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Wastewater Treatment Plant Facilities Plan

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EXPIRES: 6/30/2026

Prepared for

City of Lebanon
925 Main Street
Lebanon, Oregon 97355

KJ Project No. 2176005.00

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Executive Summary

This wastewater facilities planning document (Plan) was prepared for the City of Lebanon (City). It provides recommendations for how the City will meet its wastewater treatment needs for the next 20 years and is intended to adhere to planning guidelines (Oregon DEQ, 2019) set forth by the Oregon Department of Environmental Quality (DEQ). The Plan may be used to seek Clean Water State Revolving Fund (SRF) funding for the improvements identified and recommended herein for the City's wastewater treatment plant (WWTP).

The Plan includes the following elements:

- **Basis of Project Planning** including descriptions of environmental resources, climate, population growth, and wastewater flows and loads for the WWTP service area.
- **Regulatory Considerations** providing a summary of the WWTP's existing National Pollutant Discharge Elimination System (NPDES) permit, potential permit changes, and summaries of regulatory requirements for recycled water and biosolids.
- **Existing Facilities** including an overview of the City's existing sewage collection system with infiltration and inflow (I&I) analysis, and descriptions of the WWTP's liquid stream treatment process, solids stream treatment processes, and general WWTP facilities.
- **Need for Project** including a review of past WWTP performance, summary of recent NPDES permit violations, capacity evaluations of the WWTP's liquid stream and solids stream treatment processes, and summary of identified deficiencies.
- **Alternatives Considered** that describe screening criteria and the selection of improvement alternatives for both liquid stream and solids stream treatment processes to receive additional evaluation.
- **Alternatives Selection** that summarizes the alternatives selection process for WWTP improvements and includes the basis for selection of an I&I cost effectiveness analysis.
- **Proposed Project** describing recommended WWTP improvements with design data, potential project phasing, engineer's opinion of probable construction cost (OPCC) for each project phase, and implementation schedules.
- **Additional Recommendations** that include recommendations for financial analyses, value engineering, and environmental resource studies needed to meet potential funding agency requirements.

ES.1 Basis of Planning and Regulatory Considerations

The City's current service population is approximately 19,120. The planning year used for this Plan is 2045. Population values for the planning period were obtained from the Portland State University (PSU) coordinated population forecast for Linn County (PSU Population Research Center, 2021). Population values used for the Plan are summarized in Table ES-1.

Table ES-1: City of Lebanon Population Projection[a]

Year	Population[b]
2021	19,122
2045	22,483
2070	23,525

Notes:

(a) Based on Portland State University (PSU) coordinated population forecast for Linn County (PSU Population Research Center, 2021).

(b) Annual average growth rate per the coordinated population forecast is 0.4% for years 2020 through 2045 and 0.2% for years 2045 through 2070.

City staff have expressed concern with the PSU Population Research Center coordinated population forecast stating they anticipate higher rates of population growth. In accordance with DEQ planning guidelines, the PSU coordinated population projections were nonetheless utilized for this Plan. Due to changes in state housing demands, the City believes actual population growth will be greater than that forecasted by PSU. Population values should be re-examined immediately prior to initiating design of improvements recommended by this Plan and the most current available coordinated forecast values should be used to make changes to wastewater flow and mass load projections.

Wastewater flow and mass load projections were completed utilizing WWTP daily monitoring reports (DMRs) for the 5-year period 2017 through 2021. Per capita flow and mass load values were determined with peaking factors developed for high flow and loading conditions. DEQ guidelines for making flow projections in western Oregon (DEQ, 1996) were followed in completing the projections.

Wastewater flow values are presented in Table ES-2.

Table ES-2: City of Lebanon Wastewater Flows[a]

	Wastewater Flow Rate, MGD[b]	
Condition	Current (2021)	Design Year (2045)
Average Dry Weather Flow (ADWF)	2.3	2.7
Average Wet Weather Flow (AWWF)	6.4	7.1
Average Daily Flow (ADF)	4.3	5.0
Max Month Dry Weather Flow (MMDWF)	5.4	6.0
Max Month Wet Weather Flow (MMWWF)	9.2	10.1
Peak Day Flow (PDF)	18.1	19.4
Peak Instantaneous Flow (PIF)	29.2	31.1

Notes:

(a) Flow values rounded to nearest 0.1 MGD.

(b) MGD = million gallons per day.

Wastewater mass loading values are presented in Table ES-3.

Table ES-3: City of Lebanon Wastewater Loading[a]

Parameter	Current (2021)	Design Year (2045)	Peaking Factor
BOD_5, ppd[b]			
Average	3,200	3,800	1[c]
Max Month	5,200	6,100	1.6
Peak Day	10,400	12,200	3.2
TSS, ppd			
Average	3,400	4,000	1[c]
Max Month	6,200	7,300	1.8
Peak Day	12,000	14,200	3.5
Ammonia, ppd			
Average	420	500	1[c]
Max Month	680	800	1.6

Notes:

(a) Loading values for BOD_5 and TSS rounded to the nearest 100 pounds. Ammonia rounded to the nearest 10 pounds.
(b) ppd = pounds per day.
(c) The average condition was used as the basis for calculation of peaking factor.

In preparing this Plan, Kennedy Jenks Consultants Inc. (Kennedy Jenks) considered potential changes to the City's NPDES permit that may be made during the upcoming permit renewal process. Changes considered included those associated with the Total Maximum Daily Load (TMDL) process and other potential pollutants including toxics, ammonia, chlorine, copper, pH, and thermal plume impacts.

Anticipated effluent permit limits are presented in Table ES-4. In addition to the permit limits shown, the City should expect the DEQ to require the development and implementation of a mercury minimization plan as required by the 2021 Willamette Basin mercury TMDL.

Table ES-4: Anticipated Effluent Permit Limits

Parameter	Units	Monthly Average	Weekly Average	Daily Maximum
$CBOD_5$ (May 1 – October 31)	mg/l	10	15	-
	lbs/day	250	380	500
	% removal[a]	85	-	-
TSS (May 1 – October 31)	mg/l	10	15	-
	lbs/day	250	380	500
	% removal[a]	85	-	-
$CBOD_5$ (November 1 – April 30)	mg/l	25	40	-
	lbs/day	1,500	2,200	3,000
	% removal[a]	85	-	-
TSS (November 1 – April 30)	mg/l	30	45	-
	lbs/day	1,800	2,700	3,600
	% removal[a]	85	-	-
E. coli Bacteria[b]	#/100 mL	Shall not exceed a monthly geometric mean of 126, no single sample shall exceed 406.		
pH	SU	Instantaneous limit between a daily minimum of 6.0 and a daily maximum of 9.0.		

Parameter	Units	Monthly Average	Weekly Average	Daily Maximum
Ammonia – N	mg/L	Shall not exceed a monthly average of concentration of 0.94 and a daily maximum concentration of 1.89.		
Total Chlorine Residual	mg/L	Shall not exceed a monthly average concentration of 0.01 and a daily maximum concentration of 0.02.		
Excess Thermal Load[(c)] (April 1 – May 15)	million kcal/day	$(0.03°C)*(4.1cfs+Q_R)*2.447$		
Excess Thermal Load[(c)] (May 16 – Oct 14)	million kcal/day	$(0.05°C)*(4.9cfs+Q_R)*2.447$		
Excess Thermal Load[(c)] (Oct 15 – Nov 15)	million kcal/day	$(0.08°C)*(12.3cfs+Q_R)*2.447$		

Notes:

(a) $CBOD_5$ and TSS removal efficiency shall not be less than 85% monthly average except when monthly average flows exceed 3.0 MGD which shall not be less than 75% monthly average.

(b) If a single sample exceeds 406 organisms per 100 mL, then five consecutive re-samples may be taken at 4-hour intervals beginning within 28 hours after the original sample was taken. If the log mean of the five re-samples is less than or equal to 126 organisms per 100 mL, a violation is not triggered.

(c) Dynamic wasteload allocation (kilocalories/day) expressed as a rolling 7-day average calculated on a daily basis where Q_R is the daily mean river flow (cfs). Equations are based on values included in the *Draft Total Maximum Daily Loads for the Willamette Subbasins (Willamette Mainstem and Major Tributaries) Temperature*, dated August 2024.

ES.2 Existing WWTP Facilities and Need for Project

The City maintains and operates a sewage collection system that includes over 300,000 lineal feet of pipe, 1,400 manholes, 4,600 service laterals, and 8 pump stations. The WWTP, which is located to the northeast of the City, was originally constructed in 1954 and consisted of a grit channel, comminutor, primary clarifier, trickling filter, secondary clarifier, chlorine disinfection, aerobic digester, and sludge drying beds. A significant upgrade project was completed in 1976 with multiple additional smaller improvement projects being completed since then.

Current liquid stream unit processes include the following:

- Influent pumping
- Headworks screening (no grit removal)
- Activated sludge type secondary treatment process
- Four secondary clarifiers
- Effluent pumping
- Chlorine disinfection (with dechlorination)
- Surface water discharge to the South Santiam River.

Solids stream unit processes include the following:

- Waste activated sludge (WAS) pumping
- Sludge thickening
- Aerobic digestion
- Dewatering
- Class B biosolids lime stabilization
- Class B biosolids liquid land application program.

Capacity analyses were conducted to identify deficiencies within the existing liquid and solids stream treatment processes that should be addressed to meet current and design year 2045 conditions. General WWTP facilities were also considered in this evaluation. Deficiencies identified within the liquid stream treatment process are summarized in Table ES-5.

Table ES-5: Liquid Stream Treatment Process Deficiencies

Process Area	Deficiencies
Influent Pumping	• Firm capacity of the West Side Interceptor Pump Station (WIPS) at 16 MGD does not meet peak flow conditions • Design year 2045 PIF is 31.1 MGD • Raw Sewage Station is inoperable and has been removed from service
Headworks	• Aging structure that was constructed as part of the 1976 WWTP upgrades project • Lack of downstream flow control that reduces screening efficiency • No grit removal
Secondary Treatment	• Insufficient treatment capacity • Aeration basin volume is approximately 50% of that needed to treat design year 2045 maximum month loading conditions • Aeration system capacity is approximately 80% of that needed during design year 2045 maximum month loading and less than 50% of that needed for the maximum day condition
Secondary Clarification (Clarifiers 1 through 3)	• Aging structures and equipment originally installed in 1976 and well beyond the normal 20-year design life for mechanical equipment • Poor settling performance • Uneven flow distribution • Insufficient depth
Filtration	• Tertiary filtration needed to reliably meet effluent permit conditions for $CBOD_5$ and TSS under specific flow conditions • Existing filtration facilities are inoperable and have been removed from service
Secondary Effluent Pumping	• Firm capacity of 16.3 MGD does not meet peak flow conditions • Design year 2045 PIF is 31.1 MGD

Process Area	Deficiencies
Disinfection	• Existing chlorine contact basins do not meet DEQ requirements for contact time during peak flow conditions (approximately 70% of that required under design year 2045 PIF) • Limited freeboard during design year 2045 PIF flow condition • Sodium hypochlorite storage and pumping relocated outdoors because of poor physical condition of indoor facilities

Deficiencies identified within the solid stream treatment process are summarized in Table ES-6.

Table ES-6: Solids Stream Treatment Process Deficiencies

Process Area	Deficiencies
Sludge Thickening	• Aging equipment (more than 30 years old)
Aerobic Digestion	• Insufficient volume with inability to meet Part 503 detention time requirements for aerobic digestion (less than 70% of that needed for current conditions and less than 60% of that needed for design year 2045 conditions) • Aerobic digester was originally constructed in 1954 and is more than 70 years old • Aerator/mixers were installed in 1976 and well beyond the normal 20-year design life for mechanical equipment
Biosolids Dewatering	• Inconsistent dewatering results • WWTP staff are concerned that the existing belt filter press cannot achieve the manufacturer's stated capacity
Class B Biosolids Land Application	• DEQ authorized sites that are currently available for use do not provide sufficient area to meet annual biosolids production (approximately 80% of that needed for current conditions and 70% of that needed for design year 2045) • Biosolids storage is insufficient for wintertime storage of biosolids (approximately 30% of that needed for current conditions and 25% of that needed for design year 2045)

Deficiencies associated with general WWTP facilities are summarized in Table ES-7.

Table ES-7: General WWTP Facilities Deficiencies

Process Area	Deficiencies
Control Building	• Aging structure in poor condition that was originally constructed in 1954 and thus more than 70 years old • Insufficient laboratory and working space
Electrical Infrastructure	• Much of the existing MCCs, transformers, and panelboards were installed as part of the 1976 upgrades project and are in poor condition and obsolete • Existing 1,250-kW standby generator does not have sufficient capacity to provide backup power for design year 2045

ES.3 Alternatives Evaluation and Selection

Kennedy Jenks considered multiple alternatives to address deficiencies within existing WWTP processes. An initial screening analysis of these alternatives was conducted to eliminate alternatives that were not deemed viable. The remaining alternatives received further evaluation with preferred alternatives being selected based on economic and non-economic factors.

Alternatives to address deficiencies within the liquid stream treatment process that received additional detailed evaluation are summarized in Table ES-8.

Table ES-8: Liquid Stream Treatment Alternatives Evaluated

Process Area	Alternatives Evaluated
Influent Pumping	• Expand existing WIPS
Headworks	• Upgrade existing headworks (without grit removal) • New headworks (with grit removal)
Primary Treatment[(a)]	• Primary filtration (cloth media filtration)
Secondary Treatment	• Expand existing activated sludge process • New conventional activated sludge process • New oxidation ditch • New extended aeration process
Secondary Clarification	• New 130-foot-diameter secondary clarifier
Tertiary Filtration	• Cloth media filtration
Secondary Effluent Pumping	• Expand existing effluent pump station
Disinfection	• Expand existing chlorine disinfection system • New UV disinfection system

Note:

(a) Alternative considered in conjunction with the anaerobic digestion alternative for solids stream treatment improvements.

Alternatives to address deficiencies within the solid stream treatment process that received detailed evaluation are summarized in Table ES-9.

Table ES-9: Solid Stream Treatment Alternatives Evaluated

Process Area	Alternatives Evaluated
Solids Thickening	• New rotary drum thickener
Solids Stabilization	• Expand existing aerobic digester • New aerobic digester • New anaerobic digester[(a)]
Biosolids Dewatering	• Continued use of existing belt filter press
Biosolids Beneficial Use or Disposal	• Class B dewatered cake land application • Class A biosolids with belt dryer • Landfill disposal

Note:

(a) Anaerobic digestion alternative assumed use of primary treatment.

Development of a recycled water alternative that would be implemented if the City lost its ability to discharge treated effluent to the South Santiam River was considered. However, as described in Section 6 of this Plan, reasonable potential analyses (RPAs) for toxics, ammonia, chlorine, copper, pH, and thermal plume as well as an evaluation of existing and potential TMDLs including the soon to be finalized temperature TMDL showed nothing to suggest the DEQ would not allow continued year-round discharge to the South Santiam River. While future changes in regulatory requirements may dictate reevaluation of this conclusion, there is currently no regulatory or economic driver for development of a recycled water program. This alternative was thus eliminated from further consideration.

An I&I cost effectiveness analysis was completed and showed that it is more cost effective to continue to convey and treat current levels of I&I than to remove it. The City should, however, continue the planned completion of its current five-phase I&I reduction program. This will eliminate direct storm water connections to the sanitary sewer system within the City's downtown area. The work will also complete smoke testing of the entire collection system. Additional direct connections discovered during that testing should be corrected. Doing so will protect the City's investment in current wastewater infrastructure and may reduce capacity needs for future WWTP improvements.

Improvements recommended for the WWTP identified during the alternative selection process are summarized in Table ES-10.

Table ES-10: Recommended WWTP Improvements

Process Area	Recommended Improvement
Influent Pumping	• Expand existing WIPS
Headworks	• Replace existing headworks with new facilities (including grit removal)
Secondary Treatment	• Replace existing activated sludge process with a new oxidation ditch

Process Area	Recommended Improvement
Secondary Clarification	• Replace existing Clarifiers 1 through 3 with one new secondary clarifier • Continue to use existing Clarifier 4
Tertiary Filtration	• Construct a new cloth media filtration system
Secondary Effluent Pumping	• Expand existing pump station
Disinfection	• Replace the existing chlorine disinfection system with a new UV disinfection system
Solids Thickening	• Replace the existing gravity belt thickener with a new rotary drum thickener
Biosolids Dewatering	• Continue to use the existing belt filter press
Biosolids Beneficial Use	• Replace the existing Class B land application program with a Class A biosolids treatment • Continue to use the existing aerobic digester with addition of coarse bubble aeration/mixing
General WWTP Facilities	• Construct a new Control Building • Provide new electrical equipment (e.g., MCC's, transformers, panelboards) as needed to support WWTP improvements • Replace the existing standby generator as electrical needs increase with implementation of recommended WWTP improvements

ES.4 Recommended Projects

Because of the significant cost and potential disruption to WWTP operations, recommended improvements should be phased and constructed over time. Priority should be given to those projects that would address the most acute deficiencies within the WWTP. Improvements to the WWTP have thus been divided into three project phases.

Phase 1 will address deficiencies within the solids stream treatment process. It will include new solids thickening equipment, a Class A biosolids treatment system, a new Control Building, and a new Electrical Building. Recommended Phase 1 improvements and the engineer's opinion of probable construction cost (OPCC) are presented in Table ES-11.

Table ES-11: Phase 1 Solids Stream Improvements Project OPCC[(a)]

Item	Total[(b)]
Construction Costs	
Solids Thickening	$1,120,000
Class A Biosolids Facilities	$17,280,000
Control Building	$2,000,000
Electrical Building	$1,630,000
Subtotal	$22,030,000
Contingency[(c)]	$6,620,000
Escalation[(d)]	$5,950,000
Total Construction	$34,600,000

Item	Total[b]
Engineering and Admin.[e]	$8,650,000
Total Project Cost	$43,250,000

Notes:

(a) Association for the Advancement of Cost Estimating (AACE) Class 4 planning level costs.
(b) Values rounded to the nearest $10,000.
(c) Contingency equal to 30%.
(d) Annual escalation rate of 6.5%.
(e) Engineering, legal, and administration equal to 25%.

Phase 2 will address capacity deficiencies within the existing liquid stream treatment process that could lead to permit violations. It will include new headworks, oxidation ditch, and secondary clarifier. Recommended Phase 2 improvements and the engineer's OPCC are presented in Table ES-12.

Table ES-12: Phase 2 Liquid Stream Improvements Project OPCC[a]

Item Description	Total[b]
Construction Costs	
Headworks	$2,070,000
Oxidation Ditch	$8,640,000
Secondary Clarifier	$6,070,000
Subtotal	$16,780,000
Contingency[c]	$5,030,000
Escalation[d]	$4,530,000
Total Construction	$26,340,000
Engineering and Admin.[e]	$6,580,000
Total Project Cost	$32,920,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Values rounded to the nearest $10,000.
(c) Contingency equal to 30%.
(d) Annual escalation rate of 6.5%.
(e) Engineering, legal, and administration equal to 25%.

Phase 3 will address remaining deficiencies within the existing liquid stream treatment stream. It will include improvements to the influent and effluent pumping systems, UV disinfection, tertiary filtration, and a new standby generator. Recommended Phase 3 improvements and the engineer's OPCC are presented in Table ES-13.

Table ES-13: Phase 3 Final Liquid Stream Improvements Project OPCC[a]

Item Description	Total[b]
Construction Costs	
Influent Pumping	$2,410,000
Effluent Pumping	$880,000
UV Disinfection	$2,080,000

Tertiary Filtration	$1,740,000
Generator and Assoc. Equip.[(c)]	$2,760,000
Subtotal	$9,870,000
Contingency[(d)]	$2,960,000
Escalation[(e)]	$2,670,000
Total Construction	$15,500,000
Engineering and Admin.[(f)]	$3,880,000
Total Project Cost	$19,380,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Values rounded to the nearest $10,000.
(c) Cost includes new standby engine generator, service entrance, switchboard, ATS and MCC's for Phase 3 improvements.
(d) Contingency equal to 30%.
(e) Annual escalation rate of 6.5 percent.
(f) Engineering, legal, and administration equal to 25 percent.

Recommended WWTP improvements and the project phase in which they will occur are shown on Figure ES-1.

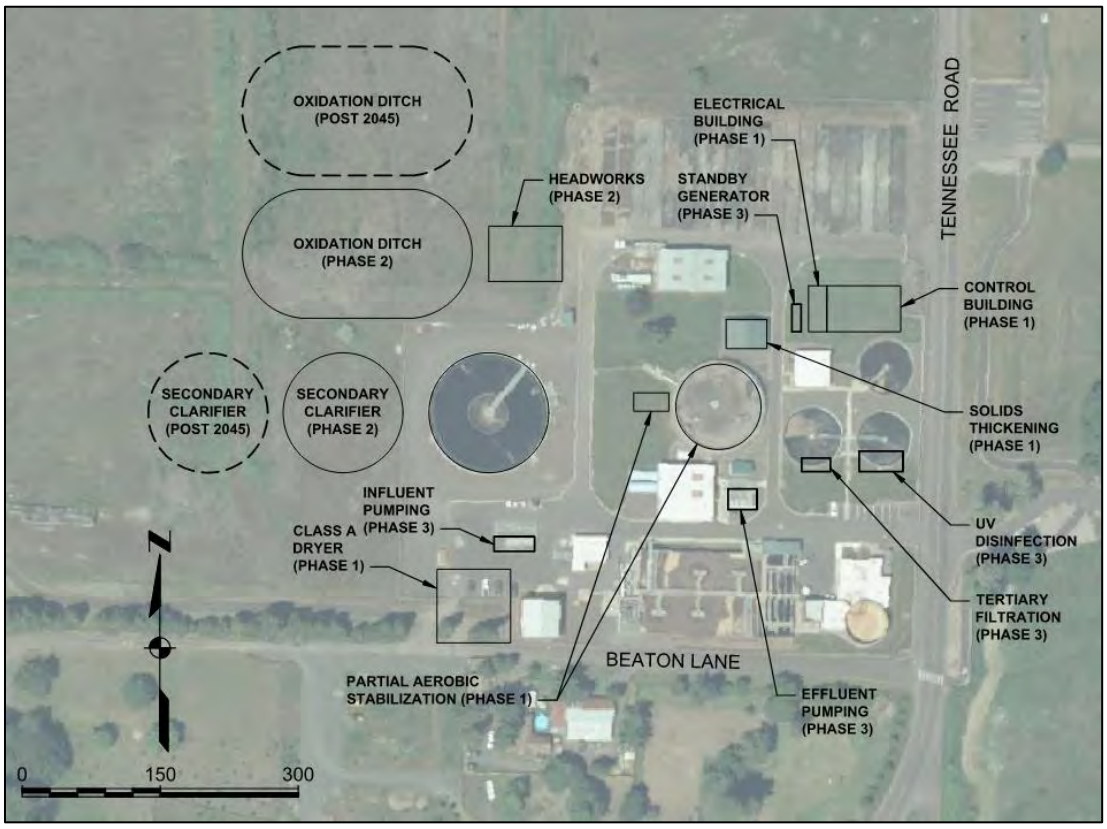


Figure ES-1: WWTP Improvements and Project Phases

Total population within the City's Urban Growth Boundary (UGB) is projected to increase by only 5% between the design year 2045 and 2070. Although it is important to consider future wastewater facility needs, specific requirements to meet ultimate buildout conditions were not

evaluated for this Plan. Instead, space should be reserved within the WWTP to allow for some level of expansion. As shown on Figure ES-1, this was done by reserving space for construction of a second oxidation ditch and a third 130-foot secondary clarifier. Doing this will provide sufficient space for future expansion even if future growth is significantly greater than that currently projected.

An implementation schedule for the three improvement phases is shown on Figure ES-2. The Phase 1 solids stream improvements project is projected to begin in January 2026 with construction being completed by the end of 2028. Design of the Phase 2 liquid stream improvements project would begin in September 2027 during construction of the Phase 1 project. Phase 2 would be complete in September 2030. Design of the Phase 3 final liquid stream project would begin in the summer of 2029 during construction of the Phase 2 project. Phase 3 would be complete in early 2032.

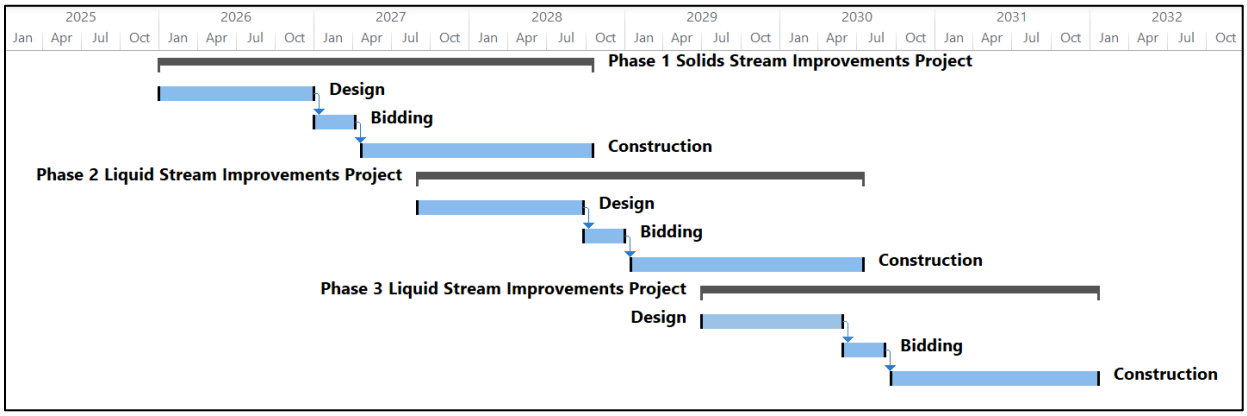


Figure ES-2: WWTP Improvements Project Phases Schedule

ES.5 Additional Recommendations

Additional steps that the City should take as it moves forward with the recommended WWTP improvements include the following:

- **Rate Study.** A rate study is recommended to provide the City with a thorough understanding of the costs of current wastewater services and potential changes in user rates (or alternative funding sources) needed to support the recommended improvements to the WWTP. The rate study should include initial data collection; development of a long-term financial model considering current revenue, forecasted costs for planned improvements, funding alternatives, and rate increase strategies; detailed financial analysis to distribute costs to customer classes; and rate analysis to evaluate alternative rate structures.

- **Value Engineering.** Depending on funding sources utilized by the City, a value engineering study may be required. For example, projects with a total cost greater than $10 million that receive DEQ's SRF funding must complete a value engineering study. Value engineering is typically done during the pre-design stage of the project and provides an independent review of project alternatives with a goal of identifying changes that could result in cost savings or improvements in reliability or performance.

- **Environmental Review.** Environmental review of improvement projects will likely be required. Specific requirements for this review will be dictated by the funding source. For example, the State Environmental Review Process (SERP) will be required for projects funded through the DEQ's SRF. Federal funding such as that administered by the Environmental Protection Agency (EPA) would require completion of environmental documentation to comply with requirements of the National Environmental Policy Act (NEPA).

Section 1: Project Planning

This section provides an introduction and background to the facilities planning process that includes a brief discussion on location, environmental resources, climate, population projections, land use, community engagement, and wastewater flows and loads.

1.1 Introduction and Background

The City of Lebanon (City) operates and maintains an activated sludge type wastewater treatment plant (WWTP) located at 33110 Tennessee Road in Lebanon, Oregon. The facility operates under National Pollutant Discharge Elimination System (NPDES) Permit Number 101771 with a permitted average dry weather flow of 3 million gallons per day (MGD). A copy of the NPDES permit is provided in Appendix A. This WWTP planning document (Plan) is intended to adhere to wastewater facility planning guidelines set forth by the Oregon Department of Environmental Quality (DEQ) and may be used to seek Clean Water State Revolving Fund (SRF) funding for the improvements identified and recommended herein.

1.2 Location

The City is located in Linn County, Oregon, within the Willamette Valley and Cascade Mountain Range foothills. The City is about 85 miles south of Portland, 45 miles north of Eugene, and 18 miles east of Corvallis. The South Santiam River runs through the City from north to south and most of the City's land area is located on the west side of the river. The terrain is mostly flat. The land slopes from south to north, with a south elevation of 380 feet to a north elevation of 330 feet. East of the river, elevation increases toward the Cascade Mountain Range. Most topography in the Lebanon Urban Growth Boundary (UGB) is conducive to development. However, Ridgeway Butte on the east side of the South Santiam River has been identified by the State as having potential for mass movement or landslide. Ridgeway Butte is located about 3.2 miles driving distance from the City's WWTP. A copy of the flood insurance rate map showing the 100-year and 500-year flood zones for the area encompassing the WWTP is provided in Appendix B. The project planning area served by the City's WWTP is presented on Figure 1-1.

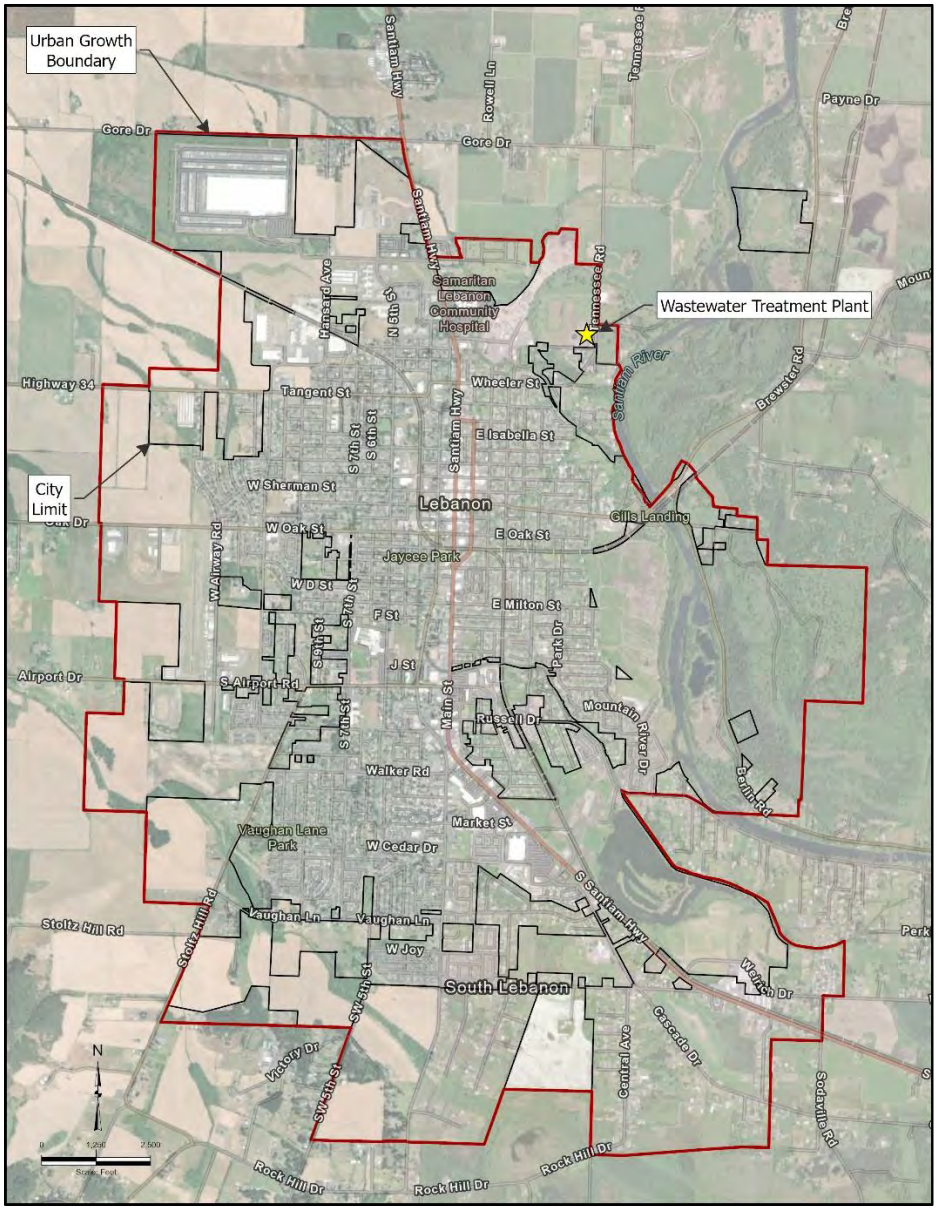


Figure 1-1: Lebanon WWTP Service Area

1.3 Environmental Resources

The following provides a summary of environmental resources within the City.

1.3.1 Water Resources

The South Santiam River runs through the City. The river's headwaters are in the central Cascade mountains at the confluence of Latiwi and Sevenmile Creeks, and it eventually flows into the Willamette River Valley (South Santiam Watershed Council, 2022). The WWTP discharges to the South Santiam River at River Mile 17.4. DEQ samples and tests the South Santiam River from approximately River Mile 0 to River Mile 33.

The City and surroundings areas have plentiful groundwater with water levels yielding moderate to large quantities of water for area wells (City of Lebanon, 2004). Domestic groundwater wells serve many residences in Lebanon. Groundwater contamination was first identified in 1990 with the main contaminants of concern being Perchloroethylene (PCE) and trichloroethylene (TCE) [Oregon Health Authority, 2009]. An approximately 0.6 square mile area including a portion of the City's downtown was found to be impacted by the contamination. However, as noted in the 2009 Health Consultation (EHA, 2009) prepared by the Environmental Health Assessment Program (EHAP) within the Oregon Public Health Division, EHAP concluded that the contaminants posed no apparent hazard to public health through exposure to water from domestic or irrigation wells. This assumed that domestic well-issuers with contaminant levels greater than safe drinking water standards were utilizing alternate sources of drinking water.

1.3.2 Threatened and Endangered Species and Wildlife

The Oregon Natural Heritage Program identifies threatened or endangered species by county in Oregon. The program lists threatened or endangered fish, amphibians, reptiles, birds, mammals, invertebrates, plants, and fungi. Lebanon is located within Linn County and has several rare, threatened, or endangered species. Notable rare, threatened, endangered fish, or fish species of concern in Linn County include steelhead, chinook salmon, and bull trout. Notable birds on this list include the northern goshawk, sandhill cranes, bald eagles, and several species of woodpecker, owl, and goose. Notable mammals on this list include the gray wolf, and several species of vole and bat (Oregon Biodiversity Information Center at Portland State University, 2019).

1.3.3 Geology and Soils

Geologic formations in Lebanon mostly consist of quaternary surficial deposits within the plains of the Willamette and South Santiam River watersheds (Oregon Department of Geology and Mineral Industries, 2002). These are alluvial deposits including broad sandbars and flood plains. Soils near the river are typically well-drained (City of Lebanon, 2004). Beneath alluvial soils is the Rowland formation, with coarse gravel and sand up to 150 feet deep. Beneath the Rowland formation are ancient Willamette River deposits which consist of sand, gravel, and bluish clay. Test hole and test pit logs from geotechnical investigations completed for the 1976 Lebanon WWTP Upgrades Project indicate that there are typically a few feet of brown sandy silt at the surface above about 20 feet of brown sandy gravel.

The most common soil types within the planning area are Clackamas gravelly silt loam and Clackamas variant silt loam (U.S. Department of Agriculture Natural Resources Conservation Service, 2019). The WWTP site soil is 87% Camas gravelly sandy loam. This soil is characterized by significant drainage and 9 to 17 inches of strongly contrasting textural stratification. The frequency of flooding is occasional to none, and the frequency of ponding for this soil type is none. It is in hydrologic soil Group A and slopes 0 to 3 percent (%). A soils map, including the WWTP and the surrounding areas, is provided in Appendix C.

1.3.4 Mineral, Aggregate, and Energy Resources

Mineral resources in the area include sand, gravel, and crushed stone. In the Lebanon area, there are currently four active mining operating permits under Oregon's Department of Geology and Mining Industries (DOGAMI). There are several gravel pits and quarries in and around Lebanon on the east side of the South Santiam River. One natural gas well is located just

outside the northwest corner of the City's UGB (Oregon Department of Geology and Mineral Industries, 2002).

1.4 Climate

The City has a temperate climate, with warm summers and cool winters. Rain is light in summer but frequently occurs in fall, winter, and spring. The average daily temperature in Lebanon is 52.03°F. Annual average rainfall is 56.65 inches, and annual average snowfall is 6.21 inches (City of Lebanon, 2012). The mean maximum temperature during July and August was just over 80°F and the mean minimum temperature during December and January was just under 40°F (U.S. Climate Data, 2022).

1.5 Population Projections

The planning year used for this Plan is 2045. Per DEQ guidelines, Portland State University (PSU) Population Research Center projection data was used to develop flow and loading projections through the planning year. Population data was retrieved from PSU's "*2021 Coordinated Population Forecast for Linn County* and *Populations for Oregon and its Counties and Incorporated Cities and Towns"*. Flow and loading projections using the population data are described later in this section. Population for the City from 2016 through 2021 is summarized in Table 1-1 below.

Table 1-1: City of Lebanon Population (2016 - 2021)

Item	Value					
Year	2016[a]	2017[a]	2018[a]	2019[a]	2020[a]	2021[b]
Population	16,435	16,720	16,920	17,135	17,335	19,122

Notes:

(a) 2010 through 2020 data from Table 4 Populations for Oregon and its Counties and Incorporated Cities and Towns, compiled April 15, 2023, PSU Population Research Center.

(b) 2021 data from Table 4 Populations for Oregon and its Counties and Incorporated Cities and Towns, compiled April 15, 2022, PSU Population Research Center.

The planning year population, annual average growth rates (AAGR) and population forecasts are summarized in Table 1-2 below. Values shown in the table are for populations within the City's UGB. Although slightly greater than those within the incorporated City limits, these values were used to allow for potential annexations that may occur prior to the 2045 planning year.

Table 1-2: City of Lebanon Population Forecast[a]

Historical Values			Estimated Values		Forecast Values			
Population		AAGR	Population	AAGR	Population		AAGR	
2000	2010	2000-2010	2020	2010-2020	2045	2070	2020-2045	2045-2070
15,913	18,295	1.4%	20,314	1.0%	22,483	23,525	0.4%	0.2%

Note:

(a) Values from Table 2 of Coordinated Population Forecast, 2021 through 2071, Linn County, Urban Growth Boundaries (UGB) & Area Outside UGBs, June 30, 2021, PSU Population Research Center.

Population growth beyond year 2045 is anticipated to slow to a rate of 0.2% and, as shown, the projected population in year 2070 is expected to be approximately 23,500. This represents only a 5% increase over the year 2045 value. While it is important to consider and plan for

wastewater facility needs beyond the 20-year planning horizon, because the rate of growth is so limited, specific requirements to meet ultimate buildout conditions were not evaluated during preparation of the Plan. Instead, as presented in Section 7, recommendations are made to reserve space within the WWTP to accommodate some level of future expansion should growth projections change significantly in the future.

City staff have expressed concern with the PSU Population Research Center coordinated population forecast stating that they anticipate higher rates of population growth. In accordance with DEQ planning guidelines, the PSU coordinated population projections were nonetheless utilized for this Plan. Due to changes in state housing demands, the City believes actual population growth will be greater than that forecasted by PSU. Population values should be reexamined immediately prior to initiating design of improvements recommended by this Plan and the most current available coordinated forecast values should be used to make changes to wastewater flow and mass load projections.

1.6 Land Use

Generalized zoning classifications in Linn County surrounding the City limits include exclusive farm use (EFU), farm/forest (F/F), and forest conservation and management (FCM) (Linn County Oregon, 2022). Within the City limits, land use is mostly residential low density, residential mixed density, industrial, mixed use, and highway commercial. Other land uses within the City include residential high density, public use, neighborhood mixed use, neighborhood commercial, and central business commercial. The WWTP is in an industrial land use area and borders other industrial land use areas and some residential and agricultural properties. A land use map for the planning area is provided in Appendix D.

1.7 Wastewater Flows and Loads

A detailed analysis of wastewater flows and loads was completed by Richwine Environmental, Inc. and a technical memorandum summarized the results. A copy of this memorandum is provided in Appendix E. The report projects wastewater flows through the 2045 planning year.

The memorandum analyzed WWTP daily monitoring reports (DMRs) from January 2017 through December 2021. Flow and loading data from January 2017 through December 2021, and population growth estimates developed by PSU were used to project flows and loading through the 2045 planning year.

1.7.1 Wastewater Flows

Wastewater flows and loading were calculated using guidelines developed by the DEQ for use in western Oregon. Definitions of the various flow conditions calculated are as follows:

- **Average dry weather flow (ADWF)** is the average flow at the WWTP during the dry weather season, defined as May through October.
- **Average wet weather flow (AWWF)** is the average flow at the WWTP during the wet weather season, defined as November through April.
- **Average daily flow (ADF)** is the annual average daily flow.

- **Maximum month dry weather flow (MMDWF)** is the dry-weather, maximum month flow rate estimated to occur once every 10 years (10% probability of occurrence). In western Oregon, this value normally occurs during May and is associated with the monthly rainfall accumulation with a 10 percent probability of occurrence.

- **Maximum month wet weather flow (MMWWF**) is the wet-weather, maximum month flow estimated to occur once every 5 years (20% probability of occurrence). This value occurs during the wet weather season when groundwater levels are high (typically occurring from January through April). The value typically occurs in January and is associated with the monthly rainfall accumulation that has a 20% probability of occurrence.

- **Peak day flow (PDF)** is the flow rate corresponding to a 1-in-5 year, 24-hour storm event that occurs during a period of high groundwater and saturated soils.

- **Peak instantaneous flow (PIF)** is the peak hour flow rate expected to occur during the peak day flow.

Values calculated for current (2021) and design year 2045 flow conditions are shown in Table 1-3. Calculation methods, per capita flows, and further information regarding these flows is summarized in the technical memorandum provided in Appendix E.

Table 1-3: City of Lebanon Wastewater Flows[a]

	Wastewater Flow Rate, MGD[b]	
Condition	Current (2021)	Design Year (2045)
ADWF	2.3	2.7
AWWF	6.4	7.1
ADF	4.3	5.0
MMDWF	5.4	6.0
MMWWF	9.2	10.1
PDF	18.1	19.4
PIF	29.2	31.1

Notes:

(a) Flow values rounded to nearest 0.1 MGD.
(b) MGD = million gallons per day.

1.7.2 Wastewater Loading

Historical 5-day biochemical oxygen demand (BOD_5) and total suspended solids (TSS) data from January 2017 through December 2021 were analyzed to determine current (2021) and design year 2045 wastewater loading values. Average annual unit loading values for BOD_5 and TSS were determined to be 0.17 lbs per capita per day (ppcd) and 0.18 ppcd, respectively. The average annual unit loading value for ammonia was determined to be 0.018 ppcd.
A summary of wastewater loading values is presented in Table 1-4 below. Details of how these values were calculated are documented in the technical memorandum provided in Appendix E.

Table 1-4: City of Lebanon Wastewater Loading[a]

Parameter	Current (2021)	Design Year (2045)	Peaking Factor
BOD_5, ppd[b]			
Average	3,200	3,800	-
Max Month	5,200	6,100	1.6
Peak Day	10,400	12,200	3.2
TSS, ppd			
Average	3,400	4,000	-
Max Month	6,200	7,300	1.8
Peak Day	12,000	14,200	3.5
Ammonia, ppd			
Average	420	500	-
Max Month	680	800	1.6

Notes:

(a) Loading values for BOD5 and TSS rounded to nearest 100 pounds. Ammonia rounded to the nearest 10 pounds.

(b) ppd = pounds per day.

Section 2: Regulatory Considerations

This section provides a summary of current effluent permit limits, changes anticipated to occur during the next permit renewal period, and regulatory requirements for biosolids and recycled water.

2.1 Existing Effluent Discharge Permit

The City's existing WWTP operates under National Pollutant Discharge Elimination System (NPDES) permit number 101771, a copy of which is included in Appendix A of this document. The permit allows the discharge of treated effluent to the South Santiam River meeting seasonal concentration and/or mass load limits for 5-day, carbonaceous biochemical oxygen demand (CBOD5), total suspended solids (TSS), E. coli bacteria, ammonia, and chlorine. Effluent requirements for pH and CBOD5 and TSS removal efficiency are also contained within the current NPDES permit. The City's permit was scheduled to expire December 31, 2004, but has been administratively extended by the Oregon Department of Environmental Quality (DEQ). The permit is currently anticipated to be renewed in 2025.

Effluent limits contained in the City's current NPDES permit for CBOD5 and TSS are presented in Tables 2-1 and 2-2. Effluent permit limits for other parameters including bacteria, pH, ammonia, CBOD5 and TSS removal efficiency and total chlorine residual are presented in Table 2-3.

Table 2-1: $CBOD_5$ and TSS Limits (May 1 – October 31)[(a)]

Parameter	Average Effluent Concentrations Monthly	Weekly	Monthly Average lbs/day	Weekly Average lbs/day	Daily Maximum lbs
$CBOD_5$	10 mg/l[(b)]	15 mg/l	250	380	500
TSS	10 mg/l	15 mg/l	250	380	500

Notes:

(a) Limits applicable May 1 through October 31.
(b) mg/l = milligrams per liter.

Table 2-2: $CBOD_5$ and TSS Limits (November 1 – April 30)[(a)]

Parameter	Average Effluent Concentrations Monthly	Weekly	Monthly Average lb/day	Weekly Average lb/day	Daily Maximum lbs
$CBOD_5$	25 mg/l[(b)]	40 mg/l	1,500	2,200	3,000
TSS	30 mg/l	45 mg/l	1,800	2,700	3,600

Notes:

(a) Limits applicable November 1 through April 30.
(b) mg/l = milligrams per liter

Table 2-3: Other Parameter Limits (Year-round)

Parameter	Limit
E. coli Bacteria	Shall not exceed 126 organisms per 100 mL monthly geometric mean. No single sample shall exceed 406 organisms per 100 mL[(a)]
pH	Shall be within the range of 6.0 – 9.0
Ammonia – Nitrogen	Shall not exceed a monthly average concentration of 0.94 mg/l and a daily maximum concentration of 1.89 mg/l
$CBOD_5$ and TSS Removal Efficiency	Shall not be less than 85% monthly average except when monthly average flows exceed 3.0 MGD which shall not be less than 75% monthly average
Total Chlorine Residual	Shall not exceed a monthly average concentration of 0.01 mg/l and a daily maximum concentration of 0.02 mg/l

Note:

(a) If a single sample exceeds 406 organisms per 100 mL, then five consecutive re-samples may be taken at 4-hour intervals beginning within 28 hours after the original sample was taken. If the log mean of the five re-samples is less than or equal to 126 organisms per 100 mL, a violation is not triggered.

2.2 Potential Permit Changes

In anticipation of the renewal process for the City's NPDES permit, Kennedy Jenks considered new potential permit requirements that might be added. These include those associated with the Total Maximum Daily Load (TMDL) process and other potential pollutants or water quality parameters including toxics, ammonia, chlorine, copper, pH, and thermal plume impacts.

2.2.1 TMDL Related Permit Requirements

Treated WWTP effluent is discharged to the South Santiam River via a multi-port diffuser at river mile 17.4 which is within the South Santiam River assessment unit between Foster Dam and the confluence with the North Santiam River. This portion of the South Santiam River is listed in the DEQ's 2022 Integrated Report (Oregon DEQ, 2022) as being impaired (i.e., does not meet water quality standards) for the following four parameters:

- Biocriteria
- Temperature
- Alkalinity
- Methylmercury

The listing of impaired water bodies is commonly referred to as the "303(d) List" in reference to Section 303(d) of the federal Clean Water Act. When a water body is listed as being impaired, a TMDL must be developed that determines the loading capacity or the amount of a pollutant that a water body can receive while still meeting water quality standards. When implemented, the TMDL allocates a portion of the loading capacity to point and nonpoint sources while retaining certain percentages as a margin of safety, to account for seasonal variations, and as reserve capacity for future growth. The portion of a waterbody's loading capacity assigned to a

point source (e.g., WWTP) is a wasteload allocation (WLA). WLAs ultimately result in additional waste discharged limits generated during the NDPES permit renewal process.

A brief discussion on the four water quality parameters responsible for the impaired designation of the South Santiam River and potential permit impacts of the listings is presented below.

2.2.1.1 Biocriteria

The South Santiam River has been listed as being impaired for the biocriteria water quality parameter. The beneficial use designation impacted by this impairment is fish and aquatic life support. The water quality standard for biocriteria as included in the Oregon Administrative Rules (OAR) is as follows:

OAR 340-041-0011 Biocriteria: Waters of the State must be of sufficient quality to support aquatic species without detrimental changes in the resident biological communities.

The DEQ considers “detrimental changes in resident biological communities” to be a form of pollution and notes that “EPA guidance recommends using biological community assessments as an indicator for aquatic life beneficial use support” (Oregon DEQ, May 2022). The protocol used by the DEQ to assess the health of aquatic biological communities is based on freshwater macroinvertebrates (e.g., insects, crustaceans, snails, clams, worms, etc.). Sites throughout Oregon the DEQ considers to be least impacted by human activities are used as reference sites to “predict the variety and number of aquatic species expected in Oregon streams and to make inferences about the condition of biological communities in the waters” (Oregon DEQ, May 2022). When sampling data from the stream being evaluated shows more than a specific percentage loss of the macroinvertebrate population (15 to 20 percent for the model region applicable to the South Santiam River) compared to that expected in the absence of human activity, the water body is listed as being impaired and a TMDL must be developed. This is done even if the pollutant or underlying cause for the impairment is unknown.

The DEQ has not developed a biocriteria TMDL. While the DEQ has indicated in email correspondence that a standard may be developed by the time the City’s NPDES permit is renewed, this is not considered to be likely. For this reason, the biocriteria listing is not expected to result in additional restrictions being added to the City’s NDPES permit during the next renewal cycle.

2.2.1.2 Temperature

There were approximately 1,200 miles of Willamette Basin rivers and streams included in the 303(d) list for temperature impairment when the 2006 Willamette Basin Total Maximum Daily Load (2006 TMDL) was issued. The South Santiam River between Foster Dam and the confluence with the North Santiam River is currently listed for year-round temperature impairment related to salmon and trout rearing and migration. The river is also listed for temperature for salmonid and steel head spawning (October 15 through May 15). The temperature criterion for salmon and trout rearing and migration is 18.0°C while the criterion for spawning is 13.0°C.

The 2006 Temperature TDML for the Willamette River mainstem and major tributaries which includes the South Santiam River up to Foster Dam is being replaced. It is scheduled to be issued to the U.S. Environmental Protection Agency (EPA) in May 2025 for EPA action (approval or disapproval) by June 28, 2025. The DEQ issued the draft TMDL for a 45-day public

comment period during the summer of 2024. A discussion of the potential implications on the new TMDL and comparison with the existing 2006 TMDL limits is presented below.

2.2.1.2.1 2006 Temperature TMDL

The 2006 TMDL thermal wasteload allocations (WLA) for the Lebanon WWTP and associated parameters used by the DEQ in setting the allocations are shown in Table 2-4. The WLAs shown are applicable April through October and thus limit thermal discharge only during that period. Three WLAs were assigned: one value applicable to the salmon and trout rearing and migration period (May 16 through October 14) and two values applicable during the salmon and steelhead spawning period (October 15 through May 15). The two values shown for the spawning period differ based on wet weather and dry weather flow seasons as defined in the City's existing NPDES permit, November 1 through April 30 and May 1 through October 31, respectively.

Table 2-4: 2006 Thermal Wasteload Allocations (WLA) for the Lebanon WWTP[a]

Designated Fish Use	Rearing and Migration	Spawning	
Biological Criteria, °C	18	13	
WLA Period	May 16 – Oct 14	Apr 1 – Apr 30	May 1 – May 15 Oct 15 – Oct 31
HUA[b], °C	0.0512	0.0513	0.0673
River Flow, 7Q10[c], cfs[d]	510	665	665
Effluent Discharge[e], cfs (MGD)	6.96 (4.50)	13.92 (9.0)	6.96 (4.50)
WLA (7Q10), Million Kcal/day	65	85	111

Notes:

(a) Values as presented in Oregon DEQ, Willamette Basin Total Maximum Daily Load (TMDL), 2006 (Oregon DEQ, 2006).
(b) Human use allowance (HUA) assigned to the Lebanon WWTP point source. This is the fraction of the 0.3°C allowable human induced temperature increase authorized in waters listed for temperature that is assigned to the Lebanon WWTP.
(c) 7Q10: Annual minimum 7-day average river flow with a recurrence interval of 10 years.
(d) CFS = cubic feet per second.
(e) WWTP effluent discharge assumed by the DEQ in calculating the WLA.

The 2006 thermal WLAs for municipal sources that discharge to the entire Santiam River (including the Lebanon WWTP) were calculated based on current design flow limits and were thus at their maximum limit. This precluded the use of flow based WLAs that would otherwise allow sources to increase their thermal load with increased river flow. The WLA values shown in Table 2-4 are thus static values and cannot be adjusted upwards during periods when river flow is greater than the designated 7Q10 value (minimum 7-day average river flow with 10-year recurrence interval).

Because the WWTP has been operating under an administratively extended NPDES permit issued prior to implementation of the 2006 TMDL, the current permit does not contain excess thermal load limits. However, Kennedy Jenks analyzed WWTP effluent temperature and flow data from 2010 through 2023 to determine whether the WLAs in the 2006 TMDL could have

been met. A summary of that analysis is presented in Table 2-5. Years in which the existing WWTP would not have met the 2006 TMDL thermal allocations are shown and the associated excess effluent volume that would have been discharged given the effluent temperature during the period are noted.

Table 2-5: Periods the Lebanon WWTP Would Have Exceeded 2006 Temperature TMDL

Period	Excess Effluent Volume (MGal)[(a)(b)]
Oct 25, 2010	0.3
Sep 30 to Oct 2, 2013	2.5
Oct 15 to Oct 31, 2016	18.8
Apr 3 to May 15, 2023	33.4

Notes:

(a) Values rounded to the nearest 0.1 MGal.

(b) Effluent volume greater than that which could have been discharged had the 2006 TMDL WLAs for the Lebanon WWTP been implemented.

As shown in Table 2-5, there have been four periods since 2010 during which the WWTP would have exceeded the 2006 TMDL thermal WLAs had they been implemented. The largest of these events occurred in 2023. However, as presented on Figure 2-1, Kennedy Jenks compared 7 day rolling average effluent temperature values for that year with those of prior years and believes the 2023 temperature data to be suspect.

As shown on the figure, effluent temperatures for the WWTP followed a predictable seasonal pattern with minimum and maximum temperatures occurring in January through April and August through September, respectively. However, in the fall of 2022, effluent temperatures did not decrease in the same manner as in prior years. Instead, they remained between 17°C and 19°C throughout the fall of 2022 and spring of 2023. Furthermore, the average temperature in April 2023 was 18.1°C which was about 3°C to 5°C higher than during the same month for years 2010 through 2022.

The City reported that the monitoring location for determining effluent temperature was changed in the fall of 2022. Upon further discussion with the City, it is believed that the new monitoring location did not accurately reflect the actual temperature of the WWTP effluent. The City has subsequently returned the monitoring location to its original position and will continue to evaluate effluent temperatures to confirm that the higher temperatures in the fall of 2022 through spring of 2023 were an anomaly and not representative of actual conditions.

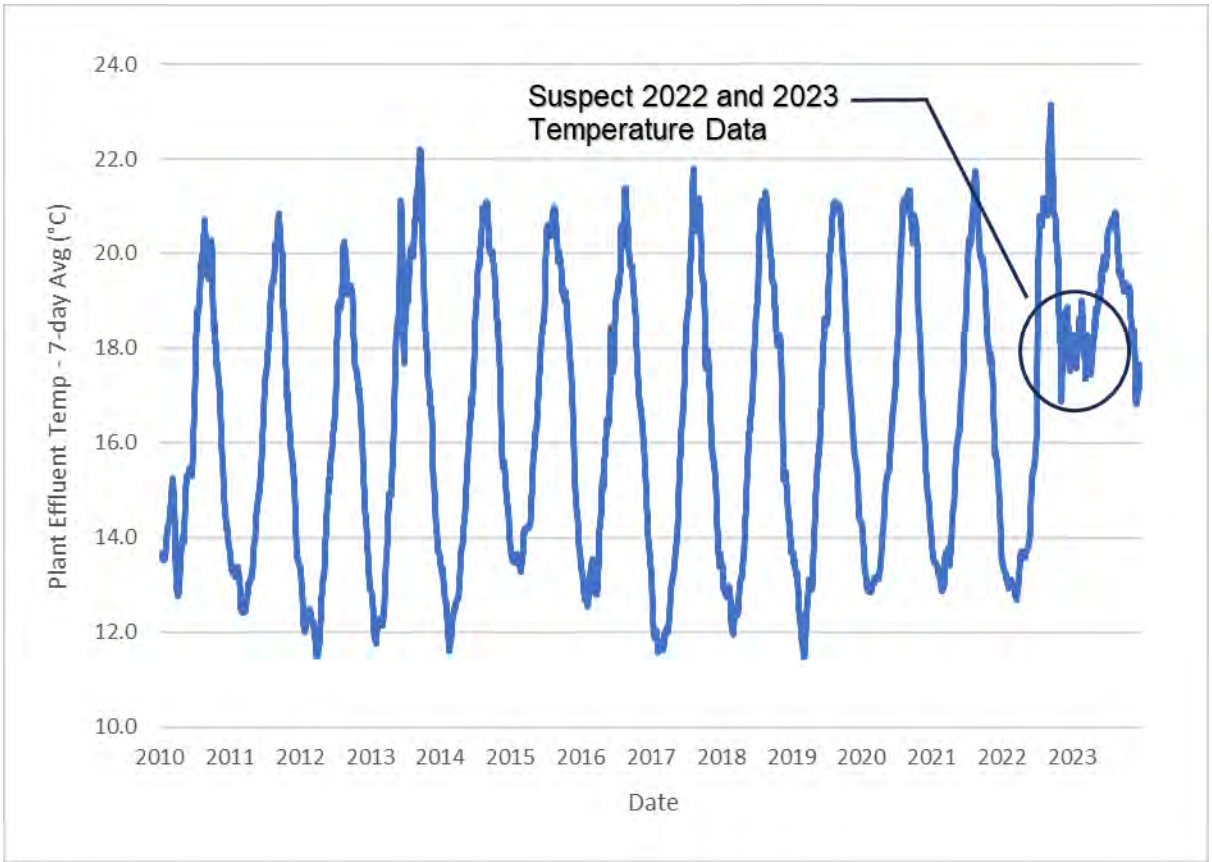


Figure 2-1: Lebanon WWTP 7-day Average Effluent Temperature

Even if the suspect 2022 and 2023 data are discounted, there have been three periods since 2010 during which the City would not have been able to meet the 2006 TMDL thermal WLAs. As shown in Table 2-5, in 2016 the WWTP would have had to store, divert, or cool more than 18 MGal of effluent to meet the WWTP's thermal load allocations had they been incorporated in the City's NPDES permit. If stored for later discharge to the river, it would have required a 6.1-acre storage basin (assuming a water depth of 10 feet). For reference, the decommissioned wastewater lagoons on the WWTP site have a combined area of approximately 7.5 acres.

2.2.1.2.2 2024 Draft TMDL

As previously noted, the 2006 temperature TDML for the Willamette River mainstem and major tributaries is being replaced and is scheduled for EPA action (i.e., approval or disapproval) by June 28, 2025. The DEQ issued the draft TMDL for a 45-day public comment period during the summer of 2024. Proposed WLAs for the Lebanon WWTP and the associated parameters used by the DEQ in setting those WLAs contained in the draft TMDL documents are shown in Table 2-6.

The DEQ has extended the period during which the WLAs apply. The critical period contained in the 2006 TMDL was April 1 through October 31. The overall critical period is now April 1 through November 15. As shown in Table 2-6, thermal allocations for the rearing and migration period and the April 1 through May 15 spawning period have decreased while the allocation for the October 15 through November 15 spawning period has increased.

Table 2-6: 2024 Thermal Wasteload Allocations (WLA) for the Lebanon WWTP[a]

Designated Fish Use	Rearing and Migration	Spawning	
Biological Criteria, °C	18	13	
WLA Period	May 16 – Oct 14	Apr 1 – May 15	Oct 15 – Nov 15
HUA[b], °C	0.05	0.03	0.08
River Flow, 7Q10[c], cfs	506	1,043	726
Effluent Discharge[d], cfs (MGD)	4.9 (3.17)	4.1 (2.65)	12.3 (7.95)
WLA (7Q10), Million Kcal/day	62.50	76.857	144.51

Notes:

(a) As presented in Oregon DEQ, *Draft Total Maximum Daily Loads for the Willamette Subbasins (Willamette Mainstem and Major Tributaries) Temperature*, August 2024 (Oregon DEQ, 2024).
(b) Human use allowance (HUA) assigned to the Lebanon WWTP point source. This is the fraction of the 0.3°C allowable human induced temperature increase authorized in waters listed for temperature that is assigned to the Lebanon WWTP.
(c) 7Q10: Annual minimum 7-day average river flow with a recurrence interval of 10 years.
(d) WWTP effluent discharge assumed by the DEQ in calculating the WLA.

Of special note, the draft 2024 TMDL documents state that thermal WLAs may be implemented in one of two ways. These include incorporating the WLAs shown in Table 2-6 as either static numeric limits or alternatively as dynamic limits that would be calculated daily. This contrasts with the 2006 TMDL that only allowed the implementation of static values for the South Santiam River. Dynamic limits allow sources to increase their thermal loading when river flows are greater than the 7Q10 value. Although it requires continuous monitoring of daily river flow data and calculation of daily thermal limits, the use of dynamic limits result in higher thermal WLAs which would be a significant advantage to the City.

Kennedy Jenks again analyzed WWTP effluent temperature and flow data from 2010 through 2023 to see if the WLAs could have been met if they were included in the City's NPDES permit. Static and dynamic limits were evaluated separately. Results of the static WLA analysis is summarized in Table 2-7.

Table 2-7: Periods the Lebanon WWTP Would Have Exceeded Draft 2024 Temperature TMDL (Static Limits)

Period	Excess Effluent Volume (MGal)[(a)(b)]
Jun 12 - Jun 13, 2013	0.5
Sep 5 - Sep 7, 2013	3.1
Sept 24 - Oct 3, 2013	14.1
Oct 16 - Oct 27, 2016	5.3
Sep 15, 2019	0.4
Nov 11, 2021	1.0
Nov 7, 2022	0.2
Apr 1 - May 11, 2023	93.1
Nov 7, 2023	0.1

Notes:

(a) Values rounded to the nearest 0.1 MGal.

(b) Effluent volume greater than that which could have been discharged assuming implementation of static WLAs proposed in the draft 2024 Temperature TMDL.

As shown in Table 2-7, there have been nine periods since 2010 during which the existing WWTP would have exceeded the draft 2024 Temperature TMDL had the static WLA's been implemented and incorporated into the City's NPDES permit. Although, as previously discussed, the occurrences starting in the fall of 2022 through the fall of 2023 could be discounted, the proposed 2024 Temperature TMDL would still have resulted in six instances during which the WWTP would have exceeded excess thermal load limits. In 2013, more than 14 MGal of effluent would have had to have been stored, cooled, or otherwise diverted to meet static thermal load allocations had they been incorporated in the City's NPDES permit.

In contrast, Kennedy Jenks' analysis of effluent flow and temperature data from 2010 through 2023 showed that there would have been no cases in which the WWTP would have exceeded excess thermal load limit had they been dynamically calculated. The results of this analysis are represented on Figure 2-2 showing the 7-day rolling excess thermal load (ETL) value for the WWTP expressed as a percent of the dynamic WLA value.

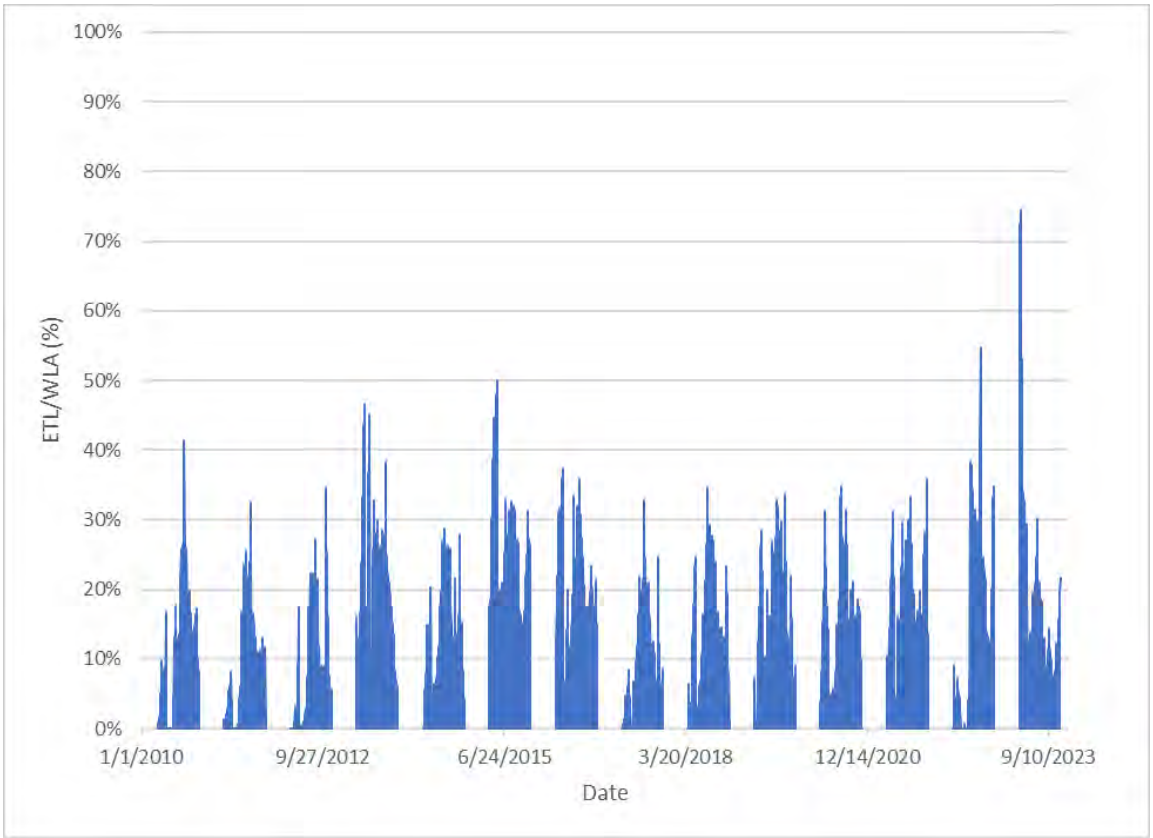


Figure 2-2: Lebanon WWTP Daily ETL as Percent of Dynamic WLA Values

As shown on Figure 2-2, ETL values were less than 60 percent of the allowable WLA value except for a short period in 2023 during which time they increased to slightly greater than 70 percent. Again, as previously discussed, temperature data during this period is not believed to reflect actual effluent temperatures and should not be included without additional data

collection. Kennedy Jenks' analysis suggests that even with moderate increases in flow to the WWTP, the City should be able to meet requirements of the draft 2024 Temperature TMDL without the need for additional facilities. This conclusion is predicated on the assumption the DEQ allows the use of dynamic WLA calculation as included in the draft 2024 Temperature TMDL documents.

2.2.1.3 Alkalinity

Alkalinity is a measure of the buffering capacity or ability of water to resist changes in pH. "EPA's recommendation for the aquatic life freshwater criterion for alkalinity is '20 mg/L or more as CaCO3 except where natural concentrations are less" (Oregon DEQ, May 2022). The South Santiam River has been listed as being impaired for alkalinity. The designated beneficial use impacted is fish and aquatic life support.

The DEQ has not yet developed guidance, a regulatory standard, or timing for a standard for alkalinity. Thus, determining if alkalinity should be addressed in near-term facilities improvements is challenging. Alkalinity in freshwater systems can be affected by multiple processes including "photosynthetic activity and other inorganic and organic chemical reactions" (Oregon DEQ, May 2022). Because development of a TMDL for alkalinity will likely consider other pollutants such as pH, chlorophyll a, aquatic weeds or algae growth, and dissolved oxygen, it is difficult to predict what impact if any the 303(d) listing for alkalinity may have on the City's next NPDES permit.

2.2.1.4 Methylmercury

The South Santiam River does not meet the human health criteria for methylmercury and thus has been listed as being impaired for failure to attain the designated beneficial uses for fishing and water supply. The original 2006 Willamette Basin Mercury TMDL was revised by the DEQ in 2011 to incorporate the revised methylmercury criterion in fish tissue. EPA's approval of the revised mercury TMDL was challenged in 2012. In 2017, the federal courts ordered that the 2006 TMDL remain in place. The DEQ issued a revised mercury TMDL on November 22, 2019 which was subsequently disapproved by the EPA on November 29, 2019. The EPA issued the final TMDL for mercury in the Willamette Basin, Oregon on February 4, 2021.

The TMDL recognizes that the primary source of mercury within the Willamette River Basin is from atmospheric deposition onto land or water mainly from sources outside of Oregon. Once deposited, the primary pathway to surface waters is through erosion of mercury containing sediment. Non-point sources are thus a more significant source of mercury than point sources such as wastewater treatment facilities.

While the DEQ considered individual wasteload allocations for point sources, they "determined that aggregated wasteload allocations were most appropriate" as it was in keeping with the EPA's guidance for "TMDL development when air deposition is the dominant source of mercury" (Oregon DEQ, 2019). Adding to this decision, the DEQ notes "that combined wastewater points source loads contribute only one percent of the total basin load of mercury" (Oregon DEQ, 2019).

The resulting aggregated wasteload allocation applicable to the South Santiam River for major and minor domestic WWTPs is a 10-percent reduction of grams of total mercury per day. The DEQ determined that to achieve the stated reduction in total mercury, permit renewals for major WWTPs "will include enforceable conditions for monitoring and reporting of total mercury and

development and implementation of mercury minimization programs" (Oregon DEQ, 2019). The City's next NPDES permit is thus anticipated to include the requirement to develop and implement a Mercury Minimization Plan (MMP).

Required elements of the MMP are expected to include the following:

- Identification and evaluation of current and potential mercury sources, including industrial, commercial, and residential
- Summary of mercury reduction activities implemented during the past 5 years
- Mercury results for samples collected within the previous 5 years
- Annual average effluent mercury concentrations and mass loads
- Annual average biosolids concentrations and mass loads
- Changes that may affect mercury, such as those to operations, treatment, and chemicals used at the WWTP
- An implementation plan that includes specific methods for reducing mercury

After implementation of the MMP, if the DEQ determines mercury levels have not decreased, additional changes may be required and the DEQ may reopen the NPDES permit and modify permit conditions.

2.2.2 Other Pollutants and Water Quality Parameters

When renewing NPDES permits, the DEQ establishes "effluent limits for any pollutant discharged under the permit that causes, has the reasonable potential to cause, or contributes to an exceedance of a water quality standard in the receiving water" (Oregon DEQ, 2023). A reasonable potential analysis (RPA) is an evaluation conducted to determine if the discharge of a pollutant has a reasonable potential to exceed water quality standards.

Kennedy Jenks performed RPAs for the City's discharge utilizing spreadsheets developed by and available from the DEQ for the following pollutants and water quality parameters:

- Toxics including metals, cyanide, and organic compounds
- Ammonia
- Chlorine
- Copper
- pH
- Thermal plume

Effluent data used for the analyses were obtained from monthly discharge monitoring reports (DMRs) for the WWTP and results from supplemental testing as required by the DEQ in a letter dated August 30, 2021. Supplemental testing included data for the Copper Biotic Ligand Model (BLM) and Aluminum and Toxic Pollutants.

Data for ambient conditions within the South Santiam River were obtained from three sources. These consisted of the supplemental testing data required by the DEQ for the Copper BLM and two river water quality monitoring stations. Monitoring station data were obtained from the Ambient Water Quality Monitoring System (AWQMS) that is used by the DEQ. The AWQMS provides access to water quality data from multiple agencies throughout Oregon. The monitoring stations used included one upstream U.S. Geological Survey (USGS) station near Waterloo, Oregon (USGS 14187500). Although it is preferable to use upstream monitoring stations when determining background water quality conditions, sufficient upstream data was not available. As a result, the downstream DEQ monitoring station (10366-ORDEQ) at Highway 220 near Crabtree, Oregon was used.

When determining the potential impact that WWTP effluent may have on receiving streams, the DEQ allows consideration of a regulatory mixing zone (RMZ) which is a limited area around the point of discharge beyond which pollutant concentrations must meet water quality standards. The RMZ defined within the City's current NPDES permit is the portion of the South Santiam River beginning 10 feet upstream and continuing 100 feet downstream from the point of discharge. The Zone of Immediate Dilution (ZID) is a smaller area within the RMZ beyond the limits of which acutely toxic (lethal) conditions must be prevented. The ZID is defined in the City's NPDES permit as the portion of the RMZ that is within 10 feet of the point of discharge.

The City submitted a mixing zone study (December 17, 2007) to the DEQ with dilution values proposed for use in establishing future effluent permit limits. Dilution is the reduction in pollutant concentration that can be achieved within the RMZ. For example, a dilution value of 20 in the RMZ would decrease the pollutant concentration to 5% of that in the undiluted WWTP effluent.

The DEQ reviewed the mixing zone study and issued a memorandum (February 4, 2008) that set dilution values to be used by permit writers in conducting RPAs and establishing effluent permit limits (Oregon DEQ, 2008). These values are presented in Table 2-8 and were used by Kennedy Jenks in performing RPAs for this Plan. A copy of the mixing zone study is provided in Appendix F.

Table 2-8: Dilution Values Applicable for Reasonable Potential Analyses

Design Flow[(a)]	Criteria	Location[(b)]	Dilution
ADWF[(c)]	Acute	ZID[(d)]	18
ADWF	Chronic	RMZ[(e)]	27
ADWF	Human Health – Carcinogen	RMZ	31
AAF[(f)]	Human Health – Non-Carcinogen	RMZ	38

Notes:

(a) Permitted design WWTP flows.
(b) Location within the mixing zone at which the noted dilution value applies.
(c) ADWF = Average dry weather flow equal to 3 MGD.
(d) ZID = Zone of Immediate Dilution. The ZID extends 10 feet from the point of discharge. The dilution value shown is that for centerline of the ZID. The bulk average dilution for the ZID is 23.
(e) RMZ = Regulatory Mixing Zone. The mixing zone extends 100 feet downstream from the point of discharge. The dilution value shown is that for the average bulk dilution at the edge of the mixing zone.
(f) AAF = Average annual flow equal to 5.5 MGD.

2.2.2.1 Toxics

The DEQ required the City to collect and submit supplemental effluent testing for toxic pollutants in anticipation of the WWTP's NPDES permit renewal. Ten monthly sampling events were

required which the City conducted from January 2022 through October 2022. Testing was performed for 109 pollutant and water quality parameters including metals, cyanide, hardness, volatile organic compounds, acid-extractable organic compounds, and base-neutral organic compounds.

A list of the pollutants which had at least one result above its respective minimum detection limit and whether water quality criteria have been established is presented in Table 2-9. Kennedy Jenks performed an RPA for each pollutant listed with established water quality criteria using the DEQ toxics RPA spreadsheet.

Table 2-9: Toxics with at Least One Result above Detection Limit and Associated Water Quality Criteria Classification

Pollutant	Water Quality Criteria
Antimony (total)	Human Health
Arsenic (total)	No Criteria Established
Arsenic, Inorganic (total)	Human Health
Arsenic, Inorganic (dissolved)	Aquatic Life
Cadmium (total)	Aquatic Life
Cadmium (dissolved)	Aquatic Life
Bis(2-ethylhexyl) phthalate	Human Health
Bromodichloromethane[(a)]	Human Health
Chloroform	Human Health
Chromium (total)	No Criteria Established
Chromium III (total)	No Criteria Established
Cyanide (total)	Human Health
Cyanide (free)	Aquatic Life
Dibromochloromethane[(b)]	Human Health
Lead (total)	No
Lead (dissolved)	Aquatic Life
Mercury (total)	Aquatic Life
Nickel (total)	Human Health
Nickel (dissolved)	Aquatic Life
Selenium (total)	Human Health
Selenium (dissolved)	Aquatic Life
Toluene	Human Health
Trichloroethene[(c)]	Human Health
Zinc (total)	Human Health
Zinc (dissolved)	Aquatic Life

Notes:

(a) Bromodichloromethane can also be identified as Dichlorobromomethane.
(b) Dibromochloromethane can also be identified as Chlorodibromomethane.
(c) Trichloroethene can also be identified as trichloroethylene.

For each pollutant listed in Table 2-9 with an established water quality criterion, Kennedy Jenks performed an RPA using the DEQ toxics RPA spreadsheet. Results of the RPA showed that of the 21 pollutants analyzed, 15 were at or below concentrations that met water quality standards at end of pipe even without accounting for dilution within the ZID and RMZ.

The six pollutants that did not meet water quality criteria at end of pipe are shown in Table 2-10. As indicated in the table, ambient water quality data for three of the pollutants were available from the downstream water quality monitoring station at Highway 226 near Crabtree, Oregon (10366-ORDEQ). Using this data, Kennedy Jenks completed the RPA which showed that with the dilution available within the mixing zone, the pollutants did not have a reasonable potential to exceed water quality standards at either the ZID or RMZ. Because of the lack of ambient water quality data for the other three pollutants, RPAs for those constituents could not be completed. While Kennedy Jenks does not anticipate the DEQ requiring effluent limits for these pollutants, they may require the City to collect ambient data for them during the next permit renewal cycle to ensure there is no reasonable potential to exceed water quality limits.

Table 2-10: Toxics with at Least One Result above Detection Limit and Associated Water Quality Criteria Classification

Pollutant	Ambient Data Available?	Reasonable Potential to Exceed?	
		ZID (Acute)	RMZ (Chronic)
Cadmium (dissolved)	Yes[(a)]	No	No
Bromodichloromethane	No	[(b)]	[(b)]
Cyanide (free)	No	[(b)]	[(b)]
Dibromochloromethane	No	[(b)]	[(b)]
Mercury (total)	Yes[(a)]	No	No
Zinc (dissolved)	Yes[(a)]	No	No

Notes:

(a) Ambient river water quality data obtained from the DEQ water quality monitoring station at Highway 226 near Crabtree, Oregon (10366-ORDEQ). Eight data points were available for ambient concentrations of cadmium and zinc. Three data points were available for mercury.

(b) Due to the lack of ambient river water quality data for the constituent listed, the RPA could not be completed.

2.2.2.2 Ammonia

The City's current NPDES permit includes year-round limits for effluent ammonia. These include a maximum monthly average concentration of 0.94 mg/L as nitrogen (N) and a daily maximum concentration of 1.89 mg/L as N.

Kennedy Jenks performed an RPA for ammonia using the DEQ ammonia RPA spreadsheet. Effluent data for ammonia, pH, and temperature were obtained from monthly DMRs for the WWTP for years 2017 through 2023. Effluent alkalinity values were taken from data collected in 2022 and 2023 for the Copper BLM. Daily and 30-day running average values for effluent ammonia during the years 2017 through 2023 are presented on Figures 2-3 and 2-4. The figures show that prior to 2020 there were multiple instances in which the WWTP exceeded either the maximum daily or the maximum monthly limit for ammonia.

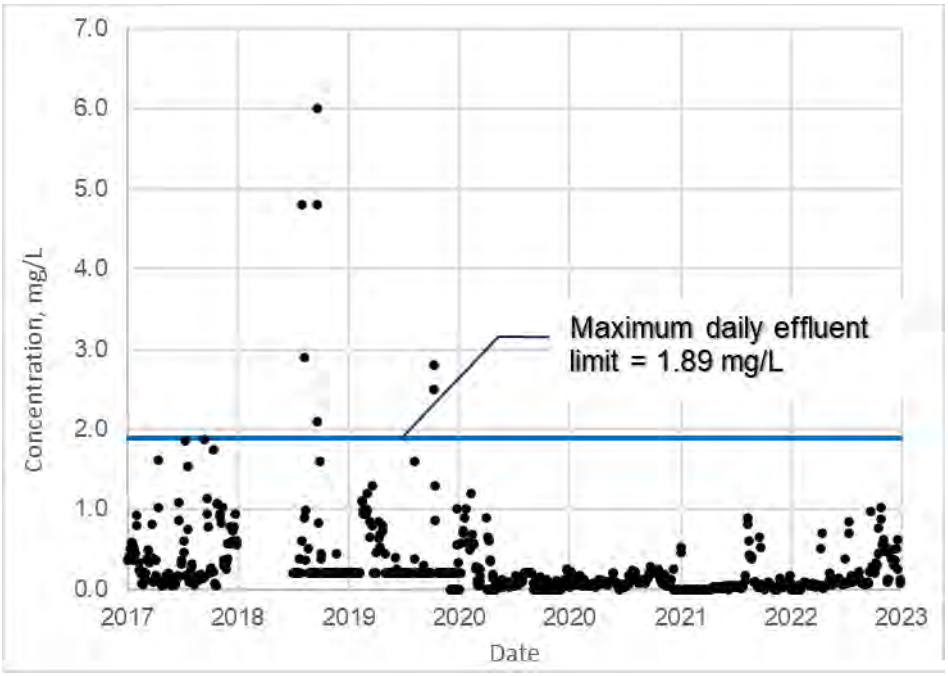


Figure 2-3: Daily Effluent Ammonia Concentrations

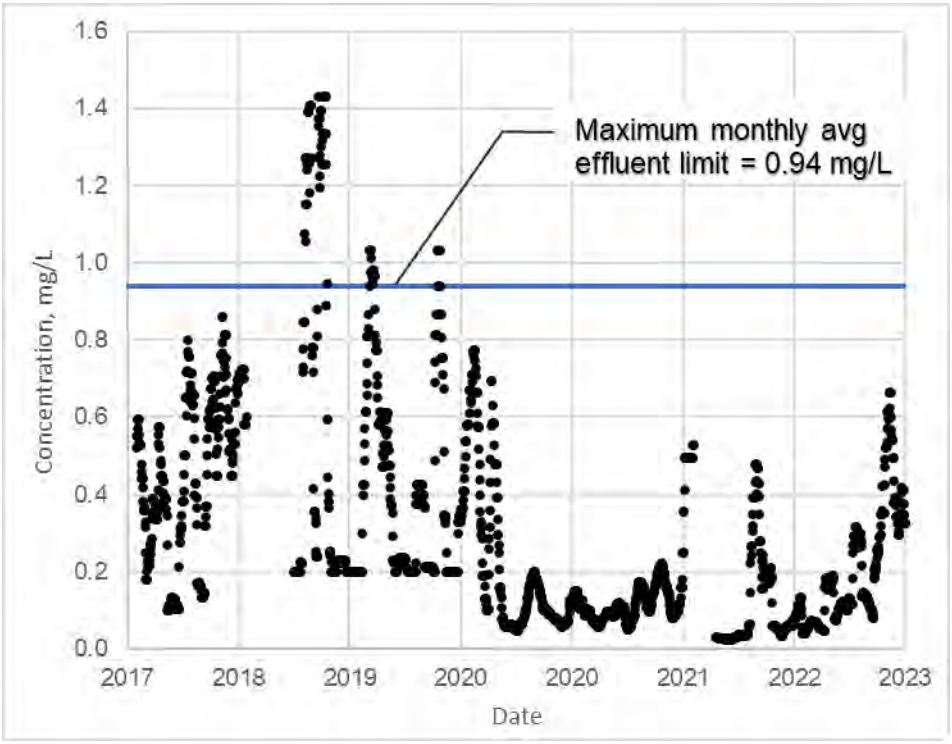


Figure 2-4: 30-Day Average Effluent Ammonia Concentration

Because only four ambient data points for ammonia and pH were available from upstream water quality monitoring stations, ammonia data from the downstream DEQ monitoring station at Highway 226 near Crabtree, Oregon (10366-ORDEQ) were used. Data collected for the Copper BLM were used for ambient pH and alkalinity. Temperature data was obtained from the upstream water quality monitoring station near Waterloo, Oregon (USGS 14187500).

Results of the RPA analysis, which are summarized in Table 2-11**,** showed no reasonable potential to exceed water quality criteria at either the ZID or RMZ. Values at the RMZ were calculated for the 4-day average condition assuming river flow equal to the 7Q10 value and the 30-day average condition assuming a river flow equal to the 30Q5 value (minimum 30-day average with a return frequency of 5 years).

Table 2-11: Ammonia RPA Results Summary

		Water Quality Criteria			Calculated Concentration		
			Chronic			RMZ	
RPA Period	Effluent Concentration (mg/L)	Acute (mg/L)	4-day avg[(a)] (mg/L)	30-day avg[(b)] (mg/L)	ZID (mg/L)	4-day avg[(a)] (mg/L)	30-day avg[(b)] (mg/L)
Summer	6.0	11.9	4.3	1.7	0.34	0.23	0.20
Winter	1.6	15.7	7.2	2.9	0.09	0.06	0.05
Year Round	6.0	14.7	4.9	2.0	0.34	0.23	0.20

Notes:

(a) Based on river flow equal to the 7Q10 value (minimum 7-day average flow with a return frequency of 10 years).
(b) Based on river flow equal to the 30Q5 value (minimum 30-day average flow with a return frequency of 5 years).

Even though, as shown on Figures 2-3 and 2-4, there were multiple instances prior to 2020 when the WWTP failed to meet effluent permit limits for ammonia, the RPA showed that there would have been no reasonable potential to exceed water quality standards in the South Santiam River. To confirm that the City's existing NPDES permit limit for ammonia is protective of water quality, Kennedy Jenks conducted an RPA assuming year-round effluent concentration equal to the daily maximum ammonia concentration limit of 1.89 mg/L. As with the RPA utilizing actual plant effluent data, this analysis also showed no reasonable potential to exceed water quality criteria.

Based on the results of both ammonia RPAs, Kennedy Jenks does not anticipate changes to the City's current NPDES permit limits for effluent ammonia during the next permit renewal cycle which is tentatively scheduled for 2025.

2.2.2.3 Copper

An aquatic life water quality standard for copper was approved for Oregon by the EPA in January 2017. The approved standard requires copper criteria to be based on the Copper BLM that accounts for the interaction between multiple water quality characteristics to predict metal toxicity to aquatic species. The interaction between the various water quality characteristics results in variable water quality criteria for copper that depend on ambient conditions.

In a letter dated August 30, 2021, the DEQ required the City to monitor WWTP effluent and the South Santiam River (upstream of the outfall) for 13 water quality parameters associated with the Copper BLM. Parameters consisted of copper, aluminum, hardness, organic carbon, pH,

temperature, calcium, magnesium, sodium, potassium, sulfate, chloride, and alkalinity. A total of 24 monthly sampling events were required. The City began sampling in January 2022 with final sampling occurring in December 2023.

Kennedy Jenks used the DEQ RPA and Copper BLM software in accordance with DEQ guidance (Oregon DEQ, 2021) to determine if the data collected showed a reasonable potential to exceed the water quality criteria for copper. The analysis showed no potential to exceed water quality criteria for the first 21 months during which data was collected (January 2022 through September 2023). However, the analysis indicated a reasonable potential to exceed water quality criteria during the months of October, November, and December 2023. Results of this analysis for the months in which a reasonable potential to exceed are summarized in Table 2-12. Values which indicate a reasonable potential to exceed water quality standards are shown in bold font.

Table 2-12: Copper RPA Results Showing Values with Potential to Exceed[a]

Sample Date	Effluent Conc. (ug/L)	River Conc. (ug/L)	ZID Conc. (ug/L)	ZID Criteria[b] (ug/L)	RMZ Conc. (ug/L)	RMZ Criteria[b] (ug/L)	100% Mix Conc. (ug/L)	100% Mix Criteria[b] (ug/L)
10/17/2023	7.93	2.65	2.94	3.61	**2.85**	2.13	**2.70**	2.02
11/14/2023	3.92	8.42	8.17[c]	4.42	**8.25**	2.63	**8.37**	2.63
12/12/2023	4.85	7.35	**7.21**	2.75	**7.26**	1.65	**7.32**	1.65

Notes:

(a) Values in bold font indicate reasonable potential to exceed water quality criteria.
(b) Water quality criteria as calculated by the Copper BLM software for the ambient river conditions at the time samples were collected.
(c) Although the concentration at the edge of the ZID exceeded the acute water quality criteria of 4.42 ug/L as calculated by the Copper BLM software, the effluent concentration for copper was 3.92 ug/L.

The 3 months during which the analysis predicted a reasonable potential to exceed water quality criteria, occurred during a period of dramatically higher ambient copper concentrations. As shown on Figure 2-5, copper concentrations in the river were equal to or less than about 0.3 ug/L until October 2023 when concentrations rapidly increased, ultimately to a maximum of more than 8.0 ug/L. However, as shown on Figure 2-6, values for effluent copper remained within the same range as prior months. This suggests ambient concentrations for copper in the river continued to be at or below those previously seen and it is likely there would have been no potential to exceed during the months of October, November, and December 2023.

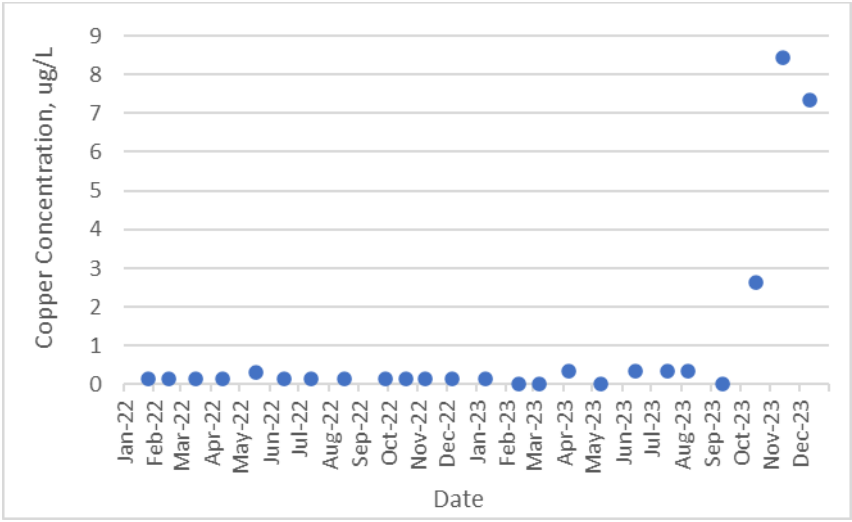


Figure 2-5: South Santiam River Copper Concentration

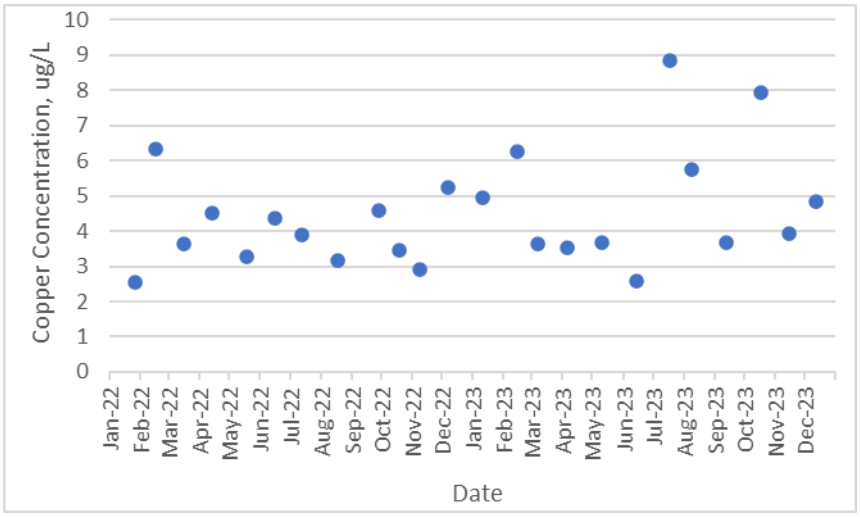


Figure 2-6: WWTP Effluent Copper Concentration

The increases in ambient copper concentrations occurred at the same time as changes were made to the operation of the Green Peter reservoir upstream of the WWTP outfall. Starting in the summer of 2023, the U.S. Army Corps of Engineers increased release of water from the dam to support salmon populations. The change in dam operation coincided with a significant change in water quality within the South Santiam River. This is illustrated by the dramatic increase in turbidity shown on Figure 2-7 starting in October of 2023 and continuing through December 2023. The increase in ambient turbidity also coincided with the increased levels of copper in the river.

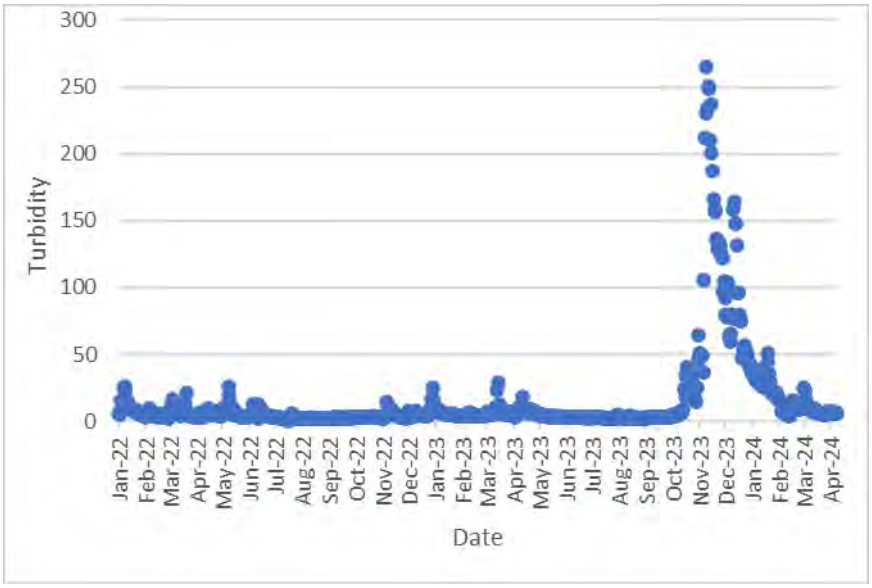


Figure 2-7: South Santiam River Turbidity

Ambient copper conditions exceeded instream water quality standards as calculated by the Copper BLM which suggest that a TMDL must be developed by the DEQ. Because the change in ambient copper concentration seems to have been closely linked to the recent change in operation of the Green Peter reservoir, it is unlikely the DEQ will impose copper limits during the next NPDES permit renewal cycle. However, additional data collection and monitoring for copper should be expected.

2.2.2.4 Chlorine

The City's current NPDES permit includes year-round limits for effluent total chlorine residual. These include a maximum monthly average concentration of 0.01 mg/L and a daily maximum concentration of 0.02 mg/L.

Kennedy Jenks performed an RPA for chlorine using the DEQ chlorine RPA spreadsheet. DEQ approved mixing zone dilution values and the City's current maximum daily chlorine limit were used as input values for the analysis. Results are summarized in Table 2-13 and show that the concentrations within the mixing zone are less than the applicable water quality standards. Thus, current permit limits would not create a reasonable potential to exceed acute water quality criteria. Based on these results, Kennedy Jenks does not anticipate changes to the City's current permit limits for effluent chlorine during the next permit renewal cycle.

Table 2-13: Chlorine RPA Results

Max. Effluent Concentration (mg/L)	Background Concentration (mg/L)	Mixing Zone Concentration[a] ZID (mg/L)	Mixing Zone Concentration[a] RMZ (mg/L)	Water Quality Criteria Acute[b] (mg/L)	Water Quality Criteria Chronic[c] (mg/L)
0.02[d]	0.00[e]	0.001	0.001	0.019	0.011

Notes:

(a) Calculated values accounting for dilution within the mixing zone. Values shown are those at the edge of the ZID and RMZ.

(b) Acute water quality criteria must be met at the edge of the ZID.
(c) Chronic water quality criteria must be met at the edge of the RMZ.
(d) Current maximum daily effluent permit limit for chlorine.
(e) DEQ assumed default value.

2.2.2.5 pH

Basin specific water quality criteria for the Willamette River basin includes criteria for pH that requires it to be within a range of 6.5 to 8.5. This differs from the City's current NPDES permit that includes a year-round pH requirement of 6.0 to 9.0.

Kennedy Jenks performed an RPA for pH using the DEQ pH RPA spreadsheet. DEQ approved mixing zone dilution values and the City's current lower and upper permit limits for pH were input to the spreadsheet. Data from the copper RPA were used for ambient pH, ambient temperature, ambient alkalinity, effluent temperature, and effluent alkalinity.

Results of the RPA are summarized in Table 2-14. Minimum and maximum pH values predicted at the mixing zone boundary were 6.6 and 7.0, respectively. Because these are within the basin specific criteria for pH of 6.5 to 8.5, there is no reasonable potential to exceed water quality standards for pH. Based on these results, Kennedy Jenks does not anticipate changes to the City's current NPDES permit limits for effluent pH during the next permit renewal cycle.

Table 2-14: pH RPA Results

Item	Lower pH Criteria	Upper pH Criteria
Ambient Characteristics[a]		
Temperature (°C)	6.0	15.4
pH	6.7	7.0
Alkalinity (mg/L)[b]	12.5	18.4
Effluent Characteristics		
Temperature (°C)[a]	12.5	20.4
pH[c]	6.0	9.0
Alkalinity (mg/L)[a][b]	29.5	44.0
pH at Mixing Zone Boundary	6.6	7.0

Notes:

(a) Based on 24 data sets collected for the copper RPA.
(b) Measured as $CaCO_3$.
(c) Values equal to current NPDES permit limits.

2.2.2.6 Temperature

Although new thermal WLAs associated with the temperature TMDL for the Willamette River mainstem and major tributaries are anticipated, OAR 340-041-0053(2)(d) also requires that effluent limits be established to prevent or minimize adverse effects to salmonids inside the mixing zone. For the South Santiam River, applicable adverse effects and the means to prevent them are described as follows:

- *Impairment of an active salmonid spawning area.* This is prevented by limiting the potential fish exposure to temperatures of 13°C or more for salmon and steelhead.

- *Acute impairment or instantaneous lethality.* This is prevented by limiting potential fish exposure to temperatures of 32.0°C.

- *Thermal shock caused by a sudden increase in water temperature.* This is prevented by limiting potential fish exposure to temperatures of 25.0°C or more to less than 5 percent of the cross section of 100 percent of the 7Q10 flow.

- *Migration blockage.* Unless the ambient temperature is 21.0°C or greater, this adverse effect is prevented by limiting potential fish exposure to temperatures of 21.0°C or more to less than 25 percent of the cross section of the 100 percent 7Q10 flow.

Kennedy Jenks performed thermal plume RPAs for each of the potential adverse effects identified in OAR 340-041-0053(2)(d) using the DEQ temperature RPA spreadsheet. Information input to the model included DEQ approved mixing zone dilution values, ambient river temperatures, and WWTP effluent temperature values. These values and the results of the RPA are summarized in Tables 2-15 through 2-18.

Although it is not known if there is an active spawning area within or near the RMZ, an RPA for active spawning area impairment was still performed. This was done as a conservative planning measure to determine the potential for adverse effects should one be present. Results of the active spawning area impairment are presented in Table 2-15. Although the temperature calculated at the regulatory mixing zone exceeds the 13°C temperature criterion, it is within the 0.3°C human use allowance which results in there being no reasonable potential.

Table 2-15: Thermal Plume RPA: Active Spawning Area Impairment

Item	Value
RPA Input	
Dilution at Spawning Area	27
Ambient Temperature (°C)	13[(a)]
Max. 7-day Effluent Temperature (°C)	20.8[(b)]
Applicable Temperature Criterion (°C)	13
RPA Results	
Temperature at Spawning Area	13.29
Reasonable Potential?	No[(c)]

Notes:

(a) Ambient temperature data obtained from the USGS gauging station at Waterloo, Oregon. Data shows temperature can exceed 13° in May and October. The value of 13°C was thus used per DEQ instructions.

(b) Value recorded on October 24, 2022. This is the highest single day effluent temperature on record. Maximum day value used instead of 7-day average of daily maximums as the effluent temperature values are daily grab values. Note that effluent temperature values starting in the fall of 2022 and continuing into the spring of 2023 are believed to be artificially high and to have been caused by a change in monitoring location that did not accurately reflect effluent temperatures. Prior to October 24, 2022, the highest recorded effluent temperature during the spawning period was 19.3°C and occurred on October 29, 2020.

(c) Temperature at the RMZ is less than the temperature criterion (13°C) plus the human use allowance of 0.3°C.

Results of the acute impairment or instantaneous lethality RPA are presented in Table 2-16. As shown, the RPA showed no reasonable potential to exceed the applicable temperature criterion of 32°C for acute impairment or instantaneous lethality.

Table 2-16: Thermal Plume RPA: Acute Impairment or Instantaneous Lethality

Item	Value
RPA Input	
Dilution at Spawning Aea	27
Ambient Temperature (°C)	16.7[(a)]
Max. Daily Effluent Temperature (°C)	20.8[(b)]
Applicable Temperature Criterion (°C)	32
RPA Results	
Temperature at Spawning Area	13.85
Reasonable Potential?	No

Notes:

(a) Ambient temperature data from the USGS gauging station at Waterloo, Oregon. Maximum recorded value during the spawning period October 15 through May 15 for the period on record (October 3, 2019, through present). Value recorded on October 22, 2023.

(b) Value recorded on October 24, 2022. This is the highest single day effluent temperature on record during the spawning period October 15 through May 15. Note that effluent temperature values starting in the fall of 2022 and continuing into the spring of 2023 are believed to be artificially high and to have been caused by a change in monitoring location that did not accurately reflect effluent temperatures. Prior to October 24, 2022, the highest recorded effluent temperature during the spawning period was 19.3°C and occurred on October 29, 2020.

Results of the thermal shock RPA are presented in Table 2-17. The RPA showed that when both the effluent and ambient water temperatures are at their maximum measured values, there is no reasonable potential for the plume's temperature at 5 percent of the receiving stream's cross-sectional area to be above 25°C.

Table 2-17: Thermal Plume RPA: Thermal Shock

Item	Value
RPA Input	
7Q10 (cfs)	450[(a)]
Ambient Temperature (°C)	18.3[(b)]
Effluent Flow (MGD)	3[(a)]
Max. Daily Effluent Temperature (°C)	26.5[(c)]
Applicable Temperature Criterion (°C)	25
RPA Results	
Temperature at 5 percent cross section (°C)	19.7
Reasonable Potential?	No

Notes:

(a) Value from the February 4, 2008 mixing zone memorandum issued by the DEQ.

(b) Ambient temperature data from the USGS gauging station at Waterloo, Oregon. Value recorded on July 29, 2022. This is the highest single day ambient temperature value for the period on record (October 3, 2019 through present).

(c) Value recorded on August 31, 2022. This is the highest single day effluent temperature on record.

Results of the migration blockage RPA are presented in Table 2-18. The RPA showed that when the effluent and ambient water temperatures are at their maximum measured values, there is no reasonable potential for the plume's temperature at 25 percent of the receiving stream's cross-sectional area to be above 21°C.

Table 2-18: Migration Blockage RPA: Thermal Shock

Item	Value
RPA Input	
7Q10 (cfs)	450[(a)]
Ambient Temperature (°C)	18.3[(b)]
Effluent Flow (MGD)	3[(a)]
Max. Daily Effluent Temperature (°C)	26.5[(c)]
Applicable Temperature Criterion (°C)	21
RPA Results	
Temperature at 25 percent cross section (°C)	18.6
Reasonable Potential?	No

Notes:

(a) Value from the February 4, 2008 mixing zone memorandum issued by the DEQ.
(b) Ambient temperature data from the USGS gauging station at Waterloo, Oregon. Value recorded on July 29, 2022. This is the highest single day ambient temperature value for the period on record (October 3, 2019 through present).
(c) Value recorded on August 31, 2022. This is the highest single day effluent temperature on record.

2.3 Anticipated Effluent Permit Requirements

The City's existing NPDES permit was previously discussed with a summary of current permit requirements presented in Table 2-1**.** The following provides a summary of anticipated outfall permit limits, changes in monitoring requirements, and mercury minimization plan.

2.3.1 Anticipated Effluent Permit Limits

Permit limits for $CBOD_5$, TSS, *E. coli* bacteria, pH, ammonia, BOD/TSS removal efficiency, and total chlorine residual are expected to remain unchanged during the next permit renewal cycle. Effluent permit additions are anticipated to include an excess thermal load limit. Anticipated permit limits are presented in Table 2-19**.**

Table 2-19: Anticipated Outfall Permit Limits

Parameter	Units	Monthly Average	Weekly Average	Daily Maximum
$CBOD_5$ (May 1 – Oct 31)	mg/l	10	15	-
	lbs/day	250	380	500
	% removal[(a)]	85	-	-
TSS (May 1 – Oct 31)	mg/l	10	15	-
	lbs/day	250	380	500
	% removal[(a)]	85	-	-

Parameter	Units	Monthly Average	Weekly Average	Daily Maximum
$CBOD_5$ (Nov 1 – Apr 30)	mg/l	25	40	-
	lbs/day	1,500	2,200	3,000
	% removal[(a)]	85	-	-
TSS (Nov 1 – Apr 30)	mg/l	30	45	-
	lbs/day	1,800	2,700	3,600
	% removal[(a)]	85	-	-
E. coli Bacteria[(b)]	#/100 mL	Shall not exceed a monthly geometric mean of 126, no single sample shall exceed 406.		
pH	SU	Instantaneous limit between a daily minimum of 6.0 and a daily maximum of 9.0.		
Ammonia – N	mg/L	Shall not exceed a monthly average of concentration of 0.94 and a daily maximum concentration of 1.89.		
Total Chlorine Residual	mg/L	Shall not exceed a monthly average concentration of 0.01 and a daily maximum concentration of 0.02.		
Excess Thermal Load[(c)] (Apr 1 – May 15)	million kcal/day	(0.03°C)*(4.1cfs+Q_R)*2.447		
Excess Thermal Load[(c)] (May 16 – Oct 14)	million kcal/day	(0.05°C)*(4.9cfs+Q_R)* 2.447		
Excess Thermal Load[(c)] (Oct 15 – Nov 15)	million kcal/day	(0.08°C)*(12.3cfs+Q_R)* 2.447		

Notes:

(a) CBOD5 and TSS removal efficiency shall not be less than 85% monthly average except when monthly average flows exceed 3.0 MGD which shall not be less than 75% monthly average.

(b) If a single sample exceeds 406 organisms per 100 mL, then five consecutive re-samples may be taken at 4-hour intervals beginning within 28 hours after the original sample was taken. If the log mean of the five re-samples is less than or equal to 126 organisms per 100 mL, a violation is not triggered.

(c) Dynamic wasteload allocation (kilocalories/day) expressed as a rolling 7-day average calculated on a daily basis where Q_R is the daily mean river flow (cfs). Equations are based on values included in the *Draft Total Maximum Daily Loads for the Willamette Subbasins (Willamette Mainstem and Major Tributaries) Temperature*, dated August 2024.

The equations shown in Table 2-19 for excess thermal load limits assumes use of dynamic WLAs that are calculated on a daily basis. Although daily calculations will require additional data collection and monitoring, the use of dynamic limits will result in higher thermal WLAs. As previously shown, this should eliminate the need to store, cool, or otherwise divert WWTP effluent to meet the requirements of the 2024 Temperature TMDL.

The excess thermal load equations are based on the following:

$$WLA = (\Delta T) * (Q_E + Q_R) * C_F$$

where,

WLA = Wasteload allocation (million kcal/day), expressed as a rolling 7-day average.
ΔT = The assigned portion of the human use allowance (HUA). It is the maximum temperature increase (°C) above the applicable river temperature criterion not to be exceeded by an individual source from all outfalls combined. This is a set value based on the period of the year.

Q_E = The daily mean WWTP effluent flow (cfs). This is a set value based on the period of the year.
Q_R = The daily mean river flow rate, upstream (cfs). When the river flow is less than the 7Q10 value, Q_R is equal to the 7Q10 value. When the river flow is greater than the 7Q10 value, Q_R is equal to the daily mean river flow.
C_F = Conversion factor equal to 2.447 when flow is in cfs.

Values for ΔT and Q_E were previously presented in Table 2-6 as are contained in the Draft 2024 Temperature TMDL. They are specific to each point source and vary based on the discharge period.

2.3.2 Monitoring Requirements

Anticipated monitoring requirements are summarized in Table 2-20. Monitoring frequencies for $CBOD_5$, TSS, pH, *E. coli*, temperature, and ammonia are expected to increase from current requirements. Alkalinity, which is not currently monitored, is anticipated to be required monthly.

Table 2-20: Anticipated Monitoring Requirements

Parameter	Units	Frequency[a]
Total Flow[b]	MGD	Daily, metered
$CBOD_5$ and TSS	mg/L	3 per week, 24-hr composite
$CBOD_5$ and TSS	lbs/day	3 per week, calculation
$CBOD_5$ and TSS Removal	%	Monthly calculation
pH[c]	SU	Daily, grab or continuous
E. coli	#/100mL	3 per week, grab
Temperature, effluent	°C	Daily, continuous
Excess thermal load	million kcal/day	Daily, calculation[d]
Ammonia, N	mg/L	2 per week, 24-hr composite
Alkalinity	mg/L	1 per month, grab
Chlorine, total residual[e]	mg/L	Daily, grab or continuous
Chlorine, used	lbs/day	Daily, measurement
UV[f], dose	mJ/cm^2	Daily average, calculation
WET[g] Testing	-	4 times per permit cycle
Toxics	µg/L[h]	Quarterly for 3 years of permit cycle

Notes:

(a) Year round monitoring unless otherwise specified.
(b) Influent and/or effluent.
(c) Influent and effluent.
(d) Monitor Apr 1 – Nov 15. Value is the 7-day rolling average calculated from daily ETLs. The daily ETL is calculated as follows: $ETL = 3.785*Q_e*\Delta T$
Where:
ETL = Excess thermal load limit, million kcal/day
Q_e = Daily average effluent flow (MGD)
ΔT = Daily maximum effluent temperature (°C) minus ambient criterion. Ambient criterion for salmon and trout rearing and migration is 18°C and is applicable May 16 – Oct 14. Ambient criterion for salmon and steel head spawning is 13°C is applicable April 1 – May 15 and Oct 15 – Nov 15.
(e) Monitor if used for disinfection. Monitored pre and post de-chlorination.
(f) Monitor if used for disinfection.
(g) WET = Whole effluent toxicity.
(h) Unless otherwise specified.

In addition to the parameters shown in Table 2-20, the DEQ may require collection of ambient data for three of the six toxics shown in Table 2-10 that did not meet water quality criteria at the end of the pipe. RPAs could not be completed for these compounds due to an absence of ambient water quality data.

As previously described, ambient copper conditions exceeded instream water quality standards as calculated by the Copper BLM in October, November, and December 2023. These were the last 3 months during which the City collected Copper BLM data for ambient stream conditions. Because the data showed that ambient conditions did not meet water quality standards, the DEQ may need to develop a TMDL for copper. Because the change in ambient copper concentration seems to have been closely linked to the recent change in operation of the Green Peter reservoir, it is unlikely that the DEQ will impose copper limits during the next NPDES permit renewal cycle. However, the City should anticipate additional data collection and monitoring requirements for copper.

2.3.3 Mercury Minimization Plan

The South Santiam River does not meet the human health criteria for methylmercury. A mercury TMDL was issued by the EPA on February 4, 2021. The TMDL requires major and minor domestic WWTPs to achieve a 10-percent reduction of grams of total mercury per day. The DEQ determined that to achieve the stated reduction in total mercury, permit renewals for major WWTPs would require development and implementation of mercury minimization programs. The City should anticipate the requirement to develop and implement an MMP during the next NPDES permit renewal cycle.

Unless otherwise authorized by the DEQ, the City will be required to use the DEQ's MMP template and will need to include the following:

- Identification and evaluation of current and potential mercury sources, including industrial, commercial, and residential
- An implementation plan that includes specific methods for reducing mercury
- Mercury sample results for samples collected within the previous 5 years
- Annual average effluent mercury concentrations and mass loads
- Annual average biosolids concentrations and mass loads
- Changes that may affect mercury, such as those to operations, treatment, and chemicals used
- Summary of mercury reduction activities implemented during the past 5 years.

After implementation of the MMP, if the DEQ determines mercury levels have not decreased, additional changes may be required and the DEQ may reopen the NPDES permit and modify permit conditions.

2.4 Recycled Water Regulatory Requirements

Recycled water refers to any treated effluent from a domestic wastewater treatment system that, through treatment, is suitable for a direct beneficial purpose [OAR 340-055-0010(13)]. The DEQ allows the use of recycled water for beneficial purposes so long as the use provides a resource value, protects public health, and protects the environment (OAR 340-055-0007). Recycled water use in Oregon requires at a minimum:

- A NPDES or WPCF permit, and
- A DEQ Recycled Water Use Plan (RWUP).

2.4.1 Recycled Water Use Permit Exemptions

OAR 340-055-0013 specifically allows the use of wastewater effluent within the WWTP for landscape irrigation or in treatment plant processes (e.g., mixing of polymers) without additional treatment if the following four conditions are met:

- The water is oxidized and disinfected;
- The water is used at the site of generation;
- Aerosol drift does not occur off-site; and
- Public access is restricted.

The City is required to report the quantity of such water reuse with the monthly DMR. If recycled water is used exclusively for exempt uses, the City is not required to develop a RWUP. Exempt recycled water use should be identified in the NPDES Permit Evaluation Report.

2.4.2 Recycled Water Beneficial Purposes

OAR 340-055 identifies several approved beneficial purposes for recycled water use within defined classes of water. The DEQ Recycled Water Internal Management Directive (IMD) summarizes the beneficial purposes allowed by rule and categorizes them into three general categories as shown in Table 2-21. Beneficial purposes identified in the rule may not be authorized for use with a lower class of water. The DEQ may approve other beneficial purposes not otherwise identified in OAR 340-055-0016.

Table 2-21: Recycled Water Beneficial Uses[a]

Beneficial Use	Class A	Class B	Class C	Class D	Nondisinfected
Irrigation					
Fodder, fiber, seed crops not intended for human ingestion, commercial timber	Yes	Yes	Yes	Yes	Yes
Firewood, ornamental nursery stock, Christmas trees	Yes	Yes	Yes	Yes	No
Sod	Yes	Yes	Yes	Yes	No
Pasture for animals	Yes	Yes	Yes	Yes	No
Processed food crops	Yes	Yes	Yes	No	No
Orchards or vineyards if an irrigation method is used to apply recycled water directly to the soil	Yes	Yes	Yes	No	No

Beneficial Use	Class A	Class B	Class C	Class D	Nondisinfected
Golf courses, cemeteries, highway medians, industrial or business campuses	Yes	Yes	Yes	No	No
Any agricultural or horticultural use	Yes	No	No	No	No
Parks, playgrounds, school yards, residential landscapes, other landscapes accessible to the public	Yes	No	No	No	No
Industrial, Commercial, or Construction					
Industrial cooling	Yes	Yes	Yes	No	No
Rock crushing, aggregate washing, mixing concrete	Yes	Yes	Yes	No	No
Dust control	Yes	Yes	Yes	No	No
Nonstructural fire fighting using aircraft	Yes	Yes	Yes	No	No
Street sweeping or sanitary sewer flushing	Yes	Yes	Yes	No	No
Stand alone fire suppression systems in commercial and residential buildings	Yes	Yes	No	No	No
Non-residential toilet or urinal flushing, floor drain trap priming	Yes	Yes	No	No	No
Commercial car washing	Yes	No	No	No	No
Fountains when the water is not intended for human consumption	Yes	No	No	No	No
Impoundments or Artificial Groundwater Recharge					
Water supply for landscape impoundments including, but not limited to, golf course water ponds and non-residential landscape ponds	Yes	Yes	Yes	No	No
Restricted recreational impoundments	Yes	Yes	No	No	No
Nonrestricted recreational impoundments including, but not limited to, recreational lakes, water features accessible to the public, and public fishing ponds	Yes	No	No	No	No
Artificial groundwater recharge	Yes	No	No	No	No

Note:

(a) As shown in the DEQ Recycled Water Internal Management Directive (IMD).

Requirements for the four classes of recycled water are presented in Table 2-22.

Table 2-22: Recycled Water Classes Based on Treatment[a]

Item	Class A	Class B	Class C	Class D	Nondisinfected
Treatment Required					
Oxidized	Yes	Yes	Yes	Yes	Yes
Disinfected	Yes	Yes	Yes	Yes	No
Filtered	Yes	No	No	No	No
Water Quality Requirements					
Turbidity (NTU)					
24-hr mean	2	N/A	N/A	N/A	N/A
5% of time during a 24-hr period	5	N/A	N/A	N/A	N/A
Maximum at any time	10	N/A	N/A	N/A	N/A
Monitoring Frequency	hourly	N/A	N/A	N/A	N/A
Total coliform (organisms/100 mL)					
7-day median	2.2	2.2	23	N/A	N/A
Maximum in any sample	23	23	N/A	N/A	N/A
Maximum in 2-consecutive sample	N/A	N/A	240	N/A	N/A
Monitoring Frequency	daily	3/wk	1/wk	N/A	N/A
E. coli (organisms/100 mL)					
30-day log mean	N/A	N/A	N/A	126	N/A
Maximum in any sample	N/A	N/A	N/A	406	N/A
Monitoring Frequency	N/A	N/A	N/A	1/wk	N/A

Note:

(a) As shown in the DEQ Recycled Water IMD.

2.5 Biosolids Regulatory Requirements

The City currently produces Class B biosolids that are land applied to several agricultural sites as liquid biosolids. On occasion, the City disposes of liquid biosolids at the City of Albany's Water Reclamation Facility. Depending on the season, however, a portion of the biosolids generated at the WWTP are dewatered and disposed of at the Coffin Butte Landfill. A copy of the City's current biosolids management plan is provided in Appendix G.

The following provides a summary of regulatory requirements for Class B and Class A biosolids. Although the City currently can produce Class B biosolids, it is considering transitioning to a Class A biosolids program. Because of this, a general regulatory background on both classes of biosolids is presented.

