter-discharge and retain existing mass load limits. A new outfall would be required.
3. New Aerated Lagoons & Facultative Lagoon Expansion/Continue Summer-Hold Winter Discharge & Polish Lagoon Effluent	
Deficiency	Method of Addressing Deficiency
Hydraulic Capacity	Additional lagoons for combined storage.
Organic Capacity	Additional Aerated primary lagoon followed by deep facultative lagoons with synthetic liner. Polish effluent to meet mass load limits.
Distribution Piping Capacity	Utilize existing distribution piping between lagoons. . New lagoon constructed upstream of existing primary lagoon will require the installation of new discharge piping and new transfer structure and piping. Will require additional effluent pump station.
Disposal Method	Polish lagoon effluent to continue summer-holding winter-discharge and retain existing mass load limits. A new outfall would be required.

7.10.1 Alternative 1 – Expand Existing Plant / Summer-Hold Winter Discharge & Polish Lagoon Effluent

Under this alternative the City would expand the existing WWTP. Please refer to Figure 7-1. This alternative includes constructing one new facultative lagoon with a synthetic liner. In combination with the existing lagoons, the additional lagoon will provide the hydraulic storage capacity through the planning period. In addition DAF clarifiers and media filters would be constructed downstream of lagoon 4 to polish the wastewater to meet the winter time discharge permit limits. Therefore, this alternative does not include summer land application facilities. This alternative also includes the construction of a new headworks, distribution piping to the new lagoon, a new transfer structure between the new lagoon cell and cell 1, a new structure between cell 2 and 3, and a new transfer pump station to pump effluent from existing cell 4 to the DAF clarifiers and media filters. This alternative also includes a new operations building, chlorine contact chamber, disinfection equipment, dechlorination equipment, and outfall piping, and outfall diffuser. Since the existing lagoon cells are to remain in service, this alternative includes repairing the leak through the north dike of cell 4. This alternative can be construction within the existing UGB.

Wastewater enters the new headworks that consists of a new 12-inch Parshall flume, sampling equipment, and flow monitoring equipment. From the headworks wastewater is then directed by gravity via transfer piping to the new 24-acre facultative lagoon with a synthetic liner. As shown in Table 7-1 and Table 7-2 the lagoon size is driven by hydraulics rather than BOD loading rate. The new lagoon is proposed with a synthetic geomembrane liner rather than a native clay liner to form a seepage barrier.

From the new lagoon wastewater would be transferred by gravity to the existing lagoon cell 1 and transferred in series to cells 2, 3 and 4. To avoid short circuiting, a new transfer structure is recommended between existing cells 2 and 3 at the west end of the common dike. Due to topographic constraints a pump station is required to lift wastewater from cell 4 to the lagoon effluent polishing process. Therefore, this alternative includes the construction of a transfer pump station. The pump station would draw suction directly from the existing cell 4 and discharge to the polishing unit(s).

As described above, the City would install a lagoon effluent polishing process to further reduce effluent BOD and TSS concentrations prior to discharge to meet the effluent mass load limits. The proposed polishing process consists of DAF clarifiers followed by media filters. The DAF clarifiers and media filter process equipment would be located under a new above grade steel cover sharing a common wall with the new operations building. The media filter modules can be located outside of the covered area to reduce costs. The operations building would include an office, a lab, a water closet, electrical room, chemical storage room, and compressed air room. From the DAF clarifiers, effluent would enter the media filters. From the media filters effluent would pass through the new chlorine contact chamber. As part of the filter backwash process a backwash pump would be required to pump water from the chlorine contact chamber to backwash the filters. Sludge and backwash removed from the DAF and filters process would be routed to a plant pump station and pumped to the new lagoon cell 1. From the chlorine contact chamber disinfected wastewater would be dechlorinated and monitored at a compliance manhole. Treated effluent would then be routed to a new multi port outfall and discharged into the Yamhill River during the winter discharge period.

