CHAPTER 7

LIQUID STREAM TREATMENT ALTERNATIVES

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The liquid stream treatment facilities at the Lebanon WWTP are currently able to satisfy most of the requirements set forth in its National Pollutant Discharge Elimination System (NPDES) permit. For those permit requirements that the plant is not able to meet, the City follows the requirements of a Mutual Agreement and Order (MAO) issued by the Department of Environmental Quality (DEQ). The MAO also includes a schedule for the completion of process improvements that will address water quality concerns in the South Santiam River. In addition to the process improvements required by the DEQ, long term upgrades are necessary to ensure that the facilities can handle increased flows and loads from Lebanon's growing population as well as comply with potentially more restrictive future permit requirements. The planning and implementation of these improvements will ensure that the Lebanon WWTP continues to satisfy its permit requirements in the years to come.

The wastewater characteristics analysis contained in Chapter 5 provides the flow and load projections used in the development of the following liquid stream treatment alternatives.

CATEGORIES OF IMPROVEMENTS

Three general factors will trigger the need to upgrade liquid stream treatment processes:

- Existing and future NPDES permit requirements.
- Higher peak flows due to deterioration of the collection system and construction of new sewers in an expanding service area.
- Increased organic loads due to population growth and industrial development.

There are two specific regulatory issues contained in the City's current MAO that affect the liquid stream improvement plan. The MAO requires the City to develop a plan for addressing ammonia toxicity issues, most likely by upgrading the effluent discharge system on the South Santiam River to improve mixing. Also, the City must greatly reduce chlorine residuals in order to eliminate the potential for chlorine toxicity in the river.

More stringent future NPDES permit requirements will also be an issue for Lebanon. Although the addition of nutrient removal requirements are unlikely, a temperature management program will be required. The current permit required that the City submit a temperature management plan to the DEQ for approval by February 2002 which has been accomplished. The plan assessed the South Santiam River with respect to the temperature standard and evaluated the impact of the plant effluent on river temperature.

Changes in the characteristics of the WWTP's service area also motivate improvements. Community growth patterns in Lebanon indicate that flows and loads at the WWTP will steadily increase over the next twenty years. As projected in Chapter 5, plant flows and loads are expected to be approximately 50 percent higher than current levels by the year 2024. Therefore, some plant upgrades will be necessary to ensure that the treatment capacity of each unit process is adequate for the projected flows and loads.

ANALYSIS OF LIQUID STREAM IMPROVEMENTS BY UNIT PROCESS

Several of the liquid stream unit processes at the Lebanon WWTP will require improvements over the next twenty years. The following sections analyze alternatives for potential improvements at each liquid stream process. For those unit processes that have alternatives for improvement, an evaluation of alternatives is also included.

Raw Sewage Pump Station

The existing raw sewage pump stations have a combined firm capacity of approximately 30 mgd which is more than adequate for the projected year 2024 peak wet weather flow of 26 mgd. The new raw sewage pump station (constructed in 2002) serves the new West Side Interceptor and recently added 12 mgd of firm influent pumping capacity.

As discussed earlier, the raw sewage pump station includes a magnetic flow meter. The new meter was installed on a portion of the force main common to both pump stations. This new flow meter alleviates the need for the Parshall flume flow meter located in the secondary effluent distribution structure. As a result, the Parshall flume should be removed since it has a maximum capacity of just 16 mgd and will therefore constrict peak flows.

Headworks

The WWTP headworks consists of one mechanical bar screen and two manual bar screens. Currently, operators must rely on both the mechanical and manual screens to accommodate peak wet weather flows. The combined capacity of these screens is sufficient for existing and future peak flows only if the manual screens are regularly cleaned during storm conditions. Because the plant is not staffed 24 hours per day, the capacity of the headworks is equal to that of the mechanically cleaned bar screen, which is 7 mgd. Since the current PWWF is estimated at 21 mgd and future PWWF at 26 mgd, a capacity expansion of the headworks is necessary.

Expansion of the headworks can take place in two different ways: renovation of the existing facilities or construction of a new headworks. The following discussion presents these two alternatives.

Alternative 1—Renovation of Existing Headworks. Renovation of the existing headworks would entail the addition of a second mechanical bar screen and structural modifications to allow for a new flow path and improved hydraulic conditions. These upgrades would provide adequate capacity to accommodate projected peak flows for the next twenty years.

Although there are significant space constraints at the headworks, it appears feasible to add another mechanical bar screen to the existing headworks bypass channel. Installing bar screens in the two channels with manually raked screens would not provide adequate overall capacity to treat 26 mgd. Since the bypass channel is 1-foot wider and 2.5-feet deeper than the existing bar screen channels, the addition of a screen at this location would greatly increase the headworks capacity. The addition of a bar screen would require the following renovations:

- Structural modifications to convert the bypass channel into a fourth bar screen channel.
- Rearrangement of the screenings washer/compactor equipment.
- Addition of a high level sensor and motorized operators to the manual screen isolation gates for gate control under emergency conditions.
- Removal of the headworks bypass weir.
- Installation of a sluice gate for control of flow to the fine screens.
- Modification of electrical systems to comply with Class I, Division II requirements.

Two other structural modifications are necessary to improve the capacity and performance of the existing headworks facilities. Raising the allowable water level by 6-inches would allow for greater depths of flow through the bar screens and provide an additional increment of capacity. Also, the addition of a flow control mechanism downstream of the screens, is necessary to maintain proper velocities. An easily integrated flow control feature would be the construction of a pair of wing walls that protrude into the downstream channel to constrict flow.

Hydraulic analysis of this new headworks configuration shows that the capacity of the existing mechanical bar screen would increase to 9 mgd and the capacity of the new mechanical bar screen would be 23 mgd for a total headworks capacity of 32 mgd. This capacity is more than sufficient to meet projected peak flows through the year 2024. The design data for this alternative is included in Table 7-1 and estimated costs are presented in Table 7-2.

Table 7-1. Design Data
Headworks Alternative 1

Item	Value	
	Current	Year 2024
Mechanically cleaned bar screens		
Number	1	2
Channel width, feet	3	3, 4
Maximum flow[a], each, mgd	7	9, 23
Manually cleaned bar screens		
Number	2	2
Channel width, feet	2.5, 3	2.5, 3
Maximum flow[a], each, mgd	11, 14	14, 17

[a]Screen capacity based on maximum water velocity through the bars of 4 ft/s.

Table 7-2. Capital Cost
Headworks Alternative 1—Renovation

Description	Cost, $1,000
Headworks Renovation	482
Contingencies	96
Construction Cost	578
Engineering and Administration	115
Total Capital Cost	693

Alternative 2—Construction of a Replacement Headworks. Given the space constraints at the existing headworks facility and limited room adjacent to the headworks for future expansion, the construction of a replacement headworks is an alternative that would benefit long-term planning at the Lebanon WWTP. The renovations described above may be sufficient only for a portion of the planning period since construction of future aeration basin facilities will likely require a new headworks with grit removal facilities. In contrast, construction of a replacement headworks would be planned to allow plant expansions through build-out. A replacement headworks could also be located and designed to accommodate future modifications to the existing aeration basins such as raising of the basin walls to increase treatment capacity.

The capacity of the initial phase of the new headworks would be 26 mgd. The design would likely include just a single mechanical screen and a single manual screen of equivalent total capacity. Table 7-3 presents a cost estimate for the new facility which would integrate the screenings washer/compactor from the existing headworks.

Table 7-3. Capital Cost
Headworks Alternative 2—New Construction

Description	Cost, $1,000
New Construction	847
Contingencies	169
Construction Cost	1,016
Engineering and Administration	203
Total Capital Cost	1,219

Evaluation of Alternatives. Renovation of the existing headworks is clearly the least expensive alternative for providing the necessary screening capacity. However, since the construction of improvements to the headworks should be coordinated with future expansion plans for the aeration basins, it is recommended that an aeration basin expansion plan be developed prior to the design of the headworks improvements. It is very likely that the headworks can be renovated in a manner that will work well with whichever aeration basin expansion plan the City selects.

Operation and maintenance costs for the two headworks alternatives are the same. In terms of reliability, both options are comparable. Delaying construction of the new headworks provides a better opportunity to integrate the future headworks with the future aeration system expansion. For this reason, adding a screen to the existing headworks is the recommended alternative.

Grit Removal

The WWTP currently has no facilities for removing grit from the wastewater. In the absence of a grit removal system, the grit content of sludge is likely to be relatively high, increasing wear on the solids handling equipment and allows grit in the aerobic digesters. Under existing operations, grit is manually removed from the aeration basins during the dry weather period when flows are low and each basin can be rotated out of service for cleaning. This cleaning practice will eventually become more difficult in the future as loading levels limit the ability of operators to take a basin out of service. However, it is estimated that average dry weather flows at the plant will not exceed the capacity of one basin until the year 2021, just before the end of the planning period.

The addition of grit removal facilities at the WWTP can improve operations in several ways. A grit removal system would reduce cleaning requirements in the aeration basins, extend the useful life of solids handling equipment, and alleviate the loss of aerobic digester volume to settled grit. A grit chamber would also allow for the future installation of fine bubble diffusers in the aeration basins.