2.5.1 Introduction

Biosolids are the nutrient-rich organic material resulting from the treatment of sewage at domestic WWTPs. Through biosolids management, solid residue from the wastewater treatment process is treated to reduce or eliminate pathogens and minimize odors, forming a safe, beneficial product for land application or disposal.

Biosolids are regulated by both the EPA and the DEQ to ensure quality standards are met. EPA's regulations can be found in 40 Code of Federal Regulations Part 503 (Part 503) and DEQ's Chapter 340 Division 50 of the Oregon Administrative Rules (Division 50). Regulations address pollutant concentrations, pathogen content, odor potential, and basic operational practices. Beneficial reuse of biosolids has long been preferred over historical disposal practices such as incineration or landfilling. Land application practices and marketable biosolids products are encouraged, as an alternative to disposal, by state and federal regulatory authorities. DEQ's December 2005 biosolids guidance titled, "Implementing Oregon's Biosolids Program Internal Management Directive" (Biosolids IMD) provides very useful information for permit writers and the public regarding how Oregon administers state and federal biosolids regulations.

Approximately 95% of biosolids in Oregon are beneficially used as Class B biosolids via agricultural land application or as Class A Exceptional Quality (EQ) biosolids as a marketable product (e.g., compost). Overall, a slight reduction in the beneficial use of biosolids has occurred because of the loss of farmland to development and decrease in public acceptance of Class B biosolids land application. For the same reasons, there has been an increase in the implementation of Class A EQ biosolids programs. Overall, the regulatory outlook for biosolids management in Oregon remains supportive but there is a trend toward use of Class A EQ biosolids and the additional flexibility presented to municipalities in markets for the end product.

2.5.2 Federal Regulations

Biosolids treatment for disposal and beneficial use is regulated at the federal level by the EPA to ensure quality standards are met. Promulgated in 1993, the Part 503 regulations set forth quality standards so that biosolids are protective of human health and the environment. Under these regulations, biosolids must meet risk-based pollutant limits and controls for pathogen reduction and vector attraction reduction (VAR). The rules also describe the requirements for land application, monitoring, testing, and reporting.

The federal regulations define two classes of biosolids based on pathogen reduction: Class B and Class A. Class B biosolids are treated but still contain detectable levels of pathogens. When utilizing Class B biosolids for land application, the site must be permitted. Agronomic application rates are specified and buffer requirements, public access restrictions, and crop harvesting restrictions must be met. This allows time for any pathogens that are present to be destroyed by environmental exposures to temperature changes, sunlight, drying, and competing soil microorganisms. Class A biosolids receive additional treatment and contain insignificant levels of pathogens. Class A biosolids that meet EQ standards have fewer restrictions on their use or sale to the public. Class A EQ biosolids meet the most stringent requirement for pathogens, vector control, pollutant concentrations, and are safe for unregulated use. In most cases when a facility refers to producing a "Class A biosolids product" they meet the EQ designation as well.

2.5.2.1 Class A Pathogen Reduction Requirements

Pathogen reduction can be achieved by treating solids prior to beneficial use or disposal and through environmental attenuation. Treatment processes are available that use a variety of approaches to reduce pathogens and alter the solids so that it is a less effective medium for microbial growth (EPA, 2003). The 40 CFR Part 503 lists treatment technologies that are judged to produce biosolids with pathogens sufficiently reduced to protect public health and the environment. The regulation also allows the use of any other technologies that produce

biosolids with adequately reduced pathogens as demonstrated through microbiological monitoring.

There are six alternative methods for demonstrating Class A pathogen reduction. The objective of all these requirements is to reduce pathogen densities to below detectable limits. In addition to undergoing a treatment process, Class A biosolids must also be tested for bacteria. Class A biosolids must meet one of the following bacteria limits and one of the process treatment alternatives.

Biosolids must comply with one of the following bacteria limits:

- Fecal coliform is less than 1,000 Most Probable Number (MPN) per gram of total solids (dry weight).
- *Salmonella sp*. Bacteria density is less than 3 MPN per 4 grams total solids (dry weight).

Biosolids must meet one of the following treatment alternatives:

- *EPA Class A alternative 1:* Maintain the sludge at the time, temperature, and percent solids determined by using the formula in paragraph 503.32(a)(3) of 40 CFR Part 503.
- *EPA Class A alternative 2:* Maintain the temperature of the sludge above 52°C (126°F) for 72 hours. The sludge must be above pH 12. Air dry the sludge to 50 percent solids or higher, per paragraph 503.32(a)(4).
- *EPA Class A Alternatives 5 or 6:* Use a Process to Further Reduce Pathogens (PFRP, see Table 2-21 below) or equivalent treatment process approved by the permitting authority, per paragraph 503.32(a)(7) and (8).

Table 2-23: Process to Further Reduce Pathogens

Process	Requirement
Composting	Using either the within-vessel composting method or the static aerated pile composting method, the temperature of sewage sludge is maintained at 55°C (131°F) or higher for 3 consecutive days. Using the windrow composting method, the temperature of the sewage sludge is maintained at 55°C (131°F) or higher for 15 consecutive days or longer. During the period when the compost is maintained at 55°C (131°F) or higher, there shall be a minimum of five turnings of the windrow.
Heat Drying	Sewage sludge is dried by direct or indirect contact with hot gases to reduce the moisture content of the sewage sludge to 10% or lower. Either the temperature of the sewage sludge particles exceeds 80°C (176°F) or the wet bulb temperature of the gas in contact with the sewage sludge as the sewage sludge leaves the dryer exceeds 80°C (176°F).
Heat Treatment	Liquid sewage sludge is heated to a temperature of 180°C (356°F) or higher for 30 minutes.
Thermophilic Aerobic Digestion	Liquid sewage sludge is agitated with air or oxygen to maintain aerobic conditions and the mean cell residence time (MCRT) (i.e., the solids retention time) of the sewage sludge is 10 days at 55°C (131°F) to 60°C (140°F).

Process	Requirement
Beta Ray Irradiation	Sewage sludge is irradiated with beta rays from an electron accelerator at dosages of at least 1.0 megarad at room temperature (approximately 20°C [68°F]).
Gamma Ray Irradiation	Sewage sludge is irradiated with gamma rays from certain isotopes, such as Cobalt 60 and Cesium 137, at dosages of at least 1.0 megarad at room temperature (approximately 20°C [68°F]).
Pasteurization	The temperature of the sewage sludge is maintained at 70°C (158°F) or higher for 30 minutes or longer.

2.5.2.2 Class B Pathogen Reduction Requirements

The alternatives for Class B biosolids consist of either a treatment process such as a Process to Significantly Reduce Pathogens (PSRP) or a fecal coliform bacteria limit. Biosolids must comply with the following bacteria limit:

- The geometric mean of the density of fecal coliform must be less than 2,000,000 MPN, per gram of total solids (dry weight).

Solids must undergo one of the PSRPs shown in Table 2-22 (as listed in Appendix B of 40 CFR Part 503) or an equivalent treatment method approved by the permitting authority.

Table 2-24: Process to Significantly Reduce Pathogens

Process	Requirement
Aerobic Digestion	Sewage sludge is agitated with air or oxygen to maintain aerobic conditions for a specific MCRT (i.e., solids retention time) at a specific temperature. Values for the MCRT and temperature shall be between 40 days at 20°C (68°F) and 60 days at 15°C (59°F).
Air Drying	Sewage sludge is dried on sand beds or on paved or unpaved basins. The sewage sludge dries for a minimum of 3 months. During 2 of the 3 months, the ambient average daily temperature is above 0°C (23°F).
Anaerobic Digestion	Sewage sludge is treated in the absence of air for a specific MCRT (i.e., solids retention time) at a specific temperature. Values for the MCRT and temperature shall be between 15 days at 35°C to 55°C (95°F to 131°F) and 60 days at 20°C (68°F).
Composting	Using either the within-vessel, static aerated pile, or windrow composting methods, the temperature of the sewage sludge is raised to 40°C (104°F) or higher and remains at 40°C (104°F) or higher for 5 days. For 4 hours during the 5-day period, the temperature in the compost pile exceeds 55°C (131°F).
Lime Stabilization	Sufficient lime is added to the sewage sludge to raise the pH of the sewage sludge to 12 for ≥2 hours of contact.

2.5.2.3 Vector Attraction Reduction Requirements

The pathogens in biosolids may pose a disease risk only if there are routes by which the pathogens are brought into contact with humans or animals (EPA, 2003). A primary route for transport of pathogens is vector transmission. Vectors are any living organism capable of transmitting a pathogen from one organism to another either mechanically or biologically by playing a specific role in the life cycle of the pathogen. Vectors for pathogens would most likely include insects, rodents, and birds. The VAR is accomplished by implementing one of the following:

- Biological processes which breakdown volatile solids, reducing the available nutrients for microbial activities and odor producing potential
- Chemical or physical conditions which stop microbial activity
- Physical barriers between vectors and volatile solids in the solids.

The term “stability” is often used to describe sewage sludge or biosolids. Although it is associated with VAR, stability is not regulated by 40 CFR Part 503. Stability is generally defined as the point at which food for microbial activity is no longer available (EPA, 2003). Because stability is also related to odor generation and the continued degradation of solids, it is often considered an important parameter when producing Class A EQ biosolids for sale or distribution. Solids must undergo one of the VAR options set forth in 40 CFR Part 503, shown in Table 2-25 below.

Table 2-25: Vector Attraction Reduction Options

VAR Option	Requirement	Most Appropriate for the Following
Option 1 503.33(b)(1)	At least 38% reduction in volatile solids during sewage sludge treatment	Sewage sludge processed by: anaerobic or aerobic biological treatment
Option 2 503.33(b)(2)	Less than 17% additional volatile solids loss during bench-scale anaerobic batch digestion of the sewage sludge for 40 additional days at 30°C to 37°C (86°F to 99°F)	Only for anaerobically digested sewage sludge that cannot meet the requirements of Option 1
Option 3 503.33(b)(3)	Less than 15% additional volatile solids reduction during bench-scale aerobic batch digestion for 30 additional days at 20°C (68°F)	Only for aerobically digested liquid sewage sludge with 2% or less solids that cannot meet the requirements of Option 1 – e.g., sewage sludges treated in extended aeration plants. Sludges with 2% or greater solids must be diluted.
Option 4 503.33(b)(4)	Specific oxygen uptake rate (SOUR) at 20°C (68°F) is ≤ 1.5 mg oxygen/hr/g total sewage sludge solids	Liquid sewage sludges (2% or less solids) from aerobic processes run at temperatures between 10°C to 30°C (should not be used for composted sewage sludges)

VAR Option	Requirement	Most Appropriate for the Following
Option 5 503.33(b)(5)	Aerobic treatment of the sewage sludge for at least 14 days at over 40°C (104°F) with an average temperature of over 45°C (113°F)	Composted sewage sludge (for sewage sludges from other aerobic processes, it will likely be easier to meet Option 3 or 4)
Option 6 503.33(b)(6)	Addition of sufficient alkali to raise the pH to at least 12 at 25°C (77°F) and maintain a pH ≥ 12 for 2 hours and a pH ≥ 11.5 for 22 more hours	Alkali-treated sewage sludge (alkaline materials include lime, fly ash, kiln dust, and wood ash)
Option 7 503.33(b)(7)	Percent solids ≥ 75% prior to mixing with other materials	Sewage sludges treated by an aerobic or anaerobic process (i.e., sewage sludges that do not contain unstabilized solids generated in primary wastewater treatment)
Option 8 503.33(b)(8)	Percent solids ≥ 90% prior to mixing with other materials	Sewage sludges that contain unstabilized solids generated in primary wastewater treatment (e.g., heat-dried sewage sludges)
Option 9 503.33(b)(9)	Sewage sludge is injected into soil so that no significant amount of sewage sludge is present on the land surface 1 hour after injection, except Class A biosolids which must be injected within 8 hours after the pathogen reduction process	Sewage sludge applied to the land or placed on a surface disposal site. Domestic septage applied to agricultural land, a forest, a reclamation site, or placed on a surface disposal site.
Option 10 503.33(b)(10)	Sewage sludge is incorporated into the soil within 6 hours after application to land or placement on a surface disposal site, except Class A biosolids which must be applied to or placed on the land surface within 8 hours after the pathogen reduction process	Sewage sludge applied to the land or placed on a surface disposal site. Domestic septage applied to agricultural land, a forest, a reclamation site, or placed on a surface disposal site.

2.5.2.4 Pollutant Concentration Requirements

Biosolids for beneficial use must meet risk-based pollutant limits to protect public health and the environment. The 40 CFR Part 503 rules (Section 503.13) set regulatory limits for certain pollutants (metals) and requires biosolids be used in accordance with approved management practices including operational standards, monitoring, recordkeeping, and reporting.

The nine pollutants regulated are: arsenic, cadmium, copper, lead, mercury, molybdenum, nickel, selenium, and zinc. These limits determine how the DEQ regulates land application under a jurisdictions permit. The regulatory limits include the following:

2.5.2.4.1 Ceiling Concentration Limits

This is the maximum concentration of each pollutant allowed in biosolids for beneficial use. According to 40 CFR Part 503, biosolids containing any pollutant that exceeds the Ceiling Concentration Limits (CCLs) cannot be beneficially used. This is also known as EPA Table 1, which is shown in the second column of Table 2-26 below.

2.5.2.4.2 Cumulative Pollutant Loading Rate

The Cumulative Pollutant Loading Rate (CPLR) is the maximum amount of a pollutant that can be applied to a site over its lifetime by all biosolids applications meeting ceiling concentration limits. Biosolids applications must be discontinued when any one of the pollutants reaches its maximum CPLR. This is also known as EPA Table 2, which is shown in the third column of Table 2-24 below.

2.5.2.4.3 Pollutant Concentration Limits

The Pollutant Concentration Limits (PCLs) are used along with the pathogen reduction and VAR requirements as quality standards for Class A EQ biosolids. Biosolids with pollutant concentrations below the PCLs can be sold or given away without a permit from the EPA or the DEQ. However, these Class A EQ biosolids must still be land applied at agronomic rates. Biosolids with pollutant concentrations above the PCL require a permit, must be applied at an agronomic rate, and the cumulative amounts of pollutants must be tracked. This is also known as EPA Table 3, which is shown in the fourth column of Table 2-26 below.

2.5.2.4.4 Annual Pollutant Loading Rate

The Annual Pollutant Loading Rate (APLR) sets the maximum amount of a pollutant that can be applied during a 365-day period. These rates apply to non-EQ biosolids. This is also known as EPA Table 4, which is shown in column 5 of Table 2-26 below.

Table 2-26: Pollutant Limits and Loading Rates for Biosolids

Pollutant[a]	Ceiling Concentration Limits, EPA Table 1 (mg/kg)[b]	Cumulative Pollutant Loading Rate Limits, EPA Table 2 (mg/kg)[b]	Pollutant Concentration Limits, EPA Table 3 (mg/kg)[b]	Annual Pollutant Loading Rate Limits, EPA Table 4 (mg/kg/365-d-period)[b]
Arsenic	75	41	41	2.0
Cadmium	85	39	39	1.9
Copper	4,300	1,500	1,500	75
Lead	840	300	300	15
Mercury	57	17	17	0.85
Molybdenum	75	- [c]	- [c]	- [c]
Nickel	420	420	420	21
Selenium	100	100	100	5.0
Zinc	7,500	2,800	2,800	140

Notes:

(a) Source: EPA 40 CFR Part 503; University of Georgia Extension (March 2017).

(b) Dry-weight basis: mg/kg - milligrams per kilogram; kg/ha - kilograms per hectare.
(c) February 25, 1994 40 CFR Part 503 Rule Amendment deleted the molybdenum limits but retained the molybdenum CCL.

2.5.3 State of Oregon Regulations

In addition to federal requirements, the DEQ also implements regulations overseeing biosolids management in Oregon. The state rule, OAR 340-050, incorporates the legal requirements in 40 CFR Part 503 but goes further to require specific plans for land application, best management practices (e.g., setbacks), and additional public notice requirements.

In Oregon, biosolids are regulated under the DEQ's water quality program through a water quality permit (NPDES or Water Pollution Control Facility permit), a biosolids management plan, and land application site authorization letters. The permit, management plan, and site authorization letters are specific to a facility and include conditions relevant to both state and federal regulations. The conditions in the management plan and site authorization letters are considered an integral part of the permit and thus are enforceable. Oregon's biosolids regulations are more restrictive than federal regulations.

Each permit is open for public comment when the facility's permit is renewed. The facility's biosolids management plan is also open for public comment when the facility's permit is renewed and anytime there are significant changes to the management plan. The public comment period is at least 30 or 35 days depending on the type of permit.

2.5.4 Future Biosolids Regulations

As part of this project, Kennedy Jenks contacted the DEQ's biosolids program staff to discuss their opinion regarding current and future trends in biosolids management. Additionally, we keep abreast of national, regional, state, and local biosolids regulatory issues. The following are findings associated with these communications and industry regulatory trends:

- When discussing potential future regulatory requirements, DEQ staff stated they did not foresee any immediate changes to OAR 340-050. However, they are intending on issuing a general permit or individual Water Pollution Control Facility (WPCF) permit for biosolids land application to allow facilities an additional option for permitting beneficial use.
- The staff at DEQ mentioned there is a general trend of moving from Class B to Class A EQ programs in Oregon. The trend allows more flexibility in biosolids product use and helps protect against unforeseen regulatory changes and common public perception issues encountered with Class B land application programs. Concerns are focused on odor, emerging contaminants, and population encroachment near farmland. In the mid-Willamette Valley, these concerns are currently less acute than areas closer to the Portland-metro area.
- The EPA and DEQ are actively addressing concerns regarding per- and polyfluoroalkyl substances (PFOA and PFAS) in biosolids as well as multiple other mediums (e.g., air and land). This is a very active and evolving issue throughout the country.
    - The DEQ has designated PFAS as one of 60 priority chemicals or chemical classes for its Toxics Reduction program. In addition, DEQ is working with the Oregon Health Authority (OHA) and other federal, state, and local agencies to

address growing public health and environmental concerns being raised. DEQ and OHA are evaluating appropriate policy responses to protect public health and the environment from PFAS contamination.

  - DEQ air, land and water programs, and laboratory, are taking steps to address PFAS by identifying sites that may use PFAS in their operations, and overseeing site testing (potentially biosolids land application sites) and assessment of impacts.
  - DEQ, Oregon State University (OSU), and other organizations are partnering to further study PFAS related to biosolids land application to better understand impacts.
  - Newly developed analytical methods are being used to test for PFAS in water and the DEQ is working with the EPA and other agencies to develop and refine testing methods for soil and biosolids.
  - As of October 2024, the EPA is preparing to release a Draft Risk Assessment on PFOA and PFOS in biosolids for public comment. Policy implications from this information are unclear at this time. However, it is reasonable to assume additional regulatory requirements (e.g., testing, site restrictions on sensitive areas, etc.) will be implemented.

- PFAS is ubiquitous in the environment. Like most toxic substances, eliminating the production and/or preventing PFAS from entering the waste stream are the most effective methods to mitigate the presence of PFAS in biosolids. Current research into processes and technologies to remove PFAS from biosolids have concluded a limited suite of options. Thermal drying of biosolids (e.g., mechanical dryers) is the most probable pre-treatment step necessary prior to an application that destroys PFAS (e.g., pyrolysis).
- Kennedy Jenks recommends the City closely monitor the findings and outcomes of the PFAS issues to better understand possible impacts to Class B land application.

Section 3: Existing Facilities

This section describes the City's existing wastewater collection and treatment facilities. While the focus of this Plan is to identify and recommend improvements to the City's WWTP, a short description of collection system facilities is provided serving as background for the economic evaluation presented in Section 6 of the Plan on the cost effectiveness of infiltration and inflow (I&I) removal.

3.1 Collection System

The City maintains and operates a sewage collection system within its UGB that includes over 300,000 feet of pipe and eight pump stations. The following provides a short overview of the existing collection system and an analysis of I&I.

3.1.1 System Description

The collection system consists of a gravity-fed system with pipe diameters ranging from 6 inches to 54 inches, as summarized in Table 3-1.

Table 3-1: Collection System Inventory

Sewer Pipe Diameter, Inches	Pipe Length, Feet[(a)]
6	7,900
8	194,600
10	31,800
12	43,500
14	700
15	100
16	3,500
18	11,200
20	1,300
21	3,200
24	16,800
27	8,700
30	5,300
36	7,900
42	4,900
54	7,900

Note:
(a) Values rounded to the nearest 100 feet.

As shown on Figure 3-1, there are two main sanitary sewer interceptor systems: the East Side Interceptor (ESI) system and the West Side Interceptor (WSI) system. The ESI system generally serves the area bounded by River Drive in the south, W. Mary Street to the north, 6th Street to

the west, and the South Santiam River on the east. The WSI system generally serves the area to the west of 6th Street and extending north and south to the edges of the UGB.

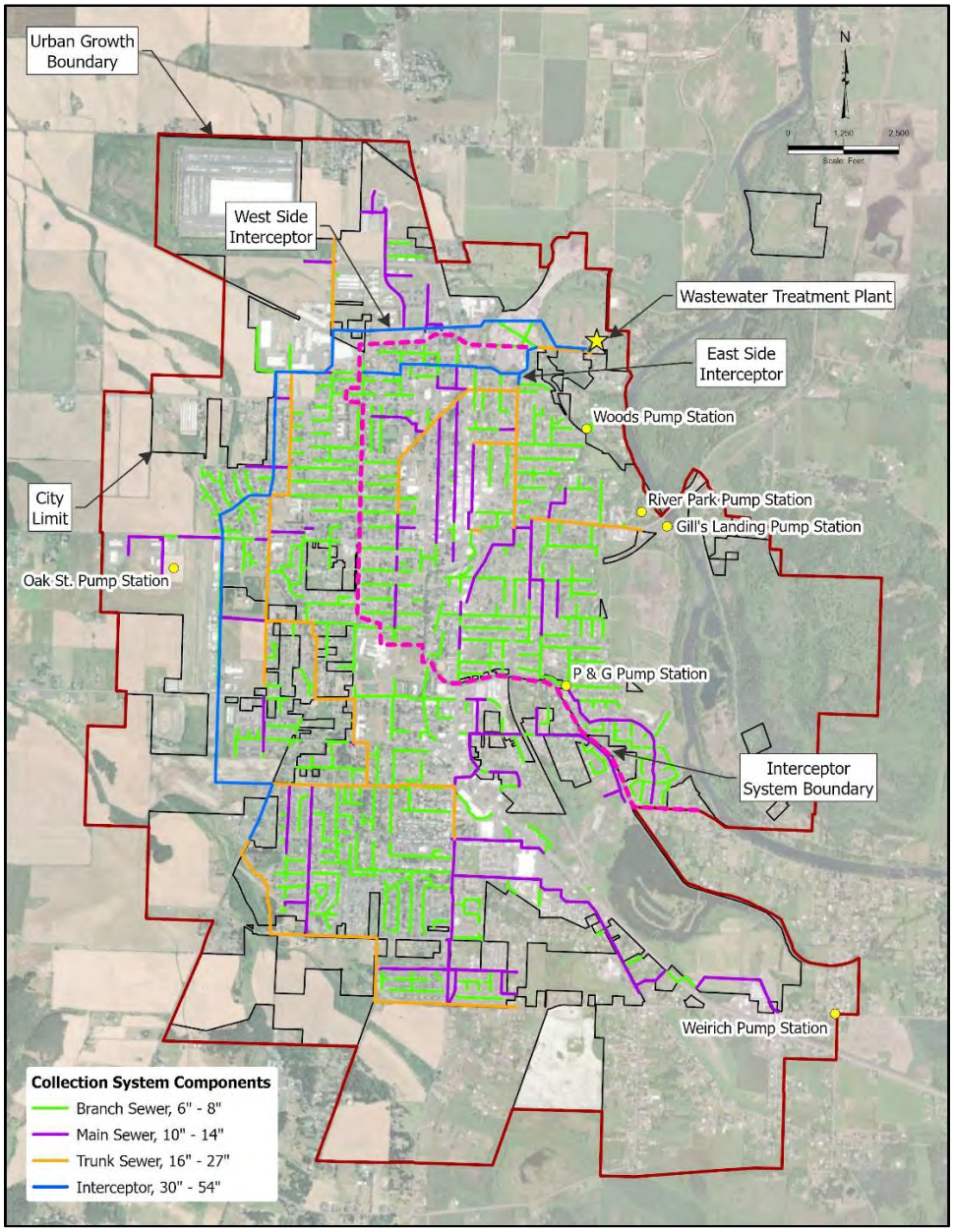


Figure 3-1: Collection System

3.1.1.1 Gravity Sewers

The City's collection system consists of gravity sewers constructed over a period of more than 100 years. The gravity system generally flows northward following the general topography of the service area ultimately discharging to the WWTP located in the northeast of the City near the South Santiam River.

The ESI system is surrounded by the WSI system on its south, west, and north sides with the South Santiam River to the east. The WSI's primary trunk line starts in the southeast corner of the service area with 10-inch gravity lines that gradually increase in size to the 18-inch and 20-inch trunk line that runs northward along Williams Street starting at the intersection of E. Grant Street. The Williams Street trunk line joins an 18-inch trunk line that collects flows from the 5th Street area at the intersection of Williams Street and Wheeler Street and increases in size to 36 inches. The combined 36-inch interceptor is joined by a 30-inch gravity line from the west on N. Williams Street. The 36-inch interceptor continues north until turning eastward and splitting into parallel 24-inch and 27-inch interceptors that convey flows from the ESI system to the WWTP.

The WSI system surrounds the ESI system serving regions to the north, west, and south. Significant improvements to the WSI were completed in 2024 with the WSI extension project. This project extended the WSI system's main trunk line around the western most side of the service area connecting to the existing 42-inch trunk line at the intersection of W. Oak Street and W. Airway Drive and continuing southward to Crowfoot Road ending at the intersection of Crowfoot Road and Hillview Drive. Size of the new WSI system trunk line ranges from 24 inches at the upstream end along Crowfoot Road to 36 inches at the connection to the existing 42-inch trunk line at the intersection of W. Oak Street and W. Airway Drive. The 42-inch trunkline continues northward along W. Sherman Street and S. 12th Street until connecting to a 54-inch interceptor at Harrison Street. A 27-inch trunk line that predates the recent WSI extension project starts near the intersection of S. Airport Road and S. 12th Street and runs parallel to the next trunk line northward along S. 12th Street and then S. 11th Street before also connecting to the 54-inch interceptor in Harrison Street. The 54-inch interceptor conveys all wastewater flow from the WSI system eastward to the WWTP starting near the intersection of Harison Street and 11th Street.

Older portions of the collection system were originally constructed as combined storm water and sanitary sewer systems. Starting in 1986, the City has worked to gradually eliminate combined sewers through the construction of a dedicated storm water system. Inflows from storm events remain a large component of wet weather flows and the City is actively managing a 5-phase, I&I reduction program. The following is a summary of the I&I reduction program:

- Phase 1 Downtown Sewer Separation: The first phase of the I&I reduction program was completed in 1998 and eliminated stormwater connections to the sanitary sewer system in three, two-block areas on Ash, Vine, and Grant Streets.

- Phase 2 Downtown Sewer Separation: The second I&I reduction phase was completed in 2000 and eliminated stormwater connections to the sanitary sewer system in three, two-block areas on Sherman, Maple, and Rose Streets.

- Phase 3 Downtown Sewer Separation: The third phase project has been partially completed with the separation of the storm water system from the sanitary sewer system in the alley from E. Oak Street to E. Maple Street between S. Main Street and Park Street. Additional elements of the Phase 3 project are included in the City's 2022 to 2027 capital improvement plan (CIP) that will eliminate stormwater connections to the sanitary sewer in three downtown areas including Ash Street (between Main Steet and Second Street), Second Street (between Ash Street and Grant Street) and Grant Street (alley

between Main Street and Park Street). The project will also complete smoke testing of the last 20% of the collection system.

- Phase 4 Downtown Sewer Separation: This project phase will be the final downtown sewer separation project. It is included in the City's 2022 to 2027 CIP and will eliminate stormwater connections to the sanitary sewer in four downtown areas including Second Street (between Rose Street and Vine Street), Sherman Street (alley between Main Street and Park Street), Second Street (between Grant Street and Oak Street), and Maple Street (between Main Street and Park Street).

- Phase 5 Residential Sewer Separation: This final phase of the I&I program includes on-going efforts to eliminate non-sanitary sewer connections outside the downtown area and primarily focuses on connections within residential areas of the City.

The City completed separate capacity evaluations for the WSI and ESI systems in 2020 and 2022, respectively. The analyses determined system capacity under both existing and future buildout conditions to identify potential deficiencies. A summary of these evaluations performed for the City by David Evans and Associates Inc. (DEA) is presented below for reference. The reader should refer to the technical memoranda prepared by DEA for additional information.

The 2020 WSI system analysis was performed assuming completion of the recently constructed WSI trunk line from W. Oak Street to Crowfoot Drive. Results showed that with the addition of the new trunk line, the WSI system will have sufficient capacity to accommodate dry weather sanitary sewer flows through full buildout. However, although the new trunk line significantly reduces the risk of sanitary sewer overflows, there are multiple locations (29 manholes) within the WSI system where overflows are still anticipated to occur during 5-year storm events (DEA, 2020). Because of this continued risk, the evaluation of the WSI system included recommendations for additional analysis to identify and eliminate sources of I&I to avoid the need for additional capacity improvements that would otherwise be required.

The 2022 ESI system analysis was conducted to identify capacity deficiencies associated with future buildout including the proposed development of the Old Mill Health Campus and Neighborhood that is south of Grant Street and west of the Santiam River (DEA, 2022). The analysis showed that the ESI system has sufficient capacity to accommodate dry weather sanitary sewer flows through full buildout. However, like the WSI system, modeling of the ESI system at full buildout and during a 5-year storm event showed the potential sewer overflows at multiple locations (4 manholes). The evaluation again included recommendations for additional evaluation to identify and eliminate sources of storm water entering the sanitary sewer collection system.

3.1.1.2 Emergency Overflows

Three designated overflow points were described in the City's 2001 I&I removal report (City of Lebanon, 2001). These included the Harrison Street Pump Station and two manholes on 15th and 16th Streets that would bypass wastewater to Burhart Creek during high flow emergency conditions. Following the removal of the Harrison Street Pump Station and upgrades to the WSI system trunklines, the City reports there are no longer designated overflow points in the City's collection system. There have been no overflows reported within the last 5 years.

3.1.1.3 Pump Stations

There are six wastewater pump stations within the existing collection system. These include the P&G Station, Wood's Station, Oak Street Station, Weirich Station, Gill's Landing Station, and River Park Station. There are two pump stations at the WWTP. These include the West Side Interceptor Pump Station (WIPS) and the Raw Sewage Pump Station the latter of which is no longer considered operational. The WIPS and Raw Sewage Pump Station are described within the section on WWTP facilities presented later in this section.

The P&G Pump Station is located near the intersection of Park Drive and Garvord Street. The pump station serves the southeastern most portion of the ESI system from Garvord Street to the south end of Mountain River Drive. The pump station consists of two constant speed, self-priming, solids handling pumps. Design Criteria for the pump station are presented in Table 3-2.

Table 3-2: P&G Pump Station Design Criteria

Item	Units	Value
Pumps		
Type	-	Self-priming, solids handling
Number	-	2
Capacity, each	gpm	250
Power, each	hp	5

The Wood's Pump Station is located near the Wood's RV Trailer Park located on Santiam Street. The pump station serves an area east of Lebanon Santiam Canal. The pump station consists of two submersible non-clog pumps. Design Criteria for the pump station are presented in Table 3-3.

Table 3-3: **Wood's Pump Station** Design Criteria

Item	Units	Value
Pumps		
Type	-	Submersible, non-clog
Number	-	2
Capacity, each	gpm	93
Power, each	hp	1

The Oak Street Pump Station is located on Montesa Way and serves the surrounding area. The pump station consists of two submersible, non-clog pumps. Design Criteria for the pump station are presented in Table 3-4.

Table 3-4: Oak Street Pump Station Design Criteria

Item	Units	Value
Pumps		
Type	-	Submersible, non-clog
Number	-	2
Capacity, each	gpm	345
Power, each	hp	5

The Weirich Pump Station is located on Weirich Drive and serves the Cheadle Lake area. The pump station consists of two self-priming, solids handling pumps. Design Criteria for the pump station are presented in Table 3-5.

Table 3-5: Weirich Pump Station Design Criteria

Item	Units	Value
Pumps		
Type	-	Self-priming, solids handling
Number	-	2
Capacity, each	gpm	350
Power, each	hp	10

The Gill's Landing Pump Station is located on E. Grant Street and serves the Gill's Landing RV Park. The pump station consists of two submersible grinder pumps. Design Criteria for the pump station are presented in Table 3-6.

Table 3-6: **Gill's Landing Pump Station Design Criteria**

Item	Units	Value
Pumps		
Type	-	Submersible, grinder
Number	-	2
Capacity, each	gpm	55
Power, each	hp	2

The River Park Pump Station is also located on E. Grant Street. The station serves the River Park facilities including the park host site, RV dumps station, and restrooms. The pump station consists of one submersible solids handling pump. Design Criteria for the pump station are presented in Table 3-7.

Table 3-7: River Park Pump Station Design Criteria

Item	Units	Value
Pumps		
Type	-	Submersible, solids handling
Number	-	1
Capacity, each	gpm	55
Power, each	hp	0.5

3.1.2 Infiltration and Inflow Analysis

I&I is water within the collection system other than base sanitary flow. Infiltration refers to the entry of groundwater through defects in the collection system including damaged pipe, faulty joints, and leaky manholes. Inflow refers to the direct entry of water into the system through

direct connections that can include storm water catch basins, basement sump pumps, and roof drains connected to private service laterals.

The Environmental Protection Agency (EPA) has guidelines with thresholds that, when paired with analysis of flow events at the WWTP, can help determine if I&I is a significant contributor to the wastewater flows and whether it is excessive. Excessive can be defined as I&I that is more economical to remove through collection system improvements than it is to convey and treat. I&I that causes sanitary sewer overflows can also be deemed excessive.

When determining if I&I is excessive, wastewater flow values are first compared with EPA guideline values. If below the guideline values, I&I can be considered non-excessive. However, if they are above these values, an economic evaluation is needed to determine if I&I is excessive.

The EPA guideline for infiltration is based on the highest 7-day average flow recorded over a 7 to 14-day period during times of high groundwater with no rainfall. If the flow during such a period is less than 120 gallons per capita per day (gcd), infiltration is considered non-excessive. A review of City rainfall data showed that from 2017 to 2021 there were 9 occurrences of 7 to 14-day periods with no rain during times of normally high groundwater (January through April). Values of the highest 7-day average WWTP flow for these periods, population, and per capita flow are presented in Table 3-8. As indicated in the table, values exceed EPA guidelines below which infiltration can be considered non-excessive. A cost effectiveness analysis is thus needed to determine if infiltration is excessive.

Table 3-8: WWTP Flows During High Groundwater and No Rainfall[(a)]

Date Range	Population[(b)]	7-day Average Flow, MGD	7-day Average Flow, gcd[(c)]
1/23/2017 – 1/29/2017	16,720	5.46	327
1/23/2019 – 1/31/2019	17,135	5.13	299
3/13/2019 – 3/20/2019	17,135	5.14	300
4/20/2019 – 4/30/2019	17,135	6.10	356
3/15/2020 – 3/22/2020	17,335	3.51	202
4/6/2020 – 4/17/2020	17,335	4.21	243
1/17/2021 – 1/23/2021	19,122	6.18	323
3/29/2021 – 4/8/2021	19,122	3.24	169
4/10/2021 – 4/22/2021	19,122	2.58	135

Notes:

(a) Maximum 7-day average WWTP flows during 7 to 14-day periods with no rainfall and high groundwater (January through April).
(b) Population values from Table 4 Populations for Oregon and its Counties and Incorporated Cities and Towns, compiled April 15, 2023, PSU Population Research Center.
(c) gcd = gallons per capita per day.

The EPA guideline for inflow is 275 gcd based on wet weather flow, defined as the highest daily flow recorded during a storm event. The maximum day WWTP flow, population, and maximum day flow on a per capita basis for the years 2017 through 2021 are shown in Table 3-9.

As shown in the table, the annual values for each year of the 5-year period were significantly higher than the EPA value below which inflow can be considered non-excessive.

Table 3-9: Maximum Day WWTP Flows

Year	Population[(a)]	Max Day Flow, MGD	Max Day Flow, gcd[(b)]
2017	16,720	15.6	930
2018	16,920	10.5	620
2019	17,135	19.1	1120
2020	17,335	16.6	960
2021	19,122	15.1	790

Notes:

(a) Population values from Table 4 Populations for Oregon and its Counties and Incorporated Cities and Towns, compiled April 15, 2023, PSU Population Research Center.

(b) gcd = gallons per capita per day.

As presented above, per capita rates of I&I exceed the EPA threshold below which I&I can be considered non-excessive. Because of this, an evaluation was performed to determine if it would be more economical to reduce I&I through improvements to the City's collection system than to convey and treat. This evaluation is summarized in Section 6 of this Plan.

3.2 Wastewater Treatment Plant

The WWTP, which is situation in the northeast portion of the City (Figure 3-2), has a permitted ADWF of 3.0 MGD and is located at the intersection of Beaton Lane and Tennessee Road. It was originally constructed in 1954 and consisted of a grit channel, comminutor, primary clarifier, trickling filter, secondary clarifier, chlorine disinfection, aerobic digester, and sludge drying beds. Structures from the original WWTP that are still in service include the trickling filter, primary clarifier, and secondary clarifier. In 1976, the trickling filter was converted for use as an aerobic digester. The primary clarifier was converted to a secondary clarifier.

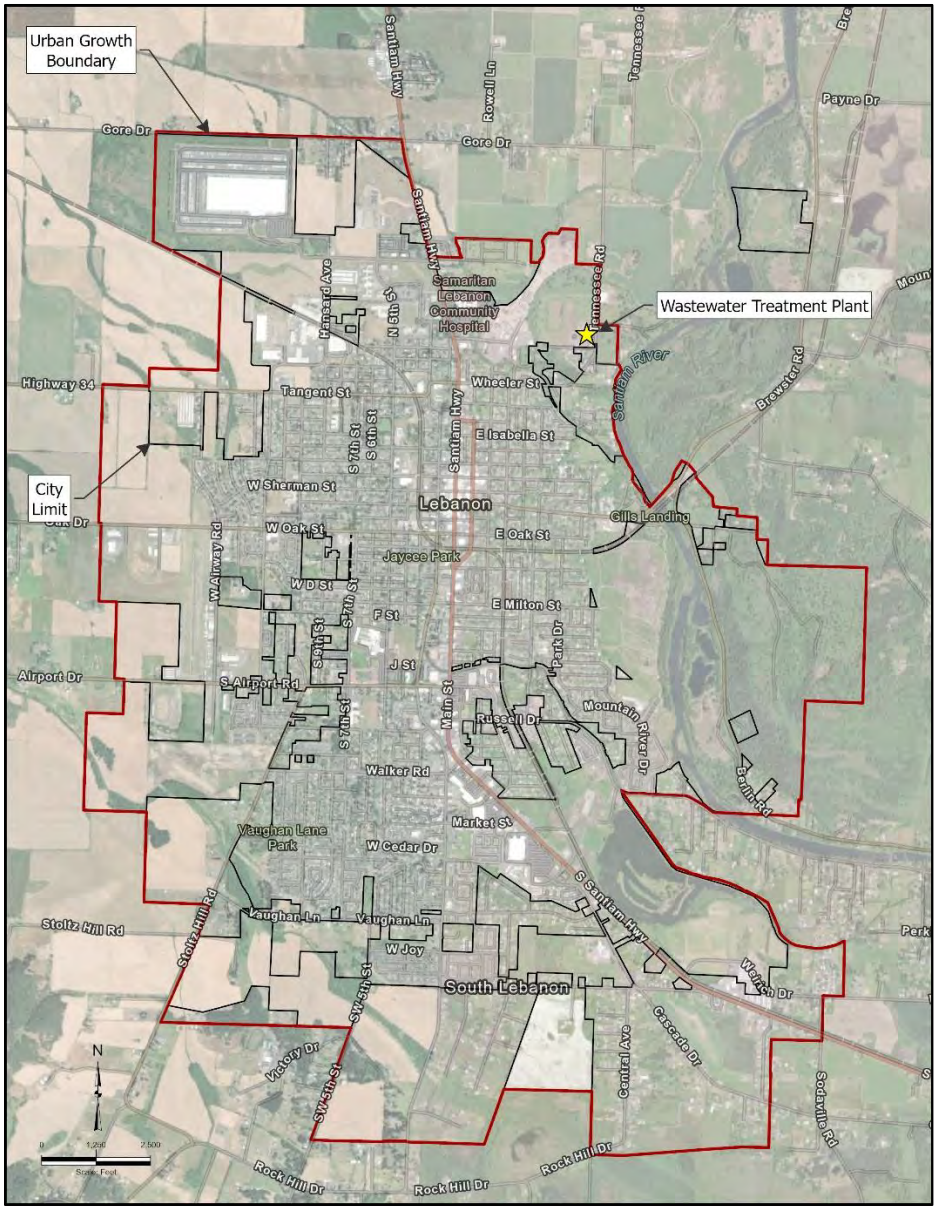


Figure 3-2: Lebanon WWTP Location Map

There have been numerous modifications to the WWTP since its original construction. The most significant of these changes are summarized in Table 3-10, below.

Table 3-10: WWTP Modifications

Year	Project Name	Project Elements
1976	Wastewater Treatment Plant Upgrades	• Raw Sewage Pump Station • Headworks • Storm Screens • Aeration Basins • Convert Primary Clarifier to Secondary Clarifier • Tertiary Filtration • Chlorine Contact Basins with Gaseous Chlorine • Sludge Pump Station • Convert Trickling Filter to Aerobic Digester • Convert Digester to Sludge Storage Tank • Control Building
1992	Sludge Thickening Facility	• Sludge Thickening Building • Gravity Belt Thickener
1999	Sodium Hypochlorite Conversion Plan	• Sodium Hypochlorite Storage and Metering
2003	West Side Interceptor Pump Station	• Submersible Influent Pump Station • New Engine Driven Standby Generator
2008	Effluent Discharge System Improvements	• 42-inch Outfall and River Diffuser • Sodium Bisulfite Dechlorination Storage and Metering
2008	Cannibal® Project	• Convert to Cannibal® Solids Reduction System • Convert Aerobic Digester to Interchange Reactor • Aeration Basin Upgrades
2012	Clarifier Expansion Improvements	• 130-foot Secondary Clarifier • RAS/WAS/Scum Pumping
2015	Effluent Pump Station Improvements	• New Submersible Pumps • Electrical Enclosure
2020	City Resumes Operation of Plant	• Eliminate Cannibal® Process • Convert Interchange Reactor to Aerobic Digester • New Belt Filter Press

Existing WWTP facilities are shown on Figure 3-3. They are described in detail in the subsequent sections of the Plan.

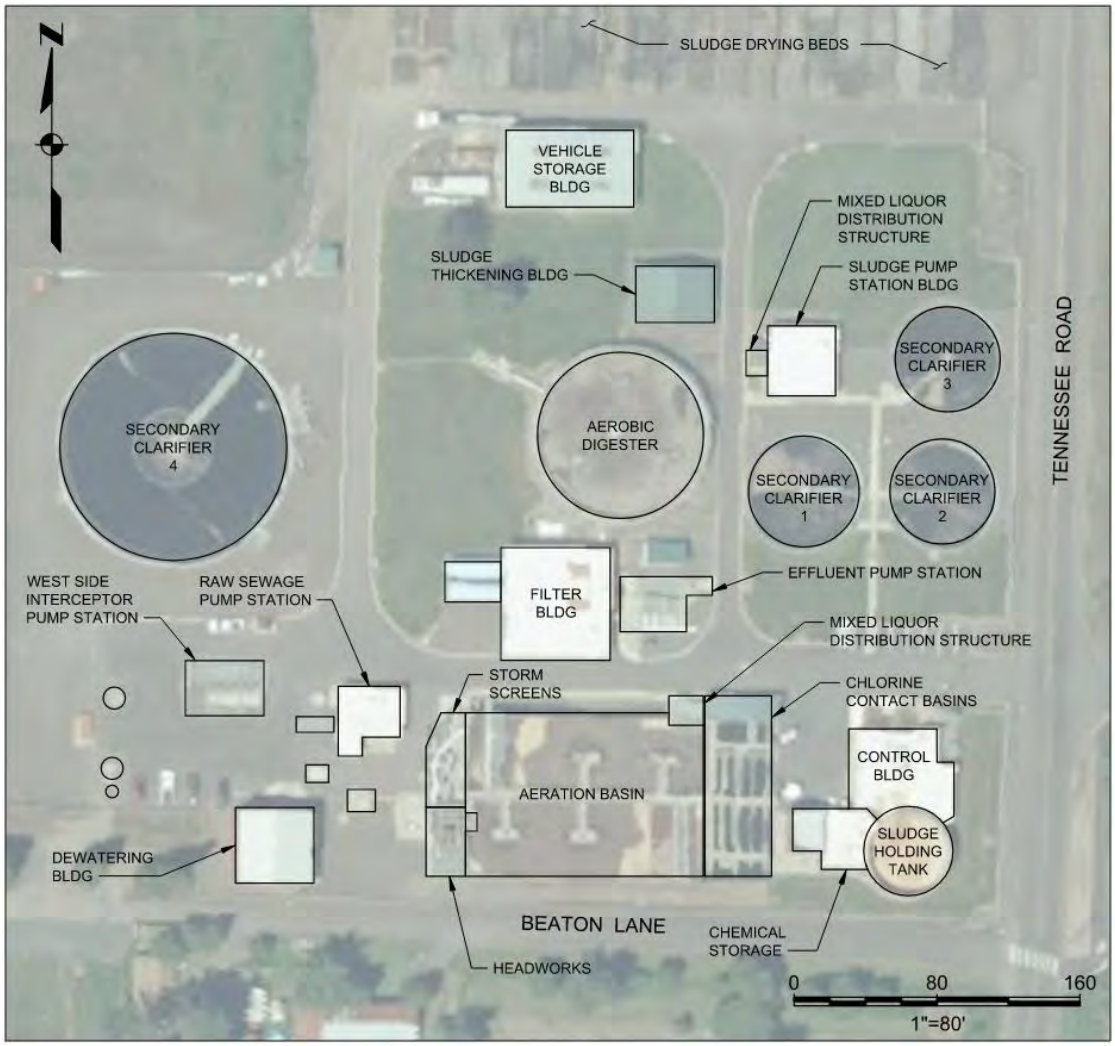


Figure 3-3: Existing Wastewater Treatment Plant Facilities

3.2.1 Liquid Stream Treatment Process

Liquid stream processes currently active at the WWTP include the following:

- Influent Pumping
- Preliminary Treatment
- Secondary Treatment (Activated Sludge)
- Secondary Clarification
- Effluent Pumping
- Chlorine Disinfection
- Dechlorination
- Outfall to the South Santiam River

The tertiary filtration facilities constructed as part of the 1976 upgrades project are still present. However, they were removed from service in 2020 and are no longer considered to be functional.

A flow schematic of the existing liquid stream treatment processes is presented on Figure 3-4 with descriptions of each unit process following.

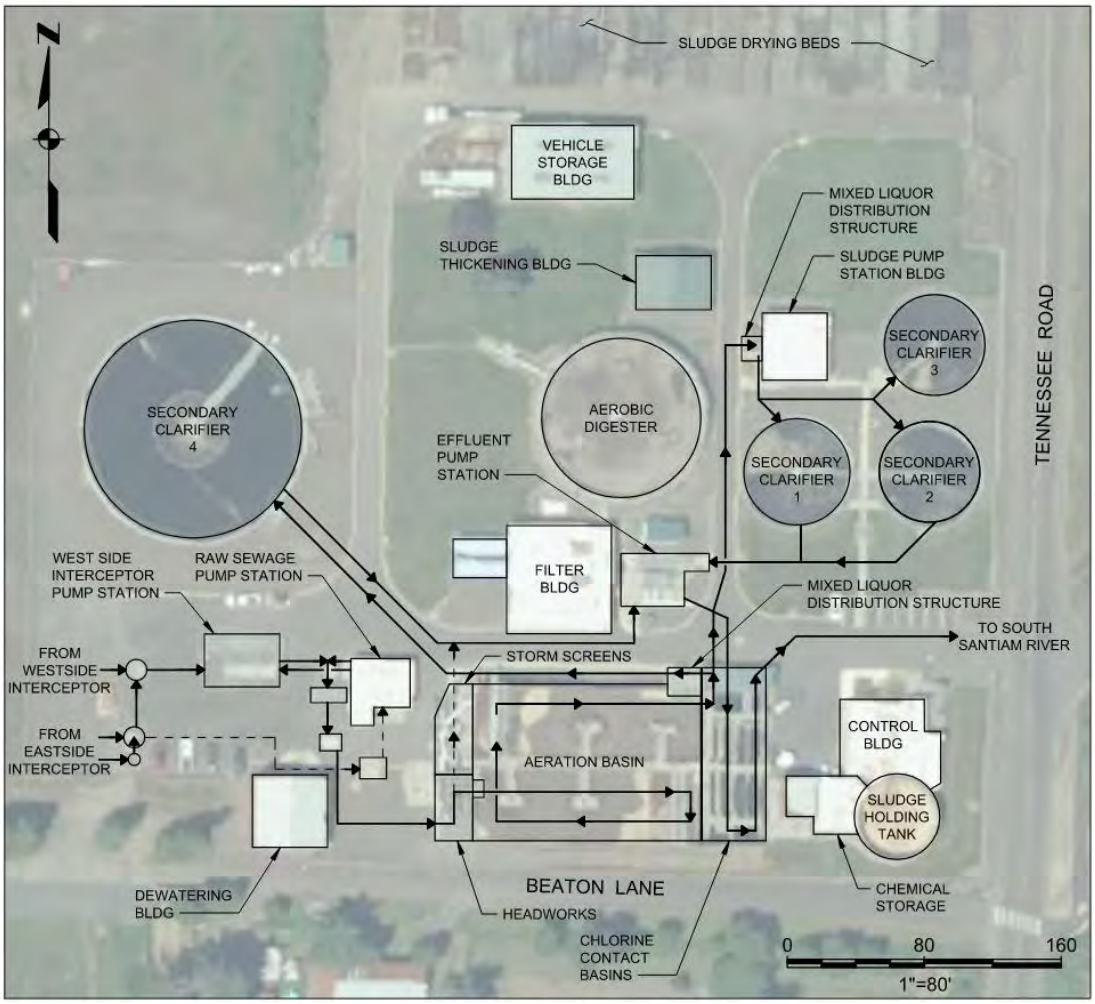


Figure 3-4: Liquid Stream Treatment Process Flow Schematic

3.2.1.1 Influent Pumping

Raw sewage from the WSI system enters the WWTP from the west via a 54-inch gravity pipeline and discharges to the 120-inch diameter WSI manhole. Flow continues to the west via another 54-inch line and into the WIPS. Raw sewage from the ESI system enters the WWTP from the west via parallel 24-inch and 27-inch pipelines that discharge to 60-inch and 120-inch diameter manholes, respectively. A 24-inch line connects the two manholes. Flow from the manholes is directed north via a 36-inch line into the WSI manhole and then to the WIPS.

The WIPS, which was completed in 2003 and shown on Figure 3-5, is a variable speed, submersible, self-cleaning wet well-type pumping station with five available pump slots. The station design anticipated phased expansion to increase capacity. The current configuration installed as part of the 2003 project includes three large (85-hp and 75-hp) pumps and one smaller (25-hp) pump. The smaller pump functions as a jockey pump operating under low flow conditions. The station has a firm capacity of 16 MGD with two large pumps running. The station was designed for a maximum firm capacity of 37 MGD with the future installation of up to five larger (150-hp) pumps.

Figure 3-5: West Side Interceptor Pump Station

WIPS pumps discharge to a common 24-inch discharge header before being directed through one of two magnetic flow meters (10-inch and 20-inch). Meter selection is made by opening and closing 10-inch and 20-inch plug valves in an upstream vault. The 10-inch meter is used for flows up to 8 MGD. The 20-inch meter is used for flows greater than 8 MGD. The 10-inch and 20-inch lines combine into a 24-inch line that conveys flow to the headworks.

Design criteria for the existing station are summarized in Table 3-11.

Table 3-11: West Side Interceptor Pump Station Design Criteria

Item	Units	Value
Pumps		
Type	-	Submersible centrifugal
Number		
Large pump[a]	-	3
Small pump[b]	-	1
Large pump capacity, each	gpm	5,500
Motor power, each		
Large pump[c]	hp	75 and 85
Small pump	hp	25
Drive type	-	Variable speed
Station capacity[c]	MGD	16

Notes:

(a) Includes one redundant pump.
(b) The smaller pump is a jockey pump that operates during low flow conditions.
(c) One 75-hp and two 85-hp pumps.
(d) Firm capacity with two large pumps running.

The older Raw Sewage Pump Station was constructed in 1976. It is a wet well/dry well type station, contains four pumps, and had an original design capacity of 26.8 MGD. The station has two, 75-hp variable speed pumps; one, 25-hp variable speed pump, and one 50-hp constant speed pump. The pumps are located within the dry well on the bottom level of the pump station. Drive shafts connect the pumps to their respective motors that are located on the ground floor along with other station electrical equipment. The Raw Sewage Pump Station was meant to serve as backup for the WIPS during peak flow events exceeding the capacity of the WIPS. However, WWTP staff report that discharge piping from the station appears to have been plugged and the station is no longer operational.

Preliminary Treatment

The headworks was constructed in 1976 as part of a large common wall structure that included the headworks, aeration basins, storm flow diversion, and chlorine contact basins (CCBs). Raw sewage enters the headworks through a 24-inch pipe and flows by gravity through one of three parallel channels. One channel contained a comminutor and the second channel contained a manual bar screen. The third channel was provided for future expansion. As shown on Figure 3-6, the headworks now includes two mechanically cleaned bar screens that discharge to a screw auger. The first mechanical screen was installed in 1999 and replaced the comminutor. The second mechanical screen was installed in 2021. A manually raked bar screen provides additional redundancy. The headworks contains no grit removal.

Figure 3-6: Existing Headworks Screening Equipment

An automatic sampler pulls a sample immediately prior to screening. This is the sampling point for compliance with NPDES permit plant influent wastewater sampling.

After screening, wastewater flows into a common channel that also receives discharge from the Drainage Pump Station. A parallel channel receives return activated sludge (RAS) from the secondary clarifiers. A series of gates within the two parallel channels allow WWTP staff to direct RAS, screened sewage, or a combination of the two to one (or both) of the aeration basins.

The flow rate through the headworks is limited to approximately 14 MGD. When flows exceed this amount, excess flow is diverted around the headworks via an overflow weir upstream of the bar screens and is directed through two rotary storm screens. The existing storm screens are shown on Figure 3-7. Effluent from the storm screens is blended with secondary effluent before entering the CCB for disinfection.

Figure 3-7: Existing Rotary Storm Screens

An automatic composite sampler draws from the influent channel immediately prior to screening. This is the sampling point for compliance with NPDES permit plant influent wastewater sampling.

Preliminary treatment design criteria are presented in Table 3-12.

Table 3-12: Preliminary Treatment Design Criteria

Item	Units	Value
Mechanical Screens		
Type	-	Climber
Number	-	2
Channel width	inch	36
Bar spacing	inch	0.5
Power	hp	3
Manual Screens		
Number	-	1
Channel width	inch	30
Bar spacing	inch	1.5
Storm Screens		
Type	-	Rotary
Number	-	2
Screen size	inch	0.05
Power	hp	3

Secondary Treatment

The WWTP's existing activated sludge treatment system was constructed in 1976 as part of a larger common wall structure that included the two parallel aeration basins, headworks, storm flow diversion, and CCBs. The basins as originally constructed were only able to operate in parallel. Alternate operating modes (e.g., series operation, step feed, sludge reaeration, etc.) were not possible. Aeration was provided by six surface aerators, three per basin. The basins were operated as aerobic, complete-mix reactors.

The aeration basins were modified as part of the 2008 Cannibal® Project. The current configuration of the basins is shown on Figure 3-8. A series of baffle walls and weirs were added with vertical shaft mixers to create two unaerated selector zones in each aeration basin. Isolation gates were provided to allow multiple operating modes including series, parallel, sludge reaeration, and step feed. WWTP staff select the appropriate operating mode to maximize performance of the secondary treatment process in response to seasonal changes in influent flows and to facilitate routine maintenance.

The 2008 project also included four new, variable speed surface aerators which replaced the original six aerators. Dissolved oxygen (DO) probes were provided to measure DO in the two basins and are used to control aerator speed and the number of aerators in operation.

Figure 3-8: Existing Aeration Basins

Following biological treatment, mixed liquor spills over a weir and into a common mixed liquor channel along the east end of the aeration basins before entering one of two flow control structures. The first flow structure is located at the east end of the north aeration basin and

directs mixed liquor to Clarifier 4 using two electrically actuated weir gates. The second flow splitting structure is located on the west side of the Sludge Pump Station. Manually actuated weir gates are used to split and control flow between Clarifiers 1, 2, and 3. Coagulant (i.e., aluminum chlorohydrate) can be added to the mixed liquor channel to enhance performance of the secondary clarifiers.

The WWTP has four secondary clarifiers. Clarifiers 1 and 2 predated the 1976 upgrades project. Clarifier 1 was originally a 60-foot-diameter, primary clarifier that was converted to a secondary clarifier during the 1976 upgrades project. This same project added a 55-feet clarifier (Clarifier 3) that matched Clarifier 2. The three clarifiers shown on Figure 3-9 have sidewall depths of 10 feet and were originally rated for a peak overflow rate of 8 MGD. They are equipped with scraper type sludge collector mechanisms that move settled sludge to central sumps where it is drawn off by the RAS pumps. There are four recessed impeller-type, RAS pumps located in the Sludge Pump Station that serve Clarifiers 1, 2, and 3. Each clarifier has a dedicated RAS pump with the fourth pump providing redundancy. The pumps are operated at variable speed to control RAS return rates.

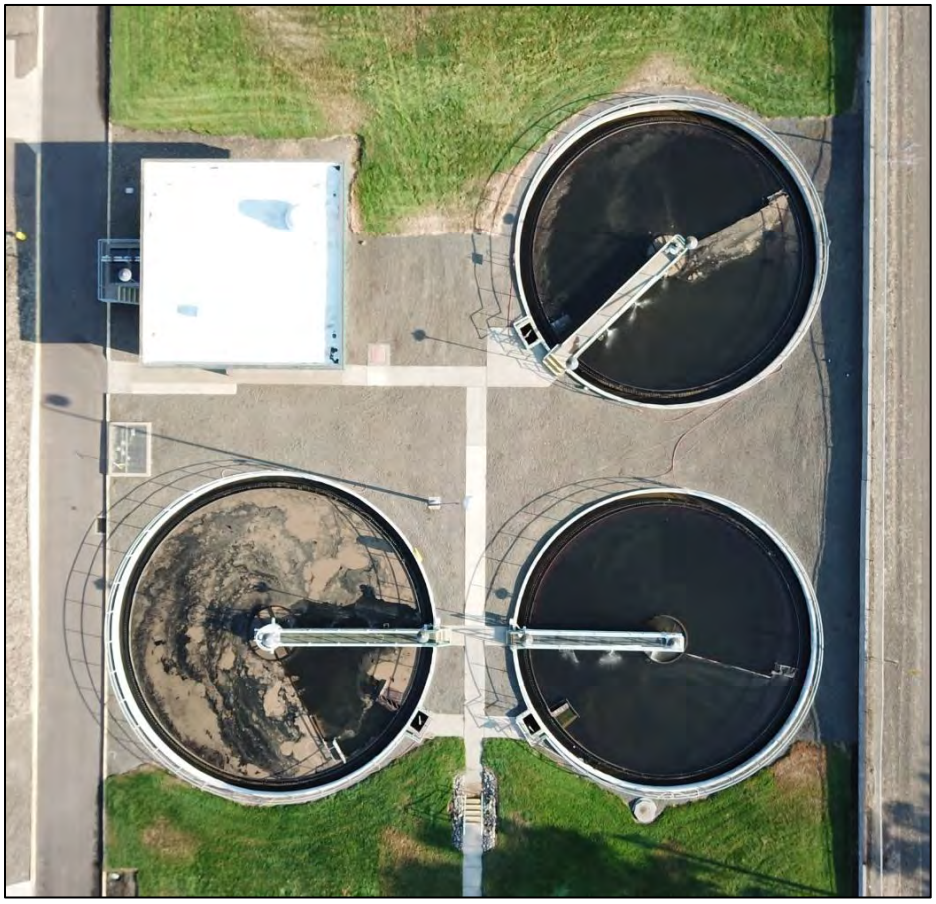


Figure 3-9: Secondary Clarifiers 1, 2, and 3

Clarifier 4, shown on Figure 3-10, was constructed in 2012 and has an inside diameter of 130 feet with a sidewall depth of 16 feet. Clarifier 4 was designed for a peak wet weather flow of 17 MGD and is equipped with a suction-type sludge collector mechanism with two suction arms. RAS is pulled towards the center column and is conveyed beneath the clarifier floor through a

20-inch RAS pipe to the adjacent RAS Pump Station that contains three variable speed, submersible pumps. The WAS and scum pumping are provided by dedicated constant speed, submersible pumps located in separate wet wells located on the outer wall of the clarifier. Because of the age, condition, and overall poor performance of Clarifiers 1, 2, and 3, Clarifier 4 operates year-round with the smaller clarifiers being kept offline except during higher flow periods of the year.

Figure 3-10: Secondary Clarifier 4

Design criteria for the existing secondary treatment process is presented in Table 3-13.

Table 3-13: Secondary Treatment Design Criteria

Item	Units	Value
Aeration Basins		
Number	-	2
Volume, total	gal	935,000
Depth	ft	11
Selectors		
Number per basin	-	2
Volume, each	gal	44,000

Item	Units	Value
Aeration		
Type	-	Variable speed, surface
Number per basin	-	2
Power	hp	75
Selector Mixing		
Type	-	Vertical shaft, propeller
Number per zone	-	1
Power	hp	5
Secondary Clarifiers		
Number	-	4
Clarifiers 1, 2, and 3		
Type	-	Scraper
Diameter, Clarifier 1	ft	60
Diameter, Clarifiers 2 and 3	ft	55
Depth	ft	10
Power, each	hp	0.5
Clarifier 4		
Type	-	Suction
Diameter	ft	130
Depth	ft	16
Power	hp	0.75
RAS Pumping		
Clarifiers 1, 2, 3		
Type	-	Recessed impeller
Number	-	4
Capacity, each	gpm	1,000
Power	hp	40
Clarifier 4		
Type	-	Submersible
Number	-	3
Capacity, each	gpm	2,400
Power	hp	25

Filtration

Tertiary filtration facilities were constructed as part of the 1976 upgrades project and consisted of two pressure type filter vessels, filter feed pumps, backwash pump, chemical feed systems, and the associated Filter Building. The filtration system was originally designed to operate between June 1 and October 31 to reduce effluent suspended solids. However, the system has not been operated for more than 20 years and was officially removed from service in 2020.

Secondary Effluent Pumping

Secondary effluent from the secondary clarifiers, as well as flow diverted through the storm screens during high flow events, are pumped to the CCBs by the Effluent Pump Station. The station was originally constructed in 1976 and later upgraded in 2015. Secondary effluent from Clarifiers 1, 2, and 3 combine and flow through a Parshall flume before entering the station's wet well. Flow from the storm screens and secondary effluent from Clarifier 4 combine in a

30-inch-diameter line before also discharging to the wet well. The wet well contains a dividing weir wall. When the filters were operational, it allowed secondary effluent to be directed to the tertiary filters.

The Effluent Pump Station shown on Figure 3-11 currently contains four, variable speed, submersible pumps that were installed in 2015. Electrical equipment for the pump station is housed in the small structure shown immediately to the north of the station. The pumps consist of two larger capacity pumps and two smaller capacity pumps. The pumps are installed within separate sections of the wet well that can be isolated from one another by manually operated slide gates. Pump speed and number of pumps running are controlled to maintain preset water level within the station's wet well. The discharge piping from each pump combines into a single 24-inch pipe that conveys flow to the CCB.

Figure 3-11: Effluent Pump Station

Design criteria for the Effluent Pump Station are presented in Table 3-14.

Table 3-14: Effluent Pump Station Design Criteria

Item	Units	Value
Effluent Pumps		
Type	-	Submersible
Number, total	-	4
Capacity		
Pumps 1 and 2	gpm	7,650
Pumps 3 and 4	gpm	1,840
Station capacity[(a)]	mgd	16.3
Power		
Pumps 1 and 2	hp	100
Pumps 3 and 4	hp	20

Note:

(a) Firm capacity with largest pump out of service.

Disinfection

Sodium hypochlorite is used to disinfect secondary effluent prior to discharge to the South Santiam River. Secondary effluent from the Effluent Pump Station is distributed to the existing CCBs via a common 36-inch pipe header. There are two parallel CCBs that can be operated in series or in parallel. Sluice gates in the common header allow secondary effluent to be directed to either of the two existing CCBs. Additional sluice gates allow the CCBs to be operated in series or in parallel. Each basin can be independently removed from service for routine maintenance. The CCBs empty to a common discharge chamber with flow leaving the structure over a fixed weir. The CCBs are shown on Figure 3-12.

Figure 3-12: Chlorine Contact Basins (CCBs)

The original chlorine gas disinfection system was replaced with a sodium hypochlorite system in 1999. The chlorine storage room on the southwest side of the Control Building structure houses a single, 2,500-gal, polyethylene sodium hypochlorite storage tank. However, WWTP staff report that a new 4,700-gal double walled storage tank has been installed outside and the smaller storage tank is no longer in use. Sodium hypochlorite is pumped from the tank by two, flow-paced, peristaltic metering pumps. The sodium hypochlorite is injected into the Effluent Pump Station wet well. An in-line total chlorine residual analyzer measures chlorine residual within the CCB.

Sodium bisulfite is added to dechlorinate the final effluent downstream of the CCB at the fixed weir prior to discharge. The sodium bisulfite system is housed in the dechlorination building constructed as part of the 2008 effluent discharge system improvements. The dechlorination system consists of a polyethylene storage tank and two, flow-paced peristaltic metering pumps.

Design criteria for the existing disinfection process is presented in Table 3-15.

Table 3-15: Disinfection Design Criteria

Item	Units	Value
Chlorine Contact Basins		
Number	-	2
Dimensions		
Length	ft	75
Width	ft	13
Depth	ft	16.5
Volume CCB	gal	240,000
Total volume[(a)]	gal	280,000
Contact time		
ADWF (2.3 MGD)	min	150
PDF (18.1 MGD)	min	19
PIF (29.2 MGD)	min	12
Hypochlorite Dosing		
Storage volume	gal	4,700
Metering pumps		
Number	-	2
Capacity	gph	15.8
Bisulfite Dosing		
Storage volume	gal	1,830
Metering pumps		
Number	-	2
Capacity	gph	7.9

Note:

(a) Total volume within the CCB structure including the discharge chamber (volume equal to 40,000 gal). Although this volume provides some additional contact time, it is not part of the main serpentine type structure so was not included in the calculation of the detention times shown.

Final Effluent Discharge

After the final effluent weir, dechlorinated effluent enters a common 36-inch header and flow split structure where effluent sampling is taken for NPDES permit compliance. Dechlorinated effluent is conveyed to the South Santiam River through a 1,500-foot-long, 42-inch-diameter outfall constructed in 2008. Discharge to the river is via a multi-port diffuser buried beneath the bottom of the riverbed. The diffuser is approximately 24 feet long and consists of eleven, evenly spaced, 12-inch duckbill diffusers on 12-inch-diameter HDPE risers and is located at river mile 17.4.

3.2.2 Solids Stream Treatment Process

Solids stream treatment processes currently in use at the WWTP include the following:

- WAS pumping
- Sludge thickening

- Aerobic digestion
- Dewatering
- Sludge drying beds
- Class B biosolids land application.

A flow schematic of the existing solids stream treatment processes is presented in Figure 3-13 with descriptions of each unit process following.

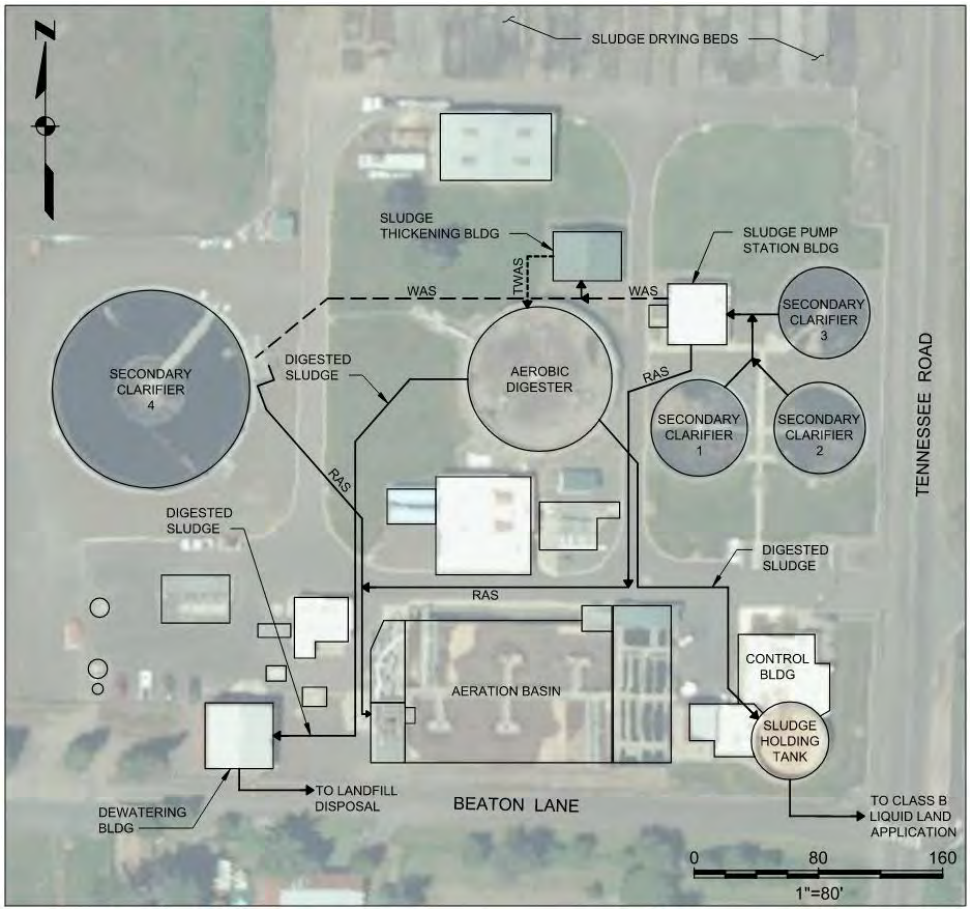


Figure 3-13: Solids Stream Treatment Process Flow Schematic

3.2.2.1 Waste Activated Sludge Pumping

The WAS pumping facilities consist of one horizontal, recessed impeller type pump located inside the Sludge Pump Station Building originally constructed in 1976. The pump, shown on Figure 3-14, draws from the RAS pump suction header serving secondary Clarifiers 1, 2, and 3. Valving allows the pump to direct WAS to the Sludge Thickening Building, sludge truck pump, or aerobic digester.

Secondary Clarifier 4, which was constructed in 2012, has one submersible solids handling WAS pump within a wet well immediately adjacent to the clarifier. The WAS from Secondary Clarifier 4 is pumped directly to the Gravity Belt Thickener (GBT).

Figure 3-14: WAS Pump for Clarifiers 1,2, and 3

Design criteria for the existing WAS pumping system is presented in Table 3-16.

Table 3-16: WAS Pumping Design Criteria

Item	Units	Value
Clarifiers 1,2, and 3		
Type	-	Horizontal, recessed impeller
Number	-	1
Capacity	gpm	100
Power	hp	5
Clarifier 4		
Type	-	Submersible, non-clog
Number	-	1
Capacity	gpm	250
Power	hp	5

3.2.2.2 Sludge Thickening

Sludge thickening facilities were constructed in 1992 and consist of a GBT, sludge feed pump, sludge grinder, thickened sludge pump, and polymer feed equipment. Sludge thickening facilities are contained within the 1,400squarefoot Sludge Thickening Building. The existing GBT is shown on Figure 3-15**.**

Figure 3-15: Gravity Belt Thickener

The WAS from the existing secondary clarifiers is pumped to the sludge feed pump inside the Sludge Thickening Building. The feed pump, which is a vertical, screw-type, centrifugal pump. conveys the WAS through a sludge grinder and then to the GBT. A liquid emulsion polymer feed system, installed in 2020, injects polymer immediately downstream of the sludge grinder. Injecting polymer increases the efficiency of the GBT which, although capable of producing sludge concentrations ranging from 3 to 8%, normally thickens sludge to between 4 and 5%. A progressive cavity pump pulls thickened sludge from the thickened sludge hopper and pumps it to the aerobic digester for stabilization. Filtrate from the GBT drains by gravity to the Drainage Pumping Station where it is pumped to the headworks. Design criteria for the existing sludge thickening system are contained in Table 3-17.

Table 3-17: Sludge Thickening Design Criteria

Item	Units	Value
Sludge Thickening		
Type	-	Gravity belt
Number	-	1
Belt width	meter	2
Capacity	lbs/hr[(a)]	1,600
Sludge concentration	percent	4 to 5
Power	hp	3

Item	Units	Value
Sludge Feed Pump		
Type	-	Centrifugal, screw
Capacity	gpm	340
Power	hp	5
Sludge Grinder		
Capacity	gpm	400
Power	hp	3
Thickened Sludge Pump		
Type	-	Progressive cavity
Capacity	gpm	100
Power	hp	10
Polymer Feed Pump		
Number	-	1
Capacity	gal/hr	0.001 to 7.13

Note:

(a) Dry pounds per hour.

3.2.2.3 Aerobic Digestion

Aerobic digestion is used to stabilize wastewater solids from the liquid stream secondary treatment process. The WWTP's 530,000-gallon aerobic digester was originally constructed in 1954 as a trickling filter but was converted to an aerobic digester in 1976. In 2008, it was modified for use as an interchange reactor as part of the conversion to the Cannibal® solids reduction system. Because of poor performance, the Cannibal® system was decommissioned in 2020 and the interchange reactor was returned to use as aerobic digester when City staff took over operation of the WWTP.

The aerobic digester is shown on Figure 3-16**.** Aeration and mixing of digester contents is provided by three floating aerators. The floating mixer shown in the center of the digester is no longer in service and has been removed.

Figure 3-16: Aerobic Digester

When hauled offsite for liquid land application, digested sludge is temporarily stored in a 265,000-gallon sludge holding tank. The sludge holding tank is part of the Control Building structure and was originally an anerobic digester when constructed in 1954. It was converted for use as a sludge holding tank in 1976. A positive displacement blower provides mixing and aeration air. The sludge holding tank and Control Building are shown on Figure 3-17.

Figure 3-17: Sludge Holding Tank and Control Building

The existing aerobic digester does not provide adequate detention time to meet regulatory requirements for Class B biosolids. Because of this, lime stabilization is used by WWTP staff to produce Class B biosolids for land application. This is done by combining lime slurry with the partially stabilized biosolids in the sludge holding tank. After holding for at least the regulatory required minimum detention time, the resulting lime stabilized Class B liquid biosolids are pumped by a self-priming pump to the City's sludge truck for land application. There are two pumps used for truck loading. The pump normally used for this service is a self-priming solids handling pump located in the Sludge Pump Station Building. A second pump, located in the Control Building, can also be used for truck filling but is primarily used for recirculation of sludge in the Sludge Holding Tank.

Design criteria for the existing aerobic digestion system is presented in Table 3-18.

Table 3-18: Aerobic Digestion Design Criteria

Item	Units	Value
Aerobic Digester		
Dimensions		
Diameter	ft	95
Depth	ft	10
Volume	gal	530,000
Aeration/Mixing		
Type	-	Floating mechanical
Number	-	3
Power, each	hp	20

Item	Units	Value
Digested Sludge Holding Tank		
Dimensions		
Diameter	ft	45
Depth	ft	20
Volume	gal	265,000[(a)]
Aeration/Mixing		
Type	-	Air
Blower		
Type	-	Positive displacement
Number	-	1
Power	hp	60
Sludge Truck Pump		
Type	-	Self-priming
Number	-	1
Capacity	gpm	300

Note:

(a) Volume shown includes the volume contained in the cone bottom of the sludge holding tank (28,000 gal).

3.2.2.4 Biosolids Dewatering

Biosolids dewatering facilities consist of a belt filter press (BFP), sludge feed pump, polymer addition system, wash water pump, and dewatered cake conveyor. Dewatering facilities were installed in 2022 inside a pre-engineered metal building originally constructed as part of the Cannibal® process to house RAS screening facilities. When the Cannibal® process was decommissioned because of poor performance, the RAS screening equipment was removed from service and later replaced by the current dewatering equipment.

When dewatering facilities are operating, digested sludge flows by gravity from the aerobic digester to the Dewatering Building. A variable speed, progressive cavity sludge feed pump inside the Dewatering Building pumps the sludge into the flocculation tank at the inlet end of the BFP. The sludge flows into the gravity dewatering section of the BFP (shown on Figure 3-18) with the partially dewatered cake then being further dewatered within the pressure zone of the BFP. The dewatered cake drops into a shaftless screw conveyor that deposits the sludge onto the floor of the Dewatering Building. City staff use a front-end loader to move the dewatered cake into 20-cubicyard (cy) roll-off bins for disposal.

Liquid emulsion polymer is injected upstream of the flocculation tank to increase the dewatering efficiency resulting in the production of dewatered cake with typical solids concentration between 16 and 17%. A multi-stage vertical type pump provides high pressure wash water for belt cleaning. Filtrate from the BFP drains by gravity to the Drainage Pumping Station where it is pumped to the headworks.

Figure 3-18: Biosolids Dewatering Belt Filter Press

Design criteria for the dewatering facilities are present in Table 3-19. WWTP staff reported difficulties with the newly installed BFP and have struggled to achieve consistent dewatering results and the manufacturer's rated capacity for the BFP.

Table 3-19: Dewatering Design Criteria

Item	Units	Value
Biosolids Dewatering		
Type	-	Belt filter press
Number	-	1
Belt width	meter	1.7
Capacity	lbs/hr[(a)]	600[(b)]
Cake concentration	percent	16 to 17
Polymer Feed Pump		
Type	-	Diaphragm
Capacity	gal/hr	1.8
Sludge Feed Pump		
Type	-	Progressive cavity
Capacity	gpm	125
Power	hp	5
Washwater Pump		
Type	-	Multi-stage inline centrifugal
Number	-	1
Power	hp	5

Item	Units	Value
Dewatered Cake Conveyor		
Type	-	Shaftless screw
Power	hp	2

Note:

(a) Dry pounds per hour.
(b) Manufacturer's rated capacity. WWTP staff have reported that they have been unable to achieve this capacity.

3.2.2.5 Sludge Drying Beds

The WWTP has 20 sludge drying beds with a total area of 37,000 square feet. The drying beds (shown on Figure 3-19) were originally constructed 1976 and are in extremely poor condition. Because of their condition, the City uses the drying beds only in emergency situations and for vactor waste.

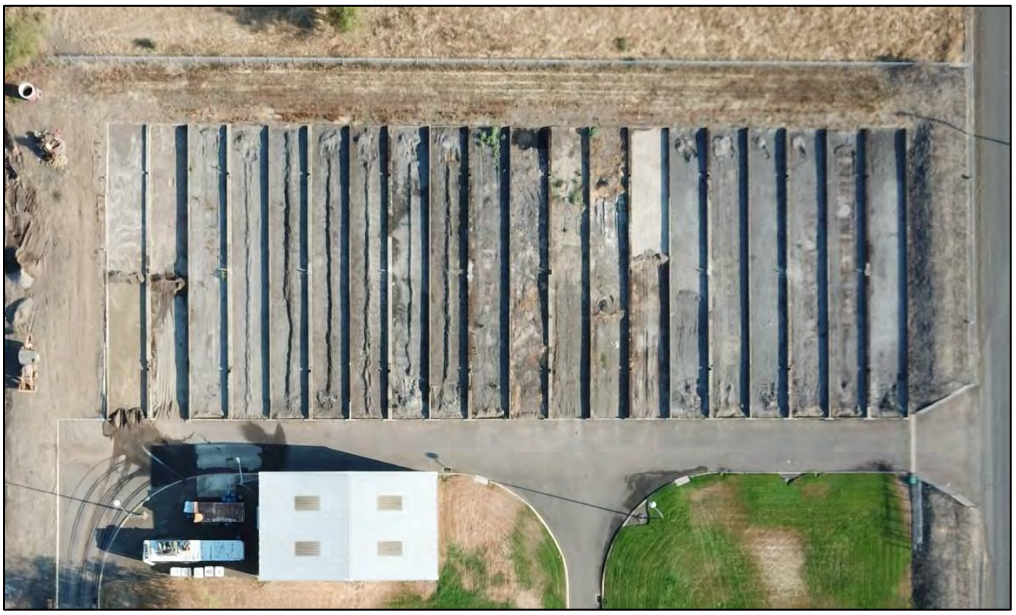


Figure 3-19: Sludge Drying Beds

3.2.2.6 Beneficial Use or Disposal

The City produces aerobically (partially) digested, lime stabilized Class B biosolids that are liquid land applied to five DEQ authorized application sites with a total combined area of approximately 370 acres. Of these five authorized sites, only two are currently in use. Both are used to produce grass hay. One of the sites (Sommers) has a total area of 106 acres and was originally authorized for year-round biosolids application (weather dependent). Of the total potential area, approximately 40 to 60 acres are currently available for use and it is now just authorized for dry season application. The City's 4,800-gallon liquid sludge truck is used to haul to the field at the Sommers site where a self-driven irrigation hose reel system applies the liquid biosolids.

The second site (Morgan) currently in use is authorized for summertime application only. It has a total area of 68 acres although only about 40 acres are usable. Land application at the site

typically begins in June or July and must be completed prior to the end of October unless specifically authorized by the DEQ. Unlike the Sommer site, the City's sludge truck is fitted with a splash plate which is used to apply the liquid Class B biosolids directly to the field at the Morgan site (Figure 3-20).

Figure 3-20: Class B Liquid Land Application

Of the almost 400 dry tons of biosolids produced by the WWTP in 2023, more than 390 were land applied. However, this amount of biosolids was applied when all the City's five authorized application sites were used. Because the City is currently only using two of the five authorized sites and year-round application is no longer allowed, the amount of biosolids that will be land applied is anticipated to drop significantly.

Design criteria for the City's Class B land application program is presented in Table 3-20.

Table 3-20: Biosolids Land Application Design Criteria

Parameter	Units	Value
Biosolids		
Type	-	Lime stabilized, Class B liquid
Concentration	percent	3 to 5
Application Sites[(a)]		
Number	-	5
Total area	acres	600
Sludge Truck Volume	gal	4,800

Note:

(a) Although there are five authorized application sites, only two are currently in use that have a combined total area of approximately 180 acres.

Excess biosolids that cannot be land applied are dewatered and hauled for disposal at the Coffin Butte Landfill. The landfill is about 30 miles northwest of the WWTP in Benton County. While the City landfilled less than 10 dry tons of biosolids in 2023, this amount is expected to increase significantly with the reduction in land application sites currently in use. It should be noted that the landfill operators, although seeking to expand, report there to be approximately

12.9 years of remaining capacity as of January 2023 (Coffin Butte Landfill and Pacific Region Compost Annual Report, 2023).

3.2.3 General Wastewater Treatment Plant Facilities

General WWTP facilities include an existing Control Building, electrical and standby power systems, utility water pumping, and plant drain pumping facilities.

3.2.3.1 Control Building

The WWTP's Control Building shown on Figure 3-21 was originally constructed in 1954 and modified in 1976 along with the conversion of the adjoining anaerobic digester to a sludge holding tank. The building was expanded with new laboratory facilities, mechanical room, gaseous chlorine storage, and electrical room. Office space and bathroom facilities were also provided. Total building space was increased from about 1,200 to 2,700 square feet.

The chlorine gas disinfection system was replaced with a sodium hypochlorite system in 1999. The chlorine storage room on the southwest side of the Control Building which housed chlorine gas cylinders was converted for use as sodium hypochlorite storage and dosing.

Figure 3-21: Control Building

3.2.3.2 Electrical and Standby Power

Electrical facilities including motor control centers (MCCs), transformers, panel boards, switch gear, and standby generator are in multiple buildings throughout the WWTP. Electrical equipment associated with Clarifier 4 and the Effluent Pump Station are in separate, standalone

structures near each of these unit processes. The age of electrical equipment ranges from that of the facilities constructed as part of the 1976 WWTP project to that of the more recently installed 2022 dewatering system. A summary of the main existing electrical equipment, the building in which it is located, equipment served, and year installed is presented in Table 3-21.

Table 3-21: Electrical Equipment Summary

Building	Electrical Equipment	Equipment Powered	Year Installed
Filter Building	MCC-1A	Filter Pumps[(a)]	1976
		Unit Heaters	1976
		Instrument Air	1976
	MCC-2A	Instrument Air	1976
		Unit Heaters	1976
	Switch Gear		1976
	Standby Generator		2003
	Automatic Transfer Switch		2003
Control Building	MCC-1B	Plant Water Pumps	1976
		HVAC	1976
		Wash Water Pumps	1976
		Anoxic Zone Mixers	2008
Raw Sewage Pump Station	MCC-1C	Raw Sewage Pumps[(b)]	1976
		Rotary Storm Screen	1976
		Headworks Screen	1999
		Headworks Screen	2021
		Unit Heaters	1976
		WIPS Pumps 1 and 2	2003
	MCC-2C	Raw Sewage Pumps[(b)]	1976
		Rotary Storm Screen	1976
		Screw Conveyor	1976
		WIPS Pumps 3 and 5	2003
Sludge Pump Station	MCC-1D	Sump Pumps	1976
		Unit Heaters	1976
		RAS Pumps	1976
		Digester Aerators	1976
		WAS Pump	1976
		Scum Pump	1976
		Clarifiers 1 and 2	1976
		Thickening Building	1992
		MCC-4D	2011
	MCC-2D	RAS Pumps	1976
		Digester Aerators	1976
		Clarifier 3	1976
Dewatering Building	MCC-1E	Belt Filter Press	2022
Clarifier 4[(c)]	MCC-4D	Clarifier 4	2011

Building	Electrical Equipment	Equipment Powered	Year Installed
		WAS Pump	2011
		RAS Pumps	2011
		Scum Pump	2011
Effluent Pump Station[(c)]	MCC-20A	Effluent Pumps	2015

Notes:

(a) Tertiary filtration removed from service in 2020.
(b) No longer considered operable.
(c) Electrical equipment in a standalone structure near unit process.

The WWTP's existing diesel driven standby generator is in the northeast portion of the existing Filter Building. The 250-kW engine generator installed as part of the 1976 WWTP project was replaced in 2003 with the 1,250-kW engine generator shown on Figure 3-22.

Figure 3-22: Standby Engine Generator

Design criteria for the existing standby engine generator are presented in Table 3-22.

Table 3-22: Standby Generator Design Criteria

Parameter	Units	Value
Capacity	kW	1,250
Fuel		
Type	-	Diesel
Capacity	gal	1,000

3.2.3.3 Utility Water

The utility water system provides water for non-potable use (e.g., scum sprays, utility water stations, and irrigation) around the WWTP and includes the plant water pump, plant water strainer, plant water booster pump, irrigation pump, and irrigation water strainer. Utility water facilities were installed in 1976 and are located at the north end of the CCB within a metal paneled enclosure as shown on Figure 3-23. The plant water booster pump, which is supplied with City water, is used for redundancy should the plant water pump fail. A backwash pump is located with the other utility water facilities. It was originally installed for use with the existing tertiary filters. Like the filters, it is no longer in use.

Figure 3-23: Utility Water Pumping Facilities

Design criteria for utility water facilities are presented in Table 3-23.

Table 3-23: Utility Water Facilities Design Criteria

Item	Units	Value
Plant Water Pump		
Type	-	Centrifugal
Capacity	gpm	175
Power	hp	20
Plant Water Booster Pump		
Type	-	Centrifugal
Capacity	gpm	150
Power	hp	7.5
Irrigation Pump		
Type	-	Centrifugal
Capacity	gpm	200
Power	hp	15

3.2.3.4 Plant Drain Pumping

The Drainage Pump Station pumps plant recycle and stormwater from catch basins within the WWTP directly to the headworks through a 6-inch pipeline. The station, which was constructed in 2003, consists of two constant speed submersible pumps within a 72-inch manhole. Because the station pumps directly to the headworks, plant recycle and onsite stormwater flows are not included in influent flow measurements. Design criteria for utility water facilities are presented in Table 3-24.

Table 3-24: Plant Drain Pumping Design Criteria

Item	Units	Value
Plant Drain Pumps		
Type	-	Submersible non-clog
Number	-	2
Capacity, each	gpm	500
Power	hp	7.5

Section 4: Need for Project

This section provides information to demonstrate the need for improvements to the City's WWTP facilities. A discussion on the recent performance of the WWTP is presented followed by information specific to the liquid stream and solids stream treatment processes. Regulatory requirements, reliability, and capacity deficiencies including those needed for growth are included within the information presented.

4.1 WWTP Performance

A summary of overall WWTP performance for years 2023 and 2024 for effluent discharge parameters for $CBOD_5$, TSS, bacteria, ammonia and residual chlorine is presented below. Effluent values were compared with discharge limits contained in the WWTP's existing NPDES permit. Values in bold represent those exceeding permit limits. As noted in Section 3 of the Plan, the City took over direct operation of the WWTP in 2020. While the last 2 years of performance data is summarized here, Appendix H contains a complete summary for years 2020 through 2024.

A monthly summary of effluent $CBOD_5$ data is presented in Table 4-1. As shown in the table, the WWTP exceeded effluent standards for $CBOD_5$ in July 2023.

Table 4-1: WWTP Effluent $CBOD_5$ (2023 – 2024)

Month	Influent Flow, MGD		Effluent Concentration, mg/L		Effluent Mass, lbs/day			Percent Removal
	Avg.	Max Day	Avg.	Max Week	Avg.	Max Week	Max Day	Avg.
Jan 2023	5.7	9.1	2	3	109	134	138	95
Feb 2023	3.9	4.6	4	7	132	132	235	94
Mar 2023	5.0	8.6	4	5	178	369	446	91
Apr 2023	6.9	11.6	9	13	603	1,011	1,250	79
May 2023	3.4	4.7	3	5	98	151	169	97
Jun 2023	2.1	2.4	3	4	50	80	85	98
Jul 2023	1.5	2.4	**13**	**39**	187	**599**	**1,108**	92
Aug 2023	1.3	1.4	3	4	29	37	42	99
Sep 2023	1.4	1.9	2	5	26	55	55	98
Oct 2023	2.1	3.3	5	7	85	120	130	94
Nov 2023	3.8	7.1	5	7	180	290	337	93
Dec 2023	6.7	12.0	5	5	304	419	602	89
Jan 2024	8.83	15.2	6	8	492	725	924	80
Feb 2024	6.38	8.40	5	7	252	732	337	88

Month	Influent Flow, MGD		Effluent Concentration, mg/L		Effluent Mass, lbs/day			Percent Removal
	Avg.	Max Day	Avg.	Max Week	Avg.	Max Week	Max Day	Avg.
Mar 2024	6.77	13.4	4	6	214	339	407	90
Apr 2024	4.04	5.3	3	4	104	139	169	96
May 2024	4.9	10.9	3	4	128	192	220	97
Jun 2024	2.7	4.61	3	4	70	120	120	98
Jul 2024	1.8	2.1	3	3	40	51	61	99
Aug 2024	1.7	2.3	4	6	59	99	100	98
Sep 2024	1.7	2.1	3	4	48	52	53	98
Oct 2024	2.2	4.2	3	3	56	65	87	98
Nov 2024	6.2	9.8	3	3	131	184	188	95
Dec 2024	6.7	14.3	3	3	184	191	677	96

A monthly summary of effluent TSS data is presented in Table 4-2. As shown in the table, the WWTP exceeded effluent standards for TSS in July 2023. These permit exceedances coincided with the $CBOD_5$ exceedances presented in Table 4-1.

Table 4-2: WWTP Effluent TSS (2023 – 2024)

Month	Influent Flow, MGD		Effluent Concentration,		Effluent Mass, lbs/day			Percent Removal
	Avg.	Max Day	Avg.	Max Week	Avg.	Max Week	Max Day	Avg.
Jan 2023	5.7	9.1	11	15	516	817	1,123	85
Feb 2023	3.9	4.6	11	17	365	542	635	89
Mar 2023	5	8.6	13	29	540	1,353	1,708	82
Apr 2023	6.9	11.6	14	34	981	2,326	2,633	75
May 2023	3.4	4.7	4	7	132	235	236	96
Jun 2023	2.1	2.4	3	4	49	68	68	99
Jul 2023	1.5	2.4	**102**	**394**	**1,540**	**5,990**	**11,781**	**57**
Aug 2023	1.3	1.4	3	4	30	43	53	99
Sep 2023	1.4	1.9	4	8	48	97	115	97
Oct 2023	2.1	3.3	10	12	184	238	330	92
Nov 2023	3.8	7.1	12	13	402	617	760	89
Dec 2023	6.7	12	10	12	649	970	1,196	83
Jan 2024	8.8	15.2	18	30	1,443	2,447	3,466	**68**

Month	Influent Flow, MGD		Effluent Concentration,		Effluent Mass, lbs/day			Percent Removal
	Avg.	Max Day	Avg.	Max Week	Avg.	Max Week	Max Day	Avg.
Feb 2024	6.4	8.4	8	19	440	1,936	708	89
Mar 2024	6.8	13.4	6	8	360	598	673	91
Apr 2024	4.0	5.3	4	5	130	177	205	99
May 2024	4.9	10.9	5	6	199	346	356	98
Jun 2024	2.7	4.6	4	6	103	178	231	99
Jul 2024	1.8	2.1	2	3	30	51	66	100
Aug 2024	1.7	2.3	6	14	92	246	261	99
Sep 2024	1.7	2.1	3	3	37	42	44	99
Oct 2024	2.2	4.2	3	3	55	60	117	99
Nov 2024	6.2	9.8	3	4	174	263	277	96
Dec 2024	6.7	14.3	4	5	228	303	735	98

A monthly summary of effluent bacteria, ammonia, and chlorine residual is presented in Table 4-3. As shown in the table, the WWTP exceeded effluent standards for chlorine residual in both May and July of 2023. There was one exceedance of effluent limits for bacteria that happened in July 2023. There were no reported exceedances for ammonia in 2023 or 2024.

Table 4-3: WWTP Effluent E. coli, Ammonia, and Chlorine (2023 – 2024)

Month	Influent Flow, MGD		*E. coli* Bacteria, MPN		Ammonia, mg/L		Chlorine Residual, mg/L	
	Avg.	Max Day	Avg.	Max Sample	Avg.	Max Day	Avg.	Max Day
Jan 2023	5.7	9.1	3.5	12	0.02	0.04	0.00	0.01
Feb 2023	3.9	4.6	6.1	12	0.07	0.09	0.00	0.01
Mar 2023	5.0	8.6	2.5	8	0.05	0.08	**0.02**	**0.51**
Apr 2023	6.9	11.6	2.5	5	0.18	0.71	0.00	0.01
May 2023	3.4	4.7	2.3	7	0.11	0.21	0.01	0.02
Jun 2023	2.1	2.4	5.6	16	0.13	0.39	0.01	0.02
Jul 2023	1.5	2.4	8.2	**1,987**	0.29	0.85	**0.02**	**0.36**
Aug 2023	1.3	1.4	2.3	5	0.13	0.22	0.00	0.02
Sep 2023	1.4	1.9	1.1	2	0.26	0.97	0.00	0.01
Oct 2023	2.1	3.3	1.1	1	0.54	1.02	0.01	0.02
Nov 2023	3.8	7.1	17.2	93	0.35	0.55	0.00	0.01

Month	Influent Flow, MGD		*E. coli* Bacteria, MPN		Ammonia, mg/L		Chlorine Residual, mg/L	
	Avg.	Max Day	Avg.	Max Sample	Avg.	Max Day	Avg.	Max Day
Dec 2023	6.7	12.0	3.0	22	0.33	0.62	0.00	0.01
Jan 2024	8.8	15.2	5.7	24	0.41	0.56	0.00	0.01
Feb 2024	6.4	8.4	3.2	11	0.41	1.69	0.00	0.00
Mar 2024	6.8	13.4	2.1	5	0.09	0.20	0.00	0.02
Apr 2024	4.0	5.3	1.3	3	0.08	0.21	0.00	0.01
May 2024	4.9	10.9	1.9	4	0.13	0.61	0.01	0.02
Jun 2024	2.7	4.6	1.9	3	0.05	0.06	0.01	0.01
Jul 2024	1.8	2.1	1.9	6	0.08	0.20	0.00	0.01
Aug 2024	1.7	2.3	3.1	24	0.41	1.81	0.00	0.02
Sep 2024	1.7	2.1	1.5	4	0.07	0.11	0.00	0.00
Oct 2024	2.2	4.2	1.4	3	0.21	0.50	0.00	0.02
Nov 2024	6.2	9.8	1.4	3	0.16	0.27	0.00	0.00
Dec 2024	6.7	14.3	1.3	8	0.34	1.11	0.00	0.01

Although the WWTP is generally operating well and producing high quality effluent, there have been permit violations in the last several years. A summary of NPDES permit violations are presented in Table 4-4 spanning years 2020 through 2024. The City assumed direct operation of the WWTP in 2020. In addition to the permit violations for effluent quality presented in the table below, there have been 19 violations related to quality assurance failures since 2020 and 11 months during which an insufficient number of samples were collected. There have been two overflow events, one in April 2022 and the second in July 2023.

Table 4-4: NPDES Permit Violations (2020 - 2024)

Date	Parameter	Permit Limitation	Reported Value
July 10, 2023	Daily $CBOD_5$ load	500 lbs/day	1,108 lbs/day
July 9-15, 2023	Weekly $CBOD_5$ load	380 lbs/day	599 lbs/day
July 9-15, 2023	Weekly $CBOD_5$ concentration	15 mg/L	39 mg/L
July 2023	Monthly $CBOD_5$ concentration	10 mg/L	13 mg/L
May 8-14, 2022	Weekly TSS load	380 lbs/day	401 lbs/day
May 15-21, 2022	Weekly TSS load	380 lbs/day	453 lbs/day
May 2022	Monthly TSS load	250 lbs/day	333 lbs/day
June 12-18, 2022	Weekly TSS load	380 lbs/day	404 lbs/day
June 2022	Monthly TSS load	250 lbs/day	341 lbs/day
Dec 25-31, 2022	Weekly TSS load	2,700 lbs/day	3,388 lbs/day
Dec 27, 2022	Daily TSS load	3,600 lbs/day	14,305 lbs/day
Dec 25-31, 2022	Weekly TSS concentration	45 mg/L	66.3 mg/L
December, 2022	Monthly TSS concentration	30 mg/L	45 mg/L

Date	Parameter	Permit Limitation	Reported Value
July 10, 2023	Daily TSS load	500 lbs/day	11,781 lbs/day
July 9-15, 2023	Weekly TSS load	2,700 lbs/day	3,388 lbs/day
July 2023	Monthly TSS load	250 lbs/day	1,540 lbs/day
July 9-15, 2023	Weekly TSS concentration	15 mg/L	394 mg/L
July 2023	Monthly TSS concentration	10 mg/L	102 mg/L
December 2022	TSS removal efficiency	75%	65%
July 2023	TSS removal efficiency	85%	57%
January 2024	TSS removal efficiency	75%	68%
March 28, 2023	Total chlorine residual	0.01 mg/L	0.51 mg/L
July 31, 2023	Total chlorine residual	0.01 mg/L	0.36 mg/L
April 19, 2021	*E. coli* concentration	126 org./ 100 ml	770 org./ 100 ml
July 10, 2023	Overflows and discharge of sewage to waters of the State	Prohibited	1 MGal untreated wastewater discharged

In 2020, the City assumed control and operation of the WWTP with its own staff. The WWTP had previously been managed and operated by a third-party contract operations company. Since assuming the responsibility for WWTP operations, the City has faced staffing challenges including significant staff turnover and an industry wide shortage of qualified treatment plant operators. In early 2024, the City hired new WWTP supervisory staff. Operations have stabilized and improved markedly since then.

4.2 Need for Liquid Stream Treatment Process Improvements

Hydraulic and biological treatment capacity analyses were conducted to identify potential deficiencies within the existing liquid stream treatment process under projected design year 2045 flow and load conditions. The analysis assumed the DEQ will implement the anticipated effluent permit requirements presented in Section 2 of this Plan.

4.2.1 Influent Pumping

WWTP influent pumping facilities include the Raw Sewage Pump Station and the West Side Interceptor Pump Station (WIPS).

The Raw Sewage Pump Station, constructed in 1975, is a wet well/dry well type station. The station consists of four solids handling centrifugal pumps with an original design capacity of 28 MGD. Although meant to serve as backup for the WIPS during peak flow events, WWTP staff report that the station is no longer operational and has been removed from service.

The WIPS is a submersible-type pump station designed and constructed to accommodate up to five, 150-hp, submersible pumps with a firm capacity (capacity with the largest pump out of service) of 37 MGD. The WIPS, constructed in 2003, is in good structural condition and currently contains three large pumps (85 hp and 75 hp) and one smaller (25 hp) jockey pump. The firm capacity of the pump station is 16 MGD.

The DEQ requires that wastewater pump stations have a firm capacity equivalent to the peak instantaneous flow (PIF). Current (year 2021) PIF for the WWTP is 29.2 MGD. Design year 2045 PIF is 31.1 MGD. Because the Raw Sewage Pump Station is no longer operable and the

firm capacity of the WIPS is 16 MGD, the WWTP does not have sufficient influent pumping capacity and requires improvement. A summary of the influent pumping capacity evaluation is presented in Table 4-5.

Table 4-5: Influent Pumping Capacity Evaluation

Parameter	Value
Basis	PIF[(a)]
Design Criteria	Firm capacity[(b)]
Required Capacity, MGD	
Current (2021)	29.2
Design Year (2045)	31.1
Existing Capacity[(c)], MGD	16

Notes:

(a) Peak instantaneous flow (PIF). The peak hour flow occurring during the peak day flow.
(b) Pumping station capacity with the largest pump out of service.
(c) Firm capacity of the WIPS only. The Raw Sewage Pump Station is inoperable.

4.2.2 Preliminary Treatment

WWTP preliminary treatment facilities (i.e., headworks) were constructed in 1976 as part of a larger structure that included the headworks, aeration basins, storm flow diversion facility, and chlorine contact basins (CCBs). Originally containing a single comminutor and parallel manual bar screen, the headworks currently consists of two mechanical bar screens and a parallel manual bar screen. The first mechanical screen was installed in 1999 with the second being installed in 2021. There are no grit removal facilities.

The DEQ requires that headworks be sized for the PIF with a minimum of two screens. For facilities with a single mechanical screen, the second screen may be a mechanical bar screen. Grit removal facilities, if provided, do not require redundancy.

Hydraulic modeling showed the existing headworks can convey the projected design year 2045 PIF while maintaining approximately 2 feet of freeboard between the water surface and the top of the headworks walls. Modeling assumed the two mechanical screens were running and were 30% plugged or blinded with screenings.

While the existing headworks can convey the design year PIF without overtopping, modeling showed that velocity through the screens is greater than 7 feet per second (fps). Typical design practice limits velocity through mechanical bar screens to less than 4 fps to reduce the potential for material to be swept through the screens. The high flow velocity through the screens is caused by a lack of downstream flow control with resulting minimal water depth in the channel immediately after the screens. Typical design practice is to provide some form of downstream flow control (e.g., weirs or flumes) to modulate the water depth downstream of the screens such that velocity through the bar screens is kept within a range of 1 to 4 fps.

Although capable of conveying the design year PIF, the existing headworks structure is nearly 50-years old, contains aging equipment, and lacks downstream flow control to limit velocity through the screens which reduces the performance of the screens. The absence of grit removal increases wear on downstream equipment, increases the amount of sludge that must

be treated and handled, and adds additional maintenance from the periodic removal of grit from downstream processes (e.g., aeration basins).

4.2.3 Secondary Treatment

The WWTP's existing secondary treatment process consists of an activated sludge treatment system and four secondary clarifiers.

4.2.3.1 Activated Sludge Treatment Process

The existing activated sludge process was constructed in 1976 and is part of a larger common wall structure that includes the headworks, two parallel aeration basins, and the CCBs. Improvements to the activated sludge process were made in 2008 which included the addition of unaerated selector zones with vertical mixers, new variable speed surface aerators, and modifications needed to allow the basins to operate in different modes including series, parallel, step feed, and sludge reaeration. Adding other operating modes (e.g., step feed and sludge reaeration) has allowed WWTP staff to respond to high flow events to help prevent washing out the aeration basins and overloading the secondary clarifiers.

The capacity of the existing aeration basins was evaluated using process modeling software and deficiencies were identified.

4.2.3.1.1 Process Model

A process model was developed to simulate the existing WWTP using the BioWin wastewater process simulator by EnviroSim Associates Ltd. The schematic developed for the model is presented on Figure 4-1 and included the two aeration basins, four secondary clarifiers, the solids treatment stream consisting of thickening and aerobic digestion, RAS and WAS systems, and other return streams.

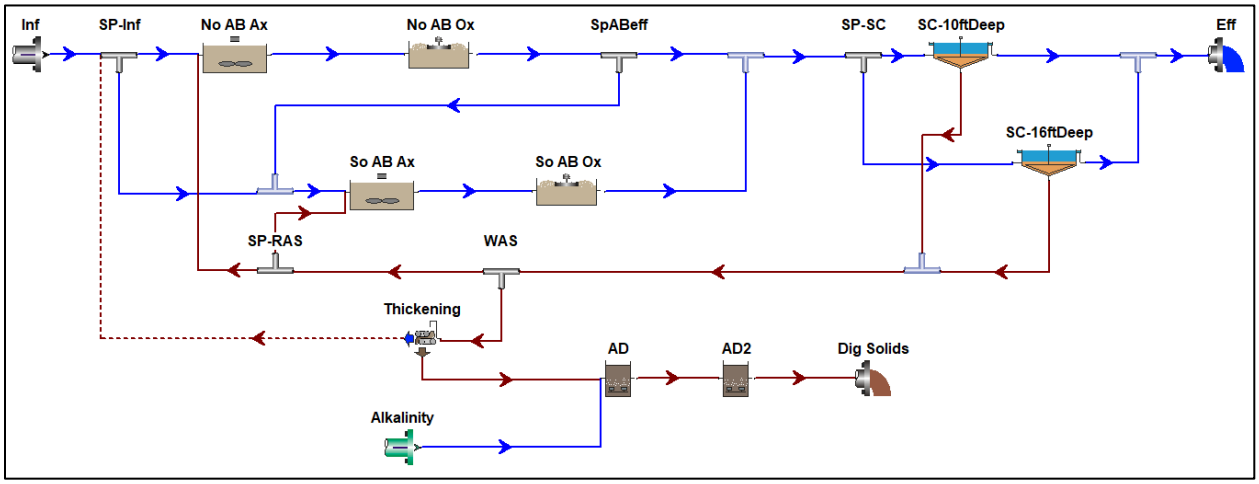


Figure 4-1: Process Simulator Layout

To reduce predictive error, the process model was calibrated and validated using historical operation and performance data and influent characterization. The goal of model calibration is to minimize the discrepancy between simulator predictions and a chosen dataset while altering model input, kinetic, and stoichiometric parameters as little as possible. Calibration is deemed

satisfactory when simulation results are within 20% of actual values from two distinct datasets under steady-state operating conditions. Calibration of the process model of the existing WWTP was completed based on average conditions observed during two periods in July 2020 and March 2021.

4.2.3.1.2 Conditions Assumed for Capacity Assessment

Influent wastewater mass loading assumed for the capacity assessment are as presented in Section 1 of this Plan and summarized in Table 4-6 below. Maximum month loading is used to determine the capacity of the aeration basins. Maximum day loading is used to determine the capacity of the aeration system to deliver sufficient air.

Table 4-6: Influent Wastewater Mass Loading

Parameter	Current (2021)	Design Year (2045)
BOD_5, ppd		
Average	3,200	3,800
Max Month	5,200	6,100
Max Day	-	12,200
TSS, ppd		
Average	3,400	4,000
Max Month	6,200	7,300
Max Day	-	14,200
Ammonia-N, ppd		
Average	420	490
Max Month	680	800
Max Day	-	1,600

Additional wastewater characteristics assumed include alkalinity, pH, and temperature, the values for which are presented in Table 4-7.

Table 4-7: Additional Influent Wastewater Characteristics

Parameter	Value
Alkalinity, mg/L	250
pH	7
Temperature, °C	
Average	17
Min Month[(a)]	12
Max Month[(b)]	21

Notes:

(a) Minimum average monthly temperature used to evaluate biological treatment capacity.
(b) Maximum average monthly temperature used to determine aeration capacity needed.

Effluent quality requirements were assumed to match the anticipated effluent permit limits presented in Section 2 of this Plan. Permit limits specifically affected by the secondary treatment are summarized in Table 4-8.

Table 4-8: Effluent Discharge Limits Specific to Secondary Treatment

Parameter	Units	Monthly Average	Weekly Average	Daily Maximum
$CBOD_5$ (May 1 – October 31)	mg/l	10	15	-
	lbs/day	250	380	500
	% removal[(a)]	85	-	-
TSS (May 1 – October 31)	mg/l	10	15	-
	lbs/day	250	380	500
	% removal[(a)]	85	-	-
$CBOD_5$ (November 1 – April 30)	mg/l	25	40	-
	lbs/day	1,500	2,200	3,000
	% removal[(a)]	85	-	-
TSS (November 1 – April 30)	mg/l	30	45	-
	lbs/day	1,800	2,700	3,600
	% removal[(a)]	85	-	-
Ammonia – N	mg/L	Shall not exceed a monthly average of concentration of 0.94 and a daily maximum concentration of 1.89.		

Note:

(a) $CBOD_5$ and TSS removal efficiency shall not be less than 85% monthly average except when monthly average flows exceed 3.0 MGD which shall not be less than 75% monthly average.

Of the permit limits shown, the requirement for year-round nitrification (monthly effluent ammonia limit of 0.94 mg/L and daily maximum of 1.89 mg/L) controls the biological capacity of the existing activated sludge process. This is because nitrifying bacteria, which are responsible for the conversion of ammonia to nitrite and then to nitrate, have a significantly lower growth rate than the bacteria that remove carbonaceous BOD_5. Temperature has a significant effect on nitrifying bacteria dramatically slowing down their activity and rate of ammonia conversion. As such, to meet the strict ammonia limit, the system must maintain a relatively long solids retention time (SRT) and provide adequate air to ensure nearly complete nitrification. The required operating conditions are summarized in Table 4-9 and include SRT, dissolved oxygen (DO) concentration, and mixed liquor suspended solids (MLSS) concentration. The value for minimum required SRT is greater during the wet weather portion of the year (November through April) when temperatures are cooler.

Table 4-9: Secondary Process Operating Conditions

Operating Parameter	Value
Total SRT[(a)], days	
Dry Weather[(b)]	11
Wet Weather[(c)]	13
DO Concentration, mg/L	2[(d)]
MLSS[(e)], mg/L	3,000-4,000

Notes:

(a) SRT required to help ensure reliable nitrification.
(b) Determined at a temperature equal to the average annual value of 17°C.
(c) Determined at the minimum monthly average temperature value of 12°C.
(d) Required DO concentration is 2 mg/L except during maximum day loading conditions when it is allowed to drop to 1 mg/L.
(e) Maximum allowable range for MLSS.

4.2.3.1.3 Activated Sludge Process Capacity Assessment

The capacity of biological treatment processes is expressed in terms of the maximum average monthly mass load that can be treated while maintaining SRT, DO, and MLSS within normal operating ranges (see Table 4-9). Because of the year-round ammonia limit, the capacity of the aeration basins is controlled by the growth rate of nitrifying bacteria which slows as temperatures drop. The capacity assessment must thus consider an appropriately low (i.e., conservative) temperature to ensure the ability to reliably meet the effluent limit for ammonia. Using the minimum anticipated average monthly temperature is appropriate and was used for this capacity analysis.

The scenarios used to determine the capacity of the City's existing WWTP include those presented in Table 4-10.

Table 4-10: Biological Treatment Capacity Assessment Scenarios

Scenario	Description
Max. Month/Avg. Temp	• Maximum average monthly mass load • Two aeration basins in operation • Average annual temperature
Max. Month/Min. Temp	• Maximum average monthly mass load • Two aeration basins in operation • Minimum average monthly temperature

Results of the process model simulation and values of the estimated treatment capacity of the existing aeration basins in terms of BOD_5 loading are presented in Table 4-11.

Table 4-11: Biological Treatment Capacity of Existing Aeration Basins

Item	Value
Max. Month/Avg Temp	
Temperature, °C	17
BOD_5 Loading Capacity, ppd	3,400
Max. Month/Min. Temp	
Temperature, °C	12
BOD_5 Loading Capacity, ppd	2,900

Notes:

(a) Required DO concentration is 2 mg/L except during maximum day loading conditions when it is allowed to drop to 1 mg/L.
(b) Maximum allowable MLSS.

The process simulation results show the effect of temperature on the capacity of the aeration basins with greater capacity at higher temperatures. Comparing these values with the current and projected design year 2045 loading shows that while the existing aeration basins have enough capacity to meet the current average annual loading condition, they are approximately 40% to 50% short of that needed to meet maximum month conditions. The existing aeration basins are thus essentially 50% short of the volume needed to contain the amount of biomass required to treat the project design condition. A summary of this comparison and the deficiency in capacity are presented in Table 4-12.

Table 4-12: Aeration Basin Treatment Capacity Deficiency

Condition	Current (2021)			Design Year(2045)		
	Loading (ppd)	Capacity (ppd)	Deficiency (%)	Loading (ppd)	Capacity (ppd)	Deficiency (%)
Avg. Annual BOD_5 at 17°C	3,200	3,400	NA[(a)]	3,800	3,400	11%
Max. Month BOD_5 at 12°C	5,200	2,900	44%	6,100	2,900	52%

Note:

(a) Maximum capacity of the existing aeration basins (3,400 ppd at 17°C) is greater than projected load. Therefore, there is no deficiency under this loading condition.

While the existing WWTP has successfully met effluent discharge requirements for $CBOD_5$, TSS and ammonia and should continue to do so under normal operating and loading conditions, it does not have sufficient capacity to reliably treat the anticipated maximum month load under appropriately conservative temperature conditions. The City thus faces the risk of permit violation most specifically for ammonia since that is the controlling biological process.

4.2.3.1.4 Aeration Capacity Assessment

Each of the two existing aeration basins contains two 75-hp, surface-type aerators. In addition to assessing the overall aeration basin capacity and volume needed to treat maximum month loading conditions, the capacity of the existing aeration system was evaluated and compared to peak demand conditions. Aeration systems should be capable of providing sufficient air to maintain DO levels at 2 mg/L during maximum month mass loading and 1 mg/L during peak day mass loading. Because additional aeration air is needed as the temperature rises, aeration needs should be assessed at the maximum average month temperature. Results of the capacity assessment assuming normal design values for oxygen transfer efficiencies for surface aerators are summarized in Table 4-13.

Table 4-13: Aeration Capacity Assessment

Loading Condition	DO[(a)] (mg/L)	Current (2021) DO Required (lbs/hr)	Current (2021) Capacity (lbs/hr)	Current (2021) Deficiency (%)	Design Year (2045) DO Required (lbs/hr)	Design Year (2045) Capacity (lbs/hr)	Design Year (2045) Deficiency (%)
Max. Month at 21°C[(b)]	2	610	600	2%	730	600	18%
Max. Day at 21°C[(b)]	1	NA[(c)]	NA[(c)]	NA[(c)]	1,290	600	54%

Notes:

(a) Value shows is the target dissolved oxygen concentration within the aeration basins. Under normal operating conditions, the aeration system should be capable of maintaining the DO at 2 mg/L. During peak day loading conditions, normal design practice is a target value of 1 mg/L.

(b) Estimated maximum average month temperature.
(c) Maximum day aeration capacity required calculated only for the design year 2045 condition.

As shown in Table 4-13, the existing surface aerators can essentially meet current maximum month loading and temperature condition with an estimated deficit of only 2%. However, under design year 2045 conditions, the existing aerators are approximately 20% short of the capacity needed for maximum month conditions and a little more than 50% of that needed for maximum day conditions even at a reduced DO concentration of 1 mg/L.

4.2.3.2 Secondary Clarifiers

The WWTP has four secondary clarifiers. Clarifier 1 was originally a 60-ft-diameter, primary clarifier that was part of the original facility constructed in 1954. It was converted to a secondary clarifier during the 1976 upgrades project at which time two 55-ft clarifiers (Clarifier 2 and 3) were constructed. Clarifiers 1, 2, and 3 are relatively shallow with sidewall depths of 10 ft. Each has a scraper-type sludge removal mechanism. Clarifier 4, constructed in 2012, has a diameter of 130 ft and a sidewall depth of 16 ft. It has a suction-type sludge removal mechanism with two suction arms.

A state point analysis was conducted to estimate the capacity of the existing secondary clarifiers and compare it with the current and design year 2045 flow conditions. State point analysis provides a means of estimating clarifier capacity based on typical settling characteristics of mixed liquor adjusted for differences in the specific sludge settleability of a specific biological process. This is normally done by adjusting the settling parameters based on the sludge volume index (SVI). The SVI, which is specifically defined as the volume of 1 gram (g) of mixed liquor after settling for 30 minutes, provides a relative measure of the settleability of the activated sludge. Better settling sludges have lower SVI values. For example, well operating activated sludge processes typically produce mixed liquor with SVI values of less than 100 mL/g. Higher values can be indicative of sludge settleability issues such as filamentous growth. The existing activated sludge system has produced SVI values typically ranging from 70 to 110 mL/g.