7.10.2 Alternative 2 – New Facultative Lagoons/Summer-Hold Winter Discharge & Polish Lagoon Effluent

This alternative is similar to Alternative 1 with the exception that none of the existing lagoons or treatment plant will be used. Please refer to Figure 7-2. The new lined facultative lagoons will be placed at a higher elevation in order to eliminate the need for a transfer pump station to lift the wastewater from the lagoons to the polishing process. All lagoon effluent is routed through a polishing process consisting of DAF clarifiers followed by media filters 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. Under this alternative two new lagoons cells are proposed. These consist of a new 24-acre cell 1 followed by a 10-acre cell 2. The lagoons would also include ancillary improvements such as, a headworks, discharge piping, and transfer structures. Due to size constraints this alternative would be constructed outside the City UGB which would require land use approval from the County prior to construction.

7.10.3 Alternative 3 – New Aerated & Facultative Lagoon Expansion/ Summer-Hold Winter Discharge & Polish Lagoon Effluent

This alternative is essentially the same as Alternative 1 with the addition of two, 0.65-acre aerated lagoons and a deep facultative lagoon (10-ft active storage) installed between the new headworks and the existing lagoon cell 1. Please refer to Figure 7-3 for an illustration of this alternative. The aerated lagoons remove a substantial amount of BOD allowing the new facultative lagoons to be constructed deeper which reduces the overall footprint of the new facultative lagoon. Effluent leaving the aerated lagoons would pass into the new 12-ft deep (10-ft of active storage), 16-acre lagoon. From the new 16-acre lagoon effluent is then transferred by gravity into the existing lagoon cell 1. From this point on, this alternative is identical to Alternative 1 as it includes utilizing the existing lagoons, a new transfer pump station, new DAF and media filter polishing processes, a new chlorine contact chamber, new disinfection and dechlorination equipment, a new operations building, new outfall piping, and a new outfall diffuser.

7.11. EVALUATION OF PRINCIPLE TREATMENT SYSTEM ALTERNATIVES

As described above, three principal alternatives have been identified to address treatment system deficiencies. In this subsection, each alternative is compared to arrive at the best treatment plan. A cost estimate was completed to compare the capital 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 8. 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 G. Since these alternatives have nearly the same operational scheme, operation and maintenance costs were not included in the cost estimate. However, a qualitative discussion of the differences in operation and maintenance costs is included in the following section. The results of the capital cost analysis are presented in Table 7-8. The advantages and disadvantages of each alternative are listed in Table 7-9.

Table 7-8 | Cost Comparison of Principal Treatment Alternatives

Alternative	Total Estimated Project Cost
1. Expand Existing Plant/Polish Effluent/Continue Summer-Hold Winter Discharge	$9,473,000
2. New Facultative Lagoons/Polish Effluent/Continue Summer-Hold Winter Discharge	$10,834,000
3. New Aerated & Facultative Lagoon Expansion/Polish Effluent/Continue Summer-Hold Winter Discharge	$9,387,000

(1) Costs are in 2011 dollars and assume dry weather construction, publicly bid project, ENR 20 cities index = 9,103. See Section 3.6 for basis of project cost estimates (i.e., 10% construction contingency, 20% engineering, 5% legal, permits, easements and administration).

Table 7-9 | Comparison of Principal Treatment Alternatives

Alternatives	
Advantages	**Disadvantages**
1. Expand Existing Plant/ Polish Effluent/Continue Summer - Hold Winter Discharge	
• Filtering step could be deferred to later in the planning period • Lower power cost (no aerated lagoons) • Utilizes existing lagoons	• Operational complexity • Higher risk of seepage problems in the existing lagoons
2. New Facultative Lagoons/Polish Effluent/Continue Summer - Hold Winter Discharge	
• Filtering step could be deferred to later in the planning period • Low risk of lagoon seepage problems from existing lagoons • Lowest power costs (no aerated lagoons or transfer pump station)	• Higher capital cost • Higher land costs
3. New Aerated & Facultative Lagoon Expansion/Polish Effluent/Continue Summer-Hold Winter Discharge	
• Filtering step could be deferred to later in the planning period • Smaller footprint • Less land acquisition	• Higher operational complexity • Higher risk of seepage problems in the existing lagoons • Higher power costs • Higher maintenance costs

Alternative 2 is the highest capital cost alternative, but the lowest operation costs due to the lack of aeration lagoons and the transfer pump station. Another major advantage of this alternative is that it does not rely on the existing lagoons. Seepage from the existing lagoons may be viewed as a liability for the City. The new lagoons wound be constructed with synthetic liners which would have very low seepage rates. The existing lagoons could be decommissioned and removed from service thereby eliminating the seepage problem. Aside from capital cost, another major drawback of Alternative 2 is that it has the largest footprint and land acquisition costs. Alternative 2 was eliminated due to this higher overall cost.