The most cost-effective system for grit removal would be a vortex chamber located downstream of the bar screens. Aerated grit chambers tend to be approximately 30 percent more expensive. To accommodate the year 2024 peak flow of 26 mgd, at least two grit chambers would be required. Due to space constraints at the existing headworks, this type of grit removal system would likely be located in the vicinity of the existing fine screens. The grit chambers would be located in an elevated concrete structure to allow for a hydraulic drop into the existing aeration basins. The estimated costs for a vortex grit chamber are included in Table 7-4.

Table 7-4. Capital Cost
Grit Removal System

Description	Cost, $1,000
Vortex Grit Chambers	680
Contingencies	136
Construction Cost	816
Engineering and Administration	163
Total Capital Cost	979

The addition of grit removal facilities to the treatment system is only critical if fine bubble diffusers are installed in the downstream aeration basins. Otherwise, the benefits of grit removal do not warrant the cost. Therefore, evaluation of the need for a grit removal system must be made within the context of the planning for the aeration system, which is included in the following section.

Aeration Basins

There are several different alternatives for upgrading the treatment capacity of the aeration basins to ensure adequate treatment of future flows and loads. In addition to expanding the capacity of this unit process, the aeration basin upgrades should include modifications that allow for different modes of operation to improve plant performance during various flow and loading conditions. Following is a description of upgrades that would improve operational flexibility as well as alternatives for expanding the treatment capacity of the aeration basins.

Upgrades to Improve Operational Flexibility. The aeration basins currently operate as a plug flow process with high mixing energy. Return activated sludge (RAS) is mixed with the raw sewage just downstream of the bar screens to create mixed liquor. The mixed liquor enters the aeration basins where it is mixed and aerated and then discharges to the clarifiers at the downstream end of the basins. In the plug flow mode of operation, the solids in the aeration basins are vulnerable to wash-out during peak flow events. Solids wash-outs can overload secondary clarifiers and create performance problems. To improve plant performance during high flow conditions, many treatment plants have the ability to introduce raw sewage at a downstream location in the aeration basins as well. This mode of operation is known as sludge reaeration or contact stabilization. The ability to operate in this mode allows the plant to keep the solids inventory in the upstream portion of the aeration basins isolated from the main flow path, thus protecting the secondary clarifiers from a severe solids wash-out. Since sludge reaeration offers increased treatment capacity for a given basin volume, upgrades to allow this mode of operation are critical for extending the treatment capacity of the existing aeration basins. Although treatment performance for this mode of operation is generally not as good as plug flow, sludge reaeration would only occur during high flow conditions to avoid the solids wash-out that can occur while in the plug flow mode.

Although the addition of sludge reaeration capability is most important, there are other modes of operation that could be desirable as well. These operating modes include step feed, anaerobic selector, and anoxic selector.

Step feed mode provides a solids retention time (SRT) between that of plug flow and sludge reaeration. In this mode of operation, the RAS is introduced at the upstream end of the aeration basin and raw sewage is added at several different downstream locations. In switching between plug flow and sludge reaeration, WWTP operators often use step feed mode as an intermediate operating point to make the transition of SRT and mixed liquor concentrations more gradual. Again, while this mode of operation does not perform as well as plug flow, step feed would only be used during high flows to reduce risk of the solids wash-out that can occur while in plug flow mode.

In the anaerobic selector mode, RAS and raw sewage are combined in an unaerated compartment at the upstream end of the aeration basin. If the unaerated zone is anaerobic, the growth of organisms that improve sludge settleability will be favored. This operating mode can also aid biological phosphorus removal. High dissolved oxygen levels in the raw sewage and nitrates in RAS can inhibit the anaerobic selector process, but these difficulties can be overcome by enlarging the unaerated zone.

Many nitrifying activated sludge plants operate in anoxic selector mode. The configuration of an anoxic selector process is similar to that of an anaerobic selector, except that nitrate-rich mixed liquor is recycled back to the unaerated zone. The mixed liquor recycle rate is typically about three times the influent flow rate and denitrification occurs in the anoxic zone. While improvements in sludge settleability are not as pronounced as with an anaerobic selector, anoxic selectors offer several advantages over conventional complete mix mode:

- Denitrification requires a source of organic material. By removing soluble BOD in the upstream unaerated zone, the oxygen demand and air requirements in subsequent aerated compartments is reduced.
- Nitrification consumes approximately seven parts alkalinity for each part ammonia removed. Denitrification returns about half of this alkalinity back to the process.
- Anoxic selectors can often discourage the growth of poor-settling filamentous microorganisms, improving sludge settleability.

It is important to note that incorporating a selector mode without also providing the ability to operate in sludge reaeration mode would necessitate the construction of a third aeration basin.

With the exception of sludge reaeration, these various modes of operation are not critical to maintaining adequate plant performance at this time. The ability to operate in sludge reaeration mode is most important since it significantly increases the capacity of the existing aeration basins during peak flows and allows the City to postpone expansion of the aeration basins. In combination with secondary clarification improvements described later in this chapter, it is likely that the improved aeration basins will be capable of providing adequate treatment of the projected flows and loads until late in the planning period; and perhaps for the duration of the planning period if the City is successful in obtaining a waiver on mass load limits for maximum month wet weather flow periods. The base construction cost (not including contingency) for modifying raw sewage and RAS piping to allow operation in sludge reaeration mode is approximately $241,000. Since this cost is common to all alternatives, it is included in each of the following cost estimates.

Alternative 1—Replacement of Existing Surface Aerators. Once upgrades are completed to allow for operation in sludge reaeration mode, the existing aeration basin volume will be adequate until at least late in the planning period. However, the firm oxygen transfer capacity of the existing surface aerators is inadequate for treating the existing and future peak day load. The firm capacity is calculated under the assumption that one of the aerators is out of service. In order to satisfy the oxygen demand associated with the year 2024 loads, the existing surface aerators will need to be replaced. These aerators could be equipped with variable frequency drives so that the aeration process could be controlled by a dissolved oxygen monitoring system to reduce energy use. Table 7-5 presents a design data table for the upgraded aeration basins and Table 7-6 presents the estimated capital costs.

Table 7-5. Design Data
Aeration Basins Alternative 1

Item	Value	
	Current	Year 2024
Surface Aerators		
Number	6	6
Motor horsepower, each	20	50

Table 7-6. Capital Cost
Surface Aerator Replacement

Description	Cost, $1,000
Sludge Reaeration Modifications	241
Equipment Replacement	446
Subtotal	687
Contingencies	137
Construction Cost	824
Engineering and Administration	165
Total Capital Cost	989

Alternative 2—Conversion to Fine Bubble Diffusers. Fine bubble diffusers are an alternative to the surface aeration equipment currently employed at the WWTP. A fine bubble aeration system would consist of blowers, a blower building, air piping, and diffusers placed at the bottom of the aeration basins. The oxygen transfer efficiency of fine bubble diffusers is proportional to their submergence depth. Given adequate submergence, fine bubble diffusers have a higher oxygen transfer efficiency than surface aerators. However, since the existing aeration basins are relatively shallow (11 feet deep), the transfer efficiencies of the two types of equipment are similar. A fine bubble system would present a clear efficiency advantage only if the aeration basins were deepened to provide additional submergence. It is also important to note that conversion to fine bubble diffusers would require the addition of a grit removal system upstream of the aeration basins because cleaning basins equipped with fine bubble diffusers is relatively difficult. Table 7-7 presents a design data table for the upgraded aeration basins and Table 7-8 presents the estimated capital costs.

Table 7-7. Design Data
Aeration Basins Alternative 2

Item	Value	
	Current	Year 2024
Fine bubble diffusers		
Number	-	2,100
Aeration Blowers		
Number	-	4
Type	-	Centrifugal
Horsepower	-	100
Capacity, each, scfm	-	2,300

Table 7-8. Capital Cost
Conversion to Fine Bubble Diffusers

Description	Cost, $1,000
Sludge Reaeration Modifications	241
Aeration Blowers, Building, Piping, and Diffusers	1,218
Grit Removal System	680
Subtotal	2,139
Contingencies	428
Construction Cost	2,567
Engineering and Administration	513
Total Capital Cost	3,080

Alternative 3—Addition of Aeration Basin Volume. As discussed in the previous two examples, the existing aeration basins can be improved to accommodate projected flows and loads until at least late in the planning period without adding any tank volume if sludge reaeration improvements are installed. After that time, increasing flows and loads will require additional aeration basin volume to provide adequate treatment. Due to the space constraints at the existing facilities and limited room adjacent to the aeration basins, the additional volume would have to be acquired by one of the following expansion alternatives:

- Deepening the existing basins.
- Constructing new aeration basins at another location.
- Abandoning the existing aeration basins and building new facilities.

Although these expansion alternatives are all worth consideration, each has its drawbacks as described below:

- Deepening the aeration basins would alter the hydraulic profile of the plant and require additional renovations of upstream facilities (raw sewage pump station, headworks, and possibly grit removal system). The costs for deepening the existing basins would also be comparable to new construction in order to meet the increased structural requirements and modern seismic standards.
- Construction of a third aeration basin at another location would disperse the unit process across the plant site.
- Abandoning the existing aeration basins and building new facilities would keep this unit process centralized, allow plant expansions through build-out, and accommodate constraints of the plant's hydraulic profile, but would also be very expensive.