A summary of the state point analysis results showing the estimated capacity of the existing clarifiers is presented in Table 4-14. Values are shown in terms of maximum surface overflow rate (SOR) compared with those associated with current and design year 2045 design conditions. Scenarios evaluated include the MMDWF and PDF flow conditions. The DEQ requires secondary clarifiers to have enough capacity to treat either the MMDWF condition with the largest clarifier out of service or the PDF condition with all clarifiers in service, whichever results in the largest required capacity.

Table 4-14: Existing Secondary Clarifier Capacity Assessment

				Predicted Conditions			
		Existing Capacity[(a)]		Current (2021)		Design Year (2045)	
Flow Condition	Clarifiers in Service	Flow Rate (MGD)	SOR (gpd/sf)[(b)]	Flow Rate (MGD)	SOR (gpd/sf)	Flow Rate (MGD)	SOR (gpd/sf)
MMDWF	3	8.3	1,100	5.4	710	6.0	790
PDF	4	23	1,100	18.1	870	19.4	930

Notes:

(a) Capacity of existing clarifiers calculated assuming an SVI equal to 120 mL/g and MLSS concentration of 3,500 mg/L.
(b) gpd/sf = gallons per day per square foot.

Although the state point analysis suggests that the capacity of the existing secondary clarifiers is sufficient to meet both current and future year 2045 design conditions, it is solely based on the available surface area of the existing clarifiers. It does not consider the age of the equipment, side water depth, and overall condition of the structures themselves.

Existing Clarifiers 1, 2, and 3 are shallower than current design practice dictates with side water depths of only 10 feet. Providing additional water depth per current industry standard provides additional operational flexibility. It provides the ability to operate with a deeper sludge blanket allowing the clarifiers to better absorb changing flow and/or sludge quality changes without negatively impacting clarifier performance. Providing additional distance between the sludge blanket and the overflow weir also typically results in better effluent quality.

WWTP staff report poor performance, failing equipment, and difficulties with flow distribution with the three older clarifiers. Because of these concerns, staff minimize the use of Clarifiers 1, 2, and 3 running them only when needed during high flow months.

4.2.4 Filtration

Existing tertiary filtration facilities were constructed as part of the 1976 upgrades project. They consisted of two pressure type filter vessels, filter feed pumps, backwash pump, chemical feed systems, and the associated Filter Building. Although designed to operate between June 1 and October 31 to reduce effluent suspended solids, the filtration system has not been used for more than 20 years and was officially removed from service in 2020.

The anticipated effluent limits presented in Section 2 of this Plan include monthly average dry weather (May 1 through October 31) concentration limits for $CBOD_5$ and TSS of 10 mg/L. Monthly wet weather (November 1 through April 30) concentration limits for $CBOD_5$ and TSS are anticipated to be 25 mg/L and 30 mg/L, respectively. However, under specific flow conditions (e.g., MMDWF and MMWWF), the anticipated mass load limits and percent removal requirements can result in the need to produce effluent with concentrations well below the specified limits with values as low as 5 mg/L.

Conventional secondary treatment process can be relied upon to consistently meet effluent $CBOD_5$ and TSS concentrations down to 10 mg/L. However, it is standard practice to provide tertiary filtration to ensure reliable treatment when effluent limits are lower than this.

4.2.5 Secondary Effluent Pumping

The Effluent Pump Station was originally constructed in 1976. The pump station was upgraded in 2015 and currently contains four variable speed, submersible pumps. They include two larger capacity 100-hp pumps and two smaller capacity 20-hp pumps. While the firm capacity of the existing pump station is approximately 16.3 MGD, design of the 2015 upgrade project included provisions for future expansion of the station to provide additional capacity.

The DEQ requires that wastewater pump stations have a firm capacity equivalent to the peak instantaneous flow (PIF). Current year PIF for the WWTP is 29.2 MGD. Design year 2045 PIF is 31.1 MGD. While the existing station has been able to meet flow requirements, it does not meet DEQ requirements for firm capacity and thus requires improvement. A summary of the Effluent Pump Station capacity evaluation is presented in Table 4-15.

Table 4-15: Secondary Effluent Pump Station Capacity Evaluation

Parameter	Value
Basis	PIF[(a)]
Design Criteria	Firm capacity[(b)]
Required Capacity, MGD	
Current (2021)	29.2
Design Year (2045)	31.1
Existing Capacity, MGD	16.3

Notes:

(a) Peak instantaneous flow (PIF) which is the peak hour flow anticipated to occur during the peak day flow.
(b) Pumping station capacity with the largest pump out of service.

4.2.6 Disinfection

Secondary effluent is disinfected within two parallel CCBs. The CCBs were constructed in 1976 as part of a larger common wall structure that also included the headworks and aeration basins. The original gaseous chlorine system was replaced in 1999 with a liquid sodium hypochlorite system.

DEQ design requirements state that CCBs should be sized for at least 60 minutes (min) of contact time at ADWF, 20 min at PDF, and 15 min at PIF, whichever results in the largest basin. In addition, DEQ guidelines state that while a minimum length-to-width ration of 40:1 is required, a ratio of 72:1 is preferred.

The existing CCBs were evaluated and compared to the DEQ design requirements. Results are summarized in Table 4-16. As shown, the CCBs exceed the DEQ required contact time during ADWF conditions. However, they do not provide enough contact time during peak flow conditions being short of the required contact time by almost 30% under the design year 2045 PIF condition.

Table 4-16: Chlorine Contact Basin Contact Time Evaluation

	DEQ	Current (2021)			Design Year (2045)		
Flow Condition	Required Contact Time (min)	Flow Rate (MGD)	Contact Time (min)	Deficiency (%)	Flow Rate (MGD)	Contact Time (min)	Deficiency (%)
ADWF	60	2.3	150	NA[(a)]	2.7	128	NA[(a)]
PDF	20	18.1	19	5%	19.4	18	10%
PIF	15	29.2	12	20%	31.1	11	27%

Note:

(a) Contact time under this flow condition exceeds the DEQ required value. Therefore, there is no deficiency at this flow condition.

The length-to-width ratio of the existing CCBs when operated in series is 46:1 and thus meets the minimum DEQ required value. When operating in parallel, the length-to-width ratio is 23:1 and does not meet the value required by the DEQ for new facilities.

While the existing CCBs have historically been able to meet discharge permit limits for bacteria, contact times during peak flow conditions do not meet DEQ requirements. In addition to the deficient contact time, hydraulic modeling suggests that during design year 2045 peak hour flow there would be limited freeboard (approximately 0.5 ft with the CCBs operating in series and approximately 1 ft if operating in parallel) within the basins. Other deficiencies with the disinfection system include the poor condition of the sodium hypochlorite storage area that is within the aging Control Building structure.

4.2.7 Outfall

Dechlorinated effluent is conveyed to the South Santiam River through a 1,500-foot-long, 42-inch-diameter outfall constructed in 2008. Discharge to the river is via a multi-port diffuser buried beneath the bottom of the riverbed. The diffuser is approximately 24 foot long and consists of eleven, evenly spaced, 12-inch duckbill diffusers attached to 12-inch-diameter HDPE risers.

The City submitted a mixing zone study (December 17, 2007) to the DEQ with dilution values proposed for use in establishing future effluent permit limits. The DEQ reviewed the mixing zone study and issued a memorandum (February 4, 2008) that set dilution values that were to be used during future permit renewals. The ADWF and ADF design flows assumed in the mixing zone analysis were 3 MGD and 5.5 MGD, respectively. Design year 2045 ADWF and ADF values were estimated to be 2.7 MGD and 5.0 MGD, respectively. Because the design year values are lower than those assumed in the analysis, the results of the mixing zone analysis should still be valid. Correspondence with the DEQ during preparation of this Plan suggests that the DEQ will not require an updated mixing zone study as part of the pending permit renewal process.

Hydraulic modeling of the outfall was completed to identify potential hydraulic deficiencies. Conditions were evaluated under the design year 2045 PIF (31.1 MGD) peak flow condition assuming a discharge water surface equal to the South Santiam River's 100-year flood elevation. Results showed sufficient hydraulic capacity with approximately 18 inches of freeboard at the outlet structure in the CCB.

4.3 Need for Solids Stream Treatment Process Improvements

Treatment capacity analyses were conducted to identify potential deficiencies within the existing solids stream treatment processes under current and projected design year 2045 load conditions. Results of those analyses are summarized below.

4.3.1 Sludge Thickening

Sludge thickening facilities were constructed in 1992 and, except for the polymer feed system which was replaced in 2020, are more than 30 years old. Facilities consist of a gravity belt thickener (GBT) with a 2-meter belt, centrifugal screw-type sludge feed pump, sludge grinder, progressive cavity type thickened WAS (TWAS) pump, and polymer feed equipment. Sludge thickening facilities are contained within the 1,400-square-foot Sludge Thickening Building.

Projected solids production values generated by the process model prepared for the existing WWTP's biological treatment process were compared with the capacity of the existing thickening facilities. Maximum week conditions are appropriate for the design of thickening facilities and the capacity of thickening facilities must consider both hydraulic and solids loading requirements. The capacity of the existing thickening facilities was calculated assuming the system operates 8 hours per day, 5 days per week. WAS concentration was assumed to be 7,500 mg/L and TWAS was assumed to be 40,000 mg/L (4%).

Results of the evaluation for the GBT are presented in Table 4-17**.** The existing GBT has a solids loading capacity of 1,600 lbs per day. As shown, this exceeds both current and design year 2045 loading conditions.

Table 4-17: Gravity Belt Thickener Capacity Evaluation

Parameter	Value
Basis	Max Week Loading[(a)]
Capacity Required[(b)]	
Solids Loading, lb/hr[(c)]	
Current (2021)	1,340
Design Year (2045)	1,570
Existing Capacity	
Solids Loading, lb/hr[(c)]	1,600

Notes:

(a) Conditions associated with maximum week solids production for activated sludge process.
(b) Assuming thickening facilities operate 40 hours per week.
(c) Dry pounds.

The hydraulic capacity of the existing sludge feed pump, sludge grinder, and TWAS pump were compared with current and design year 2045 flow rates. Results are presented in Table 4-18 and show that the capacities of the Sludge Feed Pump and Sludge Grinder are either less than or only slightly more than that needed to meet maximum week loading conditions. Capacity of the TWAS pump exceeds that needed under both current and design year 2045 conditions.

Table 4-18: Sludge Pumping Capacity Evaluation

Parameter	Value
Basis	Max Week Loading[(a)]
Capacity Required[(b)]	
Sludge Feed Pumping, gpm	
Current (2021)	360
Design Year (2045)	420
TWAS Pumping, gpm	
Current (2021)	70
Design Year (2045)	80
Existing Capacity, gpm	
Sludge Feed Pump	340
Sludge Grinder	400
TWAS Pump	100

Notes:

(a) Conditions associated with maximum week solids production for activated sludge process.
(b) Assumes thickening facilities operate 40 hours per week, WAS is 7,500 mg/L and TWAS is 4%.

Although the existing GBT has sufficient capacity to meet design year 2045 loading conditions and WWTP staff report it is functioning well, the equipment is more than 30 years old. This is well beyond the 20-year useful life typically assumed for mechanical equipment. Because of this and the capacity limits of the sludge feed pump and grinder, planned improvements to the thickening facilities, while not immediately necessary, are recommended.

4.3.2 Aerobic Digestion

The WWTP's 530,0000-gallon aerobic digester was originally constructed as a trickling filter in 1954. It was converted to its current use as part of the 1976 WWTP upgrades project. Aeration and mixing of the digester contents is provided by three, 20-hp, floating aerator/mixers. Class B biosolids are produced that are liquid land applied to authorized application sites.

Class B biosolids must meet quality standards as contained in 40 Code of Federal Regulations Part 503 (Part 503) and Oregon Administrative Rules (OAR) Chapter 340, Division 50. These standards include requirements for both pathogen reduction and vector attraction reduction (VAR).

Five treatment methods are listed in Part 503 as Processes to Significantly Reduce Pathogens (PSRP) that, if used, meet the Part 503 pathogen reduction requirement for Class B biosolids. Aerobic digestion is a listed PSRP and must meet specific requirements for aerobic detention times that vary with temperature (i.e., detention time must be at least 40 days at 20°C [68°F] or 60 days at 15°C [59°F]).

The capacity of the existing aerobic digester was evaluated assuming a minimum detention time of 60 days as required by Part 503 for aerobic digestion at 15°C (59°F). Assuming an average design operating temperature of 15°C (59°F) is appropriate for uncovered aerobic digesters within the region. A review of effluent discharge temperatures from the WWTP further supports this assumption showing monthly average temperatures typically ranging from 12°C (54°F) to 15°C (59°F) during the winter and spring months of each year.

Results of the capacity analysis expressed in terms of detention time and solids loading to the digester are presented in Table 4-19. Calculations were based on maximum month loading and an average TWAS concentration of 4%. The evaluation showed the existing aerobic digester cannot provide the necessary 60-day detention time under current or design year 2045 maximum month loading conditions and does not meet the Part 503 requirements for use of aerobic digestion as a PSRP.

Table 4-19: Aerobic Digester Capacity Evaluation

Parameter	Value
Basis	Max Month Loading
Design Criteria	Detention Time[(a)]
Detention Time Required, days	60

Parameter	Value
Existing Detention Time, days	
Current (2021)	40
Design Year (2045)	34
Solids Loading Capacity Required, lb/day[(b)]	
Current (2021)	4,400
Design Year (2045)	5,100
Existing Solids Loading Capacity, lb/day[(b)(c)]	3,000

Notes:

(a) Detention time of 60 days is required based on a temperature of 15°C (59°F).
(b) Rounded to the nearest 100 lbs/day.
(c) Assuming TWAS concentration equal to 4%.

Part 503 regulations include ten options for Class B biosolids to meet the VAR requirement. The option most used for aerobic digestion is to achieve a volatile solids reduction (VSR) value of at least 38%. VSR values for properly sized and functioning aerobic digesters typically range from 40% to 50%. Another option to demonstrate VAR compliance is to achieve a specific oxygen uptake rate (SOUR) of less than 1.5 mg O_2/hr per gram of wastewater solids at 20°C (68°F).

WWTP staff have reported the existing aerobic digester cannot reliably achieve the required values for either VSR or SOUR. This is likely due to insufficient digester volume and relatively short detention time as described above and presented in Table 4-19. WWTP staff have had to rely on lime stabilization (another option listed in Part 503) to meet Class B biosolids VAR requirements. This option requires that the pH of the solids being treated be raised by alkali addition to at least 12 and then, without the addition of more alkali, remain at 12 for 2 hours and then at 11.5 or higher for at least an additional 22 hours. WWTP staff do this by combining partially stabilized solids from the aerobic digester with lime slurry in the sludge holding tank. The sludge is held for the required minimum time to meet the biosolids requirements and then hauled to approved sites for liquid land application.

The existing aerobic digester does not have enough volume to meet minimum detention time requirements to meet Part 503 PSRP requirements for aerobic digestion. It provides less than 70% of the detention time needed under current maximum month loading conditions and less than 60% under design year 2045 maximum month loading conditions. The digester is also unable to meet VAR requirements for either VSR or SOUR requiring the City to practice lime stabilization to meet regulatory requirements for Class B biosolids. Because of this and the operational challenges and costs associated with lime treatment, improvements to the solids stabilization facilities are recommended.

4.3.3 Dewatering

Existing dewatering facilities were installed by the City in 2022 inside the pre-engineered metal building originally constructed to house the now decommissioned Cannibal® process RAS screening facilities. Dewatering facilities consist of a 1.7-meter BFP, progressive cavity sludge feed pump, polymer feed system, and dewatered cake screw conveyor. The dewatering system is used when the City cannot liquid land apply Class B biosolids and dewatered cake must be hauled to the Coffin Butte Landfill for disposal.

Capacity of the existing dewatering system was compared with current and design year 2045 maximum month solids production rates. Use of the maximum month condition, while conservative, is appropriate for dewatering facilities because of the detention time provided within the aerobic digester and sludge holding tank. Results of the comparison are presented in Table 4-20. Capacity results were determined assuming a feed sludge concentration of 4% and dewatered cake concentration of 16% to 17%. Total operating time during the maximum month condition was assumed to be 40 hours per week.

Table 4-20: Dewatering Capacity Evaluation

Parameter	Value
Basis	Maximum Month
Capacity Required[(a)]	
Solids Loading, lb/hr[(b)]	
Current 2021	550
Design Year (2045)	650
Hydraulic Loading, gpm[(c)]	
Current 2021	38
Design Year (2045)	45
Existing Capacity	
Belt Filter Press	
Solids Capacity, lb/hr[(b)]	600[(d)]
Hydraulic Capacity, gpm	160
Sludge Feed Pump, gpm	125

Notes:

(a) Assuming dewatering system operates 40 hours per week.
(b) Dry pounds per hour.
(c) Hydraulic loading based on a feed sludge concentration of 4%.
(d) Manufacturer's rated capacity. WWTP staff have reported that they have been unable to achieve this capacity.

As shown in Table 4-20, the manufacturer's rated capacity of the existing dewatering system is 600 dry lbs per hour. This is slightly short of the projected design year 2045 maximum month loading of 650 dry lbs per hour. However, the design year 2045 solids loading value was calculated assuming that dewatering is limited to 40 hours per week. Running the dewatering equipment approximately 4 additional hours would make up for the slight short fall.

The existing dewatering system should have sufficient capacity to meet the projected design year 2045 maximum month condition. Additional capacity improvements should not be needed. However, WWTP staff have reported difficulties in achieving consistent dewatering results and throughput and have questioned the ability of the equipment to achieve the manufacturer's stated capacity. The City is continuing to improve the operation and performance of the BFP. Should these efforts fail, improvements may be needed to the dewatering facilities to meet future loading conditions.

4.3.4 Land Application

The City land applies aerobically (partially) digested, lime stabilized Class B biosolids to DEQ authorized application sites. Although the City has five authorized sites with a combined total

area of approximately 370 acres, only two sites are currently available for use. One of these sites (Sommers) has a total area of 108 acres and was originally authorized for year-round biosolids application (weather dependent). The Sommers site is now only authorized for dry season application and has a usable area of approximately 60 acres. The second site (Morgan) is authorized for dry season application and has a total area of 68 acres of which approximately 40 acres is usable.

The capacity of the existing liquid Class B biosolids land application program was evaluated and compared with total current and design year 2045 biosolids production values. Values for application area and storage volume were determined assuming that all Class B biosolids generated by the WWTP would be land applied. Results of the capacity evaluation are presented in Table 4-21.

The total area of the City's five authorized application sites (370 acres) exceeds that needed to meet design year 2045 biosolids generation rates. However, the City currently only has access to about 100 acres for summertime application. This is 20% short of that needed for current conditions and 40% short of that needed to meet design year 2045 conditions. Lack of liquid sludge storage also limits the City's ability to land apply Class B biosolids. The volume of the existing sludge storage tank is 265,000 gallons. This is equivalent to approximately 45 days of solids storage under current conditions and less than 40 days under design year 2045 conditions. This is about 25% of the volume needed to store solids for the 6-month period each year during which the City cannot land apply Class B biosolids.

Table 4-21: Land Application Capacity Evaluation

Parameter	Value
Basis	Average Annual
Solids Generation, DT/yr[(a)]	
Current 2021	350
Design Year (2045)	420
Sludge Storage Duration, months	6
Area Required for Land Application, acre[(b)]	
Current 2021	120
Design Year (2045)	140
Volume Required for Solids Storage, MGal[(c)]	
Current 2021	0.88
Design Year (2045)	1.04
Existing Land Application Area, acre	
Total Authorized	370
Total Current Usable	100
Existing Liquid Sludge Storage	
Volume, MGal	0.265
Days of Storage, days	
Current 2021	45
Design Year (2045)	38

Notes:

(a) Dry tons per year.
(b) Area requirement based on biosolids application rate of 3 DT/yr.
(c) Concentration of stored liquid biosolids concentration assumed to be 4%.

The City's ability to liquid land apply Class B biosolids is significantly limited by a lack of usable application area and to a greater extent the lack of solids storage. Improvements to the City's Class B liquid land application program are needed to eliminate the need for disposal of excess biosolids at the Coffin Butte Landfill. Landfilling is costly and is expected to become increasingly more so as the landfill reaches capacity and becomes less willing to accept municipal wastewater solids.

4.4 General Wastewater Treatment Plant Facilities

General WWTP facilities that require improvements include the existing Control Building and the WWTP's electrical infrastructure including motor control centers (MCCs), transformers, panel boards, switch gear, and standby generator.

4.4.1 Control Building

The WWTP's Control Building was originally constructed in 1954 and later modified in 1976 in conjunction with the conversion of the adjoining aerobic digester to a sludge holding tank. The building includes laboratory facilities, mechanical room, sodium hypochlorite storage, and an electrical room. Total building space is about 2,700 square feet.

Improvements to the Control Building are recommended. The structure is more than 70 years old, has suffered from overflows of the adjoining sludge holding tank that have damaged interior working areas, has limited office and laboratory space, and is in generally poor overall condition.

4.4.2 Electrical and Standby Power

Electrical facilities including MCCs, transformers, panel boards, switch gear, and standby generator are in multiple buildings throughout the WWTP. The age of electrical equipment ranges from 1976 for equipment installed as part of the 1976 WWTP upgrades project to that of the more recently installed dewatering equipment in 2022. The existing 1,250-kW standby generator was installed in 2003 as part of the WIPS project.

Because of the age, condition, and general obsolescence of much of the existing electrical equipment, improvements are recommended and will be needed as improvements to the liquid and solids treatment systems are implemented. The existing 1,250-kW standby generator is generally in good condition. However, it will not provide sufficient power to support the increased power demand associated with anticipated improvements to the WWTP. Replacement of the standby generator is recommended and should be done as power demand approaches that of the existing generator.

4.5 Summary of Deficiencies

A summary of deficiencies identified within the liquid stream treatment process, solids stream treatment process, and general WWTP facilities are provided in Table 4-22. Subsequent sections of this Plan describe the alternatives considered to address the identified deficiencies, the selection process to identify the preferred alternative, estimated costs, and recommendations for phasing.

Table 4-22: WWTP Deficiencies Summary

Process Area or Facility	Deficiencies
Influent Pumping	• Firm capacity of the WIPS at 16 MGD does not meet peak flow conditions • Design year 2045 PIF is 31.1 MGD • Raw Sewage Station is inoperable and has been removed from service
Headworks	• Aging structure that was constructed as part of the 1976 WWTP upgrades project • Lack of downstream flow control that reduces screening efficiency • No grit removal
Secondary Treatment	• Insufficient treatment capacity • Aeration basin volume is approximately 50% of that needed to treat design year 2045 maximum month loading conditions • Aeration system capacity is approximately 80% of that needed during design year 2045 maximum month loading and less than 50% of that needed for the maximum day condition
Secondary Clarification (Clarifiers 1 through 3)	• Aging structures and equipment that was originally installed in 1976 and well beyond the normal 20-year design life for mechanical equipment • Poor settling performance • Uneven flow distribution • Insufficient depth
Filtration	• Tertiary filtration will be needed to reliably meet effluent permit conditions for $CBOD_5$ and TSS under specific flow conditions • Existing filtration facilities are inoperable and have been removed from service
Secondary Effluent Pumping	• Firm capacity of 16.3 MGD does not meet peak flow conditions • Design year 2045 PIF is 31.1 MGD
Disinfection	• Existing CCBs do not meet DEQ requirements for contact time during peak flow conditions (approximately 70% of that required under design year 2045 PIF) • Limited freeboard during design year 2045 PIF flow condition • Poor physical condition of sodium hypochlorite storage and dosing facilities

Process Area or Facility	Deficiencies
Sludge Thickening	• Aging equipment (more than 30 years old)
Aerobic Digestion	• Insufficient volume with inability to meet Part 503 detention time requirements for aerobic digestion (less than 70% of that needed for current conditions and less than 60% of that needed for design year 2045 conditions) • Aerobic digester was originally constructed in 1954 and is more than 70 years old • Aerator/mixers were originally installed in 1976 and well beyond the normal 20-year design life for mechanical equipment
Biosolids Dewatering	• Inconsistent dewatering results • WWTP staff concerned the BFP cannot achieve its rated capacity
Class B Biosolids Land Application	• DEQ authorized sites that are currently available for use do not provide sufficient area to meet annual biosolids production (approximately 80% of that needed for current conditions and 70% of that needed for design year 2045) • Biosolids storage is insufficient for wintertime storage of biosolids (approximately 30% of that needed for current conditions and 25% of that needed for design year 2045)
Control Building	• Aging structure in poor condition that was originally constructed in 1954 and thus more than 70 years old • Insufficient laboratory and working space
Electrical Infrastructure	• Much of the existing MCCs, transformers, and panelboards were installed as part of the 1976 upgrades project, are in poor condition, and obsolete • Existing 1,250-kW standby generator will not have sufficient capacity to provide backup power for design year 2045

Section 5: Wastewater Treatment Alternatives Considered

This section provides a summary of alternatives considered for improvements to the City's WWTP. This work included initial screening analyses to eliminate alternatives that were not deemed to be viable.

5.1 Alternatives Screening Evaluation Criteria

A matrix-based screening analysis of potential alternatives was performed for most of the WWTP's major process areas including those associated with liquid and solids treatment processes. The scoring matrixes developed were used in consultation with City staff to reduce the number of alternatives that would receive further evaluation to those believed to be most viable and applicable in addressing WWTP deficiencies. The screening analysis was done to increase the efficiency of the planning process while ensuring the range of potential alternatives theoretically available to the City for consideration were documented.

Evaluation criteria including both economic and non-economic factors that were considered in the screening analysis are summarized below. Each criterion was weighted in terms of relative importance towards an alternatives overall score. Weighting of the criteria differed slightly between evaluations of liquid stream and solids stream treatment alternatives.

- **Capital Cost.** A qualitative assessment of the construction cost of each alternative relative to other alternatives being considered. Lower cost alternatives receive a higher score.

- **Operations and Maintenance Cost.** A qualitative assessment of the anticipated operation and maintenance cost of each alternative relative to other alternatives being considered. Lower cost alternatives receive a higher score.

- **Technology Maturity.** Provides an assessment of the relative maturity of the technology associated with an alternative. Processes with few full-scale installations would receive a lower relative score than those of alternatives with many full-scale installations.

- **Operational Complexity and Staffing Demands.** Scores alternatives based on their relative complexity and anticipated staff effort needed. Process alternatives with large numbers of equipment sub-systems or those that require significant amount of operator intervention would receive a lower relative score than simpler systems with less need for operator attention.

- **Ease of Permitting.** Assesses alternatives based on the anticipated permitting process. Considerations may include perceived acceptance by DEQ, land use permitting, and environmental permitting.

- **Potential for Community Objections.** Scores alternatives based on potential negative impacts to the community. Considerations may include traffic impacts, potential for objectionable odors, noise, and visual impacts. Alternatives with less potential for negative impacts to the community receive a higher score that those with greater potential for negative impact.

- **Beneficial Use of End Product.** Assesses an alternative's ability to promote beneficial use of recycled water or biosolids (e.g., land application of Class B biosolids, unrestricted use of Class A biosolids, etc.). Criteria may not be applicable to all process alternatives.

Alternatives were scored for each criterion on a scale of one (1) to five (5), with 1 being lowest (worst) and 5 being highest (best). The weighting factor is a percentage-based multiplier that places greater emphasis on specific criteria deemed to be of higher value.

5.2 Liquid Stream Treatment Alternatives

The following provides a description of alternatives for improving the WWTP's liquid stream treatment processes and those selected for further evaluation.

5.2.1 Influent Pumping

The City currently receives raw sewage from the westside and eastside interceptors that converge at the southwest side of the WWTP. There are two existing pumping stations onsite. The first (not currently in use) is an older, dry-pit/wet-pit type pumping station (Raw Sewage Pumping Station). The second (currently in use) is a self-cleaning submersible-type pump station (West Side Interceptor Pumping Station [WIPS]). The current firm capacity of the WIPS is 12 MGD with the largest pump out of service. The Raw Sewage Pumping Station, constructed in 1974 and historically used to convey inflows which exceeded the capacity of the WIPS, is currently inoperable. The City's current influent pumping capabilities are not sufficient to meet current (or future design year 2045) peak flow conditions. Two alternatives were considered to increase pumping capacity.

5.2.1.1 Raw Sewage Pumping Station

The existing Raw Sewage Pumping Station consists of two square-shaped interconnected wet wells separated from four centrifugal pumps which sit in an adjacent dry well. It contains three variable speed pumps and one constant speed pump. Historically, the pumping station was used to manage influent flows that exceeded the WIPS capacity. When the City assumed control of the WWTP in 2020, the Raw Sewage Pumping Station was found to be inoperable. Given the age and condition of the pump station, this alternative was eliminated from further consideration.

5.2.1.2 West Side Interceptor Pumping Station

The WIPS was constructed as a modern self-cleaning, submersible-type wet well pumping station designed to be expandable to meet future higher flows. The station contains one 75-horsepower (hp) pump, two 85-hp pumps, and one 25-hp pump with an empty space for an additional pump. Firm capacity of the station is 12 MGD.

This alternative would replace and upsize the pumps so that the station would contain four 135 hp high flow pumps and one smaller 50 hp pump to accommodate periods of low flow. Firm capacity of the pumping station would meet the projected year 2045 peak flow condition of 31.1 MGD. Other modifications would include piping modifications and electrical and controls upgrades to accommodate the greater power demand of the new pumps.

5.2.2 Preliminary Treatment

Preliminary treatment provided in the headworks of treatment plants provide mechanical removal of large solids and debris from the raw wastewater. This reduces solids loading on downstream processes, lessens the potential for clogging of pumps, reduces wear on equipment caused by abrasive particles, and increases the visual quality of the plant's digested sludge by removing recognizable solids from the waste stream.

Preliminary treatment typically consists of two removal processes. The first is screening which removes large debris from the liquid stream. Screen openings range in size depending on the needs of downstream processes and can be manually or mechanically cleaned. The second removal process is grit removal which is settling inorganic grit in chambers of various designs and removing it for offsite disposal.

Alternatives considered for preliminary treatment included: 1) improving the existing headworks and 2) construction of a new headworks with new mechanical screening and grit removal.

5.2.2.1 Improve Existing Headworks

The existing headworks provides screening in two channels via mechanically cleaned bar screens. A side channel contains a manually-raked bar screen for use during periods when the mechanical bar screens are out-of-service. There are no existing grit removal facilities.

This preliminary treatment alternative would maintain use of the existing headworks structure and would be limited to replacement of the existing climber screens with new mechanically cleaned screens.

5.2.2.2 Construct New Headworks

The existing headworks screens large debris from raw wastewater but does not have facilities to remove grit. Grit in the raw wastewater can cause excess abrasion and wear on downstream equipment; can form deposits in tanks, pipelines, and other areas or the plant; and can increase the frequency of tank cleaning. 'Grit' typically refers to sand, gravel, or other solid particles with relatively high specific gravities such that they have settling velocities higher than small organic particles and may be removed from plant influent through settlement. Grit removal technologies typically include aerated grit and vortex-type systems.

Vortex-type systems are more common and were thus selected as the technology for use in developing this analysis. Vortex-type grit removal systems consist of an inverted concrete cylinder and paddle that generates a toroidal velocity in the incoming sewage. Grit then settles out of the sewage to the bottom of the cylinder. An example of a vortex-type grit removal system is presented on Figure 5-1 below.

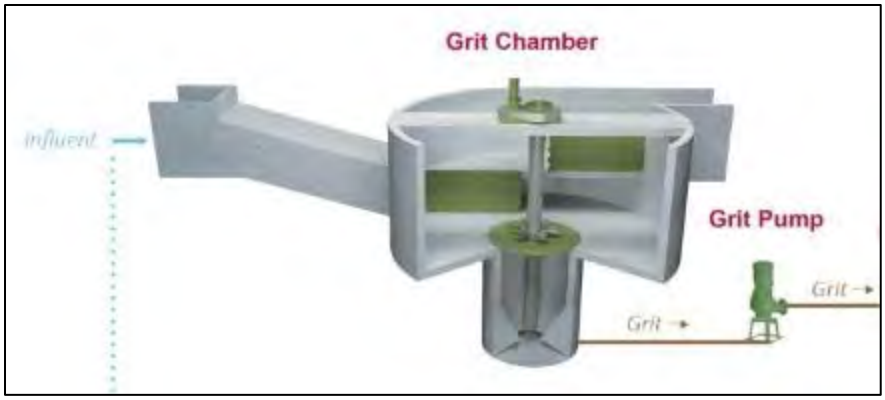


Figure 5-1: Vortex-Type Grit Removal Chamber
(Source: Smith & Loveless)

A pump intermittently pumps settled grit from the cylinder and discharges it to a cyclone separator and to a grit classifier for cleaning and dewatering. The dewatered grit is discharged to a waste bin for disposal offsite.

This alternative would construct a new headworks structure and would include two new mechanically cleaned bar screens, a manually cleaned bar screen, and a vortex-type grit removal system.

5.2.3 Primary Treatment

Primary treatment follows preliminary treatment and is designed to reduce solids loading (i.e., TSS and BOD) on the downstream secondary process. It is commonly used for larger treatment facilities where anaerobic digestion is practiced for solids stabilization. It can typically be relied upon to reduce TSS loading to downstream processes by approximately 60% and BOD loading by approximately 30%. It does so by removing a significant portion of the larger, heavier particulate matter in the raw wastewater. Smaller, lighter particulate matter and dissolved solids are not typically removed in primary treatment processes.

Because primary treatment can significantly reduce solids loading to the downstream processes, the capacity of the secondary treatment process can be reduced with associated cost savings in terms of both physical facility sizing (capital cost) and in long term energy costs for aeration. However, savings in the secondary treatment process must be weighed against the additional cost of increased solids treatment needed to accommodate the additional loading coming directly from the primary treatment process.

Generally, primary treatment is coupled with anaerobic digestion to increase the cost effectiveness of implementing anaerobic digestion which is normally only considered for treatment plants with average annual flows greater than about 5 MGD. Generally, the use of anaerobic digestion is not cost effective for smaller treatment plants. As presented in Section 1 of the Plan, projected year 2045 average annual flow for the WWTP is 5 MGD.

While not normally used unless combined with anaerobic digestion, primary treatment could be warranted when the downstream secondary process cannot accommodate peak flow conditions. Primary treatment in these cases could permit use of split stream treatment during

high flow events. Flow would be split between the primary and secondary treatment processes and recombined prior to disinfection. This would be much like the WWTP's current practice of diverting a portion of the influent raw sewage through its storm screens during high flow events but would result in significantly better treatment.

Alternatives considered for primary treatment included conventional primary clarifiers, enhanced primary clarifiers, and primary filtration.

5.2.3.1 Conventional Primary Clarifiers

Conventional primary clarification removes suspended particulate matter from the water by providing quiescent conditions under which settling can occur. The settled material (primary sludge) is mechanically collected and pumped to the solids treatment system. Important design criteria for primary clarifiers include surface overflow rate (ratio of influent flow rate to clarifier surface area) and depth. Surface overflow rates typically range from about 1,000 gallons per day per square foot (gpd/sf) under average flow conditions up to about 2,500 gpd/sf under peak flow conditions. Clarifier depths typically range from about 10 to 15 feet (ft).

Circular or rectangular tanks can be used for primary clarification. Rectangular tanks have an advantage in that common wall construction can be utilized resulting in space savings for constrained WWTP sites. For both types of clarifiers, primary sludge is removed from the clarifier floor with mechanical scraping systems and conveyed to sludge hoppers from which it flows by gravity or more commonly is pumped. Sludge hoppers can be designed to promote thickening of the sludge prior to pumping to the solids treatment system.

In addition to mechanical sludge collector and primary sludge pumps, primary clarifiers include equipment for primary scum removal. Both rectangular and circular primary clarifiers have surface skimmers that collect and move floating material to a central location where it is removed. Sloped scum beaches are typically used for circular clarifiers and tilting pipe skimmers for rectangular clarifiers. The material is then commonly pumped directly to the solids treatment process.

5.2.3.2 Enhanced Primary Clarifiers

Like conventional primary clarification, enhanced primary clarification creates quiescent conditions under which particulate material in the influent wastewater can settle by gravity and be removed. However, enhanced primary clarification includes the addition of chemical coagulants to enhance settling performance by generating larger particle sizes that settle more quickly. Sludge and scum removal systems for enhanced primary clarification are like those for conventional primary clarification. However, chemical feed systems (e.g., storage, metering, and rapid mixing) are needed.

Advantages of enhanced primary clarification include increased surface overflow rates, smaller clarifiers, and increased solids removal (e.g., TSS removal greater than 60% and BOD removal greater than 30%). Disadvantages include additional complexity with the addition of chemical feed systems, increased sludge mass, and increased operations cost associated with chemical consumption.

5.2.3.3 Primary Filtration

Filtration can be used to provide primary treatment of wastewater. Unlike clarification that relies on gravity to remove particulate matter from the wastewater, filtration removes suspended solids by passing the water through a physical barrier. Particles larger than the pore size of the physical barrier (i.e., filter media) are prevented from passing through simply due to the physical size of the openings. A portion of the smaller suspended solids can be removed through adhesion and absorption to particles already trapped on the surface of the filter media. The filter media, which is a synthetic cloth, must be intermittently purged of the accumulated solids by backwashing to control headloss across the filter.

The filter medium is typically a pile fabric that consists of coarse, randomly woven fiber attached to an underlying woven backing. The pile fabric creates depth for solids storage and helps improve removal efficiency. Nominal pore size for the material is 5 micrometers (µm). The filter medium is supported by a series of disks that are mounted vertically inside a steel or concrete tank. The filter material covers both sides of the disks, which are normally stationary and submerged. Effluent flows by gravity through the disk filter and the solids are trapped on the filter material. When the accumulation of solids on the filter material creates sufficiently high headloss as measured by a level probe, an automated cleaning cycle is initiated. During the cleaning cycle, the disks are rotated while solids are cleaned from the filter medium with a high-pressure spray wash or vacuum suction. Solids removed from the filter media are pumped to the solids treatment process. An example of cloth covered filter disks is shown in Figure 5-2 below.

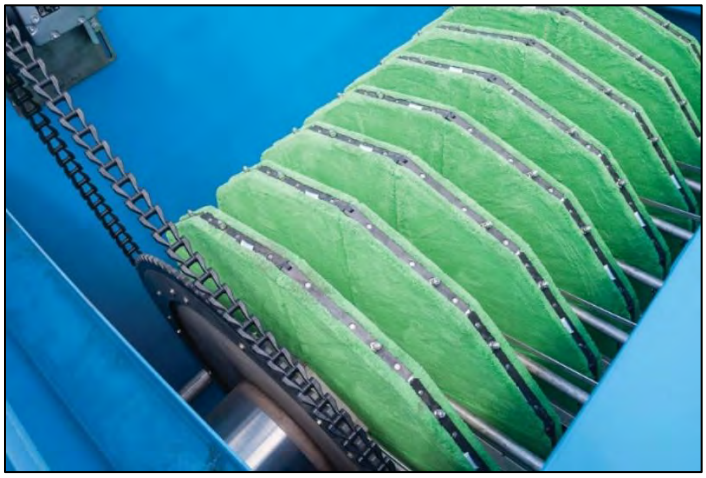


Figure 5-2: Primary Filtration Cloth Media
(Source: Aqua-Aerobics Systems)

Disk filters come as packaged units complete with steel tanks or installed into concrete tanks constructed onsite. For primary filtration, each disk filter assembly (or train) includes inlet and outlet weirs, filter disks, a motorized rotating assembly, backwash system including backwash pump, waste sludge pump, manual and automated control valves, and controls. Controls include starters, protection devices, adjustable frequency drives, relays, and instrumentation.

Advantages of primary filtration when compared to other primary treatment technologies include smaller footprint, lower cost, and potential higher removal efficiencies (compared with conventional primary clarification).

5.2.3.4 Primary Treatment Alternatives Screening

A matrix-based screening analysis of the three primary treatment alternatives was performed to identify one treatment technology that would be used when developing overall project alternatives requiring the use of primary treatment (e.g., anaerobic digestion). Results of this analysis are summarized in Table 5-1 below.

Table 5-1: Primary Treatment Alternatives Screening Analysis

Primary Treatment Alternative	Capital Cost (25%)	O&M Cost (25%)	Technology Maturity (12.5%)	Operational Complexity (12.5%)	Ease of Permitting (10%)	Potential for Community Objection (10%)	Beneficial End Use (5%)	Weighted Score
Conventional Clarification	2	4	5	5	5	3	NA	**3.7**
Enhanced Clarification	3	3	4	4	5	4	NA	**3.6**
Primary Filtration	5	3	3	4	4	5	NA	**4.0**

As shown in Table 5-1, primary filtration received the highest score and was selected as the preferred primary treatment technology. As further described in Section 6, it was used in evaluating overall treatment alternatives that require primary treatment. Should the City choose to provide primary treatment as part of an overall improvement project for the WWTP, additional evaluation during preliminary design of the project should be done prior to final selection of a primary technology.

In general, primary filtration is less costly to construct than conventional primary clarifiers but is anticipated to have higher O&M costs. Filtration is more mechanically involved than conventional primary clarifiers resulting in a lower score for operational complexity. Because it is a newer technology than conventional clarification, it also scored slightly lower in terms of permitting ease. Filtration ranked highest in terms of potential for community objections because of its smaller footprint. Disk filters are also more easily covered to prevent nuisance odors than the larger tanks associated with either conventional or enhanced primary clarification alternatives. The beneficial use criterion was not applicable in ranking the three primary treatment alternatives so was not considered.

5.2.4 Secondary Treatment

Secondary treatment processes are two step processes. The first is typically a biological process that utilizes microorganisms to remove organic (e.g., BOD) and in some cases inorganic (e.g., ammonia) from the influent wastewater. Both dissolved and particulate contaminants are removed and used for cell growth by the microorganisms within the secondary

process. The resulting biomass is then physically separated from the water by gravity (e.g., secondary clarification) or by filtration (e.g., membrane filtration) prior to further downstream treatment.

Seven alternatives were considered for initial screening with four being selected for further evaluation. Alternatives included 1) expansion of the City's existing treatment process, 2) new conventional activated sludge system, 3) oxidation ditches, 4) extended aeration, 5) sequencing batch reactors, 6) membrane bioreactors, and 7) intensification processes.

Other treatment alternatives such as trickling filters and lagoons were not deemed appropriate to address the deficiencies identified within the City's WWTP and were not considered.

5.2.4.1 Expand Existing Treatment Process

The WWTP's existing treatment process is a suspended growth biological treatment process that includes two separate aeration basins. Vertical turbine aerators are used to maintain dissolved oxygen (DO) levels within the mixed liquor (ML). The basins have anoxic selector zones that help improve sludge settleability. Mechanical mixing is used in these zones to maintain solids in suspension. Piping and isolation gates give WWTP staff the ability to operate the basins in step feed or sludge re-aeration modes. These modes are used during high flow events to avoid overloading the existing secondary clarifiers by increasing the concentration of mixed liquor suspended solids (MLSS) within specific sections of the basins. Figure 5-3 shows the WWTP's existing biological treatment process.

Figure 5-3: Existing Secondary Treatment Process

This alternative assumes continued use of the WWTP's two existing treatment basins. However, because the capacity of the existing treatment process is only equal to about 55% of the estimated current maximum month loading and 50% of year 2045 maximum month loading, two new basins would be needed to double the effective capacity of the system. The new basins would be like the existing basins and would include vertical turbine aerators, anoxic zones,

mechanical mixing, and the ability to operate in step feed and sludge re-aeration modes during peak flow conditions.

Advantages to retaining use of the existing facilities include potentially lower capital cost and staff familiarity. Disadvantages include the potential for higher long term operating costs associated with higher energy use, general age and condition of existing structures, potential difficulties in making modifications to the existing structures, and limited area adjacent to the existing structures for expansion.

5.2.4.2 Conventional Activated Sludge

Like the WWTP's existing secondary treatment process, conventional activated sludge (CAS) systems are a type of suspended growth process. However, in CAS systems fine bubble aeration is used instead of vertical turbine mixers resulting in greater basin depths (approximately 18 ft deep compared to 11 ft for the City's existing basins). This alternative would eliminate the City's existing treatment process entirely replacing it with a new CAS system.

CAS systems can be designed to ensure complete nitrification with an SRT of 13 days needed under the minimum monthly temperature condition determined from review of the WWTP's operating data. Maximum design MLSS concentration is about 4,000 mg/L under maximum month loading. Biological selector zones (anoxic and anaerobic) and internal ML recycling systems are typically provided in CAS systems to support biological nutrient removal and improve sludge settleability.

The ability to operate in step feed and sludge re-aeration modes where the influent wastewater is introduced at varying distances downstream of the front of the aeration basin is commonly incorporated into the design of CAS systems. This is especially helpful for communities with highly variable flows and peaking factors. Operating in these modes would enable WWTP staff to retain solids within the aeration basins during peak flow events helping to reduce the potential for solids washout.

Equipment associated with CAS systems normally include aeration blowers, fine bubble diffusers with associated control valves, mechanical mixing in selector zones, internal recycle pumping, DO instrumentation, and associated aeration system controls.

Disadvantages of CAS systems include their relative complexity and higher cost especially when compared to expanding the WWTP's existing secondary treatment system. Advantages include better process control to help optimize both nutrient removal and sludge settleability, relatively energy efficient, and ability to accommodate highly variable influent flows (peaking factors).

5.2.4.3 Oxidation Ditch

Oxidation ditch systems, another category of suspended growth treatment, are oval in shape and use surface brush aerators or vertical turbine style mixers to aerate and continuously circulate mixed liquor around the circular pathway created by the basin shape. Oxygen concentrations within the basin can be manipulated to create alternating aerobic and anoxic conditions as the mixed liquor flows past the aeration equipment. Separate selector basins can be constructed to promote denitrification and improve sludge settleability. Oxidation ditches can be designed to ensure complete nitrification with an SRT of 13 days needed under the minimum

monthly temperature condition determined from review of the WWTP's operating data. Maximum design MLSS concentrations are about 4,000 mg/L under maximum month loading.

Conventional oxidation ditches are configured with a single circular path. A variation of this design is the Orbal® system available from Evoqua. As shown on Figure 5-4 below, the Orbal® style of oxidation ditch contains multiple, independent concentric rings. Openings in the dividing walls allow mixed liquor to flow between rings. During normal operation, wastewater is introduced into the outer ring where it combines with the mixed liquor. Flow then progresses from the outer to inner rings of the basin with mixed liquor ultimately exiting via the inner most ring. The Orbal® style design is inherently well suited for WWTPs with high flow peaking factors. During high flow events, influent wastewater can be directed to one of basin's inner rings instead of the outer ring. This increases the mixed liquor concentration in the outer ring, increases solids retention in the process basin, and reduces loading to the secondary clarifiers helping to prevent solids washout from the secondary process. This mode of operation is much like the sludge step feed or sludge re-aeration modes typically designed into conventional activated sludge processes.

Figure 5-4: Orbal® Style Oxidation Ditch (Source: Evoqua)

Advantages of oxidation ditches include their ability to accommodate peak flow events and lack of mechanical complexity. Equipment is essentially limited to the surface aeration equipment and DO instrumentation used to control aerator speed.

5.2.4.4 Extended Aeration

Extended aeration systems are suspended growth biological systems that have SRTs greater than 30 to 40 days. One example of such systems is the Biolac® wastewater treatment system from Parkson. It is a low rate activated sludge process that typically utilizes lined earthen basins with aeration air being provided via suspended fine bubble diffusers. An example of a Biolac® treatment system is present on Figure 5-5.

Figure 5-5: Biolac® Extended Aeration System (Source: Parkson)

As shown on Figure 5-5, floating aeration chains span the width of the aeration basins. The aeration chains are constructed of flexible high-density polyethylene tubing that float on the water surface. Fine bubble diffusers are attached to the floating aeration tubing and are suspended just above the basin floor. The aeration chains are anchored on both sides of the basin via a cabling system and are connected to an air header running along the length of the basin. Electrically actuated valves on the air header are used to control the flow of air into the floating aeration chains. Because the aeration chains are constructed of flexible tubing, air released from the diffusers causes the chains to oscillate back and forth in a recurring pattern. The oscillation of the aeration chains provides some supplemental mixing. The chains are spaced evenly to cover the entire basin floor with the oscillating patterns. Adjusting the tension on the aeration chains changes the magnitude of oscillation.

When nutrient removal is not required, the Biolac® system aeration chains are operated with the same air flow to each aeration chain, and the aeration basin is uniformly oxygenated with a constant DO throughout the basin. The process can be modified to perform denitrification by using “wave oxidation,” which controls the air flow to each aeration chain so that some chains operate at a very low air flow while others operate at a higher air flow, promoting several aerobic and anoxic zones within the basin at the same time. This allows biological nitrification and denitrification to occur simultaneously. Timers are used to cycle the air flow to each aeration chain, creating a moving “wave” of changing aerobic and anoxic zones.

Equipment associated with the Biolac® system includes aeration blowers, fine bubble diffusers, electrically actuated air control valves, DO instrumentation, and an automated control system. Advantages of extended aeration systems include lower capital cost and their relative simplicity to operate. Disadvantages include their significantly larger size, poorer sludge settleability, and limited ability to accommodate high flow events.

5.2.4.5 Sequencing Batch Reactors

A sequencing batch reactor (SBR) is a fill-and-draw system in which all steps of the activated sludge process occur in one reactor. At least two basins are used for municipal wastewater treatment with a continuous influent flow, so that one basin is in the fill mode while the other reactor completes the biological treatment, allows solids to settle, and decants the effluent.

Specific steps taken as each SBR basin progresses through the batch process consist of the following:

- Fill – Basin is filled with wastewater and may be aerated or anoxic during this phase. During an anoxic fill, the reactor is mixed without aeration using mechanical mixing equipment. An anoxic fill phase can function as an anoxic selector to provide denitrification and improve sludge settling characteristics.

- React – Influent wastewater flow into the basis stops and is redirected to the second basin that is in fill mode. Aeration begins. Mechanical mixing may also be provided to supplement mixing from aeration to reduce the air requirement. Oxidation and nitrification are accomplished under aerobic conditions. Denitrification can be accomplished by alternating between aerobic and anoxic conditions. In addition, an anaerobic reaction phase may be included after anoxic fill and before aeration to enhance biological phosphorus removal.

- Settle – Mixing and aeration cease. Solids settle and separate from the treated effluent under quiescent conditions.

- Draw – Mixing and aeration systems remain off. Effluent is withdrawn from just below the liquid surface by means of a decanter. The decanter floats to allow shorter settling periods and variable decant levels. Grease, scum, and other floating solids are largely excluded from the effluent because the decanter withdraws effluent from below the liquid surface.

- Waste – Sludge is wasted from the bottom of the reactor to maintain the operating SRT. After wasting, the reactor is ready to repeat the cycle.

Mixed liquor remains in the reactor during all cycles thereby eliminating the need for separate secondary clarifiers and return activated sludge (RAS) pumping. Aeration may be accomplished by jet aerators or coarse or fine bubble diffusers. Separate mixing can reduce the aeration demand and provide flexibility by facilitating operation under anoxic and anaerobic conditions. Because the flow of effluent from the reactor is condensed into a short period of time, effluent flows out of the reactor at a much higher rate than when it enters the reactor. As a result, post-equalization is often included to avoid having to oversize downstream unit processes.

Advantages of SBRs include elimination of secondary clarifiers and associated RAS pumping system. Operating conditions can also be closely controlled to facilitate biological nutrient removal. Disadvantages of SBRs include their relatively high level of complexity. Because they are a batch system, SBRs are not well suited for systems with highly variable flow and large peaking factors.

5.2.4.6 Membrane Bioreactors

A membrane is a thin film that separates two phases (liquid and solid) and acts as a selective barrier preventing the transport of certain materials. As the liquid being filtered flows through the membrane, particles are transported to the membrane surface where they are removed.

Membrane bioreactors (MBRs) are a comparatively new technology that can produce effluent of exceptional quality. The process is a modification of the conventional activated sludge process,

with membranes immersed in the process to separate solids eliminating the need for secondary clarification. Average pore size for wastewater membranes range from about 0.04 to 0.2 micrometers (µm). The small pore size behaves as a physical barrier to essentially all suspended solids, while still allowing water to pass through. Water is typically drawn through the membranes by downstream permeate pumps.

Membranes can be provided in either a hollow fiber configuration or as flat sheets. When applied within secondary treatment processes, the direction of water flow for both configurations is outside to in. An example of a hollow-type membrane is presented on Figure 5-6 below. In this example, the grouping of hollow fiber membranes (i.e., cassette) is being removed from biological reactor tank for routine inspection.

Figure 5-6: Hollow Fiber Membrane Removal

Accumulation of solids including mineral and biological fouling is controlled through various cleaning mechanisms. These include air scour, chemical cleaning, membrane relaxation, and/or membrane backwashing and are performed automatically by the MBR's control system. However, periodic intensive chemical cleans must be initiated manually. Continuous use of air scouring to reduce the accumulation of material on the membrane surface requires large quantities of air that, coupled with permeate pumping, result in significantly higher energy use compared to conventional and extended aeration activated sludge processes.

Because the membranes provide a physical means of separating solids from the water, secondary clarifiers are not needed. This results in the ability of membrane systems to operate at significantly higher MLSS concentration (e.g., around 10,000 mg/L compared to a maximum of 4,000 mg/l for other typical processes). Required basin size is thus much less than other activated sludge processes.

A significant advantage of MBRs is that they produce tertiary quality effluent without the need for additional unit processes. Because the membrane forms a physical barrier removing essentially all suspended solids, sludge settleability is eliminated as an operational concern for

WWTP staff. Disadvantages of MBRs typically include higher capital and operating costs (energy, chemical, and membrane replacement). Additional pretreatment including fine screening (typically 2-millimeter openings) is also required to protect the membranes from stringy material. MBR systems are also not well suited for systems with highly variable influent flow rates and can only accommodate peaking factors of about 2 to 1 at water temperatures typical of the Pacific Northwest.

5.2.4.7 Intensification Processes

Intensification processes are newer technologies that increase the capacity of the biological treatment system per unit of basin volume. Examples of these processes include the following:

- **Moving Bed Biofilm Reactor (MBBR).** MBBR systems are a type of fixed film treatment that uses loose synthetic (plastic) carrier media placed within the aeration basin. Biofilm forms on the free moving carrier media. Screens are used to retain the media in the aeration basins. Diffused aeration is used to supply oxygen needed to support biological activity.

- **Integrated Fixed-Film Activated Sludge (IFAS).** IFAS is like MMBR in that it is a fixed film type process that utilizes synthetic carrier media. However, IFAS systems include return of activated sludge from the secondary clarifiers.

- **Aerobic Granular Sludge (AGS).** AGS is an SBR like process, specifically designed to create growth environments that favor the formation of dense granular sludge. Aerobic, anoxic, and anaerobic zones within the sludge granule can result in nutrient removal. Because it is an SBR like process, secondary clarifiers are not needed.

- **Ballasted Activated Sludge.** Ballasted activated sludge systems such as the BioMag® system from Evoqua introduces fine particles of magnetite to the activated sludge system. Biological floc forms around the magnetite. Because magnetite is about five times denser than water, the resulting floc settles much more rapidly. This allows the biological system to be designed for MLSS concentrations that are two to three times greater than that of other suspended growth treatment systems without decreasing the performance of the secondary clarifiers. The magnetite is mechanically recovered from the waste activated sludge (WAS) stream and returned to the biological process.

Advantages of intensification processes include their ability to provide significantly more treatment capacity within a given biological reactor volume. This can be especially advantageous for space limited sites or when existing facilities can be retrofitted to allow their use. Disadvantages include increased complexity, potential for higher operating costs, and relative newness of the technologies. Intensification processes were deemed unsuitable for use in retrofitting the WWTP's secondary treatment process because the shallow depth of the existing basins would severely limit the oxygen transfer efficiency of aeration systems. This would make it unlikely that sufficient air (oxygen) could be supplied to support high biomass concentrations.

5.2.4.8 Secondary Treatment Alternatives Screening

A matrix-based screening analysis of the seven secondary treatment alternatives was performed. Results of this analysis are summarized in Table 5-2 below. The top four alternatives were selected for further evaluation as presented in Section 6 of the Plan.

Table 5-2: Secondary Treatment Alternatives Screening Analysis

Secondary Treatment Alternative	Capital Cost (25%)	O&M Cost (25%)	Technology Maturity (12.5%)	Operational Complexity (12.5%)	Ease of Permitting (10%)	Potential for Community Objection (10%)	Beneficial End Use (5%)	Weighted Score
Existing Process	5	3	5	5	5	4	4	**4.4**
CAS	3	4	5	4	5	4	4	**4.0**
Oxidation Ditch	3	5	5	5	5	4	4	**4.4**
Extended Aeration	4	5	4	5	5	3	4	**4.4**
SBR	4	3	4	3	5	4	4	**3.7**
MBR	2	2	4	3	5	4	5	**3.0**
Intensification	4	3	3	4	4	4	4	**3.6**

As shown in Table 5-2, the four secondary treatment processes receiving the highest scores consisted of 1) expansion of the existing system, 2) conventional activated sludge (CAS), 3) oxidation ditch, and 4) extended aeration. These four alternatives received further evaluation as detailed in Section 6 of this Plan.

Considerations factoring into the lower ranking of the three alternatives removed by this initial screening evaluation included the following:

- Sequencing batch reactor (SBR). Because they are a batch process, SBRs are not well suited in cases with highly variable influent flows. They are also more operationally complex than other alternatives considered with perceived higher O&M costs.

- Membrane Bioreactor (MBR). Although capable of producing extremely high-quality effluent which resulted in the MBR alternative receiving the highest score for beneficial use (i.e., potential for recycled water), effluent quality did not outweigh higher initial capital and long-term O&M costs associated with MBR treatment. In addition, an MBR based treatment system would not use the City's existing 130-ft secondary clarifier which would then essentially become a wasted asset.

- Intensification. Intensification processes scored lower in terms of their perceived higher O&M costs. Maturity of intensification processes was also considered to be much lower when compared with the other more conventional secondary treatment options.

5.2.4.9 Secondary Clarifiers

Following secondary biological treatment, suspended solids must be removed to clarify effluent. The settled sludge is returned to the secondary treatment process RAS. Excess is removed (WAS) The WWTP's existing secondary clarifiers include three smaller clarifiers built in 1976 with diameters ranging from 50 to 60 ft and a side water depth of 10 ft. A larger, 130-ft diameter, 16-ft deep clarifier was constructed in 2012. The smaller clarifiers are shallow, in poor mechanical condition, and do not perform well during high flow events.

This alternative would replace the three, small existing clarifiers with a new 130-ft clarifier that would match the clarifier constructed in 2012. This alternative would also include ancillary solids pumping infrastructure for RAS, WAS, and scum.

5.2.5 Tertiary Filtration

Filtration is a process that removes suspended solids from the liquid stream by passing the liquid through a physical barrier (i.e., woven mesh, granular media, or membrane). Suspended solids larger than the pore size of the filter media are prevented simply based on size. Smaller suspended solids are also removed through adhesion and absorption to particles already trapped on the surface of the filter media. The filter media must be intermittently purged of the accumulated solids by backwashing to control headloss across the filter media.

The City's existing NPDES permit contains summer and winter time effluent discharge limits that, under specific flow conditions, would require the WWTP to produce effluent with BOD and TSS concentrations below 10 mg/L. While conventional secondary treatment process can be relied upon to consistently meet effluent BOD and TSS concentrations down to 10 mg/L, tertiary filtration is recommended for concentrations below 10 mg/L.

Conditions under which effluent concentration limits would be below 10 mg/L are as follows:

- **Dry weather (May 1 through October 31).** The City's existing NPDES permit dry weather monthly mass load limit for BOD and TSS is 250 lbs/day. Under design year 2045 ADF projected flow conditions, the mass load limit will require the effluent concentrations of BOD and TSS be no more than 6 mg/L. Under design year 2045 MMDWF conditions, effluent concentrations would need to be no more than 5 mg/L.

- **Wet weather (November 1 through April 30).** The City's existing NPDES permit requires 85% monthly average removal efficiencies for BOD and TSS except when the monthly average flow exceeds 3.0 MGD when the required removal efficiency is reduced to 75%. Because the projected design year 2045 MMWWF exceeds 3.0 MGD, the current NPDES would only require 75% removal efficiency. Under projected design year 2045 average annual loading, effluent BOD and TSS concentration would be greater than 10 mg/L. However, should the DEQ eliminate relaxation of the 85% removal efficiency in the next permit cycle, effluent concentrations for BOD and TSS would need to be no more than 7 mg/L.

Under the City's existing NPDES permit, tertiary filtration would only be required to meet effluent limits during the dry weather period and could be designed with a capacity equal to that of the design year 2045 MMDWF flow condition (5.4 MGD). However, for the purposes of this evaluation and to allow for future changes in the City's NPDES permit that would require year-

round BOD and TSS removal efficiency to be 85%, the capacity of the tertiary filtration system was assumed to be equal to the design year 2045 MMWWF condition (10.1 MGD).

Alternatives for tertiary filtration include 1) synthetic/cloth media filtration, 2) granular media filtration, and 3) membrane filtration.

5.2.5.1 Synthetic/Cloth Media Filtration

Synthetic/cloth media filtration alternatives include cloth disk filters and compressible media filters. However, only cloth disk filters like those described for primary filtration above were considered for this analysis due to their more common use.

The filter medium for cloth disk filters is typically a pile fabric that consists of coarse, randomly woven fiber attached to an underlying woven backing. The pile fabric creates depth within the filter medium for solids storage and helps improve removal efficiency. Nominal pore size for pile type fabric medium ranges from 5 to 10 µm. The filter medium is supported by a series of disks that are mounted vertically inside a tank which results in a relatively small footprint. The medium covers both sides of the disks, which are normally stationary and submerged. Effluent flows by gravity through the disk filter and the solids are trapped on the filter material. When the accumulation of solids on the filter material creates sufficiently high headloss as measured by a level probe, an automated cleaning cycle is initiated. During the automated cleaning cycle, the disks are rotated while solids are cleaned from the filter medium with a high-pressure spray wash or vacuum suction. Backwash water is returned to the secondary treatment process.

Disk filters come as prepacked packaged units complete with steel tanks or installed into concrete tanks constructed onsite. For use in tertiary filtration, each disk filter unit includes inlet and outlet weirs, filter disks, a motorized rotating assembly, backwash system including backwash pump, manual and automated control valves, and controls. Controls include starters, protection devices, adjustable frequency drives, relays, and instrumentation.

5.2.5.2 Granular Media Filtration

There are several different types of granular medium filters which can be classified according to flow direction (i.e., up-flow or down-flow), driving force (i.e., gravity or pressure), flow rate (i.e., slow or rapid rate filtration), and type of media (i.e., single, dual, or multimedia). Down-flow filters are the most common conventional granular filter type used for tertiary filtration. The driving force is either gravity or applied pressure with pressure type filters being more commonly used at smaller plants. Amongst the granular media filter alternatives, rapid rate filtration is usually preferred for tertiary filtration to reduce cost and space requirements.

The size of the granular filter media, normally described by its effective size, is one of the main parameters affecting solids removal performance and headloss development across the filter. Dual-media (and multimedia) filters have an advantage over single media filters by allowing suspended solids to penetrate the filter bed to a greater degree, thus providing increased solids storage capacity and longer filter runs. The top media layer in dual-media, down-flow filters is typically anthracite with sand being used for the bottom layer. Total media depth for dual-media filters is typically about 36 inches (18 to 24 inches of anthracite and 12 to 18 inches of sand). Media depth for single media, conventional granular filters (composed of sand or anthracite) is typically 6 inches less than dual-media conventional granular filters.

The granular medium is intermittently purged of accumulated particles by backwashing, with filtered water being forced back through the filter in the reverse direction of normal flow. Accumulated solids are stripped from the media and transported back to the secondary treatment process. Forward flow is temporarily halted when a filter is backwashed. Air is typically used to help clean the media.

Granular filtration systems can be constructed within concrete or steel tanks. Other system components typically include an influent distribution system, underdrains, filter media, backwash pumping systems, air scour systems, chemical (i.e., coagulant and/or polymer) feed equipment, instrumentation, and controls.

5.2.5.3 Membrane Filtration

A membrane is a thin film that separates two phases (liquid and solid) and acts as a selective barrier preventing the transport of certain materials. As the liquid being filtered flows through the membrane, particles are transported to the membrane surface where they are removed.

Pressure driven membranes typically used for tertiary filtration can be categorized into two types: microfiltration (MF) or ultrafiltration (UF) with the main difference being the average pore size of the membrane material with MF pore size being larger at about 50 nanometers (nm). UF membrane pore size typically ranges from 2 to 50 nm. Because of their smaller effective pore size, membrane filtration can produce higher quality effluent than the other filter alternatives considered. However, initial capital costs and ongoing O&M costs are normally greater.

Membranes used for tertiary filtration are available in several different configurations including tubular, hollow fiber, and flat sheet. Flow can be inside to out or outside to in. The membranes can be placed within tanks or inside pressure vessels. System components typically associated with membrane filters include the membrane filters themselves, feed pumping systems, air scour and water backwashing systems, chemical cleaning equipment, instrumentation, and controls.

5.2.5.4 Tertiary Filtration Alternatives Screening

A matrix-based screening analysis of three tertiary filter alternatives was performed to identify one alternative for further development of overall recommended improvements for the WWTP. Results of this analysis are summarized in Table 5-3 below.

Table 5-3: Tertiary Filtration Alternatives Screening Analysis

Filtration Alternative	Capital Cost (25%)	O&M Cost (25%)	Technology Maturity (12.5%)	Operational Complexity (12.5%)	Ease of Permitting (10%)	Potential for Community Objection (10%)	Beneficial End Use (5%)	Weighted Score
Cloth Media	5	5	4	4	4	5	4	**4.6**

Filtration Alternative	Capital Cost (25%)	O&M Cost (25%)	Technology Maturity (12.5%)	Operational Complexity (12.5%)	Ease of Permitting (10%)	Potential for Community Objection (10%)	Beneficial End Use (5%)	Weighted Score
Granular Media	4	4	5	4	5	5	4	**4.3**
Membrane	2	3	4	3	4	5	5	**3.3**

The intent of the screening analysis for tertiary filtration was to identify one technology that would act as a generic filtration alternative to be carried forward for use in more detailed overall project alternatives development. As shown in Table 5-3, cloth disk filtration received the highest score and was thus selected for further evaluation. While cloth disk filtration was selected as the recommended tertiary filtration technology for the purposes of this Plan, ultimate selection of a specific filtration technology should be done during the preliminary design phase of the recommended project.

In general, cloth disk filters are less costly to construct and operate than the other alternatives considered and thus scored higher in those categories. Although it has been in use for more than 20 years in the wastewater industry, cloth filtration is still a newer technology than granular media filtration so ranked slightly lower in terms of technology maturity. Ease of permitting was also slightly lower than granular media filtration for this same reason due to it being somewhat newer to reviewing agencies. While cloth disk filter systems are mechanical in nature and include a rotary drive system, pumping, instrumentation, and controls, they are much like other mechanical treatment systems associated with wastewater treatment and less complex than other technologies such as membrane filtration. Because there were no perceived community objections to providing tertiary filtration, all alternatives received high scores for potential for community objection. While cloth disk filters and granular media filters can produce high quality effluent, membrane filtration would result in the highest effluent quality of the technologies considered and thus ranked highest for the beneficial end use criteria.

Membrane filtration ranked lowest of the three alternatives considered. This was due to its relatively high cost and complexity. Membrane filtration did, however, rank highest in terms of potential for beneficial use due to its ability to produce the highest effluent quality of the alternatives evaluated.

5.2.6 Disinfection

Disinfection is the process by which microorganisms remaining in the effluent are deactivated or destroyed. The existing WWTP disinfects secondary effluent with chlorine by injecting sodium hypochlorite upstream of the City's 0.28 million gallon (MG) chlorine contact basin (CCB). Effluent is dechlorinated with sodium bisulfite prior to its discharge to the South Santiam River. The existing CCB does not meet DEQ required detention times under high flow conditions (peak day and peak hour).

Four alternatives were considered for initial screening of technologies to address deficiencies identified with the WWTP's existing disinfection system. Alternatives included 1) continuing to chlorinate, 2) ultraviolet (UV) light, 3) ozonation, and 4) alternate technologies (e.g., peracetic acid and pasteurization). Chlorination and UV disinfection were selected for further evaluation.

5.2.6.1 Chlorination

This disinfection alternative assumes continued use of sodium hypochlorite to disinfect treatment plant effluent. The use of sodium bisulfite for dechlorination prior to final discharge was also assumed to continue.

The City's existing chlorine disinfection system consists of two parallel CCBs with a combined volume of about 0.24 MG. Sluice gates and piping within the basins allow the CCBs to operate in parallel or in series. This allows basins to be removed from service for cleaning during periods of low flow. Sodium hypochlorite storage and metering facilities consist of two peristaltic metering pumps and a new 4,700-gallon double walled storage tank that have been placed outside. The existing sodium bisulfite storage and metering system is located immediately east of the CCBs in facilities constructed in 2008. These facilities consist of two diaphragm metering pumps and a single 1,830-gallon storage tank.

Because the existing CCBs do not meet DEQ requirements for detention time, this alternative assumes construction of at least one new CCB. Construction of new sodium hypochlorite and sodium bisulfite storage and metering facilities would also be included. New sodium hypochlorite storage and metering facilities would be needed because of the poor condition of the existing control building complex and the City's desire to replace it. Expansion of the existing CCB would most likely occur to the east where the existing sodium bisulfite facility is located necessitating its replacement.

Because of the age of the existing CCBs and potential difficulty entailed with expanding the existing structure, complete replacement of the existing CCBs should be considered as a sub-alternative.

5.2.6.2 Ultraviolet Light

Ultraviolet (UV) disinfection is accomplished by exposing secondary effluent to UV light. UV light occurs at wavelengths between those of X-rays and visible lights. Exposure of microorganisms to UV light alters their DNA, prevents cell replication, and results in rapid die-off of the organisms. The use of UV disinfection would eliminate the use of chlorine and therefore the need for dechlorination.

The main equipment components of UV systems include UV lamps, ballasts, and their power/control panels. Special bulbs containing argon and mercury vapor produce UV light with a wavelength between 200 and 300 nm. Peak efficiency for disinfection with UV light occurs at approximately 260 nm. The bulbs are placed within quartz sleeves that protect the lamps from direct contact with the effluent. The outside surface of the quartz sleeves must be cleaned regularly. This is typically accomplished with automated cleaning systems incorporated into the UV system equipment.

UV systems are available in two physical configurations: open channel and closed vessel. UV lamps in the open channel configuration are placed within a channel that is open to the atmosphere. Water flow is by gravity. Channels may be concrete or stainless steel and are filled

with groupings of UV lamps called banks. Banks may be arranged horizontally or vertically. If arranged vertically, these banks typically have mechanical systems that raise or lower them from the channel for maintenance work. Figure 5-7 below shows an example of an open-channel type UV disinfection system.

Figure 5-7: Open Channel UV System (Source: Trojan Technologies, Inc.)

Closed-vessel systems consist of in-line flanged fittings containing UV lamps. Flow is often split at a header and multiple pipe branches are used to provide the design UV dose. Lamps must be accessible for routine maintenance and be placed above grade or in an access vault. For this alternatives evaluation, the open channel arrangement was used because of its wide use, scalability, and simplicity of construction. Should UV disinfection be selected as the preferred improvement alternative for inclusion in an overall improvements project for the WWTP, additional evaluation during preliminary design of the project should be done prior to final selection of UV system configuration.

Design criteria for all disinfection alternatives include the level of disinfection required, reliability, and flow rate. The peak instantaneous flow condition is used as the primary flow criteria for UV systems. Additional criteria specific to UV systems include UV transmittance (UVT) and total suspended solids (TSS).

UVT is a measure of the clarity of the wastewater to passage of UV light. It is the main parameter controlling the level of UV energy required to disinfect a particular wastewater stream. UVT monitoring is often integral to UV systems and provides an input to the system's programmable logic controller (along with flow rate) to determine the appropriate UV dosage. Factors influencing UVT include color, suspended and colloidal solids, and chemical contaminants such as iron or various organic compounds.

Quality of the secondary effluent, specifically TSS, also has a significant impact on the sizing of UV disinfection equipment. Suspended solids can shield pathogens from exposure to the UV light. Higher TSS concentrations require more UV light and thus more energy. If UV disinfection is ultimately chosen, data on UV transmittance for the secondary effluent from the City's WWTP will need to be collected during detailed design.

5.2.6.3 Ozonation

Although uncommon, ozone can be used for wastewater disinfection. Like chlorine, ozone is a strong oxidizer which results in its ability to destroy microorganisms in the water. Electrolysis is used to produce ozone from air or high purity oxygen. Because ozone gas rapidly decomposes to oxygen, it is generated onsite and immediately fed into a contactor vessel where it diffuses into the wastewater and provides disinfection. Excess ozone that does not dissolve into the water leaves the reactor vessel and is treated and destroyed prior to discharge as oxygen to the atmosphere.

System components typically include an electrical power supply, feed gas supply (e.g., air or high purity oxygen), ozone generator, contactor vessel, off-gas removal equipment, instrumentation, and controls.

5.2.6.4 Alternative Disinfection Technologies

There are other alternative disinfection technologies that have received some consideration within the industry. Examples of these include the use of peracetic acid and pasteurization neither of which has seen significant full-scale use.

Peracetic acid is a combination of acetic acid and hydrogen peroxide. Its use in wastewater disinfection has been considered to reduce the potential for the generation of disinfection byproducts such as those associated with chlorination.

Pasteurization, which is the heating of water to temperatures below boiling for a specific time, has also been considered for destruction of water borne pathogens in wastewater. Time and temperature required are inversely related with shorter time periods required at higher temperatures.

5.2.6.5 Disinfection Alternatives Screening

A matrix-based screening analysis of the four disinfection alternatives was performed to identify two alternatives for additional consideration in determining recommended improvements to the WWTP's disinfection system. Results of this analysis are summarized in Table 5-4 below.

Table 5-4: Disinfection Alternatives Screening Analysis

Disinfection Alternative	Capital Cost (25%)	O&M Cost (25%)	Technology Maturity (12.5%)	Operational Complexity (12.5%)	Ease of Permitting (10%)	Potential for Community Objection (10%)	Beneficial End Use (5%)	Weighted Score
Chlorination	4	4	5	5	5	4	4	**4.4**
UV	5	5	5	4	5	5	5	**4.9**
Ozonation	3	3	4	3	4	4	4	**3.4**
Other Technologies	3	3	3	3	3	3	3	**3.0**

As shown in Table 5-4, the two disinfection alternatives processes receiving the highest scores were chlorination and UV. These two alternatives received further evaluation as detailed in Section 6 of this Plan.

Considerations factoring into the lower ranking of the two disinfection alternatives removed by this initial screening evaluation included the following:

- Ozonation. Capital and O&M costs for ozonation were deemed higher than either chlorination or UV. Ozonation has not been widely applied within the wastewater industry and there is less application experience than with chlorine and UV. Ozone may also breakdown the organic carbon remaining in the treated effluent making it more biologically available. This could result in higher effluent BOD than might otherwise be present.

- Other Technologies. Like ozonation, capital and O&M costs for alternative technologies (e.g., peracetic acid, pasteurization, etc.) were deemed to be higher than either chlorination or UV. While studied for use in the disinfection of wastewater, there are few actual full-scale applications. Pasteurization also has the potential to increase effluent temperatures and thus thermal loading to receiving waters. Peracetic acid is not as widely available as chlorine.

5.2.7 Effluent Pumping

In its current configuration, the WWTP receives raw sewage at the WIPS and provides enough lift to drive plant flows through preliminary and secondary treatment via gravity. An additional pumping station, the Effluent Pumping Station, lifts secondary effluent up into the disinfection system (CCBs) where it then flows by gravity through final discharge to the South Santiam River.

The existing Effluent Pumping Station contains four pumps, two 100-hp pumps and two 20-hp pumps. Based on information contained in City-provided pump curves for the pumps, it was determined that the existing firm capacity of the pump station is about 16.3 MGD. To increase

the pump station capacity to handle projected design year 2045 peak hour flows, two alternatives were considered.

5.2.7.1 Improve Existing Effluent Pumping Station

The existing pump station was originally constructed in 1976. In 2015, the station was improved with the installation of two 100-hp pumps and two 20-hp pumps. Space was shown on the record drawings for the 2015 project for expansion of the wet well and anticipated installation of a third 100-hp pump. Figure 5-8, taken from 2015 project record drawings, shows the existing wet well and its anticipated expansion. Electrical provisions were also made during the 2015 pump station rehabilitation to allow future installation of a 100-hp variable speed drive for the future pump.

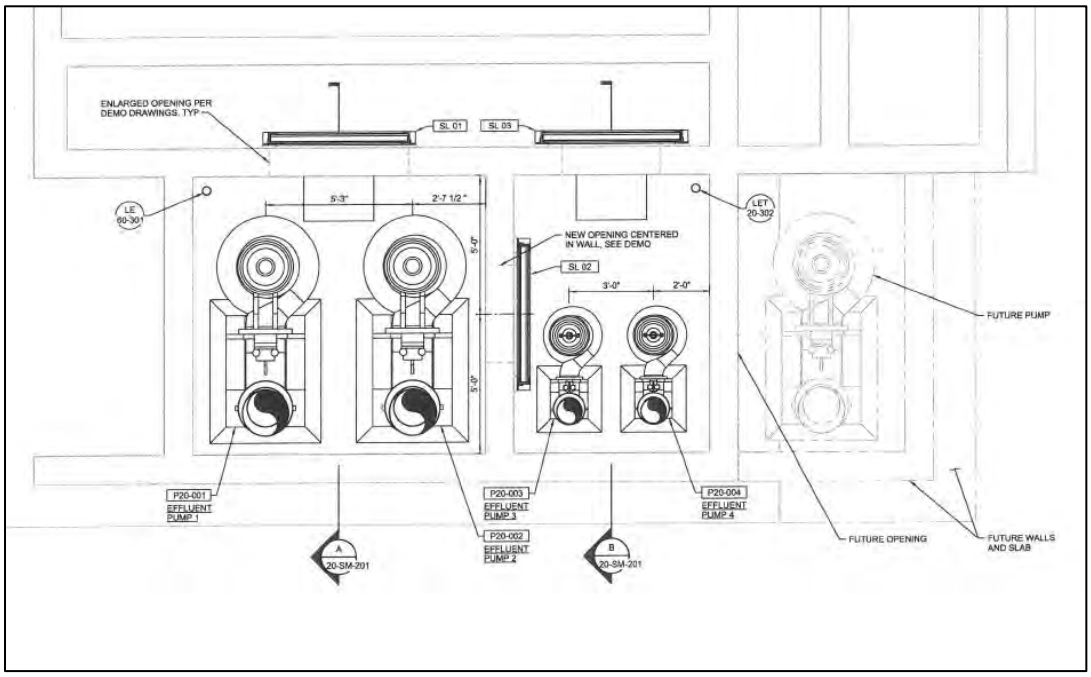


Figure 5-8: Plan of Existing Effluent Pump Station Wet Well

Although expansion of the pumping station was anticipated to include the addition of a single larger pump, hydraulic calculations done as part of this Plan shows that this arrangement would not increase the station's firm capacity enough to meet design year 2045 peak flows. Therefore, this alternative consists of replacing the two existing 20-hp pumps with a 100-hp pump in addition to expansion of the wet well. The station would thus contain four, identical, 100-hp pumps.

5.2.7.2 Eliminate Effluent Pumping

This alternative considered the feasibility of eliminating effluent pumping altogether to reduce both capital and long-term operating and maintenance costs. Kennedy Jenks reviewed elevations of existing WWTP structures as shown on City-provided record drawings and compared them with the projected 100-year flood river elevation (i.e., 340.00 ft above mean sea level [NAVD88]). This review showed that effluent weirs of the existing secondary clarifiers are about 8 inches below the projected 100-year flood river elevation. Therefore, it is not possible to

eliminate use of an effluent pumping station unless all structures upstream of disinfection, including the relatively new 130-ft secondary clarifier, are replaced and constructed at a higher elevation. Thus, this alternative was eliminated from further consideration.

5.3 Recycled Water Alternatives

Recycled water alternatives are presented in this section to address the potential need for a dry weather effluent strategy should thermal load allocations to the South Santiam River change. In the future it may become necessary to treat (i.e., cool) or divert effluent before discharge to mitigate temperature impacts to the receiving stream.

This section presents information related to the diversion of effluent to a Class C recycled water system that includes application of treated and disinfected effluent to agricultural lands (irrigation) to produce crops in accordance with Oregon DEQ Recycled Water rules (OAR 340-055) and guidance (Oregon DEQ Recycled Water Internal Management Directive, 2009). Class C Recycled Water requirements are summarized in Table 5-5. A Class C recycled water program was assumed instead of Class A or B programs because of the perceived availability of suitable agricultural irrigation sites and lower cost.

Table 5-5: Class C Recycled Water Requirements

Parameter	Value[(a)]
Agricultural Uses	Irrigation of Christmas trees, pasture, processed food, and seed crops
Setback Requirements	70-ft from property lines 100-ft from water supply source
Treatment	Effluent must be oxidized and disinfected
Disinfection Criteria	Must not exceed a median of 23 total coliform organisms per 100 milliliters, and 240 coliform organisms per 100 milliliters in any two consecutive samples
Monitoring Criteria	Once per week minimum

Note:
(a) Sprinkler Irrigation, OAR 340-55.

The proposed Class C recycled water system would include gravity conveyance of disinfected secondary effluent from the disinfection process to a 75-acre storage reservoir located on the WWTP site, or lands adjacent to the WWTP that would be purchased by the City. The storage reservoir would be sized to store flows during the shoulder months of May and October when there is typically little demand for irrigation water, as well as to store flows during wetter than average summers. A preliminary evaluation was performed to determine the feasibility of eliminating the storage reservoir. However, because the area needed for irrigation would range from about 5,000 to 10,000 acres depending on the precipitation condition assumed, it was not found to be feasible and all further alternatives considered assumed use of storage.

Other system components would include a recycled water pumping station that would convey stored effluent to the reuse site. A booster pumping station and sprinkler irrigation system at the reuse site would convey and apply recycled water. A process flow diagram depicting the recycled water system is shown on Figure 5-9.

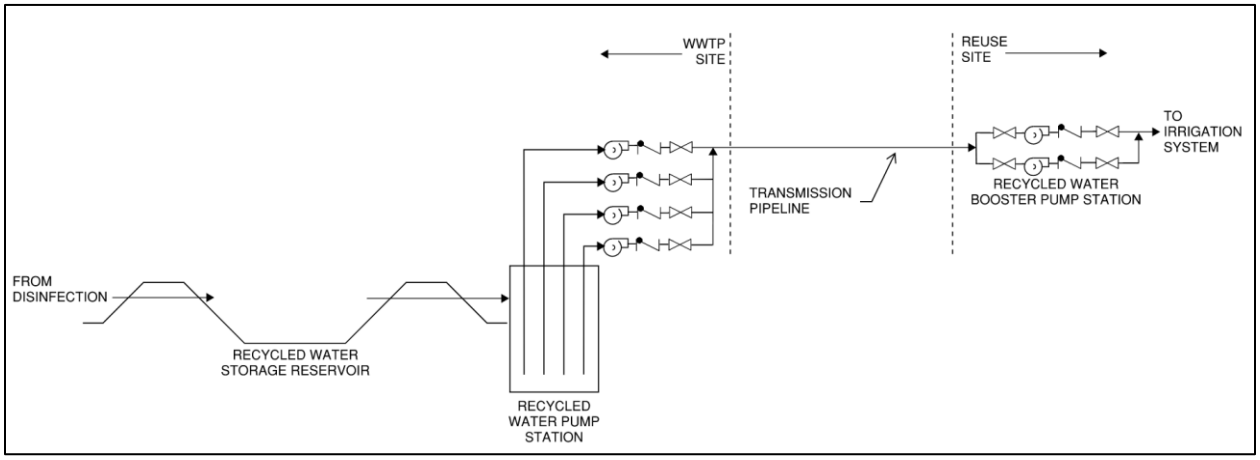


Figure 5-9: Recycled Water Process Flow Diagram

For this evaluation, it was assumed that the City would enter into an effluent reuse agreement with a property owner or owners with suitable agricultural land located within 3 miles of the WWTP site. The 75-acre storage reservoir, WWTP, and potential agricultural sites within a 3-mile radius are shown on Figure 5-10.

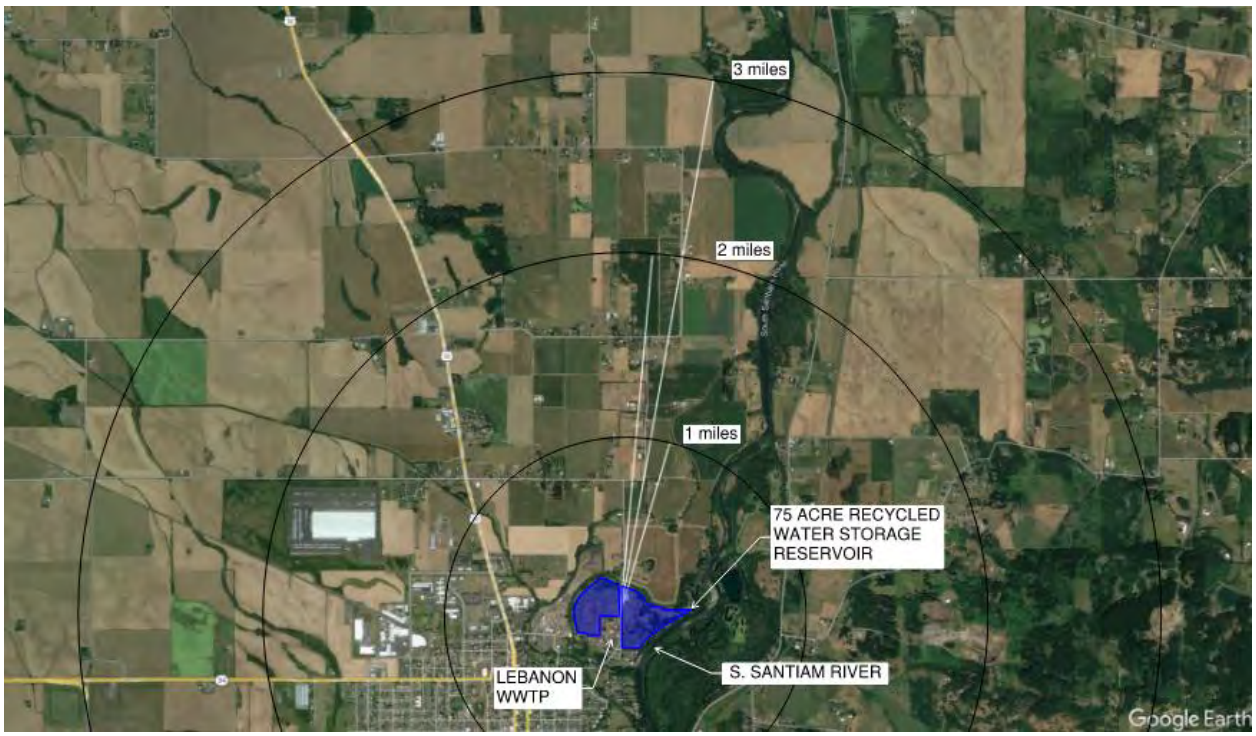


Figure 5-10: Recycled Water Proposed Improvements

To implement the recycled water program, the City would need to develop a Recycled Water Use Plan meeting DEQ requirements that would 1) describe the proposed reuse site(s), including soil, topography, and groundwater conditions, 2) project future recycled water flows, and 3) review crop agronomic data to demonstrate that the proposed application rates would not adversely impact surface or ground water quality at the site.

Two discharge alternatives were considered for initial screening of recycled water alternatives. Alternative 1 assumed that 50% of WWTP effluent from June through September would be conveyed to the reuse site with the balance being discharged to the river. During the months of May and October, 100% of WWTP effluent would be stored. Effluent stored in May would be discharged to the irrigation system during June through September. Effluent stored in October would be gradually discharged to the river over the winter months.

Alternative 2 assumed that no effluent would be discharged from May through October. Irrigation would occur May through September. A portion of the effluent in May would be stored and discharged to irrigation in June through September. Because of the minimal irrigation demand in October, it was assumed that all effluent during that month would be stored and then discharged over the winter months.

Alternatives included the potential use of two agricultural crops: alfalfa and tall fescue. Two precipitation conditions were also considered: average annual conditions and 1-in 10-year rainfall conditions.

Projected recycled water quantities for both alternatives are summarized in Table 5-6.

Table 5-6: Monthly Recycled Water Quantities

Discharge Alternative[b]	Recycled Water Quantities, MG[a]						
	May	June	July	August	September	October	Total
1[b]	0	41	42	42	41	0	165
2[c]	217	81	84	84	81	0	630

Notes:

(a) Projected design year 2045 effluent flows are shown assuming MMDWF for May and ADWF for June through October.

(b) Alternative 1 assumes that 50% of WWTP effluent June through September would be irrigated with the remaining 50% being discharged to the river. During May, 100% of WWTP effluent would be stored and then discharged to irrigation June through September. During October, 100% of effluent would be stored and then discharged to the river November through April.

(c) Alternative 2 assumes that no effluent would be discharged to the river for months May through October. Irrigation would occur in May through September. All effluent would be stored in October and then discharged to the river November through April.

Monthly irrigation acreages for each alternative considered are presented in Table 5-7. All alternatives assumed use of a 10-ft deep, 75-acre storage reservoir.

Table 5-7: Acres Under Irrigation by Month[a]

Crop	Rainfall	Discharge Alternative[b][c]	May	Jun	Jul	Aug	Sep	Oct
Alfalfa	Avg	1	0	234	234	234	234	0
		2	923	923	923	923	923	0
	1 in 10 yr	1	0	410	410	410	0	0
		2	0	1,587	1,587	1,587	0	0
Tall Fescue	Avg	1	0	212	212	212	212	0
		2	801	801	801	801	801	0
	1 in 10 yr	1	0	311	311	311	0	0
		2	0	1,202	1,202	1,202	0	0

Notes:

(a) Acreages are based on evapotranspiration data published in Oregon State University Crop Water Use and Irrigation Requirements (1999) and rainfall data published by the Oregon State Prism Climate Group. Agronomic rates for nutrient loading were obtained from Oregon State University Fertilizer Guides to determine of nitrogen loading exceeded agronomic rates. No instance of excessive loading was found.

(b) Alternative 1 assumes that 50% of WWTP effluent in June through September would be irrigated with the remaining 50% being discharged to the river. During May, 100% of WWTP effluent would be stored and then discharged to irrigation June through September. During October, 100% of effluent would be stored and then discharged to the river November through April.

(c) Alternative 2 assumes that no effluent would be discharged to the river for months May through October. Irrigation would occur May through September. All effluent would be stored in October and then discharged to the river November through April.

(d) Precipitation during the 1- in 10-year condition is projected to meet or exceed irrigation demands in May and September. WTTP during these months and the 1- in 10-year rainfall condition must thus be stored.

The acreage requirements shown for May and October assume utilization of a 10-ft deep, 75-acre storage reservoir. The volume of the storage reservoir was set to limit the number of acres under irrigation and was assumed to hold the design MMDWF for May during 1- in 10-year rainfall conditions. The surface area and depth of the reservoir were set to maximize evaporation over the course of the summer to reduce the quantity of water to be irrigated. While providing storage significantly reduces the number of acres needed for irrigation, its benefit begins to diminish at about 75 acres when further increases have little impact on the area needed for irrigation.

Because the implementation of a recycled water program would require development of a Recycled Water Use Plan and an NPDES permit renewal, it is often more advantageous to draft a plan that encompasses all potential future beneficial uses and sites to serve the permit holder with more flexibility. For these reasons, the more conservative Alternative 2 was retained for further evaluation.

5.4 Solids Stream Treatment Alternatives

The following provides a description of alternatives considered to improve the WWTP's solids stream treatment processes and those selected for further evaluation.

5.4.1 Thickening Alternatives

Thickening processes increase the solids concentration of waste sludge streams. This decreases the volume of waste sludge that must be treated in downstream stabilization processes (e.g., anaerobic digestion, aerobic digestion, etc.) thereby decreasing the size of these processes. The WWTP currently thickens WAS prior to further treatment. The continued use of thickening to increase the solids concentration of waste sludge to 4% was assumed when evaluating the solids stabilization alternatives considered in this Plan.

The WWTP's existing gravity belt thickener (GBT) is 30 years old having been installed in 1993. While City staff report that the existing GBT is functional and it has sufficient capacity to meet year design year 2045 loading conditions, it is well beyond the 20-year useful life typically assumed for mechanical equipment. Because of the GBT's age and presumed need for replacement, a screening analysis was done to identify one thickening technology for use in developing overall project alternatives. The thickening alternatives considered were 1) rotary drum thickeners, 2) GBTs, 3) centrifuges, and 4) dissolved air flotation.

5.4.1.1 Rotary Drum Thickener

In rotary drum thickening, sludge conditioned with polymer is pumped into a rotating drum screen. Free water drains through the screen openings and collects in a trough underdrain. Thickened sludge is conveyed through the rotating drum and out the discharge end via a continuous internal screw or angled flights. The drum is sometimes inclined to aid in thickening. An example of a single-drum style rotary drum thickener (RDT) is shown on Figure 5-11.

Figure 5-11: Rotary Drum Thickener (Source: FKC)

RDTs have slow rotational speeds and are typically less than 30 revolutions per minute (rpm). As a result, they have relatively low operational noise, require less power, and have less wear on moving parts than other thickening alternatives. RDTs can be fully enclosed to simplify odor control and can be operated with minimal attention due to their simplicity and reliability. Like GBTs, thickening performance depends strongly on effective coagulant addition of the inlet sludge. RDTs can thicken WAS to 4% to 9% total solids (TS).

5.4.1.2 Gravity Belt Thickener

The WWTP currently uses the GBT shown on Figure 5-12 to thicken WAS prior to digestion. As a thickening technology, GBTs were originally developed based on the gravity section of a belt filter press, where most of the dewatering occurs. This process utilizes gravity to drain water from the polymer-conditioned sludge as it travels over a filter belt that is under tension. The water is collected beneath the belt as the concentrated sludge is carried above the belt to the discharge end of the thickener. GBTs can typically thicken inlet sludge to 4% to 8% TS with the appropriate polymer addition. The correct type and dosage of polymer is critical to achieving target total solids percentages.

Figure 5-12: Existing Gravity Belt Thickener

GBTs consume relatively low amounts of power and operate quietly. However, they are more mechanically complex than RDTs and typically require more space. GBTs are more sensitive to grease and high organics content than other technologies, as grease can blind the belt and lower the total surface area available to drain the sludge. As a result, GBTs require significant operator attention if feed solids might vary in concentration or organics content.

5.4.1.3 Centrifuge

Centrifuge thickeners consist of a central scroll conveyor rotating inside a cylindrical rotating bowl. Solids are conveyed between the annular space between the bowl and the scroll. Centrifuge thickeners separate sludge particles from water by inducing centrifugal forces with high-speed rotation of the bowl. The bowl and the scroll are controlled by separate drives and rotate at different speeds.

The scroll continuously conveys solids to a discharge end. Water leaves the unit opposite the solids discharge end and returns to the liquid stream process for treatment. The bowl is

conically shaped which helps lift solids out of the liquid, allowing them to concentrate on an inclined surface before being discharged.

Centrifuges typically require less polymer and are the smallest of the thickening technologies considered. Like RDTs, they are fully enclosed making it easier to control odors. Energy use is higher than other technologies. Their high rotational speed (typically 4,000 to 6,000 rpm for the solid bowls) also means greater noise levels and greater potential for higher wear and more maintenance. Relatively complex maintenance requirements may be beyond the abilities of WWTP maintenance staff.

5.4.1.4 Dissolved Air Floatation

Dissolved Air Flotation Thickening (DAFT) is a solids-liquid separation technology that utilizes pressured air, rather than mechanical means, to thicken sludge. Pressurized air is introduced into liquid-phase sludge. The sludge is directed into a flotation tank where it is depressurized. Microscopic air bubbles come out of the solution and attach to solid flocs. These flocs float to the surface of the tank, where they are mechanically skimmed from the tank into a float box and removed as thickened solids. The liquid underflow is recycled to the head of the process or sent back to the head of the plant. Heavier solids settle to the bottom of the tank and are removed. Flotation tank designs can be circular or rectangular. Figure 5-13 is a representation of a rectangular type DAFT.

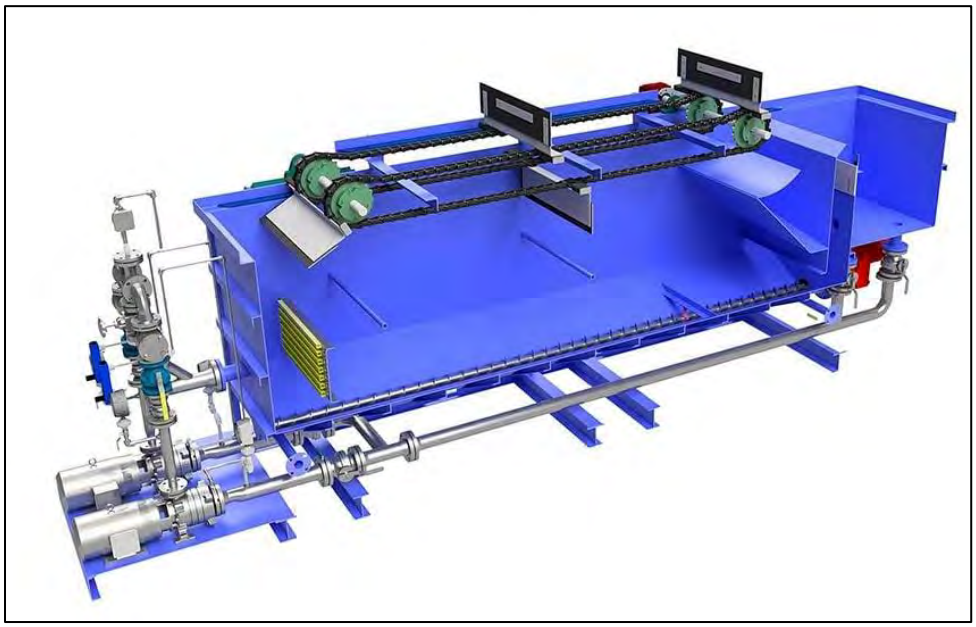


Figure 5-13: Rectangular DAFT Tank (Source: WesTech)

A DAFT is most suitable for WAS or extended-aeration solids where the specific gravity of solids is close to 1. Primary or trickling filter solids are more difficult and less economical to remove using DAFT. Polymer conditioning of sludge inflow is commonly used to increase solids flotation. The DAFT systems often are packaged with an air compressor system, and can be noisy. DAFT systems are mechanically complex, more operationally complex, and require more maintenance than other separation technologies. A DAFT can produce thickened solids with 3% to 6% TS.

5.4.1.5 Thickening Alternatives Screening

A matrix-based screening analysis of four thickening alternatives was performed to identify one generic thickening alternative for further development of overall recommended improvements for the WWTP. Results of this analysis are summarized in Table 5-8 below.

Table 5-8: Thickening Alternatives Screening Analysis

Thickening Alternative	Capital Cost (25%)	O&M Cost (25%)	Technology Maturity (12%)	Operational Complexity (12%)	Ease of Permitting (8%)	Potential for Community Objection (8%)	Beneficial End Use (10%)	Weighted Score
Rotary Drum Thickener	4	3	5	4	5	5	5	**4.1**
Gravity Belt	3	2	5	4	5	5	5	**3.6**
Centrifuge	4	2	5	3	5	5	5	**3.8**
DAFT	3	2	5	3	4	4	5	**3.4**

As shown in Table 5-8, rotary drum thickening received the highest score and was thus selected as the thickening technology assumed when developing the recommended overall treatment plant improvement project. Although rotary drum thickening was selected as the recommended thickening technology for the purposes of this Plan, ultimate selection of a specific technology should be done during the preliminary design phase of the recommended project.

Considerations that factored into the lower ranking of the thickening alternatives removed by this initial screening evaluation included the following:

- Gravity Belt Thickening. Capital and O&M costs for GBTs were deemed to be higher than RDTs. GBTs are also more mechanically complex.
- Centrifuge. O&M costs were deemed to be higher with greater power requirements. Centrifuges are also more operationally complex with more sophisticated drive and control systems needed for the high speeds involved.
- DAFT. Capital and O&M costs for DAFTs were deemed to be higher than RDTs. DAFTs require the most space of the technologies considered and would most likely not fit within the existing thickening building.

5.4.2 Stabilization Alternatives

Solids generated and collected in the liquid stream often have high water and volatile solids content, making them unstable and likely to putrefy. This in turn makes them more attractive to vectors (like vermin or insects) as well as odorific. Stabilization processes (such as digestion, lime addition, composting, etc.) are often employed to reduce the ability of the solids to putrefy. Digestion processes result in a net reduction of total solids mass reducing the amount of

downstream handling required for their ultimate beneficial use or disposal. Depending on the type and degree of stabilization, biosolids may be disposed of or beneficially reused (as soil amendment, marketable fertilizer, etc.).

The City currently thickens WAS before partially stabilizing it in an existing 0.53 MG aerobic digester. The existing aerobic digester does not have sufficient volume and is unable to reliably produce Class B biosolids. As a result, slurry lime is routinely added to further stabilize the sludge and meet regulatory requirements for Class B biosolids.

This section reviews alternatives considered to improve the WWTP's solids stabilization process and describes rationales for retaining or eliminating them from further evaluation.

5.4.2.1 Aerobic Digestion

Aerobic digestion is employed at the City's existing WWTP and is the biological conversion of organic matter in the presence of oxygen to carbon dioxide, water, ammonia, new cellular biomass, and energy. In the presence of adequate oxygen and declining food supply, the microorganisms convert their own protoplasm to energy for cell maintenance. Aerobic digestion is an exothermic biological process that occurs within an optimal range between 20°C and 35°C (68°F to 95°F). Aerobic digestion typically reduces volatile solids by 35% to 50%.

Aerobic digestion is commonly used at treatment plants with flow rates less than 5 MGD but has been installed in plants with flows up to 50 MGD. The process requires higher energy to operate aeration equipment compared with anaerobic digestion. The space requirement is also slightly larger than anaerobic. However, the process is considered more stable under variable feed conditions and less labor intensive to operate than anaerobic digestion.

Federal regulatory requirements for use of aerobic digestion in meeting Class B requirements consist of the following:

- 40 CFR 503.32(b)(3) – Aerobic digestion is allowed as a process to significantly reduce pathogens (PSRP) for Class B biosolids. Mean cell retention time (MCRT) and temperature must be between 40 days at 20°C and 60 days at 15°C.

- 40 CFR 503.33(b)(1) – When aerobic digestion achieves a minimum volatile solids destruction of 38%, it may be used to satisfy the vector attraction reduction (VAR) requirements of this regulation.

- 40 CFR 503.33(b)(3) – When aerobic digestion does not achieve a minimum volatile solids destruction of 38%, additional bench testing may be used to satisfy VAR requirements of this regulation.

Digesters can be covered or uncovered. Covering aerobic digesters can help maintain higher temperatures during the winter to better achieve desired volatile solids destruction in less time. Covering may also improve the control of odors.

Conventional aerobic digestion uses air to provide oxygen to the sludge for the cell reproductive process. Oxygen transfer is typically achieved through diffused air but can also be supplied by submersible jet aeration. Ammonia is oxidized to nitrate in the aerobic process, causing reduced alkalinity and pH. The air can be cycled off to promote denitrification and lower the nitrate

concentrations in the return stream. This can help stabilize the alkalinity and raises the pH by producing carbon dioxide.

Digester mixing is needed to prevent solids from settling out of suspension. Mixing energy can be provided by the aeration air and typically controls the amount of air needed. Digester mixing can also be provided by mechanical devices such as surface mounted or submersible mixers, jet (pump) mixing, or draft tubes.

Advantages of aerobic digestion include its ability to produce Class B biosolids at ambient temperatures. It is a relatively simple process compared to other stabilization technologies with fewer mechanical systems. Aerobic digestion is also one of the most used stabilization processes especially for smaller sized wastewater treatment facilities. Because methane is not generated there are less safety concerns as compared to anaerobic digestion.

Disadvantages of aerobic digestion include its relatively high energy use. Capital costs can be greater than for anaerobic digestion for mid-to-large treatment facilities. Stabilization is reduced at colder water temperatures (i.e., winters in the Pacific Northwest) resulting in the need for longer detention times or addition of covers to help maintain temperature. Aerobic digestion also does not allow for the generation and reuse of biogas.

5.4.2.2 Autothermal Thermophilic Aerobic Digestion (ATAD)

Autothermal Thermophilic Aerobic Digestion (ATAD) is aerobic digestion conducted at higher temperatures than conventional aerobic digestion. The typical operating range for the process is between 55°C and 75°C (131°F and 167°F) as opposed to 20°C and 35°C (68°F and 95°F) for conventional aerobic digestion. Because the aerobic digestion reaction is exothermic, all necessary heat can be provided by the process itself when proper feed conditions are met. ATAD reactors are covered and insulated to retain the heat generated by the process to help ensure correct temperatures are maintained.

Advantages of the ATAD process include a lower residence time and associated tank volume needed to meet regulatory requirements for Class B biosolids. It can also be used to produce Class A biosolids in a shorter amount of time than needed by conventional aerobic digestion to meet Class B requirements. The MCRT for ATAD to meet Class A requirements is 10 days at temperatures between 55°C and 60°C (131°F and 140°F) compared to 40 days at 20°C (68°F).

However, disadvantages of ATAD include the greater complexity of the process (as compared to conventional aerobic digestion). ATAD requires more equipment including tank insulation, post-process cooling and storage tanks, and off-gas treatment systems. Foaming is a more prevalent issue with ATAD than with conventional aerobic digestion and dewatering of the resulting sludge can be more difficult. The process also has much greater potential for odor.

5.4.2.3 Anaerobic Digestion (Mesophilic)

Anaerobic digestion is a multi-step biological process wherein anaerobic bacteria convert organic matter into methane and carbon dioxide (sometimes called biogas) in the absence of oxygen. The process stabilizes the organic matter in wastewater solids and reduces pathogens. The destruction of volatile solids and conversion to biogas results in an overall reduction of total solids quantity. Volatile solids are typically reduced 40% to 65%. Anaerobic digestion requires more equipment and higher capital costs than aerobic digestion for smaller plants. However, it can become economically advantageous for plants with flows greater than 5 MGD, especially if

biogas is utilized to offset heating or energy costs (through co-generation of electricity, using a biogas-fired boiler to maintain process temperatures, etc.). Biogas production can be augmented by the inclusion of high-strength feedstocks delivered from outside the plant, such as Fats, Oils, and Grease (FOG) or food waste.

Typical anaerobic digesters are circular and covered with fixed steel or concrete covers or floating steel or membrane covers which allow some storage of digester gas in the digester. Figure 5-14 shows an example of a circular, fixed steel cover.

Figure 5-14: Fixed Steel Digester Cover (Source: OTI)

The footprint of anaerobic digesters is typically smaller than aerobic digesters for the same loading, as MCRT to meet Class B requirements in anerobic digesters is considerably lower than aerobic digesters. Under ideal process conditions, 15 days is needed for anaerobic digestion compared to 40 to 60 days for aerobic digestion. However, anaerobic digesters are more sensitive to process conditions and are more prone to upsets. Anaerobic digesters also require ancillary equipment such as biogas safety equipment; sludge mixers (by pump mixing, gas mixing, plunger mixing, propeller, etc.); boilers; and a heating system (such as a hot water loop and pump).

Conventional mesophilic digesters operate within a temperature range of 30°C to 38°C (85°F to 100°F). Because digestion rates are highly dependent on temperature, boilers and hot water loops are utilized to maintain a temperature setpoint. Dual-fuel boilers designed to combust both biogas and natural gas are typically used to provide heat. The use of biogas can offset some or even all the heating requirements of the digester. Besides temperature, other process conditions that can cause process upsets and must be monitored include pH, ammonia, heavy metals, sulfides, etc. that can cause inhibition of methane-forming bacteria. Feed stock for anerobic digesters is typically comprised of primary sludge and WAS. The use of primary treatment is essential to making anaerobic digestion cost effective and was assumed when evaluating anaerobic digestion for this analysis.

Federal regulatory requirements for anaerobic digestions use in meeting Class B requirements consist of the following:

- 40 CFR 503.32(b)(3) – Anaerobic digestion is allowed as a PSRP for Class B biosolids. MCRT and temperature must be between 15 days at 35°C to 55°C and 60 days at 20°C.

- 40 CFR 503.33(b)(1) – When anaerobic digestion achieves a minimum volatile solids destruction of 38%, it may be used to satisfy the VAR requirements of this regulation.

- 40 CFR 503.33(b)(2) – When anaerobic digestion does not achieve a minimum volatile solids destruction of 38%, additional bench testing may be used to satisfy VAR requirements of this regulation.

Advantages of anaerobic digestion when compared with aerobic digestion include the ability to meet Class B biosolids requirements with less energy and residence time (i.e., reactor volume). It can also generate biogas, which can be used to offset operational costs. Volatile solids destruction can exceed 55% resulting in lower sludge volumes reducing costs of downstream handling processes. Anaerobic digestion also has the potential to increase biogas production and revenue (via tipping fees) from outside feedstocks (e.g., FOG and food waste).

Disadvantages of anaerobic digestion can include higher operation and maintenance costs associated with sludge heating and mixing equipment. Primary treatment is needed to increase cost efficiency. Anaerobic digestion is also more complex (compared to aerobic digestion) and has a greater potential for process upset with a slower startup and recovery period following upsets. Finally, generation of biogas (i.e., methane) can present safety concerns.

5.4.2.4 Anaerobic Digestion (Thermophilic)

Thermophilic anaerobic digestion occurs at higher temperatures ranging between 50°C and 60°C (122°F to 140°F). The elevated temperature range increases the rate of anaerobic activity. Solids destruction and biogas output tend to be higher for thermophilic digestion than mesophilic digestion. Thermophilic digestion can produce biosolids that meet Class A regulatory requirements. Operation of thermophilic digesters increases digester stability, such that variations in FOG loading can be handled with lower potential of digester upset. The improved volatile solids destruction can result in a slight improvement in dewaterability.

While MCRT (i.e., reactor volume) is less for thermophilic digestion compared to mesophilic digestion, net energy and long-term operating and maintenance costs are higher. This technology is much less widely utilized than mesophilic digestion and is most advantageous in geographies where footprint space is limited (e.g., large metropolitan areas). Given its limited use, thermophilic digestion could be more difficult to permit.

5.4.2.5 Anaerobic Digestion (Thermal Hydrolysis)

Thermal hydrolysis is a bath-type pretreatment process that uses heat and pressure to treat primary sludge and WAS streams prior to anaerobic digestion. Pretreatment occurs in a two-stage process. In the first stage, the combined solids stream is pressurized and heated to a temperature of 340°F for 30 minutes. The material is then conveyed to a flash tank operating at atmospheric pressure. The drop-in pressure bursts the cells, releasing material that is more easily utilized by microbes in the digester increasing their activity. This increased microbial activity results in greater volatile solids destruction with an associated increase in gas production. Thermal hydrolysis meets regulatory requirements for Class A biosolids if used to treat both the primary sludge and WAS streams going into the digester. This pretreatment technology can also be utilized to just treat the WAS stream.

Advantages of thermal hydrolysis with anaerobic digestion include greater solids destruction and gas production. It also improves the ability to dewater the digested sludge, resulting in drier biosolids with a typical range of values between 28% and 32% solids. Thermal hydrolysis is also useful when a digester is approaching capacity limits and there are space constraints that would prevent construction of additional digester volume.

Although it has a proven record in Europe, thermal hydrolysis is considered an emerging technology with few installations within the United States. The process increases the complexity of the anaerobic digestion process and requires a specially trained operator to be onsite during operation of the boiler that produces steam. The thermal hydrolysis process also results in a higher level of ammonia in the dewatering side stream than conventional anaerobic digestion. This side stream is fed back to the head of the plant, increasing the ammonia in the liquid stream.

5.4.2.6 Stabilization Alternatives Screening

A matrix-based screening analysis of five stabilization alternatives was performed to identify two alternatives for further development of overall recommended improvements for the WWTP. Results of this analysis are summarized in Table 5-9 below.

Table 5-9: **Stabilization Alternatives Screening Analysis**

Stabilization Alternative	Capital Cost (25%)	O&M Cost (25%)	Technology Maturity (12%)	Operational Complexity (12%)	Ease of Permitting (8%)	Potential for Community Objection (8%)	Beneficial End Use (10%)	Weighted Score
Aerobic (Conventional)	5	3	5	5	5	4	5	**4.4**
ATAD	2	2	4	3	3	1	2	**2.4**
Anaerobic (Mesophilic)	2	4	5	4	5	4	5	**3.8**
Anaerobic (Thermal Hydrolysis)	1	1	2	1	2	2	3	**1.5**
Anaerobic (Thermophilic)	2	2	3	2	4	2	2	**2.3**

As shown in Table 5-9, aerobic digestion and anaerobic (Mesophilic) digestion received the highest score and were thus selected for further evaluation for implementation as part of a recommended overall treatment plant improvement project. The two selected technologies are commonly used and are very mature technologies that have been integrated into biosolids regulations at the state and federal levels. Given its long-standing use by the City and expectation for lower initial capital cost, aerobic digestion scored higher than anaerobic (Mesophilic) digestion.

ATAD, thermophilic digestion, and thermal hydrolysis were not retained for future consideration. Although these technologies can produce Class A biosolids, they are less commonly used and, in the case of thermal hydrolysis, can still be considered as emerging technologies. In general, they are more mechanically and operationally complex and are expected to be more costly both

in terms of initial capital and long-term operation and maintenance. These alternatives were thus removed from further consideration.

5.4.3 Dewatering Alternatives

Dewatering removes water from waste sludge streams and stabilized biosolids. Stabilized biosolids are typically dewatered to solids concentrations between about 15 and 25%. Doing so significantly reduces the weight and volume of material that must be hauled, stored, and potentially undergo further treatment.

A screening analysis of four dewatering technologies was done to identify one that would act as a generic dewatering alternative to be carried forward when developing overall project alternatives. These included 1) belt filter presses, 2) screw presses, 3) centrifuges, and 4) fan presses.

5.4.3.1 Belt Filter Press

Belt filter presses (BFPs) are dewatering devices that use the principles of chemical conditioning, gravity drainage, and mechanically applied pressure to dewater sludge. In a BFP dewatering process, polymer is added to flocculate the solids. The flocculated solids are then distributed on a gravity belt section where the free water drains through a moving, porous fabric belt (like that of a GBT). The solids are then pressed between two belts that move in a serpentine pattern over a series of rollers. Each pass over a roller provides additional pressing force and releases additional water. A scraper blade removes the final dewatered sludge cake from the belts. Filtrate is returned to the liquid treatment process and the dewatered solids (cake) are captured and transported for further treatment (i.e., Class A processes) or removed for Class B beneficial use or landfilled.

Stabilized solids can typically be dewatered to concentrations between 15% and 20% with 95% capture solids. BFPs are not constructed as enclosed pieces of equipment and may be a source of significant odor.

In 2022, the City installed a new BFP manufactured by OR-TEC Incorporated. The unit has a design capacity of 600 lbs dry solids per hour. Assuming that the BFP is operated 40 hours per week, a capacity of about 650 lbs of dry solids per hour is needed to meet design year 2045, maximum 30-day solids production conditions. Because this value is very close to the capacity of the installed BFP, the existing BFP should meet design year 2045 conditions. However, WWTP staff have reported difficulties with inconsistent dewatering results and have been unable to achieve the manufacturer's stated capacity. Figure 5-15 presents an illustration of a BFP like that currently in use by the City.

Figure 5-15: Belt Filter Press (Source: OR-TEC)

Advantages of BFPs include the City's familiarity with the equipment and ability of plant staff to maintain the equipment. Because the existing BFP was only recently installed and has capacity to meet anticipated design year 2045 conditions, cost of the BFP dewatering alterative will be the lowest of the alternatives being considered.

Disadvantages of BFPs include potential for odor unless properly ducted and ventilated. BFPs can also require a higher level of operator attention than other technologies to changing sludge feed characteristics. They are more mechanically complex than dewatering technologies like screw presses and fan presses and they require more space than other technologies.

5.4.3.2 Screw Press

A screw press consists of a tapered screw with a surrounding screen. Solids conveyed down the length of the screw are dewatered through compression between the tapered screw and the reducing diameter of the surrounding screen. In screw press operation, polymer is added to flocculate the solids ahead of the press. After polymer addition, the sludge flows into a flocculation tank with a slow speed mixer. The conditioned sludge then enters a head box on the top of the unit before flowing into the screw press. Solids move through the unit along the length of a tapered screw enclosed by an outer screen. In the initial screen sections, water drains by gravity. As solids move along the screw, the internal pressure increases, forcing water to drain out through the outer screen. An example screw press installation is presented in Figure 5-16 below.

Figure 5-16: Screw Press

Screw presses are slow moving machines with rotational speeds less than 2 rpm. Stabilized solids are typically dewatered to concentrations between 16% and 20% with 95% capture solids. Screw presses are enclosed within a stainless-steel enclosure with ports provided for connection of foul air ducting.

Advantages of screw presses include the slow speed at which they operate. This results in lower noise levels and less wear on moving parts. Energy use is lower than other technologies such as centrifuges. The press is enclosed, minimizing odor control requirements. The simple operation of a screw press allows for an automated shutdown, making non-staffed operation feasible. The units can also be started and stopped each day for shift operation with minimal cleanup.

While smaller than BFPs, screw presses require more space than centrifuges. Because the City has a relatively new BFP that meets design year 2045 conditions, installation of a new screw press would be more costly.