Alternatives 1 and 3 operate in a similar manner. They are summer hold winter discharge facilities that use DAF clarifiers and media filters to polish the lagoon effluent. The main difference between these alternatives is that Alternative 3 includes aerated lagoons and a smaller overall lagoon footprint. Alternative 1 includes only facultative lagoons with no aeration equipment. Alternatives 1 and 3 are similar in overall costs with Alternative 3 being slightly less. However, the difference in cost is relatively insignificant and would be offset by the higher Operation and Maintenance costs associated with the aerated lagoons included in Alternative 3. Based on today's power costs, the aerated lagoons in Alternative 3 will consume approximately $14,000 of power each year, an annual cost that Alternative 1 does not require. Therefore, over the planning period Alternative 1 is considered to be the more cost effective option.

As demonstrated in Table 7-8, Alternative 3 has the lowest capital costs. However, as previously discussed this alternative also has the highest operation and maintenance costs due to the aerated lagoons. Alternative 3 has the smallest footprint and therefore, the lowest land cost. The cost estimates are based land acquisition costs of $5,000 per acre. If the land costs are greater than the this assumed purchase price, Alternative 3 may potentially be more cost effective over the planning period.

As shown in Figure 7-1 (Alternative 1) and Figure 7-3 (Alternative 3) there is an alternative location for the new WWTP building, DAF equipment cover, filters and chlorine contact chamber. Therefore, these alternatives allow for some flexibility in the final location of these facilities. Also, in both Alternatives 1 and 3 there is a ±5-acre gravel area that is currently used for storage along the east side of the property. If the acquisition costs of this area are substantial the WWTP improvements could be expanded to the north in both Alternative 1 and 3.

7.12. RECOMMENDED TREATMENT PLAN

Due to lower life cycle costs and ease of operation, Alternative 1 is the recommended treatment plan. A schematic layout for the recommended improvements is included in Figure 7-1. It is recommended that improvements be constructed in two phases. A detailed cost estimate that lists each of components included in each phase is included in Table 7-11. Phase I includes all the treatment plant components except for the construction of the media filters. Phase II includes the construction of media filters. The purpose of phase I is to increase the organic treatment and hydraulic storage capacity of the plant in order to meet the new NPDES permit. The existing treatment facilities lack organic treatment and hydraulic storage capacity. Therefore, the phase I improvements are required early in the planning period. Based on the population growth and flow projections described in Section 5, the Phase I improvements should enable the City to comply with the NPDES discharge permit limits until approximately 2020. Beyond 2020, the WWTP may have trouble producing effluent of the needed quality at certain times of the year due to the limitations of the DAF clarifiers. If this proves to be the case, the media filters can be added to enable the City meet the permit limits for the remainder of the planning period. This phased approach provides the City with some flexibility in the event that growth is slower than anticipated.

Design Criteria for the recommended treatment plant improvements are listed in Table 7-10.