In any event, these different expansion alternatives for the aeration basins need to be considered during planning for the WWTP. Since totally new aeration basin facilities would be the most expensive alternative, Table 7-9 presents the worst case scenario cost estimates. A cost estimate is provided for two scenarios: a completely new aeration basin system and a new third aeration basin that would operate in conjunction with the existing facilities. The new aeration basins would be deeper and would rely on a fine bubble diffusion system for oxygen transfer to ensure high efficiencies for the treatment process.

Table 7-9. Capital Cost
Aeration Basins Alternative 3 New Construction

Description	Completely New Aeration Basin System Cost, $1,000	New Third Aeration Basin Cost, $1,000
New Aeration Basins	4,460	2,480
Contingencies	892	496
Construction Cost	5,352	2,976
Engineering and Administration	1,070	595
Total Capital Cost	6,422	3,571

Evaluation of Alternatives. Construction of sludge reaeration improvements and replacement of the existing surface aerators is clearly the most cost effective alternative for providing the required aeration capacity. These improvements would postpone the need for additional aeration basin volume until late in the planning period and perhaps through the end of the next planning period if the City is able to obtain a waiver during maximum wet weather periods. The considerable expense of converting to fine bubble diffusers and constructing a new blower facility is not justified because there is little gain in aeration efficiency. Once the capacity of the existing aeration basins is exceeded, the City will need to consider construction of a new aeration system at a different location or possibly expansion of the aeration basins to the south if the adjacent property can be acquired.

Secondary Sedimentation

The total capacity of the three existing clarifiers used for secondary sedimentation is approximately 9 mgd based on standard design overflow rates for clarifiers of such diameter and depth. However, the introduction of a coagulant chemical upstream of the clarifiers allows them to operate at significantly higher flow rates. Although these clarifiers have operated at total flows ranging up to even 16 mgd for brief periods of time in the past, these flow rates are beyond the intended design capacity of the facilities. To avoid the ongoing incursion of chemical costs during normal operation of the secondary clarifiers, new facilities are sized based on standard design overflow rates in the absence of chemical coagulants.

Since the existing clarifiers have already been fitted with baffles to optimize their capacity, new facilities will need to be constructed to handle the 17 mgd deficit between the existing capacity and future PWWF of 26 mgd. One alternative for new facilities is to increase secondary sedimentation capacity by building an additional secondary clarifier. A new clarifier would allow full secondary treatment of peak flows. An alternative is to treat high flows in a separate system of ballasted sand sedimentation basins. This system is a more efficient sedimentation process that is being tested at a number of communities in Oregon, but it does not provide full secondary treatment of the wastewater nor remove soluble BOD very affectively. The EPA is still evaluating the acceptability of ballasted sand sedimentation relative to full secondary treatment.

Alternative 1—Additional Secondary Treatment. The addition of a new 110-foot-diameter secondary clarifier would be needed to treat the year 2024 PWWF of 26 mgd. The new clarifier would be capable of handling more than 17 mgd at a design overflow rate of 1,850 gallons per square foot per day. Additional space should be reserved for secondary sedimentation in the general vicinity, since ultimately another 90-foot-diameter clarifier would be necessary to accommodate the projected build-out PWWF of 36 mgd. Also, the City could consider replacing the three existing small diameter clarifiers at the end of their useful life by constructing a single 110-foot unit for the build-out condition.

Construction of a new clarifier would also require modifications to the mixed liquor distribution system. A new distribution system and flow splitting structure would be needed to direct flow to the new clarifier. The new distribution system would also serve future clarifiers.

As an option, the new distribution structure and clarifier could be constructed at a higher elevation to take advantage of approximately five feet of wasted elevation difference between the aeration basins and existing clarifiers. This arrangement would generally improve design flexibility and operational efficiency in the future.

Design data for the expansion of secondary sedimentation capacity is included in Table 7-10 and estimated capital costs are presented in Table 7-11.

Table 7-10. Design Data
Secondary Sedimentation

Item	Value	
	Current	Year 2020
Secondary Clarifiers		
Number	3	4
Dimensions, each		
Diameter, ft	2 @ 55, 1 @ 60	2 @ 55, 1 @ 60, 1 @ 110
Depth, ft	10	10, 18
Peak overflow rate, gpd/sq ft	1,200	1,200 and 1,850
RAS Pumps		
Number	4	6
Capacity, each, mgd	1.45	4 @ 1.45, 2 @ 3.5

Table 7-11. Capital Cost
Secondary Sedimentation

Description	Cost, $1,000
New Clarifier / RAS Pumps	1,880
New Yard Piping / Distribution Structure	520
Subtotal	2,400
Contingencies	480
Construction Cost	2,880
Engineering and Administration	576
Total Capital Cost	3,456

Alternative 2—Ballasted Sand Sedimentation. Under this alternative, plant flows less than the treatment capacity of the existing secondary clarifiers would receive full secondary treatment, identical to the current treatment scheme. Flows in excess of the capacity of the secondary clarifiers would pass through the bar screens with the other wastewater, but would then be split away and routed to a new ballasted sand sedimentation system. Effluent from the ballasted sand sedimentation system would combine with secondary clarifier effluent prior to disinfection.

For the development of this alternative, it is assumed that flows up to 12 mgd would receive full secondary treatment in the existing clarifiers. Given the year 2024 peak wet weather flow of 26 mgd, a ballasted sand sedimentation system would be sized to treat a peak flow of 14 mgd. The ballasted sand process consists of three tanks. A coagulant is added to the wastewater upstream of the first tank. Polymer and fine silica sand are added into the first tank and mixed to ensure contact with wastewater solids. The wastewater is stirred in the second tank to promote floc formation. Settling occurs in the third tank. The settled sludge is pumped through a cyclone to separate the wastewater solids from the sand, which can be reused in the process. Overflow rates for the process are reported to be greater than 86,000 gallons per day per square foot (gpd/ft^2). This compares to the 1,500 to 1,850 gpd/ft^2 peak overflow rates typical for secondary sedimentation basins.

Manufacturers of ballasted sand sedimentation systems report removal efficiencies of greater than 70 percent for TSS and 40 to 50 percent for BOD. Given that the BOD and TSS concentration of the raw wastewater should be relatively low during peak flow events, the effluent from a sand sedimentation system should be of reasonably good quality.

Ballasted sand sedimentation systems require relatively little space—a 14-mgd system with two separate process trains would have a footprint of approximately 30 feet wide by 40 feet long. Compared to secondary sedimentation basins, capital costs for a ballasted sand sedimentation system would be relatively low. However, due to the cost of chemicals and sand, operating costs are relatively high. Therefore, ballasted sand sedimentation systems are best suited for periodic, short-term operation, such as for treatment of peak flows.

Table 7-12 provides design data on the ballasted sand sedimentation alternative.

Table 7-12. Design Data
Ballasted Sand Sedimentation

Item	Value	
	Current	Year 2024
Number of trains	--	2
Total peak flow capacity, mgd	--	14
Basin width, each, ft	--	15
Basin length, each, ft	--	40
Overflow rate at nominal capacity, gpd/sq ft	--	86,400

Estimated capital costs for the ballasted sand sedimentation alternative are summarized in Table 7-13.

Table 7-13. Capital Cost
Ballasted Sand Sedimentation

Item	Alternative 1 cost, $1,000
Ballasted Sand Sedimentation	1,900
Contingency	380
Construction Cost	2,280
Engineering and Administration	456
Total Capital Cost	2,736

Evaluation of Alternatives. At this time, Lebanon's NPDES permit requires full secondary treatment of peak wet weather flows. Based on this requirement, the less expensive ballasted sand sedimentation alternative is not a viable option. However, the EPA may consider ballasted sand sedimentation systems to be equivalent to secondary treatment and ongoing evaluations of ballasted sand sedimentation systems are underway at various locations in Oregon. Depending on the results, the alternative may become viable in the future. Therefore, for the time being, the City should continue planning for full secondary treatment through the addition of a new secondary clarifier. Meanwhile, the City should monitor the ongoing evaluations of the ballasted sand system since it represents an opportunity for significant capital cost savings. These savings could be assessed relative to the increased operation and maintenance costs associated with a ballasted sand sedimentation system.

Filtration

Filtration capacity may be an issue at the WWTP depending on the details of the City's dry weather treatment strategy. Continued discharge to the South Santiam River during the dry weather period while plant flows and loads steadily increase due to community growth would ultimately create a problem due to the City's fixed mass load discharge limit. However, if the City pursued a subsurface discharge strategy or a dry weather reuse strategy, these mass load discharge limits may not be a concern.

At the projected year 2024 maximum month dry weather flow of 7 mgd, effluent BOD and TSS concentrations would have to average 4 mg/L to comply with the dry weather mass discharge limit of 250 pounds per day discharged to the river. Consistently meeting these limits without filtration would be difficult. To achieve an average concentration of 4 mg/L under these conditions, it is recommended that an additional 3 mgd of filtration capacity be provided, bringing the total filtration capacity to 6 mgd. Assuming the filters produced effluent with an average 3 mg/L BOD and TSS, the unfiltered effluent could contain an average 10 mg/L and still meet the overall 4 mg/L target. If meeting the mass discharge limits still proves problematic even with additional filtration capacity in place, the City would be justified in approaching the Environmental Quality Commission to request a modification of the dry weather mass discharge limits, especially during the problematic, high flow shoulder months of May and October.