5.4.3.3 Centrifuge

As previously discussed, basic centrifuge operation and unit construction is similar for dewatering and thickening applications. However, centrifuges used for dewatering have lower solids throughput than those used for thickening. Centrifuges can produce dewatered cake with higher solids concentrations than other dewatering technologies typically achieving 20% to 25% with solids capture rates greater than 95%. Higher power and maintenance costs associated with centrifuge dewatering can be offset where higher percentage solids drastically reduce plant operating costs (e.g., solids are hauled for disposal following dewatering and a tipping fee is charged). Conditioning with polymer is required to prevent floc shear and to improve centrate quality and solids capture.

Centrifuge dewatering is a closed process, which makes for easy containment of odors. The liquids and solids discharge from the bottom of the machine by gravity. The controlled discharge and containment of the dewatering mechanism allow for localized odor control at the liquids and

solids discharge ports. Centrifuges require the least space of the dewatering technologies considered and can produce dewatered cake with the highest solids concentration.

Disadvantages of centrifuges include higher operating speeds and associated noise. Centrifuge maintenance is more difficult and may be beyond the capabilities of WWTP staff. Dewatered cake from a centrifuge can be more odorous and odor control should be provided on the cake and centrate outlets. Energy use is also higher than the other technologies considered.

5.4.3.4 Rotary Fan Press

Rotary fan presses (RFPs) use continuous pressure and rotational speed to remove water and produce dewatered cake. Solids are fed into the inlet and travel slowly through the interior channel in a circular fashion. Dewatering occurs through central screens which drain the filtrate by gravity to an outlet located at the bottom of the unit. The cake is extruded as the screens turn slowly and the material behind it builds and advances. Filtrate is returned to the plant's liquid stream treatment system. RFPs typically produce 14% to 16% dewatered cake (assuming stabilized feed sludge). Figure 5-17 provides an example of an RFP.

Figure 5-17: Rotary Fan Press (Source: Evoqua)

RFPs are enclosed with almost no external exposure to pathogens, gases, odors, or process material. Like screw presses, they require minimal operator attention, operate at low speeds, have low energy and maintenance costs, and low operating noise. Additional press channels can be added in line to a single motor/gear unit to increase capacity in the future.

RFPs require more space than centrifuges. Because the City has a relatively new BFP that meets design year 2045 conditions, installation of a new RFP would be more costly than continuing to use the existing BFP.

5.4.3.5 Dewatering Alternatives Screening

A matrix-based screening analysis of four dewatering alternatives was performed to identify one generic dewatering alternative for further development of overall recommended improvements for the WWTP. Results of this analysis are summarized in Table 5-10 below.

Table 5-10: Dewatering Alternatives Screening Analysis

Dewatering Alternative	Capital Cost (25%)	O&M Cost (25%)	Technology Maturity (12%)	Operational Complexity (12%)	Ease of Permitting (8%)	Potential for Community Objection (8%)	Beneficial End Use (10%)	Weighted Score
Belt Filter Press	5	4	5	4	5	5	3	**4.4**
Screw Press	2	5	5	5	5	5	4	**4.2**
Centrifuge	2	3	5	4	5	5	5	**3.6**
Fan Press	2	4	4	4	4	5	4	**3.6**

As shown in Table 5-10, the BFP dewatering technology received the highest score and was selected to be carried forward as the assumed dewatering technology. The main differentiator between the BFP and screw press alternatives was that the City recently (2022) installed a new BFP. The rated capacity of the BFP should be sufficient to meet anticipated design year 2045 solids production values. Scoring for capital cost was thus skewed heavily in favor of continuing use of the BFP. However, WWTP staff have reported issues with the new BFP experiencing inconsistent dewatering results and an inability to meet the manufacturer's rated capacity. Should the City's efforts to improve the performance of the BFP fail, new dewatering facilities will be needed. In this case, the screw press alternative would be the recommended alternative.

5.4.4 Class B Beneficial Use Alternatives

Two Class B biosolids beneficial use alternatives were evaluated with one being selected for further consideration. The Class B beneficial use alternatives were 1) liquid land application and 2) dewatered cake land application.

5.4.4.1 Liquid Land Application

This alternative would continue the City's current practice of land applying Class B liquid biosolids. The City has five authorized application sites with a total combined area of approximately 370 acres. The application window is dependent upon the weather conditions and specific site conditions but generally extends from March through October. Liquid biosolids are applied at a solids concentration ranging between 3% and 5%. The City uses a City-owned application truck to land apply liquid biosolids via a splash plate or feeding the liquid biosolids into a large spray irrigation canon.

The City is challenged by the lack of storage for biosolids in the wintertime (approximately 5 months is needed) and currently must dewater biosolids for landfilling. To eliminate the need for wintertime landfilling, this alternative would need to include construction of liquid biosolids storage facilities (e.g., facultative sludge lagoon). Assuming annual land application of liquid biosolids, sludge dredging facilities would also be needed.

5.4.4.2 Dewatered Cake Land Application

Like the City's existing liquid land application program, a dewatered cake land application alternative would continue to utilize the City's five authorized biosolids application sites. However, instead of applying liquid biosolids at concentrations between 3% and 5%, the biosolids would be dewatered to between 15% and 20% solids concentration prior to land application. Doing so would significantly reduce the volume of biosolids that would be hauled and land applied. For example, increasing the solids content of biosolids from 4% to an 18% dewatered cake would reduce its volume by more than 75%. This would dramatically reduce hauling and labor costs when compared with a liquid land application program.

For the purposes of this evaluation, it was assumed that the WWTP's existing BFP would be used to dewater stabilized Class B biosolids. As noted elsewhere, WWTP staff have reported issues with the new BFP experiencing inconsistent dewatering results and an inability to meet the manufacturer's rated capacity. Should the City's efforts to improve the performance of the BFP fail, new dewatering facilities will be needed. Other equipment and facilities that would be needed include a dewatered cake building for wintertime cake storage, cake conveyors, front end loader, and slinger truck.

5.4.4.3 Class B Beneficial Use Alternatives Screening

A matrix-based screening analysis of the two Class B beneficial use alternatives was performed to select one alternative for use in comparing overall biosolid treatment and handling options for potential implementation by the City. Results of this analysis are summarized in Table 5-11 below.

Table 5-11: Class B Beneficial Use Alternatives Screening Analysis

Dewatering Alternative	Capital Cost (25%)	O&M Cost (25%)	Technology Maturity (12%)	Operational Complexity (12%)	Ease of Permitting (8%)	Potential for Community Objection (8%)	Beneficial End Use (10%)	Weighted Score
Dewatered Cake Land Application	4	3	5	3	1	2	4	**3.4**
Liquid Land Application	4	2	5	2	2	2	4	**3.1**

As shown in Table 5-11, dewatered cake land application received the highest score and was selected to be carried forward as the assumed Class B beneficial use alternative. Dewatered cake is preferable to liquid land application primarily because it should result in significantly less labor and costs associated with transportation to/from land application sites (i.e., potentially 75% lower solids volume). Capital costs were anticipated to be similar when comparing construction of facilities needed for liquid solids storage versus dewatered cake storage. Ease of permitting for both alternatives received low scores. This was because of the risk associated with the potential for additional regulatory restrictions and public perception concerns. The

dewatered cake alternative scored lower in this category because of changes that would be needed in the City's biosolids management plan to allow for land application of dewatered cake.

5.4.5 Class A Beneficial Use Alternatives

The City has stated a desire to produce Class A biosolids. Doing so would result in greater control by the City and reliability for disposal (e.g., beneficial use) of their biosolids. As will be described below, federal and state regulatory requirements for Class A biosolids as contained in EPA's 40 CFR Part 503 and DEQ's OAR 340-050 vary depending on the treatment technology employed.

A screening analysis of three Class A treatment alternatives was done to identify one that would act as a generic Class A alternative that would be carried forward when developing overall project alternatives. Technologies that were evaluated included 1) composting, 2) thermal drying, and 3) lime/heat pasteurization.

5.4.5.1 Composting

Composting is an exothermic biological process that utilizes naturally occurring microbes to convert organic solids under aerobic conditions into stabilized end products. The heat generated during the process is maintained at specific levels and times to achieve pathogen reduction and vector attraction requirements contained in the Class A biosolids regulations. The final composting end product is highly marketable and desirable for use by the public as a valuable soil amendment.

There are four commonly used methods of composting. These include: 1) aerated static pile, 2) covered aerated static pile (CASP), 3) windrow, and 3) in-vessel systems. Each method involves mixing dewatered biosolids with a "bulking" material that provides carbon and increases the porosity of the mixture to facilitate oxygen and heat transfer. Bulking material can include wood waste, hog fuel, green waste, or yard debris. The resulting mixture is placed in a vessel or covered pile where microbial activity causes the temperature of the mixture to rise during an "active composting" period. Mechanically aerated systems use forced air to keep the composting process within an optimum temperature range to help facilitate the composting process. After active composting, the material is cured and distributed for public use.

Federal regulatory requirements applicable to composting in meeting Class A biosolids requirements consist of the following:

- 40 CFR 503.32(a)(7) – Either the density of fecal coliform in the sewage sludge shall be less than 1000 Most Probable Number (MPN) per gram of total solids (dry weight basis), or the density of *Salmonella,* sp. bacteria in the sewage sludge shall be less than 3 MPN per 4 grams of total solids (dry weight basis).

- 40 CFR 503 Appendix B – Processes to Further Reduce Pathogens (PFRP). Using either the within-vessel composting method or the static aerated pile composting method, the temperature of sewage sludge is maintained at 55°C (131°F) or higher for 3 consecutive days.

- 40 CFR 503.33(b)(5) – Vector Attraction Reduction (VAR). Aerobic treatment of the sewage sludge for at least 14 days at over 40°C (104°F) with an average temperature of over 45°C (113°F).

An example of a CASP is shown below on Figure 5-18.

Figure 5-18: Covered Aerated Static Pile Composting System

Composting is a relatively simple, reliable technology that is widely used for Class A biosolids production. It is a relatively low-cost technology that can be phased in over time. Composted biosolids are highly marketable and typically well accepted by the public. Energy use is the lowest of the Class A alternatives under consideration. However, because of the addition of bulking agents, the volume of solids for disposal is much higher than other alternatives. This means that the footprint of composting operations is higher as is the amount of material that must ultimately be distributed for beneficial use. Composting is a labor-intensive process and can require additional operating staff for successful implementation.

A Class A EQ biosolids composting facility is currently being operated by the City of Albany, Oregon. One variation of the composting alternative is to form an intergovernmental partnership with Albany to produce Class A biosolids at a shared facility. This would entail hauling Class B biosolids to Albany for further treatment to achieve Class A EQ biosolids product.

In this partnership, the City would be able to minimize capital improvements needed to implement a Class A composting (or other Class A technology) of its own. There is a concern with loss of control and the reliance on another entity for the management of City biosolids. Albany's existing composting facility does not have adequate capacity to accept the City's biosolids. Thus, the partnership would require expansion of Albany's existing facility.

5.4.5.2 Thermal Drying

Thermal drying technology removes water via evaporation from dewatered biosolids, dramatically reducing its weight and volume. The high temperatures utilized by a dryer ensure that the EPA time and temperature requirements for Class A biosolids are met.

Federal regulatory requirements applicable to thermal drying in meeting Class A biosolids requirements consist of the following:

- 40 CFR 503.32(a)(7) – Either the density of fecal coliform in the sewage sludge shall be less than 1000 MPN per gram of total solids (dry weight basis), or the density of *Salmonella,* sp. bacteria in the sewage sludge shall be less than 3 MPN per 4 grams of total solids (dry weight basis).

- 40 CFR 503 Appendix B – Processes to Further Reduce Pathogens (PFRP). Sewage sludge is dried by direct or indirect contact with hot gases to reduce the moisture content of the sewage sludge to 10% or lower. Either the temperature of the sewage sludge particles exceeds 80°C (176°F) or the wet bulb temperature of the gas in contact with the sewage sludge as the sewage sludge leaves the dryer exceeds 80°C (176°F).

- 40 CFR 503.33(b)(8) – Vector Attraction Reduction (VAR). Percent solids ≥ 90% prior to mixing with other materials.

Thermal drying can be accomplished by one of two main drying technologies: direct convection or indirect conduction. Direct dryers expose the biosolids to open flame or burner gases. Due to safety concerns, they were not considered in this evaluation. Indirect dryers physically separate the biosolids being dried from the heating medium. Indirect dryer types typically used for small to medium sized installations include paddle dryers and belt dryers.

Paddle dryers have jacketed shells through which the heated medium (typically oil) is circulated. The heated oil is also circulated through hollow-core paddles inside the dryer that provide mixing and additional heating surface for the biosolids. Evaporated water leaves the dryer vessel and passes through a condenser. The remaining exhaust air passes through odor control prior to discharge to the environment. Dried biosolids are discharged to a cooling conveyor and transported to storage. Because of the extended startup time needed to heat the drying vessel and paddles, paddle dryers are best used when continuous 24-hour operation over a period of multiple days is possible.

Belt driers convey biosolids through the drying vessel on a conveyor belt. Drying air is indirectly heated by a gas furnace or hot water boiler and is then passed through the heating vessel. Moisture from the biosolids evaporates into the heated air stream and is removed via an exhaust gas stream. Exhaust air passes through a condensing unit to remove water with the remaining exhaust gas passing through odor control prior to being discharged to the atmosphere. The heated/dried biosolids are discharged to a cooling conveyor that moves the Class A material to storage.

Although more costly than paddle dryers, belt driers do not take as long to start up and are better suited to intermittent operation (e.g., operating 8 hours per day). Because dryers are not typically operated unattended and the City's existing WWTP is not staffed 24 hours per day, intermittent operation was deemed to be a necessary requirement for drying. For this reason, belt dryers were used as the assumed dryer technology for this evaluation.

Figure 5-19 provides an example illustration of a belt dryer.

Figure 5-19: Thermal Drying – Belt Dryer (Source: Andritz)

Thermal drying generates the least volume of biosolids because of the high solids concentration (>90%) and the absence of bulking agents such as those associated with composting and lime addition processes. This greatly reduces the amount of storage needed as well as transportation costs. Class A biosolids generated by thermal drying can be marketed to many types of end users, as they are easily blended into landscape products.

However, the drying process is energy- and mechanically-intensive, and therefore expensive to operate. Additionally, the equipment has a per-session heat-up and cool-down cycle, and equipment manufacturers recommend that each operating session be extended as long as possible to minimize the number of these cycles. Finally, because of the biosolid's high organic content, dust generated during the drying cycle is flammable. Precautions must be taken to design the heat-drying process, equipment, and storage to minimize the potential for explosion or fire.

5.4.5.3 Lime/Heat Pasteurization

Lime/heat pasteurization is a method that uses both lime and heat to meet VAR and PFRP requirements for Class A biosolids. Federal regulatory requirements applicable to lime/heat pasteurization in meeting Class A biosolids requirements consist of the following:

- 40 CFR 503.32(a)(7) – Either the density of fecal coliform in the sewage sludge shall be less than 1000 MPN per gram of total solids (dry weight basis), or the density of *Salmonella,* sp. bacteria in the sewage sludge shall be less than 3 MPN per 4 grams of total solids (dry weight basis).

- 40 CFR 503 Appendix B – Processes to Further Reduce Pathogens (PFRP). The temperature of the sewage sludge is maintained at 70°C (158°F) or higher for 30 minutes or longer.

- 40 CFR 503.33(b)(6) – Vector Attraction Reduction (VAR). Addition of sufficient alkali to raise the pH to at least 12°C at 25°C (77°F) and maintain a pH ≥ 12 for 2 hours and a pH ≥ 11.5 for 22 more hours.

Several lime/heat pasteurization systems exist. One example is the lime pasteurization system manufactured by RDP Technologies. In this system, dewatered solids/cake is heated prior to lime addition to enhance the hydration reaction that occurs during “lime slaking” and optimize its exothermic effect. Lime is then augered from silos, mixed with dewatered solids, and the mixture is conveyed to the system’s pasteurization vessel. The pasteurization vessel consists of an insulated belt conveyor and electrical heating elements to raise the sludge temperature. The solids/lime mixture is held at 70°C (158°F) for at least 30 minutes in the pasteurization vessel to produce Class A biosolids. The final product solids concentration is approximately 35% after pasteurization.

Another example of a lime heat pasteurization system is manufactured by FKC. Major components include a flocculation tank, rotary screen thickener, polymer system, and steam heated screw press. Biosolids are fed to the system from a digester and thickened in a rotary drum screen. Lime is added to the liquid biosolids and mixed in the lime addition tank to raise the pH to meet VAR requirements. In the screw press, the thickened biosolids are simultaneously dewatered and heated to meet PFRP requirements. In the screw press, the elevated pH and temperature improve dewaterability which results in cake dryness ranging from 35% to 50% total solids. FKC’s lime/heat pasteurization system is shown on Figure 5-20 below.

Figure 5-20: Lime/Heat Pasteurization (Source: FKC)

Lime/heat pasteurization produces Class A biosolids with a significant lime content that is desirable for agricultural land application in Western Oregon where soils are typically acidic. However, the lime addition produces significantly more biosolids that must then be stored, transported, and handled for ultimate beneficial use. Other disadvantages include the cost of lime which has been volatile and has increased markedly in recent years. Lime/heat

pasteurization is also mechanically intensive, and lime handling can pose a health and safety risk for plant staff.

5.4.5.4 Class A Beneficial Use Alternatives Screening

A matrix-based screening analysis of the three Class A beneficial use alternatives was performed to select one alternative to carry forward for use in evaluating overall biosolid treatment and handling options for potential implementation by the City. Results of this analysis are summarized in Table 5-12 below.

Table 5-12: Class A Beneficial Use Alternatives Screening Analysis

Dewatering Alternative	Capital Cost (25%)	O&M Cost (25%)	Technology Maturity (12%)	Operational Complexity (12%)	Ease of Permitting (8%)	Potential for Community Objection (8%)	Beneficial End Use (10%)	Weighted Score
Composting								
Independent	4	3	5	3	5	3	5	**3.9**
Regional	4	3	5	4	1	3	5	**3.7**
Thermal Drying	3	4	4	5	5	5	4	**4.0**
Lime/Heat Pasteurization	3	1	3	1	4	2	3	**2.3**

As shown in Table 5-12, thermal drying received the highest score and was selected to be carried forward as the assumed Class A beneficial use alternative. Considerations that factored into ranking of the other Class A alternatives included the following:

- Composting. Composting was expected to have a lower initial capital cost than other alternatives considered. However, composting is labor intensive with additional staff needed to operate a full-scale composting system that would treat all biosolids generated by the City's WWTP. O&M costs were thus anticipated to be greater than those associated with thermal drying. Although composting is mechanically less complex than composting, the need for significantly greater staff attention resulted in a lower ranking for operational complexity. Potential for community objection associated with anticipated increase in truck traffic also resulted in a lower ranking in that category. Composting was ranked the highest in terms of beneficial end use because the marketability and public desirability of the final composting product was considered to the highest of the alternatives considered. Of note was the low ranking of the regional composting alternatives in terms of ease of permitting. This was due to the anticipated complexity and effort needed to develop and maintain an intergovernmental partnership with other communities.

- Thermal Drying. Thermal drying, specifically belt drying, received the highest ranking of the alternatives considered. Although initial capital costs were anticipated to be greater than composting (and equal with that of lime/heat pasteurization), long term O&M costs

should be lower due to lower labor costs. As noted above, although thermal drying is more mechanically complex than composting, it ranked higher in terms of operational complexity because of its lower need for operator attention. The final volume of the Class A material is many times less than with the other alternatives making storage, handling, and ultimate beneficial use significantly easier. While much less material is generated in terms of volume, the marketability and desirability of the final product from belt drying, while good, is not quite as high as that associated with Class A composting.

- Lime/Heat Pasteurization. This alternative ranked significantly lower than the other Class A alternatives considered. It is more costly with both initial capital and long-term O&M costs being equal to or greater than other alternatives. O&M costs are greater in large part because of the cost and market volatility of lime. It is operationally the most complex of the alternatives both in terms of mechanical complexity and the difficulties often associated with handling lime. Beneficial end use also ranked the lowest of the alternatives because of the difficulty in handling and limited demand for the final product.

5.4.6 Biosolids Disposal Alternatives

This alternative would simply entail landfilling all waste solids generated at the City's WWTP. Doing so would eliminate the need for capital improvements and the additional O&M associated with the Class B and Class A biosolids alternatives considered. Waste solids would be stored temporarily in the existing aerobic digester, dewatered using the City's existing BFP, and then transported for final disposal at the Coffin Butte Landfill (Coffin Butte). The City currently disposes wastewater screenings and excess dewatered wastewater solids that cannot otherwise be land applied at Coffin Butte. The landfill is in Benton County about 30 miles northwest of the WWTP.

The most significant advantage of this alternative would be its simplicity. The complexity and labor requirements associated with processes needed to generate Class B and Class A biosolids would be avoided. Initial capital costs would also be less of any other solids alternative.

Disadvantages of landfilling would include significant on-going costs associated with hauling and disposing of solids at Coffin Butte. Perhaps most significant of all is the decreasing capacity and willingness of the landfill to accept municipal wastewater sludge. This would most likely result in ever increasing tipping fees and ultimate inability to send solids to the landfill leaving the City with no means to dispose of solids being generated.

Section 6: Alternatives Selection

This section provides a summary of the alternatives selection process for improvements to the City's WWTP. This process includes both economic and non-economic analyses of the alternatives under consideration. An infiltration and inflow (I&I) cost effectiveness analysis is also presented that describes potential financial benefits of reducing I&I in relation to the recommended improvements.

6.1 Basis for Alternatives Selection

Assumptions used in the life cycle cost evaluation of alternatives are summarized in Table 6-1 below. Additional alternative specific cost assumptions are documented within the respective alternative section.

Table 6-1: **Life Cycle Cost Evaluation Assumptions**

Parameter	Value	Notes
Planning Period, years	20	Planning horizon assumed to be year 2045
Real Discount Rate, %	2.0	20-yr inflation free rate of return OMB[(a)] Circular A-94 Appendix C (December 12, 2022)
Construction Cost Index	13,175	ENR Cost Index, March 2023
Consumer Price Index	301.8	Bureau of Labor Statistics, March 2023
Annual Cost Escalation, %	6.5	Annual construction cost escalation until midpoint of construction
Construction Mid-Point, years	3	Used for annual cost escalation
Cost Contingency, %	30	Conceptual level estimate
Engineering and Administration, %	25	Cost for engineering as a percent of construction
Labor Cost, $/hr	53.43	Value provided by City
Utility Rates		
Electrical cost, $/kWh[(b)]	0.086	Value provided by City
Natural gas, $/therm[(c)]	1.18	Value provided by City
Asset Life Expectancy, years		For salvage value estimates
Earthwork	70	
Concrete	50	
Piping systems	30	
Mechanical equipment	20	
Electrical and controls	20	
Buildings	40	

Notes:

(a) OMB = Office of Management and Budget.
(b) kWh = kilowatt hour.
(c) therm = 100,000 British Thermal Units (BTU).

Non-economic factors considered in selection of alternatives are summarized below. Each criterion was weighted in terms of relative importance towards an alternatives overall score.

- **Sustainability.** Scores alternatives based on minimizing risk to continue to meet water quality objectives during the planning period. Considerations include operational complexity, anticipated staff effort and certification needed, redundancy, and reliability.

- **Ease of Permitting.** Assesses alternatives based on the anticipated permitting process. Considerations may include perceived acceptance by the Department of Environmental Quality (DEQ) to meet National Pollutant Discharge Elimination System (NPDES) permit requirements, land use permitting, and environmental permitting.

- **Community Considerations.** Scores alternatives based on potential negative impacts to the community. Considerations may include traffic impacts, potential for objectionable odors, noise, and visual impacts.

- **Water Conservation.** Provides a measure of an alternative's potential for water reuse and/or water use efficiency.

- **Energy Conservation.** Provides a measure of an alternative's energy characteristics including total energy use, greenhouse gas emissions, and potential as a renewable energy source.

- **Environmental Considerations.** Scores alternatives based on minimizing impacts to flood plains, wetlands, endangered species, and historical and archeological resources. Also considers waste generation and ease of handling wastes.

Alternatives were scored for each criterion on a scale of one (1) to five (5), with 1 being lowest (worst) and 5 being highest (best). The weighting factor is a percentage-based multiplier that places greater emphasis on specific criteria deemed to be of higher value.

6.2 Liquid Stream Treatment Alternatives

The following provides a summary of the alternative selection process for improvements to the liquid stream treatment processes of the City's WWTP. Processes evaluated included 1) influent pumping, 2) preliminary treatment (headworks), 3) primary treatment, 4) biological secondary treatment including clarification, 5) tertiary filtration, 6) disinfection, and 7) effluent pumping.

6.2.1 Influent Pumping

As described in Section 5, two influent pumping alternatives were considered. These included rehabilitating the currently inoperable Raw Sewage Pumping Station or increasing the capacity of the West Side Interceptor Pump Station (WIPS) that was constructed in 2002 and was designed to facilitate expansion to meet projected buildout flows. Expanding the capacity of the WIPS to meet projected design year 2045 flows was selected as the preferred alternative.

Expanding the capacity of the WIPS would include replacing the station's existing three main, 75-horsepower (hp) submersible pumps and one 25-hp low flow pump with four (3 duty, 1 standby) 135-hp pumps and one 50-hp low flow pump to increase the station's hydraulic capacity. Existing station piping and magnetic flowmeters would also be replaced because of

the increased pump capacity. Planning level design criteria for influent pumping are presented in Table 6-2.

Table 6-2: Influent Pumping Station Design Criteria

Parameter	Units	Value
Pumps		
Type	-	Submersible centrifugal
Number		
Large pump[a]	-	4
Small pump	-	1
Capacity, each		
Large pump	gpm[b]	7,200
Small pump	gpm	3,600
TDH[c]	ft[d]	44
Motor power, each		
Large pump	hp[e]	135
Small pump	hp	50
Drive type	-	Variable speed
Station capacity	MGD[f]	31[g]

Notes:

(a) Includes one redundant pump.
(b) gpm = gallons per minute.
(c) TDH = total dynamic head.
(d) ft = feet.
(e) hp = horsepower.
(f) MGD = million gallons per day.
(g) Firm capacity with one large pump out of service.

Estimated costs for expanding the existing WIPS, including a total 20-year net present value, are presented in Table 6-3 below.

Table 6-3: Influent Pumping Station Cost Summary[a][b]

Item	Value
Capital Cost	
Construction	$2,410,000
Contingency	$720,000
Escalation	$650,000
Engineering and Admin.	$950,000
Total capital cost	$4,730,000
O&M Cost	
Annual average[c][d]	$34,000
20-yr net present value	$560,000
Salvage Value	$440,000
Total Net Present Value	$4,850,000

Notes:

(a) AACE Class 4 planning level costs.

(b) Unless otherwise stated, values rounded to the nearest $10,000.
(c) Based on an annual average labor cost of approximately $5,000,annual energy cost of about $21,000, and equipment maintenance of about $8,000.
(d) Rounded to the nearest $1,000.

6.2.2 Preliminary Treatment

As described in Section 5, two alternatives were considered for preliminary treatment. These include: 1) improving the existing headworks and 2) replacing the existing headworks with new facilities.

The alternative to improve the existing headworks would consist of replacing the existing climber bar screens with two new, mechanically cleaned screens each with capacity equal to half of the 2045 projected peak flow condition. Both screens would continue to feed the existing common screenings washer/compactor. Ancillary equipment such as slide gates within the existing headworks would be replaced but no changes would be made to the existing concrete structure itself.

The second alternative would be to completely replace the existing headworks with new facilities. This would include a new concrete channel structure, mechanically cleaned screens, manual bar rack, and grit removal. Use of a vortex type grit removal chamber with associated grit pump, cyclone separator, and grit classifier was assumed. Dewatered grit would be discharged with washed/compacted screenings to dumpsters for offsite disposal.

Planning level design criteria for both preliminary treatment options are presented in Table 6-4.

Table 6-4: Headworks Design Criteria

Parameter	Units	Retain Existing Headworks	New Headworks
Bar Screen			
Type	-	Climber	Climber
Number	-	2	2
Capacity, each	MGD	15.5	15.5
Opening width	in	0.5	0.25
Motor power	hp	1.5	1.5
Screenings Washer			
Number	-	1	1
Motor power	hp	1.5	1.5
Grit Chamber			
Type	-	NA[(a)]	Vortex
Number	-		1
Diameter	ft		18
Drive power	hp		2
Grit Pump			
Type	-	NA[(a)]	Recessed impeller
Capacity	gpm		250
Motor power	hp		5
Grit Classifier			

Parameter	Units	Retain Existing Headworks	New Headworks
Number	-	NA[(a)]	1
Motor power	hp		1

Note:

(a) The existing headworks do not have grit removal.

6.2.2.1 Economic Comparison of Headworks Alternatives

Estimated costs for the two preliminary treatment alternatives, including a total 20-year net present value, are presented in Table 6-5.

Table 6-5: Economic Comparison of Headworks Alternatives[(a)(b)]

Item	Retain Existing Headworks	New Headworks
Capital cost		
Construction	$1,110,000	$2,420,000
Contingency	$330,000	$720,000
Escalation	$300,000	$650,000
Engineering and Admin.	$440,000	$950,000
Total capital cost	$2,180,000	$4,740,000
O&M cost		
Annual average[(c)]	$7,000[(d)]	$12,000[(e)]
20-yr net present value	$120,000	$200,000
Salvage value	$10,000	$650,000
Total net present value	$2,290,000	$4,290,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $10,000.
(c) Rounded to the nearest $1,000.
(d) Based on an annual average labor cost of approximately $3,000, annual energy cost of about $1,500, and reoccurring equipment maintenance of about $1,500.
(e) Based on an annual average labor cost of approximately $5,000, annual energy cost of about $2,000, and reoccurring equipment maintenance of about $4,000.

6.2.2.2 Non-economic Comparison of Headworks Alternatives

A comparison of the preliminary treatment alternatives considering non-economic factors was performed. The results of this analysis are summarized in Table 6-6.

Table 6-6: Non-Economic Comparison of Headworks Treatment Alternatives

Headworks Alternative	Sustainability (30%)	Ease of Permitting (20%)	Community Considerations (20%)	Water Conservation (10%)	Energy Conservation (10%)	Environmental Considerations (10%)	Weighted Score
Retain Existing Headworks	3	5	3	0	5	4	3.5
New Headworks	5	5	4	0	4	4	4.1

As shown in Table 6-6, providing new headworks facilities scored higher in the non-economic comparison of preliminary treatment alternatives than continuing to utilize existing facilities. Reasoning for the scores assigned within each of the six categories is summarized below.

- **Sustainability.** The new headworks alternative ranked higher than continuing to use the existing facilities. Although the addition of grit removal would increase complexity of the system with more equipment to operate and maintain, the new facilities could be designed to improve access and reduce effort needed to maintain and keep facilities clean. Grit removal would reduce wear and tear on downstream equipment. It would also reduce the potential for material to settle out in the secondary treatment process thus reducing labor needed for routine maintenance and cleaning of associated tanks.

- **Ease of Permitting.** The two alternatives considered should have no difficulty in gaining regulatory approval and were thus ranked the same.

- **Community Considerations.** Neither alternative would vary significantly from the other in terms of their potential for negative impact on the community. The main concern with preliminary treatment is the potential for odor. New headworks facilities could include covered channels that would reduce the potential for nuisance odors so was ranked slightly higher.

- **Water Conservation.** Neither alternative would impact the City's ability to implement a recycled water program. In addition, water consumption should be the same. For these reasons, a value of zero was assigned for this category.

- **Energy Conservation.** Retaining use of the existing headworks ranked higher for energy conservation than constructing new facilities. New headworks would increase energy use because of the addition of grit removal that would include electrically driven vortex grit chamber, grit pump, and grit classifier. The overall carbon footprint of constructing new headworks would also be greater because of the construction materials (e.g., concrete, steel, etc.) that would not be needed if the existing headworks were retained.

- **Environmental Considerations.** Neither of the preliminary treatment alternatives is anticipated to negatively impact flood plains or biological, historical, or cultural resources. Constructing new headworks would, however, create more construction related waste.

6.2.2.3 Recommended Headworks Alternative

Constructing new headworks facilities is recommended. While more costly than upgrading the existing headworks, the existing headworks structure is almost 50 years old and would not be located near the new secondary treatment process that is also recommended within this section. New headworks will simplify construction of yard piping within the WWTP and reduce maintenance labor. New facilities will also provide grit removal that will protect downstream equipment, reduce accumulation of grit within the secondary treatment process thus saving on maintenance, and will lower the quantity of inert solids within the solids treatment process.

6.2.3 Primary Treatment

As described in Section 5, a matrix-based screening analysis of three primary treatment alternatives was performed to identify one treatment technology that would be used when developing overall project alternatives requiring the use of primary treatment (e.g., anaerobic digestion). The alternatives considered include 1) conventional primary clarification, 2) enhanced primary clarification, and 3) primary filtration. Of these, primary filtration was selected as the alternative for further consideration.

Primary treatment was selected for evaluation in part for its being a prerequisite for anaerobic digestion. Because anaerobic digestion is typically sized for maximum month loading conditions, capacity of the primary filtration system was assumed to be 15 million gallons per day (MGD). The design year 2045 MMWWF and PDF were estimated to be 10.1 MGD and 19.4 MGD, respectively. Assuming a filter capacity within this range of flows ensures that the filters will have sufficient capacity to capture the maximum month loading condition. In addition, as presented later in the discussion on secondary treatment alternatives, providing primary filtration with a capacity of 15 MGD was needed when evaluating the extended aeration alternative.

Primary filtration would be constructed immediately downstream of the headworks and would receive screened and degritted raw wastewater. A flow splitting structure would be provided for use in splitting influent flows above 15 MGD (capacity of the primary filters). At influent flows greater than 15 MGD, all secondary treatment alternatives (except for extended aeration) would continue to receive primary effluent except for the portion above 15 MGD. The portion above 15 MGD would flow around the primary filters and would recombine with the primary filter effluent prior to being sent to the secondary treatment process.

If extended aeration was selected as the preferred secondary treatment alternative, the flow splitting structure would be configured such that during peak flow events (i.e., flows greater than 15 MGD), the WWTP would operate under split flow treatment. In this mode, up to 15 MGD of the screened and degritted raw wastewater would receive secondary treatment without first flowing through the primary filters. The balance of the influent wastewater would only receive primary treatment. When operating in parallel in the split flow treatment mode, effluent from the secondary and primary treatment processes would combine immediately prior to disinfection.

Planning level design criteria for primary filtration are presented in Table 6-7. Primary filter manufacturers include Aqua-Aerobics Systems and Veolia. Because they have more primary filtration installations in the United States than others, the planning level sizing and costs presented here were based on systems as manufactured by Aqua-Aerobics Systems.

Table 6-7: Primary Filtration Design Criteria

Parameter	Units	Value
Filtration type	-	Synthetic cloth disk
Capacity, peak[(a)]	MGD	15.0
Number of trains	-	2
Filter disks per train	-	14
Total filter area	ft^2	3,010
Filtration rate, peak flow	gpm/ft^2	3.5
Filter tanks		
Construction type	-	Concrete
Volume, each	gal	21,800
Filter drive power, each	hp	5
Backwash pumps		
Type	-	Centrifugal
Number	-	2
Power, each	hp	20
Solids waste pumps		
Type	-	Non-clog centrifugal
Number	-	2
Power, each	hp	20

Note:

(a) Total capacity with all filter trains in service.

Estimated costs for primary filtration, including a total 20-year net present value, are presented in Table 6-8. These costs would only be applicable for WWTP improvement projects that would include either 1) extended aeration as the secondary treatment alternative or 2) anaerobic digestion.

Table 6-8: Primary Filtration Cost Summary[(a)(b)]

Item	Value
Capital cost	
Construction	$4,330,000
Contingency	$1,300,000
Escalation	$1,170,000
Engineering and Admin.	$1,700,000
Total capital cost	$8,500,000
O&M cost	
Annual average[(c)(d)]	$46,000
20-yr net present value	$740,000
Salvage value	$800,000
Total net present value	$8,440,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $10,000.
(c) Based on an annual average labor cost of approximately $19,000, annual energy cost of about $5,000, and reoccurring major equipment maintenance (filter media replacement) of about $115,000 every 5 years.
(d) Rounded to the nearest $1,000.

6.2.4 Secondary Treatment

Four alternatives were selected for consideration to address deficiencies in the WWTP's secondary treatment process. These included 1) expanding the WWTP's existing treatment process, 2) conventional activated sludge, 3) oxidation ditch, and 4) extended aeration. The later three alternatives assumed complete replacement of the WWTP's existing treatment process with the demolition of the current facilities.

6.2.4.1 Expand Existing Treatment Process

This alternative assumes continued use of the WWTP's two existing treatment basins with construction of two additional basins of equal size and volume to double the effective capacity of the system. The new basins would be similarly designed with vertical turbine aerators, anoxic zones, mechanical mixing, and the ability to operate in step feed and sludge re-aeration modes during peak flow conditions. Design criteria for this alternative are presented in Table 6-9.

Construction of additional matching basins would be challenging. While it would be preferable to place the new basins immediately adjacent to the existing aeration basins, space is limited and doing so would require demolishing the existing Filter Building and Effluent Pumping Station to the north. In addition, because of changes in building codes that have occurred since construction of the original basins in 1976, the new basins would have to be constructed as an independent structure and could not include common wall construction with the existing basins. This would increase both the construction cost and area required for this alternative over what it might otherwise be.

Table 6-9: Expand Existing Treatment Process Design Criteria

Parameter	Units	Value
Aeration Basins		
Number (new/existing)	-	2 / 2
Construction type	-	Concrete
Volume per basin	MG	0.47
Total volume	MG	1.8
Depth, water	ft	11
SRT	days	13
MLSS, max	mg/L	4,000[(b)]
Aeration		
Type	-	Vertical, turbine
Number (new/existing)	-	4 / 4
Power, each	hp	75
Anoxic Zone Mixers		

Parameter	Units	Value
Type	-	Submersible
Number (new/existing)	-	4 / 4
Power, each	hp	5

Note:

(a) mg/L = milligrams per liter.
(b) Maximum value assuming design year maximum month loading conditions, minimum monthly temperature, and SRT value shown.

Estimated costs for this alternative to expand the existing secondary treatment process including capital, annual O&M, and salvage value are summarized in Table 6-13 and compared with the other three alternatives considered.

6.2.4.2 Conventional Activated Sludge

This alternative assumes the existing secondary treatment process is completely replaced with a new conventional activated sludge process. The new process would include fine bubble aeration, ability to produce fully nitrified effluent, and operate in the following process modes: 1) plug flow, 2) step feed, and 3) sludge reaeration. Anoxic zones with submersible mixers would be provided to achieve partial denitrification, improve sludge settleability, and partially recover alkalinity consumed by the nitrification process.

The new basins would be significantly deeper at 18 ft than those of the existing treatment process at 11 ft. This would reduce the overall footprint of the secondary treatment process and would result in better aeration efficiency. The new activated sludge process could be constructed at a separate location on the WWTP site while maintaining continued operation of the existing secondary treatment process. Aeration air would be provided by positive displacement blowers that would be in a new blower building.

Design criteria for the conventional activated sludge alternative are presented in Table 6-10.

Table 6-10: Conventional Activated Sludge Design Criteria

Parameter	Units	Value
Aeration Basins		
Number	-	3
Construction type	-	Concrete
Volume per basin	MG	0.7
Total volume	MG	2.1
Depth, water	ft	18
SRT	days	13
MLSS, max	mg/L	4,000[(a)]
Aeration		
Type	-	Fine bubble diffusers
Blowers		
Number	-	4[(b)]
Power, each	hp	150

Parameter	Units	Value
Anoxic Zone Mixers		
Type	-	Submersible
Number	-	6[(c)]
Power, each		2.5
ML Return Pumps		
Type	-	Submersible, axial
Number	-	3
Power, each	hp	8.3

Notes:

(a) Maximum value assuming design year maximum month loading conditions, minimum monthly temperature, and SRT value shown.
(b) Number includes one redundant blower.
(c) Two anoxic zones per basin assumed, each with one mixer.

Estimated costs for this alternative to replace the existing secondary treatment process with a new conventional activated sludge process including capital, annual O&M, and salvage value are summarized in Table 6-13. Costs shown are compared with the other three alternatives considered.

6.2.4.3 Oxidation Ditch

This alternative assumes that the existing secondary treatment process would be replaced with an oxidation ditch. As previously described in Section 5, an Orbal® type oxidation ditch by Evoqua was used as the basis for this alternative. This type of oxidation ditch is constructed with multiple, independent concentric rings that allows it to operate in modes like that of the conventional activated sludge process (e.g., plug flow, step feed, and sludge reaeration). This makes the Orbal® style process design inherently well suited for applications with high flow peaking factors. Like the conventional activated sludge process, it too can achieve nitrification. Conditions within the process can also be controlled to promote denitrification.

The oxidation ditch alternative differs from the conventional activated sludge alternative in that aeration is provided by a series of horizontal surface disc aerators instead of fine bubble diffusion. This would eliminate the need for aeration blowers and the associated building.

For an Orbal® type oxidation ditch, a single ditch would be constructed with a water depth of about 14 ft. Like the conventional activated sludge process, it could be constructed at a separate location on the treatment plant site while maintaining continued operation of the existing secondary treatment process.

Design criteria for the conventional activated sludge alternative are presented in Table 6-11.

Table 6-11: Oxidation Ditch Design Criteria

Parameter	Units	Value
Oxidation Ditch Basins		
Number	-	1[(a)]
Construction type	-	Concrete
Number of Channels	-	3
Total volume	MG	2.3[(b)]
Depth, water	ft	14
SRT	days	13.5
MLSS, max	mg/L	4,000[(c)]
Aeration		
Type	-	Horizontal, surface disc
Number	-	6[(d)]
Power, each[(d)]		
Small aerators	hp	30
Large aerators	hp	75

Notes:

(a) Assumes construction of the Orbal® type oxidation ditch system by Evoqua with concentric internal channels. Other ditch designs are available but may entail construction of multiple separate ditches.
(b) Total volume includes outer channel volume of 1.0 MG, middle channel volume of 0.8 MG, and inner channel volume of 0.5 MG.
(c) Maximum value assuming design year maximum month loading conditions, minimum monthly temperature, and SRT value shown.
(d) Mixers include three 30-hp shaft mounted disc mixers and three 75-hp shaft mounted disc mixers.

Estimated costs for this alternative to replace the existing secondary treatment process with a new oxidation ditch (specifically an Orbal® type) including capital, annual O&M, and salvage value are summarized in Table 6-13 below. Costs shown are compared with the other three alternatives considered.

6.2.4.4 Extended Aeration

The extended aeration alternative assumes the existing secondary treatment process would be replaced with a new extended aeration type biological process. As previously described, the extended aeration process is a low rate activated sludge process with SRTs greater than 30 to 40 days compared to the other processes considered with SRTs of 13 days.

Extended aeration is a modified lagoon type process that typically utilizes lined earthen basins with aeration air being provided via suspended fine bubble diffusers. Electrically actuated valves on the air supply lines are used to control the flow of air into the basins. Aeration can be controlled to produce fully nitrified effluent and achieve partial denitrification. Like the conventional activated sludge process, aeration air would be provided by positive displacement blowers that would be located in a new blower building.

An extended aeration system would completely replace the existing secondary treatment process. It would include two, parallel, lined, earthen basins, each with a water depth of about 10 ft. Like the other new process alternatives, it could be constructed at a separate location on

the treatment plant site while maintaining continued operation of the existing secondary treatment process during construction. However, the area required for an extended aeration system would be about four times greater than that of the other alternatives considered.

Design criteria for the conventional activated sludge alternative are presented in Table 6-12.

Table 6-12: Extended Aeration Design Criteria

Parameter	Units	Value
Aeration Basins		
Number	-	2
Construction type	-	Lined earthen basin
Volume per basin	MG	2.5
Total volume	MG	5.1
Depth, water	ft	10
SRT	days	40
MLSS, max	mg/L	3,000[(a)]
Aeration		
Type	-	Fine bubble diffusers
Blowers		
Number	-	4[(b)]
Power, each	hp	150

Notes:

(a) Maximum value assuming design year maximum month loading conditions, minimum monthly temperature, and SRT value shown.

(b) Number includes one redundant blower.

Estimated costs for the extended aeration alternative including capital, annual O&M, and salvage value are summarized in Table 6-13 below and are compared with those of the other alternatives considered. The costs shown for the extended aeration alternative include construction of primary filtration that would be needed to treat flows greater than 15 MGD which would be the hydraulic capacity of an extended aeration system.

6.2.4.5 Economic Comparison of Secondary Treatment Alternatives

Estimated costs for the four secondary treatment alternatives considered are summarized in Table 6-13. As shown in the table, the least cost alternative both in terms of initial capital and net present value is to expand the existing treatment process. The second closest alternative is construction of a new oxidation ditch. The significantly lower cost of expanding the existing system is a result of requiring approximately half the tank volume of either the oxidation ditch or conventional activated sludge alternatives. However, this lower cost requires continued use of existing tanks that are almost 50 years old. The age of the existing facilities is reflected in the higher salvage value of the alternatives that assume replacement of the existing aeration basins.

Table 6-13: Economic Comparison of Secondary Treatment Alternatives[a][b]

Item	Expand Existing Treatment	Conventional Activated Sludge	Oxidation Ditch	Extended Aeration[c]
Capital cost				
Construction	$4,600,000	$8,910,000	$8,640,000	$11,000,000
Contingency	$1,380,000	$2,670,000	$2,590,000	$3,300,000
Escalation	$1,240,000	$2,410,000	$2,330,000	$2,970,000
Engineering and Admin.	$1,810,000	$3,500,000	$3,390,000	$4,320,000
Total capital cost	$9,030,000	$17,490,000	$16,950,000	$21,600,000
O&M cost				
Annual average[d]	$103,000	$87,000	$87,000	$124,000
20-yr net present value	$1,770,000	$1,640,000	$1,500,000	$2,040,000
Salvage value	$2,430,000	$4,960,000	$5,550,000	$3,680,000
Total net present value	$8,370,000	$14,170,000	$12,900,000	$19,960,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $10,000.
(c) Costs of the extended aeration alternative include those associated with providing primary filtration which is needed to treat flows greater than 15 MGD.
(d) Rounded to the nearest $1,000.

6.2.4.6 Non-economic Comparison of Secondary Treatment Alternatives

A comparison of the secondary treatment alternatives considering non-economic factors was performed. The results of this analysis are summarized in Table 6-14.

Table 6-14: Non-Economic Comparison of Secondary Treatment Alternatives

Secondary Treatment Alternative	Sustainability (30%)	Ease of Permitting (20%)	Community Considerations (20%)	Water Conservation (10%)	Energy Conservation (10%)	Environmental Considerations (10%)	Weighted Score
Existing Process	3	4	4	0	4	5	**3.4**
Conventional Activated Sludge	4	5	5	0	3	4	**3.9**
Oxidation Ditch	5	5	5	0	3	4	**4.2**
Extended Aeration	3	4	4	0	5	3	**3.3**

As shown in Table 6-14, the oxidation ditch alternative scored the highest in the non-economic comparison of secondary treatment alternatives. Reasoning for the scores assigned within each of the six categories is summarized below.

- **Sustainability.** The oxidation ditch alternative ranked the highest of the secondary treatment alternatives for its relative simplicity, proven reliability, and ability to meet future anticipated permit requirements. Conventional activated sludge ranked slightly lower because of the need for aeration blowers and being slightly more complex both mechanically and operationally. Although WWTP staff are accustomed to operating the existing treatment process, it ranked lower as new facilities should be better able to meet future permit limits. New facilities should also result in less operational and maintenance effort over the 20-year planning period than continued use of aging facilities. Although it is a relatively simple process, extended aeration was ranked lower because of the need for primary effluent diversion during peak flow events. This increases the operational complexity. Extended aeration is also less well suited for future changes in permit limits that could require nutrient removal.

- **Ease of Permitting.** None of the secondary treatment alternatives should be difficult to gain approval from state or local agencies. However, the existing treatment process and extended aeration alternatives were ranked slightly lower. This was because of potential concerns that the DEQ may have with the shallow depth of the existing aeration basins and potential difficulty in meeting future permit conditions when compared to a new secondary treatment process. Extended aeration received a lower ranking because of the potential DEQ concerns with primary flow diversion during peak flow events.

- **Community Considerations.** The alternatives for secondary treatment all ranked similarly with none anticipated to have significant negative impacts to the community. Continued use of the existing treatment process ranked slightly lower because it would entail construction of two additional aeration basins that would most likely be closer to the WWTP's property line than the other alternatives. This could potentially create additional noise during construction. The extended aeration alternative was also ranked slightly lower simply due to its much greater size and associated potential for visual impacts.

- **Water Conservation.** None of the alternatives considered would impact the City's ability to implement a recycled water program. In addition, water consumption should be similar for all alternatives. For these reasons, a value of zero was assigned to all alternatives for this category.

- **Energy Conservation.** Extended aeration ranked highest in terms of energy conservation. Annual energy use is expected to be less as would be its overall carbon footprint. This would mainly be because of its use of earthen berms for construction of the basins which has a smaller carbon footprint than reinforced concrete which would be used in constructing the other alternatives. Continued use of the existing treatment process was ranked second highest because of its reuse of existing concrete structures which resulted in a lower overall carbon footprint than the conventional activated sludge and oxidation ditch alternatives.

- **Environmental Considerations.** None of the secondary treatment alternatives should have significant negative impacts to flood plains or biological, historical, or cultural resources. However, retaining the use of the existing secondary treatment facilities would result in less construction related waste than the other alternatives and was thus ranked higher. The extended aeration alternative ranked lowest because of its

significantly greater size and the area occupied that would not otherwise be available for greenspace.

6.2.4.7 Recommended Secondary Treatment Alternative

Constructing a new oxidation ditch based secondary treatment process is recommended. While the total net present value of the oxidation ditch alternative is greater than that of expanding existing facilities, it is lower than the other alternatives considered. The oxidation ditch treatment process ranked significantly higher in the non-economic comparison of alternatives than expanding the existing almost 50-year-old treatment process. Oxidation ditches are well suited to the City's highly variable wet weather flows and are operationally simple, well proven, and reliable.

While potentially less costly initially, expanding the existing treatment process would be challenging because it is surrounded by other existing structures. New basins would have to be located away from the existing basins. This would require construction of additional yard piping and means to split flows between new and existing facilities making control and operation more difficult.

6.2.4.8 Secondary Clarification

As described in Section 5, construction of a new 130-ft secondary clarifier is recommended. The new clarifier would match the WWTP's large secondary clarifier constructed in 2012. It would replace the WWTP's three smaller, aging, secondary clarifiers, two of which were constructed prior to 1976. New RAS, WAS, and scum pumping systems would be provided with the new clarifier.

Planning level design criteria for secondary clarification are presented in Table 6-15.

Table 6-15: Secondary Clarifier Design Criteria

Parameter	Units	Value
Clarifiers		
Number (new/existing)	-	1/1
Diameter	ft	130
Depth, water	ft	16
Overflow rate, peak day		
MMDWF	gpd/ft^2	450
Peak Day	gpd/ft^2	730
Drive power, each	hp	1
RAS Pumps		
Type	-	Submersible non-clog
Number, each clarifier	-	3
Capacity, each	gpm	2,430
Motor power	hp	25
Drive type	-	Variable Speed

Parameter	Units	Value
WAS Pumps		
Type	-	Submersible non-clog
Number, each clarifier	-	1
Capacity, each	gpm	300
Motor power	hp	3
Drive type	-	Constant Speed
Scum Pump		
Type	-	Submersible non-clog
Number, each clarifier	-	1
Capacity, each	gpm	200
Motor power	hp	5
Drive type	-	Constant Speed

Estimated costs for a new secondary clarifier including a total 20-year net present value are presented in Table 6-16.

Table 6-16: Secondary Clarifier Cost Summary[(a)(b)]

Item	Value
Capital cost	
Construction	$6,070,000
Contingency	$1,820,000
Escalation	$1,640,000
Engineering and Admin.	$2,380,000
Total capital cost	$11,910,000
O&M cost	
Annual average[(c)(d)]	$29,000
20-yr net present value[(e)]	$470,000
Salvage value[(e)]	$3,080,000
Total net present value	$9,300,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $10,000.
(c) Includes annual average costs of about $4,000 for labor, $12,000 for energy, and $13,000 for maintenance.
(d) Rounded to the nearest $1,000.
(e) Value shown is for new secondary clarifier only.

6.2.5 Tertiary Treatment

The use of tertiary filtration is recommended to help ensure that the City can meet effluent discharge limits that under specific flow conditions will require effluent BOD and TSS concentrations below 10 mg/L. Although conventional secondary treatment processes can be relied upon to consistently meet effluent BOD and TSS concentrations down to 10 mg/L, tertiary filtration is recommended for concentrations below this value. As discussed in Section 5, capacity of the tertiary filtration system should be equal to the projected design year 2045 MMWWF condition (approximately 10 MGD) to allow for potential changes in the City's NPDES

permit that would require year-round BOD and TSS removal efficiency of 85%. Current discharge limits relax the mass removal efficiency requirement for the WWTP to 75% when the monthly average flow rate exceeds 3.0 MGD.

Of the three tertiary filtration technologies initially considered, synthetic/cloth media filtration was selected as the alternative for further consideration in developing recommended WWTP improvements. Planning level design criteria for a synthetic/cloth media type tertiary filtration system are presented in Table 6-17. Tertiary filter manufacturers include Aqua-Aerobics Systems and Veolia. Because they have more synthetic cloth type filter installations in the United States than others, the planning level sizing and costs presented here were based on Aqua-Aerobics Systems.

Tertiary filtration would be constructed within a concrete tank hydraulically located between the Effluent Pumping Station and the disinfection system. A flow splitting structure would be provided that would direct secondary effluent flows above 10 MGD (capacity of the tertiary filters) directly to the disinfection system.

Table 6-17: Tertiary Filtration Design Criteria

Parameter	Units	Value
Filtration type	-	Synthetic cloth disk
Capacity, peak	MGD	10.0
Number of trains	-	1
Filter disks per train	-	14
Total filter area	ft^2	1,510
Filtration rate, peak flow	gpm/ft^2	4.6
Nominal Pore Size	µm	5 to 10
Filter tanks		
Construction type	-	Concrete
Volume	gal	15,400
Filter drive power	hp	5
Backwash pumps		
Type	-	Centrifugal
Number	-	1
Power, each	hp	20

Notes:

(a) Maximum value assuming design year maximum month loading conditions, minimum monthly temperature, and SRT value shown.
(b) Number includes one redundant blower.

Estimated costs for tertiary filtration including a total 20-year net present value are presented in Table 6-18.

Table 6-18: Tertiary Filtration Cost Summary[a][b]

Item	Value
Capital cost	
Construction	$1,740,000
Contingency	$520,000
Escalation	$470,000
Engineering and Admin.	$680,000
Total capital cost	$3,410,000
O&M cost	
Annual average[c][d]	$23,000
20-yr net present value	$560,000
Salvage value	$270,000
Total net present value	$3,700,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $10,000.
(c) Includes an annual average labor cost of approximately $6,000, annual energy cost of about $3,000, and reoccurring major equipment maintenance (filter media replacement) of about $52,000 every 5 years.
(d) Rounded to the nearest $1,000.

6.2.6 Disinfection

Two alternatives were selected for consideration to improve the WWTP's disinfection system. These consisted of continued use of chlorination with sodium hypochlorite and ultraviolet (UV) disinfection. As is described below, the chlorination alternative included two sub-alternatives. These included 1) continuing to use the existing two CCBs with construction of a third and 2) replacing the existing CCBs with three new basins.

6.2.6.1 Chlorination

The WWTP's two existing CCBs do not meet DEQ requirements for detention time. In addition, during design year 2045 peak hour flow, hydraulic modeling suggests there would be limited freeboard (approximately 0.5 ft with the CCBs operating in series and approximately 1 ft if operating in parallel). This alternative assumes the continued use of sodium hypochlorite with construction of additional tank volume to meet DEQ requirements and provide additional freeboard.

Two alternatives to provide the necessary tank volume were considered. The first would entail construction of a third CCB immediately adjacent to the existing CCBs. The second would eliminate the existing CCBs (originally constructed in 1976) with three entirely new basins. Both alternatives would provide the same overall volume and would meet the needed DEQ detention time criteria. Design criteria for the chlorination alternative are presented in Table 6-19.

Because of changes in the building code that have occurred since construction of the original basins in 1976, the new CCB would have to be constructed as an independent structure and could not share a common wall or be connected to the existing structure. Expansion of the existing CCBs would occur on the east side of the basins and would necessitate removal of the

existing dechlorination facility. Thus, both chlorination alternatives assumed construction of a new dechlorination system.

New sodium hypochlorite storage and metering facilities were also assumed for both alternatives. This is because of the poor condition of the existing control building complex and the City's desire to replace it.

Table 6-19: Chorine Disinfection Design Criteria

Parameter	Units	Value
Chlorine Contact Basins		
Number	-	3[(a)]
Construction type	-	Concrete
Volume, each	gal	120,000
Depth	ft	16.5
Length:width ratio[(b)]	-	23:1
Detention time[(c)(d)]		
ADF[(e)]	min	128
PDF	min	26
PIF	min	16
Flash mixer, power	hp	7.5
Sodium Hypochlorite Feed		
Dose, avg	mg/L	4
Storage tanks		
Number	-	2
Volume, each	gal	4,000
Feed pumps		
Type	-	Peristaltic
Number	-	2
Capacity, each	gph	52
Sodium Bisulfite Feed		
Storage tanks		
Number	-	2
Volume, each	gal	1,200
Feed pumps		
Type	-	Peristaltic
Number	-	2
Capacity, each	gph	16

Notes:

(a) Both alternatives would have the same number of CCBs. The alternative that includes continued use of the two existing CCBs requires addition of a third equal sized CCB.
(b) Value shown assumes parallel operation of the three CCBs. CCBs would be configured to also allow series operation which would increase the length:width ratio.
(c) Detention time at projected year 2045 influent flow conditions.
(d) The DEQ requires that CCB volume be set to achieve at least 60 minutes of contact time at average dry weather flow, 20 minutes for peak day, and 15 minutes for peak hour, whichever results in the largest volume.
(e) Value shown assumes one of the three CCBs is offline for maintenance.

Estimated costs for the two chlorine disinfection alternatives, including capital, annual O&M, and salvage value, are summarized in Table 6-21 and compared with the UV disinfection alternative.

6.2.6.2 Ultraviolet Light Disinfection

As described in Section 5, an open channel, vertically mounted, UV disinfection system was chosen as the basis of design for this alternative. The UV lamp banks would be placed in a single concrete channel constructed above grade at an elevation to allow continued gravity flow through the WWTP's existing river outfall. Multiple UV lamp banks would be provided and would include one redundant bank. Level control would be provided by a fixed downstream weir provided by the UV system manufacturer. Electric or hydraulically powered lifting equipment would be provided as an integral part of the UV system for routine removal and maintenance of the UV system lamps.

Although DEQ guidelines do not require parallel UV channels, doing so would allow one train to be removed from service during summertime low flow periods. This would reduce the effort needed to clean the UV channels. Although single channel UV disinfection systems are not uncommon and are less costly, the City should evaluate the advantages and disadvantages of parallel UV channels prior to detailed design of proposed disinfection improvements.

Controls (which include the system PLC), electrical distribution to the UV banks (including UV ballasts), and a bank lifting system would be located near the channel in outdoor-rated panels or in an electrical enclosure such as a motor control center (MCC) room. Costs for this option do not account for the construction of a separate electrical facility.

Planning level design criteria for UV disinfection are presented in Table 6-20. UV disinfection systems of the type and configuration described are available from multiple manufacturers including Trojan Technologies and Xylem/Wedeco. This evaluation assumed a Trojan Technologies UV disinfection system. Systems provided by other manufacturers would be similar.

Table 6-20: UV Disinfection Design Criteria

Parameter	Units	Value
Type	-	Low pressure
Configuration	-	Open channel
Construction type	-	Concrete
Number of trains	-	1
Banks per train	-	4[(a)]
Lamps per bank	-	22
Minimum design dose[(b)]	mJ/cm^2	30
UV transmittance	%	60
Channel dimensions		
Length	ft	36
Width	ft	5
Depth	ft	7.8
Level control	-	Fixed weir

Note:

(a) Total number of UV banks includes 3 duty banks and 1 redundant bank.

(b) Minimum allowable design dose at projected design year 2045 peak hour flow with all duty UV-banks in service assuming minimum UV transmittance, end of life lamp output, and application of quartz sleeve fouling factor.

Estimated costs for a UV disinfection, including capital, annual O&M, and salvage value, are summarized in Table 6-21 and are compared with the chlorine disinfection alternative.

6.2.6.3 Economic Comparison of Disinfection Alternatives

Costs for the two overall disinfection alternatives, including sub-alternatives for chlorination, are summarized in Table 6-21. As shown in the table, the least cost alternative in terms of initial capital is UV disinfection. UV disinfection also has the lowest 20-year O&M cost. However, expanding the existing CCBs has a greater 20-year salvage value. This alternative thus has a lower total net present value than UV disinfection.

The difference in salvage values is caused by the large proportion of the CCB cost which is concrete that has an assigned asset life expectancy of 50 years. UV disinfection would require less concrete construction and a greater percentage of its overall value is associated with the UV equipment itself which has an assigned asset life expectancy of 20 years. While expansion of the existing CCBs has the lowest total net present value of the alternatives considered, this is predicated on continued use of existing tanks that are almost 50 years old.

Table 6-21: Economic Comparison of Disinfection Alternatives[(a)(b)]

	Chlorination		
Item	**Expand Existing**	**New CCBs**	**UV**
Capital cost			
Construction	$2,520,000	$4,120,000	$2,080,000
Contingency	$750,000	$1,240,000	$630,000
Escalation	$680,000	$1,110,000	$560,000
Engineering	$990,000	$1,620,000	$820,000
Total capital cost	$4,940,000	$8,090,000	$4,090,000
O&M cost			
Annual average[(c)]	$210,000[(d)]	$210,000[(d)]	$47,000[(e)]
20-yr net present value	$3,610,000	$3,610,000	$770,000
Salvage value	$1,490,000	$2,570,000	$450,000
Total net present value	$7,060,000	$9,130,000	$4,410,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $10,000.
(c) Rounded to the nearest $1,000.
(d) Includes annual average costs of about $20,000 for labor, $3,000 for energy, and $190,000 for chemicals.
(e) Includes annual average costs of about $20,000 for labor, $13,000 for energy, and $15,000 for replacement of lamps and other UV system components (e.g., ballasts, quartz sleeves, and wipers).

6.2.6.4 Non-economic Comparison of Disinfection Alternatives

A comparison of the disinfection alternatives considering non-economic factors was performed. The results of this analysis are summarized in Table 6-22.

Table 6-22: Non-Economic Comparison of Disinfection Alternatives

Disinfection Alternative	Sustainability (30%)	Ease of Permitting (20%)	Community Considerations (20%)	Water Conservation (10%)	Energy Conservation (10%)	Environmental Considerations (10%)	Weighted Score
Chlorination							
Expand Existing	3	4	4	3	4	4	**3.6**
New CCBs	4	5	4	4	3	4	**4.1**
UV	5	5	5	5	5	5	**5.0**

As shown in Table 6-21, the UV disinfection alternative scored the highest in the non-economic comparison of disinfection alternatives. Reasoning for the scores assigned within each of the six categories is summarized below.

- **Sustainability.** Converting to UV disinfection ranked highest of the three disinfection alternatives considered. This was because of reduced complexity, elimination of chemical handling, improved safety, and ease of future expansion to meet changing permit requirements. The continued use of the existing CCBs, which would include construction of one new CCB, ranked lowest of the alternatives. This was because of the associated chemical feed systems and handling, difficulty of future expansion, and deficiencies in the existing design (e.g., length to width ratio) that may make it more difficult to meet discharge limits.

- **Ease of Permitting.** None of the disinfection alternatives considered should be difficult to gain approval from state or local agencies. However, continued use of the existing CCBs may face additional scrutiny from the DEQ because of its low length to width ratio.

- **Community Considerations.** UV disinfection ranked the highest of the three disinfection alternatives. This is because of its small size with less potential for construction period impacts (e.g., noise, traffic, visual). Because sodium hypochlorite would not be used as the primary disinfection agent, its use would be drastically reduced. The use of sodium bisulfite for dechlorination would be eliminated entirely. This would reduce truck traffic and potential concerns with chemical use.

- **Water Conservation.** Chlorination was ranked lower than UV disinfection because it is less suitable for use in disinfecting nitrified effluent to recycled water standards. The City's NPDES permit requires the WWTP to nitrify. Because of this, using chlorine for

disinfection could negatively impact the City's ability to generate recycled water. In the absence of sufficient ammonia, dosing chlorine at the high concentrations needed to achieve recycled water standard can cause conditions to move in and out of breakpoint chlorination making it difficult to achieve consistent disinfection results.

- **Energy Conservation.** UV disinfection ranked higher than both chlorination alternatives in terms of energy conservation. Although the estimated annual energy (electric) consumption for chlorination is about 40,000 kW-hrs compared with 150,000 kW-hrs for UV disinfection, the greenhouse gas emissions for chlorination is estimated to be more than five times greater than that of UV disinfection. The significant difference in estimated greenhouse gas emissions is mainly associated with the emissions from the production and transport of sodium hypochlorite. In addition, construction of CCBs require significantly more structural material (e.g., concrete, steel, etc.) compared to UV facilities.

- **Environmental Considerations.** None of the disinfection alternatives should have significant negative impacts to flood plains or biological, historical, or cultural resources. UV disinfection ranked higher than chlorination because of the reduction or complete elimination of chemicals even though construction of UV facilities would create construction related waste (e.g., demolition of the existing CCBs). Additionally, UV light with the addition of some chemicals is showing promising results in the destruction of certain "forever" constituents (e.g., specific types of PFAS).

6.2.6.5 Recommended Disinfection Alternative

Replacing the existing CCBs with a new UV disinfection system is recommended. The UV disinfection alternative has the lowest initial capital cost and net present worth value of the disinfection alternatives considered. UV disinfection also scored significantly higher than other alternatives in the non-economic comparison. Advantages of a new UV disinfection system include reduced complexity, elimination of chemical handling, improved safety, and ease of future expansion to meet changing permit requirements.

6.2.7 Effluent Pumping

As described in Section 5, the WWTP's existing hydraulic grade line and elevation of its secondary clarifier results in the continued need for effluent pumping. Improvements to the existing pumping station are needed to increase its capacity to meet projected design year 2045 peak flow conditions. Improvements to the existing station would include construction of an additional wet well bay to house a new 100-hp pump. In addition, the station's two small 20-hp pumps would be replaced with a single 100-hp pump. The station would thus have a total of four, 100-hp pumps (3 duty and 1 standby).

Planning level design criteria for secondary effluent pump station modifications are presented in Table 6-23.

Table 6-23: Effluent Pump Station Design Criteria

Parameter	Units	Value
Pumps		
Type	-	Submersible centrifugal
Number	-	4[(a)]
Capacity, each	gpm	7,200
TDH	ft	40
Motor power, each	hp	100
Drive type	-	Variable speed
Station capacity	MGD	31[(b)]

Notes:

(a) Includes one redundant pump.
(b) Firm capacity with one pump out of service.

Estimated costs for modifications to the existing effluent pumping station, including capital, annual O&M, and salvage value, are summarized in Table 6-24.

Table 6-24: Effluent Pumping Station Cost Summary[(a)(b)]

Item	Value
Capital cost	
Construction	$880,000
Contingency	$260,000
Escalation	$240,000
Engineering and Admin.	$350,000
Total capital cost	$1,730,000
O&M cost	
Annual average[(c)(d)]	$24,000
20-yr net present value	$400,000
Salvage value	$270,000
Total net present value	$1,860,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $10,000.
(c) Includes an annual average labor cost of approximately $4,000, annual energy cost of about $15,000, and equipment maintenance of about $4,000.
(d) Rounded to the nearest $1,000.

6.3 Recycled Water Alternatives

As described in Section 5, recycled water Alternative 2 was selected for further evaluation. This alternative would be implemented if the City lost its ability to discharge to the South Santiam River during the months of May through October. It represents the more conservative design. Improvements would include construction of a 75-acre recycled water storage reservoir, recycled water pumps station, and transmission pipeline. A booster pump station and irrigation equipment would also be required; however, the design of these facilities and funding for the facilities could be shared with the property owner(s). The improvements were based on

projected design year 2045 dry weather effluent flows, including MMDWF conditions for the month of May. Planning level design criteria for the recycled water system are presented in Table 6-25.

Table 6-25: Recycled Water Design Criteria

Parameter	Units	Value
Storage Reservoir		
Area	acres	75
Depth	ft	10
Reuse Pumps		
Type	-	Vertical turbine
Number	-	4
Drive Type	-	Variable speed
Capacity, each	gpm	2 @ 500
		2 @ 4,700[(a)]
Motor power, each		
	hp	2 @ 7.5
	hp	2 @ 300
Flow, total		
Minimum	gpm	200
Maximum	gpm	5,700
TDH	ft	40
Minimum	ft	88
Maximum	ft	133
Station capacity	gpm	5,700[(b)]
Transmission Pipeline		
Length	ft	10,560
Diameter	in	24
Material	-	HDPE

Notes:

(a) Includes one redundant pump.
(a) Firm capacity with one pump out of service.

Estimated costs for the recycled water system, exclusive of the booster pump station and irrigation equipment, are summarized in Table 6-26. Costs include initial capital, annual O&M, and salvage value.

Table 6-26: Recycled Water Cost Summary[a][b]

Item	Value
Capital cost	
Construction	$17,930,000
Land Acquisition	$1,500,000
Contingency	$5,380,000
Escalation	$4,850,000
Engineering and Admin.	$7,040,000
Total capital cost	$36,700,000
O&M cost	
Annual average[c]	$72,000
20-yr net present value	$1,100,000
Salvage value	$5,480,000
Total net present value	$32,320,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $1,000.
(c) Includes an annual average labor cost of approximately $6,000, annual energy cost of about $56,000, and equipment maintenance of about $9,000.

As presented in Section 2 of this Plan, Reasonable Potential Analyses (RPAs) for toxics, ammonia, chlorine, copper, pH and thermal plume as well as an evaluation of existing and potential Total Maximum Daily Loads (TMDLs) including the soon to be finalized temperature TMDL showed nothing to suggest the DEQ would not allow continued year-round discharge to the South Santiam River. While future changes in regulatory requirements may dictate reevaluation of this conclusion, there is currently no regulatory or economic driver to suggest development of a recycled water program.

6.4 Solids Stream Treatment Alternatives

The following provides a summary of the alternative selection process for improvements to the solids stream treatment processes of the City's WWTP. Processes evaluated included 1) solids thickening, 2) stabilization (i.e., digestion), 3) Class B beneficial use, 4) Class A beneficial use, and 5) landfill disposal.

6.4.1 Solids Thickening Alternatives

The continued use of solids thickening is recommended to reduce the needed volume of potential downstream stabilization processes (e.g., aerobic digestion or anaerobic digestion). Although the WWTP currently uses a gravity belt thickener (GBT) that has sufficient capacity to meet projected design year 2045 solids production, the existing equipment was installed in 1993 and is well beyond the 20-year useful life typically assumed for mechanical equipment. Four alternatives were thus selected for initial consideration to replace the existing GBT. These included 1) rotary drum thickeners (RDTs), 2) a new GBT, 3) centrifuges, and 4) dissolved air flotation. As described in Section 5, the initial screening analysis selected rotary drum thickening as the preferred generic thickening technology to replace the existing GBT. Costs associated with new thickening facilities will be incorporated as appropriate into overall WWTP improvement projects recommended in this Plan.

Design criteria for solids thickening are presented in Table 6-27. Improvements to the WWTP's existing thickening facilities were assumed to include a new RDT, polymer feed system, sludge feed pump, and thickened sludge pump. Improvements to the building heating, ventilation system, and air conditioning system were also assumed.

Table 6-27: Solids Thickening Design Criteria

Parameter	Units	Value
Thickener		
Type	-	Rotary drum
Number	-	1
Capacity[(a)]	gpm	420
TWAS Concentration	%	4
Motor power, each	hp	3
Sludge Feed Pump		
Type	-	Rotary lobe
Number	-	1
Capacity	gpm	420
Motor power, each	hp	25
TWAS Pump		
Type	-	Rotary lobe
Number	-	1
Capacity	gpm	80
Motor power, each	hp	7.5

Note:

(a) Based on an inlet WAS concentration of 0.75% and outlet TWAS concentration shown.

Estimated costs for thickening assuming replacement of the WWTP's existing GBT with a new rotary drum thickener and other associated improvements are presented in Table 6-28.

Table 6-28: Solids Thickening Cost Summary[(a)(b)]

Item	Value
Capital cost	
Construction	$1,120,000
Contingency	$340,000
Escalation	$300,000
Engineering and Admin.	$440,000
Total capital cost	$2,200,000
O&M cost	
Annual average[(c)(d)]	$180,000
20-yr net present value	$2,880,000
Salvage value	$340,000
Total net present value	$4,740,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $10,000.

(c) Includes an annual average labor cost of about $15,000, energy cost of $1,000, polymer cost of $160,000 and equipment maintenance of $4,000.
(d) Rounded to the nearest $1,000.

6.4.2 Stabilization Alternatives

Aerobic digestion and anaerobic digestion were the two alternatives selected for further consideration to provide stabilization of biosolids needed to meet Class B regulatory requirements.

6.4.2.1 Aerobic Digestion

As described in Section 5, the WWTP's existing 530,000-gallon aerobic digester, originally constructed in 1976, does not have sufficient volume to reliably produce Class B biosolids. Two aerobic digestion alternatives were considered to address existing deficiencies. These consisted of 1) continued use of the existing digester with construction of additional digester volume and 2) complete replacement of the existing aerobic digester with a new larger aerobic digestion facility.

Continued use of the existing aerobic digester would necessitate construction of a new separate aerobic digester to provide the additional volume needed to achieve detention times to meet Class B requirements. Coarse bubble aeration would be used to provide needed aeration for both the existing and new digester. Mechanical aeration equipment in the existing digester would be replaced with a coarse bubble aeration system. Coarse bubble aeration would also be used in the new digester. Positive displacement blowers would be provided to supply air to both digesters. Submersible mixers would be provided in each digester to ensure complete mixing of digester contents.

Planning level design criteria assuming the continued use of the existing aerobic digester are presented in Table 6-29.

Table 6-29: Aerobic Digestion Design Criteria with Existing Digester

Parameter	Units	Value
Digesters		
Number	-	2
Number of cells		4
Existing digester	-	1
New digester	-	2
Volume, total		
Existing digester	gal	530,000
New digester	gal	410,000
Total	gal	940,000
Depth, water		
Existing digester	ft	10
New digester	ft	17
HRT[(a)]	days	60

Parameter	Units	Value
Aeration		
Type	-	Coarse bubble
Blowers		
Type	-	Positive displacement
Number[(b)]	-	4
Capacity, each	scfm	1,250
Motor power, each	hp	100
Blower building	sf	1,200
Mixing		
Type	-	Submersible propeller
Number, total[(c)]	-	4
Motor power, each	hp	10

Notes:

(a) Value under projected design year 2045 maximum month loading conditions. HRT of 60 days is required to meet 40 CFR Part 503 and OAR 340-050 requirements for Class B biosolids treatment at design temperature 15°C.
(b) Total number needed to supply aeration air to both new and existing digesters. Number includes one redundant blower.
(c) Two mixers per digester.

The second aerobic digestion alternative considered assumed complete replacement of the existing digester with a new larger digester structure. A coarse bubble aeration system with positive displacement blowers and blower building would be provided. Submersible mixers would be provided to ensure mixing of digester contents.

Planning level design criteria for aerobic digestion alternatives are presented in Table 6-30.

Table 6-30: Aerobic Digester Design Criteria with All New Digester Facilities

Parameter	Units	Value
Digester		
Number	-	1
Number of cells	-	4
Volume, total	gal	940,000
Depth, water	ft	17
HRT[(a)]	days	60
Aeration		
Type	-	Coarse bubble
Blowers		
Type	-	Positive displacement
Number[(b)]	-	4
Capacity, each	scfm	1,250
Motor power, each	hp	100
Blower building	sf	1,200

Parameter	Units	Value
Mixing		
Type	-	Submersible propeller
Number, total[(c)]	-	4
Motor power, each	hp	10

Notes:

(a) HRT to meet 40 CFR Part 503 and OAR 340-050 requirements for Class B biosolids treatment at design temperature 15°C.
(b) Number includes one redundant blower.
(c) One mixer per digester cell.

6.4.2.2 Anaerobic Digestion

This alternative would replace the existing aerobic digester with new anaerobic digestion facilities. These would include construction of a new 40-ft-diameter concrete anaerobic digester, heat exchange equipment, gas handling equipment, waste gas burner, and control building. Addition of primary treatment (i.e., primary filtration) was assumed and included in estimated costs for the anaerobic digestion alternative.

Digester cover alternatives include floating steel, fixed steel, and fixed concrete. Use of a fixed steel digester cover was assumed for the purposes of this analysis due to its cost effectiveness and wide use. Digester mixing was assumed to be provided by a linear motion type mixer an example of which is shown on Figure 6-1. Other mixing alternatives, including pumped mixing and draft tube mixing systems, could be considered but are typically more costly. Should anaerobic digestion be selected as the preferred stabilization alternative, additional evaluation of digester covers and mixing should be performed prior to detailed design.

Figure 6-1: Linear Motion Mixer (Source: Ovivo)

Planning level design criteria for anaerobic digestion is presented in Table 6-31. Costs for all stabilization alternatives, including anaerobic digestion, are presented in Table 6-32.

Table 6-31: Anaerobic Digestion Design Criteria

Parameter	Units	Value
Digester		
Type	-	Mesophilic
Number	-	1
Volume	gal	330,000
Diameter	ft	40
Depth, water	ft	38
Cover type	-	Fixed, steel
HRT[(a)]	days	20
Mixing		
Type	-	Linear motion
Number	-	1
Motor power, each	hp	7.5
Control Building	sf	2,600
Digester Heating		
Recirculation pumps		
Type	-	Screw centrifugal
Number	-	1
Capacity	gpm	500
Motor power	hp	7.5
Heat Exchangers		
Type	-	Spiral
Number	-	1
Boiler		
Number	-	1
Fuel	-	Duel[(b)]
Capacity	kBTU/hr	360
Hot water pumps		
Type	-	Centrifugal
Number	-	1
Motor power, each	hp	3
Waste Gas Burner		
Type	-	Candle stick
Capacity	scfh	2,600

Notes:

(a) Value under projected design year 2045 maximum month loading conditions. A minimum HRT of 15 days is required to meet 40 CFR Part 503 requirements for Class B biosolids treatment at design temperature 35°C.

(b) Capable of utilizing both natural gas and digester gas.

6.4.2.3 Economic Comparison of Stabilization Alternatives

Estimated costs for stabilization alternatives including sub-alternatives for aerobic digestion are summarized in Table 6-32. As shown, the least cost alternative in terms of both initial capital cost and long-term O&M costs is aerobic digestion assuming continued use of the existing 540,000-gallon digester with addition of a new separate 430,000-gallon digester. The cost of this alternative includes converting the existing digester from surface aeration to coarse bubble aeration with the addition of associated blowers and blower building. It relies on the continued use of the existing digester whose original construction predates the 1976 WWTP improvement project that converted the structure from its original configuration as a trickling filter into an aerobic digester.

Of the alternatives considered, anaerobic digestion is significantly more costly. The higher cost of the alternative is partially because of the need for primary treatment, the costs of which are included in the values shown.

Table 6-32: Economic Comparison of Stabilization Alternatives[a][b]

Item	Aerobic Digestion		Anaerobic Digestion[c]
	Expand Existing	New Digesters	
Capital cost			
Construction	$3,960,000	$5,160,000	$9,950,000
Contingency	$1,190,000	$1,550,000	$2,990,000
Escalation	$1,070,000	$1,400,000	$2,690,000
Engineering and Admin.	$1,560,000	$2,030,000	$3,910,000
Total capital cost	$7,780,000	$10,140,000	$19,540,000
O&M cost			
Annual average[d]	$267,000[e]	$267,000[e]	$153,000[f]
20-yr net present value	$4,370,000	$4,370,000	$2,490,000
Salvage value	$1,880,000	$2,800,000	$3,250,000
Total net present value	$10,270,000	$11,710,000	$18,780,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $10,000.
(c) Costs shown for anaerobic digestion includes the addition of primary filtration which would be needed for anaerobic digestion.
(d) Rounded to the nearest $1,000.
(e) O&M costs assumed to be the same for both aerobic digestion alternatives because both entail the same digester volume and aeration blower system. Cost includes annual average costs of about $70,000 for labor, $190,000 for energy, and $6,000 for maintenance.
(f) Includes annual average costs of about $85,000 for labor, $30,000 for energy, and $38,000 for maintenance.

6.4.2.4 Non-economic Comparison of Stabilization Alternatives

A comparison of the stabilization alternatives considering non-economic factors was performed. The results of this analysis are summarized in Table 6-33.

Table 6-33: Non-Economic Comparison of Stabilization Alternatives

Stabilization Alternative	Sustainability (30%)	Ease of Permitting (20%)	Community Considerations (20%)	Water Conservation (10%)	Energy Conservation (10%)	Environmental Considerations (10%)	Weighted Score
Aerobic Digestion							
Expand Existing	4	4	5	0	2	5	**3.7**
New Digesters	5	5	5	0	3	4	**4.2**
Anaerobic Digestion	3	5	4	0	5	4	**3.6**

As shown in Table 6-33, construction of new aerobic digesters scored the highest in the non-economic comparison of the stabilization alternatives considered. Reasoning for the scores assigned within each of the six categories is summarized below.

- **Sustainability.** Construction of new aerobic digesters ranked the highest of the three stabilization alternatives considered. WWTP staff are accustomed to aerobic digestion. When compared to anaerobic digestion, it has fewer mechanical systems and is less operationally complex. When compared with the continued use of the existing digesters, construction of new aerobic digesters should also result in facilities that are easier to maintain and better able to meet regulatory requirements for Class B biosolids.

- **Ease of Permitting.** None of the stabilization alternatives considered should be difficult to gain approval from state or local agencies. However, the DEQ may be more likely to question continued use of the existing aerobic digester and its ability to meet Class B regulatory requirements.

- **Community Considerations.** The alternatives for stabilization ranked similarly with none anticipated to have significant negative impacts to the community. However, anaerobic digestion was ranked slightly lower because of the potential for odors associated with primary treatment and the digestion process itself. Anaerobic digestion would also require routine flaring of excess digester gas that may create concern with the local community.

- **Water Conservation.** None of the alternatives considered would impact the City's ability to implement a recycled water program. In addition, water consumption should be similar for all alternatives. For these reasons, a value of zero was assigned to all alternatives for this category.

- **Energy Conservation.** Anaerobic digestion ranked highest in terms of energy conservation. Annual energy use is expected to be less. Calculations showed that the overall carbon footprint of anaerobic digestion as compared to aerobic digestion would also be lower. This was in part due to the expectation that biogas generation would meet all heating needs of the digester. Expanding the existing digester ranked higher than constructing all new facilities because of the lower overall carbon footprint generated by reducing the amount of new construction materials needed.

- **Environmental Considerations.** None of the stabilization alternatives considered is anticipated to negatively impact flood plains or biological, historical, or archeological resources. However, even though it would include construction of some new facilities, continued use of the existing aerobic digesters was ranked higher because it would result in less construction related waste such as that created by demolition of the existing facilities.

6.4.2.5 Recommended Stabilization Alternative

Aerobic digestion is recommended for stabilization of biosolids. WWTP staff are accustomed to the use of aerobic digestion, and it is significantly less costly and less operationally complex than anaerobic digestion. Anaerobic digestion would also require the addition of primary treatment to the WWTP which adds considerable cost. Continued use of the existing aerobic digester is also recommended in lieu of constructing all new facilities. Doing so will be less costly. Although ranked lower in the non-economic comparison, continued use of the existing digester is better suited to the recommended Class A alternative discussed below that does not require full stabilization of biosolids to Class B regulatory requirements.

6.4.3 Dewatering Alternatives

As described in Section 5, an initial screening analysis of four solids dewatering alternatives was performed to identify one treatment technology that would be used when developing overall project alternatives for the WWTP. The alternatives considered included 1) belt filter presses (BFPs), 2) rotary screw presses, 3) centrifuges, and 4) fan presses. In 2022, the City installed a new BFP with capacity to meet projected design year 2045 solids production values. Because of this, BFPs were selected as the preferred dewatering alternative.

Design criteria for the existing dewatering system are presented in Table 6-34. Because the existing BFP has enough capacity to meet future operation conditions, an economic analysis of dewatering was not performed. However, WWTP staff have reported issues with the new BFP experiencing inconsistent dewatering results and an inability to meet the manufacturer's rated capacity. Should the City's efforts to improve the performance of the BFP fail, new dewatering facilities will be needed.

Table 6-34: Solids Dewatering Design Criteria[(a)]

Parameter	Units	Value
Dewatering Unit		
Type	-	Belt filter press
Number	-	1
Belt width	meter	1.7

Parameter	Units	Value
Capacity[(a)]	dry lbs/hr	600
Cake concentration	%	16
Polymer dose	lbs/dry ton	8 to 12
Drive motors		
Number	-	2
Power, each	hp	1.5
Sludge Feed Pump		
Type	-	Progressive cavity
Number	-	1
Capacity	gpm	25 to 125
Motor power, each	hp	5
Cake Conveyor		
Type	-	Shaftless screw
Number	-	1
Motor power, each	hp	2

Note:

(a) Values shown for existing BFP dewatering system.

6.4.4 Beneficial Use or Disposal Alternatives

Three options were considered related to the ultimate use or disposal of biosolids produced by the WWTP. These included 1) Class B beneficial use, 2) Class A beneficial use, and 3) landfill disposal of unstabilized solids.

6.4.4.1 Class B Beneficial Use Alternative

As described in Section 5, a screening analysis of two Class B biosolids beneficial use alternatives was performed that identified dewatered cake land application as the preferred alternative. The Class B beneficial use alternative assumes continued use of the City's five existing authorized biosolids application sites. Solids would be dewatered utilizing the WWTP's existing BFP. New facilities needed for land application of dewatered cake would include a new storage building, conveyors, and cake handling equipment including slinger truck and wheel loader. Design criteria for the existing dewatering system are presented in Table 6-35.

Table 6-35: Class B Dewatered Cake Land Application Design Criteria

Parameter	Units	Value
Biosolids production[(a)]	Dry tons/yr	420
Application Sites		
Number	-	5[(b)]
Total area	acres	370
Application Season[(c)]	-	May thru October
Cake Storage[(d)]	ft^2	8,700
Conveyors		
Type	-	screw
Number	-	2
Motor power, each	hp	7.5

Parameter	Units	Value
Slinger Truck		
Number	-	1
Capacity	ton	10
Wheeled Loader		
Number	-	1
Capacity	CY	5

Notes:

(a) Projected design year 2045 average annual Class B biosolids production assuming aerobic digestion.
(b) There are five authorized application sites with only two currently in use.
(c) Specific application window is weather dependent but generally spans a 6-month period.
(d) Sized to permit 7 months of wintertime storage and temporary summertime storage for truck loadout during the application season. Although the City is authorized to land apply Class B biosolids for 6 months each year, 7 months of storage was assumed to account for annual variations in precipitation that may reduce the City's ability to land apply.

Estimated costs for new facilities needed specifically for land application of Class B dewatered cake are summarized in the first column of cost values shown on Table 6-36. These include a new wintertime storage building, conveyors, and cake handling equipment including slinger truck and wheel loader. Also shown are costs associated with improvements needed to produce Class B biosolids assuming expansion of the existing aerobic digester is the preferred Class B stabilization alternative. The totals shown, inclusive of both land application and aerobic digestion, are the estimated costs that would result should the City choose to implement a Class B land application program.

Table 6-36: Class B Dewatered Cake Land Application Cost Summary[(a)(b)]

Item	Land Application[(c)]	Aerobic Digestion[(d)]	Total
Capital cost			
Construction	$3,830,000	$3,960,000	$7,790,000
Contingency	$1,150,000	$1,190,000	$2,340,000
Escalation	$1,040,000	$1,070,000	$2,110,000
Engineering and Admin.	$980,000[(e)]	$1,560,000	$2,540,000
Total capital cost	$7,000,000	$7,780,000	$14,780,000
O&M cost			
Annual average[(f)]	$79,000[(g)]	$267,000[(h)]	$346,000
20-yr net present value	$1,290,000	$4,370,000	$5,660,000
Salvage value	$1,800,000	$1,880,000	$3,680,000
Total net present value	$6,490,000	$10,270,000	$16,760,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $10,000.
(c) Assumes expansion of the WWTP's existing aerobic digester and conversion to coarse bubble aeration.
(d) Assumes expansion of WWTP's existing digester with construction of additional digester volume to meet projected design year 2045 solids production.

(e) Engineering associated with cake storage buildings and conveyors.
(f) Rounded to the nearest $1,000.
(g) Includes annual average costs of about $57,000 for labor and $22,000 for maintenance and consumables.
(h) Includes annual average costs of about $70,000 for labor, $190,000 for energy, and $6,000 for maintenance.

6.4.4.2 Class A Beneficial Use Alternative

As described in Section 5, a screening analysis of three Class A biosolids beneficial use alternatives was performed. The alternatives considered were 1) composting, 2) thermal drying, and 3) lime/heat pasteurization. Of these, thermal drying was identified as the preferred Class A beneficial use alternative.

The Class A beneficial use alternative assumes construction of a thermal drying system sized to treat projected design year 2045 solids production for the WWTP. As previously described, dryer types that would be appropriate for this application include belt and paddle type configurations. Because they are better suited for intermittent operation (e.g., 8 hours per day), use of a belt type dryer was assumed for the purposes of this evaluation. Paddle dryers have longer startup and shutdown requirements and are thus better suited for continuous 24-hour operation.

Design criteria for a belt type dryer system are presented in Table 6-37. Thermal dryer facilities would include a 6,500 square foot, pre-engineered metal building and odor control. Dryer equipment would include the belt dryer, dewatered cake hopper, screw conveyors, and bagging system (e.g., super sacks) for use in distributing the final Class A product. Depending on the belt dryer manufacturer, additional system components may be included. For example, the heat source used may either be an indirect gas burner or a hot water boiler. Other components may include hot water circulation pumps, screw conveyors, exhaust air condensing units, belt cooler, and screener/crushers.

Table 6-37: Class A Thermal Dryer Design Criteria

Parameter	Units	Value
Biosolids production[(a)]		
Average annual	dry lbs/day	2,300
Maximum month	dry lbs/day	3,700
Operating time	hrs/week	40[(b)]
Biosolids Concentration		
Dewatered cake	%	18
Class A Product	%	92
Dryer type	-	Belt
Fuel type	-	Natural gas
Dryer Capacity	dry lbs/hr	650
Dryer building	sf	6,500

Notes:
(a) Projected design year 2045 average annual Class B biosolids production assuming aerobic digestion.
(b) Operation assumed to be 8 hours per day, 5 days per week.

Estimated costs for the Class A belt dryer alternative are summarized in Table 6-38 and Table 6-39. Values were developed for two potential dryer manufacturers: Centrisys and Andritz. As shown, the capital costs associated with the two manufacturers differ significantly with that of Andritz being about 75% greater than that of the Centrisys based system. Both manufacturers would require the same size building (e.g., 6,500 sf) and would have similar labor and energy costs. However, the planning level estimates provided by the manufacturers for their equipment differed by almost two times. The dryer system provided by Andritz is more mechanically complex than that provided by Centrisys and included a greater amount of equipment internal to the dryer system itself. While both manufacturers have installations in the United States, the number of Andritz installations exceeds those of Centrisys. Because an in-depth evaluation and selection of specific dryer manufacturers was outside the scope of this evaluation, costs for both systems were developed and are shown below to provide a range of potential costs that could be associated with implementing a Class A dryer system.

Although thermal dryers can produce Class A biosolids from unstabilized wastewater solids, doing so significantly increases the risk of nuisance odors generated when the final product is wetted. Because this would have a significant impact on the public acceptance of the final product, providing even partial stabilization before drying is recommended. Continued use of the WWTP's existing aerobic digester would provide more than 30 days of detention time under projected design year 2045 solids production conditions. Providing this amount of stabilization should greatly reduce the potential for odor from the final product. For this reason, costs shown in Table 6-38 and Table 6-39 include those assuming continued use of the existing aerobic digester with conversion from mechanical to coarse bubble aeration.

Table 6-38: Class A Thermal Dryer Cost Summary: Centrisys[(a)(b)]

Item	Centrisys Dryer	Partial Stabilization[(c)]	Total
Capital cost			
Construction	$9,110,000	$1,190,000	$10,300,000
Contingency	$2,730,000	$360,000	$3,090,000
Escalation	$2,460,000	$320,000	$2,780,000
Engineering and Admin.	$3,580,000	$470,000	$4,040,000
Total capital cost	$17,880,000	$2,340,000	$20,210,000
O&M cost			
Annual average[(d)]	$116,000	$150,000	$266,000[(e)]
20-yr net present value	$1,870,000	$2,460,000	$4,330,000
Salvage value	$1,460,000	$540,000	$2,000,000
Total net present value	$18,290,000	$4,260,000	$22,540,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $10,000.
(c) Assuming continued use of the existing aerobic digester with conversion to coarse bubble aeration.
(d) Rounded to the nearest $1,000.
(e) Includes annual average costs of about $76,000 for labor, $165,000 for energy, and $25,000 for maintenance.

Table 6-39: Class A Thermal Dryer Cost Summary: Andritz[a][b]

Item	Andritz Dryer	Partial Stabilization[c]	Total
Capital cost			
Construction	$16,090,000	$1,190,000	$17,280,000
Contingency	$4,830,000	$360,000	$5,190,000
Escalation	$4,350,000	$320,000	$4,670,000
Engineering and Admin.	$6,320,000	$470,000	$6,790,000
Total capital cost	$31,590,000	$2,340,000	$33,930,000
O&M cost			
Annual average[d]	$138,000	$150,000	$288,000[e]
20-yr net present value	$2,230,000	$2,460,000	$4,690,000
Salvage value	$1,460,000	$540,000	$2,000,000
Total net present value	$32,360,000	$4,260,000	$36,620,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $10,000.
(c) Assuming continued use of the existing aerobic digester with conversion to coarse bubble aeration.
(d) Rounded to the nearest $1,000.
(e) Includes annual average costs of about $76,000 for labor, $165,000 for energy, and $47,000 for maintenance.

6.4.4.3 Biosolids Disposal Alternatives

Landfill disposal was the third of three overall options considered related to the ultimate use and/or disposal of biosolids produced by the WWTP. It would include temporary storage of wastewater solids in the WWTP's existing aerobic digester. The WWTP's existing BFP would be used for solids dewatering with dewatered cake being conveyed to roll-off bins that would be used by the waste hauler for transport to the landfill. For the purposes of this evaluation, it was assumed that solids would be landfilled at the Coffin Butte Landfill until year 2035 after which time it would stop accepting wastewater solids. For years 2035 until 2045, it was assumed the solids would then be transported to the Columbia Ridge Landfill in Arlington, Oregon. A pre-engineered building would be provided for temporary storage of the roll-off bins. The building would be provided with an odor control system to reduce the potential for nuisance odors associated with the handling and temporary storage of unstabilized wastewater solids.

Design criteria for the landfill disposal alternative are presented in Table 6-40.

Table 6-40: Landfill Disposal Design Criteria

Parameter	Units	Value
Wastewater solids production[a]	Wet tons/yr	3,200
Landfill location		
Until 2035	-	Coffin Butte
Post 2035	-	Columbia Ridge
Weight per load[b]	ton	10

Parameter	Units	Value
Hauling frequency[(c)]	trips/week	6
Storage building	sf	1,650

Notes:

(a) Projected design year 2045 average annual wastewater solids production assuming no stabilization.
(b) Includes approximately 1.5 ton of bin liner material (e.g., woody debris) required by landfill to facilitate dumping of dewatered wastewater solids.
(c) Average annual frequency for projected design year 2045 solids production.

Estimated costs for the landfill disposal alternative are summarized in Table 6-41 and assume continued use of the existing aerobic digester mechanical aeration and mixing systems. The aeration system, while still in use, was installed in 1976 when the digester was converted from its original use as a trickling filter and is well beyond the 20-year useful life typically assumed for mechanical equipment. Replacing the existing mechanical aeration/mixing systems would add significant cost to this alternative.

Table 6-41: Biosolids Landfill Disposal Cost Summary[(a)(b)]

Item	Value
Capital cost	
Construction	$1,120,000
Contingency	$340,000
Escalation	$300,000
Engineering and Admin.	$440,000
Total capital cost	$2,220,000
O&M cost	
Annual average[(c)(d)]	$650,000
20-yr net present value	$9,940,000
Salvage value	$340,000
Total net present value	$11,800,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $10,000.
(c) Includes annual average costs of about $18,000 for labor and $630,000 for hauling/tipping. Hauling and tipping values calculated assuming solids are landfilled at Coffin Butte Landfill until 2035 after which time it was assumed it would no longer accept wastewater solids. Solids would then be transported to the Columbia Ridge Landfill in Arlington, Oregon.
(d) Rounded to the nearest $1,000.

6.4.4.4 Economic Comparison of Beneficial Use or Disposal Alternatives

Estimated costs for beneficial use or disposal alternatives including sub-alternatives for two thermal drying options are summarized in Table 6-42. As shown, the least cost alternative is landfill disposal with significantly lower initial capital cost. However, long term O&M costs for landfilling are greater than the other alternatives considered. This was because of the potential for Coffin Butte Landfill ceasing to accept wastewater solids at its facilities. As described above, this evaluation assumed that Coffin Butte would not be available after year 2035 at which time solids would be transported to the Columbia Ridge Landfill in Arlington, Oregon.

Of the alternatives considered, Class A beneficial use via thermal drying was estimated to be the costliest. However, as is shown below, Class A beneficial use was the highest ranked alternative in terms of the non-economic factors.

Table 6-42: Economic Comparison of Beneficial Use or Disposal Alternatives[a][b]

Item	Class B Beneficial Use[c]	Class A Beneficial Use[d] Centrisys Dryer	Class A Beneficial Use[d] Andritz Dryer	Landfill Disposal[e]
Capital Cost				
Construction	$7,790,000	$10,300,000	$17,280,000	$1,120,000
Contingency	$2,340,000	$3,090,000	$5,190,000	$340,000
Escalation	$2,110,000	$2,780,000	$4,670,000	$300,000
Engineering and Admin.	$2,540,000	$4,040,000	$6,790,000	$440,000
Total capital cost	$14,780,000	$20,210,000	$33,930,000	$2,200,000
O&M Cost				
Annual average[f]	$346,000	$266,000	$288,000	$650,000
20-yr net present value	$5,660,000	$4,330,000	$4,690,000	$9,940,000
Salvage Value	$3,680,000	$2,000,000	$2,000,000	$340,000
Total Net Present Value	$16,760,000	$22,540,000	$36,620,000	$11,800,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $10,000.
(c) Costs shown include those associated with the continued use and expansion of the existing aerobic digester. Conversion of the existing digester to coarse bubble aeration is included.
(d) Costs shown include those associated with the continued use of the existing aerobic digester and conversion to coarse bubble aeration.
(e) Assumes continued use of the existing aerobic digester for temporary liquid sludge holding. No improvements to the existing digester are included.
(f) Rounded to the nearest $1,000.

6.4.4.5 Non-economic Comparison of Beneficial Use or Disposal Alternatives

A comparison of the beneficial use or disposal alternatives considering non-economic factors was performed. The results of this analysis are summarized in Table 6-43.

Table 6-43: Non-Economic Comparison of Beneficial Use or Disposal Alternatives

Beneficial Use and Disposal Alternative	Sustainability (30%)	Ease of Permitting (20%)	Community Considerations (20%)	Water Conservation (10%)	Energy Conservation (10%)	Environmental Considerations (10%)	Weighted Score
Class B Beneficial Use	4	4	3	0	5	5	3.6
Class A Beneficial Use	5	5	5	0	3	4	4.2
Landfill Disposal	2	5	2	0	1	2	2.6

As shown in Table 6-43, Class A beneficial use with thermal drying scored the highest in the non-economic comparison of the alternatives considered for beneficial use or disposal of biosolids. Reasoning for the scores assigned within each of the six categories is summarized below.

- **Sustainability.** Of the three alternatives considered, landfilling ranked the lowest for sustainability. The low ranking was associated with WWTP staff having to handle unstabilized wastewater solids and this alternatives' lack of sustainability in that the willingness of landfills to accept waste solids may cease at any time. Although mechanically less complex than Class A thermal drying, Class B beneficial use would require significant staff effort for land application of dewatered cake.

- **Ease of Permitting.** None of the beneficial use or disposal alternatives considered should face significant difficulty in gaining approval from state or local agencies. However, land application of Class B biosolids will require modifications to the City's existing biosolids management plan before dewatered cake can be land applied. Land application will also require more on-going regulatory effort in terms of annual reporting. For these reasons, it was thus ranked lowest of the alternatives considered.

- **Community Considerations.** The production of Class A biosolids via thermal drying ranked the highest of the alternatives considered. Public acceptance of Class A biosolids is greater and due to its much reduced mass there would be significantly less traffic associated with hauling. Class B land application was ranked lower because of its associated truck traffic. Class B land application has also faced significant public scrutiny in other communities and the City could face similar future opposition. Landfilling of unclassified waste solids was ranked lowest because of truck traffic associated with hauling and greater potential for odor.

- **Water Conservation.** None of the alternatives considered would impact the City's ability to implement a recycled water program. In addition, water consumption should be similar

for all alternatives. For these reasons, a value of zero was assigned to all alternatives for this category.

- **Energy Conservation.** Landfill disposal was ranked lowest of the alternatives considered despite having lower estimated annual energy consumption. This was because its 20-year equivalent greenhouse gas emissions were estimated to be more than five times greater than Class B beneficial use and four times greater than Class A beneficial use. This was almost solely due to anticipated methane emissions that would be generated from landfilling biosolids. The Class B beneficial use alternative ranked higher than Class A because of its lower annual energy use.

- **Environmental Considerations.** None of the alternatives considered are anticipated to have significant impacts to flood plains or to biological, historical, or archeological resources. Landfilling was, however, ranked significantly lower because of the hauling and disposing of a potentially important resource that could otherwise be beneficially used to increase soil health and reduce the community's use of industrially produced fertilizers and soil amendments. The two beneficial use alternatives were ranked similarly. Class B beneficial use would utilize existing digester facilities and thus result in less construction related waste than Class A beneficial use.

6.4.4.6 Recommended Beneficial Use or Disposal Alternative

Although more costly than the other alternatives considered, the long-term development of a Class A biosolids beneficial use program with thermal drying is recommended. This is because of the significant uncertainty of future public acceptance of Class B land application and landfills ceasing to accept wastewater solids at their facilities. Class A biosolids beneficial use with thermal drying ranked highest in the non-economic comparison of the alternatives in part due to its better long-term sustainability, future uncertainty of the other alternatives, greater public acceptance, and (compared to landfilling) lower environmental impact. In addition, there would be far less biosolids produced as the result of implementing the Class A biosolids alternative and thus less labor assigned to managing the material.

6.5 I&I Cost Effectiveness Analysis and Recommendations

Inflow and Infiltration (I&I) can be addressed by conveying and treating it or by removing it through improvements to the collection system. An I&I analysis helps to determine the cost effectiveness of these alternatives.

6.5.1 I&I Cost Effectiveness Analysis

I&I is considered excessive if it is more economical to remove than to convey and treat it. The first step in evaluating the cost effectiveness of I&I removal is to compare wastewater flows with guidelines developed by the Environmental Protection Agency (EPA). These guidelines define per capita flows below which I&I can be considered non-excessive. If above the levels set by these guidelines, an economic evaluation is needed to determine if it is excessive.

EPA guidelines for non-excessive I&I are 125 gallons per capita per day (gpcd) and 275 gpcd for infiltration and inflow, respectively. If influent flow rates to the WWTP are below these threshold values, I&I can be considered non-excessive. If greater than the EPA values, a cost effectiveness analysis is needed to determine if it is less costly to remove than to convey and

treat. If it is less costly to remove, the I&I is deemed to be excessive. An evaluation of WWTP flows was presented in Section 3 of this Plan that showed both infiltration and inflow are above the EPA guideline values thus dictating the need for a cost effectiveness analysis to determine if I&I is excessive.

I&I affects sizing of treatment processes whose design is driven by peak flow conditions. These include pumping systems, preliminary treatment such as screening and grit removal, clarification processes such as primary and secondary clarifiers, and disinfection. Design of biological secondary unit treatment processes such as those considered in this Plan are not affected by peak flow rate but are instead designed based on mass loading. Mass loading is dependent on base sanitary sewer flows being generated and not the amount of extraneous water or I&I entering the collection system. Likewise, solids treatment systems are not affected by I&I.

The analysis presented here is based on the financial impact of providing more hydraulic treatment capacity than might otherwise be needed if I&I was reduced. Unit processes specific to this Plan that would be affected include the following:

- Influent pumping.
- Preliminary treatment including screening and grit removal.
- Secondary clarification.
- Effluent pumping.
- Disinfection.

Capital costs for the improvements previously described in this section for these unit processes were estimated for two alternatives. The first alternative assumed no reduction in current levels of I&I with the processes sized to treat projected design year 2045 influent flows. The second alternative assumed that I&I would be reduced to the EPA peak day threshold value of 275 gpcd below which inflow can be considered non-excessive.

Assuming the EPA threshold value and a projected design year 2045 population of 22,483 (PSU, 2021) would result in a peak day flow rate of 6.2 MGD. This is approximately 68% less than the projected design year 2045 peak day value of 19.4 MGD assuming existing rates of I&I. While reducing I&I to achieve a 68% reduction in flow would be an unusually aggressive goal, this value was used for the purposes of this evaluation. This was done to determine the maximum possible reduction in treatment cost that could potentially be achieved. The costs of the two alternatives are presented in Table 6-44 and show that $13.4 million could potentially be saved if the extremely aggressive 68% reduction in flow were possible.

Table 6-44: Potential Impact of I&I Removal on WWTP Improvement Costs[a][b]

Unit Process	Capital Cost No I/I Reduction	Capital Cost With I/I Reduction[c]	Difference
Influent Pumping (WIPS)	$4,700,000	$0	$4,700,000
Headworks[d]	$4,700,000	$4,100,000	$600,000
Secondary Clarifier	$11,900,000	$7,300,000	$4,600,000
Effluent Pumping	$1,700,000	$0	$1,700,000
Disinfection[e]	$4,100,000	$2,300,000	$1,800,000
Total Capital Cost	$27,100,000	$13,700,000	$13,400,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $100,000.
(c) Assumes peak WWTP flows reduced by 68%.
(d) Assumes construction of the new headworks alternative.
(e) Assumes construction of UV disinfection.

Reducing peak flow to the plant by 68% would eliminate the need to improve both the WIPS and Effluent Pump Station. This is because both are relatively new and their capacity would not have to be increased if the peak flow was reduced. Similarly, a 68% reduction in peak flow would increase the contact time within the existing CCBs above the minimum DEQ required values. While this could potentially eliminate the need for a new CCB, the cost change from chlorination to a new smaller capacity UV disinfection system is shown. This was done because of the age of the existing CCBs (constructed in 1976) and poor condition of the sodium hypochlorite storage facilities.

To reduce peak flows by 68%, significant improvements to the City's collection system would be needed. The costs of collection system improvements would have to be less than the potential $136.4 million reduction in WWTP improvements for it to be more cost effective than continuing to convey and treat. For reference, a planning level estimate of the replacement cost of the existing collection system was prepared and is summarized in Table 6-45.

Table 6-45: Estimated Replacement Cost of Collection System[(a)(b)]

System Component	Quantity	Replacement Cost
Gravity Pipe		
6 to 12-inch dia.	278,000 ft	$73,000,000
14 to 24-inch dia.	37,000 ft	$31,000,000
27 to 54-inch dia.	35,000 ft	$74,000,000
Manholes	1,400	$21,000,000
Laterals	4,600	$30,000,000
Total Replacement Cost		$229,000,000

Notes:

(a) AACE Class 4 planning level costs.
(b) Unless otherwise stated, values rounded to the nearest $1 million.

With a total replacement cost of $229 million, reducing the cost of improvements to the WWTP by $13.4 million would be enough to replace less than 6% of the collection system. Given the size and complexity of the City's collection system, projects equivalent to replacing this modest amount of the collection system cannot be expected to achieve the 68% reduction of peak flow needed to make it cost effective. Moreover, the City already has an on-going I&I removal program that has primarily focused on identifying and removing sources of inflow, the removal of which typically results in the most significant reduction in peak flow and is normally the most cost effective. However, despite these efforts, the WWTP has continued to experience relatively high peak flows. This further suggests that while improvements equivalent to the replacement of less than 6% of the collection system might reduce peak flows, they would not be sufficient to achieve the high level of I&I reduction that a 68% reduction in peak flows would require.

The City has a five phase, I&I reduction program. As described in Section 3 of the Plan, Phases 1 through 4 were specifically developed to eliminate inflow within the downtown area of the City by providing a separate stormwater system. Phases 1 and 2 have been completed. Phases 3 and 4 are included in the City's 2022-2027 capital improvement plan. Completion of these phases would eliminate all direct stormwater connections to the sanitary sewer system within the downtown area. The City has budgeted approximately $1.7 million for this work which has been estimated to reduce inflow by 1.3 MGD. The work will also complete smoke testing of the last 20% of the collection system.

6.5.2 I&I Cost Effectiveness Analysis Recommendations

As described in Section 7 of the Plan, recommended improvements to the WWTP are divided into three phases. The first phase will improve the WWTP's solids treatment process. The second phase will address significant capacity deficiencies in the WWTP's secondary treatment process and includes construction of a new oxidation ditch type process, headworks, and secondary clarifier. The third phase includes improvements to the WIPS, Effluent Pump Station, and disinfection.

Because capacity of solids treatment processes are not affected by flow, there is no need for the City to consider potential I&I reduction when determining timing of the solids stream improvements project. The City can thus proceed with the Phase 1 improvements project regardless of other efforts to reduce I&I.

The Phase 2 improvements project addresses deficiencies in unit process within the secondary treatment stream whose sizing is based on flow (e.g., headworks and secondary clarification). Because of this, the City should complete the planned I&I reduction projects in the downtown area and evaluate impact on peak flows to the WWTP prior to initiating work on the Phase 2 project. However, as shown in Table 6-45, even reducing flow by 68% would only result in an estimated $5.2 million in savings. While these savings could be used to fund improvements to the collection system, it would be equivalent to the replacement of only about 2% of the existing system. Because of this, while the City should complete the planned I&I reduction projects prior to initiating the Phase 2 project, it is not likely to result in significant costs saving.

The third improvement phase includes increasing the capacity of the WIPS and Effluent Pumping Station. It also includes construction of a new UV disinfection system. Because the capacity of these processes is directly related to flow, potential future reduction in I&I and thus peak flows to the WWTP should be considered. The City should thus complete the planned I&I reduction projects in the downtown area and evaluate impact on peak flows to the WWTP prior to the third improvement phase at the WWTP. The City should also complete smoke testing of the final 20% of the collection system as planned and continue its efforts to identify and eliminate sources of I&I into the collection system.

Section 7: Proposed Project

This section provides a description of the recommended improvements to the wastewater treatment system, recommendations for phasing, engineer's opinion of probable construction costs, and financial analysis.

7.1 Recommended Improvements

Recommended improvements to the wastewater treatment system include the following:

- Influent Pumping: Expand existing West Side Interceptor Pump Station (WIPS).
- Preliminary Treatment: Construct new headworks facilities including grit removal.
- Secondary Treatment: Replace the existing secondary treatment process with an oxidation ditch. Construct a new 130-foot (ft) secondary clarifier.
- Tertiary Treatment: Provide a cloth media tertiary filtration with 10 million gallons per day (MGD) capacity.
- Disinfection: Replace the existing chlorine disinfection system with ultraviolet (UV) disinfection.
- Effluent Pumping: Increase capacity of the existing Effluent Pump Station.
- Solids Thickening: Provide a new rotary drum thickener.
- Class A Biosolids: Construct a Class A belt drying system. Replace the existing digester's mixing/aeration system with coarse bubble aeration.
- General Site Improvements: Construct a new Control Building with new laboratory and provide new electrical, instrumentation, and control infrastructure.
- Inflow and Infiltration (I&I) Reduction: Complete Phases 3 and 4 of the City's existing I&I reduction program that includes eliminating inflow sources in the downtown area of the City and completing smoke testing of the collection system.

While the manufacturer's rated capacity of the existing belt filter press (BFP) is sufficient to meet design year 2045 operating conditions, WWTP staff have reported issues with the new BFP experiencing inconsistent dewatering results and an inability to meet the manufacturer's rated capacity. Should the City's efforts to improve the performance of the BFP fail, new dewatering facilities will also be needed.

As described in Section 1 of this Plan and in accordance with DEQ planning guidelines, recommended improvements were developed using wastewater flow and mass load projections based on the most recent PSU Population Research Center coordinated population forecast.

Because City staff believe growth will exceed that projected by PSU, immediately prior to initiating design of improvements to the WWTP, the most recent coordinated population forecast should be reviewed and values compared with those used to develop this Plan. Adjustments in design capacity values should then be made to accommodate differences in population estimates.

Design data for the treatment facilities that have recommended improvements are presented in Table 7-1.

Table 7-1: Design Data for Recommended Improvements

Item Description	Value
West Side Interceptor Pump Station (WIPS)	
Type	Submersible, non-clog
Number of pumps	
Large pumps	4
Small pumps	1
Firm capacity, mgd	31
Pump capacity	
Large pumps, gpm	7,200
Small pumps, gpm	3,600
Power	
Large pumps, hp	135
Small pumps, hp	50
Drive type	Variable speed
Headworks	
Mechanical screen	
Type	Climber
Number of screens	2
Capacity, ea, mgd	15.5
Opening, in	0.25
Power, ea, hp	1.5
Screenings washer	
Number	1
Power, hp	1.5
Grit removal	
Type	Vortex
Number	1
Diameter, ft.	18
Power, hp	2
Secondary Treatment	
Type	Oxidation ditch, Orbal®
No.	1
No. of channels	3
Total volume, MGal	2.3
Water depth, ft.	14
SRT, days	13.5
MLSS, mg/L	4,000

Item Description	Value
Mixers/aerators	
Type	Horizontal, surface disk
Number	6
Power, total, hp	315
Secondary Clarifiers	
Type	Suction, peripheral weir
Number, new/existing	1/1
Diameter, ft.	130
Water depth, ft.	16
Power, each, hp	1
RAS pumps	
Type	Submersible, non-clog
Number, each clarifier	3
Capacity, each, gpm	2,430
Power, each, hp	25
WAS Pump	
Type	Submersible, non-clog
Number, each clarifier	1
Capacity, each, gpm	300
Power, each, hp	3
Scum Pump	
Type	Submersible, non-clog
Number, each clarifier	1
Capacity, each, gpm	200
Power, each, hp	5
Tertiary Filtration	
Type	Synthetic cloth disk
Capacity, peak, mgd	10
Number of trains	1
Filter tanks	
Type	Concrete
Volume, gal.	15,400
Power, filter drive, hp	5
Backwash Pumps	
Type	Centrifugal
Number	1
Power, hp	20
Disinfection	
Type	UV, Low pressure
Configuration	Open channel
Number of trains	1
Banks per train, total	4
Number redundant banks	1
Minimum dose, mJ/cm2	30

Item Description	Value
Effluent Pumping	
Type	Submersible, centrifugal
Number, duty/standby	3/1
Station firm capacity, mgd	31
Pump capacity, each, gpm	7,200
Power, each, hp	100
Solids Thickening	
Thickener	
Type	Rotary drum
Number	1
Capacity, gpm	420
TWAS concentration, %	4
Power, hp	3
Sludge feed pump	
Type	Rotary lobe
Number	1
Capacity, gpm	420
Power, hp	25
TWAS pump	
Type	Rotary lobe
Number	1
Capacity, gpm	80
Power, hp	7.5
Partial Aerobic Stabilization	
Type	Existing digester
Volume, gal.	530,000
HRT	34
Aeration, type	Coarse bubble
Aeration blowers	
Type	Positive displacement
Number	2
Capacity, each, scfm	1,250
Power, each, hp	100
Mixing	
Type	Submersible propeller
Number	2
Power, each, hp	10
Class A Biosolids Treatment	
Type	Thermal dryer, belt-type
Number	1
Operating time, hrs/week	40
Biosolids concentration	
Dewatered cake, %	18
Class A biosolids, %	92

Item Description	Value
Fuel type	Natural gas
Dryer capacity, dry lbs/hr	650
Operating time, hrs/week	40
Dryer building size, sf	6,500

7.1.1 Influent Pumping

Influent pumping facilities at the wastewater treatment plant (WWTP), which include the currently inoperable Raw Sewage Pumping Station and the WIPS, do not have sufficient capacity to meet design year 2045 flows. The WIPS was constructed in 2002 and was designed to facilitate expansion to meet projected buildout flows. Expansion of the WIPS was selected as the recommended influent pumping alternative and will include replacing the station's four existing submersible pumps with five new pumps. The new pumps will consist of four, 135-horsepower (hp) submersible pumps and one, 50-hp pump. The smaller 50-hp pump will be used during lower flow periods to reduce the potential for excess cycling of the influent pumps.

7.1.2 Preliminary Treatment

Two preliminary treatment alternatives were considered. They included making improvements to the existing headworks or replacing them with new preliminary treatment facilities. Because of the age of the existing headworks and lack of grit removal, construction of new headworks with grit removal was selected as the preferred alternative. The recommended headworks will include new above grade concrete structures, two mechanically cleaned screens, screenings washer/compactor, a manual bar screen, vortex type grit removal chamber, and associated grit pump, cyclone separator, and grit classifier.

7.1.3 Secondary Treatment

Biological process modeling completed as part of this Plan showed the existing secondary treatment process does not have sufficient capacity to meet current maximum month loading conditions. Moreover, estimated capacity of the existing secondary treatment process is approximately 50% of that needed to meet projected design year 2045 loading conditions.