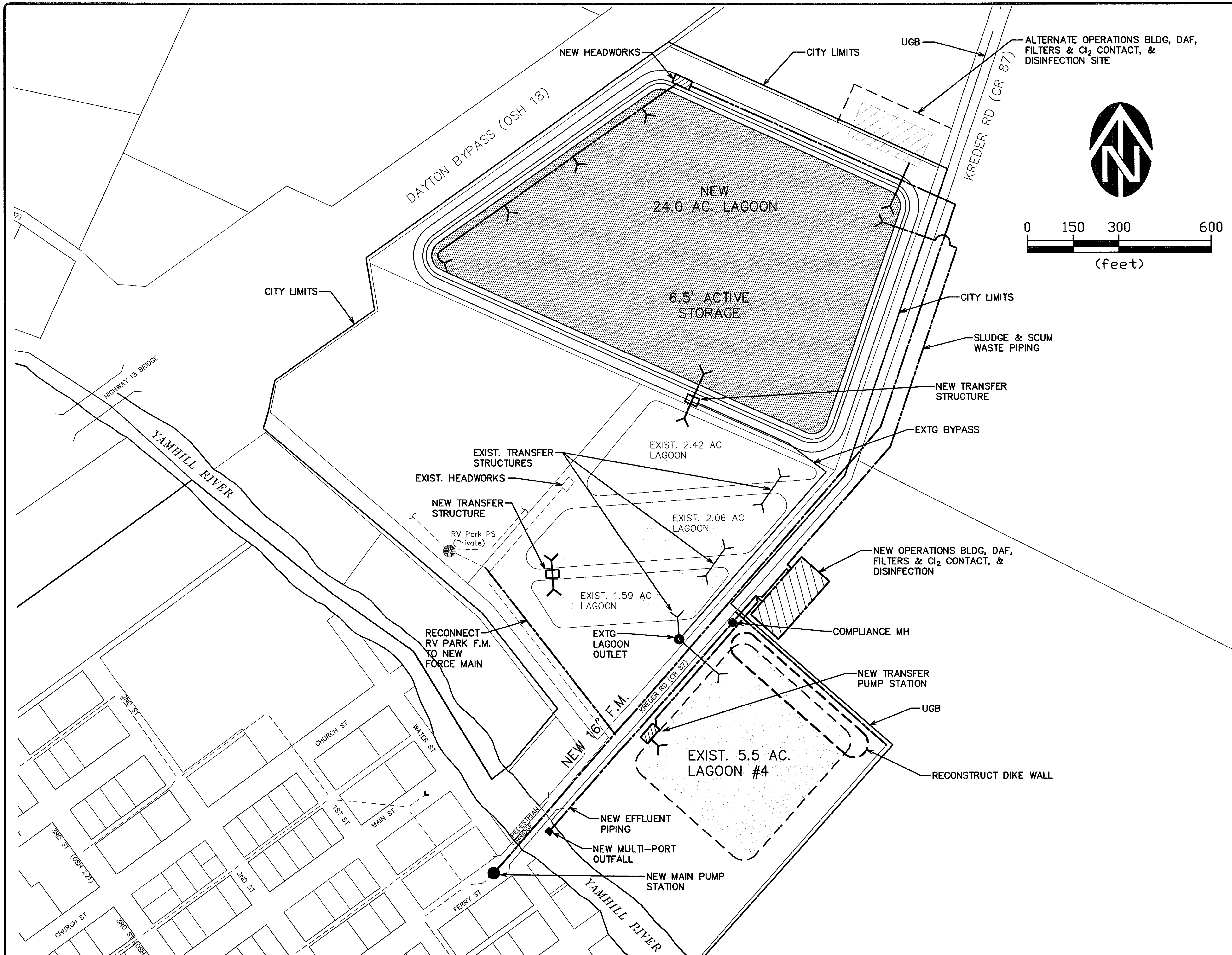
Table 7-10 | Design Data for Recommended Treatment Plant Improvements