Design data for the filtration expansion is included in Table 7-14 and estimated capital costs are presented in Table 7-15.

Table 7-14. Design Data
Filtration

Item	Value	
	Current	Year 2024
Pressure Filters		
Number	2	4
Capacity, mgd	3	6

Table 7-15. Capital Cost
Filtration

Description	Cost, $1,000
New Filters	1,222
Contingencies	244
Construction Cost	1,466
Engineering and Administration	293
Total Capital Cost	1,759

The addition of more filtration capacity is only necessary if the WWTP continues discharging all wastewater to the river during the dry weather season. Otherwise, the existing filtration system is adequate for the duration of the planning period. Therefore, evaluation of the need for filtration improvements must be made within the context of a comparison between continued river discharge, an effluent reuse program, and a subsurface river discharge strategy. This discussion is included later in the chapter.

Disinfection System

The WWTP currently relies on sodium hypochlorite and a chlorine contact chamber to provide disinfection of the plant effluent. There are two considerations that would motivate upgrades to this system: the potential need for capacity expansions at the chlorine contact chamber and the potential need for dechlorination improvements to eliminate chlorine toxicity.

The DEQ's criteria for chlorine contact times are 15 minutes at peak wet weather flow, 20 minutes at peak day flow, and 60 minutes at average dry weather flow. The DEQ typically allows contact times of 15 minutes during peak wet weather flow for plants that have demonstrated their capability of consistently meeting bacteria limits with the existing facilities. Since the existing chlorine contact chamber is capable of meeting the DEQ contact time criteria at the projected year 2024 flows, the capacity of the disinfection system does not need to be expanded.

The plant's NPDES permit requires that residual chlorine in the plant effluent not exceed a monthly average of 0.01 mg/L and a daily maximum of 0.02 mg/L. If the City continues direct discharge to the South Santiam River, compliance with this residual chlorine limit will require either the addition of dechlorination facilities to the existing system or conversion to a UV disinfection system. Although the evaluations of subsurface discharge to the river contend that dechlorination will not be necessary, the ability to periodically dechlorinate may still be required since the areas targeted for rapid infiltration of the treated effluent do become hydraulically connected to the South Santiam River during periods of high river stage. As a result, chlorine toxicity may still be an issue even for the subsurface river discharge approach.

The following discussion presents three upgrade alternatives for satisfying the capacity and chlorine residual considerations.

Alternative 1—Minor Chlorination System Improvements and Dechlorination. Since it is likely that the City will be able to obtain DEQ approval for a chlorine contact time of 15 minutes for the year 2024 peak wet weather flow, no additional basin volume will be required for the duration of the planning period. Under this scenario, the only necessary disinfection system improvements would be increased chemical feed/storage capacity, minor improvements at the contact basins, and a new dechlorination system.

Included in this alternative would be additional hypochlorite solution storage and new metering pumps along with improvements such as baffling at the chlorine contact basins to improve performance. To accommodate the chlorine residual limits, this alternative includes a sodium bisulfite dechlorination system. The primary components of a dechlorination system include storage tanks for the sodium bisulfite solution, metering pumps, and a chemical mixing system. Table 7-16 provides design data for this alternative and Table 7-17 summarizes the capital cost estimate. The capital cost estimate assumes that the new dechlorination system would be located in a new building.

Table 7-16. Design Data
Disinfection Alternative 1

Item	Value	
	Current	Year 2024
Chlorine Contact Basins		
Volume, mg	0.281	0.281
Capacity[a], mgd	27	27
Hypochlorite Disinfection System		
Storage volume, gallons	2,500	5,000
Metering pumps	2	2
Bisulfite Dechlorination System		
Storage volume, gallons	-	2,500
Metering pumps	-	2

[a]Capacity calculated for 15 minutes of contact time.

Table 7-17. Capital Cost
Minor Chlorination Improvements and Dechlorination

Description	Cost, $1,000
Chlorination Improvements	75
New Dechlorination System	275
Subtotal	350
Contingencies	70
Construction Cost	420
Engineering and Administration	84
Total Capital Cost	504

Alternative 2—Chlorination System Expansion and Dechlorination. In order to provide 30 minutes of contact time during the year 2024 peak flow of 26 mgd, the WWTP's disinfection system would require a capacity expansion of 12.5 mgd through the addition of 260,000 gallons of chlorine contact basin volume. Due to space constraints adjacent to the existing chlorine contact basin, this additional volume would need to be constructed at a separate location. If desirable, the new disinfection facilities could be constructed at a lower elevation, which would allow gravity flow from the secondary clarifiers. The disinfection system would then be capable of discharging to the outfall by gravity under most conditions, switching to a pumped discharge only when necessary due to high plant flows or high river levels.

Included in this capacity expansion would be additional hypochlorite solution storage and new metering pumps. Like Alternative 1, this alternative also includes a new building for the sodium bisulfite dechlorination system. Table 7-18 provides design data for this alternative and Table 7-19 summarizes the capital cost estimate.

Table 7-18. Design Data
Disinfection Alternative 2

Item	Value	
	Current	Year 2024
Chlorine Contact Basins		
Volume, mg	0.281	0.541
Capacity[a], mgd	13.5	26
Hypochlorite Disinfection System		
Storage volume, gallons	2,500	5,000
Metering pumps	2	2
Bisulfite Dechlorination System		
Storage volume, gallons	-	2,500
Metering pumps	-	4

[a]Capacity calculated for 30 minutes of contact time.

Table 7-19. Capital Cost
Chlorine Disinfection and Dechlorination

Description	Cost, $1,000
New Chlorine Contact Basins and Hypochlorite Feed System	1,712
New Dechlorination System	275
Subtotal	1,987
Contingencies	397
Construction Cost	2,384
Engineering and Administration	477
Total Capital Cost	2,861

Alternative 3—Conversion to UV Disinfection. An alternative to expansion of the chlorine disinfection system is conversion to UV disinfection. Under this alternative, new UV disinfection facilities would be constructed in one of the existing chlorine contact chamber basins or at a new location on the plant site. If constructed at a new location, it would be possible to build the UV disinfection system at a lower elevation as described in the previous alternative. Again, this configuration would provide the capability for gravity flow to the river under typical flow conditions; pumping would be required during peak wet weather events. For the purposes of this comparison of alternatives, it is assumed that the UV disinfection system would be built at a new location.

It is important to note that while UV disinfection performance is more than adequate for meeting the bacteria limits in the plant's NPDES permit, UV systems cannot meet the stricter bacteria limits required for Level III reclaimed water unless the effluent is first filtered. For example, the UV system included in the cost estimate is capable of satisfying only Level II reclaimed water disinfection limits under the following specific conditions:

- Peak hour dry weather flow does not exceed 12 mgd.
- Ultraviolet transmittance is 65 percent.
- Maximum TSS concentration of 5 mg/L.
- Turbidity less than 2 NTU.
- Maximum mean particle size of 20 microns.

Therefore, if the City were to convert to UV disinfection but wanted to retain the ability to produce Level III reclaimed water, the WWTP would need to maintain the existing hypochlorite feed system and a portion of the existing chlorine contact chamber. Since the hypochlorite system is already in place, it does not affect the capital cost of converting to UV.

UV systems are available using medium-pressure and low-pressure lamps. While low-pressure and medium-pressure UV systems utilize the same basic disinfection mechanism, the systems are significantly different in both operation and appearance. Low-intensity, low-pressure lamps are highly efficient at producing light at the germicidal wavelength of 254 nanometers. In contrast, medium-pressure lamps produce light at a wide range of wavelengths, some of which has little germicidal effect. Therefore, medium-pressure lamps are significantly less energy efficient. However, because medium-pressure lamps have much higher output, fewer lamps are needed. A medium-pressure system typically requires less than one-tenth as many lamps as a comparable low-pressure system. Other significant differences between low-pressure and medium-pressure UV systems include:

- Medium-pressure systems are equipped with automatic cleaning systems, reducing labor requirements when frequent cleaning is necessary.

- Medium-pressure lamps cost approximately six times more than low pressure lamps and last approximately one-half as long. However, because medium-pressure systems require less than one-tenth as many lamps, lamp replacement material costs are comparable. Lamp replacement labor is higher for low-pressure systems because there are more lamps.

- Medium-pressure systems have a smaller footprint and lower structure costs.

A present worth comparison that accounts for both capital and operating costs indicates that a medium-pressure system is slightly more cost-effective for the WWTP. However, the difference in present worth is less than ten percent. For the purposes of this facilities plan, it is assumed that a medium-pressure system would be constructed.

Table 7-20 presents design data information for the UV disinfection alternative and Table 7-21 summarizes the estimated capital costs.

Table 7-20. Design Data
Disinfection Alternative 3

Item	Value	
	Current	Year 2024
UV Disinfection System		
Number of Channels	-	1
Number of Modules	-	3
Number of Lamps	-	96

Table 7-21. Capital Cost
UV Disinfection

Description	Cost, $1,000
New UV Disinfection System	977
Contingencies	195
Construction Cost	1,172
Engineering and Administration	234
Total Capital Cost	1,406

Evaluation of Alternatives. Alternative 1 is clearly the most cost-effective approach due to the avoided capital costs associated with capacity expansion and process conversion. Although both Alternatives 2 and 3 are based on more traditional design criteria and would be expected to provide better assurance of consistent treatment performance, they have much higher capital costs. Further, the treatment performance of Alternative 1 can be adequately assured by using a control system to increase chemical dosage rates as necessary during periods of high flow/low contact time. Since other treatment plants have demonstrated successful disinfection performance using contact times as low as 15 minutes, it is recommended that the City pursue Alternative 1 and defer investments in additional chlorine contact basins until after the year 2024.