Four alternatives were considered to provide the necessary additional treatment capacity. These included expanding the existing activated sludge system or replacing the existing facilities with one of three potential technologies: 1) conventional activated sludge, 2) oxidation ditch, or 3) extended aeration. The oxidation ditch alternative was selected as the preferred treatment alternative because of its lower cost, simplicity of operation, ability to accommodate high peak flows, and proven reliability.

As previously described, oxidation ditch systems are a category of suspended growth treatment that use surface brush aerators or vertical turbine style mixers to aerate and continuously circulate mixed liquor around the circular pathway created by the basins oval shape. Conventional oxidation ditches are configured with a single circular path. A variation of this design is the Orbal® system by Evoqua. This style of oxidation ditch contains multiple, independent concentric rings that allows flow to progress from the outer rings to inner rings. The design is inherently well suited for WWTPs like the City's with highly variable influent flow rates. While use of the Orbal® type oxidation ditch has been assumed for planning purposes, selection

of the specific type of oxidation ditch should be done during the preliminary design phase for the recommended project.

Construction of a new, 130-ft diameter, secondary clarifier is recommended to replace the WWTP's, three smaller, aging, secondary clarifiers. The new clarifier will be like the WWTP's existing 130-ft diameter secondary clarifier and will include new return activated sludge (RAS), waste activated sludge (WAS), and scum pumping systems.

7.1.4 Tertiary Filtration

The use of tertiary filtration is recommended to help ensure the City can meet effluent discharge limits that, under certain circumstances, would require effluent biochemical oxygen demand (BOD) and total suspended solids(TSS) concentrations to be less than 10 mg/L. Of the three filtration technologies considered, synthetic/cloth media filtration was the selected alternative. Capacity of the filtration system will be 10 MGD. The filters will be constructed within concrete tanks located between the Effluent Pump Station and disinfection system.

7.1.5 Disinfection

The WWTP's existing two parallel chlorine contact basins (CCBs) do not have sufficient volume to provide the Department of Environmental Quality (DEQ) required detention times under high flow conditions (peak day and peak hour). Alternatives considered to address the identified deficiency included continued use of sodium hypochlorite to disinfect treatment plant effluent or conversion to UV disinfection. Continued use of sodium hypochlorite would require construction of new CCBs.

Because of its lower cost, reduced complexity, elimination of potentially hazardous chemicals, and ease of future expansion, UV disinfection was selected as the preferred disinfection alternative. The UV disinfection system will be an open channel, low-pressure type system. Although a single channel type UV disinfection system has been assumed for the purposes of this Plan, the advantages and disadvantages of providing a parallel channel type system should be evaluated during detailed design.

7.1.6 Effluent Pumping

The existing Effluent Pump Station does not have sufficient capacity to meet projected design year 2045 peak flow conditions. Expansion of the existing pumping station is recommended and will include construction of an additional wet well bay. Two new, 100-hp pumps will be provided one of which will replace the station's two, 20-hp pumps. The station will thus contain a total of four, 100-hp, variable speed, submersible pumps.

7.1.7 Solids Thickening

The continued use of solids thickening is recommended to increase the capacity of the existing aerobic digester. Although the WWTP's existing GBT has sufficient capacity to meet projected design year 2045 solids production, the existing equipment was installed in 1993 and is well beyond the 20-year useful life typically assumed for mechanical equipment.

Installation of a new rotary drum thickener is recommended. The new thickener will be installed in the existing Thickening Building. Other improvements will include a new polymer feed system,

sludge feed pump, and thickened sludge pump. Improvements will be made to the building's heating, ventilation, and air conditioning system.

7.1.8 Class A Beneficial Use

Implementation of a Class A biosolids beneficial use program is recommended to replace the City's existing Class B liquid beneficial use program. Although more costly, implementation of a Class A program would require less labor than that currently needed for land application of Class B biosolids. Class A biosolids also have a lower on-going regulatory burden and have higher community acceptance than land application of Class B biosolids which faces increasing community resistance.

Improvements needed to implement the recommended Class A beneficial use program will include construction of a belt-type thermal dryer including a new pre-engineered metal building, odor control, conveyance, and bagging system. To reduce the potential for nuisance odors, the existing aerobic digester will be retained to provide partial stabilization of the wastewater solids. The digester will be converted from mechanical mixing/aeration to coarse bubble aeration.

7.1.9 General WWTP Facilities Improvements

Improvements to general WWTP facilities include those associated with the Control Building and electrical infrastructure.

7.1.9.1 Control Building

The existing Control Building was last remodeled in 1976. The original construction date is not known. Because of its age, limited space, general condition, and changes in building codes that could significantly increase the costs of making changes to the existing building, constructing a new control building is recommended. New facilities will include office space, breakroom, locker facilities, and improved laboratory. Construction of a 3,800-square foot (sf) Control Building with the recommended facilities has been assumed for planning and budgeting purposes.

7.1.9.2 Electrical Infrastructure

Electrical infrastructure improvements needed to address deficiencies with aging equipment and to support implementation of the recommended WWTP improvements include construction of a new Electrical Building and new standby electrical generator. The new Electrical Building should be sized to allow consolidation of electrical equipment into a more central location and include facilities needed for all recommended improvements to the WWTP. Construction of a 1,000-sf building would accommodate all needed electrical facilities and is recommended. The building could either be constructed as a stand-alone structure or as part of the recommended Control Building which should reduce overall costs.

The existing 1,250-kilowatt (kW) standby engine generator, while in relatively good condition, does not have sufficient capacity to support the addition of all recommended WWTP improvements which will require total standby power capacity of 1,750-kW. A new standby engine generator will ultimately be needed and should be provided with a sound attenuating outdoor rated enclosure to eliminate the need for construction of additional building space to house the generator. Timing for installation of the new standby generator will depend on the potential phasing of recommended improvements as is further described below.

7.1.10 I/I Removal and Rehabilitation

As described in Section 3 of this Plan, the City has completed two phases of a five phase I&I removal program. Phase 3 will eliminate stormwater connections to the sanitary sewer in three downtown areas and will complete smoke testing of the last 20% of the collection system. Phase 4 will be the final downtown sewer separation project and will eliminate stormwater connections in four areas. The Phase 3 and 4 projects are included in the City's 2022-2027 Capital Improvements Program (CIP) with $0.9 million budgeted for Phase 3 and $0.8 million budgeted for Phase 4.

The City should complete the planned I&I reduction projects in the downtown area and evaluate impact on peak flows to the WWTP prior to starting the Phase 3 final liquid stream treatment improvements projects described below. Results of smoke testing should be used by the City to identify additional sources of I&I that may be addressed by future collection system improvement projects.

7.2 Project Phasing

Because of the significant cost and potential disruption to WWTP operations, recommended improvements should be phased and constructed over time. Priority should be given to those projects that would address the most acute deficiencies within the WWTP. Improvements to the WWTP have thus been divided into three project phases.

Phase 1 will address significant deficiencies within the solids treatment process that include:

- Lack of Class B land application sites. Of the City's five authorized land application sites, only two are currently available for use.

- Insufficient solids storage. The WWTP has the equivalent of 45 days of solids storage. This is about 25% of that needed to accommodate the 6-month period each year during which Class B biosolids cannot be land applied. This necessitates landfilling excess biosolids.

- Increasing landfill costs. Landfill disposal costs have increased dramatically, almost doubling within the last few years. In addition to increasing costs, the Coffin Butte Landfill may stop accepting wastewater solids at its facilities.

Because of these deficiencies and the increase in community resistance to Class B land application programs, constructing facilities capable of producing Class A biosolids is recommended as soon as practical.

Phase 2 will address capacity deficiencies within the secondary treatment process and will include new headworks, oxidation ditch, and secondary clarifier. Phase 3 will address remaining deficiencies within the existing liquid stream treatment stream. It will include improvements to the influent and effluent pumping systems, UV disinfection, tertiary filtration, and a new standby generator.

7.2.1 Phase 1 Solids Treatment Improvements Project

The Phase 1 project will provide improvements to the WWTP's solids treatment process and will include the following:

- Solids Thickening. A new rotary drum thickener will be installed to replace the WWTP's existing GBT that was originally placed into service in 1992. New polymer feed system, sludge feed pumping, thickened sludge pumping, and HVAC improvements will also be included.
- Class A Biosolids Facilities. Class A biosolids treatment facilities will include construction of a belt-type thermal dryer including new pre-engineered metal building, odor control, conveyance, and bagging system. The existing digester will continue to be used to provide partial stabilization of wastewater solids prior to drying. New blowers and a coarse bubble aeration system will be installed to replace the digester's existing mechanical mixing/aeration system.
- Control Building. A new control building will be constructed to replace the WWTP's failing building. The new building will include laboratory facilities.
- Electrical Building: A new electrical building will be constructed that will ultimately consolidate and house all electrical equipment for WWTP.

A proposed layout for Phase 1 solids stream treatment improvements is presented on Figure 7-1.

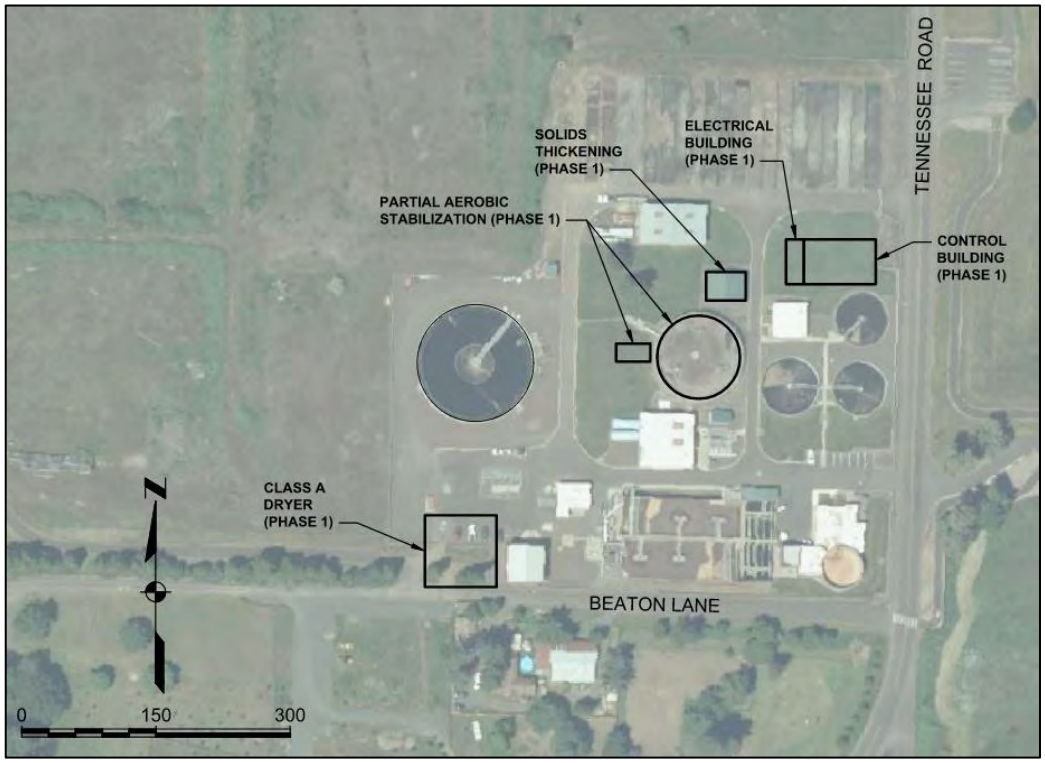


Figure 7-1: Phase 1 Solids Stream Treatment Improvements Project

Construction of a new control building is also included in the Phase 1 project. This recommendation is based on the age and poor condition of the existing building which predates plant improvements that were completed in 1976. Because of the age of existing electrical facilities and extent of new treatment facilities needed, construction of a new electrical building is recommended that will be sized to ultimately house the electrical equipment for all WWTP facilities.

The WWTP's existing 1,250-kW engine generator has sufficient capacity to provide needed standby power for the addition of Phase 1 improvements. A new larger capacity generator will not be needed until construction of Phase 3 improvements as described below.

7.2.2 Phase 2 Liquid Treatment Improvements Project

The Phase 2 improvements project will include the following:

- Oxidation Ditch. A new 2.3 million gallons (MGal) oxidation ditch will be constructed to replace the WWTP's existing activated sludge process.
- Secondary Clarifier. A new 130-ft secondary clarifier with RAS, WAS, and scum pumping will be constructed to replace the WWTP's three, smaller, aging clarifiers.
- Headworks. A new headworks will be constructed to provide needed improvements to the preliminary treatment process. It will include new mechanical screens and grit removal.

A proposed layout for Phase 2 liquid stream treatment improvements is presented on Figure 7-2.

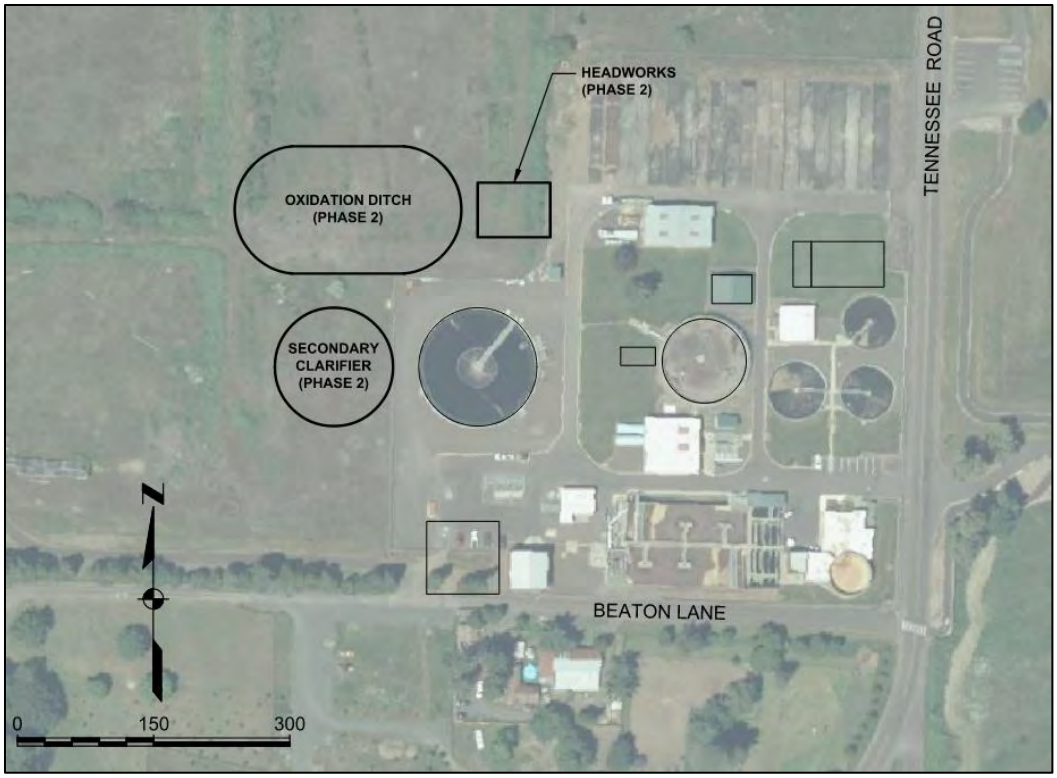


Figure 7-2: Phase 2 Liquid Stream Treatment Improvements Project

Elements of the Phase 2 project were selected to address the WWTP's most significant liquid stream treatment deficiencies. Although City staff have, with limited exceptions, met effluent discharge requirements, the existing secondary treatment process does not have sufficient capacity. The WWTP's three, smaller secondary clarifiers are aging, do not perform well, and are unreliable. These deficiencies dictate that improvements to these process be prioritized over other liquid stream processes.

Because the existing headworks structure is an integral part of the existing secondary treatment process and is in poor condition, construction of a new headworks in conjunction with the new secondary treatment process is recommended.

As described in Section 6, prior to starting the Phase 2 liquid stream treatment improvements project, the City should complete the Phase 3 and 4 I&I reduction projects. Influent flows should be revaluated once they have been completed to determine the impact on peak flows to the WWTP that would affect the sizing of the proposed liquid stream treatment process improvements.

7.2.3 Phase 3 Final Liquid Stream Improvements Project

The final phase of improvements projects will address deficiencies within the WWTP's liquid stream treatment stream not corrected in the Phase 2 project. While necessary, they are not as potentially critical as the Phase 2 improvements. They will include the following:

- Influent Pumping. Capacity of the WIPS will be increased to meet projected design year 2045 peak flow conditions. Improvements will include installation of four new larger capacity pumps and a single smaller capacity pump that will be used during low flow periods.

Effluent Pumping. Capacity of the Effluent Pump Station will be increased to meet projected design year 2045 peak flow conditions.

- Disinfection. A UV disinfection system will be constructed to replace the WWTP's existing chlorine disinfection system that cannot provide the DEQ required detention times under high flow conditions.

- Tertiary Filtration. A synthetic/cloth media filtration will be constructed to help ensure that the WWTP can meet effluent discharge limits that under certain flow conditions would require effluent BOD and TSS concentrations to be less than 10 mg/L.

- Standby Engine Generator. An outdoor rated, 1,750-kW diesel powered generator will be provided to replace the WWTP's existing 1,250-kW.

A proposed layout for Phase 3 final liquid stream treatment improvements is presented on Figure 7-3.

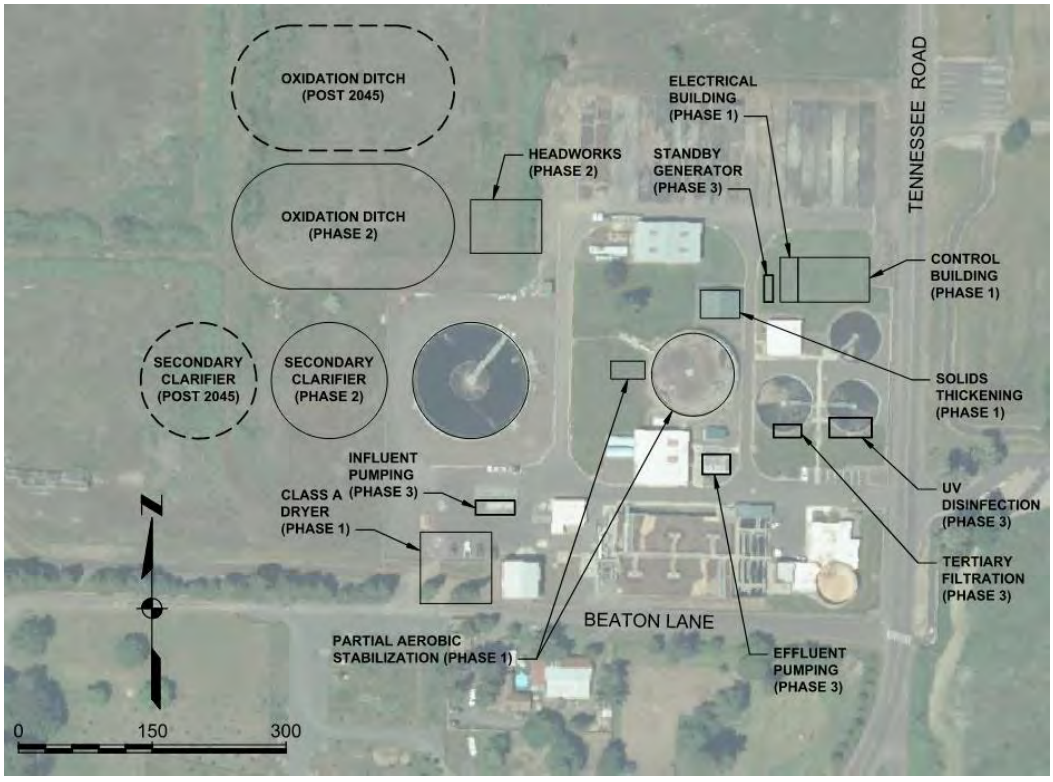


Figure 7-3: Phase 3 Final Liquid Stream Treatment Improvements Project

Although the influent and effluent pumping systems do not have sufficient firm capacity to meet projected peak hour flows, improvements to increase their capacity could potentially be delayed.

Information provided by the City shows that prior to 2021, influent flows to the WWTP were actively stored within the upstream collection system thereby reducing peak flows. Although doing so is believed to have resulted in an excessive accumulation of solids within the collection system which caused operational challenges at the headworks and potential increased degradation of the West Side Interceptor, the City could elect to restart this practice to temporarily delay the costs associated with increasing pumping capacity at the WWTP. If the City chooses to delay improvements to the pumping stations, operation of both the influent and effluent pump stations should be closely monitored. Should flows approach the firm capacity of the stations, the City should immediately take steps to implement needed improvements.

The existing CCBs do not provide the DEQ recommended minimum detention times under projected design year 2045 peak flow conditions. For example, the CCBs would provide 18 minutes of detention time during peak day flow and 11 minutes during peak instantaneous flow compared with the DEQ recommended values of 20 minutes and 15 minutes, respectively. However, the CCBs would provide more than two times the minimum recommended detention time under average flow conditions. Because of this and that the CCBs are very close to meeting the DEQ minimum detention time under peak day conditions, the City could consider delaying improvements to the disinfection system. While the City has successfully met disinfection requirements, performance should be closely monitored to ensure the ability to remain in compliance and phasing of the recommended improvements to the disinfection system re-evaluated, if needed.

Tertiary filtration is recommended to help ensure that the City meets effluent discharge requirements for BOD and TSS that under certain flow conditions could be below 10 mg/L. Conventional secondary treatment systems like the oxidation ditch recommended to replace the WWTP's existing secondary treatment process can produce effluent that meet these requirements. However, standard design practice dictates the use of tertiary filtration to ensure that they are in fact met and is thus recommended. Like disinfection though, the City could consider delaying the addition of tertiary filtration until after construction of the new secondary treatment improvements. Performance of the WWTP should be evaluated and timing for construction of a new tertiary filtration system could then be re-evaluated, if needed.

The WWTP's existing 1,250-kW standby engine generator has sufficient capacity to meet standby electrical power requirements through completion of Phase 2 improvements. However, the addition of the Phase 3 project will exceed the capacity of the existing generator. A new 1,750-kW generator will be needed and is recommended. The generator will be provided with an outdoor rated, sound attenuating enclosure to eliminate the need for additional indoor building space. Additional electrical equipment associated with the new generator will include a new service entrance, switchboard, and automatic transfer switch (ATS).

As discussed in Section 1 of this Plan, the projected annual population growth rate from 2045 to 2070 slows to 0.2%. This represents only a 5% increase in the total population between the 20-year planning horizon of this planning document (2045) and 2070. Although it is important to consider future wastewater facility needs, specific requirements to meet ultimate buildout conditions were not evaluated. Instead, space should be reserved within the WWTP to allow for some level of expansion. As shown on Figure 7-3, this was done by reserving space for construction of a second oxidation ditch and a third 130-ft secondary clarifier. Doing this will provide sufficient space for future expansion even if future growth is significantly greater than that currently projected.

7.3 Estimated Project Costs and Implementation Schedule

The engineer's opinion of probable construction cost (OPCC) for the recommended Phase 1 solids stream improvements project is presented in Table 7-2. OPCCs presented within this section are Level 4 as defined by the Association for the Advancement of Cost Estimating (AACE).

Table 7-2: OPCC for Phase 1 Solids Stream Improvements[(a)]

Item Description	Total[(b)]
Construction Costs	
Solids Thickening	$1,120,000
Class A Biosolids Facilities	$17,280,000
Control Building	$2,000,000
Electrical Building	$1,630,000
Subtotal	**$22,030,000**
Contingency[(c)]	$6,620,000
Escalation[(d)]	$5,950,000
Total Construction	**$34,600,000**
Engineering and Admin.[(e)]	$8,650,000
Total Project Cost	**$43,250,000**

Notes:

(a) AACE Class 4 planning level costs.
(b) Values rounded to the nearest $10,000.
(c) Contingency equal to 30%.
(d) Annual escalation rate of 6.5%.
(e) Engineering, legal, and administration equal to 25%.

An appropriate accuracy range for a Level 4 OPCC is -30 percent to +50 percent. Applying this to the Phase 1 improvements project estimate results in the potential cost range shown in Table 7-3.

Table 7-3: OPCC Range for Phase 1 Solids Stream Improvements Project

Low (-30 percent)	Total Estimate	High (+50 percent)
$30.3 million	$43.2 million	$64.9 million

An implementation schedule for the Phase 1 solids stream improvements project is presented on Figure 7-4. Project design is anticipated to begin in January 2026 with construction beginning in the spring of 2027. Final project closeout is projected for October 2028.

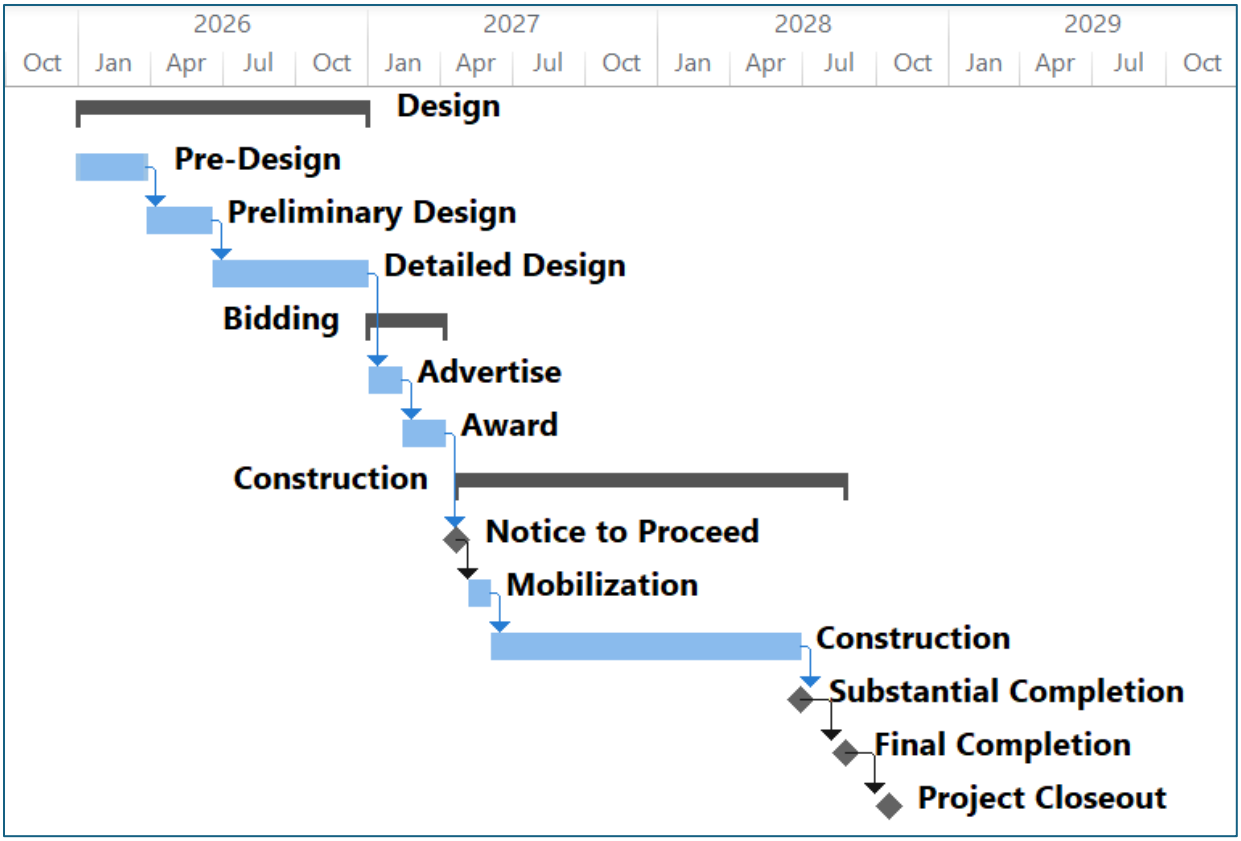


Figure 7-4: Phase 1 Solids Stream Improvements Project Schedule

The engineer's OPCC for the recommended Phase 2 liquid stream improvements project is presented in Table 7-4.

Table 7-4: OPCC for Phase 2 Liquid Stream Improvements[a]

Item Description	Total[b]
Construction Costs	
Headworks	$2,070,000
Oxidation Ditch	$8,640,000
Secondary Clarifier	$6,070,000
Subtotal	**$16,780,000**
Contingency[c]	$5,030,000
Escalation[d]	$4,530,000
Total Construction	**$26,340,000**
Engineering and Admin.[e]	$6,580,000
Total Project Cost	**$32,920,000**

Notes:

(a) AACE Class 4 planning level costs.
(b) Values rounded to the nearest $10,000.
(c) Contingency equal to 30%.
(d) Annual escalation rate of 6.5%.
(e) Engineering, legal, and administration equal to 25%.

The potential cost range for the Phase 2 liquid stream improvements project is presented in Table 7-5.

Table 7-5: OPCC Range for Phase 2 Liquid Stream Improvements Project

Low (-30 percent)	Total Estimate	High (+50 percent)
$23.0 million	$32.9 million	$49.4 million

An implementation schedule for the Phase 2 liquid stream improvements project is presented on Figure 7-5. Project design is anticipated to begin in September 2027. Doing so should result in construction beginning in January 2029 soon after completion of the Phase 1 solid stream improvements project. The Phase 2 liquid stream improvements project is projected to be complete in September 2030.

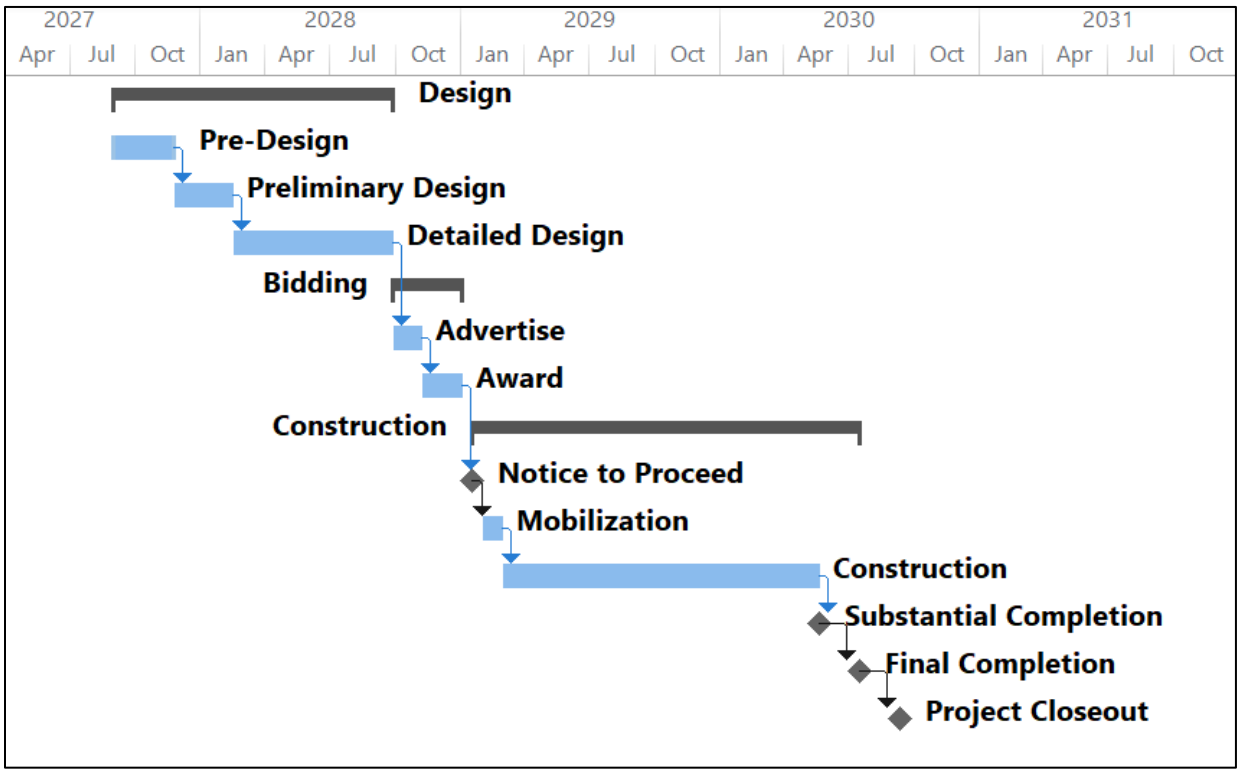


Figure 7-5: Phase 2 Liquid Stream Improvements Project Schedule

The engineer's OPCC for the recommended Phase 3 final liquid stream improvements project is presented in Table 7-6.

Table 7-6: OPCC for Phase 3 Final Liquid Stream Improvements[a]

Item Description	Total[b]
Construction Costs	
Influent Pumping	$2,410,000
Effluent Pumping	$880,000
UV Disinfection	$2,080,000
Tertiary Filtration	$1,740,000
Generator and Assoc. Equip.[c]	$2,760,000
Subtotal	**$9,870,000**
Contingency[d]	$2,960,000
Escalation[e]	$2,670,000
Total Construction	**$15,500,000**
Engineering and Admin.[f]	$3,880,000
Total Project Cost	**$19,380,000**

Notes:

(a) AACE Class 4 planning level costs.
(b) Values rounded to the nearest $10,000.
(c) Cost includes new standby engine generator, service entrance, switchboard, ATS and MCC's for Phase 3 improvements.
(d) Contingency equal to 30%.
(e) Annual escalation rate of 6.5 percent.
(f) Engineering, legal, and administration equal to 25 percent.

The potential cost range for the Phase 3 final liquid stream improvements project is presented in Table 7-7.

Table 7-7: OPCC Range for Phase 3 Final Liquid Stream Improvements Project

Low (-30 percent)	Total Estimate	High (+50 percent)
$13.6 million	$19.4 million	$29.1 million

An implementation schedule for the Phase 3 final liquid stream improvements project is presented on Figure 7-6. Project design is scheduled to begin in the summer of 2029. This should result in construction beginning in the fall of 2030 soon after completion of the Phase 2 liquid stream improvements project. The Phase 3 final liquid stream improvements project is anticipated to be complete in the spring of 2032.

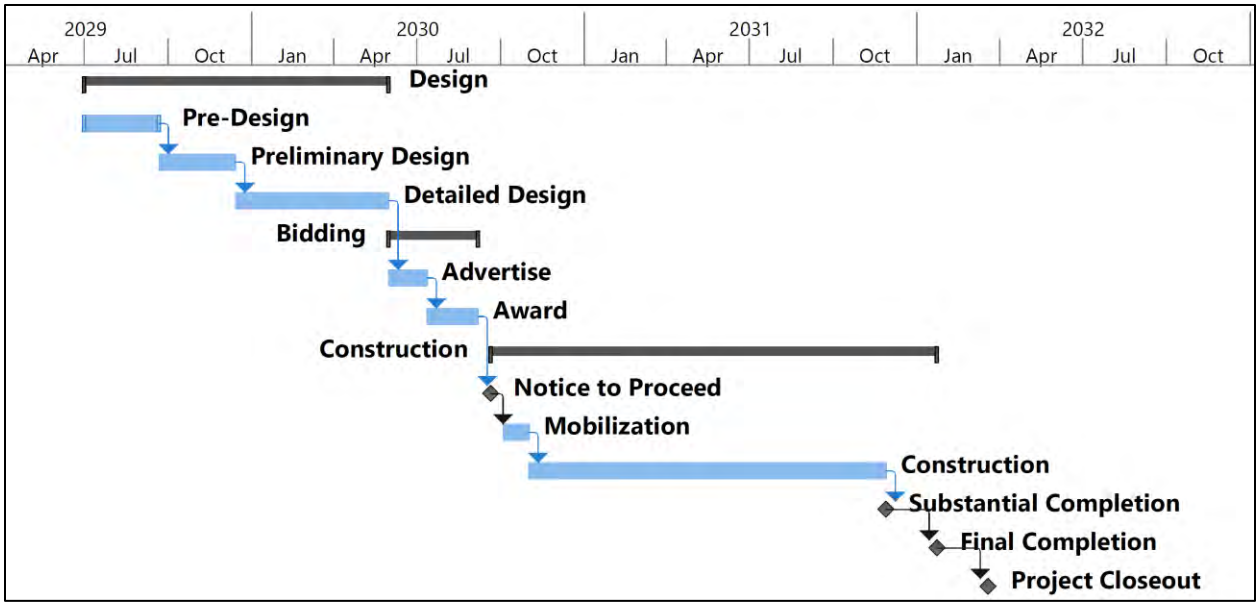


Figure 7-6: Phase 3 Final Liquid Stream Improvements Project Schedule

Section 8: Additional Recommendations

This section provides additional recommendations for the City as it moves forward with the proposed improvements to its wastewater treatment facilities. Items that should be completed include a rate study to identify needed changes to user rates, a value engineering study to identify potential cost savings, and environmental documentation for agency review of the project.

8.1 Rate Study

A rate study is recommended to provide the City with a thorough understanding of the costs of current wastewater services and need for changes in user rates or alternative funding sources to support the recommended improvements to the wastewater treatment plant (WWTP) facilities. The rate study should include the following elements as described below.

8.1.1 Data Collection

Data collection should gather information from various sources, including financial statements (e.g., income statement and balance sheet), budgets, asset schedules or inventories, and customer billing data. A review meeting should be conducted to review the available data, clarify unfulfilled data needs, identify key contact persons, and assign responsibilities. Data collection should also include identifying the steps required to obtain the customer statistics for use in the cost-of-service analysis.

8.1.2 Financial Model

The next step in a rate study should be development of a financial model to permit the evaluation of alternative rate policies and strategies. Tasks that should be included in the model developed are as follows:

- Review historical costs and revenues and forecast revenue under existing rates. This will provide information on the capacity of existing rates to support revenue requirements for recommended improvements. Determine overall revenue shortfalls by year for the forecast horizon which should be consistent with other planning timeframes utilized by the City. Identify annual increases needed to meet annual revenue requirements.

- Forecast operation and maintenance (O&M) and capital costs associated with the recommended improvement projects and insert them into the financial plan model.

- Evaluate capital funding alternatives, including bonding strategies. This task should include evaluating fiscal policies related to capital financing, including preferences for debt or equity funding. Also, analyses should be done to evaluate reserve levels for debt or equity funding, reserve levels for contingencies, and replacement of funding (i.e., explicit funding of depreciation). Finally, the feasibility of developing alternative sources of funds, including grants, special low interest loans, special fees, and system development charges (SDCs) should also be integrated into the analysis.

- Develop rate increase strategies considering the effects on utilities financial performance and condition.

8.1.3 Financial Analysis

A financial (cost-of-service) analysis generally addresses the basis for recovering revenues from customers according to the demands which they place on the utility. The cost-of-service analysis may result in different percentage rate increases among the customer classes, which may be more equitable based on the supporting analysis. Tasks that should be included in the analysis are as follows:

- Develop functional allocation of costs that would include customers, flow, strength of domestic discharge (i.e., biochemical oxygen demand [BOD] and total suspended solids [TSS]), and extra strength loading.
- Develop customer statistics the structure of which would depend on the types of rate structures the City considers.
- Distribute costs to customer classes by developing class revenue requirements based on usage/demand characteristics.

8.1.4 Rate Analysis

The rate analysis compares existing rates with alternative rate designs and compares results with the allocated costs by customer class. Alternative rate designs may include seasonal rates, block rates, and varying customer classifications. Due to the nature of restructuring rates, impacts may not be limited to customer classes nor necessarily uniform within them. The rate model should document impacts on representative customers, as well as customer classes, to provide the City information on the range of rate impacts which could result.

Specific tasks for the rate analysis should include the following:

- Review rate objectives and various alternatives for attaining those objectives and identify rate structures that should be developed.
- Develop rate model, incorporate customer statistics, and integrate revenue requirements and cost-of-service models to automate the rate development process.
- Develop and evaluate rate alternatives based on equity, sufficiency, and impacts.
- Review preliminary rate findings with City staff. Refine analyses based on that review and, if appropriate, develop phased strategies for restructuring rates.

8.2 Value Engineering

Value engineering (VE) is a cost control technique that utilizes independent experts to provide a separate review of the proposed project. The goal of VE is to identify cost savings while also looking for opportunities to increase reliability and performance. For projects that receive more than $10 million in funding from the Department of Environmental Quality (DEQ) Clean Water State Revolving Fund (CWSRF), Oregon Administrative Rule 340-054-0022 (5)(g) requires a value engineering study that “must be a specialized cost control technique specifically applicable to the wastewater treatment facility design identifying cost savings that can be made without sacrificing project reliability or efficiency.” Regardless of the source of project funding,

however, the City should consider conducting a VE study during the pre-design (10%) stage of the project to help ensure the best value is achieved when undertaking the significant improvements recommended in this Plan.

All VE studies should begin with data collection. This allows the VE team to familiarize themselves with the proposed project, its drivers, regulatory requirements, alternatives considered, and estimated costs. A VE workshop should then be conducted that typically includes the following five phases (EPA, 1984):

- Information gathering where the VE team gains a better understanding of project background, constraints, and costs. High-cost areas of the project are identified.
- Alternative speculation where the VE team identifies potential alternatives that would provide the same functionality as the systems in the recommended improvements project.
- Screen and evaluate alternatives identified in the speculation phase to select those with the greatest potential for cost savings, increased reliability, or improved performance.
- The VE team researches the selected alternatives, generates costs, and prepares descriptions of recommended alternatives.
- Recommendations of the VE workshop are presented to the City and their design consultant and a written VE report is generated.

After the VE workshop has been completed, the City and their design consultant should meet and review the VE recommendations. Additional evaluation is typically required to further vet the VE recommendations prior to their acceptance and inclusion in the recommended project.

8.3 Environmental Review

Depending on the funding source utilized by the City to support implementation of recommended WWTP improvements, some level of environmental review will likely be required to help ensure that environmental impacts are considered during project planning. For example, the State Environmental Review Process (SERP) must be followed for projects funded through the DEQ's CWSRF (Oregon DEQ, 2018).

SERP is comprised of three tracks which include the following:

- Categorical Exclusion (CE) is determined when a project or its construction will have no negative impact on the environment. Projects such as those associated with minor rehabilitation of existing facilities, reducing I&I, modifications to solids treatment processes with no liquid stream treatment expansion, or minor expansions to existing wastewater facilities serving fewer than 10,000 people may receive a CE determination from the DEQ.
- Environmental Assessments (EA) are required for projects that have the potential for negative environmental impacts. These include projects that are not eligible for a CE. Examples of such projects include, but are not limited to, new wastewater collection facilities into previously unserved areas, improvements to liquid stream treatment

processes to serve a population that exceeds the current population by more than 30%, construction of facilities expected to impact threatened or endangered species, or projects that may cause adverse impacts to air quality. After reviewing the EA (including a 30-day public comment period), the DEQ may issue a Finding of No Significant Impact (FONSI) if the project (with required mitigation measures) will not significantly impact the environment.

- Environmental Impact Statements (EIS) are required for projects which may have significant environmental impacts that will not be reduced or eliminated by mitigation steps or changes to the project. When preparing an EIS, the City must first publish a notice of intent to prepare an EIS and allow for a 30-day comment period. The City must then prepare the draft EIS conforming to the requirements found in 40 Code of Federal Regulations (CFR) Part 1502 – Environmental Impact Statement. When approved by the DEQ, the City must submit the EIS to state and federal agencies having jurisdiction, publish a notice of EIS availability with 45-day public comment period, and conduct a public hearing. The City must then prepare and submit a final EIS (including all substantive comments received with explanations of how the comments were addressed) to the DEQ. The DEQ provides copies of the final EIS to all agencies or entities who commented on the draft EIS and after a 30-day period issues a Record of Decision (ROD).

When preparing environmental documentation for review by the DEQ, recipients of CWSRF funding must consult with agencies that have regulatory authority over applicable federal environmental laws and executive orders. These agencies often referred to as “cross-cutting” agencies include those responsible to ensure compliance with the following:

- Historic/Cultural resources including the National Historic Protection Act of 1966 and the Archeological and Historic Preservation Act of 1974.
- Protection of wetlands through Executive Order 11990 signed in 1977.
- Flood plain management through Executive Order 11988 signed in 1977 and Executive Order 12148 signed in 1979.
- Farmland Protection Policy Act of 1981.
- Coastal Zone Management Act of 1972.
- Wild and Scenic Rivers Act of 1968.
- Endangered Species Act of 1973 and the Essential Fish Habitat provision added to the Magnuson-Stevens Fishery Conservation and Management Act of 1976.
- Clean Air Act of 1977.
- Safe Drinking Water Act of 1974.

Federal funding such as that administered by the Environmental Protection Agency (EPA) may require preparation of environmental documentation to comply with requirements of the National Environmental Policy Act (NEPA). Like SERP, the NEPA process includes three tracks or levels of analysis. They are as follows:

- Categorical Exclusion (CATEX) where the project may be "categorically excluded" from detailed environmental review when it does not have a significant effect on the environment. Conditions for which a CATEX is appropriate are defined by each federal agency.

- Environmental Assessment (EA) is applicable when the federal agency determines that a CATEX is not applicable to the proposed project. The EA determines if there is potential to cause significant environmental impact. If there is no potential for significant impact, then the federal agency (like the DEQ and the SERP) will issue a FONSI.

- Environmental Impact Statements (EIS) are required if the federal agency is unable to issue a FONSI. As with the SERP, the EIS required by NEPA includes a notice of intent to conduct the environmental analysis, issuance of a draft EIS for a 45-day public comment period after which the federal agency considers substantive comments, publishing the final EIS, and finally issuing of the ROD. The ROD includes an explanation of the agency's decision, presents alternatives considered, and describes mitigation and monitoring requirements, if any.

References

City of Lebanon*, City of Lebanon Comprehensive Plan*, 2004.

City of Lebanon. "Fact Sheet – Lebanon Oregon." 2012.

Google Maps, *Lebanon, Oregon,* 2022. Google Maps.

Lebanon (2004). *City of Lebanon Comprehensive Plan*, 2004.

Linn County Oregon, *Linn County Land Use Planning GIS Application*, 2022. https://gis.co.linn.or.us

Oregon Biodiversity Information Center at Portland State University. *Rare, Threatened, and Endangered Species of Oregon,* July 2019. Rare, Threatened, and Endangered Species of Oregon.

Oregon DEQ (2006), Willamette Basin Total Maximum Daily Load (TMDL), 2006

Oregon DEQ (2008). *City of Lebanon Mixing Zone Study Memorandum*, February 4, 2008.

Oregon DEQ (2018) *Oregon Clean Water State Revolving Fund*, *Applicant Guide to the State Environmental Review Process*, December 30, 2018.

Oregon DEQ (2019), *Final Revised Willamette Basin Mercury Total Maximum Daily Load*, November 22, 2019

Oregon DEQ (2019). Preparing Wastewater Planning Documents and Environmental Reports for Public Utilities Financed by Business Oregon, Oregon Department of Environmental Quality, Rural Community Assistance Corporation, United States Department of Agriculture, 2019.

Oregon DEQ (2019). Guidelines for Making Wet Weather and Peak Flow Projections for Sewage Treatment in Western Oregon: MMDWF, MMWWF, PDAF and PIF. Rev IV. 96.

Oregon DEQ (2021). *Procedure for Determining Reasonable Potential for Copper Using the Biotic Ligand Model*, Rev 1.0, 2021

Oregon DEQ (2022). *2022 Integrated Report*

Oregon DEQ (2022), *Methodology for Oregon's 2022 Water Quality Report and List of Water Quality Limited Waters,* May 2022

Oregon DEQ (2023). *Reasonable Potential Analysis Process for Toxic Pollutants*, December 13, 2023

Oregon DEQ (2024). *Draft Total Maximum Daily Loads for the Willamette Subbasins (Willamette Mainstem and Major Tributaries) Temperature*, August 2024

Oregon Department of Geology and Mineral Industries, Geologic Map of Oregon, 2002. Geologic Map of Oregon

Oregon Health Authority (2009). *Lebanon Groundwater,* 2009. Lebanon Groundwater.

PSU Population Research Center (2021), *Coordinated Population Forecast, 2021 through 2071, Linn County, Urban Growth Boundaries (UGB) & Area Outside UGBs*, June 30, 2021.

South Santiam Watershed Council. “About the South Santiam Watershed.” 2022.

U.S. Climate Data (2022). “Climate Lebanon – Oregon” 2022. Climate Lebanon – Oregon.

United States Department of Agriculture Natural Resources Conservation Service. *Web Soil Survey,* July 2019. Web Soil Survey

US EPA (2003). *Technical Background Document for the Sewage Sludge Exposure and Hazard Screening Assessment, Document No. 822-B-03-001*, December 19, 2003,

West Yost & Associates, *City of Lebanon Facilities Plan for the Wastewater Treatment Plant*, October 2004.

Appendix A: NPDES PERMIT

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM
WASTE DISCHARGE PERMIT

Department of Environmental Quality
Western Region - Salem Office
750 Front St. NE, Suite 120, Salem, OR 97301-1039
Telephone: (503) 378-8240

Issued pursuant to ORS 468B.050 and The Federal Clean Water Act

ISSUED TO:

City of Lebanon
925 Main Street
Lebanon, OR 97355

SOURCES COVERED BY THIS PERMIT:

Type of Waste	Outfall Number	Outfall Location
Treated Wastewater	001	R.M. 17.4 S. Santiam River
Muncipal Wastewater Emergency Overflows:		
Harrison St. Pump Station to Burkhart Creek via Railroad Ditch	002	R.M. 10 Burkhart Creek
16th St. Sanitary Sewer to Burkhart Creek via storm drain	003	R.M. 12 Burkhart Creek
15th St. Sanitary Sewer to Burkhart Creek via storm drain	004	R.M. 12 Burkhart Creek

FACILITY TYPE AND LOCATION:

Activated sludge sewage treatment plant
33110 Tennessee Road
Lebanon, OR 97355
Treatment System Class: III
Collection System Class: III

RECEIVING STREAM INFORMATION:

Basin: Willamette
Sub-Basin: South Santiam
Receiving Stream: South Santiam
Hydro Code: 22F-SAS 17.4D
County: Linn

EPA REFERENCE NO: OR - 002081 - 8

Issued in response to Application No. 993744 received February 28, 1996.

This permit is issued based on the land use findings in the permit record.

Gary Messer, Water Quality Manager
Western Region

FEB 11 2000
Date

PERMITTED ACTIVITIES

Until this permit expires or is modified or revoked, the permittee is authorized to construct, install, modify, or operate a wastewater collection, treatment, control and disposal system and discharge to public waters adequately treated wastewaters only from the authorized discharge point or points established in Schedule A and only in conformance with all the requirements, limitations, and conditions set forth in the attached schedules as follows:

	Page
Schedule A - Waste Discharge Limitations not to be Exceeded	2-3
Schedule B - Minimum Monitoring and Reporting Requirements	3-6
Schedule C - Compliance Conditions and Schedules	6-7
Schedule D - Special Conditions	7-10
Schedule E - Pretreatment	N/A
Schedule F - General Conditions	10-17

Unless authorized by another NPDES permit, each other direct and indirect discharge to public waters is prohibited.

SCHEDULE A

1. Waste Discharge Limitations not to be exceeded after permit issuance.

a. Outfall Number 001 -Wastewater Treatment Plant Discharge

(1) May 1 - October 31:

Parameter	Average Effluent Concentrations		Monthly Average lb/day	Weekly Average lb/day	Daily Maximum lbs
	Monthly	Weekly			
$CBOD_5$	10 mg/l	15 mg/l	250	380	500
TSS	10 mg/l	15 mg/l	250	380	500

(2) November 1 - April 30:

Parameter	Average Effluent Concentrations		Monthly Average lb/day	Weekly Average lb/day	Daily Maximum lbs
	Monthly	Weekly			
$CBOD_5$	25 mg/l	40 mg/l	1,500	2,200	3,000
TSS	30 mg/l	45 mg/l	1,800	2,700	3,600

As provided in OAR 340-041-120(9)(C), the daily mass load shall not apply on any day that the total flow exceeds 6 mgd, which is twice the design average dry weather flow.

(3)

Other Parameters (year-round)	Limitations
E. coli Bacteria	Shall not exceed 126 organisms per 100 ml monthly geometric mean. No single sample shall exceed 406 organisms per 100 ml. (See Note 1/)
pH	Shall be within the range of 6.0 - 9.0
Ammonia - N	Shall not exceed a monthly average concentration of 0.94 mg/l and a daily maximum concentration of 1.89 mg/l.
$CBOD_5$ and TSS Removal Efficiency	Shall not be less than 85% monthly average except when monthly average flows exceed 3.0 MGD which shall not be less than 75% monthly average.
Total Chlorine Residual	Shall not exceed a monthly average concentration of 0.01 mg/l and a daily maximum concentration of 0.02 mg/l.

(4) Except as provided for in OAR 340-045-0080, no wastes shall be discharged and no activities shall be conducted which violate Water Quality Standards as adopted in OAR 340-041-0445 except in the following defined mixing zone:

> That portion of the South Santiam River beginning 10 feet upstream and continuing 100 feet downstream from the point of discharge. The Zone of Immediate Dilution (ZID) shall be defined as that portion of the allowable mixing zone that is within ten (10) feet of the point of discharge

b. Outfall Number 002 - 004 -Emergency Overflow Outfalls

No wastes shall be discharged from these outfalls and no activities shall be conducted which violate Water Quality Standards as adopted in OAR 340-041-0445, unless the cause of the discharge is the result of an exceptional event beyond the reasonable control of the permittee. Overflows caused by

operational error, improperly designed facilities, or lack of preventative maintenance are not beyond the reasonable control of the permittee. These emergency overflows described above will only be allowed until the planned upgrading of the West Side Interceptor is completed. The timing will be addressed in the City's Inflow & Infiltration Removal Plan submitted to the Department for review and approval. All permitted overflows must be from a "controlled" point and, at a minimum, must have an alarm. Whenever possible, flow monitoring must also be provided.

NOTES:

1/ If a single sample exceeds 406 organisms per 100 ml, then five consecutive re-samples may be taken at four hour intervals beginning within 28 hours after the original sample was taken. If the log mean of the five re-samples is less than or equal to 126 organisms per 100 ml, a violation shall not be triggered.

SCHEDULE B

1. **Minimum Monitoring and Reporting Requirements to be met after permit issuance** (unless otherwise approved in writing by the Department).

The permittee shall monitor the parameters as specified below at the locations indicated. The laboratory used by the permittee to analyze samples shall have a quality assurance/quality control (QA/QC) program to verify the accuracy of sample analysis. If QA/QC requirements are not met for any analysis, the results shall be included in the report, but not used in calculations required by this permit. When possible, the permittee shall re-sample in a timely manner for parameters failing the QA/QC requirements, analyze the samples, and report the results.

a. Influent

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD)	Daily	Continuous
Flow Meter Calibration	2/Year	Verification
$CBOD_5$	2/Week	Composite
TSS	2/Week	Composite
PH	3/Week	Grab

b. Outfall Number 001 - Sewage Treatment Plant Discharge

Item or Parameter	Minimum Frequency	Type of Sample
$CBOD_5$	2/Week	Composite**
TSS	2/Week	Composite**
pH	3/week	Grab*
Temperature	2/Week	Record*
E. coli	2/Week	Grab* (See Note 1/)
Quantity Chlorine Used	Daily	Measurement
Chlorine Residual	Daily	Grab*
Average Percent Removed ($CBOD_5$ and TSS)	Monthly	Calculation
Pounds Discharged ($CBOD_5$ and TSS)	2/Week	Calculation
Ammonia - N	2/Week	Composite**
Toxic - Bioassay	2/Year	Acute & Chronic (See Note 3/)

The facility effluent sampling locations are the following:

* Grab samples or measurements are taken at the outlet of the chlorine contact channels or at the end of the Ultraviolet (UV) channel when the UV system is online.

** Composite samples are taken from the outlet of the chlorine contact channels or at the end of the UV channel when the UV system is online. The sampler is located in the chlorinator room of the control building.

c. Biosolids Management

The biosolids management analysis results shall be included in the annual report described in part 3.b. of this schedule. Submittal of the biosolids management analysis results with the monthly Discharge Monitoring Report is not required.

Item or Parameter	Minimum Frequency	Type of Sample
Sludge analysis including: Total solids (% dry wt.) Volatile solids (% dry wt.) Biosolids nitrogen for: NH_3 –N; NO_3-N; & TKN (% dry wt.) Sludge metals content for: As, Cd, Cu, Hg, Mo, Ni, Pb, Se & Zn, measured as total in mg/kg Phosphorus (% dry wt.) Potassium (% dry wt.) pH (standard units)	Annual	Composite sample to be representative of the product to be land applied from the aerobic digester (See Note 4/) or from the sludge drying beds (See Note 5/)
Record of % volatile solids reduction accomplished through digestion	Monthly	Calculation (See Note 6/)
Fecal coliform (per gram total solids - dry weight basis)	Monthly	Composite samples representative of the product to be land applied from either the sludge drying beds or the aerobic digester
Quantity and type of alkaline product used to stabilize biosolids (when required to meet federal pathogen and vector attraction reduction requirements in 40 CFR 503.32(b)(3) and 40 CFR 503.33(b)(6)	Each Occurrence	Measurement
Initial time when solids that received alkaline agent ascended to pH 12	Each Batch	Date, time, and actual pH measurement
2 hours after initial alkaline addition and sustained at pH 12	Each Batch	Date, time, and actual pH measurement
24 hours after initial alkaline addition and pH 11.5 was sustained	Each Batch	Date, time, and actual pH measurement
Record of locations where sludge is applied on land (Site location map to be maintained at treatment facility for review upon request by DEQ)	Each Occurrence	Date, Volume & locations where sludges were applied recorded on site location map.

d. South Santiam River - May 1 through October 31_ (See Note 2/)

Item or Parameter	Minimum Frequency	Type of Sample
Flow (upstream)	2/Week	Measurement
Temperature (upstream)	2/Week	Record
Temperature (downstream)	2/Week	Record

e. Outfall Number 002-004 - Emergency Overflows 002 through 004

Item or Parameter	Minimum Frequency	Type of Sample
Flow	Daily (During each occurrence)	Estimated Volume and Duration

2. Reporting Procedures

a. Monitoring results shall be reported on approved forms. The reporting period is the calendar month. Reports must be submitted to the Department's Western Region - Salem office by the 15th day of the following month.

b. State monitoring reports shall identify the name, certificate classification and grade level of each principal operator designated by the permittee as responsible for supervising the wastewater collection and treatment systems during the reporting period. Monitoring reports shall also identify each system classification as found on page one of this permit.

c. Monitoring reports shall also include a record of the quantity and method of use of all biosolids removed from the treatment facility and a record of all applicable equipment breakdowns and bypassing.

3. Report Submittals

a. The permittee shall have in place a program to identify and reduce inflow and infiltration into the sewage collection system. An annual report shall be submitted to the Department by February 1 of each year which details sewer collection maintenance activities that reduce inflow and infiltration. The report shall state those activities that have been done in the previous year and those activities planned for the following year.

b. For any year in which biosolids are land applied, a report shall be submitted to the Department by February 19 of the following year that describes solids handling activities for the previous year and includes, but is not limited to, the required information outlined in OAR 340-050-0035(6)(a)-(e).

NOTES:

1/ *E. coli* monitoring must be conducted according to any of the following test procedures as specified in **Standard Methods for the Examination of Water and Wastewater, 19th Edition**, or according to any test procedure that has been authorized and approved in writing by the Director or his authorized representative:

Method	Reference	Page	Method Number
mTEC agar,	MF Standard Methods, 19th Edition	9-28	9213 D
NA-MUG, MF	Standard Methods, 19th Edition	9-63	9222 G
Chromogenic Substrate, MPN	Standard Methods, 19th Edition	9-65	9223 B
Colilert QT	Idexx Laboratories, Inc.		

2/ South Santiam River flow and temperature shall be obtained at least twenty feet upstream from the outfall location. The downstream South Santiam River temperature shall be taken at the edge of the mixing zone and from within the effluent plume. All measurements shall be instantaneous values measured within a two (2) hour period.

3/ Beginning no later than six months after the installation and operation of the diffuser, the permittee shall conduct bioassay testing for a period of one (1) year in accordance with the frequency specified above. If the bioassay tests show that the effluent samples are not toxic at the dilutions determined to occur at the Zone of Immediate Dilution and the Mixing Zone, no further bioassay testing will be required during this permit cycle. Note that bioassay test results will be required along with the next NPDES permit renewal application. See Schedule D, Special Conditions, number 6.

4/ Composite samples from the digester withdrawal line shall consist of at least 4 aliquots of equal volume collected over an 8 hour period and combined.

Inorganic pollutant monitoring must be conducted according to Test Methods for Evaluating Solid Waste, Physical/Chemical Methods, Second Edition (1982) with Updates I and II and third Edition (1986) with Revision I.

5/ Composite samples from the drying bed shall be taken from reference areas in the drying bed pursuant to Test Methods for Evaluating Solid Waste, Volume 2; Field Manual, Physical/Chemical Methods, November 1986, Third Edition, Chapter 9.

Inorganic pollutant monitoring must be conducted according to Test Methods for Evaluating Solid Waste, Physical/Chemical Methods, Second Edition (1982) with Updates I and II and third Edition (1986) with Revision I.

6/ Calculation of the % volatile solids reduction is to be based on comparison of a representative grab sample of total and volatile solids entering each digester and a representative composite sample of solids exiting each digester withdrawal line (as defined in Note 4/ above) or a representative composite sample of solids from the sludge drying beds (as defined in Note 5/ above).

SCHEDULE C

Compliance Schedules and Conditions

1. By no later than two (2) years after issuance of this permit, the permittee shall submit for Department approval a temperature management plan. The temperature management plan shall include an assessment of temperature in the receiving water as it relates to the new standards. In addition, the plan shall identify the impact of the plant discharge on temperature based on both existing conditions and on improvements proposed for the outfall.

2. Within one year of permit issuance, the permittee shall submit to the Department for review and approval a proposed program and time schedule for identifying and removing inflow. Within 60 days of receiving written Department comments, the permittee shall submit a final approvable program and time schedule. The program shall consist of the following:

   a. Identification of all overflow points and verification that sewer system overflows are not occurring up to a 24-hour, 5-year storm event or equivalent;

   b. Monitoring of all pump station overflow points;

   c. A program for identifying and removing all inflow sources into the permittee's sewer system over which the permittee has legal control; and

   d. If the permittee does not have the necessary legal authority for all portions of the sewer system or treatment facility, a program and schedule for gaining legal authority to require inflow reduction and a program and schedule for removing those sources.

3. Within 180 days of permit issuance, the permittee shall submit to the Department for review and approval a proposed plan and schedule to remove the overland discharge of the City's treated effluent outfall into the South Santiam River. These modifications to the outfall shall be installed in accordance to Department approved engineering plans, specifications, and schedule.

4. The permittee is expected to meet the compliance dates which have been established in this schedule. Either prior to or no later than 14 days following any lapsed compliance date, the permittee shall submit to the Department a notice of compliance or noncompliance with the established schedule. The Director may revise a schedule of compliance if he determines good and valid cause resulting from events over which the permittee has little or no control.

SCHEDULE D

Special Conditions

1. An adequate contingency plan for prevention and handling of spills and unplanned discharges shall be in force at all times. A continuing program of employee orientation and education shall be maintained to ensure awareness of the necessity of good inplant control and quick and proper action in the event of a spill or accident.

2. All biosolids shall be managed in accordance with the current biosolids management plan approved by the Department and the site authorization letters issued by the Department. The biosolids management plan shall be kept current and remain on file with the permit. No substantial changes shall be made in solids management activities which significantly differ from operations specified under the approved plan without the prior written approval of the Department.

   This permit may be modified to incorporate any applicable standard for sewage sludge use of disposal promulgated under section 405(d) of the Clean Water Act, if the standard for sewage sludge use or disposal is more stringent than any requirements for sludge use or disposal in the permit, or controls a pollutant or practice not limited in this permit.

3. Prior to increasing thermal load from the facility (design flow or temperature), the Permittee shall notify the Department in writing and obtain necessary approval from the Department.

4. Unless otherwise approved in writing by the Department, all inflow sources identified in Schedule C, Condition 2 of this Schedule are to be permanently disconnected from the sanitary sewer system in accordance with the plan listed in Schedule C.

5. The permittee shall notify the DEQ Western Region - Salem Office (503) 378-8240 in accordance with the response times noted in the General Conditions of this permit, of any malfunction so that corrective action can be coordinated between the permittee and the Department.

6. Bioassay - Whole Effluent Toxicity Testing

   a. The permittee shall conduct whole effluent toxicity tests as specified in Schedule B of this permit.

   b. Bioassay tests may be dual end-point tests in which both acute and chronic end-points can be determined from the results of a single chronic test (the acute end-point shall be based upon a 48-hour time period).

   c. Acute Toxicity Testing - Organisms and Protocols

      (1) The permittee shall conduct 48-hour static renewal tests with the Ceriodaphnia dubia (water flea) and the Pimephales promelas (fathead minnow).

      (2) The presence of acute toxicity will be determined as specified in **Methods for Measuring the Acute Toxicity of Effluents and Receiving Waters to Freshwater and Marine Organisms**, Fourth Edition, EPA/600/4-90/027F, August 1993.

      (3) An acute bioassay test shall be considered to show toxicity if there is a statistically significant difference in survival between the control and 100 percent effluent, unless the permit specifically provides for a Zone of Immediate Dilution (ZID) for biotoxicity. If the permit specifies such a ZID, acute toxicity shall be indicated when a statistically significant difference in survival occurs at dilutions greater than that which is found to occur at the edge of the ZID.

d. Chronic Toxicity Testing - Organisms and Protocols

(1) The permittee shall conduct tests with: *Pimephales promelas* (fathead minnow) for growth and survival test endpoint, *Ceriodaphnia dubia* (water flea) for reproduction and survival test endpoint, and *Selanastrum capricornutum* (green alga) for growth test endpoint.

(2) The presence of chronic toxicity shall be estimated as specified in **Short-Term Methods for Estimating the Chronic Toxicity of Effluents and Receiving Waters to Freshwater Organisms**, Third Edition, EPA/600/4-91/002, July 1994.

(3) A chronic bioassay test shall be considered to show toxicity if a statistically significant difference in survival, growth, or reproduction occurs at dilutions greater than that which is known to occur at the edge of the mixing zone. If there is no dilution data for the edge of the mixing zone, any chronic bioassay test that shows a statistically significant effect in 100 percent effluent as compared to the control shall be considered to show toxicity.

e. Quality Assurance

Quality assurance criteria, statistical analyses and data reporting for the bioassays shall be in accordance with the EPA documents stated in this condition and the Department's **Whole Effluent Toxicity Testing Guidance Document**, January 1993.

f. Evaluation of Causes and Exceedances

(1) If toxicity is shown, as defined in sections c.(3) or d.(3) of this permit condition, another toxicity test using the same species and Department approved methodology shall be conducted within two weeks, unless otherwise approved by the Department. If the second test also indicates toxicity, the permittee shall follow the procedure described in section f.(2) of this permit condition.

(2) If two consecutive bioassay test results indicate acute and/or chronic toxicity, as defined in sections c.(3) or d.(3) of this permit condition, the permittee shall evaluate the source of the toxicity and submit a plan and time schedule for demonstrating compliance with water quality standards. Upon approval by the Department, the permittee shall implement the plan until compliance has been achieved. Evaluations shall be completed and plans submitted to the Department within 6 months unless otherwise approved in writing by the Department.

g. Reporting

Along with the test results, the permittee shall include: 1. the dates of sample collection and initiation of each toxicity test; 2. the type of production; and 3. the flow rate at the time of sample collection. Effluent at the time of sampling for bioassay testing should include split samples of required parameters stated under Schedule B, condition 1. of this permit.

h. Reopener

If bioassay testing indicates acute and/or chronic toxicity, the Department may reopen and modify this permit to include new limitations and/or conditions as determined by the Department to be appropriate, and in accordance with procedures outlined in Oregon Administrative Rules, Chapter 340, Division 45.

7. The permittee shall comply with Oregon Administrative Rules (OAR), Chapter 340, Division 49, "Regulations Pertaining To Certification of Wastewater System Operator Personnel" and accordingly:

a. The permittee shall have its wastewater system supervised by one or more operators who are certified in a classification and grade level (equal to or greater) that corresponds with the classification (collection and/or treatment) of the system to be supervised as specified on page one of this permit.

Note: A "supervisor" is defined as the person exercising authority for establishing and executing the specific practice and procedures of operating the system in accordance with the policies of the permittee and requirements of the waste discharge permit. "Supervise" means responsible for the technical operation of a system, which may affect its performance or the quality of the effluent produced. Supervisors are not required to be on-site at all times.

b. The permittee's wastewater system may not be without supervision (as required by Special Condition 7.a. above) for more than thirty (30) days. During this period, and at any time that the supervisor is not available to respond on-site (i.e. vacation, sick leave or off-call), the permittee must make available another person who is certified at no less than one grade lower then the system classification.

c. If the wastewater system has more than one daily shift, the permittee shall have the shift supervisor, if any, certified at no less than one grade lower than the system classification.

d. The permittee is responsible for ensuring the wastewater system has a properly certified supervisor available at all times to respond on-site at the request of the permittee and to any other operator.

e. The permittee shall notify the Department of Environmental Quality in writing within thirty (30) days of replacement or redesignation of certified operators responsible for supervising wastewater system operation. The notice shall be filed with the Water Quality Division, Operator Certification Program, 811 SW 6th Ave, Portland, OR 97204. This requirement is in addition to the reporting requirements contained under Schedule B of this permit.

f. Upon written request, the Department may grant the permittee reasonable time, not to exceed 120 days, to obtain the services of a qualified person to supervise the wastewater system. The written request must include justification for the time needed, a schedule for recruiting and hiring, the date the system supervisor availability ceased and the name of the alternate system supervisor(s) as required by 7.b. above.

NPDES GENERAL CONDITIONS
(SCHEDULE F)

SECTION A. STANDARD CONDITIONS

1. Duty to Comply

The permittee must comply with all conditions of this permit. Any permit noncompliance constitutes a violation of Oregon Revised Statutes (ORS) 468B.025 and is grounds for enforcement action; for permit termination, suspension, or modification; or for denial of a permit renewal application.

2. Penalties for Water Pollution and Permit Condition Violations

Oregon Law (ORS 468.140) allows the Director to impose civil penalties up to $10,000 per day for violation of a term, condition, or requirement of a permit.

In addition, a person who unlawfully pollutes water as specified in ORS 468.943 or ORS 468.946 is subject to criminal prosecution.

3. Duty to Mitigate

The permittee shall take all reasonable steps to minimize or prevent any discharge or sludge use or disposal in violation of this permit which has a reasonable likelihood of adversely affecting human health or the environment. In addition, upon request of the Department, the permittee shall correct any adverse impact on the environment or human health resulting from noncompliance with this permit, including such accelerated or additional monitoring as necessary to determine the nature and impact of the noncomplying discharge.

4. Duty to Reapply

If the permittee wishes to continue an activity regulated by this permit after the expiration date of this permit, the permittee must apply for and have the permit renewed. The application shall be submitted at least 180 days before the expiration date of this permit.

The Director may grant permission to submit an application less than 180 days in advance but no later than the permit expiration date.

5. Permit Actions

This permit may be modified, suspended, revoked and reissued, or terminated for cause including, but not limited to, the following:

a. Violation of any term, condition, or requirement of this permit, a rule, or a statute;

b. Obtaining this permit by misrepresentation or failure to disclose fully all material facts; or

c. A change in any condition that requires either a temporary or permanent reduction or elimination of the authorized discharge.

The filing of a request by the permittee for a permit modification or a notification of planned changes or anticipated noncompliance, does not stay any permit condition.

6. Toxic Pollutants

The permittee shall comply with any applicable effluent standards or prohibitions established under Section 307(a) of the Clean Water Act for toxic pollutants within the time provided in the regulations that establish those standards or prohibitions, even if the permit has not yet been modified to incorporate the requirement.

7. Property Rights

The issuance of this permit does not convey any property rights of any sort, or any exclusive privilege.

8. Permit References

Except for effluent standards or prohibitions established under Section 307(a) of the Clean Water Act for toxic pollutants and standards for sewage sludge use or disposal established under Section 405(d) of the Clean Water Act, all rules and statutes referred to in this permit are those in effect on the date this permit is issued.

SECTION B. OPERATION AND MAINTENANCE OF POLLUTION CONTROLS

1. Proper Operation and Maintenance

The permittee shall at all times properly operate and maintain all facilities and systems of treatment and control (and related appurtenances) which are installed or used by the permittee to achieve compliance with the conditions of this permit. Proper operation and maintenance also includes adequate laboratory controls, and appropriate quality assurance procedures. This provision requires the operation of back-up or auxiliary facilities or similar systems which are installed by a permittee only when the operation is necessary to achieve compliance with the conditions of the permit.

2. Duty to Halt or Reduce Activity

For industrial or commercial facilities, upon reduction, loss, or failure of the treatment facility, the permittee shall, to the extent necessary to maintain compliance with its permit, control production or all discharges or both until the facility is restored or an alternative method of treatment is provided. This requirement applies, for example, when the primary source of power of the treatment facility fails or is reduced or lost. It shall not be a defense for a permittee in an enforcement action that it would have been necessary to halt or reduce the permitted activity in order to maintain compliance with the conditions of this permit.

3. Bypass of Treatment Facilities

a. Definitions

(1) "Bypass" means intentional diversion of waste streams from any portion of the treatment facility. The term "bypass" does not include nonuse of singular or multiple units or processes of a treatment works when the nonuse is insignificant to the quality and/or quantity of the effluent produced by the treatment works. The term "bypass" does not apply if the diversion does not cause effluent limitations to be exceeded, provided the diversion is to allow essential maintenance to assure efficient operation.

(2) "Severe property damage" means substantial physical damage to property, damage to the treatment facilities or treatment processes which causes them to become inoperable, or substantial and permanent loss of natural resources which can reasonably be expected to occur in the absence of a bypass. Severe property damage does not mean economic loss caused by delays in production.

b. Prohibition of bypass.

(1) Bypass is prohibited unless:

(a) Bypass was necessary to prevent loss of life, personal injury, or severe property damage;

(b) There were no feasible alternatives to the bypass, such as the use of auxiliary treatment facilities, retention of untreated wastes, or maintenance during normal periods of equipment downtime. This condition is not satisfied if adequate backup equipment should have been installed in the exercise of reasonable engineering judgement to prevent a bypass which occurred during normal periods of equipment downtime or preventative maintenance; and

(c) The permittee submitted notices and requests as required under General Condition B.3.c.

(2) The Director may approve an anticipated bypass, after considering its adverse effects and any alternatives to bypassing, when the Director determines that it will meet the three conditions listed above in General Condition B.3.b.(1).

c. Notice and request for bypass.

(1) Anticipated bypass. If the permittee knows in advance of the need for a bypass, it shall submit prior written notice, if possible at least ten days before the date of the bypass.

(2) Unanticipated bypass. The permittee shall submit notice of an unanticipated bypass as required in General Condition D.5.

4. Upset

a. Definition. "Upset" means an exceptional incident in which there is unintentional and temporary noncompliance with technology based permit effluent limitations because of factors beyond the reasonable control of the permittee. An upset does not include noncompliance to the extent caused by operation error, improperly designed treatment facilities, inadequate treatment facilities, lack of preventative maintenance, or careless or improper operation.

b. Effect of an upset. An upset constitutes an affirmative defense to an action brought for noncompliance with such technology based permit effluent limitations if the requirements of General Condition B.4.c are met. No determination made during administrative review of claims that noncompliance was caused by upset, and before an action for noncompliance, is final administrative action subject to judicial review.

c. Conditions necessary for a demonstration of upset. A permittee who wishes to establish the affirmative defense of upset shall demonstrate, through properly signed, contemporaneous operating logs, or other relevant evidence that:

(1) An upset occurred and that the permittee can identify the causes(s) of the upset;

(2) The permitted facility was at the time being properly operated;

(3) The permittee submitted notice of the upset as required in General Condition D.5, hereof (24-hour notice); and

(4) The permittee complied with any remedial measures required under General Condition A.3 hereof.

d. Burden of proof. In any enforcement proceeding the permittee seeking to establish the occurrence of an upset has the burden of proof.

5. Treatment of Single Operational Event

For purposes of this permit, A Single Operational Event which leads to simultaneous violations of more than one pollutant parameter shall be treated as a single violation. A single operational event is an exceptional incident which causes simultaneous, unintentional, unknowing (not the result of a knowing act or omission), temporary noncompliance with more than one Clean Water Act effluent discharge pollutant parameter. A single operational event does not include Clean Water Act violations involving discharge without a NPDES permit or noncompliance to the extent caused by improperly designed or inadequate treatment facilities. Each day of a single operational event is a violation.

6. Overflows from Wastewater Conveyance Systems and Associated Pump Stations

a. Definitions

(1) "Overflow" means the diversion and discharge of waste streams from any portion of the wastewater conveyance system including pump stations, through a designed overflow device or structure, other than discharges to the wastewater treatment facility.

(2) "Severe property damage" means substantial physical damage to property, damage to the conveyance system or pump station which causes them to become inoperable, or substantial and permanent loss of natural resources which can reasonably be expected to occur in the absence of an overflow.

(3) "Uncontrolled overflow" means the diversion of waste streams other than through a designed overflow device or structure, for example to overflowing manholes or overflowing into residences, commercial establishments, or industries that may be connected to a conveyance system.

b. Prohibition of overflows. Overflows are prohibited unless:

(1) Overflows were unavoidable to prevent an uncontrolled overflow, loss of life, personal injury, or severe property damage;

(2) There were no feasible alternatives to the overflows, such as the use of auxiliary pumping or conveyance systems, or maximization of conveyance system storage; and

(3) The overflows are the result of an upset as defined in General Condition B.4. and meeting all requirements of this condition.

c. Uncontrolled overflows are prohibited where wastewater is likely to escape or be carried into the waters of the State by any means.

d. Reporting required. Unless otherwise specified in writing by the Department, all overflows and uncontrolled overflows must be reported orally to the Department within 24 hours from the time the permittee becomes aware of the overflow. Reporting procedures are described in more detail in General Condition D.5.

7. Public Notification of Effluent Violation or Overflow

If effluent limitations specified in this permit are exceeded or an overflow occurs, upon request by the Department, the permittee shall take such steps as are necessary to alert the public about the extent and nature of the discharge. Such steps may include, but are not limited to, posting of the river at access points and other places, news releases, and paid announcements on radio and television.

8. Removed Substances

Solids, sludges, filter backwash, or other pollutants removed in the course of treatment or control of wastewaters shall be disposed of in such a manner as to prevent any pollutant from such materials from entering public waters, causing nuisance conditions, or creating a public health hazard.

SECTION C. MONITORING AND RECORDS

1. Representative Sampling

Sampling and measurements taken as required herein shall be representative of the volume and nature of the monitored discharge. All samples shall be taken at the monitoring points specified in this permit and shall be taken, unless otherwise specified, before the effluent joins or is diluted by any other waste stream, body of water, or substance. Monitoring points shall not be changed without notification to and the approval of the Director.

2. Flow Measurements

Appropriate flow measurement devices and methods consistent with accepted scientific practices shall be selected and used to ensure the accuracy and reliability of measurements of the volume of monitored discharges. The devices shall be installed, calibrated and maintained to insure that the accuracy of the measurements is consistent with the accepted capability of that type of device. Devices selected shall be capable of measuring flows with a maximum deviation of less than $\pm$ 10 percent from true discharge rates throughout the range of expected discharge volumes.

3. Monitoring Procedures

Monitoring must be conducted according to test procedures approved under 40 CFR Part 136, unless other test procedures have been specified in this permit.

4. Penalties of Tampering

The Clean Water Act provides that any person who falsifies, tampers with, or knowingly renders inaccurate, any monitoring device or method required to be maintained under this permit shall, upon conviction, be punished by a fine of not more than $10,000 per violation, or by imprisonment for not more than two years, or by both. If a conviction of a person is for a violation committed after a first conviction of such person, punishment is a fine not more than $20,000 per day of violation, or by imprisonment of not more than four years or both.

5. Reporting of Monitoring Results

Monitoring results shall be summarized each month on a Discharge Monitoring Report form approved by the Department. The reports shall be submitted monthly and are to be mailed, delivered or otherwise transmitted by the 15th day of the following month unless specifically approved otherwise in Schedule B of this permit.

6. Additional Monitoring by the Permittee

If the permittee monitors any pollutant more frequently than required by this permit, using test procedures approved under 40 CFR 136 or as specified in this permit, the results of this monitoring shall be included in the calculation and reporting of the data submitted in the Discharge Monitoring Report. Such increased frequency shall also be indicated. For a pollutant parameter that may be sampled more than once per day (e.g., Total Chlorine Residual), only the average daily value shall be recorded unless otherwise specified in this permit.

7. Averaging of Measurements

Calculations for all limitations which require averaging of measurements shall utilize an arithmetic mean, except for bacteria which shall be averaged as specified in this permit.

8. Retention of Records

Except for records of monitoring information required by this permit related to the permittee's sewage sludge use and disposal activities, which shall be retained for a period of at least five years (or longer as required by 40 CFR part 503), the permittee shall retain records of all monitoring information, including all calibration and maintenance records of all original strip chart recordings for continuous monitoring instrumentation, copies of all reports required by this permit, and records of all data used to complete the application for this permit, for a period of at least 3 years from the date of the sample, measurement, report or application. This period may be extended by request of the Director at any time.

9. Records Contents

Records of monitoring information shall include:

a. The date, exact place, time and methods of sampling or measurements;

b. The individual(s) who performed the sampling or measurements;

c. The date(s) analyses were performed;

d. The individual(s) who performed the analyses;

e. The analytical techniques or methods used; and

f. The results of such analyses.

10. Inspection and Entry

The permittee shall allow the Director, or an authorized representative upon the presentation of credentials to:

a. Enter upon the permittee's premises where a regulated facility or activity is located or conducted, or where records must be kept under the conditions of this permit;

b. Have access to and copy, at reasonable times, any records that must be kept under the conditions of this permit;

c. Inspect at reasonable times any facilities, equipment (including monitoring and control equipment), practices, or operations regulated or required under this permit, and

d. Sample or monitor at reasonable times, for the purpose of assuring permit compliance or as otherwise authorized by state law, any substances or parameters at any location.

SECTION D. REPORTING REQUIREMENTS

1. Planned Changes

The permittee shall comply with Oregon Administrative Rules (OAR) 340, Division 52, "Review of Plans and Specifications". Except where exempted under OAR 340-52, no construction, installation, or modification involving disposal systems, treatment works, sewerage systems, or common sewers shall be commenced until the plans and specifications are submitted to and approved by the Department. The permittee shall give notice to the Department as soon as possible of any planned physical alternations or additions to the permitted facility.

2. Anticipated Noncompliance

The permittee shall give advance notice to the Director of any planned changes in the permitted facility or activity which may result in noncompliance with permit requirements.

3. Transfers

This permit may be transferred to a new permittee provided the transferee acquires a property interest in the permitted activity and agrees in writing to fully comply with all the terms and conditions of the permit and the rules of the Commission. No permit shall be transferred to a third party without prior written approval from the Director. The permittee shall notify the Department when a transfer of property interest takes place.

4. Compliance Schedule

Reports of compliance or noncompliance with, or any progress reports on interim and final requirements contained in any compliance schedule of this permit shall be submitted no later than 14 days following each schedule date. Any reports of noncompliance shall include the cause of noncompliance, any remedial actions taken, and the probability of meeting the next scheduled requirements.

5. Twenty-Four Hour Reporting

The permittee shall report any noncompliance which may endanger health or the environment. Any information shall be provided orally (by telephone) within 24 hours, unless otherwise specified in this permit, from the time the permittee becomes aware of the circumstances. During normal business hours, the Department's Regional office shall be called. Outside of normal business hours, the Department shall be contacted at 1-800-452-0311 (Oregon Emergency Response System).

A written submission shall also be provided within 5 days of the time the permittee becomes aware of the circumstances. If the permittee is establishing an affirmative defense of upset or bypass to any offense under ORS 468.922 to 468.946, and in which case if the original reporting notice was oral, delivered written notice must be made to the Department or other agency with regulatory jurisdiction within 4 (four) calendar days. The written submission shall contain:

a. A description of the noncompliance and its cause;

b. The period of noncompliance, including exact dates and times;

c. The estimated time noncompliance is expected to continue if it has not been corrected;

d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance; and

e. Public notification steps taken, pursuant to General Condition B.7.

The following shall be included as information which must be reported within 24 hours under this paragraph:

a. Any unanticipated bypass which exceeds any effluent limitation in this permit.

b. Any upset which exceeds any effluent limitation in this permit.

c. Violation of maximum daily discharge limitation for any of the pollutants listed by the Director in this permit.

The Department may waive the written report on a case-by-case basis if the oral report has been received within 24 hours.

6. Other Noncompliance

The permittee shall report all instances of noncompliance not reported under General Condition D.4 or D.5, at the time monitoring reports are submitted. The reports shall contain:

a. A description of the noncompliance and its cause;

b. The period of noncompliance, including exact dates and times;

c. The estimated time noncompliance is expected to continue if it has not been corrected; and

d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance.

7. Duty to Provide Information

The permittee shall furnish to the Department, within a reasonable time, any information which the Department may request to determine compliance with this permit. The permittee shall also furnish to the Department, upon request, copies of records required to be kept by this permit.

Other Information: When the permittee becomes aware that it failed to submit any relevant facts in a permit application, or submitted incorrect information in a permit application or any report to the Department, it shall promptly submit such facts or information.

8. Signatory Requirements

All applications, reports or information submitted to the Department shall be signed and certified in accordance with 40 CFR 122.22.

9. Falsification of Information

A person who supplies the Department with false information, or omits material or required information, as specified in ORS 468.953 is subject to criminal prosecution.

10. Changes to Indirect Dischargers - **[Applicable to Publicly Owned Treatment Works (POTW) only]**

The permittee must provide adequate notice to the Department of the following:

a. Any new introduction of pollutants into the POTW from an indirect discharger which would be subject to section 301 or 306 of the Clean Water Act if it were directly discharging those pollutants and;

b. Any substantial change in the volume or character of pollutants being introduced into the POTW by a source introducing pollutants into the POTW at the time of issuance of the permit.

c. For the purposes of this paragraph, adequate notice shall include information on (i) the quality and quantity of effluent introduced into the POTW, and (ii) any anticipated impact of the change on the quantity or quality of effluent to be discharged from the POTW.

11. Changes to Discharges of Toxic Pollutant - **[Applicable to existing manufacturing, commercial, mining, and silvicultural dischargers only]**

The permittee must notify the Department as soon as they know or have reason to believe of the following:

a. That any activity has occurred or will occur which would result in the discharge, on a routine or frequent basis, of any toxic pollutant which is not limited in the permit, if that discharge will exceed the highest of the following "notification levels:

(1) One hundred micrograms per liter (100 µg/L);

(2) Two hundred micrograms per liter (200 µg/L) for acrolein and acrylonitrile; five hundred micrograms per liter (500 µg/L) for 2,4-dinitrophenol and for 2-methyl-4,6-dinitrophenol; and one milligram per liter (1 mg/L) for antimony;

(3) Five (5) times the maximum concentration value reported for that pollutant in the permit application in accordance with 40 CFR 122.21(g)(7); or

(4) The level established by the Department in accordance with 40 CFR 122.44(f).

b. That any activity has occurred or will occur which would result in any discharge, on a non-routine or infrequent basis, of a toxic pollutant which is not limited in the permit, if that discharge will exceed the highest of the following "notification levels":

(1) Five hundred micrograms per liter (500 µg/L);

(2) One milligram per liter (1 mg/L) for antimony;

(3) Ten (10) times the maximum concentration value reported for that pollutant in the permit application in accordance with 40 CFR 122.21(g)(7); or

(4) The level established by the Department in accordance with 40 CFR 122.44(f).

SECTION E. DEFINITIONS

1. BOD means five-day biochemical oxygen demand.
2. TSS means total suspended solids.
3. mg/l means milligrams per liter.
4. kg means kilograms.
5. m^3/d means cubic meters per day.
6. MGD means million gallons per day.
7. Composite sample means a sample formed by collecting and mixing discrete samples taken periodically and based on time or flow.
8. FC means fecal coliform bacteria.
9. Technology based permit effluent limitations means technology-based treatment requirements as defined in 40 CFR 125.3, and concentration and mass load effluent limitations that are based on minimum design criteria specified in OAR 340-41.
10. CBOD means five day carbonaceous biochemical oxygen demand.
11. Grab sample means an individual discrete sample collected over a period of time not to exceed 15 minutes.
12. Quarter means January through March, April through June, July through September, or October through December.
13. Month means calendar month.
14. Week means a calendar week of Sunday through Saturday.
15. Total residual chlorine means combined chlorine forms plus free residual chlorine.
16. The term "bacteria" includes but is not limited to fecal coliform bacteria, total coliform bacteria, and E. coli bacteria.
17. POTW means a publicly owned treatment works.

Appendix B: Flood Insurance Rate Map

NOTES TO USERS

This map is for use in administering the National Flood Insurance Program. It does not necessarily identify all areas subject to flooding, particularly from local drainage sources of small size. The **community map repository** should be consulted for possible updated or additional flood hazard information.

To obtain more detailed information in areas where **Base Flood Elevations** (BFEs) and/or **floodways** have been determined, users are encouraged to consult the Flood Profiles and Floodway Data and/or Summary of Stillwater Elevations tables contained within the Flood Insurance Study (FIS) Report that accompanies this FIRM. Users should be aware that BFEs shown on the FIRM represent rounded whole-foot elevations. These BFEs are intended for flood insurance rating purposes only and should not be used as the sole source of flood elevation information. Accordingly, flood elevation data presented in the FIS Report should be utilized in conjunction with the FIRM for purposes of construction and/or floodplain management.

Boundaries of the **floodways** were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the National Flood Insurance Program. Floodway widths and other pertinent floodway data are provided in the Flood Insurance Study Report for this jurisdiction.

Certain areas not in Special Flood Hazard Areas may be protected by **flood control structures**. Refer to Section 2.4 "Flood Protection Measures" of the Flood Insurance Study Report for information on flood control structures for this jurisdiction.

The **projection** used in the preparation of this map was Oregon State Plane North Zone (FIPS zone 3601). The **horizontal datum** was NAD 83, GRS 1980 spheroid. Differences in datum, spheroid, projection or UTM zones used in the production of FIRMs for adjacent jurisdictions may result in slight positional differences in map features across jurisdiction boundaries. These differences do not affect the accuracy of this FIRM.

Flood elevations on this map are referenced to the North American Vertical Datum of 1988. These flood elevations must be compared to structure and ground elevations referenced to the same **vertical datum**. For information regarding conversion between the National Geodetic Vertical Datum of 1929 and the North American Vertical Datum of 1988, visit the National Geodetic Survey website at http://www.ngs.noaa.gov or contact the National Geodetic Survey at the following address:

NGS Information Services
NOAA, N/NGS12
National Geodetic Survey
SSMC-3, #9202
1315 East-West Highway
Silver Spring, Maryland 20910-3282
(301) 713-3242

To obtain current elevation, description, and/or location information for **bench marks** shown on this map, please contact the Information Services Branch of the National Geodetic Survey at **(301) 713- 3242**, or visit its website at http://www.ngs.noaa.gov.

Base map information shown on this FIRM was derived from multiple sources. Base map files were provided in digital format by the State of Oregon and the U.S. Fish and Wildlife Service. This information was compiled from Oregon Water Resources Department (2006), OR/WA Bureau of Land Management (2000), U.S. Fish and Wildlife Service (2008), Oregon Parks and Recreation Department (2008), National Geodetic Survey (2007), the City of Albany (2006), the U.S. Census Bureau (2007), and the Linn County GIS Department (2005) at a scale of 1:24,000.

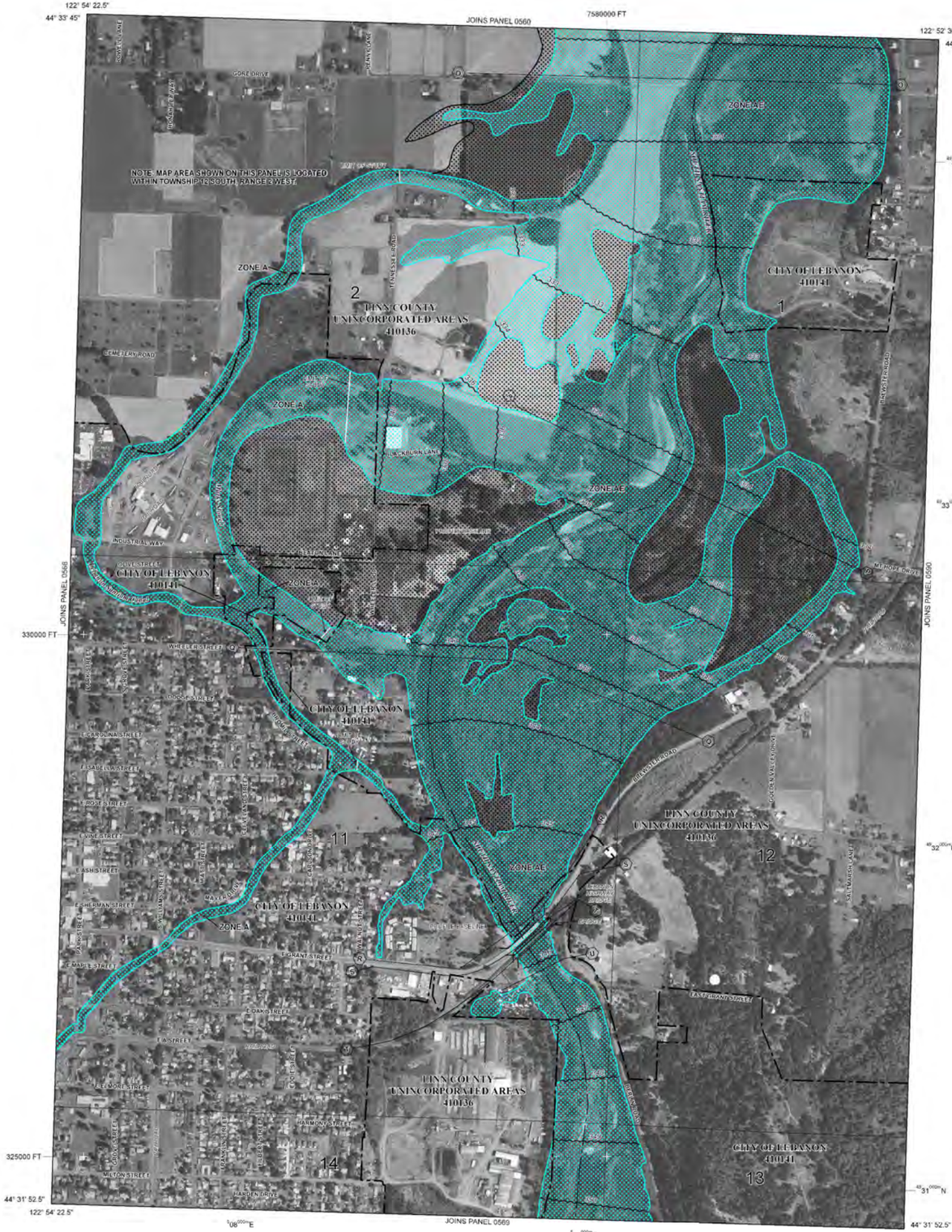
The **profile baselines** depicted on this map represent the hydraulic modeling baselines that match the flood profiles in the FIS report. As a result of improved topographic data, the **profile baseline**, in some cases, may deviate significantly from the channel centerline or appear outside the SFHA.

Corporate limits shown on this map are based on the best data available at the time of publication. Because changes due to annexations or de-annexations may have occurred after this map was published, map users should contact appropriate community officials to verify current corporate limit locations.

Please refer to the separately printed **Map Index** for an overview map of the county showing the layout of map panels; community map repository addresses; and a Listing of Communities table containing National Flood Insurance Program dates for each community as well as a listing of the panels on which each community is located.

Contact the **FEMA Map Service Center** at 1-800-358-9616 for information on available products associated with this FIRM. Available products may include previously issued Letters of Map Change, a Flood Insurance Study Report, and/or digital versions of this map. The FEMA Map Service Center may also be reached by Fax at 1-800-358-9620 and its website at http://msc.fema.gov.

If you have **questions about this map** or questions concerning the National Flood Insurance Program in general, please call **1- 877- FEMA MAP** (1-877-336-2627) or visit the FEMA website at http://www.fema.gov/business/nfip/



LEGEND

SPECIAL FLOOD HAZARD AREAS (SFHAs) SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Areas formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X	Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X	Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D	Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

- 1% Annual Chance Floodplain Boundary
- 0.2% Annual Chance Floodplain Boundary
- Floodway Boundary
- Zone D boundary
- CBRS and OPA boundary
- Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.
- 513 — Base Flood Elevation line and value; elevation in feet*
- (EL 987) — Base Flood Elevation value where uniform within zone; elevation in feet*

*Referenced to the North American Vertical Datum of 1988

- (A)—(A) Cross section line
- (23)- - - -(23) Transect line
- 45° 02' 08", 93° 02' 12" Geographic coordinates referenced to the North American Datum of 1983 (NAD 83) Western Hemisphere
- 3100000 FT 5000-foot ticks: Oregon State Plane North Zone (FIPS Zone 3601), Lambert Conformal Conic projection
- 4959000 mN 1000-meter Universal Transverse Mercator grid values, zone 10N
- DX5510 × Bench mark (see explanation in Notes to Users section of this FIRM panel)
- M1.5 River Mile

MAP REPOSITORIES
Refer to Map Repositories list on Map Index
EFFECTIVE DATE OF COUNTYWIDE FLOOD INSURANCE RATE MAP
September 29, 2010
EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL:

For community map revision history prior to countywide mapping, refer to the Community Map History table located in the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

MAP SCALE 1" = 500'

250 0 500 1000 FEET
150 0 150 300 METERS

NFIP
NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0567G

FIRM

FLOOD INSURANCE RATE MAP

LINN COUNTY, OREGON AND INCORPORATED AREAS

PANEL 567 OF 1575
(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

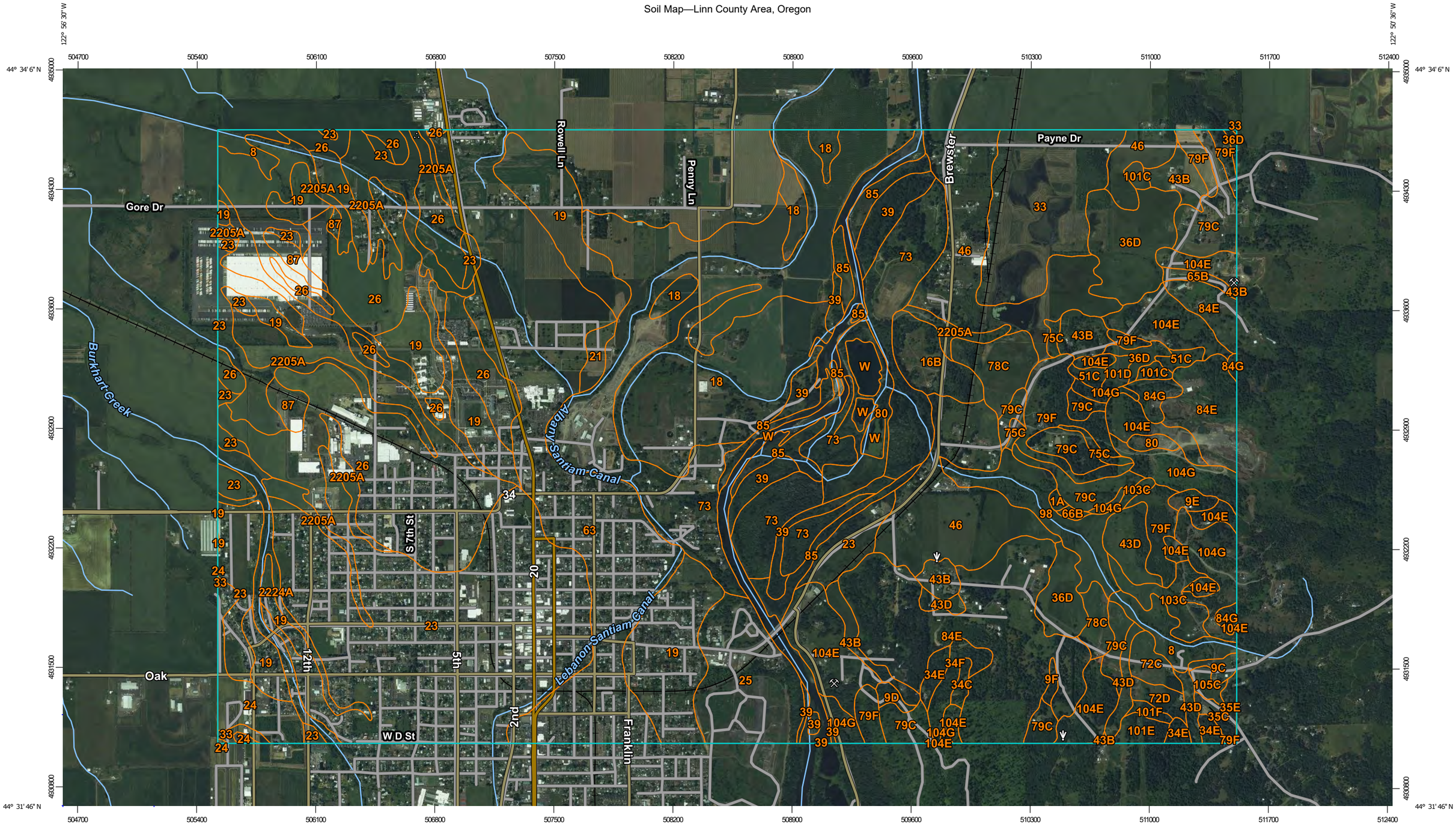
COMMUNITY	NUMBER	PANEL	SUFFIX
LEBANON, CITY OF	410141	0567	G
LINN COUNTY	410136	0567	G

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
41043C0567G
EFFECTIVE DATE
September 29, 2010
Federal Emergency Management Agency

Appendix C: Soils Map

Soil Map—Linn County Area, Oregon



Map Scale: 1:21,000 if printed on B landscape (17" x 11") sheet.

N

Meters: 0 300 600 1200 1800

Feet: 0 1000 2000 4000 6000

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 10N WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features
- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Linn County Area, Oregon
Survey Area Data: Version 20, Aug 30, 2024

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: May 28, 2020—May 29, 2020

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
1A	Abiqua silty clay loam, 0 to 3 percent slopes	2.1	0.0%
8	Bashaw silty clay	41.1	0.8%
9C	Bellpine silty clay loam, 3 to 12 percent slopes	6.4	0.1%
9D	Bellpine silty clay loam, 12 to 20 percent slopes	4.9	0.1%
9E	Bellpine silty clay loam, 20 to 30 percent slopes	6.7	0.1%
9F	Bellpine silty clay loam, 30 to 50 percent slopes	3.1	0.1%
16B	Briedwell silt loam, 0 to 7 percent slopes	39.9	0.7%
18	Camas gravelly sandy loam	224.0	4.2%
19	Chapman loam	305.3	5.7%
21	Chehalis silty clay loam	11.3	0.2%
23	Clackamas gravelly silt loam	603.4	11.3%
24	Clackamas variant silt loam	41.8	0.8%
25	Cloquato silt loam	105.0	2.0%
26	Coburg silty clay loam	469.1	8.8%
33	Dayton silt loam	147.1	2.8%
34C	Dixonville silty clay loam, 3 to 12 percent slopes	4.9	0.1%
34E	Dixonville silty clay loam, 12 to 30 percent slopes	16.9	0.3%
34F	Dixonville silty clay loam, 30 to 50 percent slopes	18.0	0.3%
35C	Dixonville-Philomath-Hazelair complex, 3 to 12 percent slopes	4.2	0.1%
35E	Dixonville-Philomath-Hazelair complex, 12 to 35 percent slopes	7.3	0.1%
36D	Dupee silt loam, 3 to 20 percent slopes	105.9	2.0%
39	Fluvents-Fluvaquents complex, nearly level	75.5	1.4%
43B	Hazelair silty clay loam, 2 to 7 percent slopes	60.8	1.1%
43D	Hazelair silty clay loam, 7 to 20 percent slopes	48.2	0.9%
46	Holcomb silt loam	298.8	5.6%

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
51C	Jory silty clay loam, 2 to 12 percent slopes	10.8	0.2%
63	Malabon silty clay loam	598.2	11.2%
65B	Marcola cobbly silty clay loam, 2 to 7 percent slopes	6.7	0.1%
66B	McAlpin silty clay loam, 3 to 6 percent slopes	2.3	0.0%
72C	Nekia silty clay loam, 2 to 12 percent slopes	15.3	0.3%
72D	Nekia silty clay loam, 12 to 20 percent slopes	11.3	0.2%
73	Newberg fine sandy loam	525.6	9.8%
75C	Panther silty clay loam, 2 to 12 percent slopes	10.9	0.2%
78C	Philomath silty clay, 3 to 12 percent slopes	50.2	0.9%
79C	Philomath cobbly silty clay, 3 to 12 percent slopes	120.8	2.3%
79F	Philomath cobbly silty clay, 12 to 45 percent slopes	97.4	1.8%
80	Pits	43.9	0.8%
84E	Ritner cobbly silty clay loam, 2 to 30 percent slopes	299.1	5.6%
84G	Ritner cobbly silty clay loam, 30 to 60 percent slopes	16.8	0.3%
85	Riverwash	73.1	1.4%
87	Salem gravelly silt loam	23.1	0.4%
98	Waldo silty clay loam	57.2	1.1%
101C	Willakenzie clay loam, 2 to 12 percent slopes	8.1	0.2%
101D	Willakenzie clay loam, 12 to 20 percent slopes	6.3	0.1%
101E	Willakenzie clay loam, 20 to 30 percent slopes	11.0	0.2%
101F	Willakenzie clay loam, 30 to 50 percent slopes	4.2	0.1%
103C	Witham silty clay, 2 to 12 percent slopes	29.3	0.5%
104E	Witzel very cobbly loam, 3 to 30 percent slopes	173.7	3.3%
104G	Witzel very cobbly loam, 30 to 70 percent slopes	86.9	1.6%
105C	Witzel variant very cobbly silt loam, 0 to 12 percent slopes	6.6	0.1%
2205A	Conser silty clay loam, 0 to 3 percent slopes	254.2	4.8%

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
2224A	Courtney gravelly silty clay loam, 0 to 3 percent slopes	44.2	0.8%
W	Water	97.6	1.8%
Totals for Area of Interest		**5,337.1**	**100.0%**

Appendix D: Land Use Map

Comprehensive Plan Map

Legend

- C-RL : Residential Low Density
- C-RM : Residential Mixed Density
- C-MU : Mixed Use
- C-CM : Commercial
- C-IND : Industrial
- C-PU : Public Use
- Taxlot
- Urban Growth Boundary

N
W E
S

0 200 400 800 1,200 1,600
Feet

1 inch = 800 feet

CITY OF LEBANON
It's easier from here.

DISCLAIMER:
The City of Lebanon's infrastructure records, drawings, and other documents have been gathered over many decades, using differing standards for quality control, documentation, and verification.
While this product is for informational usage only, it may not be suitable for legal, engineering, or surveying purposes. Prior to making any property purchases or other investments based in full or in part upon the material provided, it is specifically advised that you independently verify the information contained within our records.
If misleading or inaccurate information is brought to the City's attention, a reasonable effort will be made to fix or remove it. Such concerns should be addressed by email to gis@ci.lebanon.or.us

Effective: December 8, 2004
Revised: July 10, 2013

Appendix E: Wastewater Characteristics

TECHNICAL MEMORANDUM

TO: Charles Wright - KJ

DATE: September 22, 2025

FROM: Dale Richwine - REI

CC: Mark Cullington-KJ

SUBJECT: Technical Memorandum No. 1
Lebanon WWTP
Wastewater Characteristics

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Attachment C – Determine 5-yr Winter & 10-yr Summer Flows – DEQ Methodology
Attachment D – Determine Max Day Flow ($PDAF_5$)
Attachment E – Determine Peak Hour Flow (PHF)
Attachment F – Determine Flow Peaking Factors
Attachment G – Determine Peaking Factors and Per Capita Loads for BOD_5
Attachment H – Determine Peaking Factors and Per Capita Loads for TSS
Attachment I – Lebanon Current and Projected Wastewater Flows and Loads

EXECUTIVE SUMMARY

This memorandum documents a detailed evaluation of the influent wastewater characteristics received at the Lebanon Wastewater Treatment Plant (WWTP). The characteristics were developed using published DEQ methodology. The influent wastewater was characterized for four conditions: average annual, dry weather (July 1 – September 30), dry season (May 1 – October 31) and wet season (November 1 – April 30). This information was used to make flow and load projections and to develop design criteria for the new treatment plant.

The projected average 2045 dry season influent flow is 2.72-mgd with the average 2045 wet season flow being 7.14-mgd. The projected 2045 peak day influent flow is 19.37-mgd and the peak instantaneous flow is 31.09-mgd. The wastewater characteristics for influent flow, BOD_5 and TSS are summarized in ***Table ES-1***.

Table ES-1
Lebanon WWTP
Projected 2045 Influent Wastewater Flows and Loads

Flow (mgd)					
Process Parameter	**Ave Day**	**Max. Month**	**Max. Week**	**Max Day**	**Peak Hour**
Average Annual	5.02				
Dry Weather	2.07				
Dry Season	2.72	6.04	6.26	6.63	
Wet Season	7.14	10.09	13.84	19.37	31.09

Biochemical Oxygen Demand				
Process Parameter	**Average (lbs./day)**	**Max. Month (lbs./day)**	**Max. Week (lbs./day)**	**Max Day (lbs.)**
Average Annual	3,822	6,115	10,702	12,231
Dry Season	3,372	4,384	6,070	6,408
Wet Season	4,047	5,666	9,713	11,736

Total Suspended Solids				
Process Parameter	**Average (lbs./day)**	**Max. Month (lbs./day)**	**Max. Week (lbs./day)**	**Max Day (lbs.)**
Average Annual	4,047	7,285	12,141	14,164
Dry Season	3,148	5,666	7,869	9,128
Wet Season	4,721	7,082	10,387	13,220

The projected average 2070 (Buildout) dry season buildout flow is 2.84-mgd with the average 2070 (Buildout) wet season flow being 7.38-mgd. The projected 2070 (Buildout) peak day influent flow is 19.76-mgd and the peak instantaneous flow is 31.67-mgd.

1 WASTEWATER FACILITIES PLAN

This Technical Memorandum (TM) will provide an evaluation of the wastewater characteristics of the existing facility and use these characteristics to develop the design influent wastewater characteristics to be used for the 2022 Wastewater Facilities Plan

This memorandum documents a detailed evaluation of the wastewater characteristics based on the Lebanon Wastewater Treatment Plant (WWTP) influent wastewater. This information will be used to develop flow and load projections for the upgraded treatment facility as well as provide criteria for sizing of potential processes for future treatment.

2 BACKGROUND

The function and capacity of the City of Lebanon WWTP is directly dependent on the characteristics of the influent waste stream. The wastewater characteristics for plant flow, BOD_5, TSS, and ammonia for the 5-year study period between January 2017 and December 2021 were evaluated. An emphasis was placed on the influent peak flows between January 2021 and December 2021 as the recycle streams for this period were minimized with the decommissioning of the cannibal treatment process in September 2020.

The plant influent flows and loads were evaluated using the samples taken and analyzed at the treatment plant laboratory. An analysis of the daily rainfall data from the NPDES reports as well as the Oregon State University NOAA weather station was also evaluated. This was done to develop the correlation between rainfall and plant flow.

3 SEASONAL INFLUENCE ON FLOWS AND LOADS

The plant influent flows and loads were characterized as the waste stream entering the treatment plant for each of the permitted seasons and on an annual basis. The time periods that were used were defined as the following: average annual, dry weather, dry season, and wet season. The dry and wet seasons were developed based on the periods defined by the Oregon Administrative Rules (OAR) and specified in the plant's National Pollution Discharge Elimination System (NPDES) discharge permit Fact Sheet. The actual date range for these periods is defined as:

- Average Annual – The calendar year average load condition.
- Maximum Month – The maximum month will represent the maximum 30-day average flow and load that is experienced at the plant during the year.
- Dry Weather – The period when the influent wastewater characteristics are not influenced by rainfall. The dry weather period used in this evaluation is the

period between July 1 and September 30 each year. This period is the base flow and is directly related to the flows generated by the service area population.

- Dry Season – The dry season used in this evaluation is the period of time between May 1 and October 31 each year. This is the dry season in the NPDES permit.

- Wet Season – The wet season used in this evaluation, as defined in the NPDES permit, is the period between November 1 and April 30 each year. This period represents higher flows due to Rainfall Derived Infiltration and Inflow (RDII) in the collection system.

There is a direct relationship between the plant flow and the seasonal rainfall that is experienced in the area. In a typical Pacific Northwest community, plant flows will increase following periods of steady rainfall. This is caused when the soils become saturated and the water table level rises to where it is above the inverts of the sewers. Groundwater then leaks into cracks or poor joints in the collection system resulting in increased plant flows. This can be observed at the plant by an increase in the baseline flow rate.

4 WASTEWATER CHARACTERIZATION

The influent wastewater was characterized for flow, BOD_5, TSS, and ammonia. The results of the analysis for each of these parameters will be the basis for determining the design criteria for the new treatment plant. A detailed analysis of each parameter is provided in the attachments. A summary of the results of each evaluation is provided in the following sections.

4.1 PLANT FLOW

The plant flow was evaluated under a number of scenarios to determine the characteristics to be used to project future flows and loads. The analysis of plant flow is provided in ***Attachments A – F***. A summary of each analysis is provided in the following sections. The average monthly influent flow for the period from January 2017 to December 2021 is shown in ***Figure 1***.

4.1.1 Service Area Population

The per capita flows were developed by using the populations for the year 2021 provided in the 2021 Annual Oregon Population Report developed by the Portland State University (PSU) Population Research Center. Historical population information was taken from Table 4 – Populations for Oregon and its Counties and Incorporated Cities and Towns: July 1, 2010 – July 1, 2020, estimates. The 2021 population is an estimate from the Certified Population Estimates July 1, 2021, prepared by the PSU Population Research Center.

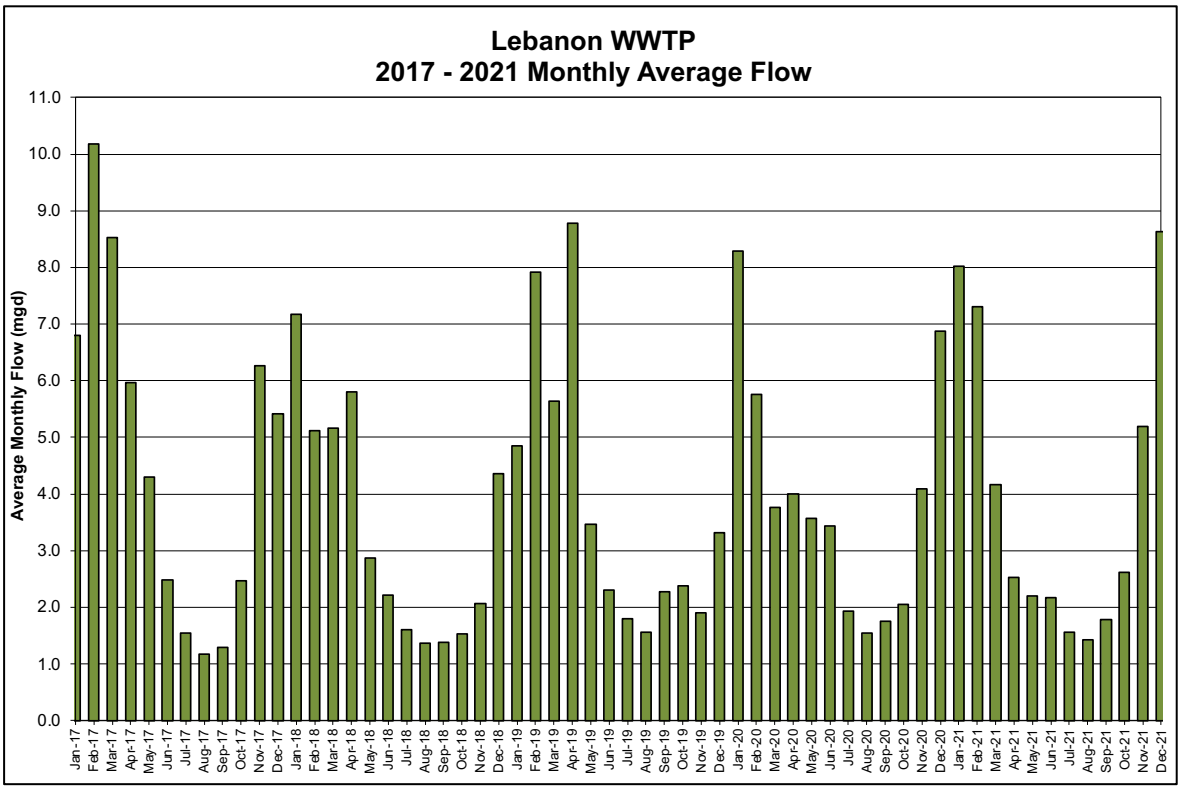

Figure 1: January 2017 – December 2021 Average Monthly Flow

December 15, 2021. The population data for the City of Lebanon is summarized in ***Table 1***.