Design Parameter	Design Criteria
Influent Flow Data (2035)	Refer to Table 5-5
Effluent Quality	
Required Effluent Quality (2035)	BOD5<11.3 mg/L, TSS<18.0 mg/L
Anticipated Effluent Quality (DAF & Filters)	BOD5<10 mg/L, TSS<10 mg/L
Headworks	
Flow Measurement	
Flume Size & Type	12" Parshall Flume
Peak Flow Capacity	10.43 MGD
Minimum Flow Capacity	0.0777 MGD
Transfer Pump Station	
Number of Pumps	3 with VFD's
Capacity	1400 gpm w/ all three pumps
Lagoons	
Cell 1 (Proposed)	
Surface Area	24 acres
Maximum Depth	8.5 feet
Working Depth	6 feet
Storage Volume	157 ac-ft
Lagoon Liner	HDPE
Cell 2, 3, & 4 (1980)	Refer to Table 4-6
Dissolved Air Flotation (DAF)	
Manufacturer	To Be Determined
Capacity	2 MGD
Number of units	2
Tank Diameter	25 feet
Hydraulic loading rate	1.9 gpm/ft^2
Chemical Feed	
Coagulant Type	To Be Determined
Feed Rate	To Be Determined
Operating Parameters	To Be Determined
Recycle Pumps	
Number & Horse Power	2, 15 HP
Gravity Filters	
Manufacturer	To Be Determined
Capacity	2 MGD
Number of Filters	4
Filtration Area (per tank)	95 ft^2
Design Filtration Rate (per tank)	4.7 gpm/ft^2
Media	
Depth & Type	4 inches gravel, 12 inches silica sand, 24 inches anthracite
Backwash	
Backwash Rate	15 gpm/ ft^2
Backwash Flow (1 filter)	1,425 gpm
Backwash Pump Number & Size	2, To Be Determined
Air Scour	
Air Scour Rate	3.0 scfm/ ft^2
Air Scour Flow (1 filter)	285 scfm
Disinfection	
Type	Sodium Hypochlorite
Feed Rate	To Be Determined
Chlorine Contact Chamber	
Volume	42,000 gallons
Contact Time	30 minutes
Maximum Flow Rate	2 MGD
Dechlorination	
Feed Solution	Calcium Thiosulfate
Feed Rate	To Be Determined

Table 7-11 | Recommended Treatment System Improvements

Lined Facultative Lagoon Expansion/ Polish Effluent/Continue Summer - Hold Winter Discharge	Total Estimated Project Cost (1)
Phase I WWTP Improvements	
Construction Costs	
Mobilization & Bond (8%)	$455,000
New headworks, flow measurement and sampling	$125,000
New facultative lagoon with synthetic liner and fencing	$1,680,000
New distribution piping (from new headworks to new lagoon)	$110,000
New lagoon transfer structures (2)	$130,000
New transfer piping (from lagoon to lagoon)	$44,000
Existing lagoon dike roadway rehabilitation	$60,000
Repair Leak in Lagoon 4	$100,000
Transfer pump station & controls	$350,000
New transfer piping (from exist. lagoon 4 to DAF & filters)	$60,000
New 3-phase power service	$50,000
Plant Office, DAF Equipment Cover & Site Work	$847,000
DAF Equipment & Piping	$781,000
Chemical Feed Equipment (Coagulant, Chlorine, disinfection)	$217,000
Plant pump station	$250,000
Plant pump station piping	$113,000
New Auxiliary power unit with automatic transfer switch	$150,000
New chlorine contact chamber	$339,000
New outfall piping	$117,000
New outfall and diffuser	$115,000
New SCADA system for Wastewater Utility	$50,000
Construction Cost Subtotal	**$6,143,000**
Soft Costs	
Purchase land for improvements	$180,000
Construction Contingency (10%)	$614,000
Engineering (20%)	$1,229,000
Legal, Administration & Permitting (5%)	$307,000
Soft Cost Subtotal	**$2,330,000**
Total Phase I WWTP Improvements	**$8,473,000**
Phase II WWTP Improvements	
Construction Costs	
Mobilization & Bond	$55,000
Filter Equipment & Piping	$686,000
Construction Cost Subtotal	**$741,000**
Soft costs	
Construction Contingency (10%)	$74,000
Engineering (20%)	$148,000
Legal, Administration & Permitting (5%)	$37,000
Soft Cost Subtotal	**$259,000**
Alternative 1 Phase II WWTP Improvements	**$1,000,000**
Grand Total Phase I & Phase II WWTP Improvements	**$9,473,000**

(1) Costs are in 2011 dollars and assume dry weather construction, publicly bid project, ENR 20 cities index = 9,103. See Section 8.2 for basis of project cost estimates (i.e., 10% construction contingency, 20% engineering, 5% legal, permits, easements and administration).