Outfall System

The existing outfall to the South Santiam River is an overland/shoreline discharge system that provides insufficient mixing of the treated effluent with the river. The existing configuration also presents potential public health and liability issues. If the City continues discharging directly to the river, this outfall system must be upgraded to provide better mixing and ensure that the WWTP meets chronic toxicity requirements at the edge of the regulatory mixing zone (RMZ) and acute toxicity requirements at the edge of the zone of initial dilution (ZID). In addition to the mixing issue, the capacity of the outfall system is significantly lower than the projected peak wet weather flows.

The best solution for maximizing mixing within an RMZ is the construction of a submerged multiport outfall diffuser. This type of diffuser system would consist of a large outfall pipe fitted with multiple discharge ports. Oriented perpendicular to the river flow, the outfall pipe would be buried beneath the riverbed with ports protruding above the river bottom. The discharge ports would be fitted with duckbill check valves to prevent backflow and sedimentation under low flow conditions and to increase discharge velocities for improved mixing. Since the existing outfall is submerged during high river stages, the multiport system could overflow to the existing pipe for additional capacity during periods of peak flow. This basic diffuser concept is assumed for each of the following capacity expansion alternatives.

All of the following capacity expansion alternatives assume that the existing outfall system is capable of a minimum level of pressurization (approximately 15 feet or 6.5 psi) such that the outfall pipe can be surcharged up to the chlorine contact tank overflow weir. Some sandbagging was necessary on outfall manholes during the 1996-97 flooding events indicating that either the manhole lids are not watertight or bolts were not in place. Therefore, conversion to watertight manhole lids is advised if necessary.

It is important to note that the mixing improvements would not be necessary if the City discontinues direct discharge to the river in favor of indirect subsurface discharge to the river. Not only would subsurface flow to the river effectively disperse the discharge plume, but there may be natural dechlorination and natural nitrification of the effluent as is passes through soil and sediments.

Alternative 1—Parallel Outfall Pipeline. The existing outfall system has a discharge capacity of approximately 21 mgd during a 100-year flood which is well below the year 2024 and build-out peak flow projections. Further, with the addition of a multiport diffuser to the outfall and the increase in head losses associated with the discharge ports, the outfall’s capacity will be reduced to approximately 15 mgd. Other than the discharge port losses, the majority of the head loss in the system is related to friction in the 1,200-foot-long, 30-inch-diameter outfall pipeline. Therefore, the easiest way to increase the outfall’s capacity is to install a parallel pipeline. The parallel pipeline would be sized to accommodate future peak wet weather flows through build-out.

Table 7-22 provides design data on the existing and future outfall system and Table 7-23 includes capital cost estimates for the Alternative 1 improvements.

Table 7-22. Design Data
Outfall Alternative 1

Item	Value	
	Current	Year 2024
Multiport Diffuser		
Pipe diameter, inches	-	36
Pipe length, feet	-	80
Number of ports	-	6
Outfall Pipelines		
Number	1	2
Diameter, inches	30	30, 42
Outfall Capacity	21	34

Table 7-23. Capital Cost
Outfall Diffuser with Parallel Pipeline

Description	Cost, $1,000
Outfall Diffuser	306
Parallel Pipeline	380
Subtotal	686
Contingencies	137
Construction Cost	823
Engineering and Administration	165
Total Capital Cost	988

Alternative 2—Outfall Diffuser System with Peak Flow Effluent Storage. Another solution for the outfall capacity problem is to provide facilities for the temporary storage of peak effluent flows. In this alternative, the capacity of the outfall system would only need to satisfy a portion of the peak day flow since the diurnal peak could be diverted to storage.

There are two existing lagoons next to the plant site that could be modified for peak flow effluent storage. Each lagoon offers approximately 11 million gallons of storage. Gravity flow from the disinfection system to the lagoons would be possible. A new pump station at the lagoon would return the stored effluent to the outfall system when peak flows subside. With the capacity of the existing outfall system (with new diffuser) at approximately 15 mgd and a year 2024 peak flow of 20 mgd, 5 mg of storage would be needed for the peak day event. However, the lagoon must have adequate capacity to store more excess wastewater than an isolated peak day flow event since the area could experience an extended storm or multiple storms in succession. Therefore, this analysis assumes that the storage lagoons would be sized to contain a volume equal to 200 percent of the peak day requirement or 10 million gallons. If the lagoons were operated to maximize their storage potential by treating all rainwater that falls into them, the full volume of storage would be maintained as available. Therefore, the required storage volume is not problematic since each existing lagoon has a storage capacity of approximately 11 million gallons, slightly more than necessary for storing peak flows.

Table 7-24 provides design data for the peak flow storage alternative and Table 7-25 summarizes the associated capital costs

Table 7-24. Design Data
Outfall Alternative 2

Item	Value	
	Current	Year 2024
Multiport Diffuser		
Pipe diameter, inches	-	30
Pipe length, feet	-	80
Number of ports	-	6
Outfall Pipeline		
Number	1	1
Diameter, inches	30	30
Outfall Capacity	21	15
Storage Lagoon		
Volume, mg	-	11
Lagoon Pump Station		
Capacity, mgd	-	7

Table 7-25. Capital Cost
Outfall Diffuser with Peak Flow Effluent Storage

Description	Cost, $1,000
Outfall Diffuser	306
Lagoon Renovation, Pump Station, and Piping	921
Subtotal	1,227
Contingencies	245
Construction Cost	1,472
Engineering and Administration	294
Total Capital Cost	1,766

Alternative 3—Pressurized Outfall System. A third alternative for increasing the capacity of the outfall is to pressurize the existing outfall during periods when the gravity flow capacity is inadequate. Two scenarios were considered. First, the existing 30-inch diameter outfall could be pressurized to provide the required peak flow capacity. The second option would require lining of the outfall to increase the pressure rating of the pipe.

To achieve the necessary peak flow capacity with the existing outfall, an internal pressure of 30 feet of water would be required. This exceeds the safe working pressure of conventional reinforced concrete pipe. To line the 30-inch pipe, a high density polyethylene pipe would be inserted inside the existing pipe. The liner pipe would have an outside diameter of 24.5 inches and an inside diameter of about 21 inches. With this pipe, average dry weather flow could be conveyed to the river by gravity, but a pump station would be required to convey most wet weather flows to the river. At the peak wet weather flow, the required velocity in the pipe would be somewhat high but not infeasible.

The capital cost for a 26 mgd pump station alone is more than $1.5 million, significantly higher than the cost for construction of a parallel gravity outfall. Therefore, the pressure system is not recommended for further consideration.

Evaluation of Alternatives. The installation of a parallel outfall pipe is significantly simpler and less costly than the construction of effluent storage facilities. Further, the parallel outfall pipe alternative includes no operation and maintenance costs while the O&M associated with the peak flow storage facilities would be significant.

Effluent Pump Station

The existing effluent pump station has a firm capacity of approximately 26 mgd which is adequate for the year 2024 peak wet weather flow of 26 mgd. Therefore, no improvements are planned for the effluent pump station.

Odor Control

The Lebanon WWTP represents a substantial community investment which provides effective wastewater treatment for the existing service area and establishes a sound base for expansion to serve future growth. The existing facilities have a replacement value of approximately $20 million. It is important that the City protect the viability of the existing plant site for long term wastewater treatment.

Public acceptance of wastewater treatment facilities as neighbors has generally been related to the level of odors experienced in the surrounding area. In communities where neighbors have experienced odors outside the plant boundaries, opposition to expansion or continued operation has surfaced, and in some cases relocation of the treatment plant has resulted. Communities that have an extensive buffer around the treatment plant have been able to avoid these controversies as well as the high cost of odor containment and treatment.

While odor controls can be constructed for the processes that generally cause the greatest amount of odor, regular operation of a wastewater treatment plant can result in occasional odors. For this reason, communities are advised to acquire properties that are adjacent to the treatment units as the first defense against future odor problems.

At the Lebanon WWTP, the potential sources for odors in order of risk are the following processes:

- Headworks including the screenings container.
- Solids processing including the aerobic digesters and thickening system.
- Aeration basin.

Especially with the aerobic digester and aeration basin, odors are generally not a regular problem. However, with any upset of the biological process, the potential exists for a severe odor condition. With these unit processes, providing containment and odor control would be very expensive. For example, other cities have constructed buildings over the headworks equipped with ventilation systems and odor scrubbers to reduce the potential for problems.

Figure 7-1 shows the existing site and the surrounding development. A substantial buffer area has always existed to the north and west. Recently, purchases of property by the City have also provided a buffer to the east. With the acquisition of a relatively modest amount of additional property, a buffer to the south side of the site could be acquired for convenient future expansion of the aeration basins. Acquisition of this property will also provide the opportunity to extend the available buffer to the south.

With the acquisition of these adjacent lands for a buffer, containment and treatment of process air should not be required during the current planning period. Continued attention to housekeeping and process performance will be necessary to minimize the potential for offsite odors.