Table 1
City of Lebanon
Service Area Population[1]

Year	2016	2017	2018	2019	2020	2021
Lebanon	16,435	16,720	16,920	17,135	17,335	19,122

Note 1: 2010 – 2020 Data from Table 4 Populations for Oregon and its Counties and Incorporated cities and Towns, PSU Population Research Center,
Note 2: 2021 Data from 2021 Certified Population Estimates July 1, 2021, prepared by Population Research Center, Portland State University

To develop the required flows and loads for design of the system to meet the desired 20-year planning horizon, it was necessary to obtain certified population forecasts for the City of Lebanon. Population projections were developed for the City of Lebanon in the "Coordinated Population Forecast for Linn County, its Urban Growth Boundaries (UGB) and Area Outside UGBs, 2017-2067. This report was developed by the Portland State Population Research Center and is the basis for planning for Linn County. This report was first published on June 30, 2017 and then updated on June 30, 2021. The population projections for the City of Lebanon as well as other areas within Linn County was provided in this report. The updated population projections in the June 30, 2021 report were used. This information for the City of Lebanon only is summarized in ***Table 2***. This study forecasts the population to grow by a total of 0.4% through 2045

and 0.2% through 2070. This shows a total population increase of 3,211 people over 50-years.

Table 2
City of Lebanon
Forecasts for Total Population

Year	Historical			Estimates		Forecast			
	2000	2010	AAGR (2000-2010)	2020	AAGR (2010-2020)	2045	2070	AAGR (2020-2045)	AAGR (2045-2070)
Lebanon	15,913	18,295	1.4%	20,314	1.0%	22,483	23,525	0.4%	0.2%

Population Forecasts prepared by: Population Research Center, Portland State University, June 30, 2021

4.1.2 Flow per Capita

Capita based flows are determined by dividing the average flow by the population for the respective population. With the Rainfall Induced Infiltration and Inflow (RDII) contributing flows to the system, a Dry Weather analysis was also completed to determine actual capita based flows. The analysis is provided in ***Attachment A***.

An evaluation was performed for period January 2016 – December 2021. To mitigate the high flows influenced by RDII, an analysis was done using the median in addition to the average to provide results that are not skewed by periodic high flows. The per capita flows are shown in ***Table 3***.

Table 3
Per Capita Flows

Period	January 2016 – December 2021	
	Average	Median
Annual Average	227	180
Dry Weather	92	89
Dry Season	122	105
Wet Season	333	299

The dry weather per capita flows were calculated using the average monthly flows for the months of July, August and September. The dry season flows used the average monthly flows for the months of May through October. The analysis of dry weather flows vs. dry season flows as shown in ***Table 3*** show that there is an influence in flow during the shoulder months of May, June and October.

This analysis provides different results based on the period being analyzed. The results are as follows:

- Dry Season Flow - When using the dry season flow for the period between May 1 and October 30, the gpd/capita is 122 gpd/capita. This is the method typically used for determining this value. In the situation for the Lebanon WWTP the flows in this period are influenced by rainfall. Therefore, the median flow for this period was determined with the result being 105 gpd/capita.

- Dry Weather Months (July - September) - The analysis was done taking the rainfall influenced flows for the months of May, June and October out of the data set. In this analysis, the flow per capita was 92 gpd/capita for the monthly average flows, 89 gpd/capita for the median flows.

The analysis of dry weather flows shows that rainfall influenced flows are limited during the months of July, August, and September. The value for the average and median is essentially the same for this period of time showing that the effects of RDII are minimal. The dry weather per capita flows for the existing flows to be used in this planning effort will be 92 gpd/capita. The results of this evaluation are summarized in ***Table 4***. The value of 92 gpd/capita will be used for future growth planning.

Table 4
Lebanon WWTP Per Capita Flows for Planning

Variable	Ave Annual	Dry Weather	Dry Season	Wet Season	Units
Flow	227	92	122	333	gal/capita/day

4.1.3 Statistical Rainfalls

The wastewater flows exhibit variation throughout the year due to the RDII in the collection system. An extensive evaluation of flow peaking was performed to characterize the wastewater flows.

4.1.3.1 10-year Summer and 5-year Winter Rainfall

The ODEQ criteria for determining the Maximum Month Wet Weather and Maximum Month Dry Weather rainfalls[1] states that the Climatography No. 20 for the local weather station should be used. There is no weather station in the Lebanon area that is listed in the Climatography of the United States No. 20. The closest station is the Oregon State University station in Corvallis, OR. Other stations in the area were the Stayton, OR station and the McKenzie Bridge Ranger Station. In addition to Climatography of the United States No. 20 information for these three weather stations, a statistical summary of the rainfall data provided in the Lebanon WWTP NPDES monthly reports for the period January 2000 through December 2021 was evaluated and a probability table was developed. This rainfall data was gathered from a manual rain gauge located outside the treatment plant administration building. This evaluation is summarized in ***Attachment B***.

These rainfall probabilities were evaluated to determine the 10-year summer and 5-year winter storms for the City of Lebanon service area, as specified in the DEQ Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon: MMDWF, MMWWF, PDAF, and PIF. The results of this analysis are summarized in ***Table 5***.

Summer Season – The 90% probability level for summer season rainfall in the months of May and October for the two data sets were not consistent. The Oregon State University Weather Station Climatography 20 values were the closest to the values

[1] DEQ Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon: MMDWF, MMWWF, PDAF, and PIF, State of Oregon, Department of Environmental Quality Guidelines.

determined from the WWTP data. The Climatography 20 values were slightly higher and were therefore the ones used in this analysis

Table 5
City of Lebanon Statistical Rainfalls

Rainfall Probability Source	Statistical Rainfall Events			
	10-Yr Summer		5-Yr Winter	
	May	October	December	January
2000 - 2021 Rainfall	3.68	6.25	9.02	9.14
Oregon Station University Station Climatography No 20	4.12	5.93	10.46	9.46

as it was determined this data would be more dependable than the small manual rain tube used at the WWTP. The probabilities showed October as the highest rainfall month at 5.93-inches, but the DEQ methodology recommends using the May statistical rainfall as water table are higher during that period. A review of monthly flow data showed that the higher monthly flows occurred during both May and October depending on the year. Therefore, the 90% probability monthly rainfall for May based on the Climatography No. 20 for a 10-year summer monthly rainfall of 4.12-inches was used as recommended in the DEQ guidance document.

- Wet Season – The 80% probability level for winter rainfall in the months of January through April. The winter storms with the highest monthly probable rainfall was 9.46-inches in January based on the Climatography No. 20. This value will be used to determine the 5-year winter probable rainfall as recommended in the DEQ guidance.

The results of this analysis for each of the three weather stations Climatography No. 20 data that was evaluated and the Lebanon WWTP rain gauge is provided in ***Attachment B***.

4.1.3.2 5-year, 24-hour storm

The ODEQ has recommended a method for determining the probable maximum day wet weather flows for a wastewater treatment plant[2]. This is done by using the statistical 5-year, 24-hour storm for the wet weather season.. The storm is used to calculate a flow using a regression from a scatter plot of the existing peak daily flows and rainfalls that occur during saturated ground conditions.

[2] DEQ Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon: MMDWF, MMWWF, PDAF, and PIF, State of Oregon, Department of Environmental Quality Guidelines.

The 5-year, 24-hour duration storm can be obtained from NOAA in Atlas 2, Volume X for this region. As shown in ***Figure 2***, the storm for the City of Lebanon is in the range of 3.0 to 3.5 inches. It was determined that a value of 3.0 inches be used.

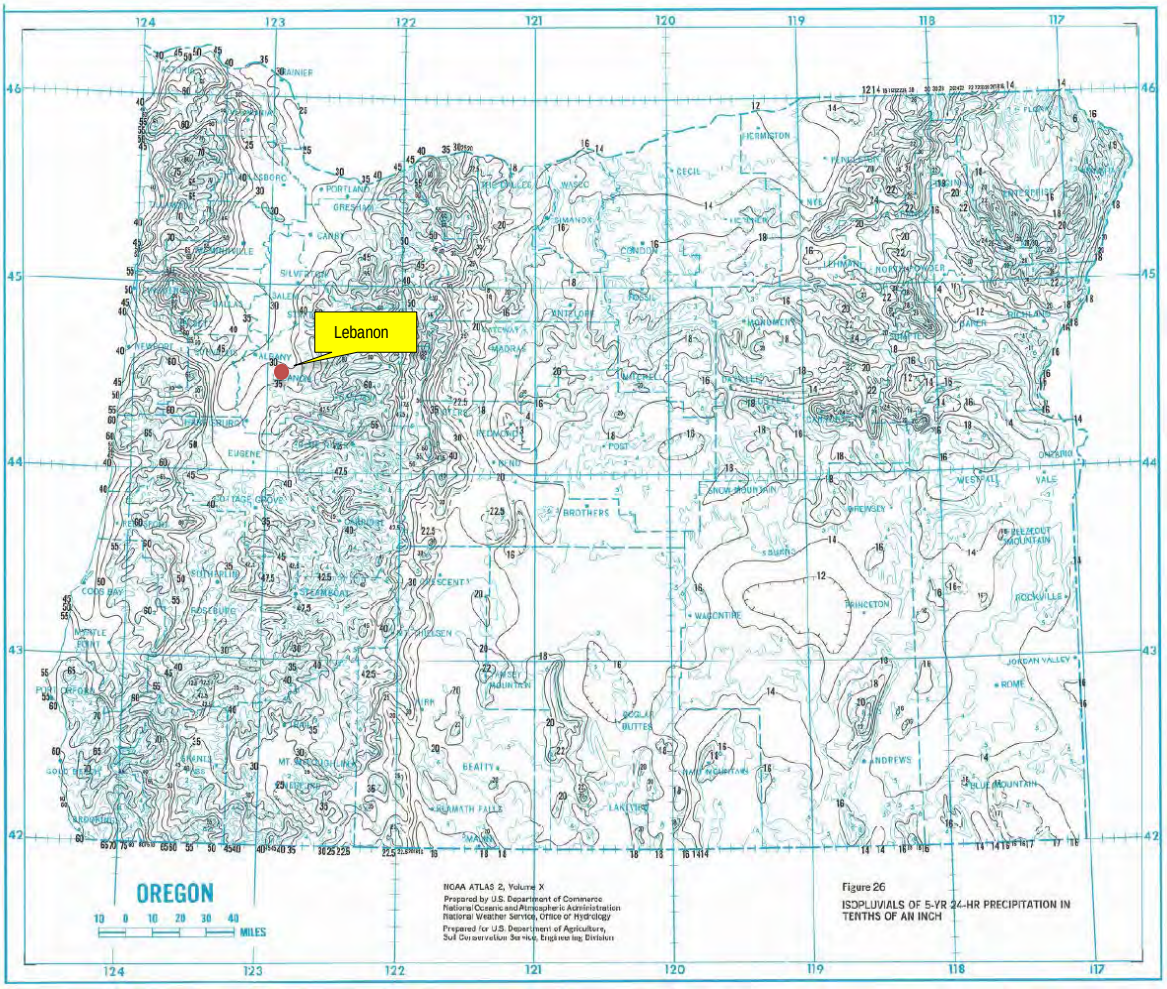

Figure 2: NOAA ATLAS 2, Volume X Isopluvials of 5-Yr, 24 Hr Precipitation

4.1.4 Flow Peaking Factors

Historical peaking factors to establish current statistical flows need to be calculated. Two methods were used to evaluate and determine the peaking factors to be used in this analysis. These were the ODEQ guidance, and an evaluation of peaking factors calculated from historical flows for the January 2017 – December 2021 period.

4.1.4.1 Maximum Month Historical Peaking Factors

Peaking factors were developed from an evaluation of historical flows. In the historical flow method, peaking factors are determined for each statistical flow as follows:

- Maximum Month Flow (MMF) -- Max. 30-day Average flow / Average Day flow
- Maximum Week Flow (MWF) -- Max. 7-day Average Flow / Average Day Flow
- Maximum Day Flow (MDF) -- Max. day flow / Average Day Flow

An analysis of plant flows for the period from January 2017 through December 2021 was performed to determine the historical peaking factors. Data was analyzed for the annual, dry season, dry weather and wet season periods. This analysis is summarized in ***Attachments A.***

The dry season flows that were used to determine the peaking factor is the average flow for the period May 1 - Oct 30 as stated in the NPDES Permit. These flows are affected by rainfall, especially in the months of May, June, and October. To mitigate the effects of RDII affected flows, an analysis was performed on flow data for the months of July, August and September to obtain the actual dry weather flows that have a minimal affect from rainfall. This resulted in a monthly average dry weather flow of 1.60-mgd. This was the dry weather flow used as the basis for calculating historical peaking factors. These peaking factors were compared with the peaking factors based on the average dry season flow of 2.13-mgd. The results are summarized in ***Table 6***.

Table 6
Lebanon WWTP
Flow Peaking Factors

Method	Average Flow	MMDWF		MMWWF	
		Flow	Peaking Factor	Flow	Peaking Factor
Dry Weather	1.60	5.36	3.35	8.83	5.52
Dry Season	2.13	5.36	2.52	8.83	4.15

4.1.4.2 Maximum Month DEQ Methodology Peaking Factors

DEQ has established guidelines for determining the statistical Maximum Month Dry Weather Flow (MMDWF) and Maximum Month Wet Weather Flow (MMWWF) for a treatment plant[3]. These guidelines outline a procedure where historical flows and rainfall are used to develop a relationship that is used to determine plant flows based on rainfall.

Creating a scatter plot of the total monthly rainfall against the average monthly flow provides the basis for a linear regression to be performed on the data to develop an equation that represents the system. This regression formula is then used to determine the MMWWF and MMDWF using the 10-year summer and 5-year winter probable rainfall.

An analysis to develop the 5-year winter rainfall and the 10-year summer rainfall was performed. The results from this evaluation (***Attachment B***) were a 10-yr May rainfall of 4.12-inches and a 5-yr Janaury rainfall of 9.46-inches.

In this analysis, scatter plots (Jan - May and Nov - Dec data), for the periods 2010-2021, 2010-2016, and 2017-2021 were developed. A regression was determined and the

[3] DEQ Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon: MMDWF, MMWWF, PDAF, and PIF, State of Oregon, Department of Environmental Quality Guidelines.

MMWWF and MMDWF was calculated for each of the periods. The plot for the period 2017-2021 was used and is shown in ***Figure 3***. This shows the flow for MMDWF being 4.93-mgd and for MMWWF being 8.34-mgd. This analysis is provided in ***Attachment C***.

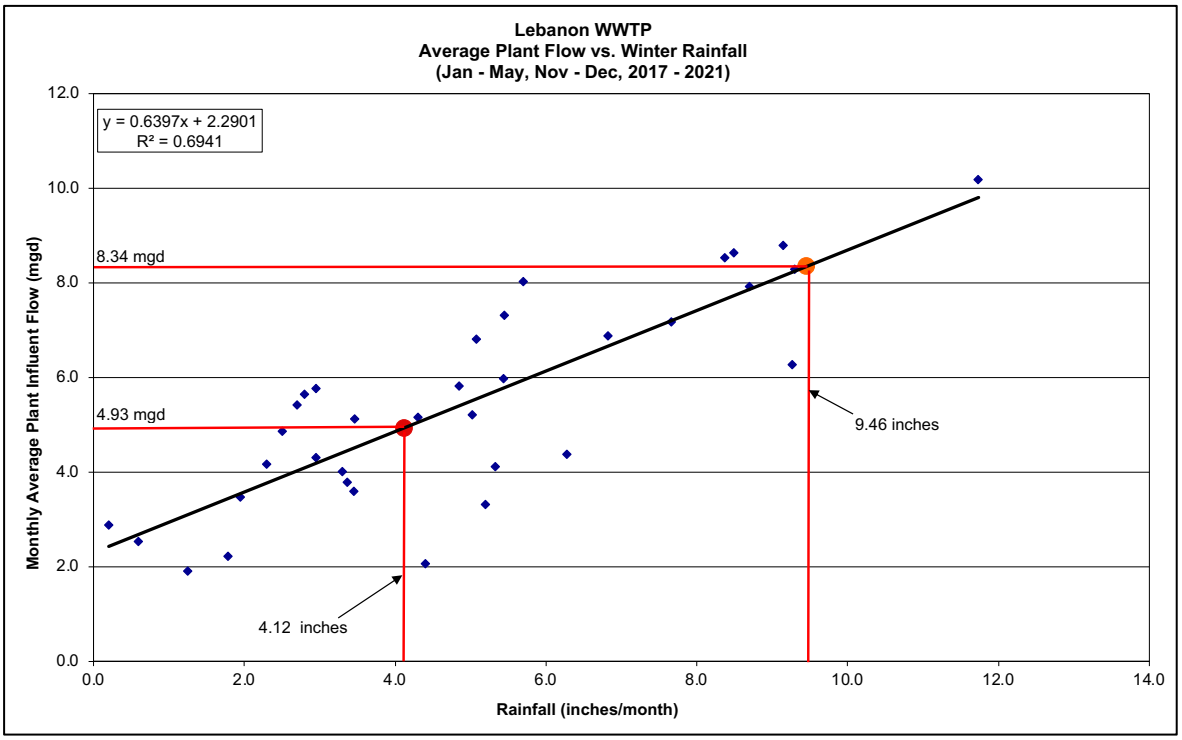

Figure 3: MMDWF and MMWWF Using DEQ Methodology

The peaking factors for the 5-year wet season (MMWWF) and the 10-year dry season (MMDWF) flows were calculated using the flows developed using the DEQ methodology. These results are summarized in ***Table 7***. These values were then compared with the peaking factors calculated from the historical flow data for both the dry season and dry weather flows. These results are summarized in ***Table 8.*** It was determined that the values for the dry weather flow of 1.6-mgd was most representative to determine flow peaking based on rainfall induced infiltration and inflow. The peaking factors based on the flows developed using the DEQ methodology were used. Therefore, the peaking factor for MMDWF will be 3.08 and the peaking factor for MMWWF will be 5.21.

Table 7
Flow Peaking Factors

Method	ADWF	MMDWF		MMWWF	
		Flow	Peaking Factor	Flow	Peaking Factor
Dry Weather	1.60	4.93	3.08	8.34	5.21
Dry Season	2.13	4.93	2.31	8.34	3.92

Table 8
Peaking Factor Summary

Peaking Factor	DEQ Methodology
MMDWF Peaking	3.08
MMWWF Peaking	5.21

4.1.4.3 Max Day Flow Peaking Factor

ODEQ has established a procedure for determining the probable maximum day flow for a wastewater treatment plant[4]. This is done by using the statistical 5-year, 24-hour storm. The storm is used to calculate a flow using a regression from a scatter plot of the existing peak daily flows and rainfalls that occurred during saturated ground conditions. The peak day flow is at the point where the regression crosses the 5-year, 24-hour storm of 3.0-inches.

In this analysis, scatter plots of the daily rainfall vs. daily flow were developed for the period January through May for the periods 2010 – 2021 and 2017 - 2021. The DEQ Methodology suggests using the daily flow data for the months of January through May. A separate scatter plot was developed for when the rainfall was greater than 0.5-in, 1-in, 1.5-in and 2.0-inches for each of the periods.

A linear regression was calculated on each plot and a formula was determined, when possible. This formula was used to determine the peak daily flow for the 5-year, 24-hour storm of 13.41-mgd using rainfall data greater than 0.5-inch. This plot is shown in ***Figure 4***.

The historical daily plant influent flow data was also evaluated to determine if the DEQ methodology was appropriate. A plot of this data is shown in ***Figure 5***. This plot shows that there have been 6-days when the plant daily flow has exceeded 16.5-mgd over the period January 1, 2000, through December 31, 2021. This data is suspect in that the Westside Interceptor has been used for flow equalization over this period. The City took over operations on September 1, 2020, and gradually pulled down the levels in the Westside Interreceptor as they lowered the excessive solids inventory that was in the treatment plant. A storm event on December 20, 2020, produced a daily flow of 16.5-mgd. This was without using the Westside Interceptor for flow equalization and is the best indicator of an actual peak day flow event. Therefore, a peak daily flow of 16.5-mgd will be used. This analysis is provided in ***Attachment D***.

[4] DEQ Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon: MMDWF, MMWWF, PDAF, and PIF, State of Oregon, Department of Environmental Quality Guidelines.

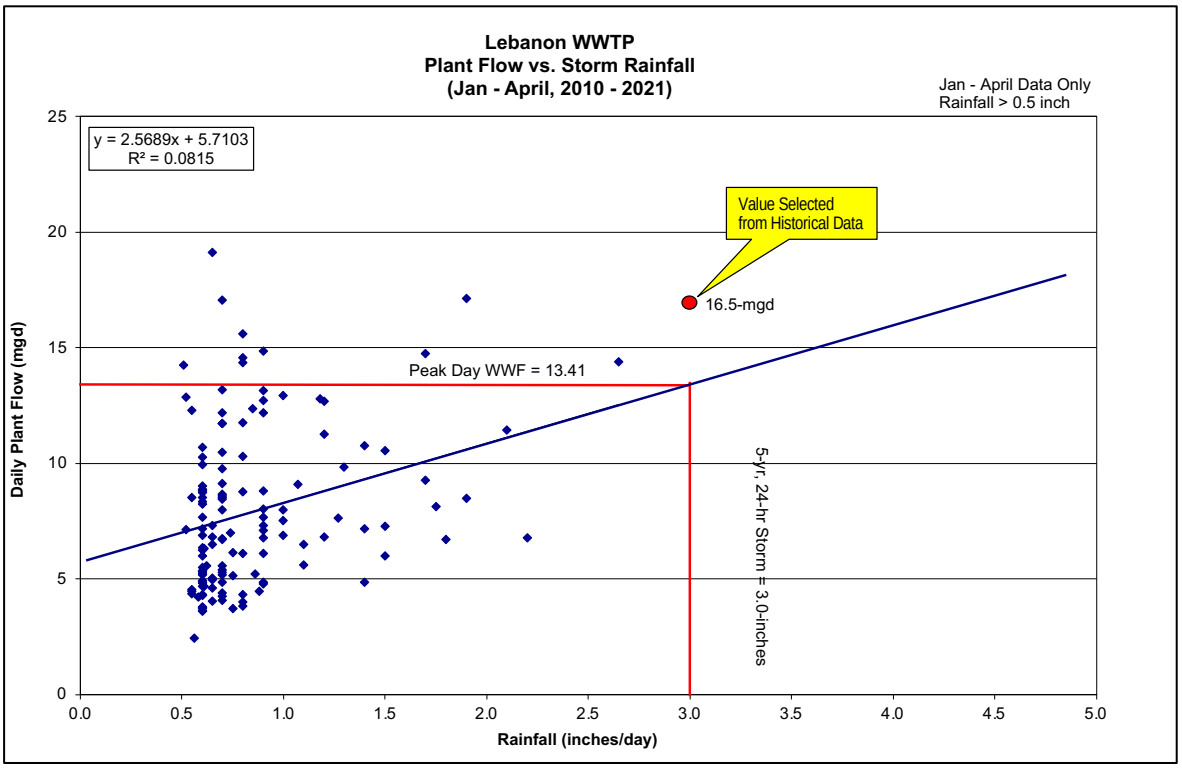

Figure 4: PDF Using DEQ Methodology

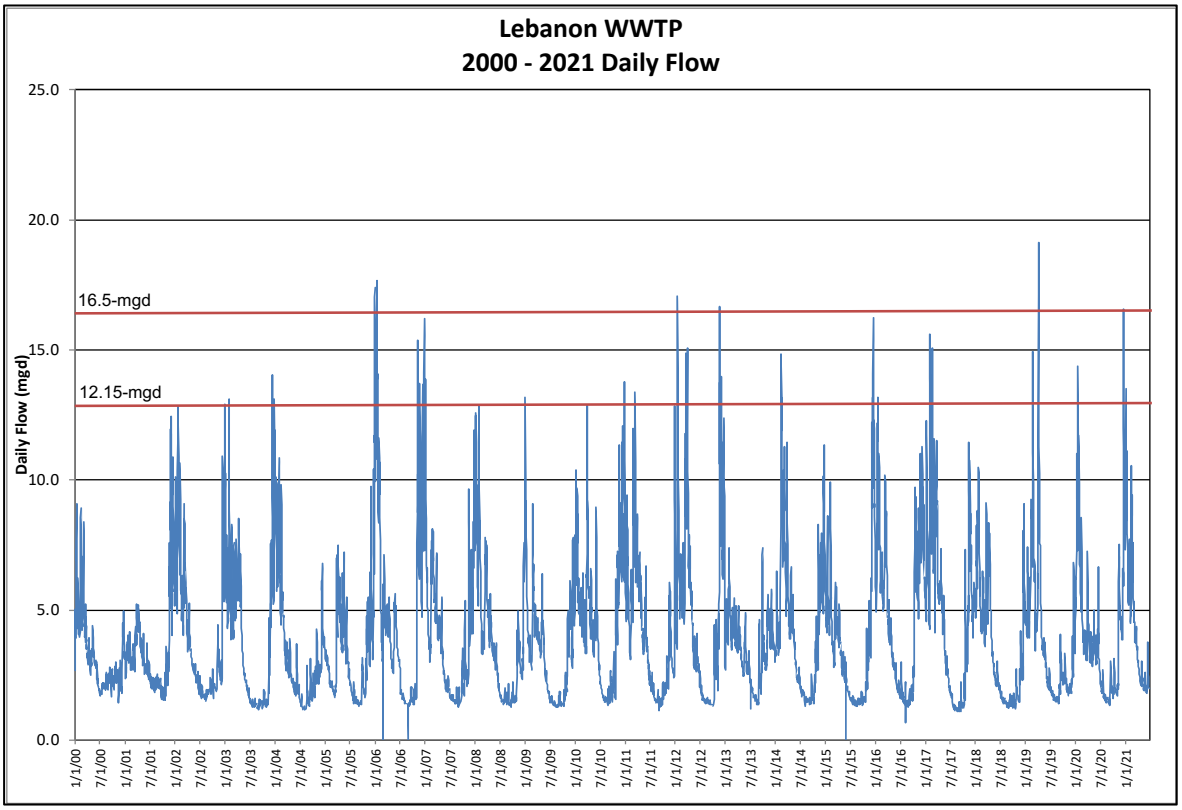

Figure 5: 2000 – 2021 Historical Plant Influent

4.1.4.4 Peak Hour Flow Determination

The PIF_5 is the peak instantaneous or peak hourly flow associated with a 5-year PDAF. That is, the peak flow resulting from a 5-year storm during high groundwater periods. The best method to determine the PIF_5 is by modeling of the collection system. When this information is not available, ODEQ has provided a methodology for estimating PIF_5 using probability of occurrence.

The DEQ methodology requires that flows for average annual (AAF), maximum month wet weather (MMWWF), maximum week wet weather (MWWWF) and peak day ($PDAF_5$) be determined. These were determined and were plotted on probability paper as shown in ***Figure 6***. On this plot, a best fit curve was plotted through the flow points. This provided an estimated peak hour flow (PIF_5) of 26.6-mgd. This analysis also provided a Maximum Week Flow (MWF) of 11.6-mgd.

4.1.4.5 Peaking Factors for Planning

The statistical flows were developed using the DEQ methodology as well as evaluating the historical data from January 2017 to December 2021 in detail. The results of this analysis were discussed in previous sections. These flows were then used to calculate the peaking factors to be used in planning. These flows are based on results of the analysis using the DEQ methodology when available. When not available, the values obtained in the historical analysis were used. The flows that were developed are summarized in ***Table 9***.

The statistical flows were then divided by the average dry weather flow to determine the flow peaking factors to be used for planning. These values are summarized in ***Table 10***. This evaluation is provided in ***Attachment F***.

Table 9
Lebanon WWTP
2021 Statistical Flows

Variable	Average	Max Month	Max Week	Max Day	Peak Hour
Historical Data					
Annual	3.95				
Dry Weather	1.60				
Dry Season	2.13	5.36	5.00	5.19	
Wet Season	5.79	8.83	12.46	16.5	27
DEQ Methodology					
Annual					
Dry Season		4.93			
Wet Season		8.34	11.6	13.4	26.6
Selected 2021 Statistical Flows					
Annual	3.95				
Dry Weather	1.60				
Dry Season	2.13	4.93	5.00	5.19	
Wet Season	5.79	8.34	11.6	16.5	26.6

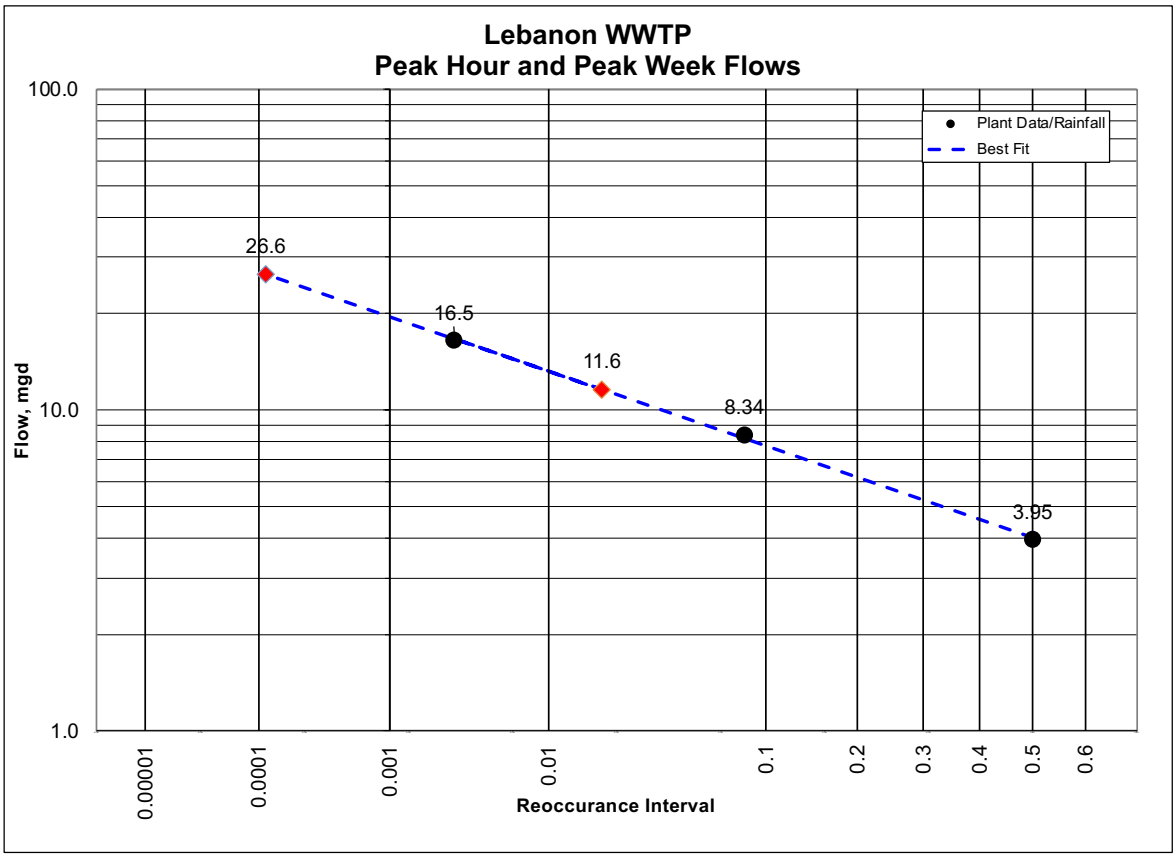

Figure 6: Lebanon WWTP Peak Instantaneous Flow Evaluation

Table 10
Lebanon WWTP
Flow Peaking Factors

Variable	Average	Max Month	Max Week	Max Day	Peak Hour
Annual	2.47				
Dry Weather	1.00				
Dry Season	1.33	3.08	3.12	3.24	
Wet Season	3.62	5.21	7.25	10.31	16.62

4.2 BIOCHEMICAL OXYGEN DEMAND (BOD_5)

The BOD_5 was evaluated to determine the characteristics for projecting future loads. A summary of the analysis is provided in the following sections. The monthly average BOD_5 concentration for the period from January 2017 to December 2021 is shown in ***Figure 7***. This plot shows the BOD_5 concentration becoming more dilute during wet weather periods due to RDII diluting the wastewater.

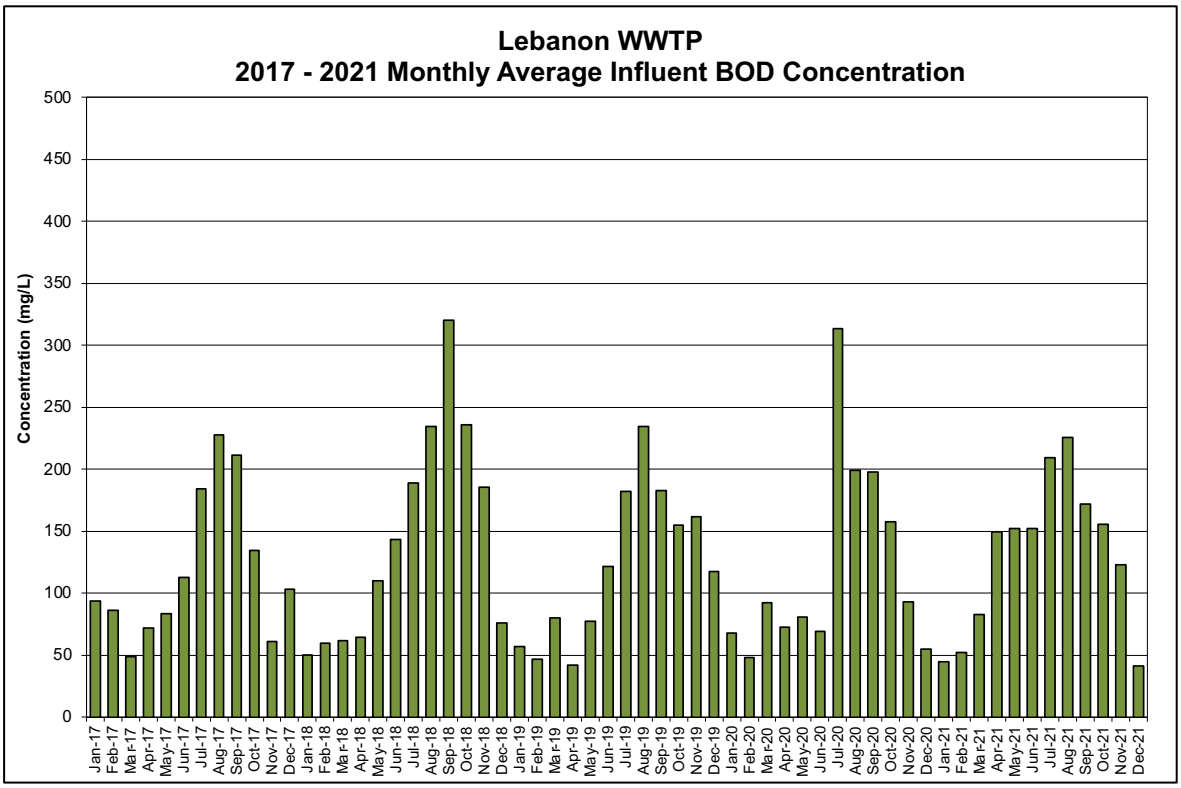

Figure 7: January 2017 to December 2021 Influent BOD_5 Concentration

The monthly average BOD_5 mass load for the period from January 2017 to December 2021 is shown in ***Figure 8***. This shows that the BOD_5 mass increases dramatically during the first high flow occurring in November, December or January. These high loads are most likely due to scouring of solids that settled in the collection system during the low flow months.

4.2.1 BOD_5 Per Capita Load

Per capita loads are calculated by taking the average day load and dividing it by the population for the City of Lebanon for the respective year. The values for each of the years of the study period were then averaged to obtain the per capita load.

BOD_5 per capita loads for municipal systems typically range from 0.18 to 0.22 depending on the demographics of the service area. For instance, a service area with older homes will have a lower per capita load than one with newer homes. This can be attributed to newer homes having garbage disposals. Another factor is percentage of commercial dischargers such as restaurants. This will result in a higher per capita load as they draw customers from outside of the service area that will contribute to the plant loads.

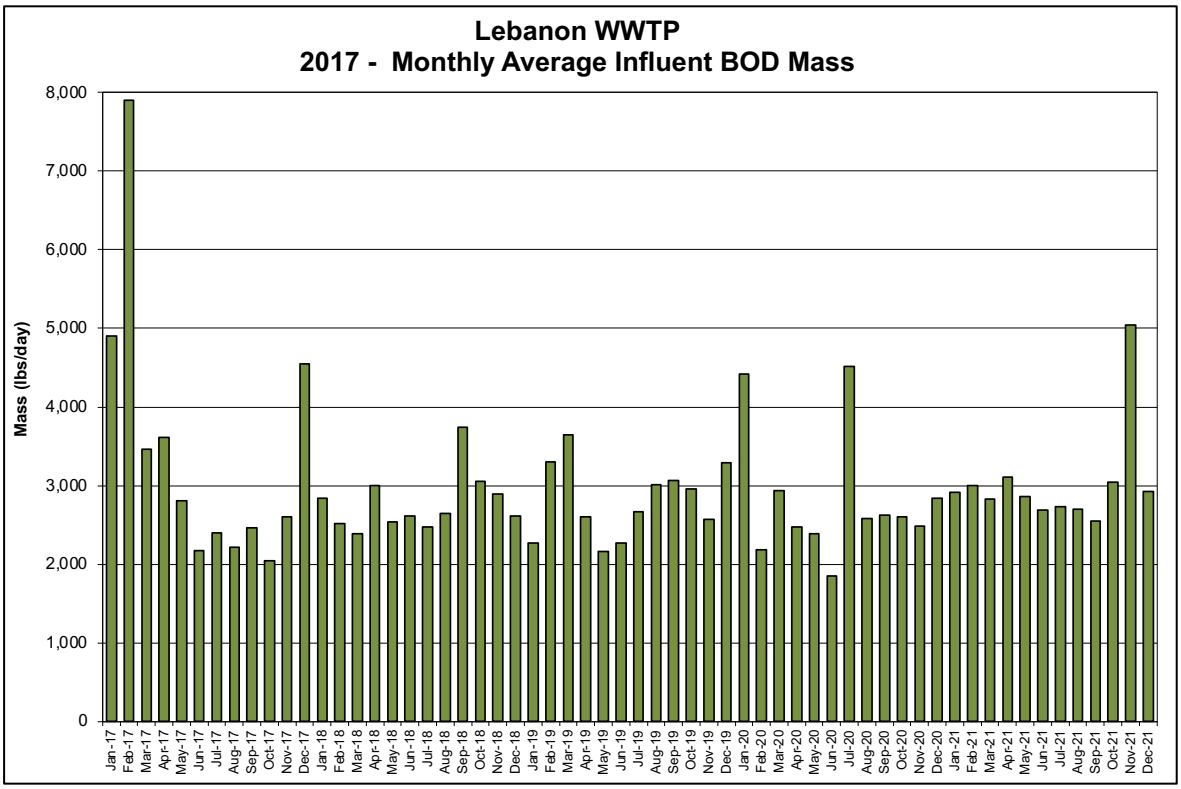

Figure 8: January 2017 to December 2021 Influent BOD_5 Mass Load

The analysis of influent BOD_5 per capita load is provided in ***Attachment G***. The per capita loads developed from the sampling are shown in ***Table 11***. This shows that the per capita loads are reasonable for a city with the demographics of Lebanon.

Table 11
Lebanon WWTP
Influent BOD_5 Characteristics

Period	Per Capita BOD_5 Load (lbs./capita/day)	Peaking Factors		
		Max Month	Max Week	Max Day
Average Annual	0.17	1.6	2.8	3.2
Summer Season	0.15	1.3	1.8	1.9
Winter Season	0.18	1.4	2.4	2.9

4.2.2 BOD_5 Peaking Factors

Peaking factors are necessary to characterize the influent wastewater and project loads into the future. Projections are made assuming that these characteristics remain the same. Peaking factors are determined as follows:

- Max Month -- Maximum 30-day average / Average Day

- Max Week -- Maximum 7-day average / Average Day
- Max Day -- Maximum day / Average Day

In this analysis, the loads for the period between January 2017 and December 2021 were evaluated. The maximum month, maximum week and max day values were developed for each of the seasonal conditions. The BOD_5 peaking factors are shown in ***Table 11***. The analysis of influent BOD_5 peaking factors is provided in ***Attachment G***.

4.3 TOTAL SUSPENDED SOLIDS (TSS)

The TSS was evaluated to determine the characteristics used to project future loads. A summary of the analysis is provided in the following sections. The monthly average TSS concentration for the period from January 2017 and December 2021 is shown in ***Figure 9***. As with the influent BOD_5, the TSS concentration varies during the wet weather periods as it typically does with other treatment plants in Western Oregon.

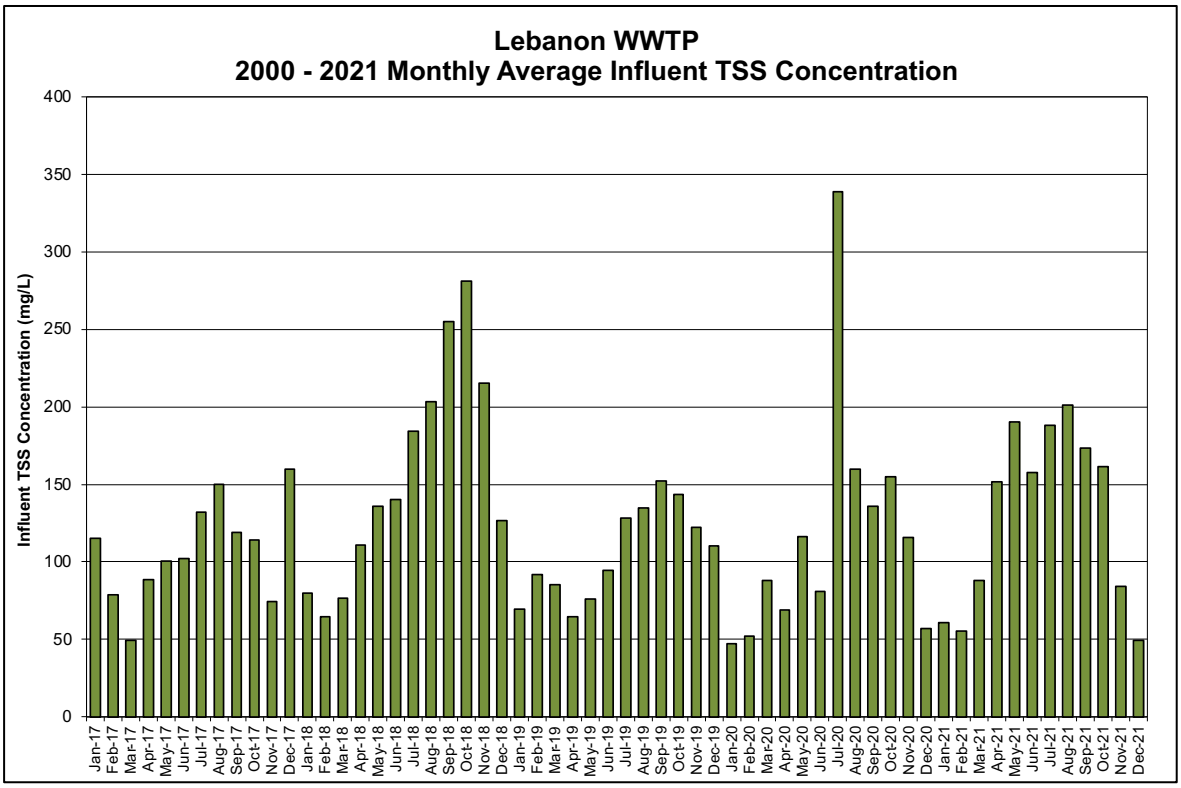

Figure 9: January 2017 to December 2021 Influent TSS Concentration

The monthly average TSS mass load for the period from January 2017 and December 2021 is shown in ***Figure 10***. This figure shows high variability in the TSS load for the years 2017 through 2020. After the influent trunk sewer was no longer used for flow equalization, the influent TSS loads stabilized in 2021.

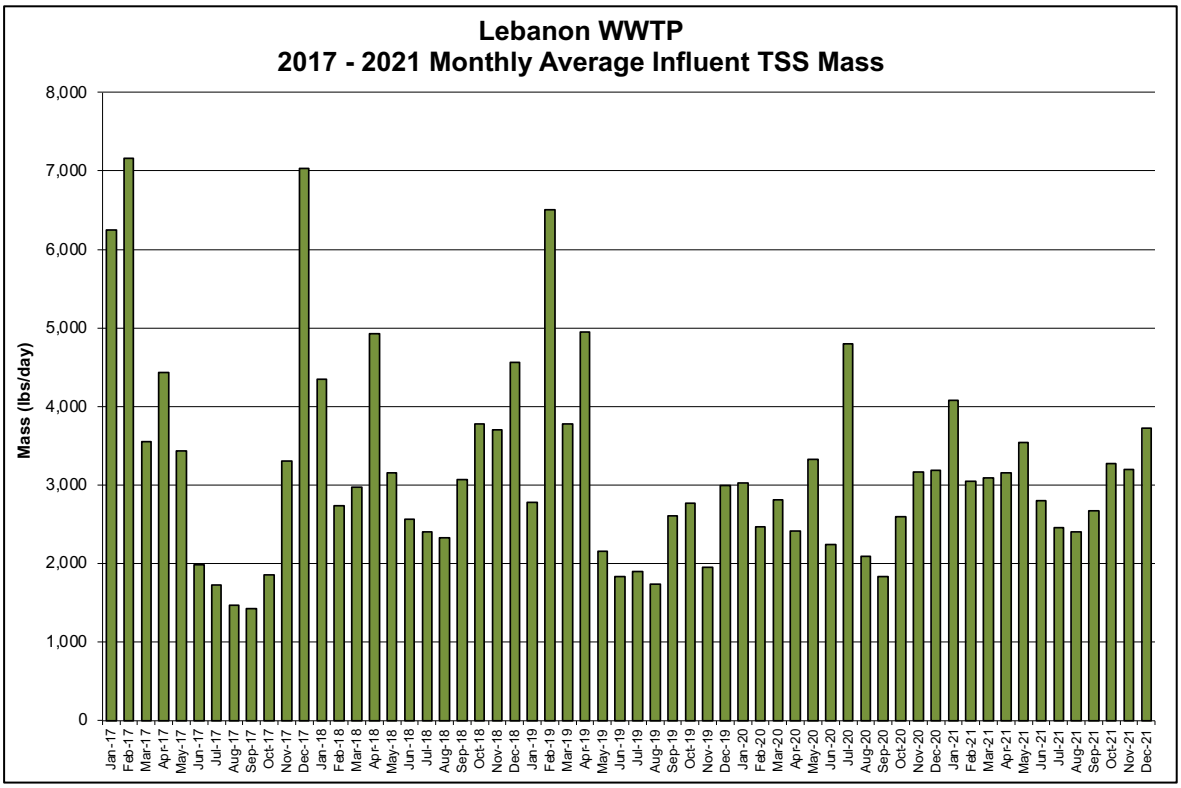

Figure 10: January 2017 to December 2021 Influent TSS Mass Load

4.3.1 TSS Per Capita Load

Per capita TSS loads are calculated by taking the average day load and dividing it by the population for the City of Lebanon for the respective year. The average of the values calculated for each of the years was used as the per capita load.

The analysis of influent TSS per capita load is provided in ***Attachment H***. The per capita loads developed from the sampling are shown in ***Table 12***.

Table 12
Lebanon WWTP
Influent TSS Characteristics

Period	Per Capita BOD_5 Load (lbs./capita/day)	Peaking Factors		
		Max Month	Max Week	Max Day
Average Annual	0.18	1.8	3.0	3.5
Summer Season	0.14	1.8	2.5	2.9
Winter Season	0.21	1.5	2.2	2.8

4.3.2 TSS Peaking Factors

Peaking factors are necessary to characterize the influent wastewater and project loads into the future. Projections are made assuming that these characteristics remain the same. Peaking factors are determined as follows:

- Max Month -- Maximum 30-day average / Average Day
- Max Week -- Maximum 7-day average / Average Day
- Max Day -- Maximum day / Average Day

In this analysis, the loads for the period between January 2017 and December 2021 were evaluated. The maximum month, maximum week and max day values were developed for each of the seasonal conditions. The TSS peaking factors are shown in ***Table 12***. The analysis of influent TSS peaking factors is provided in ***Attachment H***.

4.4 Ammonia (NH3-N)

The influent ammonia load was evaluated to determine the characteristics used to project future loads. A summary of the analysis is provided in the following sections. The monthly average ammonia concentration for the period from January 2017 and June 2021 is shown in ***Figure 11***. As with the influent BOD_5, the ammonia concentration varies during the wet weather periods as it typically does with other treatment plants in Western Oregon. A plot of the influent ammonia mass load is shown in ***Figure 12***. This plot shows a consistent ammonia mass load.

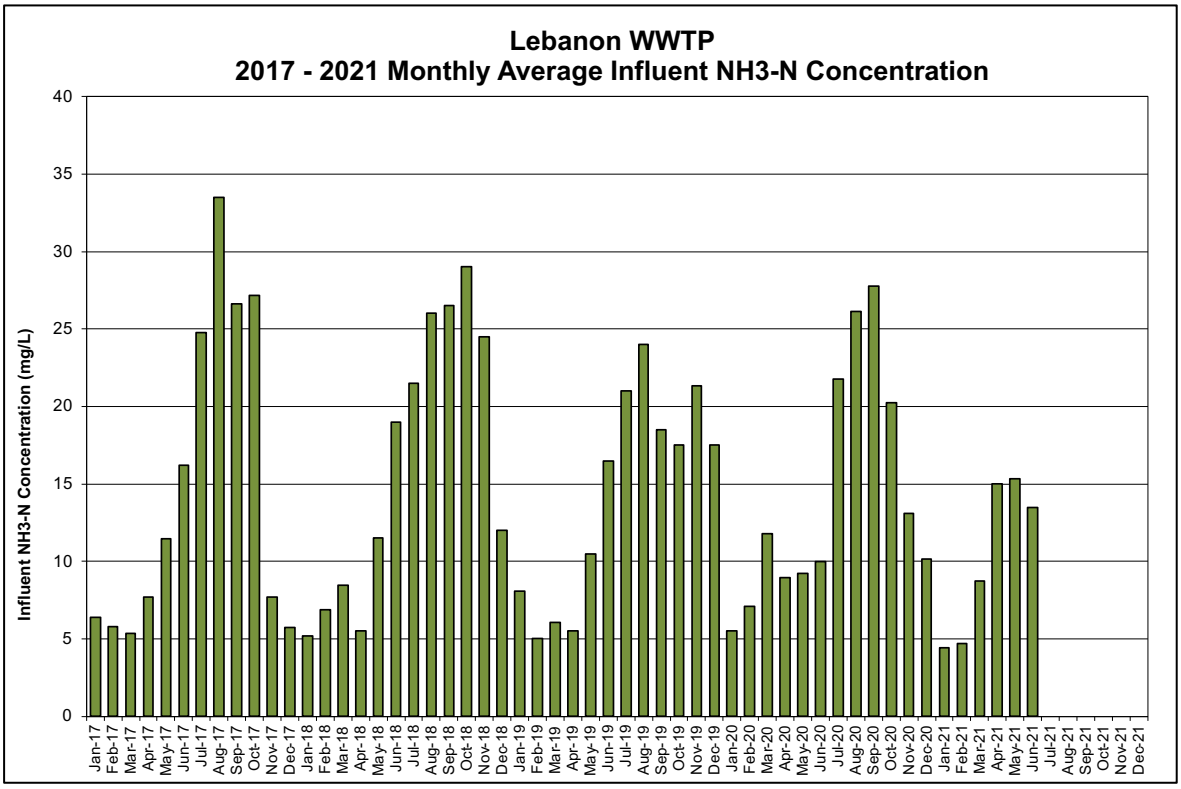

Figure 11: January 2017 – December 2021 Influent Ammonia Concentration

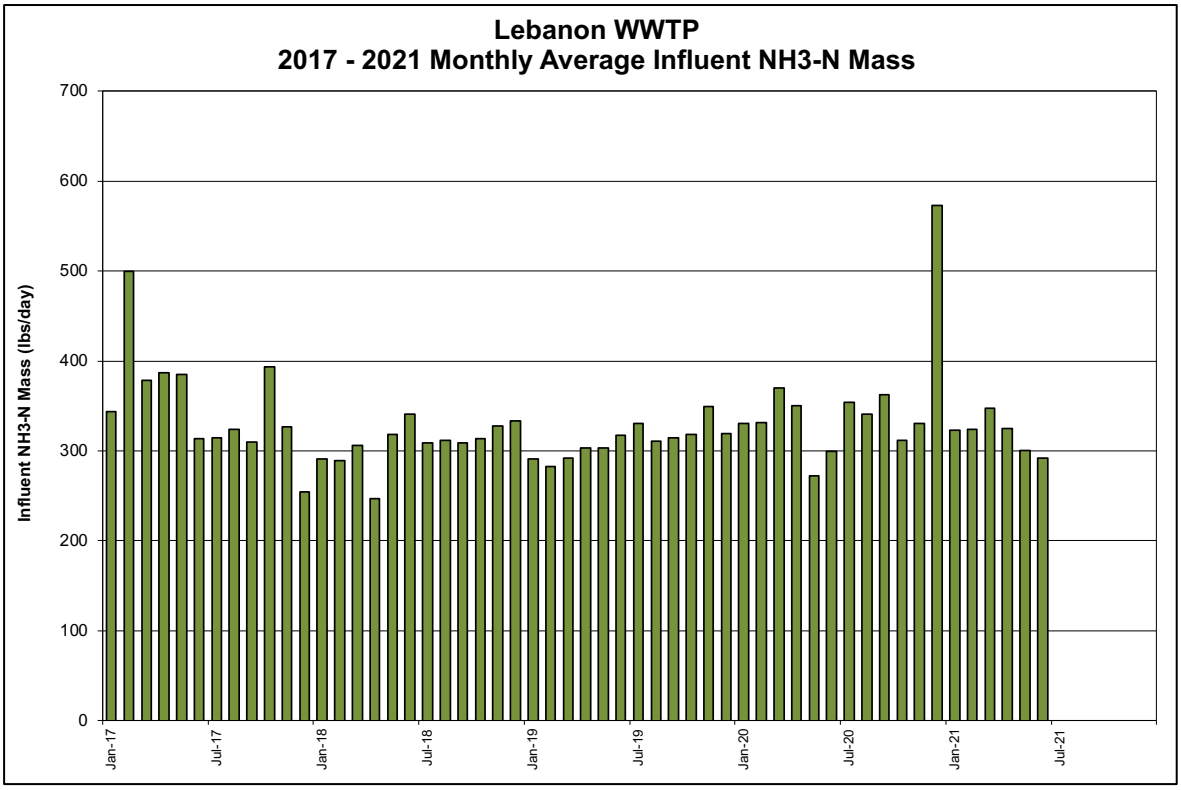

Figure 12: January 2017 – December 2021 Influent Ammonia Mass Load

A statistical evaluation was performed on the influent ammonia data for the period from January 2017 through June 2021. No data was available for the period July 2021 – December 2021. The results of this statistical analysis are summarized in ***Tables 14*** and ***15***. This analysis shows that the average influent ammonia load has ranged from 302 to 353-lbs./day. Using 350-lbs./day and the 2021 average dry weather flow of 1.76-mgd, the estimated influent ammonia concentration can be assumed to be 23.8-mg/L. This is a typical value, so an average ammonia load of 350-lbs./day is a good value to use for planning. This provides an ammonia load of 0.018-lbs./capita/day that can be used for ammonia load projections.

An additional evaluation was done to determine an ammonia to BOD_5 ratio. The analysis compared ammonia to both $CBOD_5$ and $TBOD_5$. The BOD_5 analysis in this technical memorandum was performed on $TBOD_5$. The NH_3:BOD_5 ratios for both $CBOD_5$ and $TBOD_5$ for the years 2017 through 2020 are summarized in ***Table 13***. Based on this analysis, a NH_3:$TBOD_5$ ratio of 0.13 will be used in the development of future flows and loads.

Table 13
Ammonia to BOD_5 Ratios

Year	NH_3:$CBOD_5$			NH_3:$TBOD_5$		
	Avg.	Max.	Min.	Avg.	Max.	Min.
2017	0.17	0.35	0.07	0.13	0.27	0.04
2018	0.18	0.44	0.11	0.12	0.20	0.05
2019	0.16	0.25	0.08	0.13	0.18	0.07
2020	0.17	0.24	0.10	0.13	0.22	0.07

Table 14
Lebanon WWTP
Influent NH3-N Concentration Statistics

Statistic	2017	2018	2019	2020	2021
Max	45.1	29.0	27.0	35.6	19.8
Min	2.8	3.5	3.5	3.3	2.5
Average	15.0	12.3	14.6	14.5	11.2
Median	10.1	8.0	15.0	12.0	11.0
90 Percentile	31.8	27.1	23.6	27.0	16.9
92 Percentile	34.5	27.9	24.0	27.0	16.9
98 Percentile	44.4	29.0	25.6	29.9	19.0
Std. Dev.	11.6	8.6	7.3	8.5	5.3
Count	104	40	25	69	15
Ave. Annual PF	3.0	2.4	1.9	2.5	1.8

Table 15
Lebanon WWTP
Influent NH3-N Mass Statistics

Statistic	2017	2018	2019	2020	2021
Total	36,700	12,065	7,814	24,108	4,715
Max	857	410	376	868	377
Min	179	222	273	232	272
Average	353	302	313	349	314
Median	328	299	315	336	317
90 Percentile	450	344	344	422	357
92 Percentile	490	347	347	430	364
98 Percentile	630	387	363	498	374
Std. Dev.	109	37	26	83	32
Count	104	40	25	69	15
Ave. Annual PF	2.4	1.4	1.2	2.5	1.2

5 STATISTICAL 2021 WASTEWATER CHARACTERISTICS

The statistical 2021 influent wastewater characteristics were developed using a population value of 19,122. The average dry weather influent flow was determined by taking the population for the year 2021 and multiplying it by the dry weather per capita flow of 92-gal/capita/day. Peaking factors were then applied to obtain the dry season and wet season average day flows. The maximum month, maximum week and maximum day flows and loads were then obtained by multiplying the average dry weather flow value by the appropriate peaking factor. The same process was used to calculate the average BOD_5 and TSS loads based on the average per capita load. The results of these calculations are shown in ***Table 18***. Tables for flow and load projections for the years 2021, 2025, 2030, 2035, 2040, 2045 and 2070 (Buildout) are provided in ***Attachment I***. NOTE: The values in ***Table 18*** will be slightly different than the values in ***Table 9*** as they are calculated differently. The values in ***Table 18*** are calculated from per capita flows, 2021 population and calculated peaking factors and the values in ***Table 9*** are based on the analysis of 5-years of flows between 2017 and 2021.

6 2045 PROJECTED WASTEWATER CHARACTERISTICS

The projected influent wastewater characteristics were developed using the projected population for the year 2045 of 22,483. This is 3,361 more people than the current population of 19,122. The same process was used to calculate these flows and loads as was used in the calculation of the 2021 projected wastewater characteristics. The current peaking factors were not used for new growth. It is assumed that new construction will be from materials that will have less RDII. Therefore, the peaking factors will be lower. It is assumed that the City's maintenance program will not allow further degradation of the existing system and the peaking factors for the existing system will remain the same. The unit flows to the system will be lower for future growth due to better construction materials with the BOD_5 and TSS unit flows remaining consistent. The values used for the projection of future flows are summarized in ***Table 16***. The flow peaking factors to be used for new growth are summarized in **Table 17**. The estimated flows and loads for the years 2021 and 2045 are shown in ***Tables 18*** and **19**, respectively.

Table 16
Lebanon WWTP
Future Growth Unit Flows and Loads

	Ave Annual	Dry Weather	Dry Season	Wet Season	Units
Flow	202	92	115	230	gal/capita/day
BOD_5	0.17	-	0.17	0.17	Lbs./capita/day
TSS	0.18	-	0.18	0.18	Lbs./capita/day

Table 17

Lebanon WWTP
Flow Peaking Factors for New Growth

Variable	Average	Max Month	Max Week	Max Day	Peak Hour
Annual	2.20				
Dry Weather	1.00				
Dry Season	1.25	2.00	2.50	3.00	
Wet Season	2.50	3.00	3.50	4.00	6.00

Table 18
Lebanon WWTP
2021 Influent Wastewater Flows and Loads

Flow (mgd)					
Process Parameter	**Ave Day**	**Max. Month**	**Max. Week**	**Max Day**	**Peak Hour**
Average Annual	4.34				
Dry Weather	1.76				
Dry Season	2.33	5.42	5.49	5.70	
Wet Season	6.37	9.17	12.75	18.14	29.2

Biochemical Oxygen Demand				
Process Parameter	**Average (lbs./day)**	**Max. Month (lbs./day)**	**Max. Week (lbs./day)**	**Max Day (lbs.)**
Average Annual	3,251	5,201	9,103	10,403
Dry Season	2,868	3,729	5,163	5,450
Wet Season	3,422	4,819	8,261	9,982

Total Suspended Solids				
Process Parameter	**Average (lbs./day)**	**Max. Month (lbs./day)**	**Max. Week (lbs./day)**	**Max Day (lbs.)**
Average Annual	3,442	6,196	10,326	12,047
Dry Season	2,677	4,819	6.693	7,764
Wet Season	4,016	6,023	8,834	11,244

Table 19
Lebanon WWTP
Projected 2045 Influent Wastewater Flows and Loads

Flow (mgd)					
Process Parameter	**Ave Day**	**Max. Month**	**Max. Week**	**Max Day**	**Peak Hour**
Average Annual	5.02				
Dry Weather	2.07				
Dry Season	2.72	6.04	6.26	6.63	
Wet Season	7.14	10.09	13.84	19.37	31.1

Biochemical Oxygen Demand				
Process Parameter	**Average (lbs./day)**	**Max. Month (lbs./day)**	**Max. Week (lbs./day)**	**Max Day (lbs.)**
Average Annual	3,822	6,115	10,702	12,231
Dry Season	3,372	4,384	6,070	6,408
Wet Season	4,047	5,666	9,713	11,736

Total Suspended Solids				
Process Parameter	**Average (lbs./day)**	**Max. Month (lbs./day)**	**Max. Week (lbs./day)**	**Max Day (lbs.)**
Average Annual	4,047	7,285	12,141	14,164
Dry Season	3,148	5,666	7,869	9,128
Wet Season	4,721	7,082	10,387	13,220

7 ATTACHMENTS

Attachment A – Develop Per Capita Flows
Attachment B – Determine 10-yr Summer & 5-yr Winter Rainfall
Attachment C – Determine 5-yr Winter & 10-yr Summer Flows – DEQ Methodology
Attachment D – Determine Max Day Flow ($PDAF_5$)
Attachment E – Determine Peak Hour Flow (PHF)
Attachment F – Determine Flow Peaking Factors
Attachment G – Determine Peaking Factors and Per Capita Loads for BOD_5
Attachment H – Determine Peaking Factors and Per Capita Loads for TSS
Attachment I – Lebanon WWTP Current Wastewater Flows and Loads

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ATTACHMENT A
Develop Per Capita Flows

Attachment A
Lebanon WWTP
Develop Per Capita Flows

Background

Per capita flows are developed by dividing the average flow by the population. With the high wet weather flows that are experienced, a Dry Weather analysis was also completed to determine actual population based flows.
The Wet Weather season is defined by the NPDES permit as the period from May 1 through October 31. The data that was analyzed had high rainfall induced flows in the months of May and June. Therefore, the data analzed for the low rainfall period was defined as the Dry Weather season and was July 1 through September 30.

The populations used in this analysis are those provided by the Population Research Center of Portland State University.

Evaluation

An evaluation was performed for both the period January 2010 - December 2021 and 2017 - 2021. In addition, the per capital flows for 2021 was also documented. This was done as this was the first complete period when the cannibal process was not being operated and the influent trunk sewer was not being used for flow equalization. Due to the high flows that can be influenced by RDII, an analysis was also done using the median instead of the average to provide results that are not skewed by periodic high flows.

Per Capita Flows (gpd/Capita)

Dry Season Months (May - September)

Period	2010 - 2021		2017 - 2021		2021	
	gpd/capita		gpd/capita		gpd/capita	
	Average	Median	Average	Median	Average	Median
Average Annual	249	211	227	180	207	137
Summer Season	139	115	122	105	102	97
Winter Season	360	312	333	299	312	274

Low Rainfall Months (June - September)

Statistics	*2010 - 2021*		*2017 - 2021*		*2021*	
	gpd/capita		gpd/capita		gpd/capita	
	Ave Month	Min. Month	Ave Month	Min. Month	Ave Month	Min. Month
Monthly Average	117	88	105	81	91	88
Monthly Median	113	87	102	81	83	74
Monthly Minimum	93	74	86	73	78	70

Low Rainfall Months (July - September)

Statistics	*2010 - 2021* gpd/capita		*2017 - 2021* gpd/capita		*2021* gpd/capita	
	Ave Month	Min. Month	Ave Month	Min. Month	Ave Month	Min. Month
Monthly Average	100	88	92	81	83	74
Monthly Median	97	87	89	81	77	74
Monthly Minimum	82	74	77	73	72	70

This analysis provides three different results depending on the summer months that are used. The results are as follows:

Summer Season Months:
When using the Summer Season flow which is the permitted period between May 1 and October 31, the gpd/capita is 122 gpd/capita.

No Rainfall Months (July - September):
The analysis was done taking the rainfall influenced flows for the months of May, June and October out of the data set. In this analysis, the average flow per capita was 92 gpd/capita for the monthly average flows, 89 gpd/capita for the median flows and 77 gpd/capita for the monthly minimum flows.

Discussion

The analysis of dry weather flows shows lower per captia flows for the summer permit season. It is therefore, logical to use the average value for the per capita flows for the period July - September for the existing dry weather flows. Therefore, the Dry Weather Flow per capita flow for this study will be 92 gpd/capita based on the monthly average flow for these months.

Conclusion

This analysis determined a per capita flow using the average flow for the dry weather period between July 1 and September 30 of 92 gpd/capita. This value will be used for the existing population base. This value can also be used for all future growth.

The wastewater average day per capita flows to be used for planning are summarized below.

Variable	Ave Annual	Dry Weather	Summer Season	Winter Season	Units
Flow	227	92	122	333	gal/capita/day

Table A-1
Lebanon WWTP
Influent Flow Characterization

Average Annual

Year	Service Area Population	Historical Average Annual Flows (mgd)				Historical Peaking Factors From Summer Season Flow			Annual Flow per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15,525	4.53	8.69	11.05	13.78	3.21	4.08	5.09	292
2011	15,565	3.86	7.95	9.29	13.36	3.35	3.91	5.63	248
2012	15,660	4.91	10.56	13.11	17.07	4.58	5.69	7.41	313
2013	15,690	3.43	5.17	6.57	7.39	1.97	2.51	2.82	219
2014	15,740	4.28	10.13	13.53	14.83	4.39	5.86	6.42	272
2015	15,740	3.55	10.02	12.91	16.23	5.63	7.26	9.12	225
2016	16,435	4.65	8.75	11.51	13.17	3.48	4.57	5.23	283
2017	16,720	4.66	10.20	12.84	15.60	4.64	5.84	7.10	279
2018	16,920	3.38	7.50	9.08	10.47	4.12	4.98	5.75	200
2019	17,135	3.82	8.80	15.27	19.11	3.85	6.68	8.37	223
2020	17,335	3.92	8.87	11.58	16.55	3.73	4.87	6.96	226
2021	19,122	3.95	8.77	13.54	15.08	4.48	6.92	7.71	207
Average	**16,466**	**4.08**	**8.79**	**11.69**	**14.39**	**3.95**	**5.27**	**6.47**	**249**

Summer Season

Year	Resident Service Area Population	Historical Summer Season Flows (mgd)				Historical Peaking Factors From Average			Flow per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15,525	2.71	5.57	6.94	8.94	1.96	2.56	3.30	174
2011	15,565	2.37	6.05	5.19	6.68	1.93	2.19	2.81	153
2012	15,660	2.30	7.09	5.53	5.72	1.79	2.40	2.48	147
2013	15,690	2.62	4.00	6.18	7.38	2.53	2.36	2.82	167
2014	15,740	2.31	5.11	5.65	5.58	1.89	2.45	2.41	147
2015	15,740	1.78	3.79	3.15	4.29	1.54	1.77	2.41	113
2016	16,435	2.52	5.49	8.06	9.70	1.90	3.20	3.86	153
2017	16,720	2.20	5.85	6.22	7.33	1.67	2.83	3.34	131
2018	16,920	1.82	5.74	4.10	3.76	1.54	2.25	2.06	108
2019	17,135	2.28	8.68	4.47	4.04	2.13	1.96	1.77	133
2020	17,335	2.38	3.85	4.25	6.64	1.96	1.79	2.79	137
2021	19,122	1.96	2.66	3.66	4.17	1.86	1.87	2.13	102
Average	**16,466**	**2.27**	**5.32**	**5.28**	**6.19**	**1.89**	**2.30**	**2.68**	**139**

Winter Season

Year	Resident Service Area Population	Historical Winter Season Flows (mgd)				Historical Peaking Factors From Summer Season Flow			Flow per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15,525	6.38	8.69	11.05	13.78	5.95	4.08	5.09	411
2011	15,565	5.36	7.95	9.29	13.36	5.52	3.91	5.63	344
2012	15,660	7.53	10.56	13.11	17.07	6.70	5.69	7.41	481
2013	15,690	4.26	5.17	6.57	7.39	4.13	2.51	2.82	272
2014	15,740	6.28	10.13	13.53	14.83	5.69	5.86	6.42	399
2015	15,740	5.33	10.02	12.91	16.23	4.43	7.26	9.12	339
2016	16,435	6.82	8.75	11.51	13.17	6.67	4.57	5.23	415
2017	16,720	7.17	10.20	12.84	15.60	6.64	5.84	7.10	429
2018	16,920	4.96	7.50	9.08	10.47	4.87	4.98	5.75	293
2019	17,135	5.37	8.80	15.27	19.11	4.61	6.68	8.37	314
2020	17,335	5.48	8.87	11.58	16.55	4.64	4.87	6.96	316
2021	19,122	5.97	8.77	13.54	15.08	4.48	6.92	7.71	312
Average	**16,466**	**5.91**	**8.79**	**11.69**	**14.39**	**5.36**	**5.27**	**6.47**	**360**

Table A-2
Lebanon WWTP
Influent Flow Characterization

Average Annual

Year	Resident Service Area Population	Historical Average Annual Flows (mgd)				Historical Peaking Factors From Summer Season Flow			Annual Flow per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15,525	4.30	8.69	11.05	13.78	4.44	5.65	7.04	277
2011	15,565	3.26	7.95	9.29	13.36	4.11	4.81	6.91	210
2012	15,660	3.83	10.56	13.11	17.07	5.91	7.33	9.54	245
2013	15,690	3.50	5.17	6.57	7.39	2.04	2.59	2.92	223
2014	15,740	3.65	10.13	13.53	14.83	5.38	7.18	7.87	232
2015	15,740	2.79	10.02	12.91	16.23	6.50	8.38	10.53	178
2016	16,435	4.43	8.75	11.51	13.17	4.60	6.05	6.93	270
2017	16,720	4.38	10.20	12.84	15.60	6.10	7.68	9.32	262
2018	16,920	2.50	7.50	9.08	10.47	4.86	5.88	6.78	147
2019	17,135	2.73	8.80	15.27	19.11	4.12	7.15	8.95	159
2020	17,335	3.38	8.87	11.58	16.55	4.77	6.22	8.89	195
2021	19,122	2.62	8.77	13.54	15.08	4.71	7.28	8.10	137
Average	**16,466**	**3.45**	**8.79**	**11.69**	**14.39**	**4.79**	**6.35**	**7.82**	**211**

Summer Season

Year	Resident Service Area Population	Historical Summer Season Flows (mgd)				Historical Peaking Factors From Average			Flow per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15,525	1.96	5.57	6.94	8.94	2.84	3.55	4.57	126
2011	15,565	1.93	6.05	5.19	6.68	3.13	2.68	3.46	124
2012	15,660	1.79	7.09	5.53	5.72	3.97	3.09	3.20	114
2013	15,690	2.53	4.00	6.18	7.38	1.58	2.44	2.91	162
2014	15,740	1.89	5.11	5.65	5.58	2.71	3.00	2.96	120
2015	15,740	1.54	3.79	3.15	4.29	2.46	2.04	2.78	98
2016	16,435	1.90	5.49	8.06	9.70	2.89	4.24	5.10	116
2017	16,720	1.67	5.85	6.22	7.33	3.50	3.72	4.38	100
2018	16,920	1.54	5.74	4.10	3.76	3.72	2.66	2.43	91
2019	17,135	2.13	8.68	4.47	4.04	4.06	2.09	1.89	125
2020	17,335	1.96	3.85	4.25	6.64	1.97	2.17	3.39	113
2021	19,122	1.86	2.66	3.66	4.17	1.43	1.97	2.24	97
Average	**16,466**	**1.89**	**5.32**	**5.28**	**6.19**	**2.85**	**2.80**	**3.28**	**115**

Winter Season

Year	Resident Service Area Population	Historical Winter Season Flows (mgd)				Historical Peaking Factors From Summer Season Flow			Flow per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15,525	5.95	8.69	11.05	13.78	4.44	5.65	7.04	383
2011	15,565	5.52	7.95	9.29	13.36	4.11	4.81	6.91	354
2012	15,660	6.70	10.56	13.11	17.07	5.91	7.33	9.54	428
2013	15,690	4.13	5.17	6.57	7.39	2.04	2.59	2.92	263
2014	15,740	5.69	10.13	13.53	14.83	5.38	7.18	7.87	361
2015	15,740	4.43	10.02	12.91	16.23	6.50	8.38	10.53	281
2016	16,435	6.67	8.75	11.51	13.17	4.60	6.05	6.93	406
2017	16,720	6.64	10.20	12.84	15.60	6.10	7.68	9.32	397
2018	16,920	4.87	7.50	9.08	10.47	4.86	5.88	6.78	288
2019	17,135	4.61	8.80	15.27	19.11	4.12	7.15	8.95	269
2020	17,335	4.64	8.87	11.58	16.55	4.53	5.92	8.46	268
2021	19,122	5.24	8.77	13.54	15.08	4.71	7.28	8.10	274
Average	**16,466**	**5.42**	**8.79**	**11.87**	**14.27**	**4.76**	**6.46**	**7.76**	**312**

Table A-3
Lebanon WWTP
Dry Weather Flow Evaluation

Monthly Average

Year	Resident Service Area Population	Dry Weather Flows Monthly Average (mgd) June	July	August	Sept	Monthly Average	Min Month	Monthly Average Flow per Capita	Min Month per Capita
2010	15,525	4.49	1.89	1.53	1.71	2.40	1.53	155	98
2011	15,565	3.41	2.07	1.54	1.40	2.11	1.40	135	90
2012	15,660	2.95	1.82	1.47	1.41	1.92	1.41	122	90
2013	15,690	2.80	1.92	1.54	2.78	2.26	1.54	144	98
2014	15,740	2.46	1.85	1.53	1.60	1.86	1.53	118	97
2015	15,740	1.99	1.52	1.37	1.49	1.59	1.37	101	87
2016	16,435	2.09	1.72	1.41	1.59	1.70	1.41	103	86
2017	16,720	2.48	1.55	1.18	1.29	1.62	1.18	97	70
2018	16,920	2.22	1.61	1.37	1.38	1.64	1.37	97	81
2019	17,135	2.31	1.79	1.56	2.28	1.99	1.56	116	91
2020	17,335	3.43	1.93	1.54	1.75	2.16	1.54	125	89
2021	19,122	2.17	1.56	1.42	1.79	1.74	1.42	91	74
Average	**16,466**	**2.73**	**1.77**	**1.45**	**1.71**	**1.92**	**1.44**	**117**	**88**

Monthly Median

Year	Resident Service Area Population	Dry Weather Flows Monthly Median (mgd) June	July	August	Sept	Monthly Average	Min Month	Monthly Average Flow per Capita	Min Month per Capita
2010	15,525	4.14	1.88	1.52	1.61	2.29	1.52	147	98
2011	15,565	3.10	2.09	1.53	1.40	2.03	1.40	130	90
2012	15,660	2.76	1.75	1.48	1.40	1.85	1.40	118	89
2013	15,690	2.73	1.92	1.51	2.17	2.08	1.51	133	96
2014	15,740	2.39	1.83	1.52	1.51	1.81	1.51	115	96
2015	15,740	1.91	1.51	1.36	1.46	1.56	1.36	99	86
2016	16,435	2.05	1.68	1.38	1.58	1.67	1.38	102	84
2017	16,720	2.50	1.62	1.18	1.25	1.64	1.18	98	71
2018	16,920	2.13	1.58	1.38	1.35	1.61	1.35	95	80
2019	17,135	2.27	1.77	1.56	2.28	1.97	1.56	115	91
2020	17,335	3.26	1.84	1.54	1.54	2.05	1.54	118	89
2021	19,122	1.94	1.55	1.42	1.45	1.59	1.42	83	74
Average	**16,466**	**2.60**	**1.75**	**1.45**	**1.58**	**1.85**	**1.43**	**113**	**87**

Monthly Minimum

Year	Resident Service Area Population	Dry Weathr Flows Monthly Minimum (mgd) June	July	August	Sept	Monthly Average	Min Month	Monthly Average Flow per Capita	Min Month per Capita
2010	15,525	2.50	1.51	1.34	1.39	1.69	1.34	109	86
2011	15,565	2.23	1.66	1.38	1.14	1.60	1.14	103	73
2012	15,660	2.01	1.51	1.38	1.34	1.56	1.34	100	86
2013	15,690	2.49	1.19	1.35	1.39	1.61	1.19	102	76
2014	15,740	2.13	1.62	1.30	1.37	1.60	1.30	102	82
2015	15,740	1.65	1.34	1.28	1.37	1.41	1.28	90	81
2016	16,435	1.74	1.43	0.65	1.27	1.27	0.65	77	40
2017	16,720	1.90	1.21	1.08	1.08	1.32	1.08	79	64
2018	16,920	1.92	1.39	1.24	1.23	1.44	1.23	85	72
2019	17,135	1.92	1.53	1.44	1.39	1.57	1.39	92	81
2020	17,335	2.41	1.61	1.43	1.32	1.69	1.32	98	76
2021	19,122	1.78	1.44	1.38	1.34	1.49	1.34	78	70
Average	**16,466**	**2.06**	**1.45**	**1.27**	**1.30**	**1.52**	**1.22**	**93**	**74**

Table A-5
Lebanon WWTP
Dry Weather Flow Evaluation

Monthly Average

Year	Resident Service Area Population	Dry Weather Flows Monthly Average (mgd) June	July	August	Sept	Oct	Monthly Average	Min Month	Monthly Average Flow per Capita	Min Month per Capita
2010	15,525		1.89	1.53	1.71		1.71	1.53	110	98
2011	15,565		2.07	1.54	1.40		1.67	1.40	107	90
2012	15,660		1.82	1.47	1.41		1.57	1.41	100	90
2013	15,690		1.92	1.54	2.78		2.08	1.54	132	98
2014	15,740		1.85	1.53	1.60		1.66	1.53	105	97
2015	15,740		1.52	1.37	1.49		1.46	1.37	93	87
2016	16,435		1.72	1.41	1.59		1.57	1.41	96	86
2017	16,720		1.55	1.18	1.29		1.34	1.18	80	70
2018	16,920		1.61	1.37	1.38		1.45	1.37	86	81
2019	17,135		1.79	1.56	2.28		1.88	1.56	110	91
2020	17,335		1.93	1.54	1.75		1.74	1.54	100	89
2021	19,122		1.56	1.42	1.79		1.59	1.42	83	74
Average	**16,466**		**1.77**	**1.45**	**1.71**		**1.64**	**1.44**	**100**	**88**

Monthly Median

Year	Resident Service Area Population	Dry Weather Flows Monthly Median (mgd) June	July	August	Sept	Oct	Monthly Average	Min Month	Monthly Average Flow per Capita	Min Month per Capita
2010	15,525		1.88	1.52	1.61		1.67	1.52	107	98
2011	15,565		2.09	1.53	1.40		1.67	1.40	107	90
2012	15,660		1.75	1.48	1.40		1.54	1.40	99	89
2013	15,690		1.92	1.51	2.17		1.86	1.51	119	96
2014	15,740		1.83	1.52	1.51		1.62	1.51	103	96
2015	15,740		1.51	1.36	1.46		1.44	1.36	92	86
2016	16,435		1.68	1.38	1.58		1.55	1.38	94	84
2017	16,720		1.62	1.18	1.25		1.35	1.18	81	71
2018	16,920		1.58	1.38	1.35		1.44	1.35	85	80
2019	17,135		1.77	1.56	2.28		1.87	1.56	109	91
2020	17,335		1.84	1.54	1.54		1.64	1.54	95	89
2021	19,122		1.55	1.42	1.45		1.47	1.42	77	74
Average	**16,466**		**1.75**	**1.45**	**1.58**		**1.59**	**1.43**	**97**	**87**

Monthly Minimum

Year	Resident Service Area Population	Dry Weather Flows Monthly Minimum (mgd) June	July	August	Sept	Oct	Monthly Average	Min Month	Monthly Average Flow per Capita	Min Month per Capita
2010	15,525		1.51	1.34	1.39		1.42	1.34	91	86
2011	15,565		1.66	1.38	1.14		1.40	1.14	90	73
2012	15,660		1.51	1.38	1.34		1.41	1.34	90	86
2013	15,690		1.19	1.35	1.39		1.31	1.19	83	76
2014	15,740		1.62	1.30	1.37		1.43	1.30	91	82
2015	15,740		1.34	1.28	1.37		1.33	1.28	85	81
2016	16,435		1.43	0.65	1.27		1.12	0.65	68	40
2017	16,720		1.21	1.08	1.08		1.12	1.08	67	64
2018	16,920		1.39	1.24	1.23		1.28	1.23	76	72
2019	17,135		1.53	1.44	1.39		1.46	1.39	85	81
2020	17,335		1.61	1.43	1.32		1.46	1.32	84	76
2021	19,122		1.44	1.38	1.34		1.39	1.34	72	70
Average	**16,466**		**1.454**	**1.271**	**1.303**		**1.342**	**1.217**	**82**	**74**

Table A-5
Lebanon WWTP
Influent Flow Characterization

Average Annual

Year	Service Area Population	Historical Average Annual Flows (mgd)				Historical Peaking Factors From Summer Season Flow			Annual Flow per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16,720	4.66	10.20	12.84	15.60	4.64	5.84	7.10	279
2018	16,920	3.38	7.50	9.08	10.47	4.12	4.98	5.75	200
2019	17,135	3.82	8.80	15.27	19.11	3.85	6.68	8.37	223
2020	17,335	3.92	8.87	11.58	16.55	3.73	4.87	6.96	226
2021	19,122	3.95	8.77	13.54	15.08	4.48	6.92	7.71	207
Average	**17,446**	**3.95**	**8.83**	**12.46**	**15.36**	**4.16**	**5.86**	**7.18**	**227**

Summer Season

Year	Resident Service Area Population	Historical Summer Season Flows (mgd)				Historical Peaking Factors From Average			Flow per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16,720	2.20	5.85	6.22	7.33	1.67	2.83	3.34	131
2018	16,920	1.82	5.74	4.10	3.76	1.54	2.25	2.06	108
2019	17,135	2.28	8.68	4.47	4.04	2.13	1.96	1.77	133
2020	17,335	2.38	3.85	4.25	6.64	1.96	1.79	2.79	137
2021	19,122	1.96	2.66	3.66	4.17	1.86	1.87	2.13	102
Average	**17,446**	**2.13**	**5.36**	**4.54**	**5.19**	**1.83**	**2.14**	**2.42**	**122**

Winter Season

Year	Resident Service Area Population	Historical Winter Season Flows (mgd)				Historical Peaking Factors From Summer Season Flow			Flow per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16,720	7.17	10.20	12.84	15.60	6.64	5.84	7.10	429
2018	16,920	4.96	7.50	9.08	10.47	4.87	4.98	5.75	293
2019	17,135	5.37	8.80	15.27	19.11	4.61	6.68	8.37	314
2020	17,335	5.48	8.87	11.58	16.55	4.64	4.87	6.96	316
2021	19,122	5.97	8.77	13.54	15.08	4.48	6.92	7.71	312
Average	**17,446**	**5.79**	**8.83**	**12.46**	**15.36**	**5.05**	**5.86**	**7.18**	**333**

Table A-6
Lebanon WWTP
Influent Flow Characterization

Average Annual

Year	Resident Service Area Population	Historical Average Annual Flows (mgd)				Historical Peaking Factors From Summer Season Flow			Annual Flow per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16,720	4.38	10.20	12.84	15.60	6.10	7.68	9.32	262
2018	16,920	2.50	7.50	9.08	10.47	4.86	5.88	6.78	147
2019	17,135	2.73	8.80	15.27	19.11	4.12	7.15	8.95	159
2020	17,335	3.38	8.87	11.58	16.55	4.77	6.22	8.89	195
2021	19,122	2.62	8.77	13.54	15.08	4.71	7.28	8.10	137
Average	**17,446**	**3.12**	**8.83**	**12.46**	**15.36**	**4.91**	**6.84**	**8.41**	**180**

Summer Season

Year	Resident Service Area Population	Historical Summer Season Flows (mgd)				Historical Peaking Factors From Average			Flow per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16,720	1.67	5.85	6.22	7.33	3.50	3.72	4.38	100
2018	16,920	1.54	5.74	4.10	3.76	3.72	2.66	2.43	91
2019	17,135	2.13	8.68	4.47	4.04	4.06	2.09	1.89	125
2020	17,335	1.96	3.85	4.25	6.64	1.97	2.17	3.39	113
2021	19,122	1.86	2.66	3.66	4.17	1.43	1.97	2.24	97
Average	**17,446**	**1.83**	**5.36**	**4.54**	**5.19**	**2.94**	**2.52**	**2.87**	**105**

Winter Season

Year	Resident Service Area Population	Historical Winter Season Flows (mgd)				Historical Peaking Factors From Summer Season Flow			Flow per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16,720	6.64	10.20	12.84	15.60	6.10	7.68	9.32	397
2018	16,920	4.87	7.50	9.08	10.47	4.86	5.88	6.78	288
2019	17,135	4.61	8.80	15.27	19.11	4.12	7.15	8.95	269
2020	17,335	4.64	8.87	11.58	16.55	4.53	5.92	8.46	268
2021	19,122	5.24	8.77	13.54	15.08	4.71	7.28	8.10	274
Average	**17,446**	**5.20**	**8.83**	**12.46**	**15.36**	**4.86**	**6.78**	**8.32**	**299**

Table A-7
Lebanon WWTP
Dry Weather Flow Evaluation

Monthly Average

Year	Resident Service Area Population	Dry Weather Flows Monthly Average (mgd)				Monthly Average	Min Month	Monthly Average Flow per Capita	Min Month per Capita
		June	July	August	Sept				
2017	16,720	2.48	1.55	1.18	1.29	1.62	1.18	97	70
2018	16,920	2.22	1.61	1.37	1.38	1.64	1.37	97	81
2019	17,135	2.31	1.79	1.56	2.28	1.99	1.56	116	91
2020	17,335	3.43	1.93	1.54	1.75	2.16	1.54	125	89
2021	19,122	2.17	1.56	1.42	1.79	1.74	1.42	91	74
Average	**17,446**	**2.52**	**1.69**	**1.42**	**1.70**	**1.83**	**1.42**	**105**	**81**

Monthly Median

Year	Resident Service Area Population	Dry Weather Flows Monthly Median (mgd)				Monthly Average	Min Month	Monthly Average Flow per Capita	Min Month per Capita
		June	July	August	Sept				
2017	16,720	2.50	1.62	1.18	1.25	1.64	1.18	98	71
2018	16,920	2.13	1.58	1.38	1.35	1.61	1.35	95	80
2019	17,135	2.27	1.77	1.56	2.28	1.97	1.56	115	91
2020	17,335	3.26	1.84	1.54	1.54	2.05	1.54	118	89
2021	19,122	1.94	1.55	1.42	1.45	1.59	1.42	83	74
Average	**17,446**	**2.42**	**1.67**	**1.42**	**1.58**	**1.77**	**1.41**	**102**	**81**

Monthly Minimum

Year	Resident Service Area Population	Dry Weathr Flows Monthly Minimum (mgd)				Monthly Average	Min Month	Monthly Average Flow per Capita	Min Month per Capita
		June	July	August	Sept				
2017	16,720	1.90	1.21	1.08	1.08	1.32	1.08	79	64
2018	16,920	1.92	1.39	1.24	1.23	1.44	1.23	85	72
2019	17,135	1.92	1.53	1.44	1.39	1.57	1.39	92	81
2020	17,335	2.41	1.61	1.43	1.32	1.69	1.32	98	76
2021	19,122	1.78	1.44	1.38	1.34	1.49	1.34	78	70
Average	**17,446**	**1.99**	**1.44**	**1.31**	**1.27**	**1.50**	**1.27**	**86**	**73**

Table A-8
Lebanon WWTP
Dry Weather Flow Evaluation

Monthly Average

Year	Resident Service Area Population	Dry Weather Flows Monthly Average (mgd)					Monthly Average	Min Month	Monthly Average Flow per Capita	Min Month per Capita
		June	July	August	Sept	Oct				
2017	16,720		1.55	1.18	1.29		1.34	1.18	80	70
2018	16,920		1.61	1.37	1.38		1.45	1.37	86	81
2019	17,135		1.79	1.56	2.28		1.88	1.56	110	91
2020	17,335		1.93	1.54	1.75		1.74	1.54	100	89
2021	19,122		1.56	1.42	1.79		1.59	1.42	83	74
Average	**17,446**		**1.69**	**1.42**	**1.70**		**1.60**	**1.42**	**92**	**81**

Monthly Median

Year	Resident Service Area Population	Dry Weather Flows Monthly Median (mgd)					Monthly Average	Min Month	Monthly Average Flow per Capita	Min Month per Capita
		June	July	August	Sept	Oct				
2017	16,720		1.62	1.18	1.25		1.35	1.18	81	71
2018	16,920		1.58	1.38	1.35		1.44	1.35	85	80
2019	17,135		1.77	1.56	2.28		1.87	1.56	109	91
2020	17,335		1.84	1.54	1.54		1.64	1.54	95	89
2021	19,122		1.55	1.42	1.45		1.47	1.42	77	74
Average	**17,446**		**1.67**	**1.42**	**1.58**		**1.56**	**1.41**	**89**	**81**

Monthly Minimum

Year	Resident Service Area Population	Dry Weather Flows Monthly Minimum (mgd)					Monthly Average	Min Month	Monthly Average Flow per Capita	Min Month per Capita
		June	July	August	Sept	Oct				
2017	16,720		1.21	1.08	1.08		1.12	1.08	67	64
2018	16,920		1.39	1.24	1.23		1.28	1.23	76	72
2019	17,135		1.53	1.44	1.39		1.46	1.39	85	81
2020	17,335		1.61	1.43	1.32		1.46	1.32	84	76
2021	19,122		1.44	1.38	1.34		1.39	1.34	72	70
Average	**17,446**		**1.435**	**1.314**	**1.273**		**1.341**	**1.271**	**77**	**73**

ATTACHMENT B
Determine 10-yr Summer & 5-yr Winter Rainfall

Attachment B
Lebanon WWTP
Determine 10-yr Summer & 5-yr Winter Rainfall

Background

The ODEQ criteria for determining the Maximum Month Wet Weather and Maximum Month Dry Weather rainfalls is to use the Climatography No. 20 for the local weather station. The closest weather stations with a table in Climatography No. 20 are the Oregon State University station in Corvallis, OR, and the Stayton, OR and McKenzie Bridge weather stations. The Climatography No. 20 data is based on rainfall for the period 1971 - 2001. An evaluation was done of the rainfall for the period January 2000 - December 2021 to determine if the Climatography No. 20 data for the neighborin weather stations is representative of the rainfall that occured during the period that is being evaluated.

The Maximum Monthly Average Dry-Weather Flow would be the monthly average flow in the rainiest summer month of high groundwater. West of the Oregon Cascades, the MMDWF almost invariably occurs in May.
The Maximum Monthly Average Wet-Weather Flow represents the highest monthly average attained during the winter period of high groundwater. West of the Cascades, high groundwater is usually not attained until January, and the MMWWF (maximum monthly wet-weather flow) occurs in January. Sometimes the period of October-December produces significant storms, but the ground is still dry. Heavy storms generally do not begin to cause a reliable or consistent infiltration response until January.

Evaluation

These statistics were developed to determine the 10-year summer and 5-year winter storms for the Lebanon service area, as specified in the DEQ Guidelines for Making Wet-Weather and Peak flow Projections for Sewage Treatment in Western Oregon: MMDWF, MMWWF, PDAF, and PIF.

Determine Statistical 5-year Winter and 10-year Summer Monthly Rainfalls

The 10-year summer rainfall is the rainfall with the 90% probability of occurance. The shoulder months of the dry weather season are May and October. The 90% probability values for those months were taken from the probability table that was developed for the period January 2000 - December 2021 and the Climatrography 20 data set for the Oregon State University, Stayton, OR and McKenzie Bridge Ranger Station weather stations. These are provided in the following table.

The 10-year summer season monthly rainfall is the rainfall with the 90% probability of occurance. The data from the monthly 90% probability with the highest values were May and October with October being the highest. These values were also taken from the probability table and the three nearby weather stations Climatography 20 data tables. These values are provided in the following table.

The 5-year winter monthly rainfall is the rainfall with the 80% probability of occurance. The data from the monthly 80% probability with the highest values were December and January with December being the highest. These values were also taken from the probability table and the three nearby weather stations Climatography 20 data table. These values are provided in the following table.

Station	Statistical Rainfall Events				Comment
	10-Yr Summer		5-Yr Winter		
	May	October	December	January	
2010 - 2021 WWTP Rainfall	3.68	6.25	9.02	9.14	From Table B-1
Oregon State Univ Station	4.12	5.93	10.46	9.46	From Table B-2
Stayton, OR Station	5.99	8.11	11.01	10.15	From Table B-3
McKenzie Bridge RS	7.34	10.13	15.40	13.14	From Table B-4

The statistical data for the existing WWTP data for the period 2000 - 2021 has similar statistical rainfalls to the Oregon State Universy Station but lower values than the Stayton, OR and McKenzie Bridge Ranger Station weather statiions. The Stayton, OR and McKenzie Bridge Ranger Stations are located closer to the Cascade Range which is most likley the cause for the higher rainfalls. The Treatment Plant rainfall guage is a small tube that is located next to the administration building. This tube is read manually every day by the operators. The reliability of this rainfall guage is suspect. Therefore, the Oregon State University station appears to be the most representative of rainfall in the area.

Conclusion

Summer Season 10-year Statistical Rainfall

In this analysis, The statistical analysis for the Oregon State University weather station provided in the Climatography No. 20 will be used. This Climatography No. 20 data was used as it is more conservative and the accuracy of the rainfall guage for the treatment plant is suspect. The guage is a manual guage is a tube located in front of the administration buliding. Its proximity to the administration building may affect its accuracy. This data as well as the Climatography data for each of the weather stations show that October is the highest rainfall summer permit season month. October plant flows have periodically been higher than May plant flows. The DEQ recommendation is to use the May rainfall data as this is when the water table will be the highest for the summer season. Therefore the May rainfall data will be used for determining the dry season maximum month flow.

Winter Season 5-year Statistical Rainfall

The statistical analysis for the Oregon State University weather station provided in Climatography No. 20 will be used. The highest statistical rainfall occurs in December, but the DEQ recommendation is to use the January rainfall as the water table will be highest during that period. Therefor the January statistical rainfall data will be used for determinign the wet season maximum month flow.

MMDWF

An analysis of rainfall shows that May has the probability of a 10-year summer monthly rainfall with a value of 4.12-inches.

MMWWF

An analysis of rainfall shows that January has the highest probability of a 5-year winter monthly rainfall with a value of 9.46-inches.

Table B-1
Lebanon WWTP
Janaury 2000 - December 2021 Rainfall Data Probabilties
PROBABILITY THAT THE MONTHLY PRECIPITATION WILL BE EQUAL TO OR LESS THAN THE INDICATED PRECIPITATION AMOUNT
MONTHLY PRECIPITATION (INCHES)

Probability Level	JAN	FEB	MAR	APR	MAY	JUN	SEP	OCT	NOV	DEC
0.05	2.50	1.94	2.34	1.72	0.61	0.56	0.55	1.52	2.22	2.75
0.1	2.50	2.02	2.83	2.01	0.92	0.64	0.61	1.93	3.19	3.64
0.2	2.94	2.80	3.18	2.65	1.14	0.82	0.77	2.26	4.44	5.01
0.3	3.69	3.24	3.52	2.75	1.49	1.46	1.16	2.53	4.73	5.83
0.4	4.64	3.67	4.17	3.26	1.85	1.60	1.44	3.24	5.55	6.38
0.5	5.47	4.08	4.35	3.51	2.36	1.66	1.54	3.65	5.98	6.76
0.6	8.08	4.49	5.70	3.96	2.86	1.72	1.73	3.92	6.43	7.63
0.7	8.63	4.89	6.59	4.45	3.29	1.91	2.05	4.28	7.15	8.46
0.8	9.14	5.44	6.84	4.81	3.51	2.43	2.43	5.61	8.16	9.02
0.9	9.30	6.74	8.04	5.41	3.68	2.54	3.32	6.25	9.19	11.96
0.95	9.45	8.61	8.37	6.85	3.68	2.80	4.11	7.07	12.29	13.90

NOTES:
1) This table not available in the form of Climatography of the United States No. 20 for this station.
2) This table was developed by taking the respective percentiles for total monthly rainfall data for the years 2010-202
3) These statistics are from rainfall data taken at the WWTP

Table B-2
Corvallis State Univ, OR Climatography No. 20 1971 - 2000

U.S. Department of Commerce
National Oceanic & Atmospheric Administration
National Environmental Satellite, Data,
and Information Service

Climatography of the United States No. 20 1971-2000

National Climatic Data Center
Federal Building
151 Patton Avenue
Asheville, North Carolina 28801
www.ncdc.noaa.gov

Station: CORVALLIS STATE UNIV, OR **COOP ID: 351862**

Climate Division: OR 2 **NWS Call Sign:** **Elevation: 225 Feet** **Lat: 44°38N** **Lon: 123°11W**

Precipitation (inches)

	Precipitation Totals									Mean Number of Days (3)				Precipitation Probabilities (1) Probability that the monthly/annual precipitation will be equal to or less than the indicated amount									
	Means/ Medians(1)		Extremes							Daily Precipitation				Monthly/Annual Precipitation vs Probability Levels These values were determined from the incomplete gamma distribution									
Month	Mean	Median	Highest Daily(2)	Year	Day	Highest Monthly(1)	Year	Lowest Monthly(1)	Year	>= 0.01	>= 0.10	>= 0.50	>= 1.00	.05	.10	.20	.30	.40	.50	.60	.70	.80	.90	.95
Jan	6.46	6.80	4.28	1965	28	11.59	1974	.25	1985	19.7	12.6	4.4	1.4	1.42	2.01	2.96	3.82	4.67	5.58	6.61	7.84	9.46	12.03	14.47
Feb	5.71	5.24	3.26	1996	6	13.63	1996	1.65	1973	18.0	12.0	4.2	1.0	1.77	2.31	3.12	3.81	4.48	5.17	5.94	6.83	7.99	9.79	11.46
Mar	4.59	4.24	2.07	1998	22	8.87	1974	1.04	1992	18.8	12.0	2.6	.4	1.91	2.33	2.92	3.40	3.85	4.31	4.81	5.38	6.10	7.20	8.20
Apr	2.98	2.67	1.83	1971	9	6.82	1993	1.02	1977	16.3	8.8	1.4	.2	1.00	1.28	1.69	2.04	2.38	2.73	3.11	3.55	4.12	5.00	5.82
May	2.30	2.09	1.58	1963	6	5.80	1998	.00	1992	12.7	6.5	1.2	.1	.46	.78	1.17	1.48	1.78	2.08	2.42	2.81	3.32	4.12	4.86
Jun	1.46	1.30	2.14	1952	29	4.34	1984	.29	1987	7.8	4.4	.6	.1	.36	.50	.71	.90	1.09	1.28	1.50	1.77	2.11	2.65	3.16
Jul	.57	.38	1.26	1987	19	2.55	1983	.01	1994	3.6	1.5	.3	@	.01	.03	.09	.16	.24	.35	.49	.67	.93	1.40	1.87
Aug	.73	.40	1.48	1983	29	2.67	1979	.00+	1998	3.9	2.1	.4	.1	.00	.00	.00	.07	.18	.33	.54	.83	1.26	2.02	2.80
Sep	1.47	.89	2.18	1969	18	3.58	1977	.00	1975	7.1	3.6	.9	.2	.02	.10	.27	.47	.71	.99	1.33	1.78	2.40	3.48	4.57
Oct	3.02	2.86	2.00	1995	11	7.21	1979	.14	1988	12.4	7.2	2.0	.3	.51	.77	1.21	1.63	2.05	2.52	3.04	3.69	4.54	5.93	7.25
Nov	6.94	5.90	4.45	1996	19	18.28	1973	1.03	1993	20.1	13.7	5.0	1.3	1.89	2.55	3.55	4.43	5.29	6.19	7.19	8.37	9.90	12.32	14.57
Dec	7.43	6.91	3.43	1998	28	17.11	1996	1.47	1976	20.8	12.9	5.5	1.8	2.23	2.93	3.99	4.90	5.79	6.71	7.72	8.91	10.46	12.87	15.10
Ann	43.66	42.10	4.45	Nov 1996	19	18.28	Nov 1973	.00+	Aug 1998	161.2	97.3	28.5	6.9	28.35	31.21	34.93	37.79	40.36	42.86	45.46	48.37	51.91	57.11	61.65

\+ Also occurred on an earlier date(s)
\# Denotes amounts of a trace
@ Denotes mean number of days greater than 0 but less than .05
** Statistics not computed because less than six years out of thirty had measurable precipitation

(1) From the 1971-2000 Monthly Normals
(2) Derived from station's available digital record: 1948-2001
(3) Derived from 1971-2000 serially complete daily data

Complete documentation available from:
www.ncdc.noaa.gov/oa/climate/normals/usnormals.html

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Table B-3
Stayton, OR Climatotography No. 20 1971 - 2000

U.S. Department of Commerce
National Oceanic & Atmospheric Administration
National Environmental Satellite, Data,
and Information Service

Climatography of the United States No. 20 1971-2000

National Climatic Data Center
Federal Building
151 Patton Avenue
Asheville, North Carolina 28801
www.ncdc.noaa.gov

Station: STAYTON, OR **COOP ID: 358095**

Climate Division: OR 2 **NWS Call Sign:** **Elevation: 425 Feet** **Lat: 44°47N** **Lon: 122°49W**

Precipitation (inches)

	Precipitation Totals									Mean Number of Days (3)				Precipitation Probabilities (1) Probability that the monthly/annual precipitation will be equal to or less than the indicated amount										
	Means/ Medians(1)		Extremes							Daily Precipitation				Monthly/Annual Precipitation vs Probability Levels These values were determined from the incomplete gamma distribution										
Month	Mean	Median	Highest Daily(2)	Year	Day	Highest Monthly(1)	Year	Lowest Monthly(1)	Year	>= 0.01	>= 0.10	>= 0.50	>= 1.00	.05	.10	.20	.30	.40	.50	.60	.70	.80	.90	.95
Jan	7.17	7.88	3.50	1974	15	12.97	1996	.48	1985	20.4	13.9	4.9	1.5	2.07	2.75	3.77	4.66	5.53	6.44	7.44	8.62	10.15	12.55	14.78
Feb	6.46	5.86	3.24	1996	7	14.19	1996	2.18	1973	18.4	13.0	4.5	1.2	2.52	3.12	3.97	4.68	5.35	6.03	6.76	7.62	8.70	10.36	11.88
Mar	5.37	4.99	2.10	1966	9	9.40	1974	2.06	1978	18.8	13.7	3.3	.6	2.61	3.07	3.70	4.20	4.67	5.14	5.64	6.20	6.91	7.98	8.94
Apr	4.26	4.01	1.99	1971	9	10.48	1993	1.70	1973	16.6	11.3	2.7	.3	1.62	2.01	2.58	3.05	3.50	3.96	4.46	5.03	5.76	6.89	7.92
May	3.31	3.07	1.85	1991	17	6.80	1998	.13	1992	13.9	8.6	1.8	.3	.84	1.15	1.63	2.06	2.48	2.92	3.42	4.00	4.77	5.99	7.12
Jun	2.42	2.02	2.10	1969	23	5.46	1984	.94	1972	9.5	6.0	1.5	.2	.79	1.02	1.36	1.64	1.92	2.21	2.52	2.89	3.36	4.09	4.76
Jul	.87	.47	1.74	1987	18	3.62	1983	.00	1973	4.7	2.5	.3	.1	.02	.07	.17	.29	.43	.59	.79	1.05	1.41	2.02	2.63
Aug	1.15	.94	1.68	1995	11	4.09	1978	.00+	1994	5.2	2.6	.7	.2	.00	.04	.19	.35	.54	.77	1.04	1.40	1.90	2.75	3.60
Sep	2.18	1.90	1.91	1996	15	6.07	1977	.03+	1999	7.9	4.8	1.5	.4	.07	.16	.38	.66	.99	1.40	1.91	2.58	3.55	5.24	6.95
Oct	4.03	3.89	4.47	1955	9	8.26	1979	.13	1987	13.0	8.6	2.7	.7	.58	.92	1.50	2.06	2.64	3.28	4.01	4.92	6.13	8.11	10.02
Nov	8.16	8.18	4.05	1994	1	17.62	1973	1.68	1993	20.7	14.9	6.1	1.9	2.94	3.71	4.81	5.74	6.63	7.54	8.53	9.68	11.15	13.41	15.49
Dec	8.00	8.00	3.50	1987	3	17.83	1996	2.46	1976	21.4	14.3	5.6	1.9	2.76	3.52	4.62	5.55	6.44	7.35	8.35	9.52	11.01	13.32	15.44
Ann	53.38	52.99	4.47	Oct 1955	9	17.83	Dec 1996	.00+	Aug 1994	170.5	114.2	35.6	9.3	37.57	40.62	44.53	47.50	50.14	52.70	55.34	58.27	61.82	66.97	71.43

\+ Also occurred on an earlier date(s)
\# Denotes amounts of a trace
@ Denotes mean number of days greater than 0 but less than .05
** Statistics not computed because less than six years out of thirty had measurable precipitation

(1) From the 1971-2000 Monthly Normals
(2) Derived from station's available digital record: 1951-2001
(3) Derived from 1971-2000 serially complete daily data

Complete documentation available from:
www.ncdc.noaa.gov/oa/climate/normals/usnormals.html

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Table B-3

McKenzie Bridge RS, OR Climatotography No. 20 1971 - 2000

U.S. Department of Commerce
National Oceanic & Atmospheric Administration
National Environmental Satellite, Data,
and Information Service

Climatography of the United States No. 20 1971-2000

National Climatic Data Center
Federal Building
151 Patton Avenue
Asheville, North Carolina 28801
www.ncdc.noaa.gov

Station: MC KENZIE BRIDGE R S, OR — **COOP ID: 355362**

Climate Division: OR 4 — **NWS Call Sign:** — **Elevation: 1,478 Feet** — **Lat: 44°11N** — **Lon: 122°07W**

Precipitation (inches)

	Precipitation Totals									Mean Number of Days (3)				Precipitation Probabilities (1) Probability that the monthly/annual precipitation will be equal to or less than the indicated amount										
	Means/ Medians(1)		Extremes							Daily Precipitation				Monthly/Annual Precipitation vs Probability Levels These values were determined from the incomplete gamma distribution										
Month	Mean	Median	Highest Daily(2)	Year	Day	Highest Monthly(1)	Year	Lowest Monthly(1)	Year	>= 0.01	>= 0.10	>= 0.50	>= 1.00	.05	.10	.20	.30	.40	.50	.60	.70	.80	.90	.95
Jan	9.24	9.43	6.58	1999	21	16.45	1972	.86	1985	13.9	11.0	4.9	2.4	2.57	3.45	4.78	5.94	7.07	8.26	9.57	11.13	13.14	16.31	19.26
Feb	7.99	7.41	5.28	1984	13	15.08	1999	1.91	1973	13.0	10.8	4.1	1.8	2.75	3.51	4.61	5.54	6.42	7.34	8.34	9.50	11.00	13.30	15.43
Mar	7.11	6.54	3.07	2000	20	12.37	1989	1.75	1992	15.4	11.9	3.7	1.1	2.72	3.38	4.33	5.11	5.86	6.62	7.44	8.39	9.60	11.46	13.16
Apr	5.75	5.55	3.38	1990	27	11.32	1996	2.29	1987	13.0	9.9	3.5	1.1	2.36	2.89	3.63	4.24	4.82	5.40	6.03	6.75	7.67	9.07	10.34
May	4.13	3.60	4.44	1999	12	7.92	1977	.40	1992	9.8	6.7	2.3	.6	1.12	1.51	2.11	2.63	3.15	3.68	4.28	4.98	5.90	7.34	8.69
Jun	2.85	2.91	2.20	1981	8	6.76	1984	.55	1977	7.5	5.4	1.8	.5	.77	1.04	1.45	1.81	2.17	2.54	2.95	3.43	4.07	5.06	5.99
Jul	1.00	.73	2.69	1987	18	4.37	1987	.01	1972	4.0	2.2	.4	.1	.04	.09	.19	.32	.48	.66	.89	1.20	1.62	2.37	3.12
Aug	1.42	.74	1.65	1999	7	5.03	1976	.00	1988	4.6	3.0	1.0	.3	.00	.03	.13	.28	.48	.75	1.11	1.61	2.36	3.71	5.11
Sep	2.71	2.40	2.81	1963	16	8.58	1986	.00+	1993	6.1	4.5	1.7	.5	.00	.00	.45	.89	1.36	1.90	2.54	3.36	4.48	6.38	8.26
Oct	4.97	4.71	3.30	2000	23	10.08	1996	.14	1987	7.3	5.7	2.8	1.0	.68	1.08	1.79	2.48	3.21	4.01	4.94	6.07	7.61	10.13	12.55
Nov	10.89	10.39	4.80	1961	22	21.06	1988	2.91	1976	13.7	11.6	5.1	1.8	3.58	4.61	6.13	7.42	8.66	9.95	11.35	13.00	15.11	18.39	21.42
Dec	10.68	10.31	5.56	1995	31	26.55	1996	1.52	1985	14.7	12.4	5.9	3.0	2.68	3.68	5.24	6.62	7.98	9.41	11.01	12.91	15.40	19.33	23.01
Ann	68.74	68.21	6.58	Jan 1999	21	26.55	Dec 1996	.00+	Sep 1993	123.0	95.1	37.2	14.2	46.87	51.03	56.41	60.51	64.18	67.74	71.42	75.51	80.49	87.74	94.05

\+ Also occurred on an earlier date(s)
\# Denotes amounts of a trace
@ Denotes mean number of days greater than 0 but less than .05
** Statistics not computed because less than six years out of thirty had measurable precipitation

(1) From the 1971-2000 Monthly Normals
(2) Derived from station's available digital record: 1948-2001
(3) Derived from 1971-2000 serially complete daily data

Complete documentation available from:
www.ncdc.noaa.gov/oa/climate/normals/usnormals.html

082-B

ATTACHMENT C
Determine 5-yr Winter & 10-yr Summer Flows – DEQ Methodology

Attachment C
Lebanon WWTP
DETERMINE 5-YR WINTER AND 10-YEAR SUMMER FLOWS
DEQ Methodology

Background

DEQ has established guidelines for determining the statistical Max. Month Dry Weather Flow (MMDWF) and Max. Month Wet Weather Flow (MMWWF) for a treatment plant. These guidelines outline a procedure where historical flows and rainfall are used to develop a relationship that is used to determine plant flows based on rainfall.

Flows are developed by creating a scatter plot of the total monthly rainfall against the average monthly flow. A linear regression is performed on this data to develop an equation that represents the system. This regression formula is then used to determine the MMWWF and MMDWF using a probable rainfall. The probable rainfall for the MMDWF is the historical 10-year May monthly rainfall and for the MMWWF it is the 5-year January rainfall (See Attachment B).

An analysis to develop the 5-year rainfall and the 10-year rainfall was performed. The results from this evaluation (Attachment B) were: 10-yr May - 4.12-inches, 5-yr January rainfall 9.46-inches (Climatography 20 Oregon State University Weather Station)

In this analysis, flow and rainfall data for the period from January 2017 - December 2021 was used. This will provide conservative results as it includes the 2017 wet season that was an extremely wet period with much more rainfall than the normal.

Evaluation

In this analysis, plots for each of the study years (Jan - May and Nov - Dec data), for the period 2010-2016, 2017 - 2021 and 2010 - 2021 were developed. A linear regression was then developed to develop a curve on the monthly rainfall vs. flow data. The MMWWF and MMDWF was calculated for each evauation period.

A peaking factor for each, the MMDWF and MMWWF, was calculated using the dry weather flow (ADWF) based on the average season flows for the months May - October and the dry weather flows for both the months of of June - Sept. and July - September for the years 2017 - 2021. The values in the following table were determined in Tables 5 - 8 in Attachment A. The results are as follows:

Calculated from Plot Regressions (DEQ Methodology)

Condition	ADWF	MMDWF	Peaking Factor	MMWWF	Peaking Factor
ADWF Based on July - September Average					
2017 - 2021	1.60	4.93	3.08	8.34	5.21
ADWF Based on July - September Median Flow					
2017 - 2021	1.56	4.93	3.16	8.34	5.35
ADWF Based on Summer Season Average Flows					
2017 - 2021	2.13	4.93	2.31	8.34	3.92

Conclusions

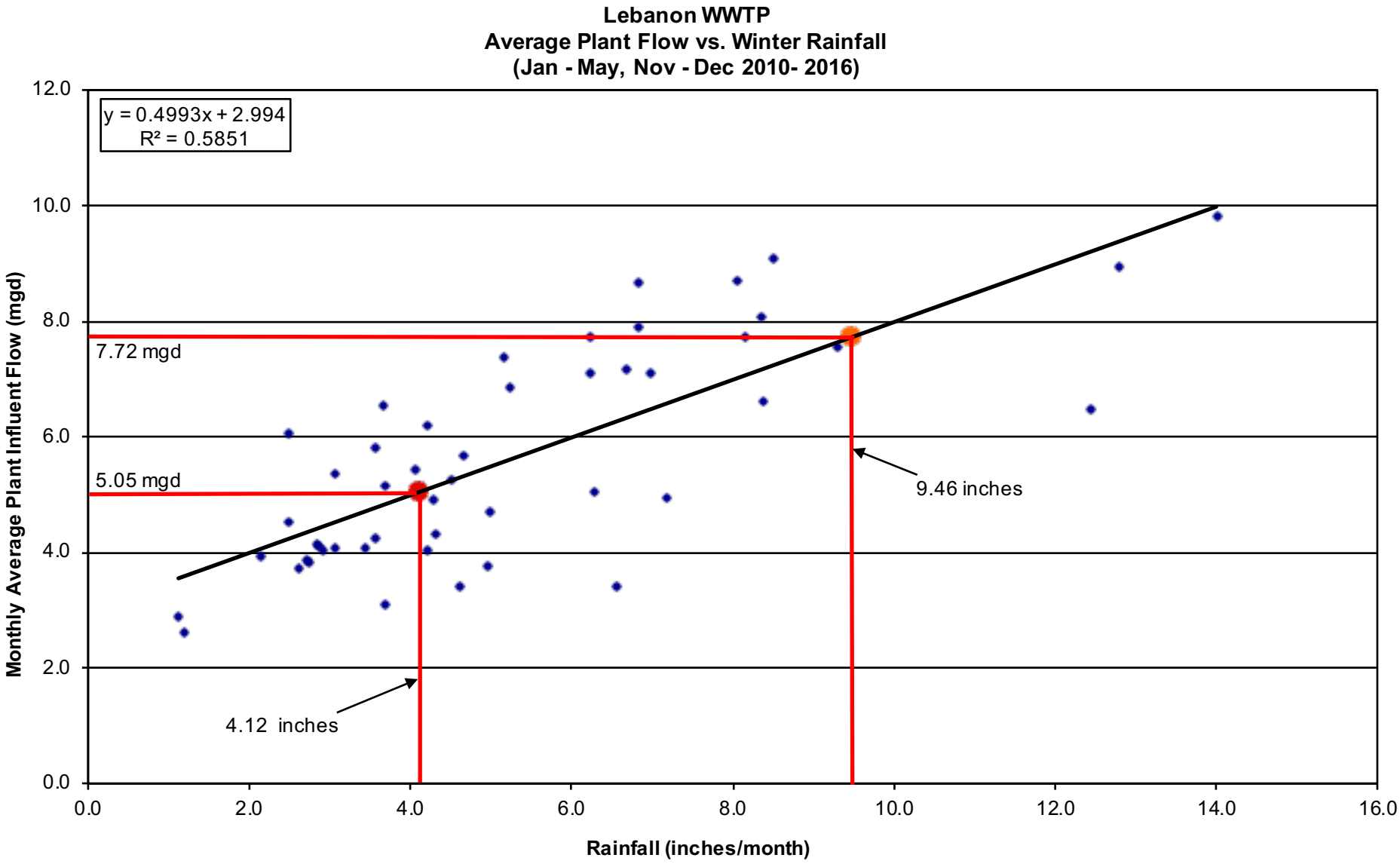
The flows for the 5-year winter (MMWWF) and the 10-year summer (MMDWF) flows were developed using the DEQ methodology for the period January 2017 - December 2021. The conclusions for this analysis are as follows:

- MMDWF = 4.93 mgd
- MMWWF = 8.34 mgd

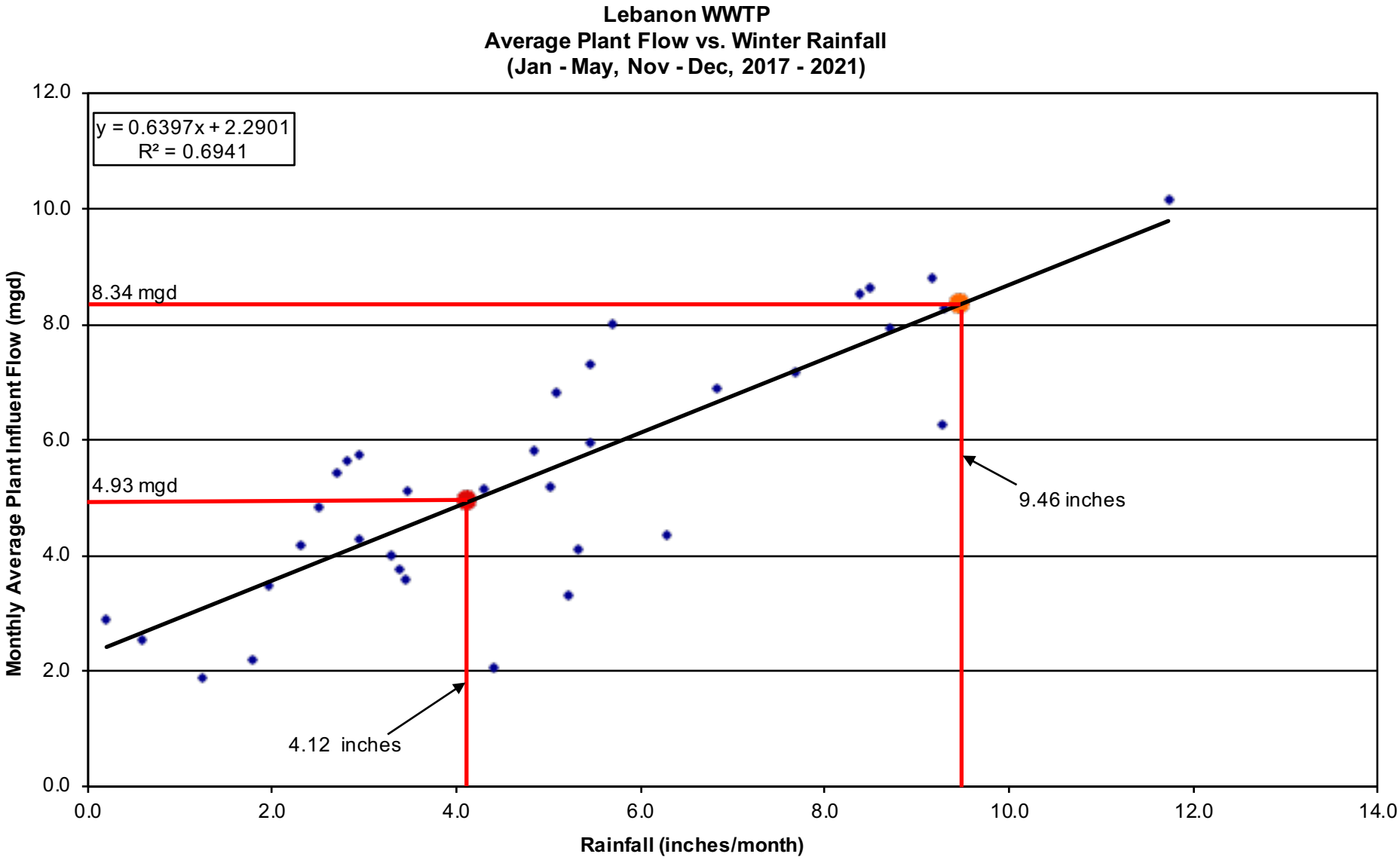
The peaking factors for MMDWF were evaluated using the July - September ADWF, the July - September Median Flow and the Summer Permit Season Average Flow for the period 2017 - 2021. The peaking factors were calculated to be 3.08, 3.16 and 2.31 for each of the flow conditions, respectively. Based on this analysis, the ADWF for the period July - September is the best period for flows not influenced by rainfall. Therefore, a peaking factor for MMDWF of 3.08 will be used based on an ADWF of 1.60-mgd.