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NO.	DATE	DESCRIPTION	BY
1		REVISIONS	

SCALE
HORIZ:
VERT:
DSN. JW
DRN. TMT/BF
CKD. JW
DATE: DEC 2011

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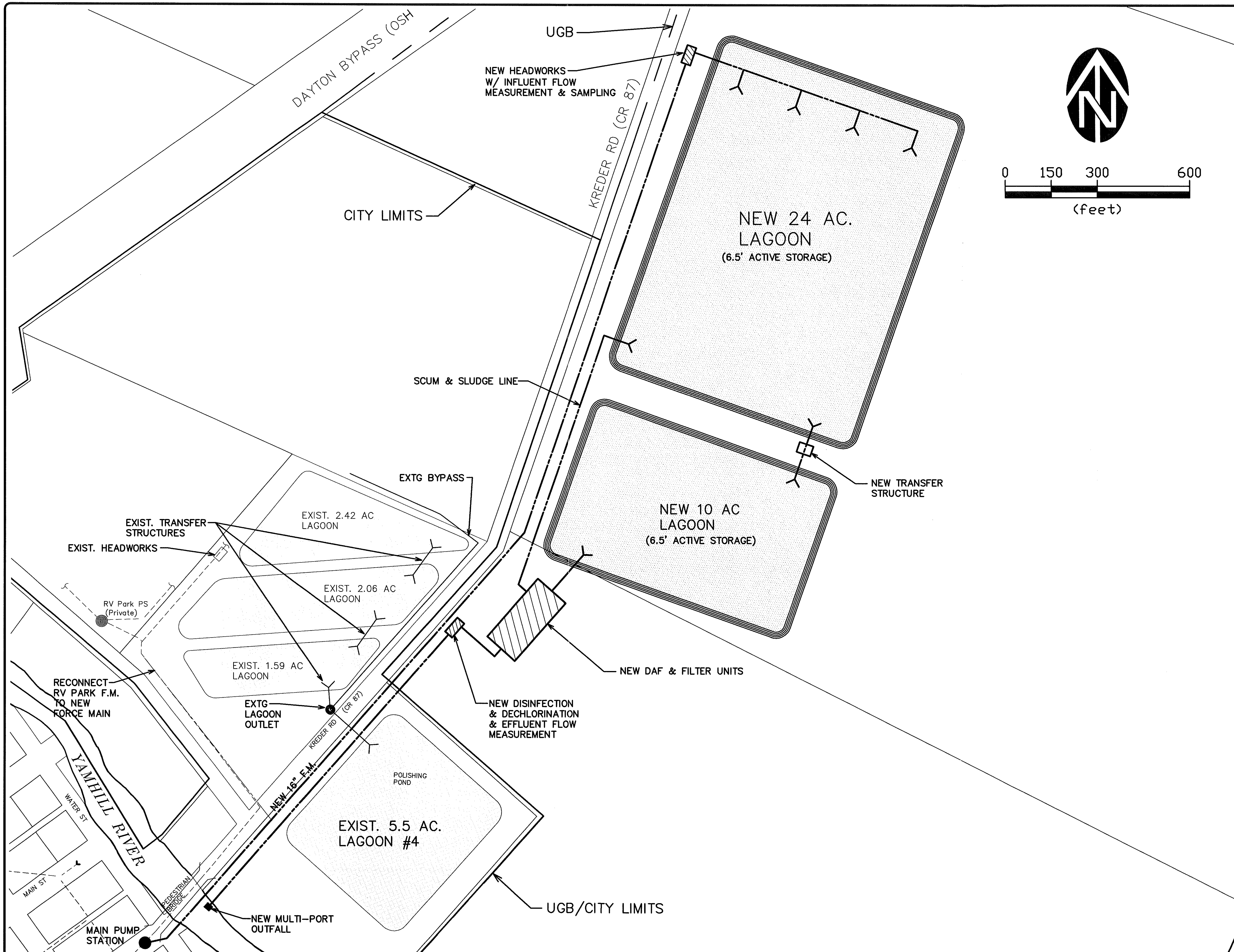
CITY OF DAYTON, OREGON