STRATEGIES FOR TREATMENT OF DRY WEATHER FLOWS

Strategies for the treatment of wastewater during the dry weather season must account for the following considerations:

- The WWTP's current dry weather mass discharge limits will not change, but influent flows and loads will increase as projected in Chapter 5.
- Compliance with the temperature standard may require Lebanon to mitigate the temperature impact on the South Santiam River.

These issues can be addressed through either the addition of treatment processes or curtailment of direct discharges to the river. This section compares three dry weather treatment strategies that address the above considerations: effluent reuse, filtration and cooling, and subsurface discharge to the river.

Dry Weather Strategy 1—Effluent Reuse

Under an effluent reuse strategy, the WWTP would produce Level III reclaimed water which is suitable for irrigation of non-food crops. For irrigators, reclaimed wastewater represents an inexpensive source of water that can satisfy a portion of a crop's nutrient requirements, thus allowing for savings in fertilizer expenses. For the City, the ability to direct effluent toward use in the irrigation of crops allows for the reduction or elimination of discharges to the South

Santiam River during the dry weather season. In this way, a reuse program would mitigate the impact of plant discharges on river temperature as well as improve the plant's ability to meet seasonal mass discharge limits.

The following general actions would be necessary to allow for effluent reuse during the dry weather season:

- Coordination of effluent reuse sites on agricultural lands within a feasible distance of the WWTP.
- Construction of pumping and piping facilities to distribute reclaimed effluent for reuse.
- Modifications to the WWTP's disinfection system to ensure compliance with Level III reclaimed water requirements.

It is anticipated that implementation of a reuse program would be implemented in phases and effluent discharges to the river during the dry weather season would be reduced over time. A graduated implementation approach allows the City adequate time to adapt the reuse program to the local water demand situation as well as the evolving regulatory environment.

Reuse Program Description. Within the general context of a reuse strategy, there are many possible differences in the specific details of how a community goes about utilizing reclaimed water. However, for the purpose of this facility planning level assessment, the following parameters are assumed:

- The City would initially contract with local farmers and provide irrigation water. Ultimately the City may purchase or lease some land for irrigation.
- The irrigated land would initially be within three miles of the WWTP.
- The City would encourage irrigation of low maintenance, relatively water intensive crops.
- The targeted acreage of cropland would accommodate the required effluent application rates even during a 1-in-10 year high rainfall season.
- The program would begin without reclaimed water storage capacity, but storage facilities would ultimately be provided.

The agricultural land surrounding the WWTP is illustrated in Figures 7-2 and 7-3. The first figure is an aerial photograph showing land to the north of Lebanon within a three mile radius of the plant. Distances are shown at one mile intervals. The second figure illustrates tracts of property in this area that are larger than 50 acres and under a single ownership. There are at least 4,400 acres of these large tracts within three miles of the WWTP. These large properties warrant the greatest attention for a wastewater reuse program since their participation would limit the required number of individual contracts.

Crops. The selection of crops for cultivation in a reuse program is directed toward those with the highest water demand and those with the lowest maintenance requirement. Crop water demand during the dry weather season shoulder months of May and October is particularly important since high precipitation rates at these times tend to greatly restrict the need to irrigate. Based on a review of potential Willamette Valley crops, alfalfa and pasture were selected as the best candidates for the evaluation of the reuse program. These crops have the highest water demand during the targeted irrigation season and the least required maintenance.

Water Balance. A water balance analysis was completed in order to quantify the amounts of land, storage, and pumping facilities required for a dry weather reuse program. The analysis was conducted under two different scenarios. The first scenario is for an initial stage in the reuse program when 50 percent of the total wastewater flow is directed toward reuse. For this scenario, it was also assumed that there would be no reuse during the relatively wetter months of May and October such that storage requirements could be limited to 175 acre-feet. The second scenario is for a reuse program when 100 percent of the wastewater flow is directed toward reuse or storage for the entire dry weather period (May through October). The analysis was based on projected year 2024 wastewater flows and average evapotranspiration rates for the alfalfa and pasture crops. Table 7-26 summarizes estimates for the irrigation land and storage requirements of various crops assuming the 1-in-10 year rainfall pattern for each scenario.

Table 7-26. Irrigation Land and Storage Requirements for Wastewater Reclamation

	1 in 10 year rainfall with reclamation of 50% of wastewater flows during June to September			1 in 10 year rainfall with reclamation of 100% of wastewater flows during May to October		
	Crop Area (acres)	Irrigation Storage[a] (ac-feet)	Storage Area (acres)	Crop Area (acres)	Irrigation Storage[a] (ac-feet)	Storage Area (acres)
Alfalfa	465	175	25	1,155	700	100
Pasture	415	175	25	1,015	700	100

[a]For average year rainfall, the entire irrigation storage volume will not be used.

Since May and October tend to have the highest level of rainfall and lowest irrigation requirements, most of the wastewater generated during these months must be stored if not immediately discharged. Since a crop's irrigation requirement is defined as the difference between the evapotranspiration (ET) rate of the crop and the amount of the ET satisfied by rainfall, an enormous amount of land would be required to irrigate wastewater flows in May and October. Therefore, much of the storage volume required for a full dry weather season reuse program is used during those months. To eliminate the need for storage initially, the City would only reclaim water during periods of high irrigation demand. The City would then phase construction of storage facilities as necessary once the basic reuse program is established.

Design Data and Capital Costs. Design data for the initial set of improvements necessary to begin implementing this alternative are listed in Table 7-27. The capital costs for the effluent reuse alternative are summarized in Table 7-28. The additional cost for the installation of new multi-port outfall diffuser and outfall pipeline capacity expansion is also included in the cost estimate for this alternative since it is not necessary for all of the dry weather strategies. Note that investment in additional improvements, including storage facilities, would be required to achieve 100 percent reuse for the entire dry weather period.

LEGEND

DESIRABLE BUFFER PARCELS

Figure 7-1

CITY OF LEBANON
WASTEWATER TREATMENT PLANT

BUFFER AREAS

WEST YOST & ASSOCIATES

CAD FILE: 516-00-02-FIG_7-1.dwg CFG FILE: 4MV-FULL.PC2 DATE: 4-1-04 VLF

N

0 1000 2000
SCALE IN FEET

NOTES:

LEGEND:

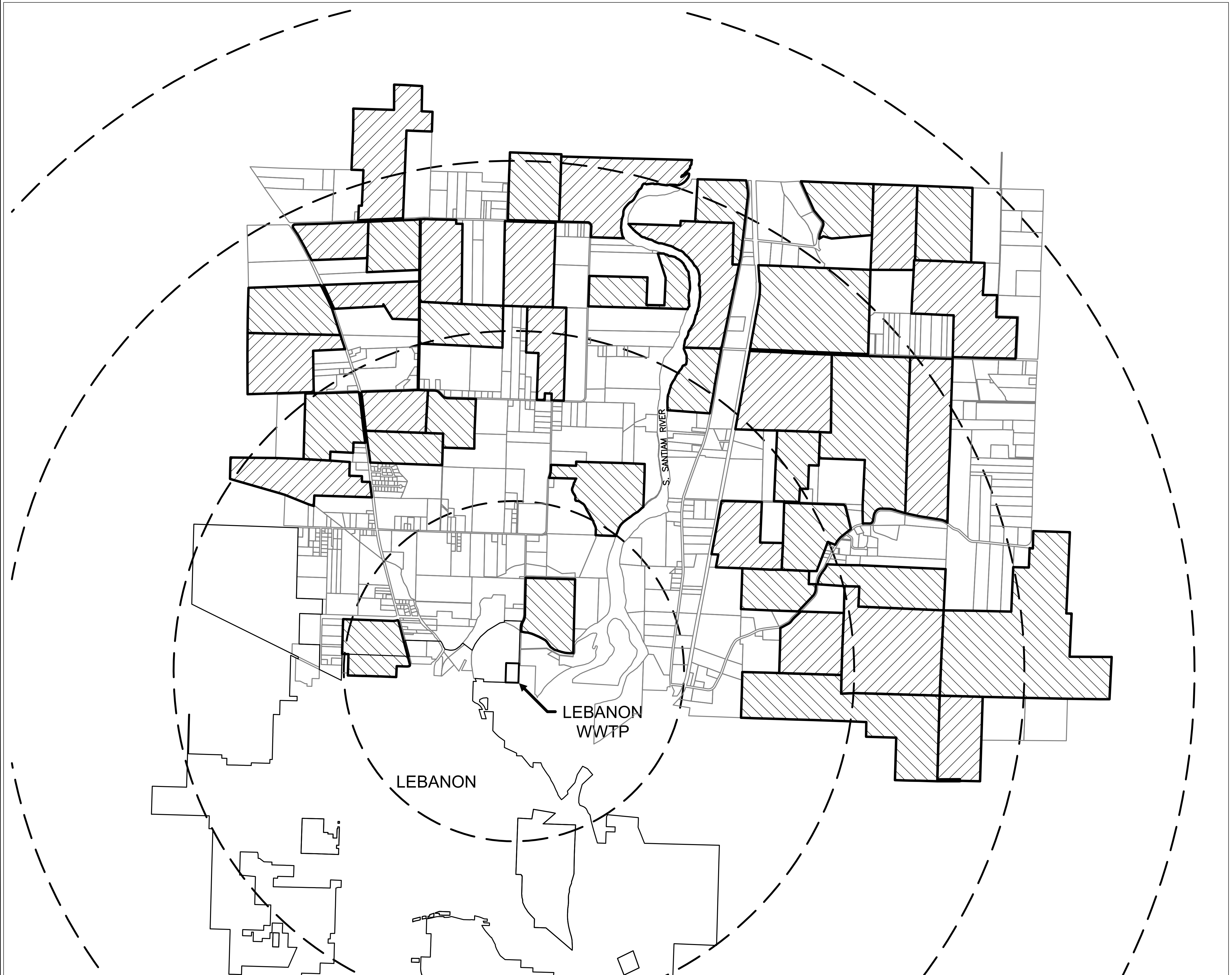
RADIUS AROUND WWTP,
1 MILE INTERVALS

FIGURE 7-2

AGRICULTURE
LAND WITH POTENTIAL
FOR REUSE

WEST YOST & ASSOCIATES

516-00-02-FIG-7-2 4MV-FULL.PC2 4-1-04 VLF



NOTES:

LEGEND:

RADIUS AROUND WWTP, 1 MILE INTERVALS

PARCELS GREATER THAN 50 ACRES

LEBANON CITY LIMITS

FIGURE 7-3

AGRICULTURAL PARCELS GREATER THAN 50 ACRES

WEST YOST & ASSOCIATES

516-00-02-FIG-7-3 4MV-FULL.PC2 4-1-04 VLF

Table 7-27. Design Data
Dry Weather Strategy 1

Item	Value	
	Current	Year 2020
Irrigation Pumping Station		
Capacity, mgd	--	5
Distribution Piping		
Diameter, inches	--	18
Length, feet	--	15,840

Table 7-28. Capital Cost
Strategy 1—Effluent Reuse

Description	Cost, $1,000
Reuse System	
Pump Station	350
Distribution Piping	2,200
Outfall Diffuser and Pipeline	686
Subtotal	3,236
Contingencies	647
Construction Cost	3,883
Engineering and Administration	777
Total Capital Cost	4,660

In addition to capital costs for the construction of physical facilities, the City would be advised to budget $100,000 in the near term for development of the reuse program. This expense represents the work necessary to explore the level of interest in reclaimed water among local farmers, identify the use locations and likely volume of water demand, and prepare contract documents for the transfer of water. In the long term, the City would also budget money for the purchase or lease of land for irrigation in order to improve efficiency of the program.

Dry Weather Strategy 2—Filtration and Cooling

Another strategy for maintaining compliance with the in-stream temperature standard is to provide filtration and cooling during the dry weather season. This strategy includes the installation of chillers to cool the effluent prior to discharge. Furthermore, as flows increase, it will be necessary to filter a portion of the plant effluent to comply with the existing mass discharge limits for BOD and TSS.

Effluent Cooling. As discussed in Chapter 6, DEQ guidelines regarding the temperature standard are currently under development. However, for the purposes of this report, it is assumed that the standard will be strictly enforced. One alterative for meeting the temperature requirement is to cool the effluent prior to discharge. While effluent cooling has not been implemented at any municipal wastewater treatment plant in Oregon, the technology to cool effluent exists.

The major components of an effluent cooling system for the WWTP would include all of the following:

- Centrifugal chiller.
- Cooling tower.
- Heat exchanger.
- Pumps and piping.
- Electrical system.
- Building.

Filtration. To continue dry season discharges to the South Santiam River while complying with mass load discharge limits, the WWTP will need to provide additional filtration capacity. Therefore, the cost for filtration must also be included in the summary of capital costs for this strategy.

Design Data and Capital Costs. Design data for this alternative are summarized in Table 7-29 and capital costs are summarized in Table 7-30. The additional cost for the installation of new multi-port outfall diffuser and outfall pipeline capacity expansion is also included in the cost estimate for this alternative since it is not necessary for all of the dry weather strategies. Effluent filtration is included as well as the cost for effluent cooling.

Table 7-29. Design Data
Dry Weather Strategy 2

Item	Value	
	Current	Year 2024
Effluent Cooling System		
Number of chillers	--	2
Capacity, each, tons refrigeration	--	600
Number of cooling towers	--	2
Number of pumps	--	4
Effluent Filtration		
Capacity, mgd	3	6
Anticipated filtered TSS, mg/L	3	3

Table 7-30. Capital Cost
Strategy 2—Filtration and Cooling

Description	Cost, $1,000
Effluent Cooling	1,353
Effluent Filtration	1,221
Outfall Diffuser and Pipeline	686
Subtotal	3,260
Contingencies	652
Total Construction Cost	3,912
Engineering and Administration	782
Total Capital Cost	4,694

Dry Weather Strategy 3—Subsurface Discharge to the River

Indirect discharge to the river by means of subsurface infiltration would also achieve compliance with the in-stream temperature standard by using the earth to cool the effluent before it reaches the river. As discussed earlier, the City has identified a promising candidate site where this discharge strategy could be implemented. Ground temperature monitoring conducted at this site for the feasibility analysis of the subsurface discharge strategy indicated that the soil maintained a steady temperature of 50 degrees F. The details of this feasibility analysis are summarized in a report prepared for the City by Kennedy Jenks Consultants (Kennedy Jenks, April 2004).

The major components of a subsurface discharge system as described in the Kennedy Jenks report include all of the following:

- Low head pump station.
- River crossing.
- Transmission piping.
- Effluent distribution systems.

In addition to the physical components of the discharge system, additional cost items include the acquisition of the candidate land and completion of the remaining technical studies and permitting.

Design Data and Capital Costs. Design data for this alternative are summarized in Table 7-31 and capital costs are summarized in Table 7-32. Since the subsurface discharge alternative alleviates the need for constructing a multi-port outfall diffuser in the river, the cost summary in Table 7-32 does not include the associated costs.

Table 7-31. Design Data
Dry Weather Strategy 3

Item	Value	
	Current	Year 2020
Effluent Pumping Station		
Capacity, mgd	--	34
Transmission Piping		
Diameter, inches	--	36
Length, feet	--	2,500

Table 7-32. Capital Cost
Strategy 3—Subsurface Discharge

Description	Cost, $1,000
Subsurface Discharge System	
Pump Station	1,200
Transmission Piping	500
River Crossing	450
Effluent Distribution Systems	75
Subtotal	2,225
Contingencies	445
Total Construction Cost	2,670
Engineering and Administration	543
Land Acquisition	280
Total Capital Cost	3,484

Evaluation of Dry Weather Strategies

Selection of an appropriate dry weather treatment strategy depends significantly on how the regulators implement the temperature standard as well as how they permit an innovative approach such as the subsurface discharge strategy. Evaluation of the alternative strategies on the basis of costs indicates that Strategy 3, the subsurface discharge strategy, should be selected as the preferred approach. The higher capital costs and ongoing operational costs associated with a mechanical cooling system indicate that Strategy 2 is not an appropriate approach. Although Strategy 1 (the reuse alternative) has become an increasingly common approach for dry weather treatment in recent years, the facilities included in the reuse cost are for an initial phase only that would not provide complete compliance with the temperature standard for the entire dry weather season. The storage facilities required for a complete dry weather reuse system could increase capital costs by several million dollars. Further, the City could incur land acquisition costs for this strategy in the event that there was insufficient demand among local farmers for the reclaimed water. Planning for a reuse program, however, could be conducted in the event that permitting of the subsurface discharge strategy proves to be problematic.

STRATEGIES FOR TREATMENT OF PEAK FLOWS

The WWTP has a current PWWF treatment capacity of approximately 12 mgd as determined by an evaluation of the existing secondary clarifiers; this compares to an estimated current peak flow of 21 mgd and a projected year 2024 PWWF of 26 mgd. Alternatives for treatment of wet weather peak flows are based on the following criteria:

- The WWTP's current mass discharge limits will not change, but flows and loads will increase as projected in Chapter 5.
- Peak flows up to the 1 in 5 year design PWWF will receive full secondary or equivalent treatment prior to discharge.

Two peak flow treatment alternatives were evaluated: peak flow attenuation through storage in lagoons and provision of additional secondary treatment capacity.

Peak Flow Strategy 1—Peak Flow Attenuation Through Storage in Basins

Under this alternative, peak flows in excess of the WWTP treatment capacity would be diverted to holding basins for temporary storage. The stored wastewater would be routed back to the WWTP after high influent flows subside. By attenuating peak wet weather flows in this manner, the required hydraulic capacity of many unit processes at the WWTP would be reduced, thus eliminating or postponing the need for certain capacity expansions. Unit processes that are sized for peak flow conditions include the headworks, clarifiers, disinfection system, and outfall. The total costs for upgrades to these facilities for future peak wet weather flows is presented in the Strategy 2 discussion.

Storage Requirements. This strategy requires that sufficient raw sewage storage volume be constructed adjacent to the plant site. The most convenient construction site would be in the vicinity of the existing lagoons. Although, the majority of one existing lagoon is targeted for digested sludge storage, the other lagoon would be available for development. The estimation of necessary storage volume is based on the following assumptions:

- The existing WWTP could treat a temporary flow of 15 mgd during peak flow conditions.
- Storage basins would be kept empty of rainwater such that the full volume is always available for storage. The rainwater would be regularly pumped to the headworks.