The peaking factors for MMWWF were evaluated using the July - September ADWF, the July - September Median Flow and the Summer Permit Season Average Flow for the period 2017 - 2021. The peaking factors were calculated to be 5.21, 5.35 and 3.92 for each of the flow conditions, respectively. Based on this analysis, the ADWF for the period July - September is the best period for flows not influenced by rainfall. Therefore, a peaking factor for MMWWF of 5.21 will be used based on an ADWF of 1.60-mgd.

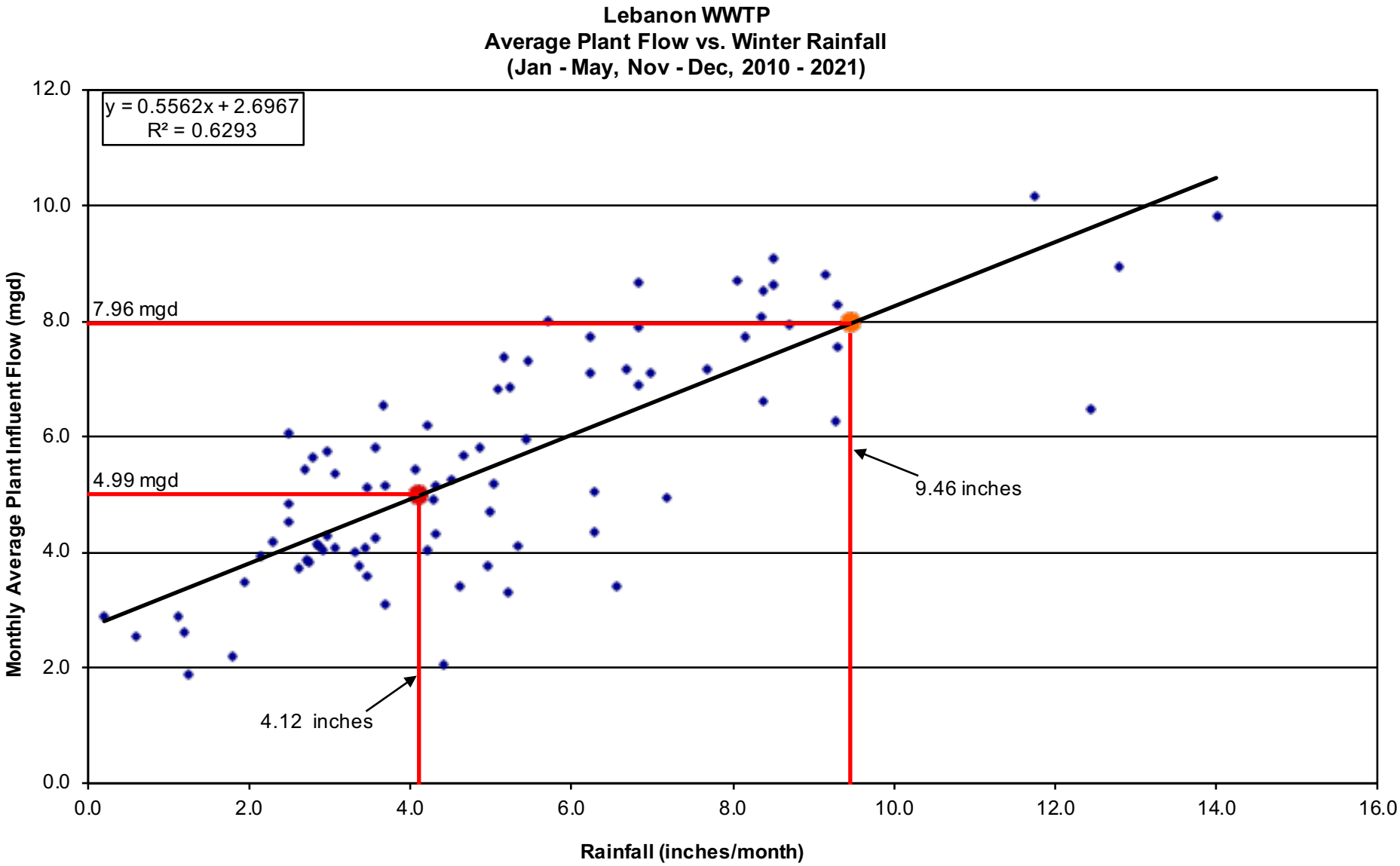
- MMDWF Peaking Factor = 3.08
- MMWWF Peaking Factor = 5.21



Richwine Environmental, Inc.
File Name: Attach C - Lebanon WWTP 10-Yr Summer 5-Yr Winter Flows.xlsx
Sheet Name: Fig 1



Richwine Environmental, Inc.
File Name: Attach C - Lebanon WWTP 10-Yr Summer 5-Yr Winter Flows.xlsx
Sheet Name: Fig 2



Richwine Environmental, Inc.
File Name: Attach C - Lebanon WWTP 10-Yr Summer 5-Yr Winter Flows.xlsx
Sheet Name: Fig 3

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ATTACHMENT D
Determine Max Day Flow ($PDAF_5$)

Attachment D
Lebanon WWTP
Determine Max Day Flow ($PDAF_5$)

Background

ODEQ has established a procedure for determining the probable maximum day flow for a wastewater treatment plant. This is done by using the statistical 5-year, 24-hour storm. The storm is used to calculate a flow using a regression from a scatter plot of the existing peak daily flows and rainfalls that occurred during saturated ground conditions.

In this analysis, a scatter plot of the daily rainfall vs. daily flow was developed for the period January through April for each period 2010 - 2021 and 2017 - 2021. A scatter plot was developed for each case when the rainfall was greater than 0.5-in, 1-in, 1.5-in and 2.0-inches for each period. A linear regression was calculated on each plot and a linear regression formula was determined. This formula was used to determine the peak daily flow for the 5-year, 24-hour storm in each case, when possible.

5-Year, 24-Hour Duration Storm

The 5-year, 24-hour duration storm (winter and annual storm) is obtained from NOAA in Atlas 2, Volume

Evaluation

There regression plots for the period 2017 - 2021 did not provide adequate linear regressions as shown in Figures D6 - D9. Figure D6 did provide an adequate regression showing a peak day flow of 12.89-mgd using flow data from rainfalls greater than 0.5-inches. The same plot for the period 2010 - 2021 provided in Figure D1 resulted in a peak day flow of 13.41-mgd. Due to the poor regression plots obtained when analyzing the 2017 - 2021 data, it was determined that it would be best to use the larger data set of 2000 - 2021.

The results of this analysis for the period 2010 - 2021 is:

Results from DEQ Methodology

Janaury 2010 - April 2021 Lebanon WWTP Influent and Rainfall Data

0.5-inch	13.41	mgd	Fig 1
1.0-inch	13.20	mgd	Fig 2
1.5-inches	13.42	mgd	Fig 3
2.0-inches	NA	mgd	Fig 4

This analysis using the DEQ Methodology shows the peak daily flow calculated values ranged from 13.20-mgd to 13.42-mgd using the rainfall data ranging from 0.5 to 1.5-inches per day. There was not adequate data for rainfalls greater than 2.0-mgd to obtain a representative regression. These regressions were performed on the 22-year period from January 2010 - April 2021. The analysis showed that the peak day flow is 13.41-mgd when using all rainfall data greater than 0.5-inches.

An analysis of the plant influent flow data was performed. A plot of this data is shown if Figure 5. This plot shows that there have been 6-days when the plant daily flow has exceeded 16.5-mgd over the period January 1, 2000 through December 31, 2021. This data is suspect in that the Westside Interceptor has been used for flow equalization ovver this period of time. The City took over operations on September 1, 2020 and gradually pulled down the levels in the Westside Intereceptor as they lowered the excessive solids inventory that was in the treatment plant. A storm event on December 20, 2020 produced a daily flow of 16.5-mgd following the City's takeover of the operation of the treatment plant. This was without using the Westside Interceptor for flow equalization and is the best indicator of an acual peak day flow event.

The PDF derived from using the DEQ Statistical Meghodology was 13.42-mgd. As shown on Figure 1. This value is low compared to the acual historical daily influent flow data. Therefore, the value dervied from evaluation of the historical influent flow data of 16.5-mgd will be used in leu of the value derived using the DEQ Methodology.

Conclusion

The 5-year, 24-hour statistical rainfall was determined to be 3.0-inches based on the NOAA Atlas 2 Volume X.

The calculated PDF using the DEQ statistically derived method is 13.42-mgd using rainfall greater than 0.5-inches for the Janaury 2010 - December 2021 period. This value is low when compared to actual historical peak day flow data. For this reason, the value developed from the observed historical peak day flow data of 16.5-mgd will be used.

Peak Daily Flow (PDF) = 16.5-mgd

The peak day peaking factor using the average dry weather flow of 1.6-mgd is therefore, 10.31.
Peak Day Flow Peaking Factor = 10.31

Table D1

Lebanon WWTP

Statistical Peaking Factor Evaluation

Based on Average Dry Season Flows

Year	ADSF	MMDSF	MMDW Peaking Factor	MMWWF	MMWW Peaking Factor	Max Day Flow	Max Day Peaking Factor
2010	2.71	5.57	2.06	6.94	2.56	8.94	3.30
2011	2.37	6.05	2.55	5.19	2.19	6.68	2.81
2012	2.30	7.09	3.08	5.53	2.40	5.72	2.48
2013	2.62	4.00	1.53	6.18	2.36	7.38	2.82
2014	2.31	5.11	2.21	5.65	2.45	5.58	2.41
2015	1.78	3.79	2.13	3.15	1.77	4.29	2.41
2016	2.52	5.49	2.18	8.06	3.20	9.70	3.86
2017	2.20	5.85	2.66	6.22	2.83	7.33	3.34
2018	1.82	5.74	3.15	4.10	2.25	3.76	2.06
2019	2.28	8.68	3.80	4.47	1.96	4.04	1.77
2020	2.38	3.85	1.62	4.25	1.79	6.64	2.79
2021	1.96	2.66	1.36	3.66	1.87	4.17	2.13
Maximum							
2010 - 2016	2.71	7.09	3.08	8.06	3.20	9.70	3.86
2017 - 2021	2.38	8.68	3.80	6.22	2.83	7.33	3.34
2010 - 2021	2.71	8.68	3.80	8.06	3.20	9.70	3.86
Average Based Peaking Factors							
2010 - 2016	2.37	5.30	2.25	5.82	2.42	6.90	2.87
2017 - 2021	2.13	5.36	2.52	4.54	2.14	5.19	2.42
2010 - 2021	2.27	5.32	2.36	5.28	2.30	6.19	2.68

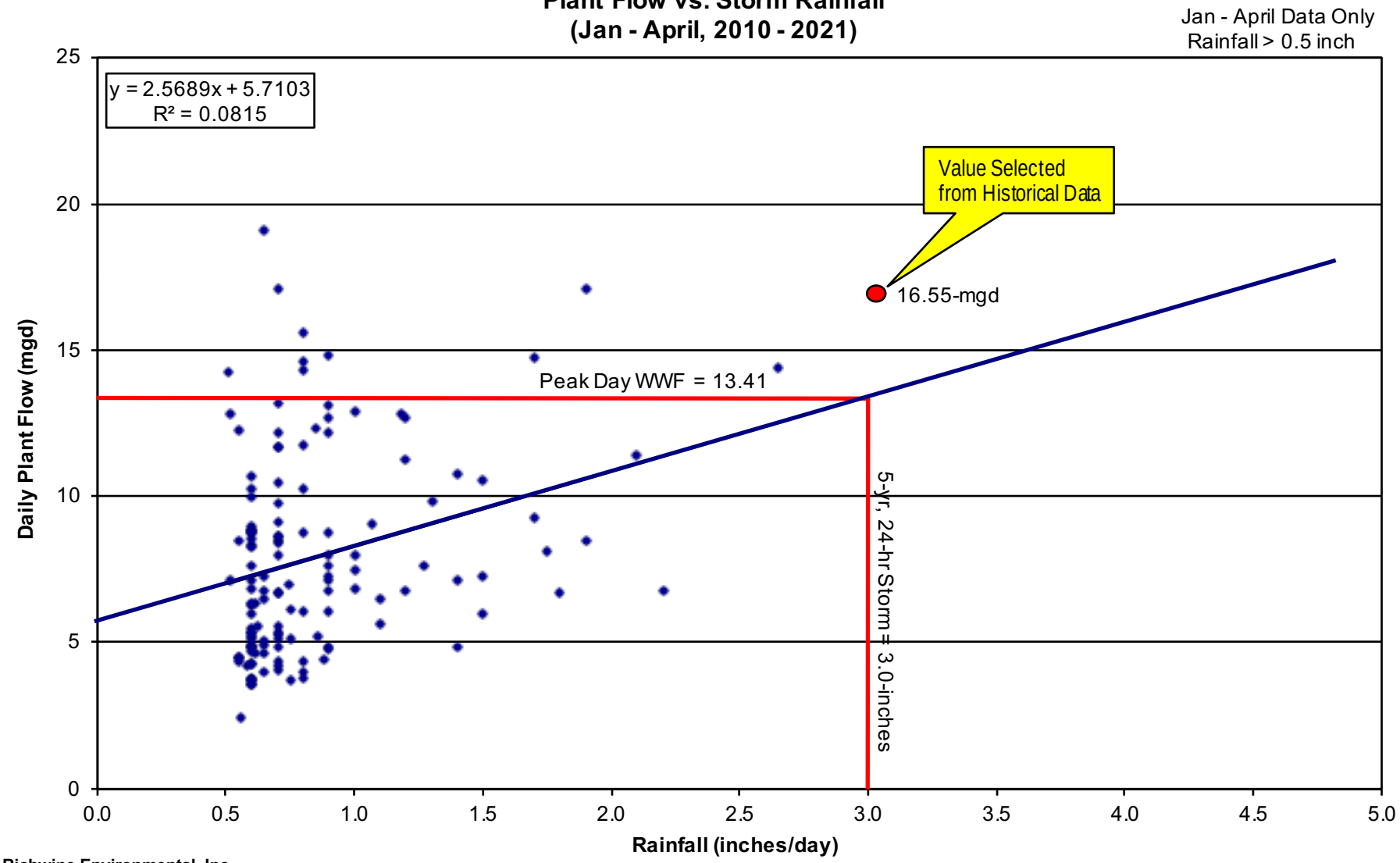
Based on Median Dry Season Flows

Year	Median DSF	MMDSF	MMDW Peaking Factor	MMWWF	MMWW Peaking Factor	Max Day Flow	Max Day Peaking Factor
2010	1.96	5.57	2.84	6.94	3.55	8.94	4.57
2011	1.93	6.05	3.13	5.19	2.68	6.68	3.46
2012	1.79	7.09	3.97	5.53	3.09	5.72	3.20
2013	2.53	4.00	1.58	6.18	2.44	7.38	2.91
2014	1.89	5.11	2.71	5.65	3.00	5.58	2.96
2015	1.54	3.79	2.46	3.15	2.04	4.29	2.78
2016	1.90	5.49	2.89	8.06	4.24	9.70	5.10
2017	1.67	5.85	3.50	6.22	3.72	7.33	4.38
2018	1.54	5.74	3.72	4.10	2.66	3.76	2.43
2019	2.13	8.68	4.06	4.47	2.09	4.04	1.89
2020	1.96	3.85	1.97	4.25	2.17	6.64	3.39
2021	1.86	2.66	1.43	3.66	1.97	4.17	2.24
Maximum							
2010 - 2016	2.53	7.09	3.97	8.06	4.24	9.70	5.10
2017 - 2021	2.13	8.68	4.06	6.22	3.72	7.33	4.38
2010 - 2021	2.53	8.68	4.06	8.06	4.24	9.70	5.10
Average Based Peaking Factors							
2010 - 2016	1.93	5.30	2.80	5.82	3.01	6.90	3.57
2017 - 2021	1.83	5.36	2.94	4.54	2.52	5.19	2.87
2010 - 2021	1.89	5.32	2.85	5.28	2.80	6.19	3.28

Based on Selected Dry Weather Flow of 1.6-mgd

Year	ADSF	MMDSF	MMDW Peaking Factor	MMWWF	MMWW Peaking Factor	Max Day Flow	Max Day Peaking Factor
2010	1.60	5.57	3.48	6.94	4.34	8.94	5.59
2011	1.60	6.05	3.78	5.19	3.24	6.68	4.18
2012	1.60	7.09	4.43	5.53	3.46	5.72	3.57
2013	1.60	4.00	2.50	6.18	3.86	7.38	4.61
2014	1.60	5.11	3.19	5.65	3.53	5.58	3.48
2015	1.60	3.79	2.37	3.15	1.97	4.29	2.68
2016	1.60	5.49	3.43	8.06	5.04	9.70	6.07
2017	1.60	5.85	3.66	6.22	3.89	7.33	4.58
2018	1.60	5.74	3.59	4.10	2.56	3.76	2.35
2019	1.60	8.68	5.42	4.47	2.79	4.04	2.53
2020	1.60	3.85	2.41	4.25	2.66	6.64	4.15
2021	1.60	2.66	1.66	3.66	2.29	4.17	2.61
Maximum							
2010 - 2016	1.60	7.09	4.43	8.06	5.04	9.70	6.07
2017 - 2021	1.60	8.68	5.42	6.22	3.89	7.33	4.58
2010 - 2021	1.60	8.68	5.42	8.06	5.04	9.70	6.07
Average Based Peaking Factors							
2010 - 2016	1.60	5.30	3.31	5.82	3.63	6.90	4.31
2017 - 2021	1.60	5.36	3.35	4.54	2.84	5.19	3.24
2010 - 2021	1.60	5.32	3.33	5.28	3.30	6.19	3.87

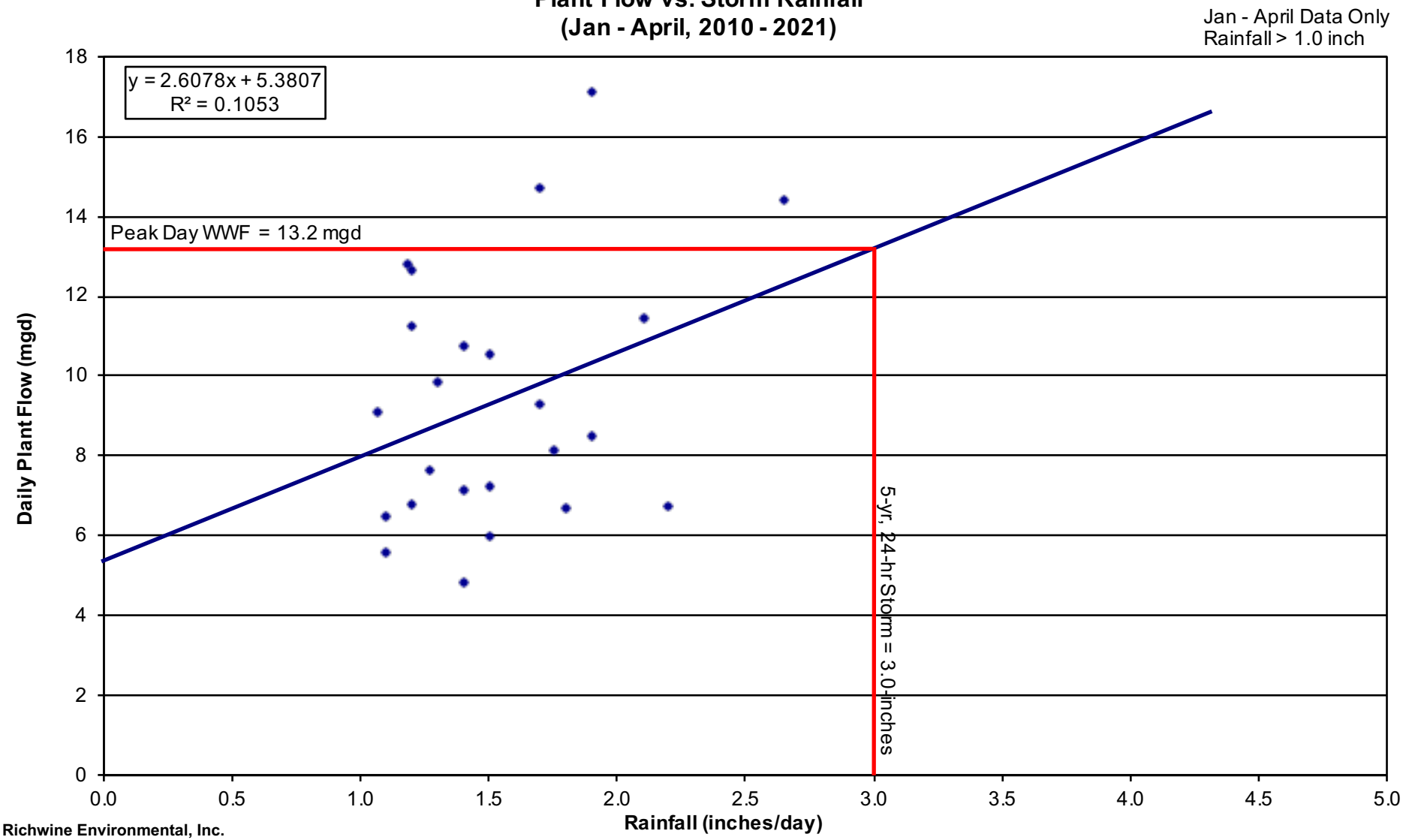
Figure D1
Lebanon WWTP
Plant Flow vs. Storm Rainfall
(Jan - April, 2010 - 2021)



Richwine Environmental, Inc.
File Name: Attach D - Lebanon WWTP DEQ PDF Calculation_ Ver 3.xlsx
Sheet Name: Fig 1

11/9/22

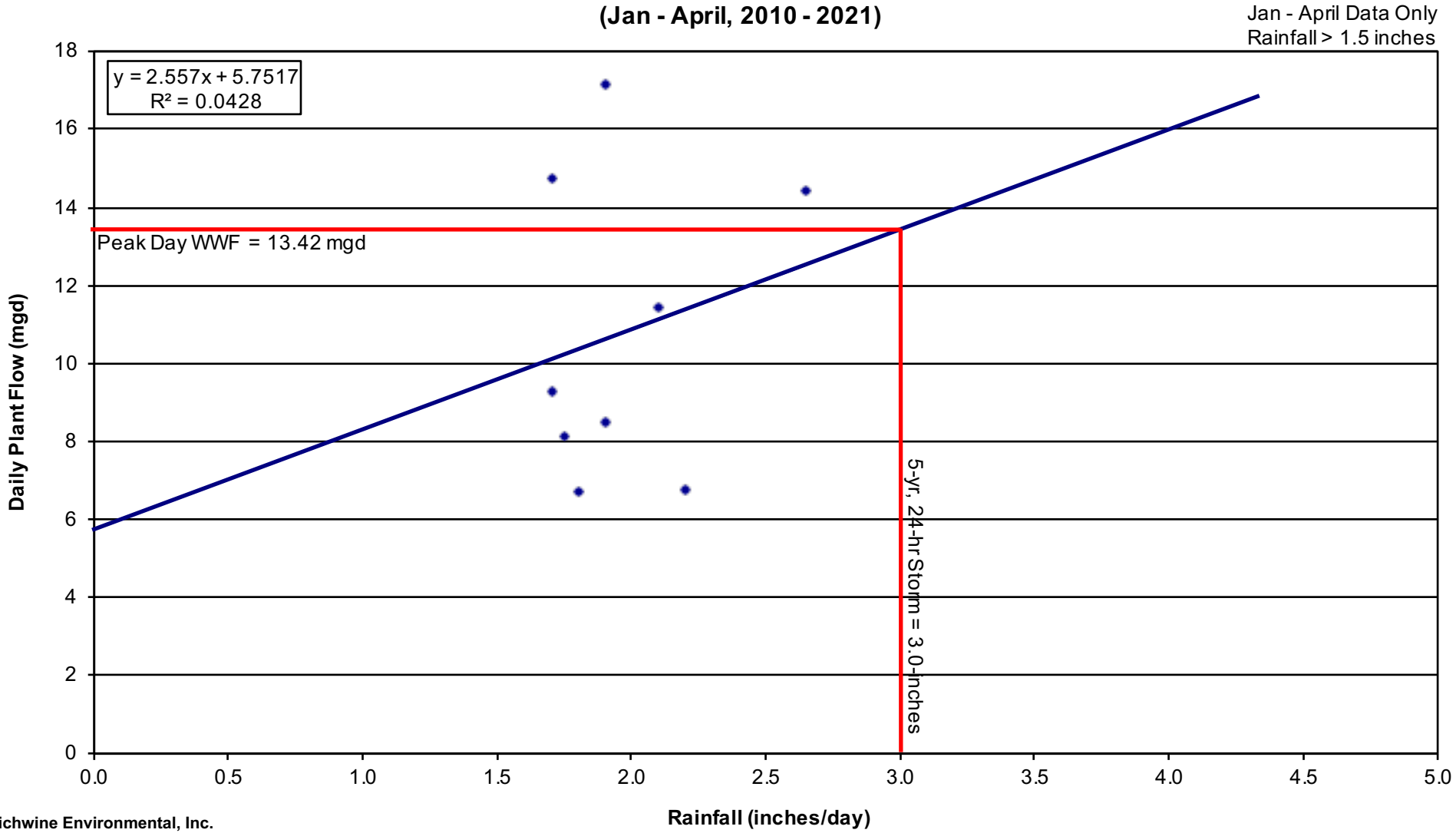
Figure D2
Lebanon WWTP
Plant Flow vs. Storm Rainfall
(Jan - April, 2010 - 2021)



Richwine Environmental, Inc.
File Name: Attach D - Lebanon WWTP DEQ PDF Calculation_ Ver 3.xlsx
Sheet Name: Fig 2

11/9/22

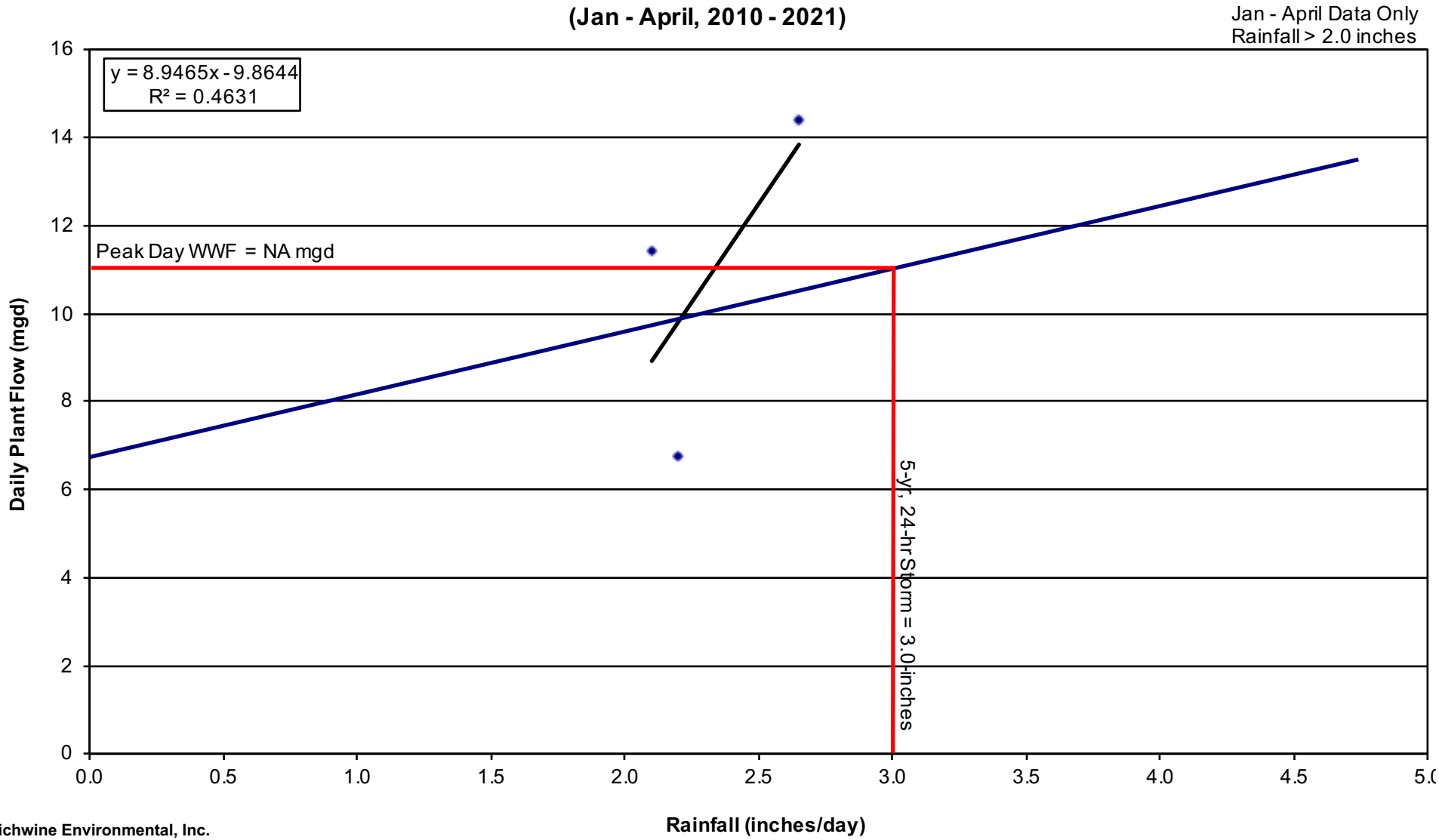
Figure D3
Lebanon WWTP
Plant Flow vs. Storm Rainfall
(Jan - April, 2010 - 2021)



Richwine Environmental, Inc.
File Name: Attach D - Lebanon WWTP DEQ PDF Calculation_ Ver 3.xlsx
Sheet Name: Fig 3

11/9/22

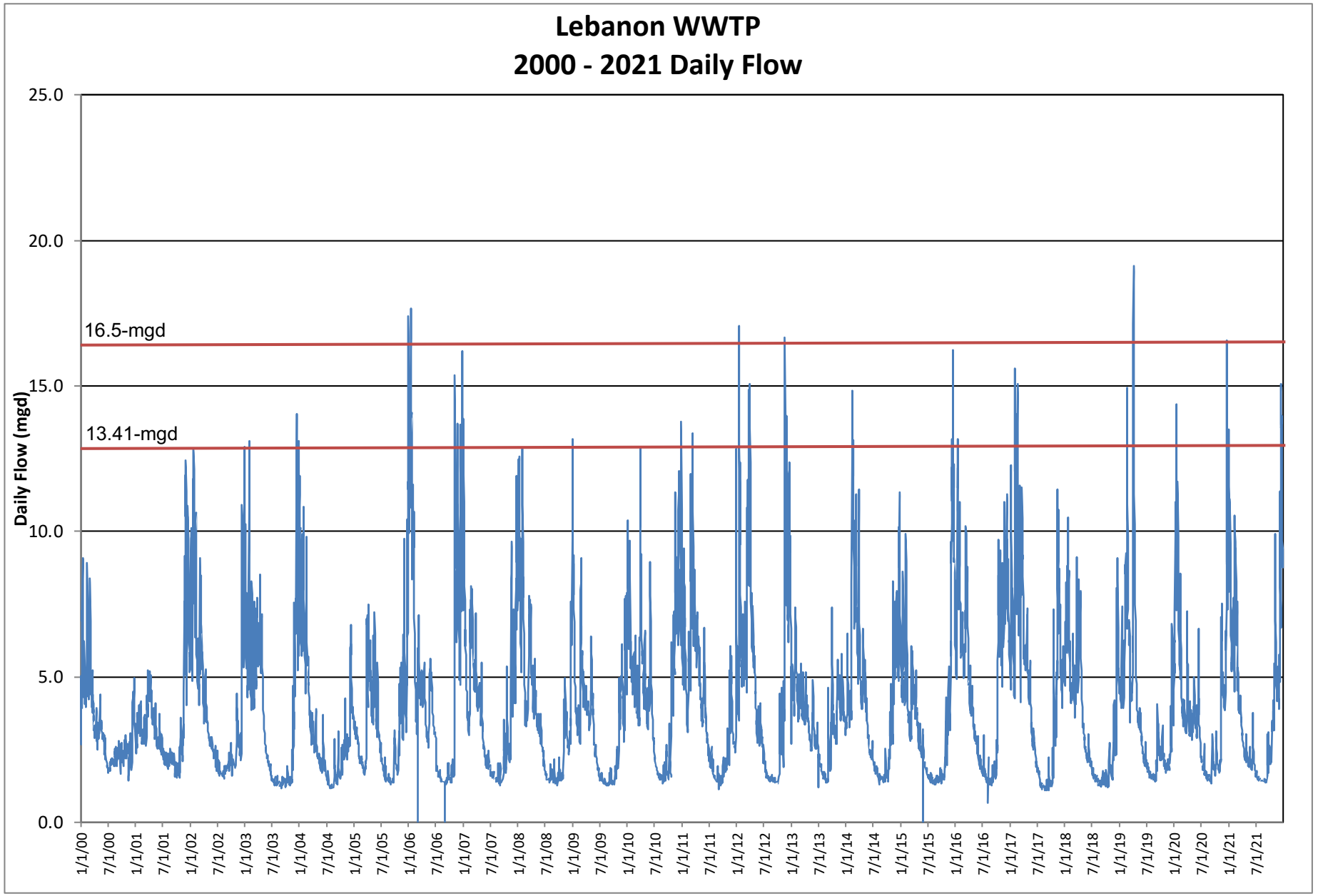
Figure D4
Lebanon WWTP
Plant Flow vs. Storm Rainfall
(Jan - April, 2010 - 2021)



Richwine Environmental, Inc.
File Name: Attach D - Lebanon WWTP DEQ PDF Calculation_ Ver 3.xlsx
Sheet Name: Fig 4

11/9/22

Figure D5


Lebanon WWTP
2000 - 2021 Daily Flow
Daily Flow (mgd)
25.0
20.0
15.0
10.0
5.0
0.0
16.5-mgd
13.41-mgd
1/1/00
7/1/00
1/1/01
7/1/01
1/1/02
7/1/02
1/1/03
7/1/03
1/1/04
7/1/04
1/1/05
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1/1/18
7/1/18
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7/1/20
1/1/21
7/1/21

Figure D6
NOAA 5-yr, 24-Hour Storm

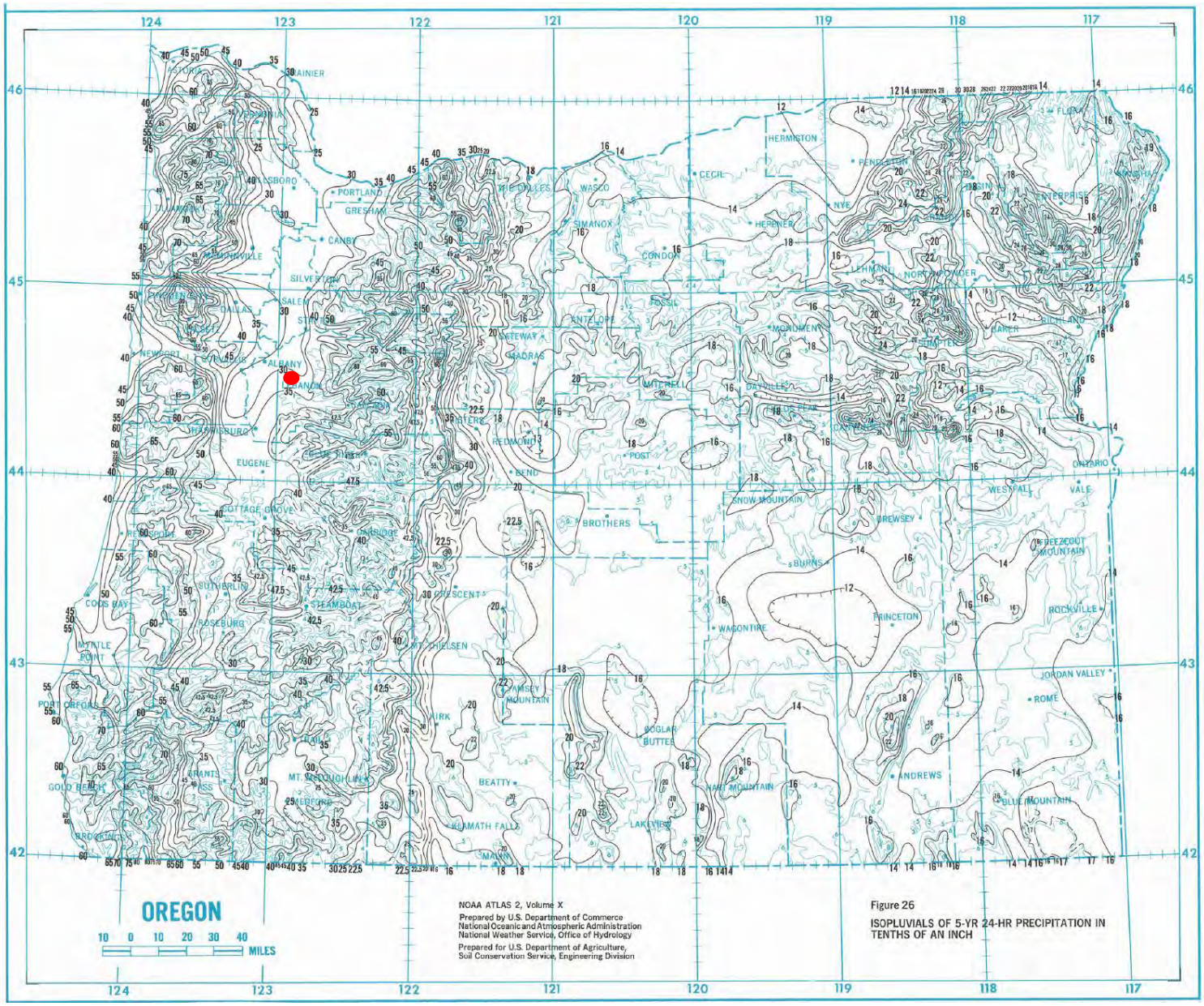
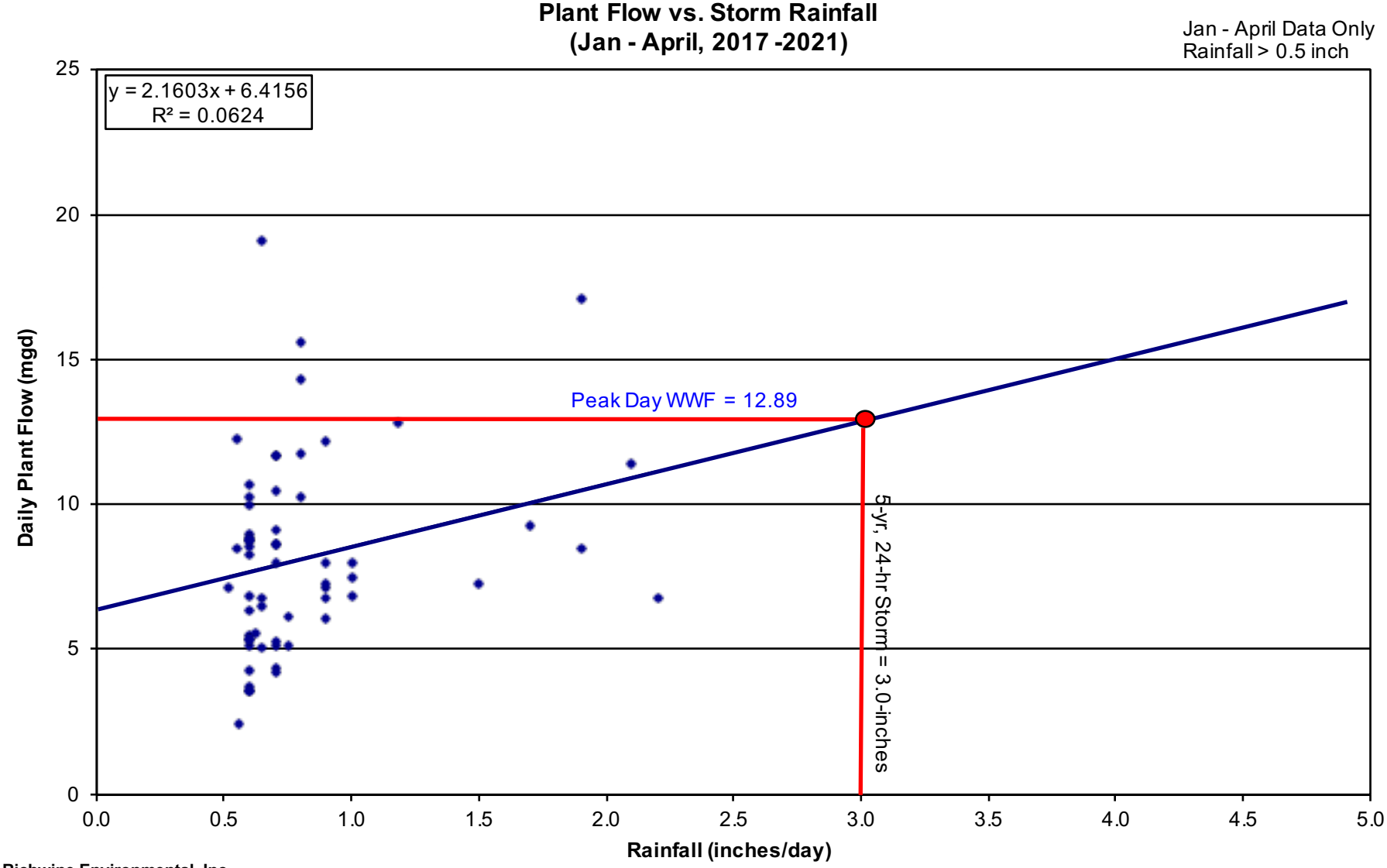


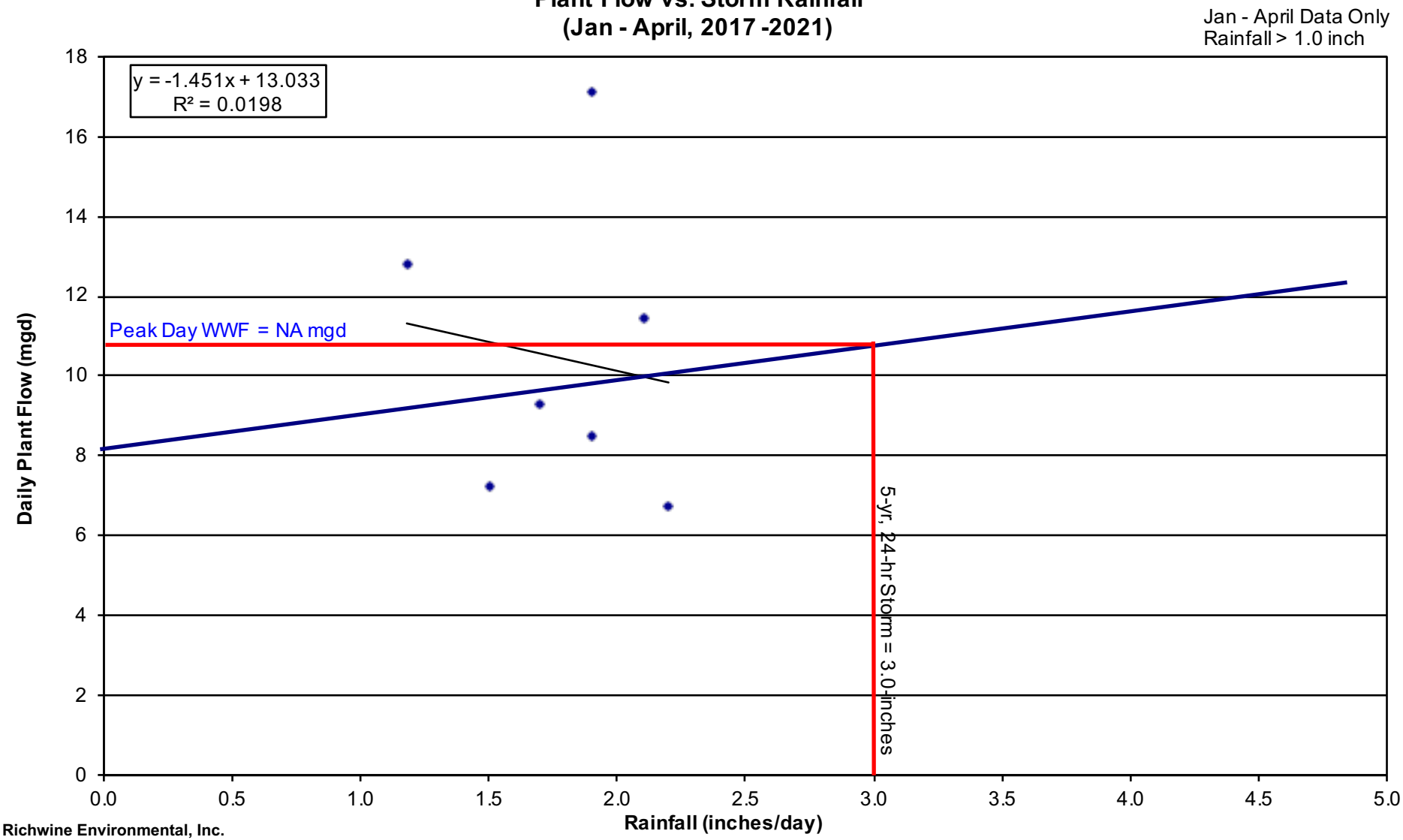
Figure D6
Lebanon WWTP
Plant Flow vs. Storm Rainfall
(Jan - April, 2017 -2021)



Richwine Environmental, Inc.
File Name: Attach D - Lebanon WWTP DEQ PDF Calculation_ Ver 3.xlsx
Sheet Name: Fig 6 (2017-2021))

11/9/22

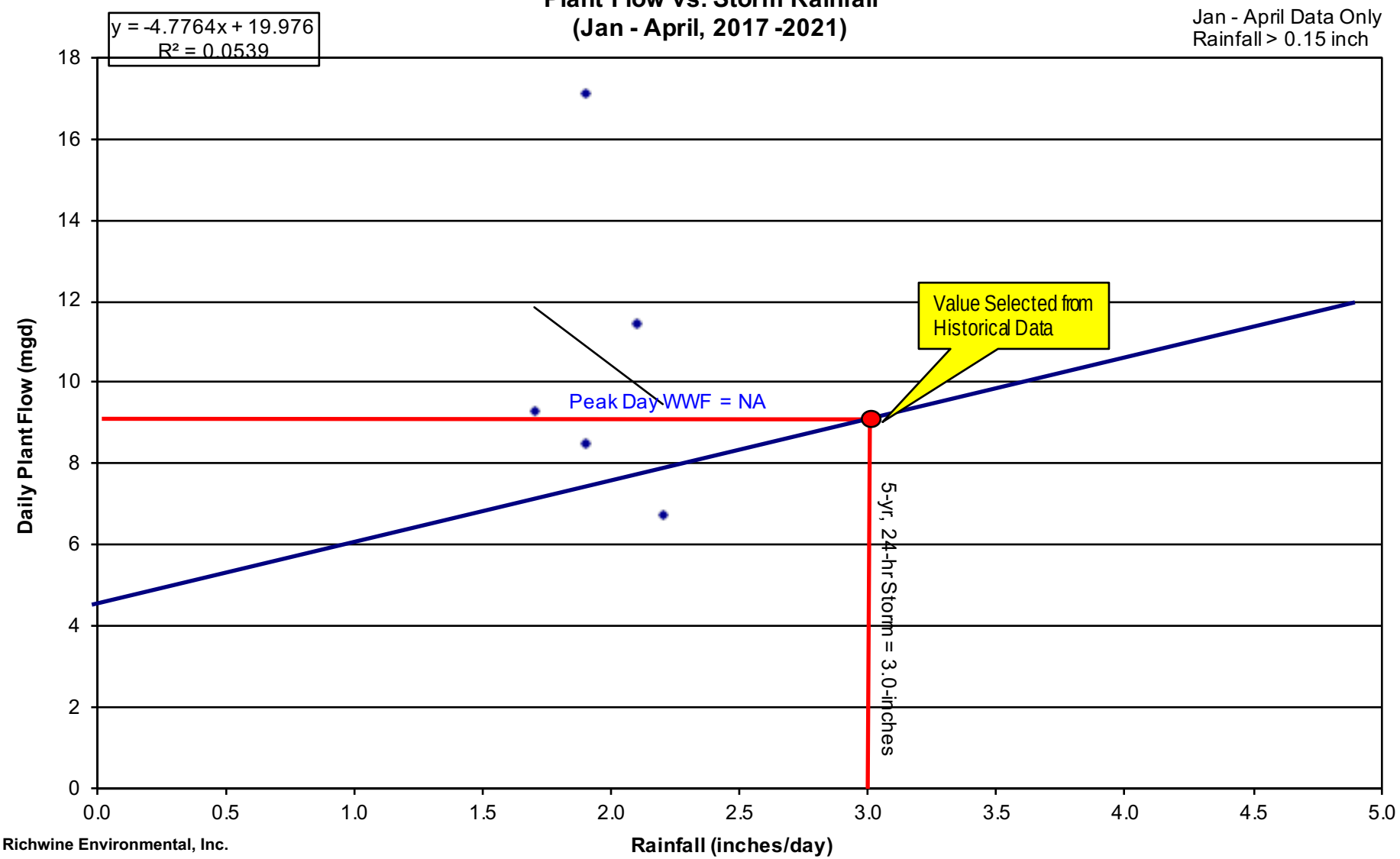
Figure D7
Lebanon WWTP
Plant Flow vs. Storm Rainfall
(Jan - April, 2017 -2021)



Richwine Environmental, Inc.
File Name: Attach D - Lebanon WWTP DEQ PDF Calculation_ Ver 3.xlsx
Sheet Name: Fig 7 (2017 -2021)

11/9/22

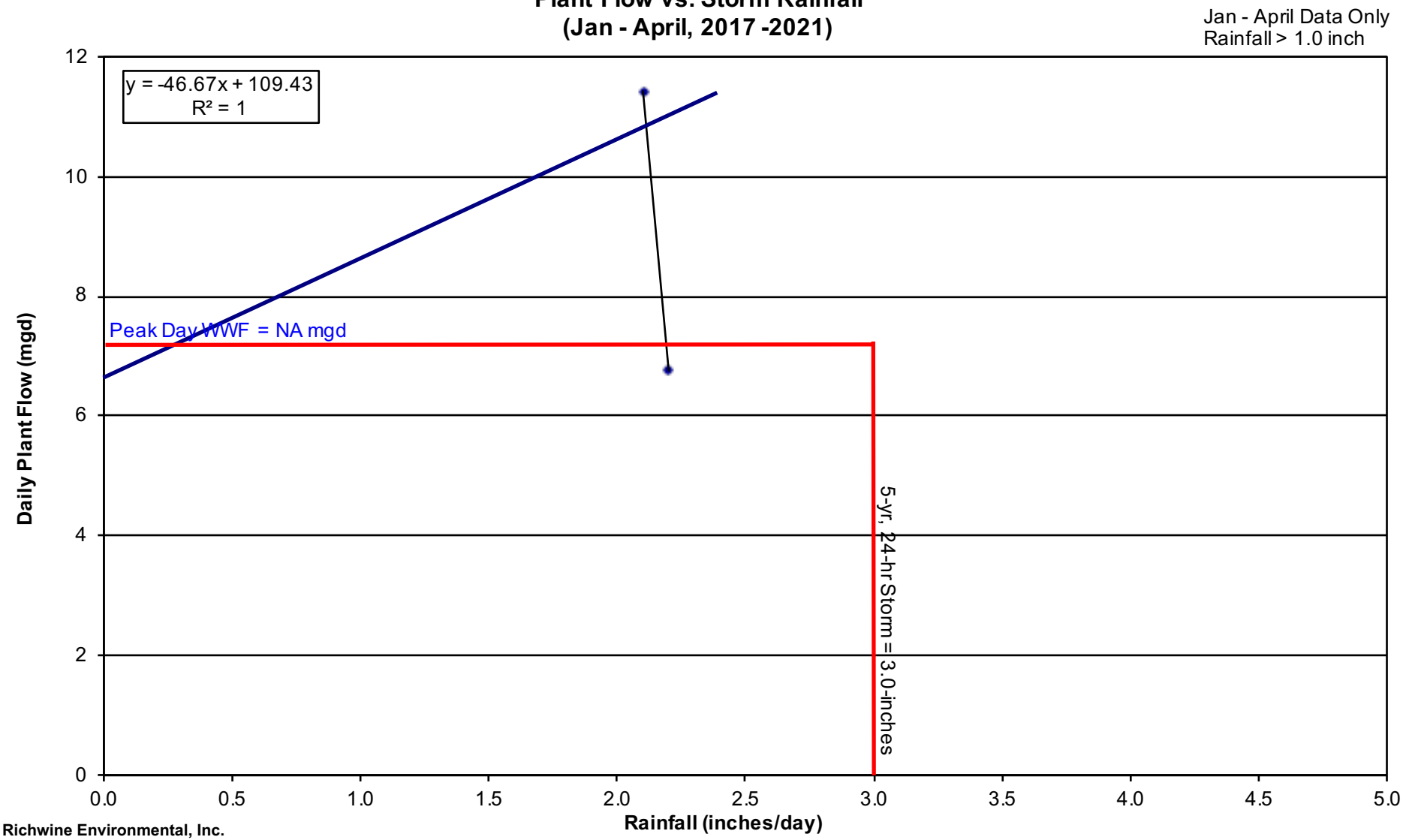
Figure D8
Lebanon WWTP
Plant Flow vs. Storm Rainfall
(Jan - April, 2017 -2021)



Richwine Environmental, Inc.
File Name: Attach D - Lebanon WWTP DEQ PDF Calculation_ Ver 3.xlsx
Sheet Name: Fig 8 (2017-2021)

11/9/22

Figure D9
Lebanon WWTP
Plant Flow vs. Storm Rainfall
(Jan - April, 2017 -2021)



Richwine Environmental, Inc.
File Name: Attach D - Lebanon WWTP DEQ PDF Calculation_ Ver 3.xlsx
Sheet Name: Fig 9 (2017-2021)

11/9/22

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ATTACHMENT E
Determine Peak Hour Flow (PIF_5)

Attachment E
Lebanon WWTP
Determine Peak Hour Flow

Background

The PIF_5 is the peak instantaneous or peak hourly flow associated with a 5-year PDAF. That is, the peak flow resulting from a 5-year storm during high groundwater periods. The best method to determine the PIF_5 is by modeling of the collection system. When this information is not available, ODEQ has provided a methodology for estimating PIF_5 using probability of occurrence.
The ODEQ methodololgy for estimating PIF5 is provided in Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon: MMDWF, MMWWF, PDAF and PIF.

An analysis of plant flows for the period of time between January 2017 thorough December 2021 was performed to determine wastewater characteristics. Data was analyzed for the annual, winter season, summer season and dry weather (July - Sept.) conditions in Attachment A Tables A1 - A8. The maximum month wet weather flow (MMWWF) was determined in Attachment C and the peak day flow (PDF) was determined in Attachment D using the DEQ Methodology or a review of historical data.

Definitions

The PIF_5 is shown on Figure 1 as a probability plot.
AAF - The average annual flow of of 3.95-mgd (Attachment A - Table 5) was plotted at a probability of 50%.

MMWWF - The maximum month wet weather flow (DEQ Methodology) of 8.34-mgd (Table C1) was plotted at a probability of 8.3%.

MWWWF - The maximum week wet weather flow was taken from the plot where the line intersected with the probability of 1.92% axis.

$PDAF_5$ - The peak day flow of 16.5 obtained from an evaluation of historical data in Attachment F was plotted at a probability of 0.27%.

PIF_5 - The peak instantaneous flow is then estimated at the 0.011% probability. A best fit line is drawn through the data and the PIF5 is estimated where the line crosses the 0.011% probability line. This evaluation then gives and estimated value of 26.6-mgd for the PIF_5.

Evaluation

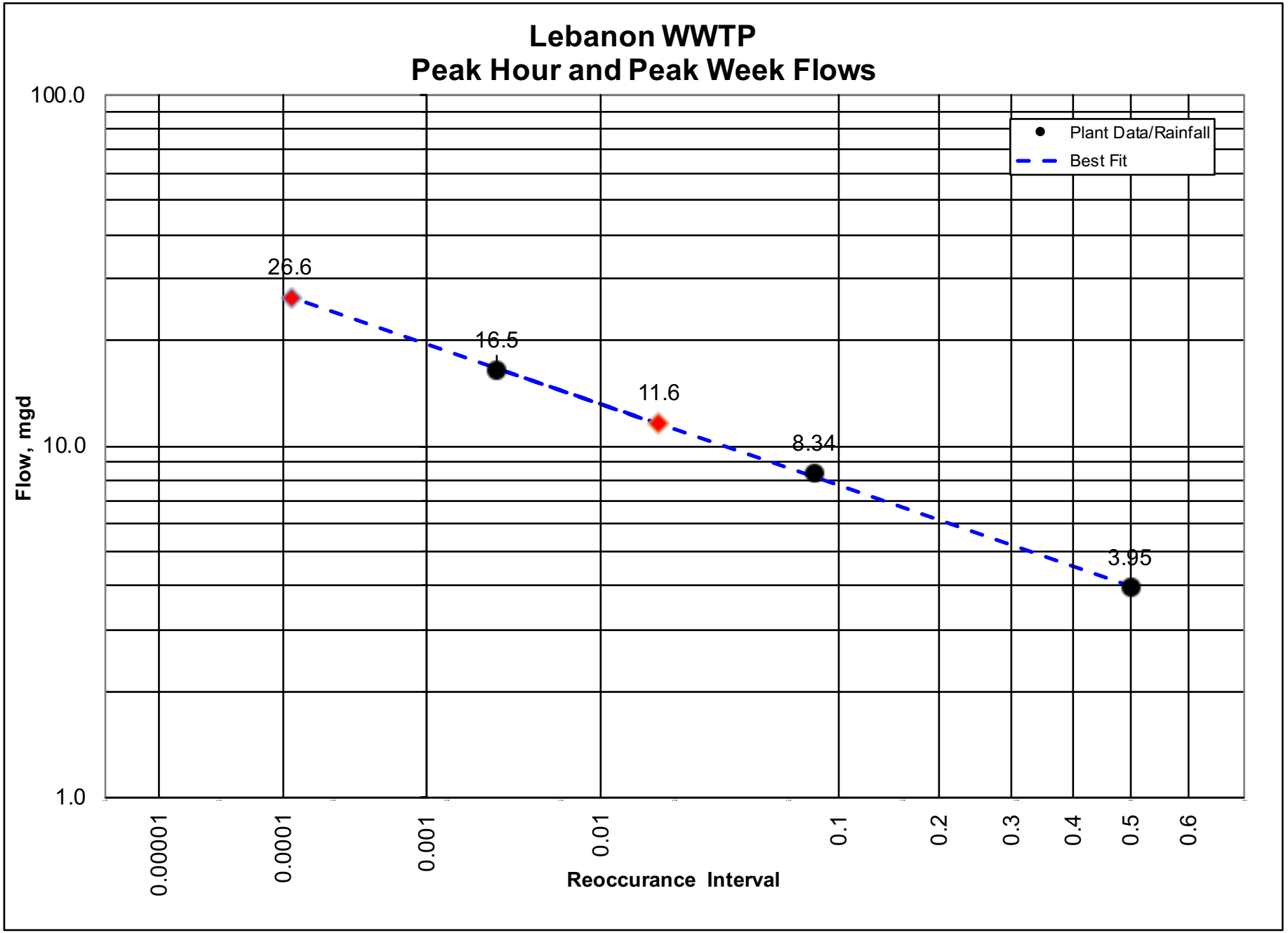
The flows for each of the 2021 statistical flow rates are as follows using the statistical flow rates developed using DEQ Methodology or review of histrical flows for each of the statistical flow values:
The results are as follows:

Data Inputs and Results		
Average Annual -	3.95	mgd
Max. Mo. Wet Weather -	8.34	mgd
Peak Week -	11.6	mgd
Peak Day Ave. Flow -	16.5	mgd
Peak Instantaneous Flow	26.6	mgd

Conclusion

The selected peak week and peak hour flows are based on the statistical analysis using the flow rates selected in Attachment F. These are as folllows:

Peak Week Flow -	**11.60**	**mgd**
Peak Instantaneous Flov	**26.60**	**mgd**


Lebanon WWTP
Peak Hour and Peak Week Flows
100.0
10.0
1.0
Flow, mgd
Plant Data/Rainfall
Best Fit
26.6
16.5
11.6
8.34
3.95
0.00001
0.0001
0.001
0.01
0.1
0.2
0.3
0.4
0.5
0.6
Reoccurance Interval

DEQ Peak Hour Flow Calculations
Probability Method

Data Processing for Log-Probability Distribution

Current **2021**		Reoccurance				r^2	0.972
Parameter		Q mgd	Interval	Probits	LN(Q)	Best Fit	ln(best fit)
Annual Average Flow (AAF)		**3.95**	0.50000	0.000	1.3737156	4.00	1.386
Max Month Wet Weather Flow (MMWWF)		**8.34**	0.08300	-1.385	2.1210632	8.14	2.097
Peak Day Flow (PDF)		**16.5**	0.00270	-2.782	2.8033604	16.70	2.815
Max Week Flow (MWF)		11.6	0.01920	-2.071	2.4495282	11.58	2.450
Peak Hour Flow (PHF)		26.6	0.00011	-3.686	3.2793594	26.56	3.279

Unit Peaking Factor Method On Chart? FALSE

Projected **2045**		Peaking	Projected	Reoccurance	
Parameter		Factor	Q (mgd)	Interval	Probits
Current Population	**19122**				
Projected Population	**27396**				
ADWF (current in mgd)	**1.640**				
Per capita ADWF (gpcpd)	92				
Average Dry Weather Flow (projected)		1.00	2.40		
Annual Average Flow (AAF)		2.44	5.85	0.50000	0.000
Max Month Wet Weather Flow (MMWWF)		4.97	11.92	0.08300	-1.385
Max Week Flow (MWF)		7.06	16.96	0.01920	-2.071
Peak Day Flow (PDF)		10.18	24.45	0.00270	-2.782
Peak Hour Flow (PHF)		16.19	38.89	0.00011	-3.686

Assume No Additional I/I On Chart? TRUE

Projected **2045**	Projected	Reoccurance	
Parameter	Q (mgd)	Interval	Probits
Average Dry Weather Flow (current)	1.76		
Average Dry Weather Flow (projected)	2.40		
Annual Average Flow (AAF)	4.64	0.50000	0.000
Max Month Wet Weather Flow (MMWWF)	8.79	0.08300	-1.385
Max Week Flow (MWF)	12.22	0.01920	-2.071
Peak Day Flow (PDF)	17.34	0.00270	-2.782
Peak Hour Flow (PHF)	27.20	0.00011	-3.686

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ATTACHMENT F
Determine Flow Peaking Factors

Attachment F
Lebanon WWTP
Determine Flow Peaking Factors

Background

Peaking factors to establish future flows need to be determined for planning. Two methods were used to determine the flows to be used in determining the current peaking factors. These methods are the ODEQ Guidance and an evaluation of historical flows. This analysis will summarize the system flows developed in Attachments A - E and determine the current peaking factors.

Peaking factors are based on the flow for the statistical period divided by the average dry weather flow. Peaking factors are determined for each variable as follows:
-- Maximum Month Flow (MMF) -- Max. 30-day Average flow / Average Dry Weather flow
-- Maximum Week Flow (MWF) -- Max. 7-day Average Flow / Average Dry Weather Flow
-- Maximum Day Flow (MDF) -- Max. day flow / Average Dry Weather Flow
-- Peak Hour Flow (PHF) -- Peak hour flow / Average Dry Weather Flow

An analysis of plant flows for the period of time between Janaury 2017 thorough December 2021 was performed and peaking factors for each year were determined using the average flow for each period (Table 2) and the median flow for each period (Table 3). Dry weather flows for the months July - September were also determined for each year (Table 4). Data was analyzed for the annual, wet weather permit season and dry weather permit season conditions. This information was compared with statistical flows based on ODEQ Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon: MMDWF, MMWWF, PDAF and PIF.

Evaluation

In this evaluation, the results of the historical flows and Peaking Factors and those developed using the DEQ Methodology were compared. The flows using the DEQ Methodology for MMDWF and MMWWF were derived in Attachment C. The flow for PDF was derived in Attachment D. The flow for PHF was derived in Attachment E.

Dry Weather Flow

The dry weather flow (DWF) that was used to determine the peaking factors is the average flow for the period July 1 - Sept. 30. The flows for these months were selected as they are not affected by rainfall as the flows in the months of May, June and October.

The data on Table 4 shows the average of the average dry season flows for the 2017-2021 period as 1.60-mgd. The median of the median dry season flows for the 2017-2021 period is 1.56-mgd. The dry weather flow is not impacted by RDII as the wet weather events occur on the shoulder months. Therefore, the average dry weather flows for this period of 1.60-mgd will be used to determined peaking factors.

Statistical Flows

The statistical flows were developed in each of the Attachments A - E for determining the current flows to the Lebanon WWTP. These statistical flows are based on the average of the 2017 - 2021 historical flows for the historical flow data. One exception was the dry season max week flow that was increased to 5.00-mgd from 4.54-mgd to provide a reasonable value between the maximum month and maximum day value. The 2017 -2021 historical flows were also used in the calculation of flows using DEQ Methodology. The exception to this was the peak day flow where historical data from 2010 - 2021 was used. The flow rates from the historical data and from the values calculated from the DEQ methodology were compared and the flows to be used in this analysis to develop peaking factors were selected. The DEQ methodology was used for all of the current flow rates with the exception of the PDF. Historical data was used for the peak day flow. The flow for each of the statistical flow rates are as follows:

Variable	Average	Max Month	Max Week	Max Day	Peak Hour
Historical Data					
Average Annual	3.95				
Dry Season	2.13	5.36	5.0	5.19	
Wet Season	5.79	8.83	12.46	16.5	27
Dry Weather	1.60				
DEQ Methodology					
Average Annual					
Dry Season		4.93			
Wet Season		8.34	11.6	13.4	26.6
Selected Data					
Average Annual	3.95				
Dry Season	2.13	4.93	5.0	5.19	
Wet Season	5.79	8.34	11.6	16.5	26.6
Dry Weather	1.60				

NOTE 1: The wet season maximum day flow of 16.5-mgd was selected based on the analysis provided in Attachment D. The results using DEQ Methodology gave a value of 13.4-mgd. The historical flow was much higher than the statistical value. A value of 16.5-mgd was selected as it was the best respresentation of the historical data that was available. Prior to 2020, the treatment plant was operated using the interceptor for flow equalization. This made the influent peak day flow data unreliable. The peak day flow of 16.5-mgd occurred following the City's takeover of treatment plant operations and is a reliable value.

NOTE 2: The DEQ Peak Instantaneous Flow (PIF) Analysis provided in Attachment E. The PIF is calculated based on a statistical analysis using the annual average flow, maximum month wet weather flow and the peak day flow. Three scenarios were evaluated using this method: data obtained from historical data, DEQ Methodology and the selected data. The results provided the peak week flow and the PIF. The peak week and PIF flows for each scenario are as follows:

The results taken from the PIF flow analysis using DEQ Methodology (Attachment E) was a Peak Week Flow of 11.6-mgd and a PHF or PIF of 26.6-mgd.

	Average Annual	MMWWF	Max Week	Max Day	Peak Hour
Historical Data	3.95	8.83	12.46	16.5	27.0
DEQ Methodology	-	8.34	11.60	13.4	26.6
Selected Data	3.95	8.34	11.60	16.5	26.6

Conclusion

The calculated statistical flow rates for the current conditions are as follows:

Statistical Flows

Variable	Average Flow	Max Month	Max Week	Max Day	Peak Hour
Average Annual	3.95				
Dry Season	2.13	4.93	5.0	5.19	
Wet Season	5.79	8.34	11.60	16.5	26.6
Dry Weather	1.60				

Dry Weather Flow

A dry weather flow of 1.60-mgd will be used as the base system flow as it is the average of the July - Sept. average flows (Table 4)

Peaking Factors

The peaking factors to be used in developing system flows were calculated from the statistical flows divided by the dry weather flow of 1.60-mgd. The peaking factors are summarized in the following table.

Variable	Average Flow	Max Month	Max Week	Max Day	Peak Hour
Average Annual	2.47				
Dry Season	1.33	3.08	3.12	3.24	
Wet Season	3.62	5.21	7.25	10.31	16.62
Dry Weather	1.00				

Per Capita Flow

The per capita flows were calculated in Attachment A by taking the average of the 5-years of average dry weather flows divided by the population for each of the respective years. This value was 92-gal/capita/day.

Variable	Ave Annual	Dry Weather	Dry Season	Wet Season	Units
Per Capita Flow	227	92	122	333	gal/capita/day

Table F1
Lebanon WWTP
Statistical Peaking Factor Evaluation

Based on Average Summer Season Flows

Year	ADSF	MMDSF	MMDS Peaking Factor	MWDSF	MWDS Peaking Factor	MDDSF
2017	2.20	5.85	2.66	6.22	2.83	7.33
2018	1.82	5.74	3.15	4.10	2.25	3.76
2019	2.28	8.68	3.80	4.47	1.96	4.04
2020	2.38	3.85	1.62	4.25	1.79	6.64
2021	1.96	2.66	1.36	3.66	1.87	4.17
Average 2016 - 2021	2.13	5.36	2.52	4.54	2.14	5.19
Median 2016 - 2021	2.20	5.74	2.66	4.25	1.96	4.17
90 Percentile 2016 - 2021	2.34	7.55	3.54	5.52	2.60	7.05
Maximum 2016 - 2021	2.38	8.68	3.80	6.22	2.83	7.33

Winter Season Flows

Year	AWSF	MMWSF	MMWS Peaking Factor	MWWSF	MWWS Peaking Factor	MDWSF
2017	7.17	10.20	1.42	12.84	1.79	15.60
2018	4.96	7.50	1.51	9.08	1.83	10.47
2019	5.37	8.80	1.64	15.27	2.84	19.11
2020	5.48	8.87	1.62	11.58	2.11	16.55
2021	5.97	8.77	1.47	13.54	2.27	15.08
Average 2016 - 2021	5.79	8.83	1.53	12.46	2.17	15.36
Median 2016 - 2021	5.48	8.80	1.51	12.84	2.11	15.60
90 Percentile 2016 - 2021	6.69	9.67	1.63	14.58	2.61	18.09
Maximum 2016 - 2021	7.17	10.20	1.64	15.27	2.84	19.11

Based on Average Dry Weather Flow (July, August, September)

Year	Average DWF	MMDSF	MM Peaking Factor	MWDSF	MW Peaking Factor	MDDSF
2017	1.34	5.85	4.37	6.22	4.64	7.33
2018	1.45	5.74	3.95	4.10	2.82	3.76
2019	1.88	8.68	4.62	4.47	2.38	4.04
2020	1.74	3.85	2.21	4.25	2.44	6.64
2021	1.59	2.66	1.68	3.66	2.30	4.17
Average 2016 - 2021	1.60	5.36	3.37	4.54	2.92	5.19
Median 2016 - 2021	1.59	5.74	3.95	4.25	2.44	4.17
90 Percentile 2016 - 2021	1.82	7.55	4.52	5.52	3.91	7.05
Maximum 2016 - 2021	1.88	8.68	4.62	6.22	4.64	7.33

Table A-2
Lebanon WWTP
Influent Flow Characterization

Average Annual

Year	Service Area Population	Historical Average Annual Flows (mgd)				Peaking Factors
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month
2017	16,720	4.66	10.20	12.84	15.60	4.64
2018	16,920	3.38	7.50	9.08	10.47	4.12
2019	17,135	3.82	8.80	15.27	19.11	3.85
2020	17,335	3.92	8.87	11.58	16.55	3.73
2021	19,122	3.95	8.77	13.54	15.08	4.48
Average	17,446	3.95	8.83	12.46	15.36	4.16

Summer Season

Year	Resident Service Area Population	Historical Summer Season Flows (mgd)				Peaking Factors
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month
2017	16,720	2.20	5.85	6.22	7.33	2.66
2018	16,920	1.82	5.74	4.10	3.76	3.15
2019	17,135	2.28	8.68	4.47	4.04	3.80
2020	17,335	2.38	3.85	4.25	6.64	1.62
2021	19,122	1.96	2.66	3.66	4.17	1.36
Average	17,446	2.13	5.36	4.54	5.19	2.52

Winter Season

Year	Resident Service Area Population	Historical Winter Season Flows (mgd)				Peaking Factors
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month
2017	16,720	7.17	10.20	12.84	15.60	4.64
2018	16,920	4.96	7.50	9.08	10.47	4.12
2019	17,135	5.37	8.80	15.27	19.11	3.85
2020	17,335	5.48	8.87	11.58	16.55	3.73
2021	19,122	5.97	8.77	13.54	15.08	4.48
Average	17,446	5.79	8.83	12.46	15.36	4.16

Table A-2
Lebanon WWTP
Influent Flow Characterization

Average Annual

Year	Resident Service Area Population	Historical Average Annual Flows (mgd)				Peaking Factors
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month
2017	16,720	4.38	10.20	12.84	15.60	6.10
2018	16,920	2.50	7.50	9.08	10.47	4.86
2019	17,135	2.73	8.80	15.27	19.11	4.12
2020	17,335	3.38	8.87	11.58	16.55	4.77
2021	19,122	2.62	8.77	13.54	15.08	4.71
Average	**17,446**	**3.12**	**8.83**	**12.46**	**15.36**	**4.91**

Summer Season

Year	Resident Service Area Population	Historical Summer Season Flows (mgd)				Peaking Factors
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month
2017	16,720	1.67	5.85	6.22	7.33	3.50
2018	16,920	1.54	5.74	4.10	3.76	3.72
2019	17,135	2.13	8.68	4.47	4.04	4.06
2020	17,335	1.96	3.85	4.25	6.64	1.97
2021	19,122	1.86	2.66	3.66	4.17	1.43
Average	**17,446**	**1.83**	**5.36**	**4.54**	**5.19**	**2.94**

Winter Season

Year	Resident Service Area Population	Historical Winter Season Flows (mgd)				Peaking Factors
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month
2017	16,720	6.64	10.20	12.84	15.60	6.10
2018	16,920	**4.87**	**7.50**	**9.08**	**10.47**	4.86
2019	17,135	4.61	8.80	15.27	19.11	4.12
2020	17,335	4.64	8.87	11.58	16.55	4.53
2021	19,122	5.24	8.77	13.54	15.08	4.71
Average	**17,446**	**5.20**	**8.83**	**12.46**	**15.36**	**4.86**

Table A-4
Lebanon WWTP
Dry Weather Flow Evaluation

Monthly Average

Year	Resident Service Area Population	Dry Weather Flows Monthly Average (mgd)				
		June	July	August	Sept	Oct
2017	16,720		1.55	1.18	1.29	
2018	16,920		1.61	1.37	1.38	
2019	17,135		1.79	1.56	2.28	
2020	17,335		1.93	1.54	1.75	
2021	19,122		1.56	1.42	1.79	
Average	17,446		1.69	1.42	1.70	

Monthly Median

Year	Resident Service Area Population	Dry Weather Flows Monthly Median (mgd)				
		June	July	August	Sept	Oct
2017	16,720		1.62	1.18	1.25	
2018	16,920		1.58	1.38	1.35	
2019	17,135		1.77	1.56	2.28	
2020	17,335		1.84	1.54	1.54	
2021	19,122		1.55	1.42	1.45	
Average	17,446		1.67	1.42	1.58	

Monthly Minimum

Year	Resident Service Area Population	Dry Weather Flows Monthly Minimum (mgd)				
		June	July	August	Sept	Oct
2017	16,720		1.21	1.08	1.08	
2018	16,920		**1.39**	**1.24**	**1.23**	
2019	17,135		1.53	1.44	1.39	
2020	17,335		1.61	1.43	1.32	
2021	19,122		1.44	1.38	1.34	
Average	17,446		1.44	1.31	1.27	

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ATTACHMENT G
Determine Peaking Factors and Per Capita Loads for BOD_5

Attachment G
Lebanon WWTP
Determine Peaking Factors and Per Capita Loads for BOD_5

Background

Peaking factors and per capita loads are necessary to characterize the influent wastewater and project BOD_5 loads into the future. Projections are made assuming that these characteristics remain the same.

Evaluaion

Peaking Factors

Peaking factors are determined as follows:
-- Max Month -- Maximum 30 day average / Average Day
-- Max Week -- Maximum 7-day average / Average Day
-- Max Day -- Maximum day / Average Day

In this analysis, the loads for the period between January 2010 and December 2021 were evaluated. The max. month, max. week and max day values were developed for each of the seasonal conditions. The peaking factor for each year were calculated and an average of the peaking factors was determined for the years 2010 - 2021 as shown in Table 1. This analysis showed wet season loads being significantly higher than dry season loads. the analysis was also done using the median vs. the average in Table 2 and for the period between January 2017 and December 2021 in Tables 3 and 4.
The values for BOD5 peaking factors to be used in the study are summarized in the following table. The BOD values were slightly lower in the 2017 - 2021 period so these values were used as they show the most recent loads.

Period	Max. Month	Max. Week	Max. Day
Average Annual	1.6	2.8	3.2
Summer Season	1.3	1.8	1.9
Winter Season	1.4	2.4	2.9

Per Capita Loads

Per capita loads are calculated by taking the average day load and dividing it by the population for the City's service area for the respective year. These values were taken from Table 3 for the period 2017 - 2020.

Period	Per Capita Load
Average Annual	0.16
Summer Season	0.15
Winter Season	0.18

Conclusion

The per capita BOD_5 loads typically range from 0.18 to 0.22 lbs/capita/day depending on the demographics of the service area. The average annual per capita BOD5 load is 0.17 lbs/capita/day.

The peaking factors and per capita loads for BOD_5 were calculated and are summarized in the tables above. These peaking factors will be used for planning purposes. The per capita loads shown in the table above will be used for planning purposes.

Table G1
Lebanon WWTP
Influent BOD Characterization

Average Annual

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average			Annual BOD per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15,525	3243	5868	10438	10486	1.81	3.22	3.23	0.21
2011	15,565	3336	5122	8187	11499	1.54	2.45	3.45	0.21
2012	15,660	3230	4638	8847	9683	1.44	2.74	3.00	0.21
2013	15,690	2999	4245	8099	11191	1.42	2.70	3.73	0.19
2014	15,740	2731	3753	7672	7967	1.37	2.81	2.92	0.17
2015	15,740	3282	6479	8360	11734	1.97	2.55	3.58	0.21
2016	16,435	3657	6630	11756	15118	1.81	3.21	4.13	0.22
2017	16,720	3059	6276	9209	9929	2.05	3.01	3.25	0.18
2018	16,920	2653	3076	5506	5506	1.16	2.08	2.08	0.16
2019	17,135	2821	4097	6751	8563	1.45	2.39	3.04	0.16
2020	17,335	2634	3282	5313	8559	1.25	2.02	3.25	0.15
2021	19,122	3046	5744	13728	13728	1.89	4.51	4.51	0.16
Average	**16466**	**3057**	**4934**	**8656**	**10330**	**1.60**	**2.81**	**3.35**	**0.19**

Summer Season

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average			Annual BOD per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15525	2816	4420	7438	10046	1.57	2.64	3.57	0.18
2011	15565	3282	4521	8055	11499	1.38	2.45	3.50	0.21
2012	15660	3040	4064	8435	8435	1.34	2.78	2.78	0.19
2013	15690	2994	4223	8099	11191	1.41	2.71	3.74	0.19
2014	15740	2653	3753	7672	7967	1.41	2.89	3.00	0.17
2015	15740	3209	4269	7267	11734	1.33	2.26	3.66	0.20
2016	16435	3104	4376	6081	7076	1.41	1.96	2.28	0.19
2017	16720	2356	3610	4084	4084	1.53	1.73	1.73	0.14
2018	16920	2589	3004	5506	5506	1.16	2.13	2.13	0.15
2019	17135	2702	3418	4328	5933	1.26	1.60	2.20	0.16
2020	17335	2486	3098	5313	5313	1.25	2.14	2.14	0.14
2021	19122	2756	3247	3887	3907	1.18	1.41	1.42	0.14
Average	**16466**	**2832**	**3834**	**6347**	**7724**	**1.35**	**2.23**	**2.68**	**0.17**

Winter Season

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average Dry Season Flow			Annual BOD per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15525	3670	5868	10438	10486	1.60	2.84	2.86	0.24
2011	15565	3389	5122	8187	8821	1.51	2.42	2.60	0.22
2012	15660	3435	4638	8847	9683	1.35	2.58	2.82	0.22
2013	15690	3005	4245	7596	8627	1.41	2.53	2.87	0.19
2014	15740	2807	3549	6159	6159	1.26	2.19	2.19	0.18
2015	15740	3354	6479	8360	9174	1.93	2.49	2.74	0.21
2016	16435	4188	6630	11756	15118	1.58	2.81	3.61	0.25
2017	16720	3820	6276	9209	9929	1.64	2.41	2.60	0.23
2018	16920	2717	3076	3847	4502	1.13	1.42	1.66	0.16
2019	17135	2937	4097	6751	8563	1.40	2.30	2.92	0.17
2020	17335	2722	3282	5058	8559	1.21	1.86	3.14	0.16
2021	19122	3367	5744	13728	13728	1.71	4.08	4.08	0.18
Average	**16466**	**3284**	**4917**	**8328**	**9446**	**1.48**	**2.49**	**2.84**	**0.20**

Table G2
Lebanon WWTP
Influent BOD Characterization

Average Annual

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average			Annual BOD per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15525	2882	5868	10438	10486	2.04	3.62	3.64	0.19
2011	15565	2834	5122	8187	11499	1.81	2.89	4.06	0.18
2012	15660	2874	4638	8847	9683	1.61	3.08	3.37	0.18
2013	15690	2748	4245	8099	11191	1.54	2.95	4.07	0.18
2014	15740	2454	3753	7672	7967	1.53	3.13	3.25	0.16
2015	15740	2791	6479	8360	11734	2.32	3.00	4.20	0.18
2016	16435	2812	6630	11756	15118	2.36	4.18	5.38	0.17
2017	16720	2605	6276	9209	9929	2.41	3.54	3.81	0.16
2018	16920	2498	3076	5506	5506	1.23	2.20	2.20	0.15
2019	17135	2555	4097	6751	8563	1.60	2.64	3.35	0.15
2020	17335	2454	3282	5313	8559	1.34	2.16	3.49	0.14
2021	19122	2866	5744	13728	13728	2.00	4.79	4.79	0.15
Average	**16466**	**2698**	**4934**	**8656**	**10330**	**1.82**	**3.18**	**3.80**	**0.16**

Summer Season

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average			Annual BOD per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15525	2544	4420	7438	10046	1.74	2.92	3.95	0.16
2011	15565	2806	4521	8055	11499	1.61	2.87	4.10	0.18
2012	15660	2840	4064	8435	8435	1.43	2.97	2.97	0.18
2013	15690	2735	4223	8099	11191	1.54	2.96	4.09	0.17
2014	15740	2241	3753	7672	7967	1.67	3.42	3.56	0.14
2015	15740	2796	4269	7267	11734	1.53	2.60	4.20	0.18
2016	16435	2695	4376	6081	7076	1.62	2.26	2.63	0.16
2017	16720	2201	3610	4084	4084	1.64	1.86	1.86	0.13
2018	16920	2463	3004	5506	5506	1.22	2.24	2.24	0.15
2019	17135	2473	3418	4328	5933	1.38	1.75	2.40	0.14
2020	17335	2425	3098	5313	5313	1.28	2.19	2.19	0.14
2021	19122	2789	3247	3887	3907	1.16	1.39	1.40	0.15
Average	**16466**	**2584**	**3834**	**6347**	**7724**	**1.49**	**2.45**	**2.96**	**0.16**

Winter Season

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average Dry Season Flow			Annual BOD per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15525	3670	5868	10438	10486	1.60	2.84	2.86	0.24
2011	15565	3389	5122	8187	8821	1.51	2.42	2.60	0.22
2012	15660	3435	4638	8847	9683	1.35	2.58	2.82	0.22
2013	15690	3005	4245	7596	8627	1.41	2.53	2.87	0.19
2014	15740	2807	3549	6159	6159	1.26	2.19	2.19	0.18
2015	15740	3354	6479	8360	9174	1.93	2.49	2.74	0.21
2016	16435	4188	6630	11756	15118	1.58	2.81	3.61	0.25
2017	16720	3820	6276	9209	9929	1.64	2.41	2.60	0.23
2018	16920	2717	3076	3847	4502	1.13	1.42	1.66	0.16
2019	17135	2937	4097	6751	8563	1.40	2.30	2.92	0.17
2020	17335	2722	3282	5058	8559	1.21	1.86	3.14	0.16
2021	19122	3367	5744	13728	13728	1.71	4.08	4.08	0.18
Average	**16466**	**3284**	**4917**	**8328**	**9446**	**1.48**	**2.49**	**2.84**	**0.20**

Table G3
Lebanon WWTP
Influent BOD Characterization

Average Annual

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average			Annual BOD per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16720	3059	6276	9209	9929	2.05	3.01	3.25	0.18
2018	16920	2653	3076	5506	5506	1.16	2.08	2.08	0.16
2019	17135	2821	4097	6751	8563	1.45	2.39	3.04	0.16
2020	17335	2634	3282	5313	8559	1.25	2.02	3.25	0.15
2021	19122	3046	5744	13728	13728	1.89	4.51	4.51	0.16
Average	**17446**	**2843**	**4495**	**8101**	**9257**	**1.56**	**2.80**	**3.22**	**0.16**

Summer Season

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average			Annual BOD per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16720	2356	3610	4084	4084	1.53	1.73	1.73	0.14
2018	16920	2589	3004	5506	5506	1.16	2.13	2.13	0.15
2019	17135	2702	3418	4328	5933	1.26	1.60	2.20	0.16
2020	17335	2486	3098	5313	5313	1.25	2.14	2.14	0.14
2021	19122	2756	3247	3887	3907	1.18	1.41	1.42	0.14
Average	**17446**	**2578**	**3275**	**4624**	**4949**	**1.28**	**1.80**	**1.92**	**0.15**

Winter Season

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average Dry Season Flow			Annual BOD per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16720	3820	6276	9209	9929	1.64	2.41	2.60	0.23
2018	16920	2717	3076	3847	4502	1.13	1.42	1.66	0.16
2019	17135	2937	4097	6751	8563	1.40	2.30	2.92	0.17
2020	17335	2722	3282	5058	8559	1.21	1.86	3.14	0.16
2021	19122	3367	5744	13728	13728	1.71	4.08	4.08	0.18
Average	**17446**	**3113**	**4495**	**7719**	**9056**	**1.42**	**2.41**	**2.88**	**0.18**

Table G4
Lebanon WWTP
Influent BOD Characterization

Average Annual

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average			Annual BOD per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16720	2605	6276	9209	9929	2.41	3.54	3.81	0.16
2018	16920	2498	3076	5506	5506	1.23	2.20	2.20	0.15
2019	17135	2555	4097	6751	8563	1.60	2.64	3.35	0.15
2020	17335	2454	3282	5313	8559	1.34	2.16	3.49	0.14
2021	19122	2866	5744	13728	13728	2.00	4.79	4.79	0.15
Average	**17446**	**2595**	**4495**	**8101**	**9257**	**1.72**	**3.07**	**3.53**	**0.15**

Summer Season

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average			Annual BOD per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16720	2201	3610	4084	4084	1.64	1.86	1.86	0.13
2018	16920	2463	3004	5506	5506	1.22	2.24	2.24	0.15
2019	17135	2473	3418	4328	5933	1.38	1.75	2.40	0.14
2020	17335	2425	3098	5313	5313	1.28	2.19	2.19	0.14
2021	19122	2789	3247	3887	3907	1.16	1.39	1.40	0.15
Average	**17446**	**2470**	**3275**	**4624**	**4949**	**1.34**	**1.89**	**2.02**	**0.14**

Winter Season

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average Dry Season Flow			Annual BOD per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16720	3820	6276	9209	9929	1.64	2.41	2.60	0.23
2018	16920	2717	3076	3847	4502	1.13	1.42	1.66	0.16
2019	17135	2937	4097	6751	8563	1.40	2.30	2.92	0.17
2020	17335	2722	3282	5058	8559	1.21	1.86	3.14	0.16
2021	19122	3367	5744	13728	13728	1.71	4.08	4.08	0.18
Average	**17446**	**3113**	**4495**	**7719**	**9056**	**1.42**	**2.41**	**2.88**	**0.18**

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ATTACHMENT H
Determine Peaking Factors and Per Capita Loads for TSS

Attachment H
Lebanon WWTP
Determine Peaking Factors and Per Capita Loads for TSS

Background

Peaking factors and per capita loads are necessary to characterize the influent wastewater and project TSS loads into the future. Projections are made assuming that these characteristics remain the same.

Evaluaion

Peaking Factors

Peaking factors are determined as follows:
-- Max Month -- Maximum 30 day average / Average Day
-- Max Week -- Maximum 7-day average / Average Day
-- Max Day -- Maximum day / Average Day

In this analysis, the loads for the period between January 2017 and December 2021 were evaluated. The max. month, max. week and max day values were developed for each of the seasonal conditions. The peaking factor for each year were calculated and an average of the peaking factors was determined for the years 2017 - 2021 as shown in Table 1. This analysis showed wet season loads being significantly higher than dry season loads. the analysis was also done using the median vs. the average in Table 2 and for the period between January 2017 and December 2021 in Tables 3 and 4.
The values for TSS peaking factors to be used in the study are summarized in the following table. The TSS values were slightly lower in the 2017 - 2021 period so these values were used as they show the most recent loads.

Period	Max. Month	Max. Week	Max. Day
Average Annual	1.8	3.0	3.5
Summer Season	1.8	2.5	2.9
Winter Season	1.5	2.2	2.8

Per Capita Loads

Per capita loads are calculated by taking the average day load and dividing it by the population for the City's service area for the respective year. These values were taken from Table 3 for the period 2017 - 2020.

Period	Per Capita Load
Average Annual	0.17
Summer Season	0.14
Winter Season	0.21

Conclusion

The per capita TSS loads typically range from 0.18 to 0.22 lbs/capita/day depending on the demographics of the service area. The average annual per capita TSS load is 0.18 lbs/capita/day.

The peaking factors and per capita loads for TSS were calculated and are summarized in the tables above. These peaking factors will be used for planning purposes. The per capita loads shown in the table above will be used for planning purposes.

Table H1
Lebanon WWTP
Influent TSS Characterization

Average Annual

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average			Annual TSS per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15,525	3643	7605	13920	20092	2.09	3.82	5.52	0.23
2011	15,565	3144	5545	13076	22822	1.76	4.16	7.26	0.20
2012	15,660	4152	7118	12190	13367	1.71	2.94	3.22	0.27
2013	15,690	3310	6125	9092	12866	1.85	2.75	3.89	0.21
2014	15,740	3386	9833	16269	16874	2.90	4.80	4.98	0.22
2015	15,740	3149	7401	9986	10588	2.35	3.17	3.36	0.20
2016	16,435	4262	12540	32435	39022	2.94	7.61	9.16	0.26
2017	16,720	3299	8684	14844	15733	2.63	4.50	4.77	0.20
2018	16,920	3257	4922	6594	9095	1.51	2.02	2.79	0.19
2019	17,135	2818	6503	10210	12955	2.31	3.62	4.60	0.16
2020	17,335	2623	3778	8472	8472	1.44	3.23	3.23	0.15
2021	19,122	3102	4079	5528	7278	1.31	1.78	2.35	0.16
Average	**16466**	**3345**	**7011**	**12718**	**15764**	**2.07**	**3.70**	**4.59**	**0.20**

Summer Season

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average			Annual TSS per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15525	2466	4713	6594	9573	1.91	2.67	3.88	0.16
2011	15565	2796	5267	13076	22822	1.88	4.68	8.16	0.18
2012	15660	3335	6012	9291	9291	1.80	2.79	2.79	0.21
2013	15690	3236	5489	9092	12866	1.70	2.81	3.98	0.21
2014	15740	2305	3393	6236	6323	1.47	2.71	2.74	0.15
2015	15740	2726	3619	5451	8513	1.33	2.00	3.12	0.17
2016	16435	2501	3906	6399	9957	1.56	2.56	3.98	0.15
2017	16720	2011	4432	5309	6095	2.20	2.64	3.03	0.12
2018	16920	2823	4761	6205	7306	1.69	2.20	2.59	0.17
2019	17135	2165	4891	4329	6161	2.26	2.00	2.85	0.13
2020	17335	2293	3465	8472	8472	1.51	3.69	3.69	0.13
2021	19122	2848	3843	5528	7278	1.35	1.94	2.56	0.15
Average	**16466**	**2625**	**4483**	**7165**	**9555**	**1.72**	**2.72**	**3.61**	**0.16**

Winter Season

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average Dry Season Flow			Annual TSS per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15525	4820	7605	13920	20092	1.58	2.89	4.17	0.31
2011	15565	3484	5545	10333	10333	1.59	2.97	2.97	0.22
2012	15660	4954	7118	12190	13367	1.44	2.46	2.70	0.32
2013	15690	3389	6125	7664	9520	1.81	2.26	2.81	0.22
2014	15740	4488	9833	16269	16874	2.19	3.63	3.76	0.29
2015	15740	3548	7401	9986	10588	2.09	2.81	2.98	0.23
2016	16435	5958	12540	32435	39022	2.10	5.44	6.55	0.36
2017	16720	4693	8684	14844	15733	1.85	3.16	3.35	0.28
2018	16920	3734	4922	6594	9095	1.32	1.77	2.44	0.22
2019	17135	3484	6503	10210	12955	1.87	2.93	3.72	0.20
2020	17335	2790	3778	4720	7428	1.35	1.69	2.66	0.16
2021	19122	3362	4079	5377	5841	1.21	1.60	1.74	0.18
Average	**16466**	**4059**	**7011**	**12045**	**14237**	**1.70**	**2.80**	**3.32**	**0.25**

Table H2
Lebanon WWTP
Influent TSS Characterization

Average Annual

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average			Annual TSS per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15525	2693	7605	13920	20092	2.82	5.17	7.46	0.17
2011	15565	2495	5545	13076	22822	2.22	5.24	9.15	0.16
2012	15660	3147	7118	12190	13367	2.26	3.87	4.25	0.20
2013	15690	2790	6125	9092	12866	2.20	3.26	4.61	0.18
2014	15740	2569	9833	16269	16874	3.83	6.33	6.57	0.16
2015	15740	2340	7401	9986	10588	3.16	4.27	4.52	0.15
2016	16435	2909	12540	32435	39022	4.31	11.15	13.41	0.18
2017	16720	2566	8684	14844	15733	3.38	5.78	6.13	0.15
2018	16920	2806	4922	6594	9095	1.75	2.35	3.24	0.17
2019	17135	2260	6503	10210	12955	2.88	4.52	5.73	0.13
2020	17335	2457	3778	8472	8472	1.54	3.45	3.45	0.14
2021	19122	2871	4079	5528	7278	1.42	1.93	2.53	0.15
Average	**16466**	**2659**	**7011**	**12718**	**15764**	**2.65**	**4.78**	**5.92**	**0.16**

Summer Season

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average			Annual TSS per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15525	2265	4713	6594	9573	2.08	2.91	4.23	0.15
2011	15565	2356	5267	13076	22822	2.24	5.55	9.69	0.15
2012	15660	2674	6012	9291	9291	2.25	3.47	3.47	0.17
2013	15690	2672	5489	9092	12866	2.05	3.40	4.82	0.17
2014	15740	1940	3393	6236	6323	1.75	3.22	3.26	0.12
2015	15740	2195	3619	5451	8513	1.65	2.48	3.88	0.14
2016	16435	1913	3906	6399	9957	2.04	3.34	5.20	0.12
2017	16720	1768	4432	5309	6095	2.51	3.00	3.45	0.11
2018	16920	2442	4761	6205	7306	1.95	2.54	2.99	0.14
2019	17135	1904	4891	4329	6161	2.57	2.27	3.24	0.11
2020	17335	1920	3465	8472	8472	1.80	4.41	4.41	0.11
2021	19122	2581	3843	5528	7278	1.49	2.14	2.82	0.13
Average	**16466**	**2219**	**4483**	**7165**	**9555**	**2.03**	**3.23**	**4.29**	**0.14**

Winter Season

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average Dry Season Flow			Annual TSS per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2010	15525	3272	7605	13920	20092	2.32	4.25	6.14	0.21
2011	15565	2681	5545	10333	10333	2.07	3.85	3.85	0.17
2012	15660	4075	7118	12190	13367	1.75	2.99	3.28	0.26
2013	15690	2909	6125	7664	9520	2.11	2.63	3.27	0.19
2014	15740	3160	9833	16269	16874	3.11	5.15	5.34	0.20
2015	15740	2435	7401	9986	10588	3.04	4.10	4.35	0.15
2016	16435	3586	12540	32435	39022	3.50	9.05	10.88	0.22
2017	16720	3550	8684	14844	15733	2.45	4.18	4.43	0.21
2018	16920	3151	4922	6594	9095	1.56	2.09	2.89	0.19
2019	17135	3011	6503	10210	12955	2.16	3.39	4.30	0.18
2020	17335	2595	3778	4720	7428	1.46	1.82	2.86	0.15
2021	19122	3167	4079	5377	5841	1.29	1.70	1.84	0.17
Average	**16466**	**3133**	**7011**	**12045**	**14237**	**2.23**	**3.77**	**4.45**	**0.19**

Table H3
Lebanon WWTP
Influent TSS Characterization

Average Annual

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average			Annual TSS per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16720	3299	8684	14844	15733	2.63	4.50	4.77	0.20
2018	16920	3257	4922	6594	9095	1.51	2.02	2.79	0.19
2019	17135	2818	6503	10210	12955	2.31	3.62	4.60	0.16
2020	17335	2623	3778	8472	8472	1.44	3.23	3.23	0.15
2021	19122	3102	4079	5528	7278	1.31	1.78	2.35	0.16
Average	**17446**	**3020**	**5593**	**9129**	**10707**	**1.84**	**3.03**	**3.55**	**0.17**

Summer Season

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average			Annual TSS per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16720	2011	4432	5309	6095	2.20	2.64	3.03	0.12
2018	16920	2823	4761	6205	7306	1.69	2.20	2.59	0.17
2019	17135	2165	4891	4329	6161	2.26	2.00	2.85	0.13
2020	17335	2293	3465	8472	8472	1.51	3.69	3.69	0.13
2021	19122	2848	3843	5528	7278	1.35	1.94	2.56	0.15
Average	**17446**	**2428**	**4278**	**5969**	**7062**	**1.80**	**2.49**	**2.94**	**0.14**

Winter Season

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average Dry Season Flow			Annual TSS per Capita
		Average	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16720	4693	8684	14844	15733	1.85	3.16	3.35	0.28
2018	16920	3734	4922	6594	9095	1.32	1.77	2.44	0.22
2019	17135	3484	6503	10210	12955	1.87	2.93	3.72	0.20
2020	17335	2790	3778	4720	7428	1.35	1.69	2.66	0.16
2021	19122	3362	4079	5377	5841	1.21	1.60	1.74	0.18
Average	**17446**	**3612**	**5593**	**8349**	**10210**	**1.52**	**2.23**	**2.78**	**0.21**

Table H4
Lebanon WWTP
Influent TSS Characterization

Average Annual

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average			Annual TSS per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16720	2566	8684	14844	15733	3.38	5.78	6.13	0.15
2018	16920	2806	4922	6594	9095	1.75	2.35	3.24	0.17
2019	17135	2260	6503	10210	12955	2.88	4.52	5.73	0.13
2020	17335	2457	3778	8472	8472	1.54	3.45	3.45	0.14
2021	19122	2871	4079	5528	7278	1.42	1.93	2.53	0.15
Average	**17446**	**2592**	**5593**	**9129**	**10707**	**2.19**	**3.61**	**4.22**	**0.15**

Summer Season

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average			Annual TSS per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16720	1768	4432	5309	6095	2.51	3.00	3.45	0.11
2018	16920	2442	4761	6205	7306	1.95	2.54	2.99	0.14
2019	17135	1904	4891	4329	6161	2.57	2.27	3.24	0.11
2020	17335	1920	3465	8472	8472	1.80	4.41	4.41	0.11
2021	19122	2581	3843	5528	7278	1.49	2.14	2.82	0.13
Average	**17446**	**2123**	**4278**	**5969**	**7062**	**2.06**	**2.87**	**3.38**	**0.12**

Winter Season

Year	Population	Historical Average Annual Loads (lbs/day)				Historical Peaking Factors From Average Dry Season Flow			Annual TSS per Capita
		Median	Maximum Month	Maximum Week	Maximum Day	Maximum Month	Maximum Week	Maximum Day	
2017	16720	3550	8684	14844	15733	2.45	4.18	4.43	0.21
2018	16920	3151	4922	6594	9095	1.56	2.09	2.89	0.19
2019	17135	3011	6503	10210	12955	2.16	3.39	4.30	0.18
2020	17335	2595	3778	4720	7428	1.46	1.82	2.86	0.15
2021	19122	3167	4079	5377	5841	1.29	1.70	1.84	0.17
Average	**17446**	**3095**	**5593**	**8349**	**10210**	**1.78**	**2.64**	**3.27**	**0.18**

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ATTACHMENT I
Current and Projected Wastewater Flows and Loads

Attachment I
Lebanon WWTP
Current and Projected Wastewater Flows and Loads

Background

The current wastewater characteristics were determined by taking the current capita based flows, loads and peaking factors and applying them to the 2021 PSU Certified Population of 19,122. Applying the 2021 population to the calculated wastewater characteristics resulted in the flows and loads shown in Table 1.

Peaking factors for flows and loads were developed from an evaluation of historical flows and loads between Janaury 2017 and December 2021. Peaking factors for maximum month, maximum week and maximum day were used in the analysis.

These results will be used to project flows and loads for 5-year periods from 2025 to 2045 and projected buildout in 2070.

Select Average Annual Growth Rate (AAGR)

Projected populations were taken from the Lin County Consolidated Population Study developed by PSU Population Research Center.

Year	2021	2025	2030	2035	2040	2045	2070
Projected Population	19,122	20,979	21,588	21,973	22,244	22,483	23,525

Wastewater Loads

The wastewater unit flows developed in Attachment A and unit loads for BOD and TSS developed in Attachments G & H were used to determine the current wastewater characteristics based on the 2021 population.

Variable	Ave Annual	Dry Weather	Dry Season	Wet Season	Units
Flow	202	92	115	230	gal/capita/day
BOD	0.16		0.15	0.18	lbs/capita/day
TSS	0.17		0.14	0.21	lbs/capita/day

Peaking Factors

The peaking factors developed for plant influent flows in Attachment F and influent BOD and TSS loads in Attachments G & H were used to determine the current wastewater characteristics based on the histoircal peaking factors and DEQ criteria. These flow were used to determine the 2021 flows.

Current Peaking Factors

Variable	Annual	Max Month	Max Week	Max Day	Peak Hr
Flow					
Ave Annual	2.47				
Dry Weather	1.00				
Dry Season	1.33	3.08	3.12	3.24	
Wet Season	3.62	5.21	7.25	10.31	16.62
BOD		1.60	2.80	3.20	
TSS		1.80	3.00	3.50	

The flow and loads will be different for new growth due to better construction materials. The unit flows and loads and peaking factors used for new growth are shown in the following tables. The BOD and TSS peaking factors are assumed to be the same as the current values.

Future Growth Wastewater Flows and Loads

	Ave Annual	Dry Weather	Dry Season	Wet Season	Units
Flow	202	92	115	230	gal/capita/day
BOD	0.17	0.17	0.17	0.17	lbs/capita/day
TSS	0.18	0.18	0.18	0.18	lbs/capita/day

New Growth Peaking Factors

TSS	Annual	Max Month	Max Week	Max Day	Peak Hr
Flow					
Ave Annual	2.20				
Dry Weather	1.00				
Dry Season	1.25	2.0	2.5	3.0	
Wet Season	2.50	3.0	3.5	4.0	6.0
BOD		1.6	2.8	3.2	
TSS		1.8	3.0	3.5	

Conclusion

The current influent wastewater characteristics based on the 2021 per capita flows and loads with the 2021 popuolation of 19,122 are summarized in Table 1. Tables 2 - 7 provide a summary of the projected flows and loads for each 5-year period from 2025 through 2045 and the assumed buildout values at 2070.

Table I1
Lebanon WWTP
2021 Influent Wastewater Flows and Loads

Flow (mgd)

Process Parameter	Ave Day	Max. Month	Max. Week	Max Day	Peak Hour
Average Annual	4.34				
Dry Weather	1.76				
Dry Season	2.33	5.42	5.49	5.70	
Wet Season	6.37	9.17	12.75	18.14	29.2

Biochemical Oxygen Demand (BOD_5)

Process Parameter	Average (lbs/day)	Max. Month (lbs/day)	Max. Week (lbs/day)	Max Day (lbs)
Average Annual	3,060	4,895	8,567	9,790
Dry Season	2,868	3,729	5,163	5,450
Wet Season	3,442	4,819	8,261	9,982

Total Suspended Solids (TSS)

Process Parameter	Average (lbs/day)	Max. Month (lbs/day)	Max. Week (lbs/day)	Max Day (lbs)
Average Annual	3,251	5,851	9,752	11,378
Dry Season	2,677	4,819	6,693	7,764
Wet Season	4,016	6,023	8,834	11,244

Table I2
Lebanon WWTP
2025 Influent Wastewater Flows and Loads

Flow (mgd)

Process Parameter	Ave Day	Max. Month	Max. Week	Max Day	Peak Hour
Average Annual	4.45				
Dry Weather	1.81				
Dry Season	2.40	5.52	5.62	5.85	
Wet Season	6.50	9.32	12.93	18.34	29.55

Biochemical Oxygen Demand (BOD_5)

Process Parameter	Average (lbs/day)	Max. Month (lbs/day)	Max. Week (lbs/day)	Max Day (lbs)
Average Annual	3,149	5,039	8,818	10,077
Dry Season	2,952	3,838	5,314	5,609
Wet Season	3,543	4,960	8,503	10,274

Total Suspended Solids (TSS)

Process Parameter	Average (lbs/day)	Max. Month (lbs/day)	Max. Week (lbs/day)	Max Day (lbs)
Average Annual	3,346	6,023	10,038	11,711
Dry Season	2,755	4,960	6,889	7,991
Wet Season	4,133	6,200	9,093	11,573

Table I3
Lebanon WWTP
2030 Influent Wastewater Flows and Loads

Flow (mgd)

Process Parameter	Ave Day	Max. Month	Max. Week	Max Day	Peak Hour
Average Annual	4.60				
Dry Weather	1.88				
Dry Season	2.48	5.65	5.78	6.05	
Wet Season	6.66	9.51	13.16	18.60	29.93

Biochemical Oxygen Demand (BOD_5)

Process Parameter	Average (lbs/day)	Max. Month (lbs/day)	Max. Week (lbs/day)	Max Day (lbs)
Average Annual	3,261	5,218	9,131	10,436
Dry Season	3,057	3,974	5,503	5,809
Wet Season	3,669	5,136	8,805	10,639

Total Suspended Solids (TSS)

Process Parameter	Average (lbs/day)	Max. Month (lbs/day)	Max. Week (lbs/day)	Max Day (lbs)
Average Annual	3,465	6,237	10,395	12,127
Dry Season	2,853	5,136	7,134	8,275
Wet Season	4,280	6,420	9,416	11,985

Table I4
Lebanon WWTP
2035 Influent Wastewater Flows and Loads

Flow (mgd)

Process Parameter	Ave Day	Max. Month	Max. Week	Max Day	Peak Hour
Average Annual	4.74				
Dry Weather	1.94				
Dry Season	2.56	5.78	5.94	6.24	
Wet Season	6.82	9.71	13.39	18.86	30.32

Biochemical Oxygen Demand (BOD_5)

Process Parameter	Average (lbs/day)	Max. Month (lbs/day)	Max. Week (lbs/day)	Max Day (lbs)
Average Annual	3,373	5,397	9,445	10,794
Dry Season	3,162	4,111	5,692	6,008
Wet Season	3,795	5,313	9,107	11,005

Total Suspended Solids (TSS)

Process Parameter	Average (lbs/day)	Max. Month (lbs/day)	Max. Week (lbs/day)	Max Day (lbs)
Average Annual	3,584	6,451	10,752	12,544
Dry Season	2,951	5,313	7,379	8,559
Wet Season	4,427	6,641	9,740	12,396

Table I5
Lebanon WWTP
2040 Influent Wastewater Flows and Loads

Flow (mgd)

Process Parameter	Ave Day	Max. Month	Max. Week	Max Day	Peak Hour
Average Annual	4.88				
Dry Weather	2.00				
Dry Season	2.64	5.91	6.10	6.43	
Wet Season	6.98	9.90	13.61	19.12	30.71

Biochemical Oxygen Demand (BOD_5)

Process Parameter	Average (lbs/day)	Max. Month (lbs/day)	Max. Week (lbs/day)	Max Day (lbs)
Average Annual	3,485	5,576	9,758	11,152
Dry Season	3,267	4,247	5,881	6,208
Wet Season	3,921	5,489	9,410	11,370

Total Suspended Solids (TSS)

Process Parameter	Average (lbs/day)	Max. Month (lbs/day)	Max. Week (lbs/day)	Max Day (lbs)
Average Annual	3,703	6,665	11,109	12,960
Dry Season	3,049	5,489	7,624	8,843
Wet Season	4,574	6,861	10,063	12,808

Table I6
Lebanon WWTP
2045 Influent Wastewater Flows and Loads

Flow (mgd)

Process Parameter	Ave Day	Max. Month	Max. Week	Max Day	Peak Hour
Average Annual	5.02				
Dry Weather	2.07				
Dry Season	2.72	6.04	6.26	6.63	
Wet Season	7.14	10.09	13.84	19.37	31.1

Biochemical Oxygen Demand (BOD_5)

Process Parameter	Average (lbs/day)	Max. Month (lbs/day)	Max. Week (lbs/day)	Max Day (lbs)
Average Annual	3,597	5,756	10,072	11,511
Dry Season	3,372	4,384	6,070	6,408
Wet Season	4,047	5,666	9,713	11,736

Total Suspended Solids (TSS)

Process Parameter	Average (lbs/day)	Max. Month (lbs/day)	Max. Week (lbs/day)	Max Day (lbs)
Average Annual	3,822	6,880	11,466	13,377
Dry Season	3,148	5,666	7,869	9,128
Wet Season	4,721	7,082	10,387	13,220

Table I6
Lebanon WWTP
2070 (Buildout) Influent Wastewater Flows and Loads

Flow (mgd)

Process Parameter	Ave Day	Max. Month	Max. Week	Max Day	Peak Hour
Average Annual	5.23				
Dry Weather	2.16				
Dry Season	2.84	6.23	6.50	6.92	
Wet Season	7.38	10.38	14.17	19.76	31.67

Biochemical Oxygen Demand (BOD_5)

Process Parameter	Average (lbs/day)	Max. Month (lbs/day)	Max. Week (lbs/day)	Max Day (lbs)
Average Annual	3,764	6,022	10,539	12,045
Dry Season	3,529	4,587	6,352	6,705
Wet Season	4,235	5,928	10,163	12,280

Total Suspended Solids (TSS)

Process Parameter	Average (lbs/day)	Max. Month (lbs/day)	Max. Week (lbs/day)	Max Day (lbs)
Average Annual	3,999	7,199	11,998	13,997
Dry Season	3,294	5,928	8,234	9,551
Wet Season	4,940	7,410	10,869	13,833

Appendix F: Outfall Mixing Zone Study

Memorandum

To: Mark Hamlin, Tim McFetridge **Date:** February 4, 2008

From: Steve Schnurbusch

Subject: City of Lebanon Mixing Zone Study

The City of Lebanon submitted a May 29, 2007 mixing zone study to the Department for review. I reviewed the study and provided comments to Mark Hamlin in a June 14, 2007 memo. These comments were forwarded to the City and their consultant for additional analysis. The City of Lebanon submitted an updated mixing zone study performed by West Yost Associates dated December 17, 2007.

I have reviewed the study and agree with the analyses performed except for a few items discussed in this memo. This memo provides a summary of the dilution needed for the permit writer, provides clarification for some of the analyses performed, and incorporates slight modifications to some of the results.

Summary of Dilution Results for Permitting Purposes

Table 1 below provides a summary of the CORMIX modeling dilution results for performing a reasonable potential analysis and developing water quality based effluent limits. The results are based on the 2024 year design flows. These dilution results can be used directly for performing a reasonable potential analysis and for establishing effluent limits. Additional modeling can be performed at existing or 5-yr projected flow rates if necessary for determining reasonable potential for the next permit renewal.