ALTERNATIVE #1
EXPAND EXISTING PLANT W/ EFFLUENT POLISHING

FIGURE
7-1

JOB NUMBER
2609.3010.0



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ALTERNATIVE #2

NEW LAGOONS W/ EFFLUENT POLISHING

FIGURE 7-2

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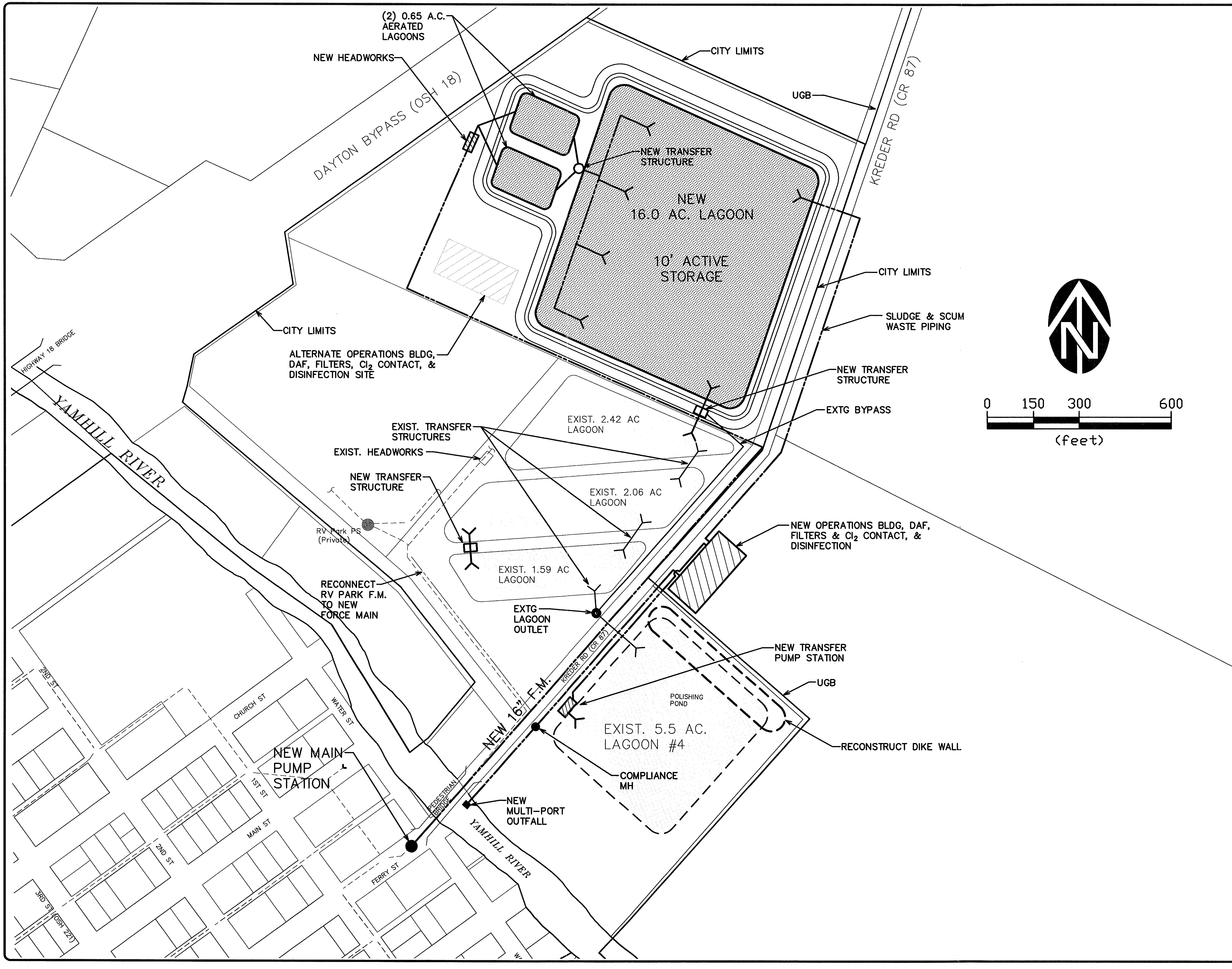

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ALTERNATIVE #3
AERATED LAGOONS W/ EFFLUENT POLISHING

FIGURE 7-3

JOB NUMBER 2609.3010.0