The year 2024 peak day flow is 20 mgd. With the WWTP treating 12 mgd, 8 million gallons would be directed to the new storage basins. Because the Lebanon area could experience an extended storm or multiple storms in succession, the storage basin must have adequate capacity to store more excess wastewater than that associated with an isolated peak day flow event. Therefore, this analysis assumes that the storage basins would be sized to contain a volume equal to 150 percent of the peak day requirement or 10.5 million gallons. If the basins were operated to maximize their storage potential by continuously treating all rainwater that falls into the lagoons, this full volume of storage would be available at all times. The basins would be lined to eliminate the potential for groundwater impacts.

For the diversion of peak flows to storage, the influent pump stations would be controlled so that excess flow was pumped directly to the lagoons. The storage lagoon would be designed to drain back to the pump stations by gravity.

Operation and Maintenance. The solids in wastewater directed to a peak flow storage basin will settle to the bottom. Since retaining raw sewage sludge in a basin will create odors, peak flow storage facilities need to be designed to permit automatic cleaning soon after a peak flow event. Although concrete storage basins are expensive, they are also most easily cleaned. Raw sewage storage basins should employ the same technologies used to store combined wastewater. Features should include tipping buckets or other flushing systems and wash down stations which expedite cleaning and reduce labor costs. To mitigate the costs associated with automated cleaning equipment, the storage facility should be constructed with multiple basins that fill in series. The primary storage basin, which is the first to fill and receives the most frequent use, would be equipped with automated cleaning equipment. Secondary storage basins, which are only used during the largest peak flow events, would rely on manual cleaning equipment such as water cannons. This concept of a concrete construction, multi-basin storage facility with both automated and manual cleaning equipment forms the basis for the following capital cost estimate.

Capital Costs. The capital cost for Strategy 1 is shown in Table 7-33. The capital costs are for a high quality storage facility, as described above. Construction could be phased over the course of the planning period.

Table 7-33. Capital Cost
Peak Flow Strategy 1—Storage in Lagoons

Item	Cost, $1,000
Storage Lagoon	14,648
Contingencies	2,930
Construction Cost	17,577
Engineering and Administration	3,515
Total Capital Cost	21,093

Peak Flow Strategy 2—Conventional Treatment

Under this strategy, the treatment facility will be expanded so the entire peak flow receives secondary treatment. Each unit process would be upgraded to allow for the treatment of the full year 2024 peak wet weather flow of 26 mgd.

Treatment Process Upgrades. The treatment processes requiring upgrades to accommodate the year 2024 PWWF include the headworks, the aeration basins, the secondary clarifiers, the disinfection system, the effluent pumps, and the outfall system. Specifically, the treatment process upgrades would include the following:

- Renovation of the headworks to accommodate a second mechanical bar screen.
- Modification of the aeration basin piping to allow for operation in sludge reaeration mode. During high flows the aeration basins would operate in a sludge reaeration mode to handle the peak flow without washing out mixed liquor solids.
- Replacement of aeration basin surface aerators with higher power units.
- Addition of a secondary clarifier and associated appurtenances.
- Conversion to a higher capacity UV disinfection system.
- Construction of a parallel outfall pipe

Costs. Estimated costs for Peak Flow Strategy 2 are summarized in Table 7-34.

Table 7-34. Capital Cost
Peak Flow Strategy 2—Conventional Treatment

Item	Cost, $1,000
Headworks Renovation	482
Aeration Basin Modifications	687
Secondary Clarifier Addition	2,400
Disinfection System Improvements	350
Outfall	988
Subtotal	4,907
Contingencies	981
Construction Cost	5,888
Engineering and Administration	1,178
Total Capital Cost	7,066

Evaluation of Alternatives

As shown in the capital cost estimate for Peak Flow Strategy 1, there is a tremendous expense associated with constructing the facilities necessary to temporarily store peak wet weather flows. While the total expense of plant capacity expansions required to provide full conventional treatment is also considerable, it is less than half the cost of raw sewage storage. Therefore, it is recommended that the City plan to provide conventional treatment for the full year 2024 peak wet weather flow of 26 mgd.

INFILTRATION AND INFLOW REMOVAL COST EFFECTIVENESS ANALYSIS

Because the levels of I/I in the Lebanon wastewater collection system exceed EPA guidelines, a cost effectiveness analysis for I/I removal is required. An I/I cost effectiveness analysis compares the cost of rehabilitating sewers and removing I/I against the cost of constructing larger treatment and conveyance facilities. There are three basic alternatives available for dealing with I/I:

- Construct facilities capable of conveying and treating the peak flows, including the significant contribution from I/I sources.
- Rehabilitate the entire collection system to reduce I/I, thereby reducing peak flows and the size and cost of new treatment and conveyance facilities.
- Rehabilitate selected segments of the collection system where I/I contributions can be reduced in a cost effective manner and the improvements restore the useful life of the collection system. Construct new treatment and conveyance facilities for the full projected peak flows. Any reduction of peak flow will extend the design period for the peak flow facilities.

I/I Removal

I/I is removed from a wastewater collection system through rehabilitation of pipes or manholes and disconnection of unwanted sources. The first step in an I/I removal program is to identify sources of I/I and prioritize rehabilitation projects.

In the fall of 1999, the City smoke tested 190,000 feet (80 percent) of the collection system to identify system defects. During this testing, 667 mainline segments were tested of which 98 segments included a total of 148 defects. Table 7-35 summarizes the defects identified. At this time, City crews are televising the reaches with defects which will lead to design and construction of the corrective measures for all identified inflow sources.

Table 7-35. Identified Defects

Description or Source	Number
Leaking sanitary manhole	24
Storm sewer manhole	2
Main sewer leaks	11
Leaking cleanout	32
Catch basin	34
Area drain	3
Transition joint	1
Leaking service lateral	39
Driveway drain	1
Downspout	1

Based on this survey and the I/I removal report submitted and approved by DEQ as part of the NPDES Schedule C requirements, the quantity of I/I and the cost effectiveness of removal was estimated. This assessment resulted in the development of priorities for rehabilitation. Table 7-36 summarizes the capital cost for the recommended improvements which would include most of the deficiencies identified above in Table 7-35.

Table 7-36. Capital Cost
I/I Removal and Rehabilitation

Description	Cost, $1,000
I/I Removal and Rehabilitation Phase III	540
I/I Removal and Rehabilitation Phase IV	450
I/I Removal and Rehabilitation Phase V	456
Subtotal	1,446
Contingencies	289
Total Construction Cost	1,735
Engineering and Administration	347
Total Capital Cost	2,082

Eliminating a number of I/I sources does not guarantee a corresponding reduction in I/I and peak flows. In many cases, eliminating a significant portion of the total number of I/I sources has resulted in little or no peak flow reduction. This can be attributed to several factors:

- When pipe defects are repaired in the upper reaches of a collection system, water migrates down the trench through the granular pipe bedding and backfill material. The water enters the collection system once it reaches a defective pipe segment or manhole downstream of the rehabilitated area.
- When large areas are rehabilitated, the groundwater level can actually rise because the sewers are no longer functioning as an underdrain system. The higher groundwater level subjects a larger portion of the collection system to possible infiltration problems.
- Some sewers are flowing full during peak storm events—no additional water can be carried. Eliminating only a portion of the I/I sources may not be enough to reduce the flow in the sewer below its full-flowing capacity.
- Few cities have replaced faulty service laterals as part of their sewer rehabilitation programs. Because the laterals are located primarily on private property, there are a number of issues that must be addressed before they can be replaced or repaired. Therefore, the I/I associated with service laterals is not removed as part of most rehabilitation programs.

The cost for comprehensive sewer system rehabilitation to essentially eliminate I/I, including engineering and contingency, is estimated at approximately $50 million. Of this, approximately $10 million is allocated to replacing service laterals and $40 million is for replacing and rehabilitating sewer mains and manholes.

Recommended I/I Reduction Program

While the cost for a comprehensive collection system rehabilitation program is not warranted, the City should nevertheless continue work on the I/I removal program as part of the NPDES Schedule C requirements and search for opportunities to reduce I/I on a case-by-case basis. Rehabilitation of all inflow sources and other significant system deficiencies is cost effective and should be integrated with the objectives of ongoing collection system maintenance programs. For example, replacement of a broken pipeline to prevent potential sinkholes in a street will also help reduce peak flows since a broken line can be a significant source of infiltration. The City's ongoing efforts to comply with the I/I removal program will provide some level of I/I reduction through correction of significant deficiencies such as connected catch basins and area drains.

Further, the City's ongoing television inspection program has the ability to reveal other deficiencies such as structural failures. In particular, television inspection of the sewer system at creek and canal crossings may be important for identifying major sources of infiltration and inflow that would not be revealed through smoke testing. In evaluating the success of this type of I/I reduction program, it is important to realize that as a collection system ages, rehabilitation work is often offset by new deficiencies and new inflow connections. Therefore, expectations for these programs should be geared toward restoring the useful life of the collection system rather than simply elimination of I/I.

Based on the work completed to date, all inflow sources and the other high priority defects have been targeted for repair and many of the repairs have been completed in recent years. The listing of these defects is shown in Appendix A. Finally, the City should adopt policies on procedures to be employed to ensure that house services are repaired when they are found to be defective.