Table 1: Dilutions Applicable for Reasonable Potential Analysis

Design Flow	Criteria	Location	Dilution
ADWF – 3 mgd	Acute	ZID	18
ADWF – 3 mgd	Chronic	MZ	27
ADWF – 3 mgd	Human Health - carcinogen	MZ	31
AAF[1] – 5.5 mgd	Human Health – non carcinogen	MZ	38
ADWF – 3 mgd	Chronic – proposed ammonia criteria	MZ	31

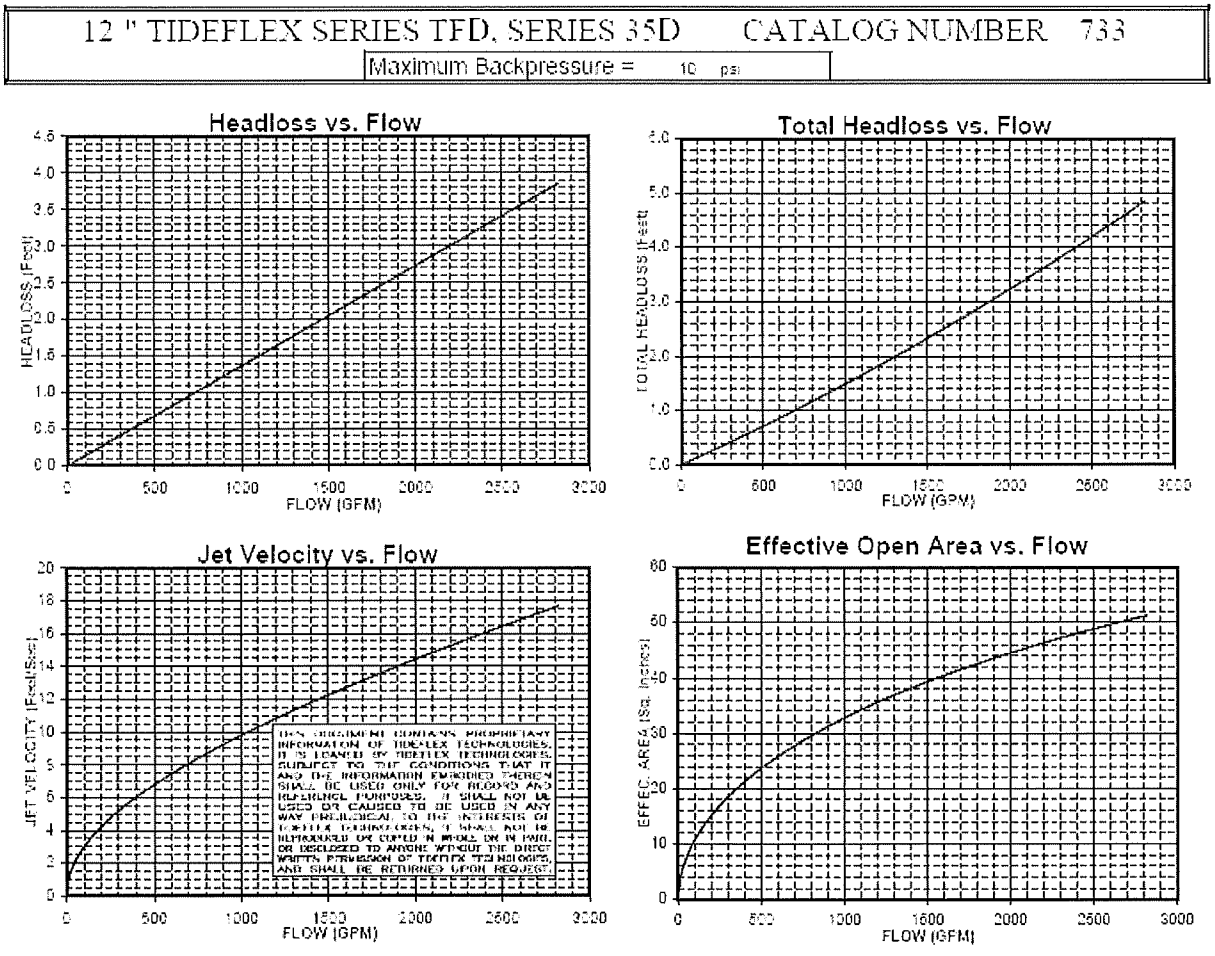
[1]AAF = Average Annual Flow (average of ADWF and AWWF)

Modeling Discussion

The Department found there were some slight modifications that needed to be made to the mixing zone analysis regarding the dilutions at the edge of the ZID, dilutions applicable to our human health criteria, and some clarification on methodology regarding the Tideflex valves.

Tideflex Valves

The diffuser consists of eleven 12-inch Tideflex valves. The opening of the Tideflex valves vary based on the flow through each port. Entering a value of 12 inches into CORMIX for the port diameter would not be correct. West Yost did not enter 12 inches as the port diameter, but they did not provide their methodology for calculating the effective port diameter. After calling West Yost Associates, they provided me with the rating curve sheet for the 12-inch Tideflex valves. The flow through each port was calculated and the rating sheet below was used to determine the jet velocity. This jet velocity was then used to back-calculate the port diameter. This back-calculation was necessary because CORMIX requires port diameter as an input parameter. For example: At an effluent flow rate of 3 mgd this corresponds to an individual port flow rate of 0.27 mgd (3 mgd/11 ports). This is equivalent to 189 gpm. Using the "Jet Velocity vs. Flow" chart below, at 189 gpm the port jet velocity is about 4 ft/s. If you divide the flow rate of 189 gpm (0.42 ft/s) by the velocity you obtain the effective port area of 0.105 ft^2. You can then calculate the effective diameter of 0.37 ft. This diameter would be entered into CORMIX for any 3 mgd effluent scenarios. At 5 mgd the effective port diameter is about 0.41 ft. These values match those provided in the CORMIX input files provided in the mixing zone study.



ZID Dilutions

The Department's Mixing Zone Internal Management Directive states the centerline dilution should be applied at the edge of the ZID where the average dilution should be applied at the edge of the mixing zone.

For this discharge scenario, the CORMIX output provides the average bulk dilution. The CORMIX manual provides a factor to divide the average dilution by to obtain an approximate centerline dilution. This is discussed on page 74 of the CORMIX User's Manual. The factor is 1.3. The mixing zone study submitted by the City did not make this adjustment to calculate the centerline dilution. I applied this factor to the dilutions reported in the mixing zone study to obtain an estimate of the centerline dilutions at the edge of the ZID. The bulk average and centerline dilutions are tabulated in Table 2.

Human Health Criteria

The carcinogenic human health criteria are to be applied under average effluent flow and stream flow conditions. Table 4-2 of the Mixing Zone IMD – Part II states the annual average effluent flow rate and the harmonic mean flow of the receiving stream should be used to determine dilutions at the edge of the mixing zone to be applied for the carcinogenic human health criteria.

The mixing zone study only provided the dilution for the average dry weather flow at the stream's harmonic mean flow. The annual average effluent flow rate should have been used. I calculated the annual average effluent flow rate by simply averaging the average dry weather flow (3 mgd) and the average wet weather flow (8 mgd). The resulting annual average flow (AAF) is 5.5 mgd. I ran CORMIX to simulate this effluent flow rate under harmonic mean flow conditions.

Table 2: Summary of Dilution Results

Effluent Parameter		Ambient & Diffuser Parameters					Dilution		
Design Flow/Yr (effluent)	Qe	Design Flow (river)	Qr	Velocity	Depth	Effective Port Diameter	ZID (bulk average)	ZID (centerline)	MZ
	mgd		cfs	ft/s	ft	ft			
ADWF(2024)	3	1Q10 7Q10	450	1.15	3.5	0.37	23	18	27
ADWF(2024)	3	30Q5	540	1.25	3.75	0.37	26	20	31
AAF(2024)	5.5	Harmonic	1380	1.7	5.6	0.41	na	na	38

All dilution results submitted in the City's mixing zone report are acceptable except that a factor of 1.3 needs to be applied to the ZID dilutions to estimate the minimum centerline dilution.

W E S T
Y O S T
& ASSOCIATES
Consulting Engineers

TRANSMITTAL MEMORANDUM

DATE: December 17, 2007 Project No. 516-01-06-06

TO: Mark Hamlin, DEQ
Steve Schnurbusch, DEQ

FROM: Mark Zinniker, West Yost Associates

ENCLOSURES:

Updated Mixing Zone Analysis

COMMENTS:

Mark and Steve – Please find attached two copies of the updated technical memorandum (TM) on our mixing zone analysis for the City of Lebanon's effluent discharge system improvements. Also included for your reference are copies of Steve's review comments on the first draft of the mixing zone analysis. Following is a brief summary of the changes made to the TM and the diffuser design in response to Steve's comments:

Instream Velocity: We've added an explanation of how ambient velocities were calculated for the various river flow conditions using a discharge curve equation (based on Manning's equation) that was calibrated to field data collected on the river cross section survey date.

Effluent Flow Rates: We've added dilution estimates for the maximum month dry weather flow to supplement the dilution estimates for the average dry weather flow. In addition, dilution estimates were developed for winter conditions. Since the average wet weather flow is very similar to the max month dry weather flow, the winter scenario modeling was conducted for the maximum month wet weather flow.

Mixing Zone Modeling and Dilution Values: This version of the TM provides modeling results based on the CORMIX 2 prediction module. The diffuser was modeled as a fanned multiport diffuser. The resulting ZID dilution values are significantly higher than the values reported in the previous draft using the CORMIX 1 prediction module. The length of the diffuser was increased as recommended to ensure that the discharge plume mixes with 25% of the river.

Environmental Mapping: We've included confirmation that ODFW was consulted during the design and permitting process for the project. The City has recently received permitting approvals for the in-river work following consultations with ODFW and other regulatory agencies.

Please let me know if you have any additional questions or comments regarding the mixing zone analysis. The City is planning to advertise the improvements for bid early in the new year.

132 East Broadway, Suite 431 Eugene, Oregon 97401 Phone 541 431-1280 Fax 541 431-1290 email: mail@westyost.com

Memorandum

To: Mark Hamlin **Date:** June 14, 2007

From: Steve Schnurbusch

Subject: Lebanon Mixing Zone Study Review

I reviewed the Lebanon mixing zone study (technical memorandum) dated May 29, 2007. In general it is a well written study and contains most of the key elements needed in a mixing zone study. However, some critical elements were left out and additional CORMIX modeling needs to be performed. This should be done by the consultant because it is likely to affect the diffuser design.

Instream Velocity

The instream velocities for various flow conditions are contained in Table 2. However, the methodology for calculating these velocities is not provided. The river was surveyed on 8/15/2005 and an average velocity of 1.5 ft/sec was apparently measured and this was extrapolated to other flow rates. The data used to determine the average velocity on 8/15/2005 should be provided along with the methodology for estimating velocities for the other flow rates.

Effluent Flow Rates

Table 4 lists the projected effluent flow rates. The ADWF of 3 mgd was used for the low flow conditions. The ADWF should be used to assess dilutions at the edge of the mixing zone during dry weather periods. The maximum month dry weather flow of 7 mgd should be used to assess dilutions at the edge of the ZID during dry weather conditions.

In addition, off-design cases that have the potential to provide lower dilutions should be considered. Sometimes, during winter conditions, the higher stream velocity carries the plume quickly downstream past the edge of the mixing zone without having time to obtain good mixing. Modeling winter time conditions may also need to be considered if there are pollutants that have the potential to have seasonal limits (i.e. ammonia).

Mixing Zone Modeling

CORMIX 1 was used to model the multiport discharge. CORMIX 1 is appropriate for single port outfalls and for testing results of multiport diffusers. CORMIX 1 was apparently used because the diffuser design called for a "fanned" diffuser orientation and the modelers thought it was better to model these ports individually and combine the individual ports graphically to determine the overall width of the plume. This was done iteratively to determine the best diffuser design to maximize mixing with 25% of the river. However this methodology does not take into account the behavior of the diffuser as a whole. Co-flowing multiport diffusers can produce their own acceleration zones and the plume actually becomes narrower in the near field before widening out. In relatively shallow streams, the momentum created by the diffuser jets creates a condition where the plume is completely mixed vertically. This type of behavior has

been observed in laboratory settings. A single port simulation will not predict this type of behavior.

CORMIX II should be used to model this discharge. There is an option to indicate a "fanned" multiport diffuser which would be appropriate for this outfall design. I performed a model simulation for one of the 3 mgd scenarios and found the model predicts an unstable discharge (as is typical in riverine conditions) that is completely fully mixed vertically in the near-field. However, with the 11-port diffuser, the dilution is less than predicted using the CORMIX 1 methodology because the plume does not spread out laterally as far. Therefore, the diffuser design should be revisited and made longer if the desire is to maximize the 25% dilution at 100 feet downstream.

Interim Dilution Values

It appears the intention of the City is to maximize the dilution by designing a diffuser to mix with 25% of the stream width. If this is the case, the final dilution can be calculated using a simple mass balance. This methodology can be used because the effluent becomes completely vertically mixed in the near field. For example, under the summer low flow condition, the effluent flow is 3 mgd (4.64 cfs) and the river flow is 450 cfs. The dilution with 25% of the river can be calculated using the following equation:

$$S = \frac{0.25Q_r + Q_e}{Q_e} = \frac{0.25(450) + 4.64}{4.64} = 25$$

This mass balance calculation only applies to the combination of ambient and effluent flow conditions the diffuser was designed around. Once the diffuser design is decided upon, the other scenarios would need to be modeled to determine the actual dilution achieved with the diffuser.

Environmental Mapping

Oregon Department of Fish and Wildlife (ODFW) should be consulted regarding the placement of the diffuser and the location of the mixing zone for potential impacts to fish and aquatic life. The mixing zone study discusses salmonid use in the area and concludes the habitat in the area is marginal. This should be verified by ODFW.

WEST YOST
ASSOCIATES
Consulting Engineers

DRAFT

TECHNICAL MEMORANDUM

DATE: December 17, 2007 Project No.: 516-01-06-06

TO: Mr. Rob Emmons
City of Lebanon

FROM: Mark Zinniker, West Yost Associates
Laura Kegley, West Yost Associates

SUBJECT: WWTP Effluent Discharge System Improvements
Modified Outfall Diffuser Design and Mixing Zone Study

INTRODUCTION

The City of Lebanon (City) has designed improvements to the wastewater treatment plant's (WWTP's) outfall system as required by the City's National Pollutant Discharge Elimination System (NPDES) permit and Mutual Agreement and Order (MAO) with the Department of Environmental Quality (DEQ). The designed improvements consist of two components: an effluent infiltration system in an upland area on the east bank of the South Santiam River and an outfall diffuser within the river. However, recent developments have caused the City to modify the implementation plan for the project. A combination of site acquisition difficulties for the infiltration system, cost escalation for the river crossing, and a workable thermal load allocation in the Willamette Basin TMDL have given the City reason to postpone the infiltration component of the design. As a result, the City's current strategy is to implement the effluent discharge system improvements in phases. Under the phased approach, construction of the outfall diffuser improvements will be completed initially and construction of the effluent infiltration system will be postponed until a later date.

The original outfall diffuser design was developed under the premise that it would provide direct discharge to the river only when effluent infiltration was not possible. Given the anticipated operational requirements for the direct river outfall, the diffuser was designed to operate when both the WWTP effluent flows and river flows are high. The original diffuser design that was selected for these conditions incorporated two ports. However, this outfall diffuser design does not maximize dilution during the critical low river flow conditions. Given the proposed phased approach and the uncertainty regarding the timing for the second phase, the new outfall diffuser will need to be capable of maximizing dilution during critical low river flow conditions. To achieve maximum mixing under these conditions, modifications to the outfall diffuser design were necessary.

This Technical Memorandum (TM) presents the revised design concept for a modified outfall diffuser in conjunction with a mixing zone study. The mixing zone study was prepared in

accordance with the DEQ's current draft of the Internal Management Directive (IMD) for mixing zone studies. The primary objectives of the TM are as follows:

- To develop a diffuser design that will maximize mixing of the effluent discharged to the South Santiam River during critical low river flow conditions.
- To provide the DEQ with detailed information on the anticipated dilution that will be provided by the diffuser for use in drafting a new NPDES permit for the City.

The methodology for conducting the mixing zone study consisted of the following primary tasks:

- Review the WWTP's existing regulatory mixing zone (RMZ) and mixing zone sizing restrictions.
- Review and analyze important South Santiam River characteristics during the critical river flow periods.
- Prepare a summary of environmental conditions for the local reach of the South Santiam River.
- Review and analyze the WWTP's effluent discharge characteristics during critical river flow periods.
- Develop a recommended diffuser design concept.
- Conduct computer modeling of the mixing between the WWTP's effluent and the river for alternative diffuser configurations under critical flow conditions.
- Estimate the dilution provided by the recommended diffuser design within the RMZ under critical flow conditions.
- Identify the hydraulic capacity of the multi-port diffuser for initial and future port sizing.

The following sections discuss each mixing zone analysis tasks.

REGULATORY MIXING ZONE AND MIXING ZONE SIZING RESTRICTIONS

The regulatory mixing zone (RMZ) for the existing outfall system is defined in the City's NPDES permit for the WWTP. The RMZ consists of a zone of initial dilution (ZID) which extends 10 feet from the point of discharge and the chronic mixing zone (CMZ) which extends 100 feet downstream from the point of discharge. Pollutants in the discharge plume must comply with acute toxicity limits at the ZID boundary and chronic toxicity limits at the CMZ boundary. The configuration of the South Santiam River at the outfall location and the RMZ for the WWTP is shown on the aerial photo in Figure 1.

The DEQ's Internal Management Directive (IMD) documents for mixing zones provide guidelines for the shape and size of an RMZ. These guidelines recommend that 75-percent of the river's cross-sectional area or volume of flow be maintained free of effluent to allow for fish passage. As a result, only 25-percent percent of the river should be utilized for mixing within the RMZ. Given these guidelines, the outfall diffuser should be designed such that the discharge

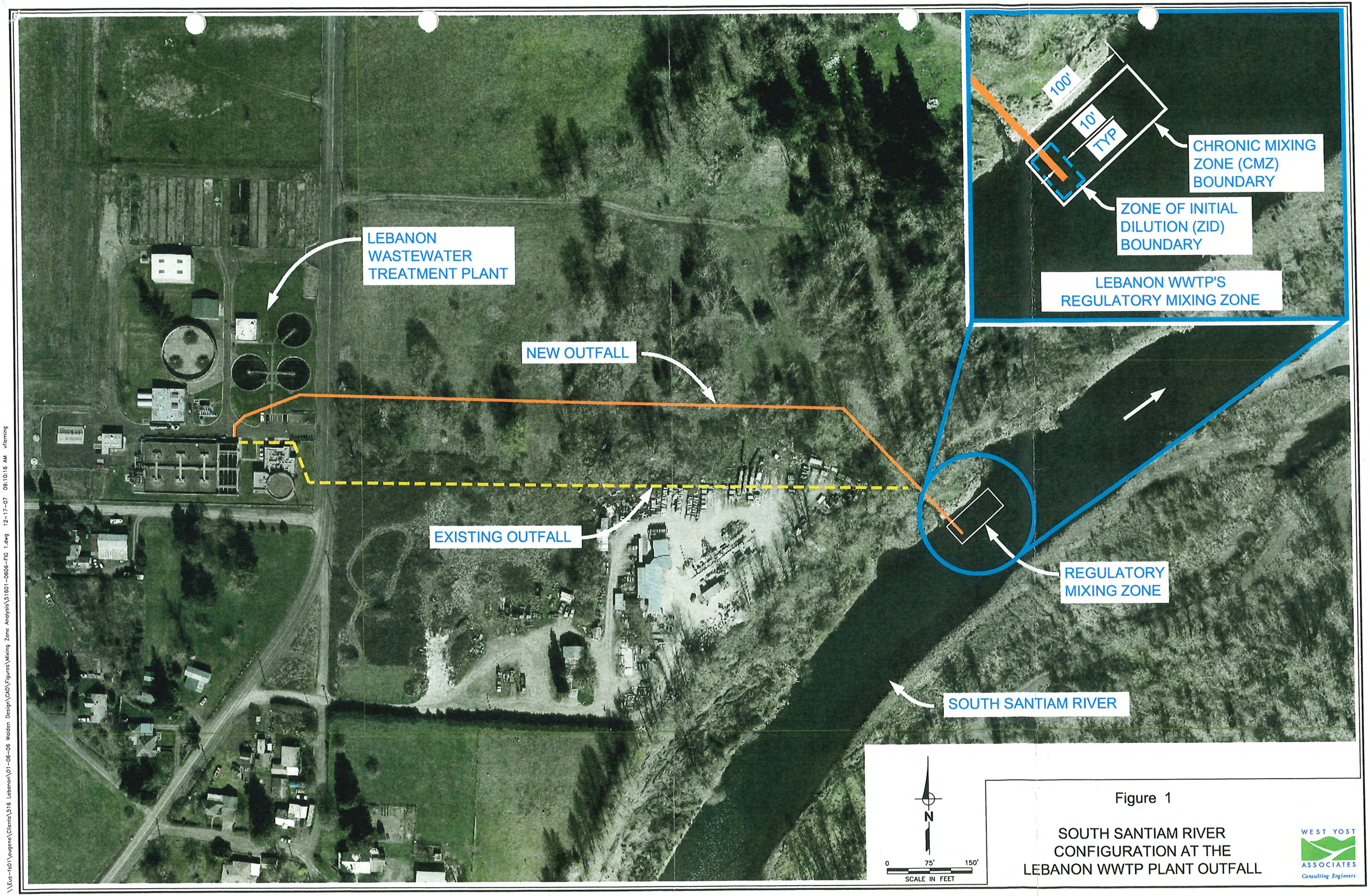


Figure 1

SOUTH SANTIAM RIVER
CONFIGURATION AT THE
LEBANON WWTP PLANT OUTFALL

WEST YOST ASSOCIATES
Consulting Engineers

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The view looking upstream from the existing outfall during the dry weather season.

plume incorporates no more than 25-percent of the river flow at the CMZ boundary under the critical low river flow conditions and build-out effluent flows. Such a diffuser would maximize the mixing that is available in the South Santiam River.

The South Santiam River flows in a relatively wide, shallow channel during the critical low flow period. Field survey work conducted in support of the new outfall system design included developing river channel cross sections. Figure 2 shows the plan and profile of the river channel at the outfall location. The river width under critical low flow conditions is approximately 130 feet, neglecting the shallowest water at the edges. By locating the outfall diffuser within the deeper water of the river, a plume width of approximately 30-feet at the CMZ boundary (100-feet downstream of the diffuser) will ensure mixing with approximately 25-percent of the river flow. The required plume width is less than 25-percent of the river width since the diffuser will be located in the deeper portion of the river.

SOUTH SANTIAM RIVER CHARACTERISTICS

The following sections address the unique characteristics of the South Santiam River at the location of the Lebanon WWTP outfall that affect the diffuser design development and mixing zone analysis.

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River Flows

Flows in the South Santiam River are monitored daily by the United States Geological Survey (USGS) at their gauging station River Mile 23.3, near Waterloo. The WWTP discharges to the river approximately six miles downstream of the Waterloo gauging station, at River Mile 17.4. In between the Waterloo gauging station and the Lebanon WWTP outfall, there is a diversion from the South Santiam River into the Lebanon-Santiam Canal (River Mile 20.6) that the USGS monitors. An input to the South Santiam River at Hamilton Creek (River Mile 21.0) is not gauged and therefore cannot be quantified, but is conservatively assumed to be negligible during critical low flow periods. The configuration of the South Santiam River near Lebanon is illustrated in Figure 3. The figure shows the location of the outfall relative to the Lebanon-Santiam Canal diversion and the Hamilton Creek input.

WYA analyzed the historical records for the gauging stations to identify the critical low flow rates for the river at the outfall location. The statistical analysis for data from the Waterloo Station was limited to the period after 1968 to account for the effects of flow regulation at Foster Dam and Green Peter Reservoir. Flow records for the Lebanon-Santiam Canal are available from 1993 to present.

The critical flow conditions identified are defined as follows:

- 1Q10: The lowest average single day low flow with a recurrence interval of 10 years. The assessment of acute toxicity at the ZID boundary is conducted using the 1Q10 flow.
- 7Q10: The lowest average seven-day low flow with a recurrence interval of 10 years. The assessment of chronic toxicity at the CMZ boundary is conducted using the 7Q10 flow.
- 30Q5: The lowest average 30-day low flow with a recurrence interval of 5 years. The assessment of long-term human health impacts for non-carcinogenic criteria are conducted using the 30Q5 flow.
- Harmonic Mean Flow: The long term mean flow value calculated by dividing the number of daily flows by the sum of the reciprocals of those daily flows. The assessment of long-term human health impacts for carcinogenic criteria are conducted using the harmonic mean flow.

The critical flow conditions for the South Santiam River are summarized below in Table 1. Since the 1Q10 and 7Q10 flows are so close and the timing of the statistical flow conditions relative to each other is uncertain, the flow at the outfall site can be conservatively estimated to be 450 cfs for both conditions. The 30Q5 flow rate is estimated to be 540 cfs at the outfall site, and the harmonic mean flow is estimated to be 1,380 cfs. Also included in Table 1 is the maximum month wet weather flow of 6,000 cfs which would be expected to correspond to the maximum month wet weather flow rate at the Lebanon WWTP.

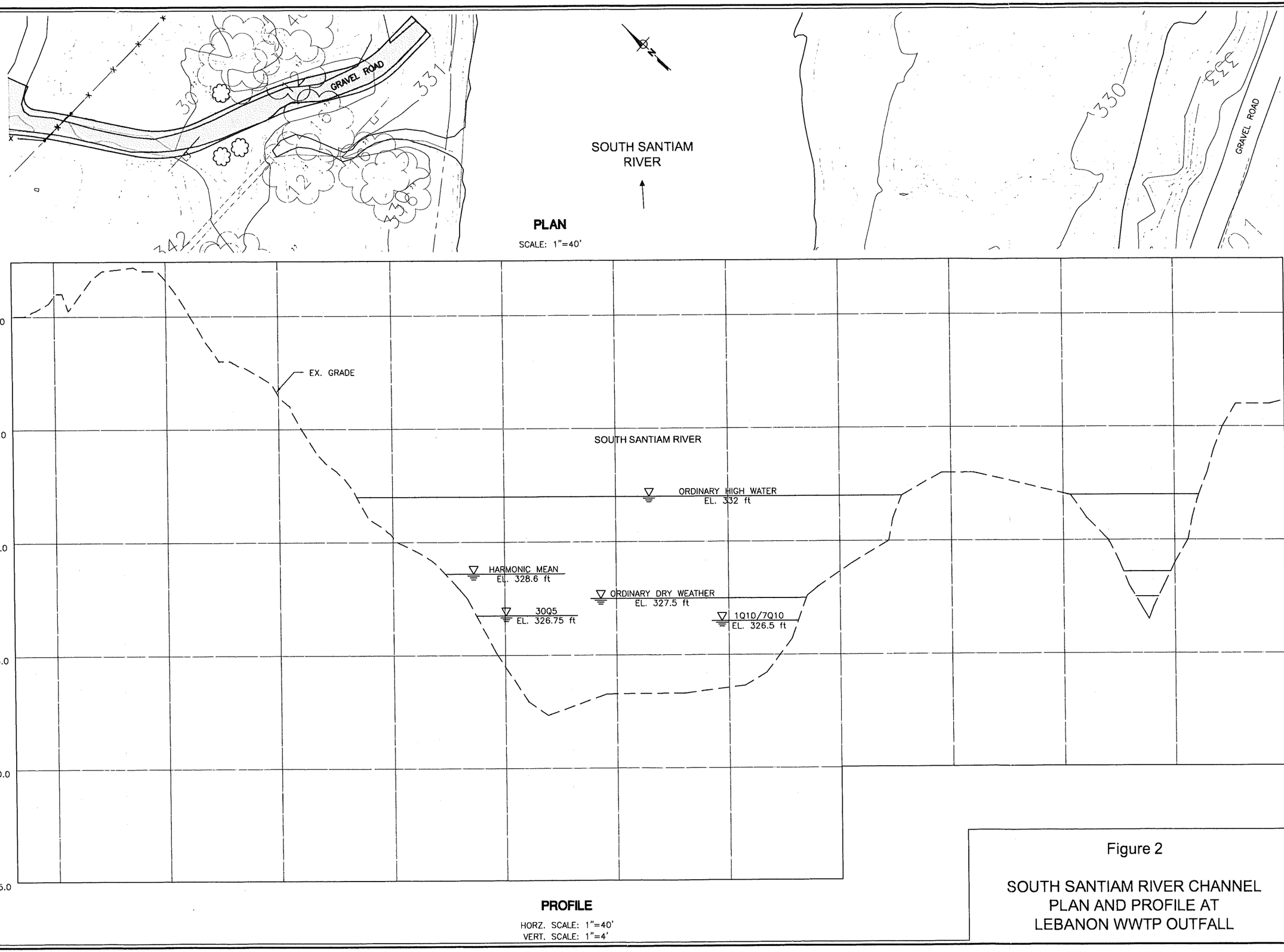

Figure 2

SOUTH SANTIAM RIVER CHANNEL
PLAN AND PROFILE AT
LEBANON WWTP OUTFALL

WEST YOST ASSOCIATES
Consulting Engineers

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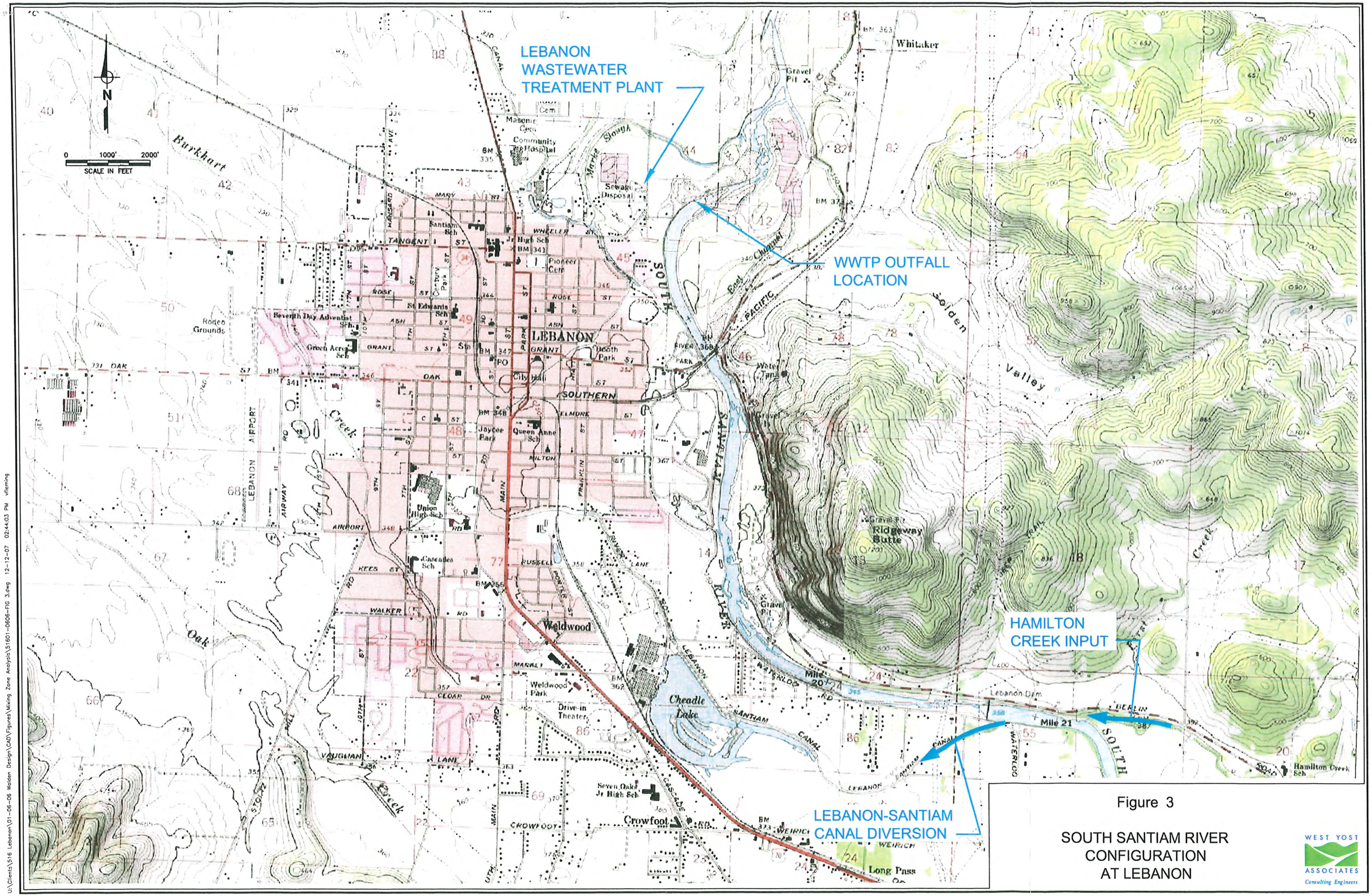

Figure 3

SOUTH SANTIAM RIVER
CONFIGURATION
AT LEBANON

WEST YOST ASSOCIATES
Consulting Engineers

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Table 1. Critical Low Flow Conditions for the South Santiam River

Flow Condition	Waterloo Gauging Station Flow Rate, cfs	Lebanon-Santiam Canal Gauging Station Flow Rate, cfs	Estimated Flow Rate at WWTP Outfall, cfs
1Q10	490	27	450
7Q10	511	31	450
30Q5	584	45	540
Harmonic Mean	1,460	77	1,380
Maximum Month Wet Weather	-	-	6,000

River Depths and Velocities

Water depths and ambient river velocities were calculated based on the field data collected during the surveying of river cross sections in combination with river flow data from the upstream USGS gauging stations. The average velocity for the river on the day of the survey was calculated using the cross-sectional area field data and the flow rates reported by the USGS at their gauging stations on the survey date. This field data was used to calibrate an open channel flow equation (based on Manning's equation) for the local reach of the river. This equation relates the average water depth to the flow rate of the river. Using the information from this equation in conjunction with the surveyed cross section of the river, the river width, average water depths, and average ambient river velocities were calculated for each of the critical river flow conditions. The resulting data are summarized in Table 2. This information provides important background data for the mixing zone modeling under each of the critical river flow conditions.

Table 2. Estimated River Width, Depth, and Velocity by Flow Condition

Flow Condition	Estimated River Flow Rate, cfs	Approximate River Width, feet	Average Water Depth, feet	Average Water Velocity, feet/second
Survey (8/15/05)	800	140	4.0	1.4
1Q10	450	130	3.0	1.2
7Q10	450	130	3.0	1.2
30Q5	540	135	3.2	1.3
Harmonic Mean	1,380	150	5.1	1.7
Maximum Month Wet Weather	6,000	190	8.0	3.9

River Temperature

The City monitored river temperatures at the WWTP outfall location as part of the development of their 2002 Temperature Management Plan. The resulting data for the upstream river temperature are illustrated in Figure 4. The highest 7-day average of the daily maximum river temperature was 66.8 degrees Fahrenheit (19 degrees Celsius). Since the high river temperature period is likely to coincide with the critical low river flow period, this ambient river temperature was used for the mixing zone modeling.

Channel Roughness

The South Santiam River in the vicinity of the Lebanon WWTP is characterized by gravel/cobble substrates and flows through a series of riffles, runs, and pools. Water depths during low river flow periods vary from a few inches over riffles to several feet in the runs and pools. The City's outfall and RMZ are located in one of the runs where the low flow water depths are always several feet. These characteristics provide an indication of an appropriate Manning's "n" coefficient for characterizing the roughness of the channel. For mixing zone modeling, there are a variety of Manning's "n" coefficients that are recommended according to the different types of channels, as summarized in Table 3. Based on the DEQ's guidance for mixing zone analyses, it is estimated that a Manning's "n" coefficient of 0.035 is an appropriate value for low flow conditions in the South Santiam River at this location.

Table 3. Summary of Typical Manning's "n" Coefficients

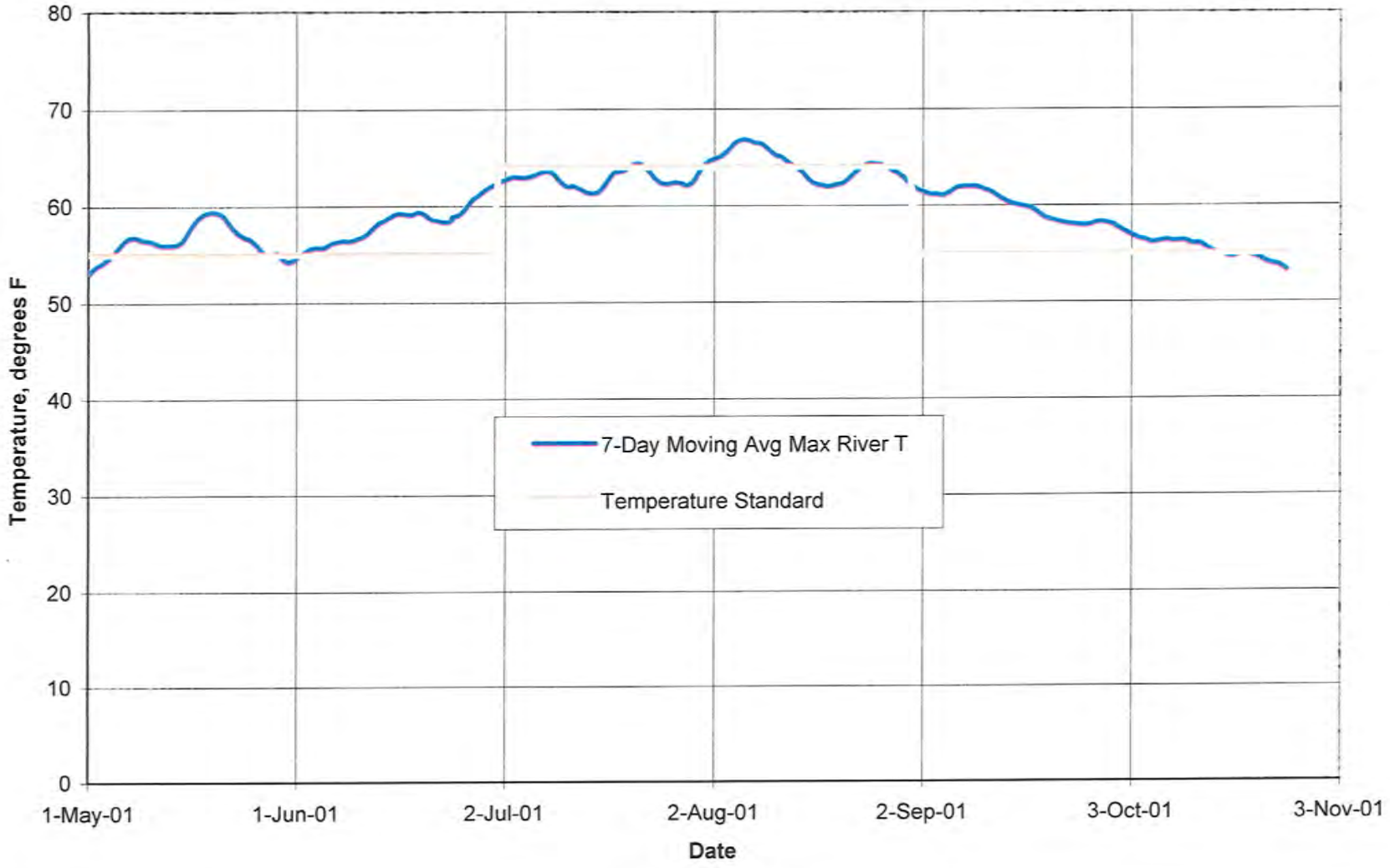
Channel Type	Manning's "n" Value
Smooth Earth, No Weeds	0.020
Smooth Earth, Some Stones and Weeds	0.025
Clean, Straight Natural Rivers	0.025 – 0.030
Winding River, Pools and Shoals	0.033 – 0.040
Winding River, Very Weedy, Overgrown	0.050 – 0.15

SOUTH SANTIAM RIVER ENVIRONMENTAL CONDITIONS

A review of the environmental conditions within the vicinity of the City's outfall is useful for identifying the specific habitats and any critical resources or beneficial uses within the segment of the river receiving the discharge. Figure 1 provided an aerial view of the area surrounding the Lebanon WWTP outfall and shows the extent of the City's RMZ.

Environmental reviews were conducted during the preliminary design phase of the project to identify potential environmental constraints on the design of improvements. The environmental studies indicate that there are two listed salmonid species in the project area, Steelhead and Chinook Salmon. These species are listed as Threatened under the Endangered Species Act. The studies indicate that this reach of the South Santiam River serves primarily as a migration corridor for adult and juvenile salmonids. It is thought that rearing of juvenile salmonids may also take

Figure 4. 7-Day Moving Average Upstream River Temperature



place in the project area; however, use is limited by the relatively high velocity main stem flows as well as the absence of backwater areas and complex cover. The potential for spawning is considered low since most fish spawn in smaller creeks and tributaries in the upper South Santiam watershed. Our review of the local area at the City's outfall and RMZ indicated that there is not any off-channel habitat, physical structures that would attract fish, or other types of refugia. Therefore, the fish spawning/rearing habitat in the general vicinity of the outfall appears to be marginal. The Oregon Department of Fish and Wildlife (ODFW) was contacted regarding the outfall improvements during the development of the environmental reports for the project and the elements of the design were adjusted according to comments received. The in-river work has since received state and federal permitting approvals following additional consultation with ODFW and other State and Federal regulatory agencies during the permitting process.

The land uses surrounding the City's outfall and the RMZ consist of a pre-cast concrete product manufacturing facility and undeveloped riparian forest. Other than the concrete facility, the local properties are large, undeveloped parcels that are either privately owned or owned by the City of Lebanon. The properties are not open to the public, so public access to the outfall area is very limited. There are no public beaches or swimming areas nearby. The closest public boat ramps are approximately two-thirds of a mile upstream and more than ten miles downstream. There are no nearby drinking water intakes and there are no other NPDES discharges to the river in the area.

WWTP DISCHARGE CHARACTERISTICS

The following sections present characteristics of the WWTP effluent discharge that affect the diffuser design development and mixing zone analysis.

WWTP Effluent Flows

Flow projections were prepared for the City's 2004 Wastewater Treatment Plant Facilities Plan. The year 2024 flow rates were projected based on a community growth rate of 1.71 percent. The build-out flow rates are predicted based on the existing Urban Growth Boundary and a projected build-out population of 33,500. The projected flow rates are summarized in Table 4.

Table 4. Projected Effluent Flow Rates

Parameter	Year 2024, mgd	Build-Out, mgd
Average Dry Weather Flow (ADWF)	3	5
Average Wet Weather Flow (AWWF)	8	14
Maximum Month Dry Weather Flow (MMDWF)	7	12
Maximum Month Wet Weather Flow (MMWWF)	12	21
Peak Day Flow (PDF)	20	26
Peak Wet Weather Flow (PWWF)	26	36

Until the second phase of improvements is constructed, the outfall diffuser will operate year round. As noted earlier, the relevant design conditions for a direct river outfall that operates year round are critical low river flow periods. Since the low river flow periods always occur during the dry weather season, the proper design flow rate for ensuring adequate mixing during critical low river flow conditions is the design Average Dry Weather Flow (ADWF). Therefore, the initial design flow rate for the mixing zone modeling of the outfall diffuser is the existing WWTP's design ADWF of 3 mgd which corresponds to the Year 2024 ADWF projection. To avoid the need for modifications to the diffuser in the foreseeable future, the design should also be capable of maximizing mixing under the Build-Out ADWF of 5 mgd. In addition, the year 2024 MMDWF of 7 mgd and Build-Out MMDWF of 12 mgd were modeled to determine the available dilution at the ZID boundary (10 feet downstream).

The DEQ recommended that the modeling analysis also include off-design scenarios. Since the AWWF is so close to the MMDWF, it was decided to model the year 2024 MMWWF of 12 mgd, and the Build-Out MMWWF of 21 mgd to demonstrate that good mixing occurs during winter conditions.

The peak flow capacity for outfall systems is typically selected to allow for build-out conditions so that re-working or expansion of the outfall system does not need to be a routine component of future plant expansions. As a result, the recommended design hydraulic capacity of the new outfall system is the Build-Out Peak Wet Weather Flow (PWWF) of 36 mgd.

WWTP Effluent Temperature

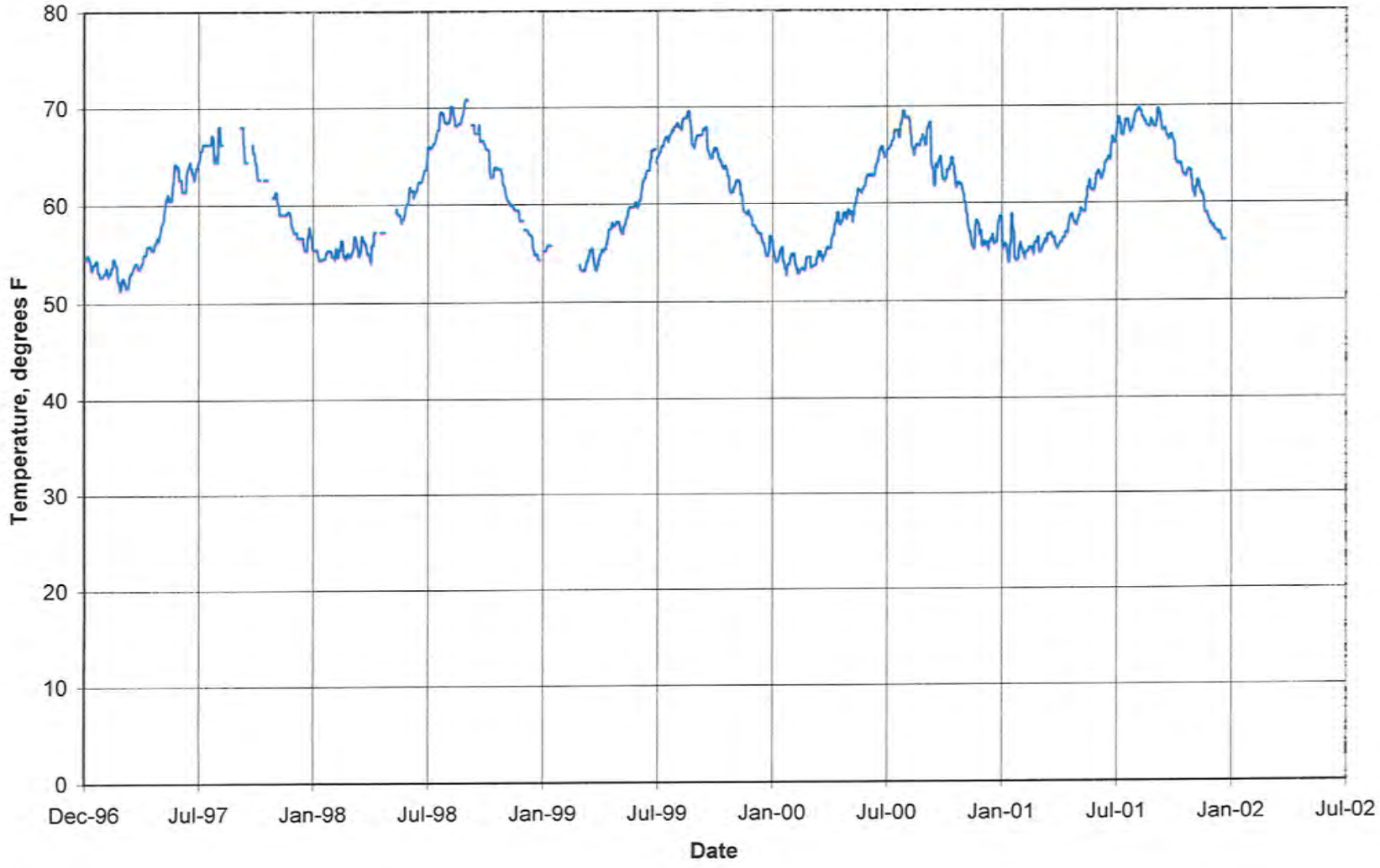
The City regularly monitors the temperature of the WWTP effluent. The effluent temperature data that was presented in their 2002 Temperature Management Plan are illustrated in Figure 5. The highest 7-day average of the daily effluent temperature was 70.8 degrees Fahrenheit (21.5 degrees Celsius). Since the high effluent temperature period is likely to coincide with the critical low river flow period, this effluent temperature was used for the mixing zone modeling. Identification of the relative difference between the river and effluent temperature is important for determining the effects of buoyancy in the mixing modeling.

RECOMMENDED DIFFUSER CONFIGURATION

A multi-port diffuser design will maximize mixing between the effluent and river water within the available mixing zone. The recommended type of diffuser system consists of a relatively large diameter outfall pipeline header fitted with multiple smaller diameter riser pipes and discharge ports equipped with duckbill type check valves. Oriented perpendicular to the river flow, the outfall pipeline header would be buried beneath the riverbed with ports protruding above the river bottom.

To minimize the necessary diffuser length as well as reduce the potential for catching debris or affecting recreational navigation in the river, the diffuser should be located within the deepest portion of the river channel. Fortunately, the deeper portion of the river channel is located near the west bank of the river, which is the easiest location to access from the Lebanon WWTP. The ports would be oriented at various angles to the river flow direction, resulting in a 'fanned'

Figure 5. 7-Day Moving Average Effluent Temperature



arrangement. The 'fanned' arrangement allows a relatively compact diffuser length to provide the required plume width at the RMZ boundary. This type of multi-port diffuser provided the basis for the mixing zone modeling, which considered alternative diffuser lengths, port spacing, and port sizing.

To increase the hydraulic capacity of the outfall system during peak flows, the recommended outfall system also includes a connection to the existing outfall system. This feature consists of a valved connection to the new outfall transmission pipeline which can divert a portion of the peak flows into the existing outfall pipeline. This connection is not anticipated to be necessary in the near term future, since the new diffuser is adequately sized for build-out conditions, but it does provide redundancy for the system. The existing outfall discharges at the shoreline of the South Santiam River.

MIXING ZONE MODELING

The EPA sponsored computer program CORMIX (Cornell Mixing Zone Expert System) was used to model mixing between the effluent discharged from a multi-port diffuser and the river. The program predicts the behavior of an aqueous pollutant plume based on key physical characteristics of the discharge flow and the receiving water body. The model predicts the geometry of a pollutant plume and dilution characteristics at discrete distances downstream from the discharge point. Knowledge of these dilution characteristics for the discharge plume is critical for identifying the dilution achieved within the mixing zone and evaluating compliance with water quality standards.

The CORMIX program consists of three basic formats for modeling typical outfall systems.

- CORMIX 1 – for submerged single port discharges.
- CORMIX 2 – for submerged multi-port diffusers.
- CORMIX 3 – for buoyant surface discharges.

For the purposes of this modeling work, CORMIX 2 was used to analyze the plume geometry and dilution at the ZID and CMZ for the diffuser.

The dimensional inputs for the CORMIX model were selected to best reflect the characteristics of the South Santiam River as surveyed in the vicinity of the City's outfall. Although the overall river channel width during the dry weather period is over 130 feet, the diffuser will be located within the deeper water on the west side of the channel. As a result, the modeling focused on a 60-foot wide section of the channel extending out from the west river bank. This portion of the river is 0.5 feet deeper than the overall channel average, requiring adjustment of the average river depths reported in Table 2 accordingly. The average river velocities within the modeled portion of the river are anticipated to be similar to the overall average for the river. Table 3 summarizes the average river width, depths, and ambient water velocities used in the modeling.

Table 3. Modeled Channel Width, Depth, and Velocity by Flow Condition

Flow Condition	Modeled Width, feet	Modeled Depth, feet	Modeled Velocity, feet/second
1Q10	60	3.5	1.15
7Q10	60	3.5	1.15
30Q5	60	3.75	1.25
Harmonic Mean	60	5.6	1.70
Maximum Month Wet Weather	60	8.5	3.9

Following are the other key parameters that were used as modeling inputs for the river:

- Channel roughness: Manning's "n" coefficient of 0.035
- Ambient water temperature: 19 degrees C
- Effluent water temperature: 21.5 degrees C

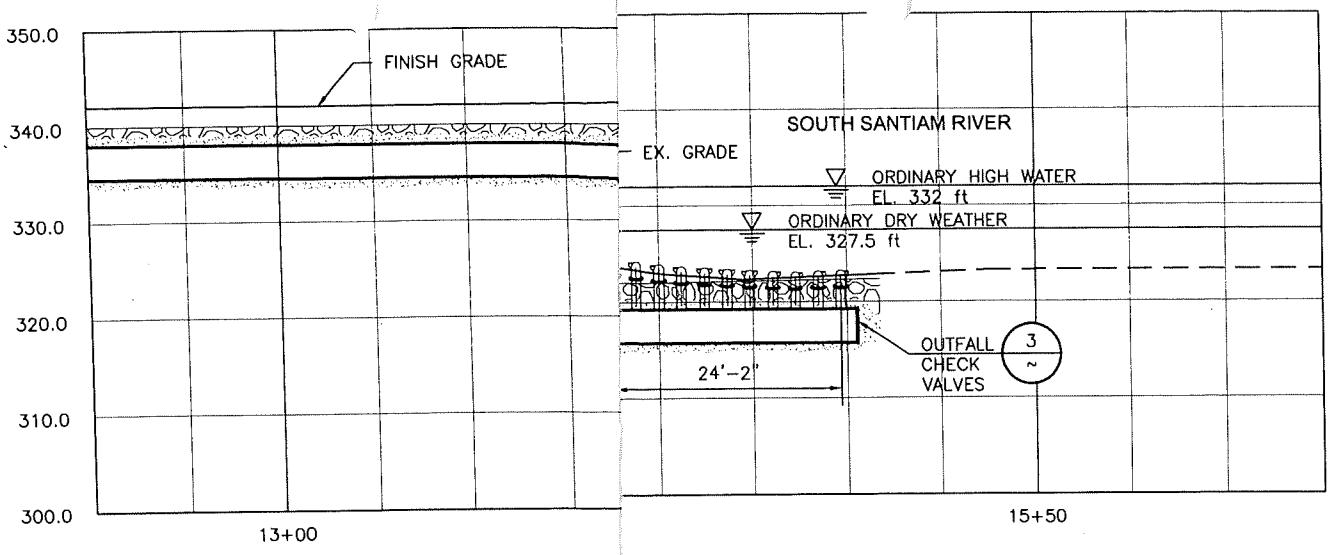
A variety of port configurations were modeled to evaluate the plume's ability to achieve complete mixing with 25-percent of the river flow within the RMZ during critical low river flow conditions. The best combination of dilution results for the various flow conditions were provided by an eleven-port diffuser with an overall length of approximately 24-feet.

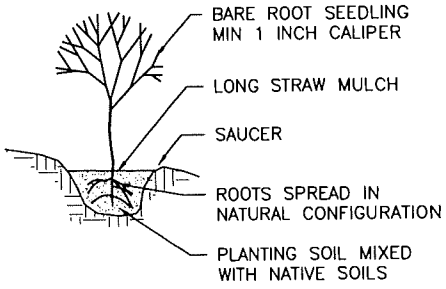
Figure 6 illustrates the recommended diffuser configuration. In section, the ports would be oriented to discharge the effluent at a slight angle (10-degrees) upward from horizontal, thus preventing plume attachment to the river bottom. In plan, the ports would be fanned at angles ranging from 10-degrees to parallel with the ambient river flow.

The CORMIX modeling predictions indicated that the discharge plumes from this type of diffuser are hydrodynamically unstable under low river flow conditions, meaning that the discharge strength is strong in relation to the layer depth and the discharge buoyancy. As the plumes move downstream, they rapidly rise while being deflected by the current. As the submerged plumes approach the water surface, mixing is strong. The plumes rise vertically and spread laterally while being advected by the ambient current. The plumes interact to provide a uniform average dilution.

The recommended riser pipe/diffuser port design consists of eleven 12-inch diameter discharge ports equipped with check valves to optimize mixing for the Year 2024 design ADWF rate of 3.0 mgd. This diffuser configuration results in a discharge plume geometry that maximizes mixing under the critical low river flow conditions.

As discussed earlier, the mixing objective is to provide complete mixing of the effluent with 25-percent of the river volume by the time the plume reaches the CMZ boundary 100 feet downstream of the diffuser. For each of the design cases, the individual plumes have fully





BARE ROOT PLANTING (NATIVE ASH)

DETAIL 1

NOT TO SCALE

CAVATION AND INSTALLATION SHALL BE ACCOMPLISHED WITH SHORE UNTED EQUIPMENT.

-RIVER WORK AREA SHALL BE ISOLATED WITH SILT FENCE TO MINIMIZE RBITY.

. DISTURBED AREAS OF THE RIVER BANK BETWEEN NORMAL WATER LEVEL 7 FEET) AND TOP OF BANK (APPROXIMATE GROUND SURFACE 342 FEET) ALL RECEIVE SPECIFIED GEOTEXTILE MATTING AND HYDROSEEDING. TURBED AREAS BEYOND THE TOP OF THE RIVER BANK SHALL RECEIVE LCH AND JUTE MATTING PRIOR TO HYDROSEEDING. SEEDING AND EPARATION SHALL BE PERFORMED WITH NATIVE GRASSES IN ACCORDANCE H CURRENT OREGON DEPARTMENT OF TRANSPORTATION STANDARDS.

ER BOTTOM ELEVATION TO BE CONFIRMED BY CONTRACTOR PRIOR TO MPLETION OF DIFFUSER SHOP DRAWINGS.

ERS TO BE ORIENTED VERTICALLY. RISER HEIGHT SHALL BE CONFIRMED LLOWING CONTRACTORS DETAILED RIVER BOTTOM SURVEY.

ECK VALVES SHALL BE 12" TIDEFLEX VALVES, MODEL 35D WITH INTEGRAL BBER ELBOW AS MANUFACTURED BY RED VALVE OR EQUAL. CONTRACTOR ALL PROVIDE CHECK VALVE SUBMITTAL AND PRODUCT DATA. PROVIDE AINLESS STEEL HARDWARE AND BACKING FLANGES.

TFALL PIPELINE SHALL BE NEGATIVELY BUOYANT BY ATTACHING BALLAST IGHT DESIGNED IN ACCORDANCE WITH SPECIFICATION SECTION 02628.

OVIDE OVERSIZED FLANGES AS NECESSARY FOR BOLT INSTALLATION TO TCH TIDEFLEX VALVE

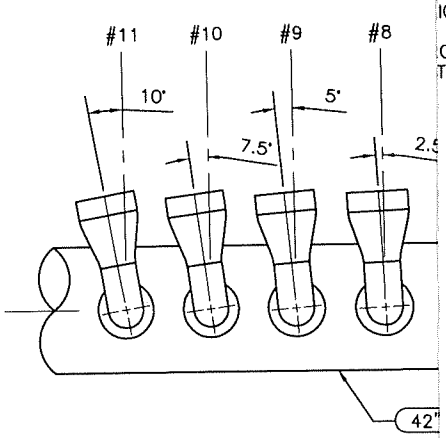


Figure 6

LEBANON WWTP OUTFALL
SECTIONS AND DETAILS

WEST YOST ASSOCIATES
Consulting Engineers

\\Eus-fs01\eugene\Clients\516 Lebanon\01-06-06 Walden Design\CAD\Figures\Mixing Zone Analysis\FIG 6.dwg 12-17-07 09:30:47 AM vfleming

DRAFT

interacted at the RMZ boundary. The the overall plume width is approximately 30-feet at 450 cfs (1Q10/7Q10), thus providing full mixing with 25-percent of the river flow within the RMZ.

The diffuser was also modeled at river flows of 540 cfs (30Q5), 1,380 cfs (harmonic mean), 6,000 cfs (maximum month wet weather flow) and effluent flow rates of 3 mgd (Design Year 2024 ADWF), 5 mgd (Build-Out ADWF), 7 mgd (Design Year 2024 MMDWF) 12 mgd (Design Year 2024 MMWWF), and 21 mgd (Build-Out MMWWF). In addition, Attachment 1 includes three-dimensional plume graphics from the CORMIX model under each of the modeled river flow conditions.

CORMIX DILUTION ESTIMATES

The results of the CORMIX modeling are summarized in Table 4. The dilution results are reported for the ZID boundary (10 feet downstream) and the CMZ boundary (100 feet downstream). The dilution values reported for the ZID boundary are for the hydrodynamic centerline of the discharge plume. Attachment 2 includes the CORMIX prediction reports for each modeled scenario.

Table 4. Dilution Results for the New Outfall Diffuser

Scenario	Plume Centerline Dilution at ZID	Bulk Average Plume Dilution at CMZ
River Flow = 450 cfs (1Q10/7Q10) Plant Flow = 3 mgd (Design/Year 2024 ADWF)	22.7	26.6
River Flow = 450 cfs (1Q10/7Q10) Plant Flow = 5 mgd (Build-Out ADWF)	14.6	17.0
River Flow = 540 cfs (30Q5) Plant Flow = 3 mgd (Design/Year 2024 ADWF)	26.0	30.6
River Flow = 540 cfs (30Q5) Plant Flow = 5 mgd (Build-Out ADWF)	16.5	19.3
River Flow = 1,380 cfs (Harmonic Mean) Plant Flow = 3 mgd (Design/Year 2024 ADWF)	51.2	62.1
River Flow = 1,380 cfs (Harmonic Mean) Plant Flow = 5 mgd (Build-Out ADWF)	31.4	37.9
River Flow = 1,380 cfs (Harmonic Mean) Plant Flow = 7 mgd (Design/Year 2024 MMDWF)	23.2	27.9
River Flow = 1,380 cfs (Harmonic Mean) Plant Flow = 12 mgd (Design/Year 2024 MMWWF)	14.6	17.3
River Flow = 6,000 cfs (MMWWF) Plant Flow = 12 mgd (Design/Year 2024 MMWWF)	44.3	55.6
River Flow = 6,000 cfs (MMWWF) Plant Flow = 21 mgd (Build-Out MMWWF)	26.2	32.7

HYDRAULIC PROFILE ANALYSIS

Analysis of the hydraulic profile for the outfall system is important for determining the capacity of the system during 100-year flood conditions. The available head for operating the outfall diffuser under gravity flow conditions is determined by the water surface elevation of the river and maximum allowable water surface elevation at the chlorine contact basin. The 100-year flood elevation is 339.6 feet and the weir elevation at the chlorine contact structure is 350.12 feet, indicating an available head of approximately 10.5 feet. With a discharge port size of 12-inches, the new outfall system will have a hydraulic capacity of 36 mgd*.

The existing outfall system, which has an available capacity of 19 mgd, will remain available for service during extreme wet weather conditions. The overflow to the existing outfall system will be available to activate during flood conditions or during maintenance activities on the new outfall. The combined capacities of the new and existing outfalls will be at least 55 mgd. This ample capacity also provides the City with the flexibility to develop discharge improvements on the east side of the river, which would involve additional head loss due to the greater distance from the WWTP.

(*In the event that an infiltration system is ultimately constructed on the east side of the river, flow splitting facilities would need to be integrated into the outfall system which would increase head losses and reduce the identified hydraulic capacity of the new outfall system. However, the overall outfall system capacity, including the existing outfall pipeline, will remain more than adequate for accommodating the Build-Out PWWF of 36 mgd even with the addition of an infiltration system.)

RECOMMENDED DIFFUSER DESIGN

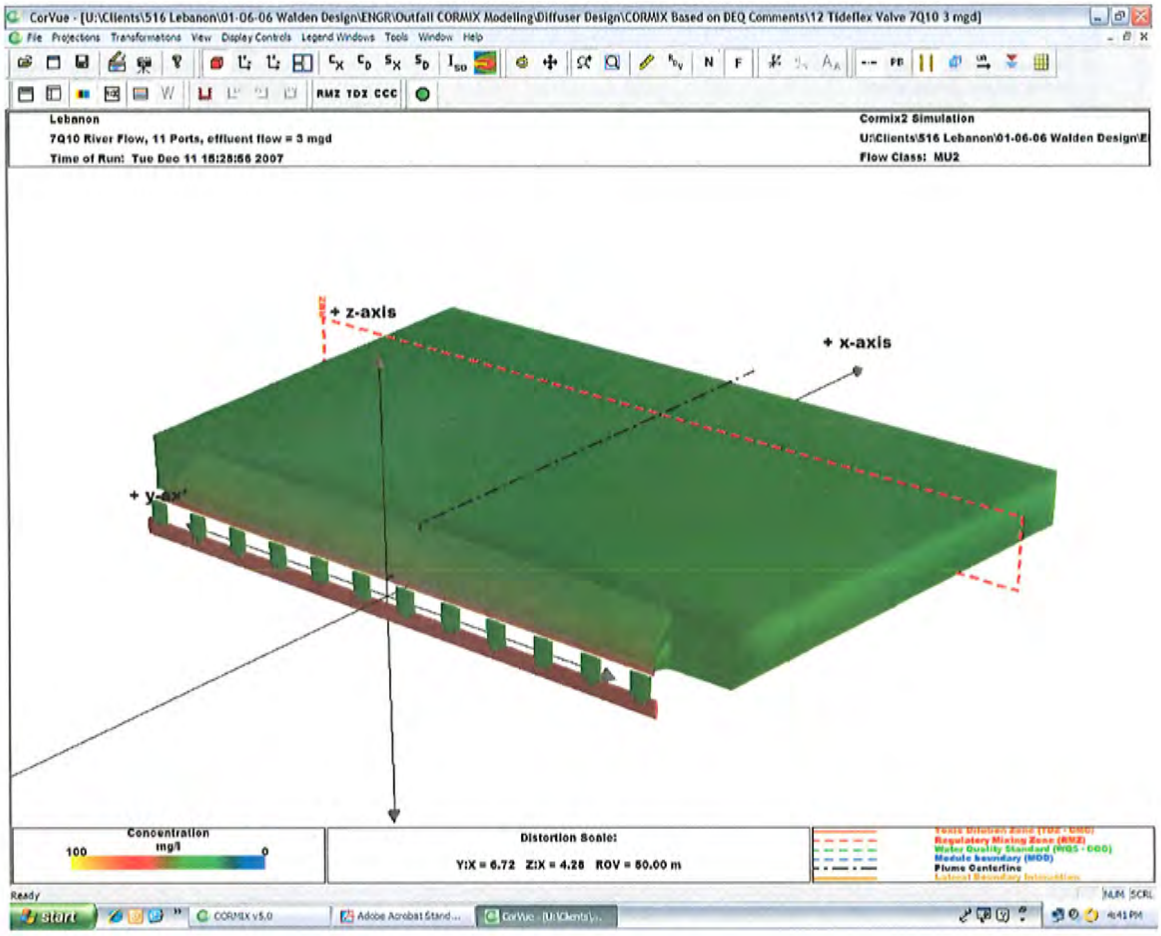
Based on the mixing zone modeling and hydraulic profile analysis results, we recommend an eleven-port outfall diffuser for maximizing mixing within the RMZ. The recommended design data for the diffuser are summarized in Table 5.

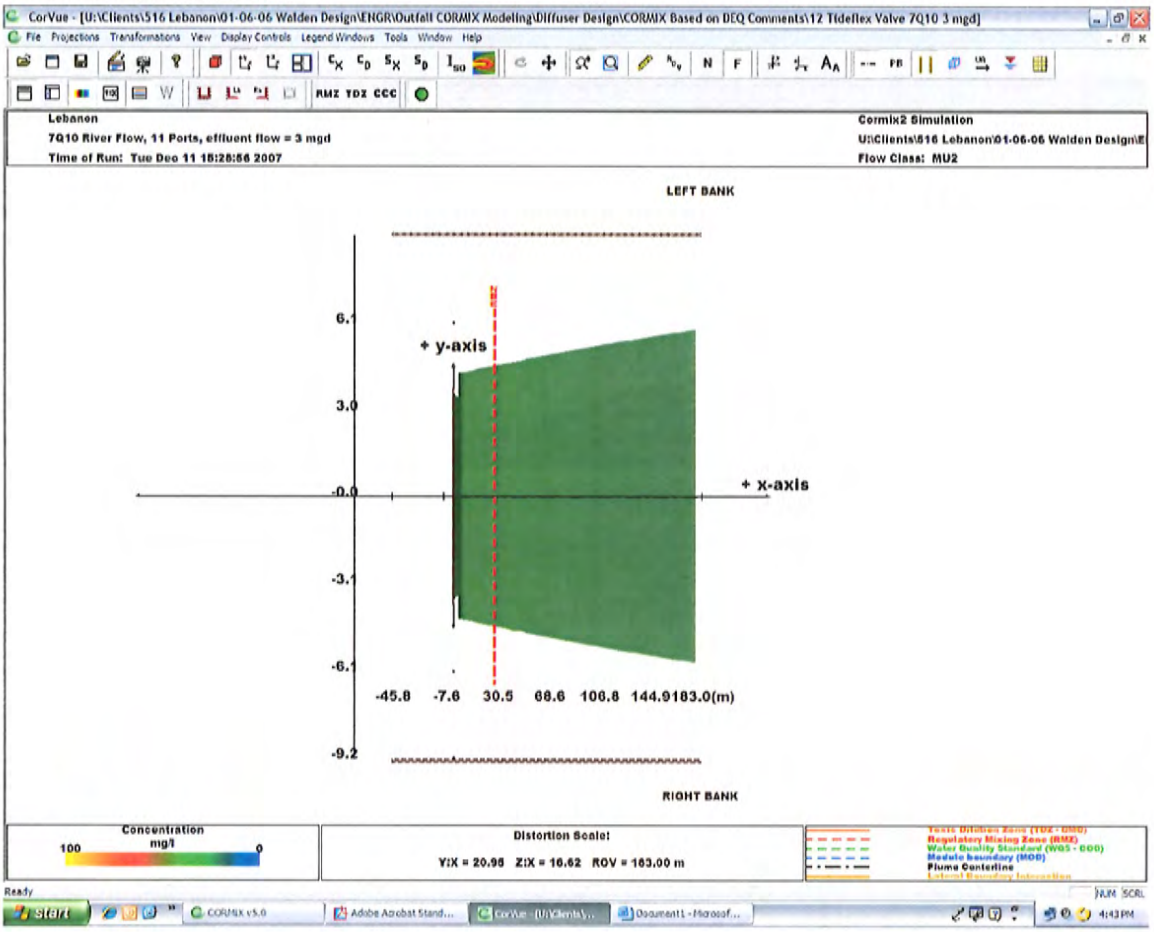
Table 5. Design Data for the Multi-Port Outfall Diffuser

	Initial Diffuser
Outfall pipe diameter, inches	42
Diffuser length, feet	24
Riser pipe diameter, inches	12
Number of ports	11
Port spacing, on center, feet-inches	2' 5"
Port orientation	Fanned
Port diameter, inches	12
Check Valve Size, inches	12

ATTACHMENT 1

1Q10/7Q10 River Flow, 3 mgd Effluent Flow

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File Projections Transformations View Display Controls Legend Windows Tools Window Help
RMZ TDZ CCC
Lebanon
7Q10 River Flow, 11 Ports, effluent flow = 3 mgd
Time of Run: Tue Dec 11 15:28:56 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
LEFT BANK
6.1
3.0
-0.0
-3.1
-6.1
-9.2
+ y-axis
+ X-axis
-45.8 -7.6 30.5 68.6 106.8 144.9183.0(m)
RIGHT BANK
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 20.96 Z:X = 16.62 ROV = 183.00 m
Toxic Dilution Zone (TDZ - CMC)
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - CCC)
Mixing boundary (MOB)
Plume Centerline
Lateral Boundary Interaction
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NUM SCRL
Start
CORMIX v5.0
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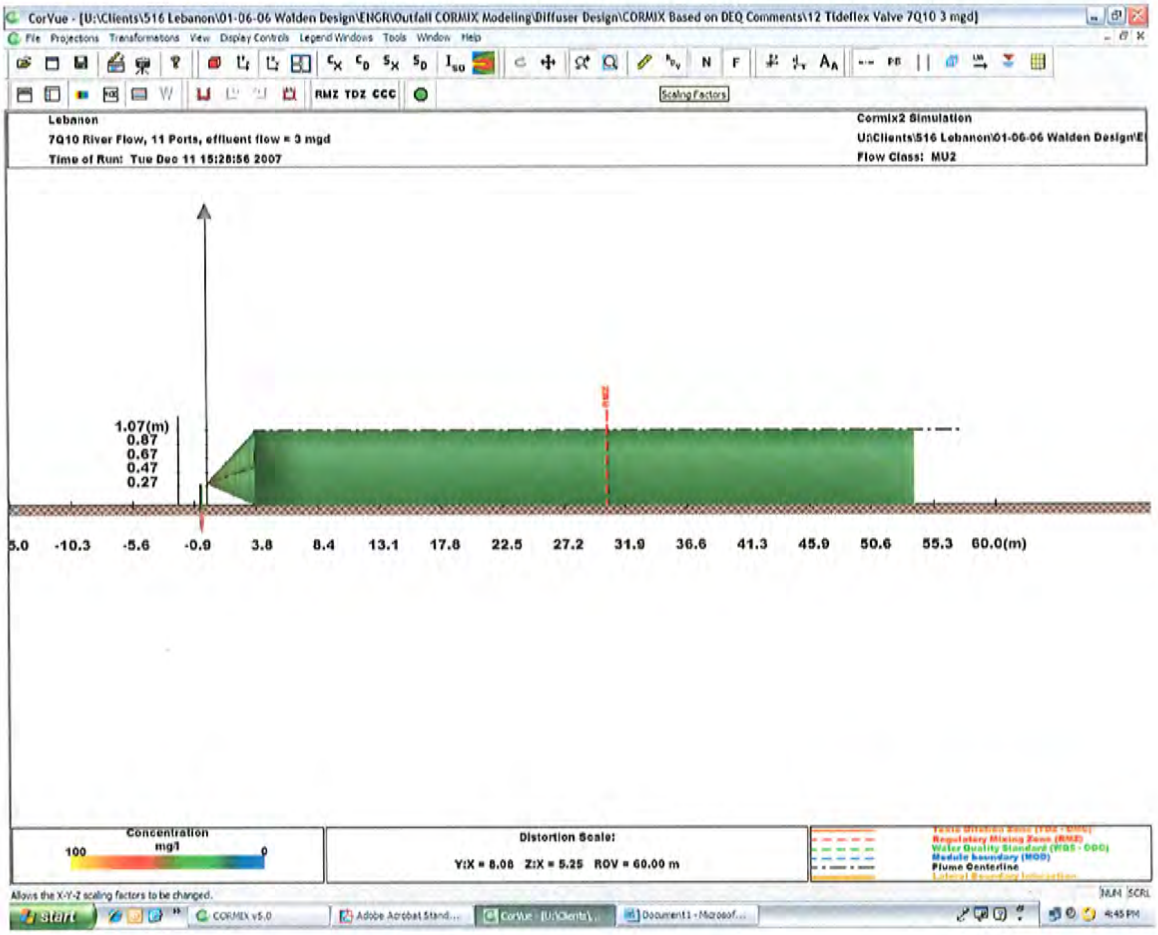
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File Projections Transformations View Display Controls Legend Windows Tools Window Help
RMZ TDZ CCC
Scaling Factors
Lebanon
7Q10 River Flow, 11 Ports, effluent flow = 3 mgd
Time of Run: Tue Dec 11 15:28:56 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
1.07(m)
0.87
0.67
0.47
0.27
5.0 -10.3 -5.6 -0.9 3.8 8.4 13.1 17.8 22.5 27.2 31.9 36.6 41.3 45.9 50.6 55.3 60.0(m)
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 8.08 Z:X = 5.25 ROV = 60.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - COC)
Module boundary (MOD)
Plume Centerline
Allows the X-Y-Z scaling factors to be changed.
NUM SCRL
start
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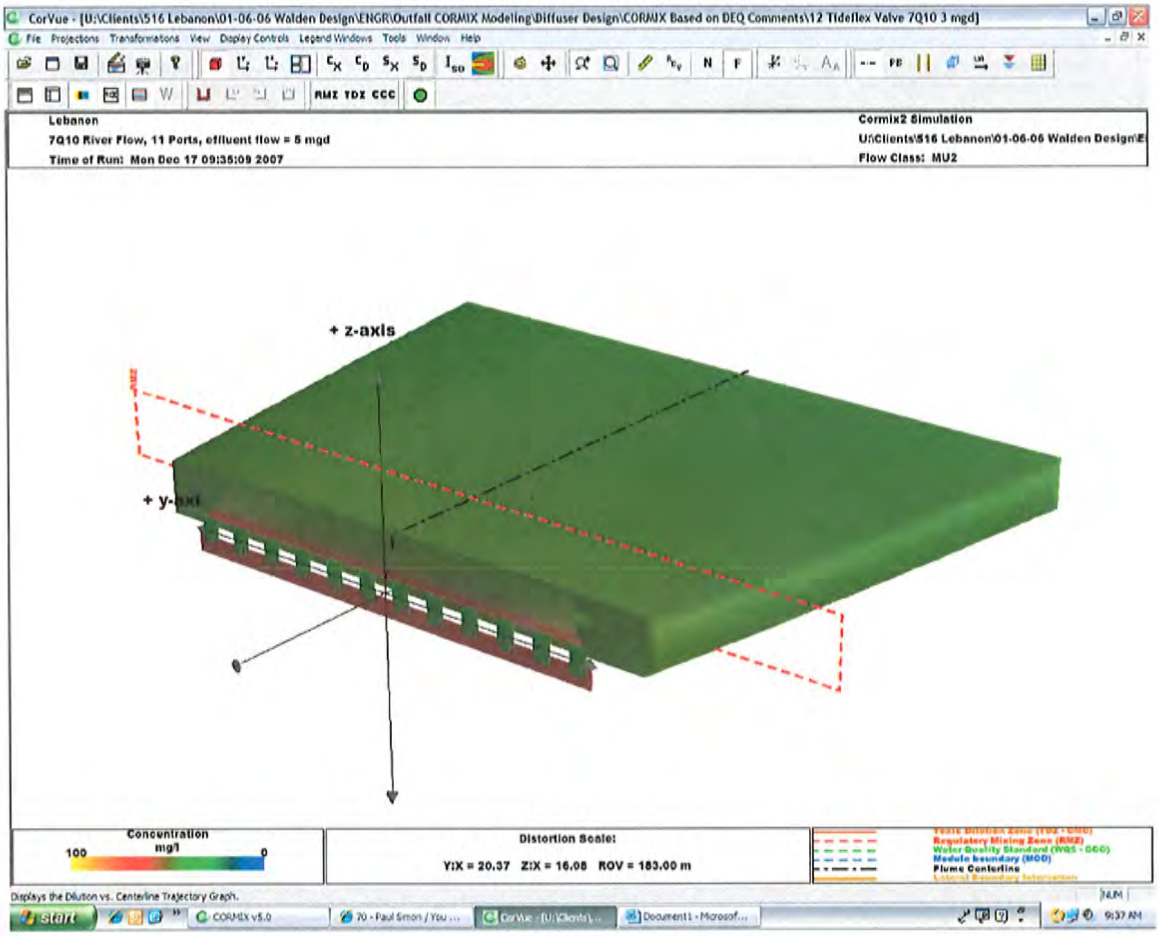
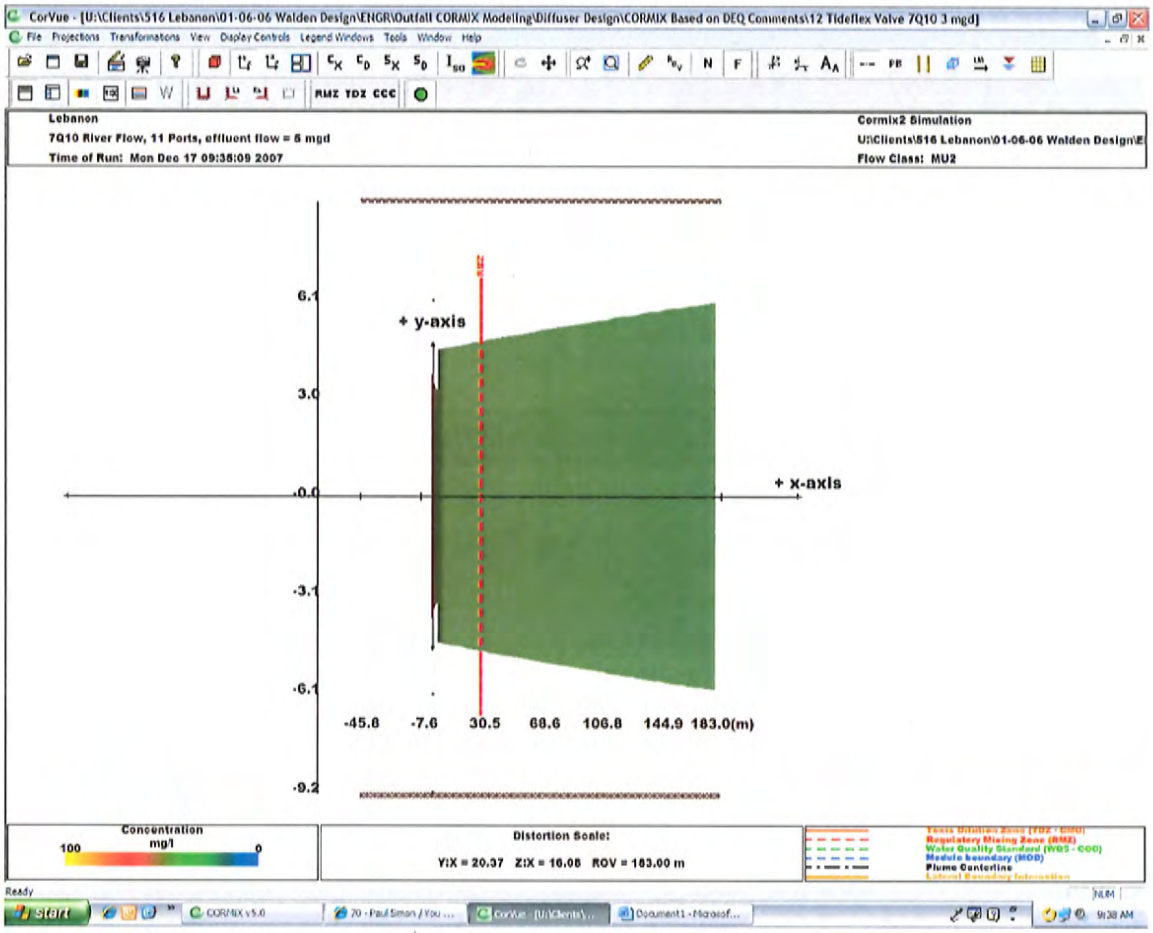
1Q10/7Q10 River Flow, 5 mgd Effluent Flow

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File Projections Transformations View Display Controls Legend Windows Tools Window Help
Lebanon
7Q10 River Flow, 11 Ports, effluent flow = 6 mgd
Time of Run: Mon Dec 17 09:35:09 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
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+ y-axis
+ x-axis
6.1
3.0
0.0
-3.1
-6.1
-9.2
-45.8
-7.6
30.5
68.6
106.8
144.9
183.0(m)
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 20.37 Z:X = 16.08 ROV = 183.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - CCC)
Mobile boundary (MOB)
Plume Centerline
Ready
start
CORMIX v5.0
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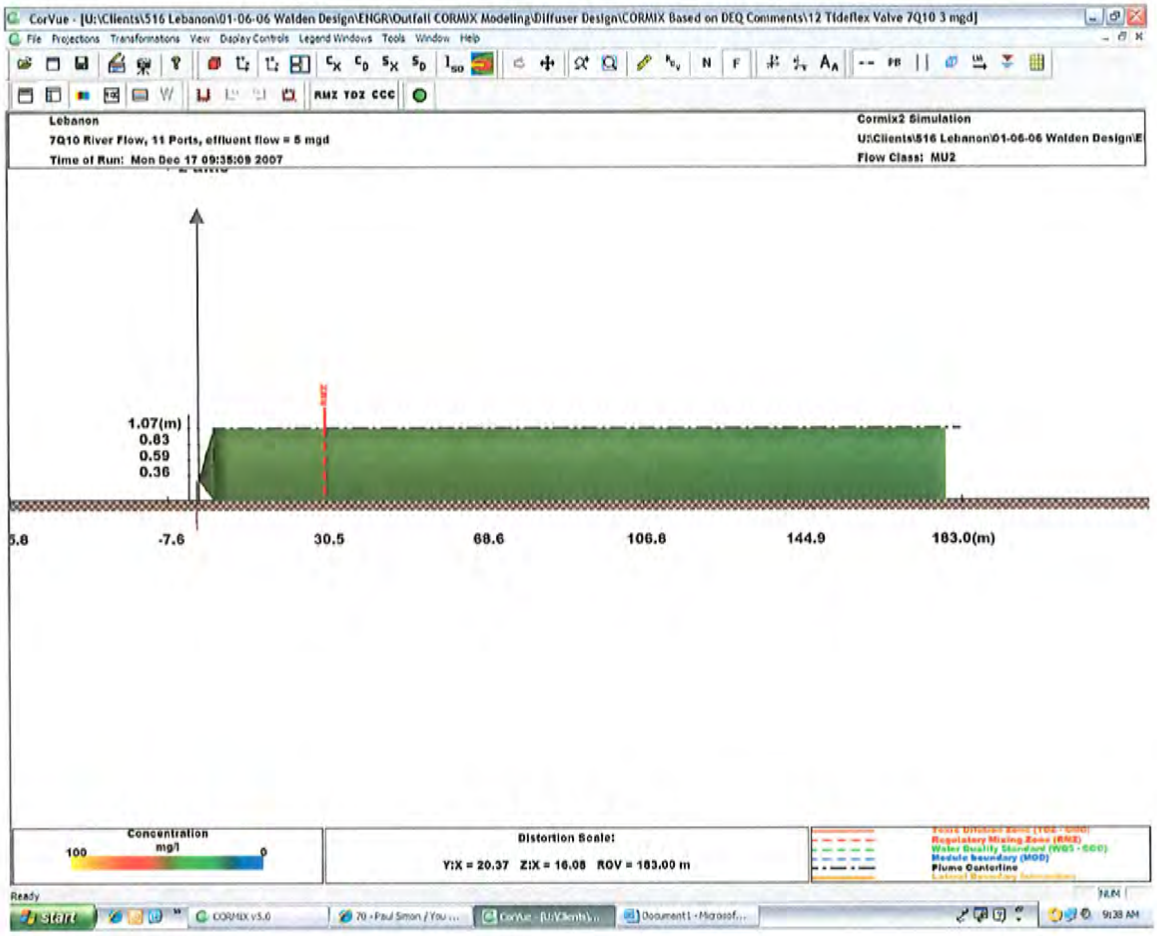
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File Projections Transformations View Display Controls Legend Windows Tools Window Help
RMZ TDZ CCC
Lebanon
7Q10 River Flow, 11 Ports, effluent flow = 5 mgd
Time of Run: Mon Dec 17 09:35:09 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
1.07(m)
0.83
0.59
0.36
5.8
-7.6
30.5
68.6
106.8
144.9
183.0(m)
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 20.37 Z:X = 16.08 ROV = 183.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - CCC)
Mixing boundary (MOD)
Plume Centerline
Ready
NUM
start
CORMIX v5.0
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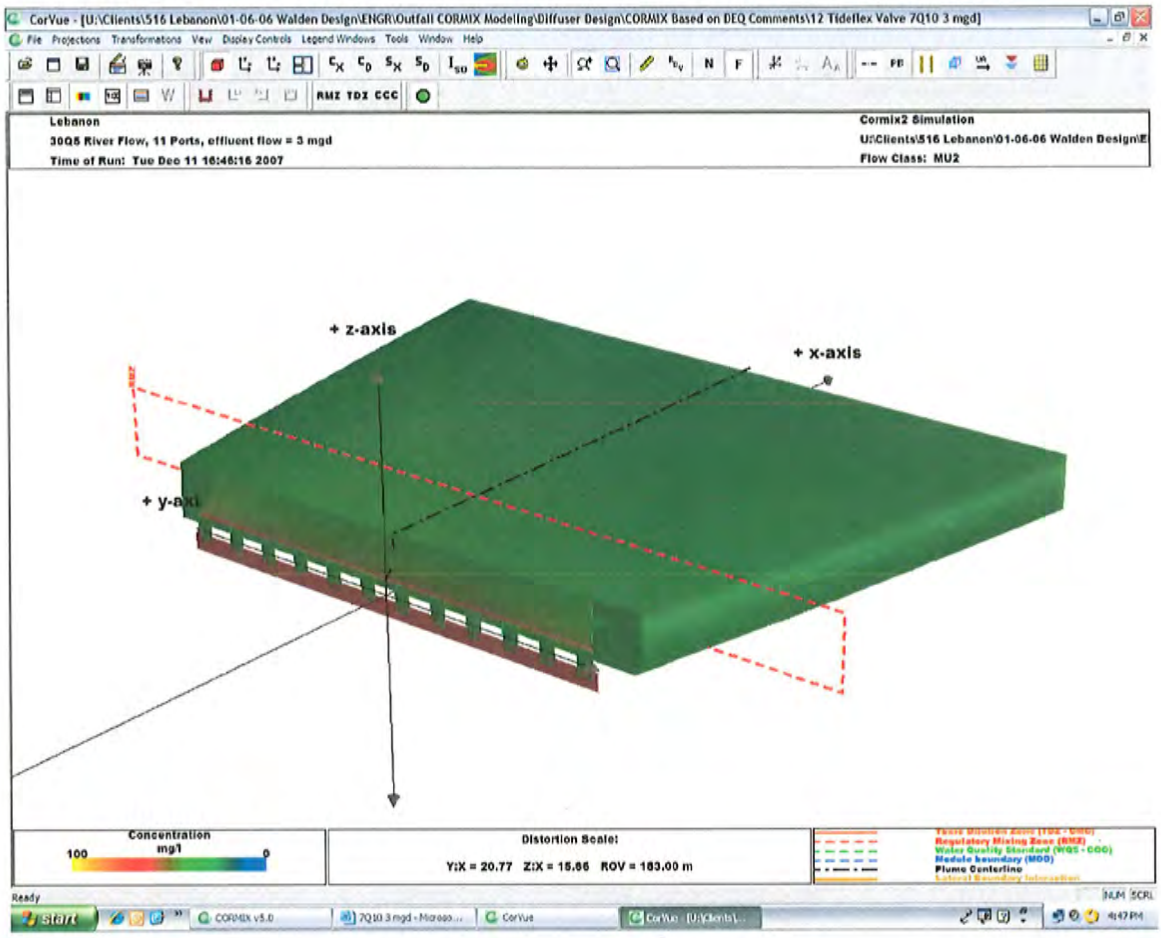
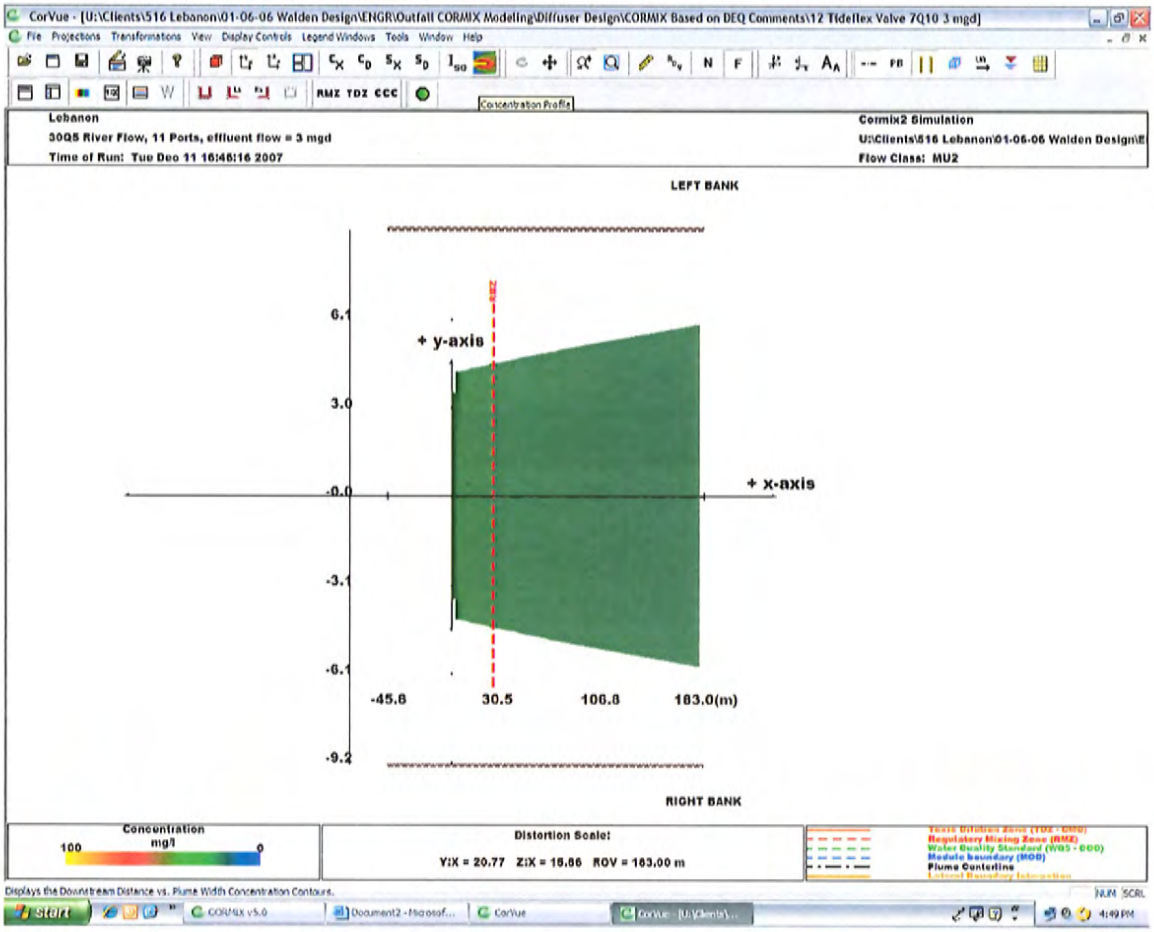
30Q5 River Flow, 3 mgd Effluent Flow

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File Projections Transformations View Display Controls Legend Windows Tools Window Help
RMZ TDZ CCC
Concentration Profile
Lebanon
30Q5 River Flow, 11 Ports, effluent flow = 3 mgd
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Cormix2 Simulation
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LEFT BANK
RMZ
6.1
3.0
-0.0
-3.1
-6.1
-9.2
+ y-axis
+ x-axis
-45.8
30.5
106.8
183.0(m)
RIGHT BANK
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 20.77 Z:X = 18.86 ROV = 183.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - CCC)
Mixing Zone boundary (MOD)
Plume Centerline
Displays the Downstream Distance vs. Plume Width Concentration Contours.
NUM SCRL
start
CORMIX v5.0
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CorVue
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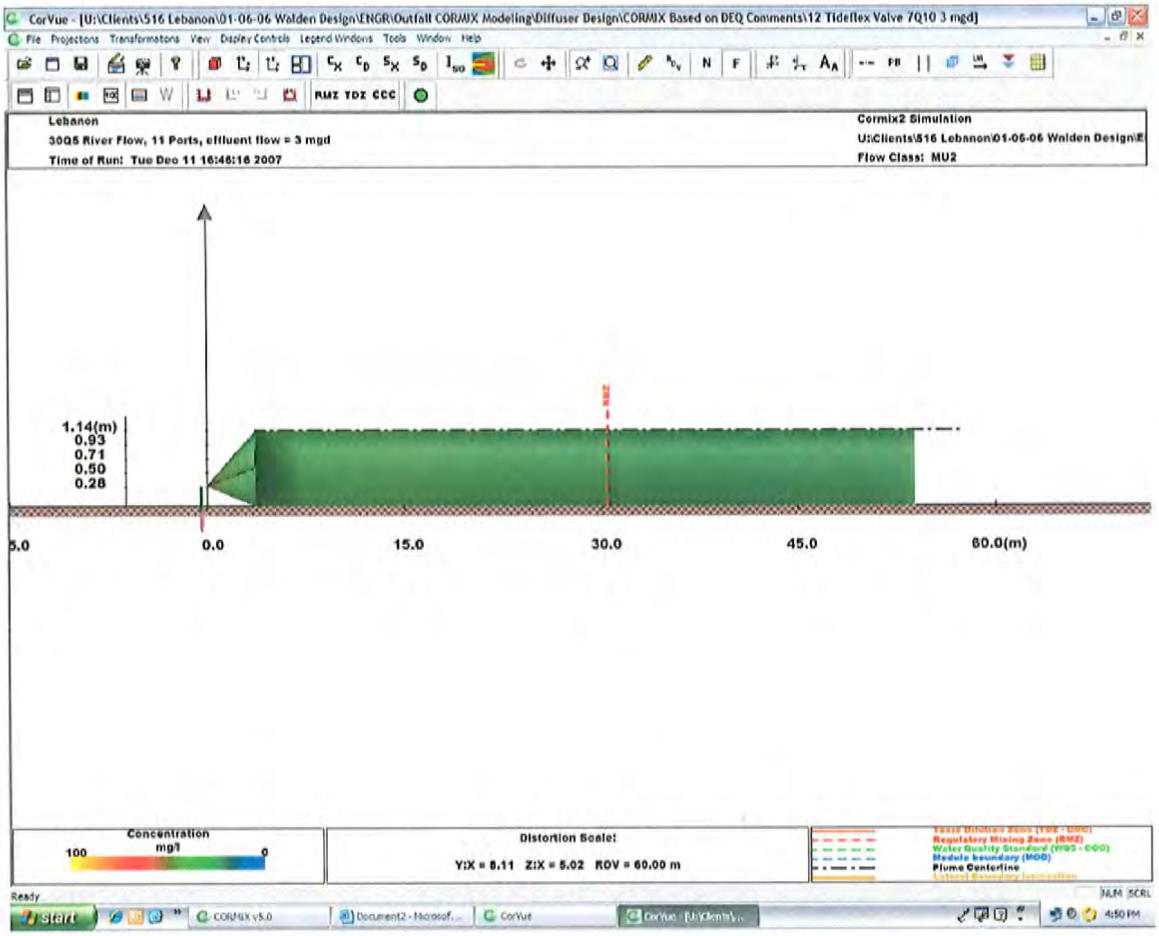
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File Projections Transformations View Display Controls Legend Windows Tools Window Help
RMZ TDZ CCC
Lebanon
30Q5 River Flow, 11 Ports, effluent flow = 3 mgd
Time of Run: Tue Dec 11 16:46:16 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
RMZ
1.14(m)
0.93
0.71
0.50
0.28
5.0
0.0
15.0
30.0
45.0
60.0(m)
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 6.11 Z:X = 5.02 ROV = 60.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - CCC)
Mixing boundary (MOD)
Plume Centerline
Ready
NUM SCRL
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CORMIX v5.0
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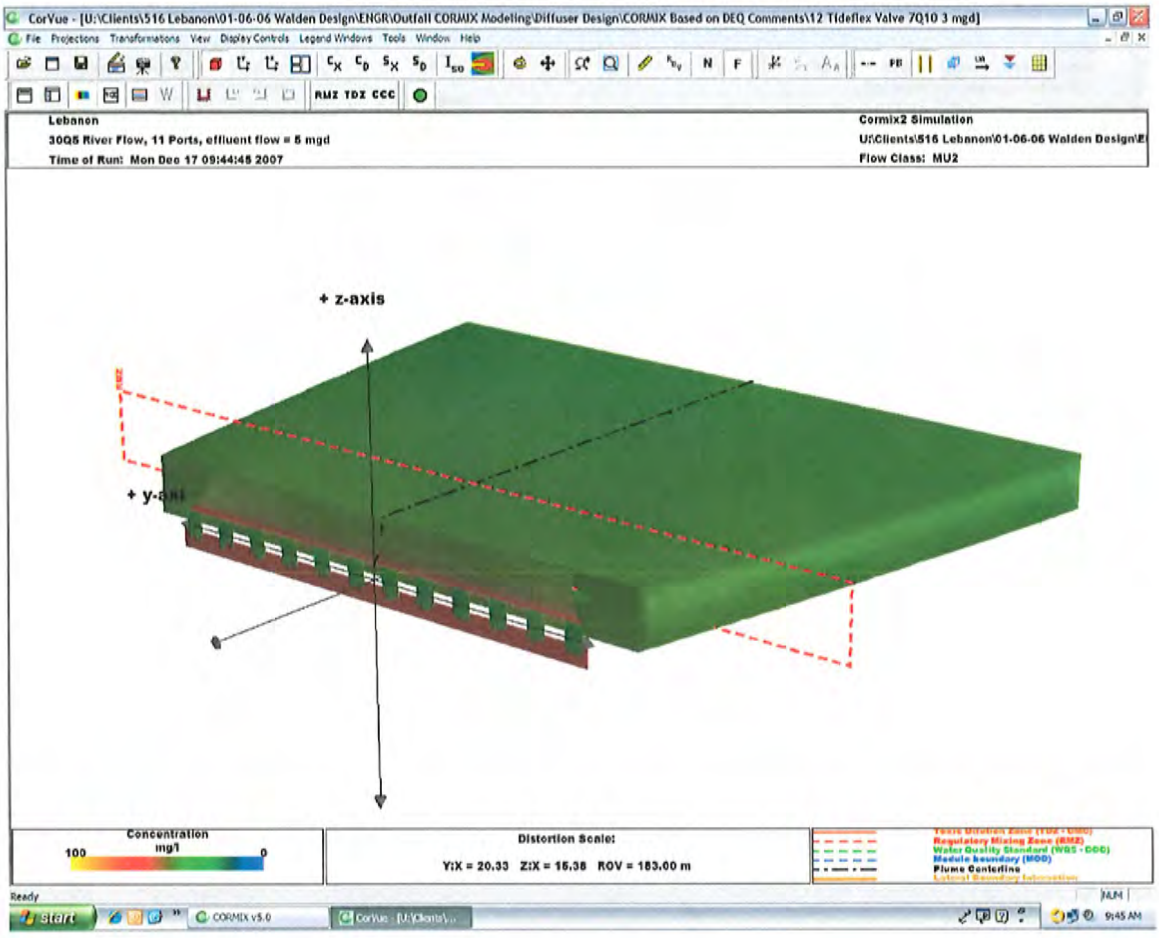
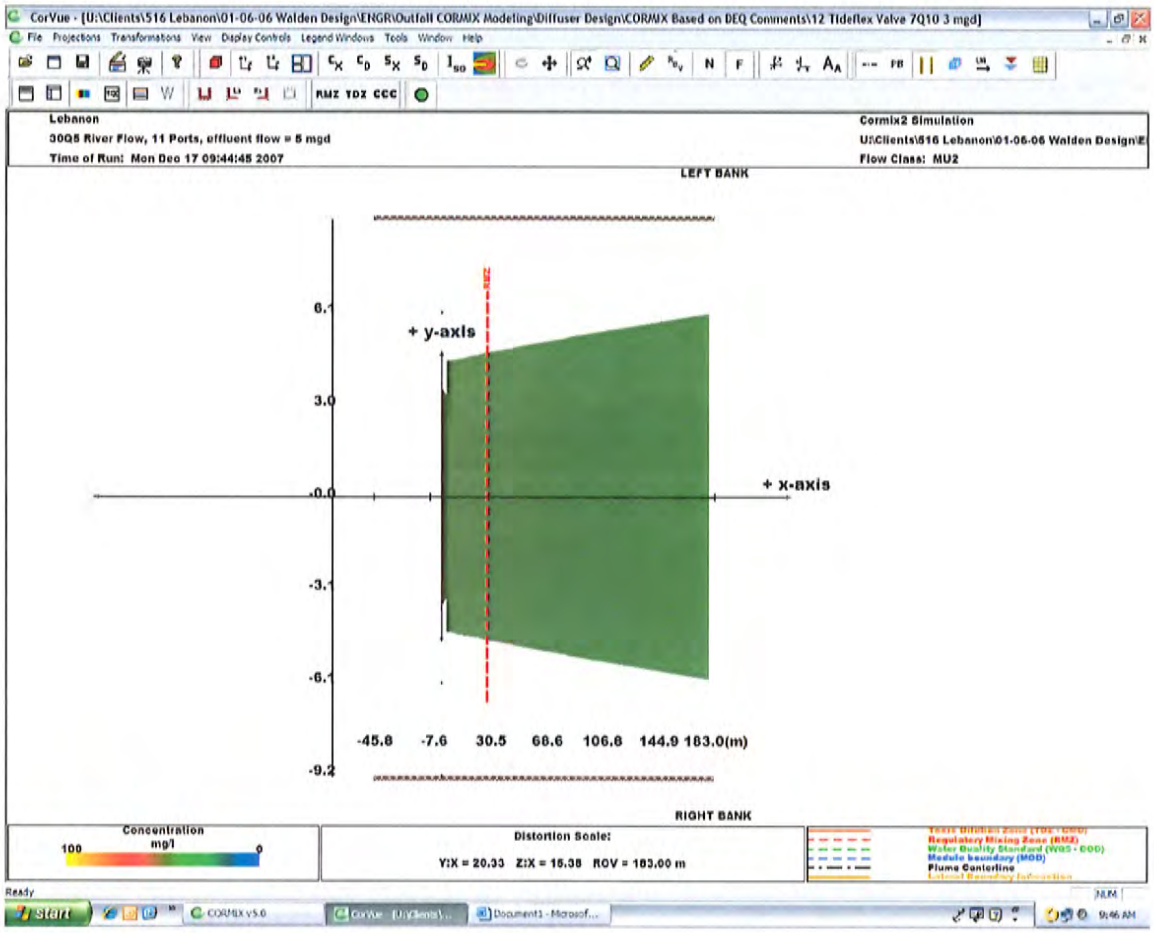
30Q5 River Flow, 5 mgd Effluent Flow

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File Projections Transformations View Display Controls Legend Windows Tools Window Help
Lebanon
30Q5 River Flow, 11 Ports, effluent flow = 5 mgd
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Cormix2 Simulation
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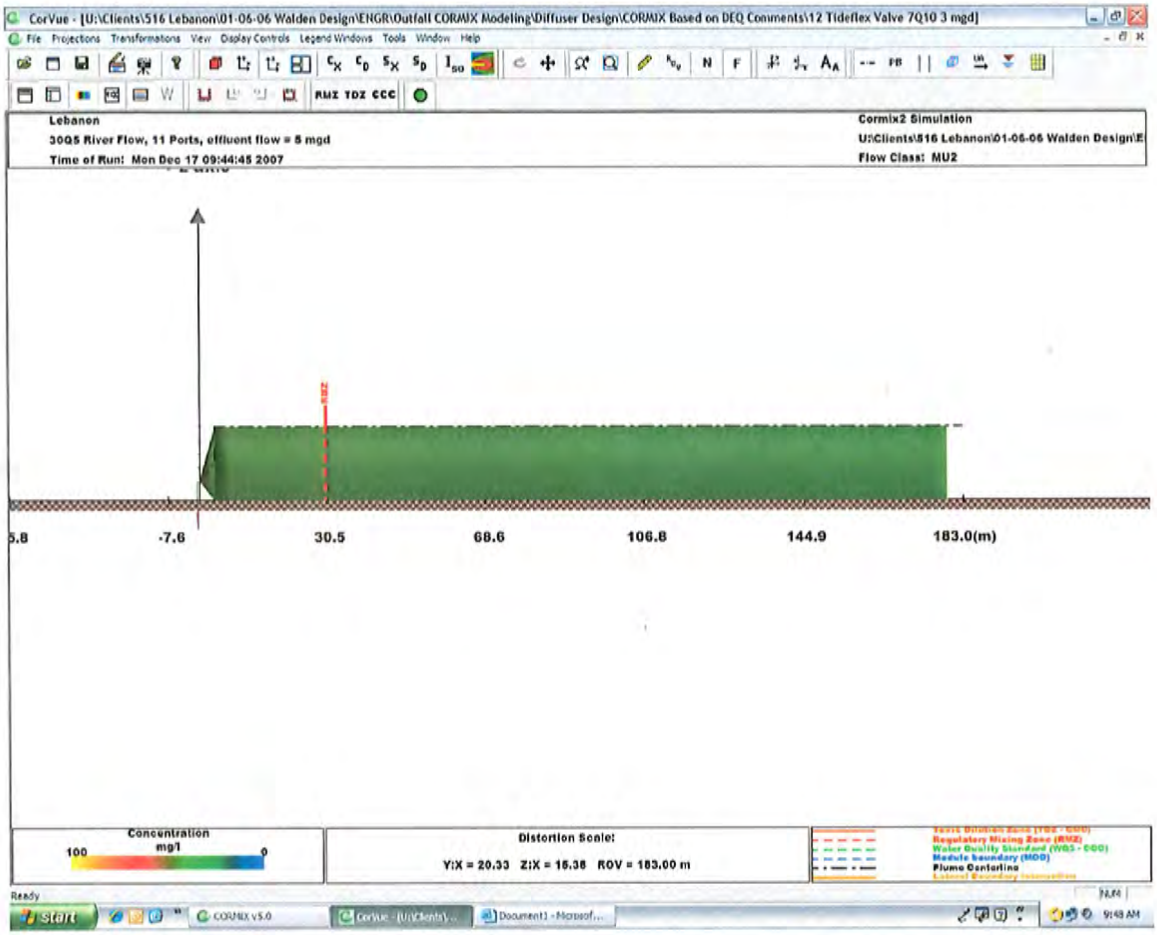
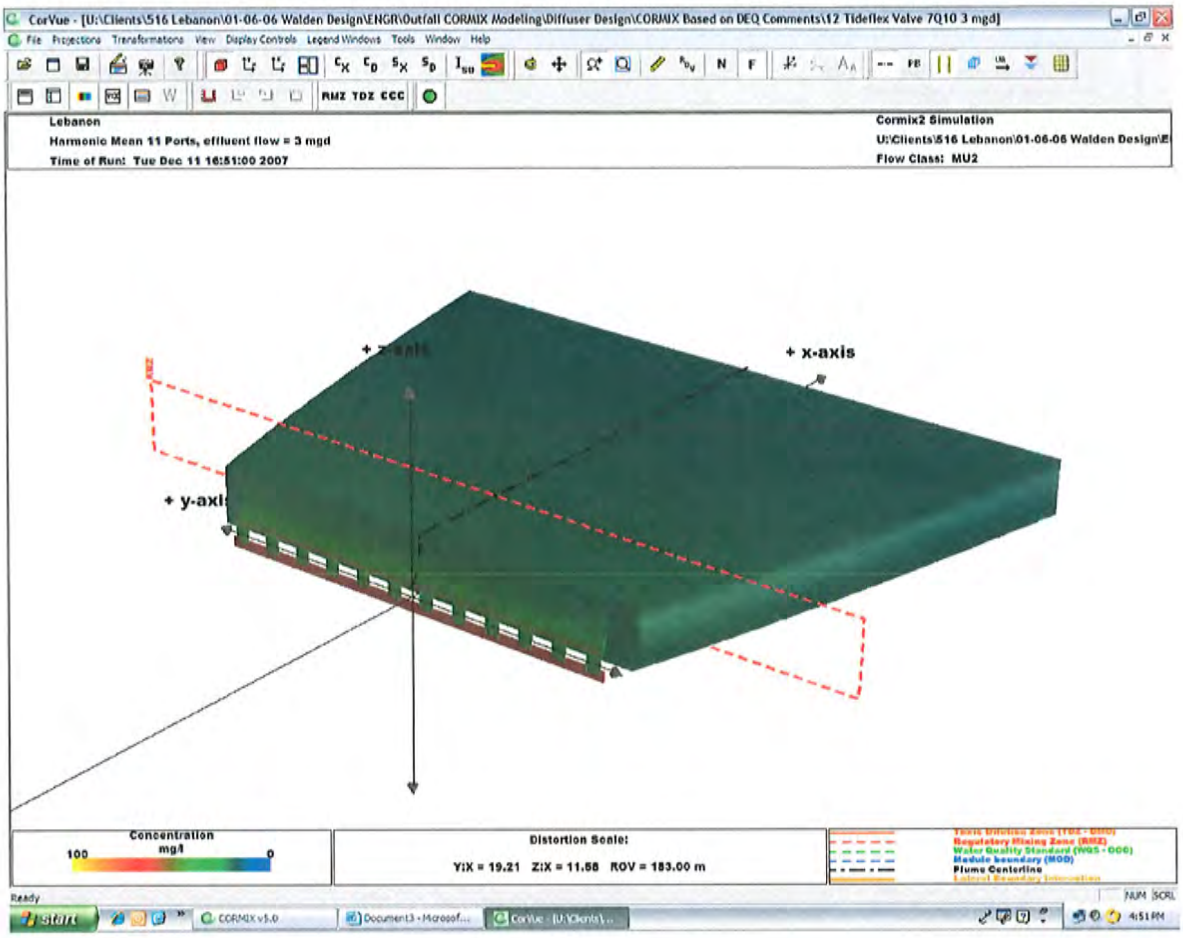
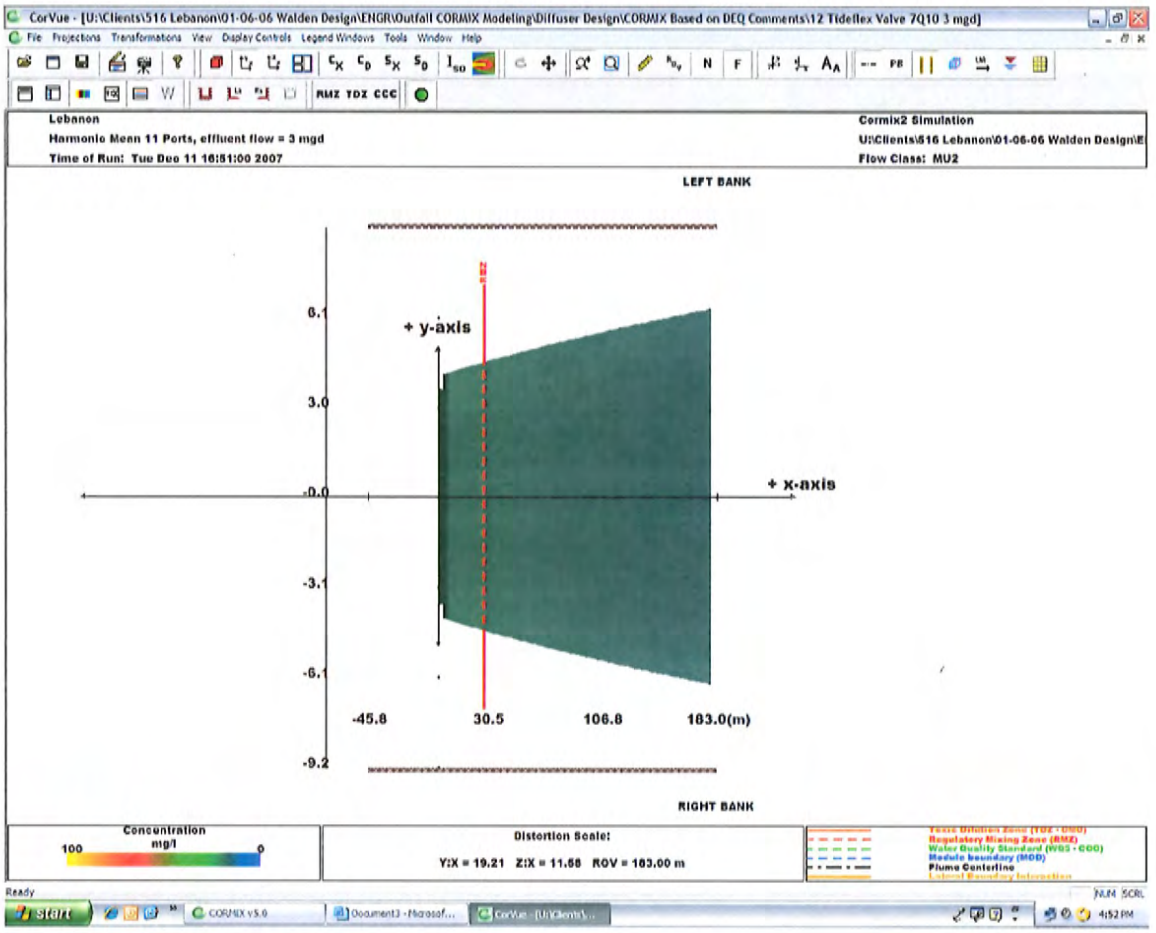
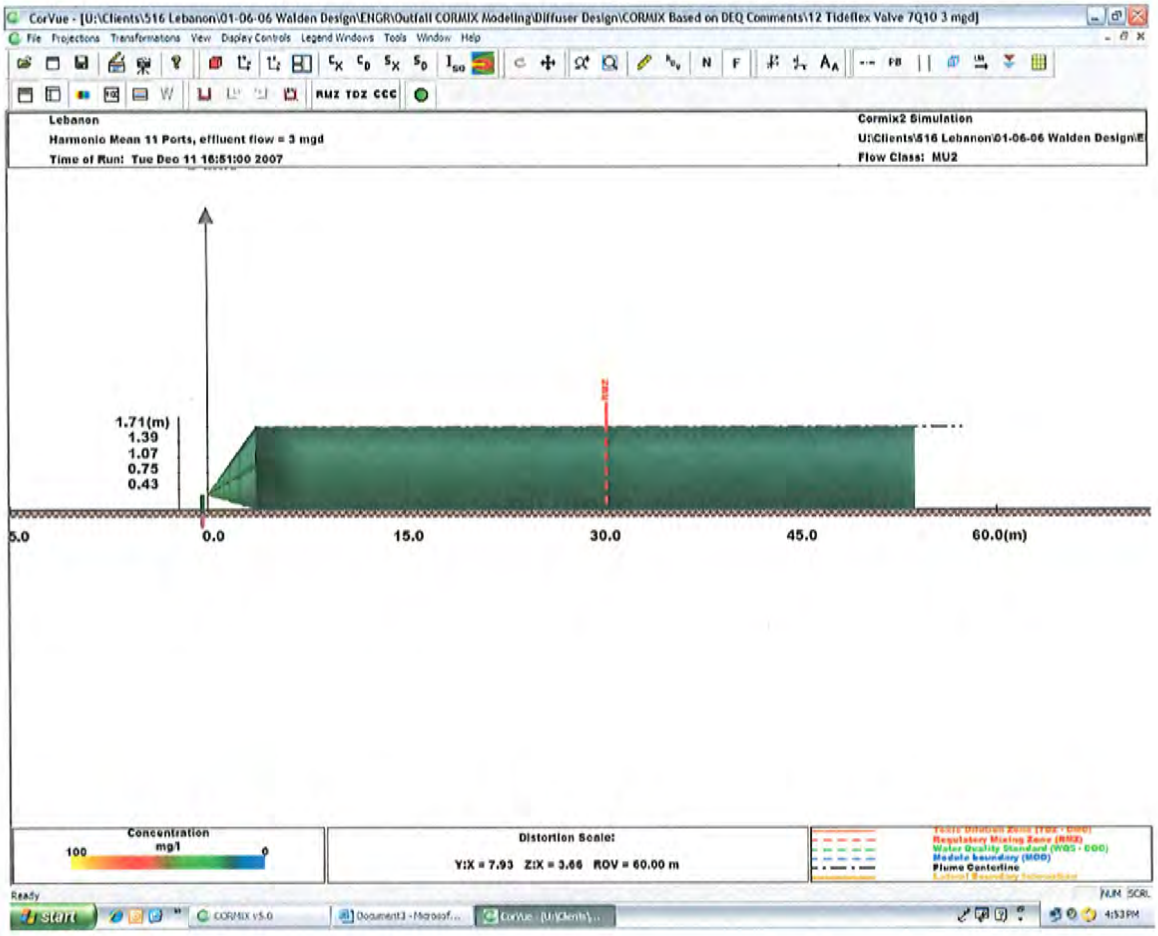
LEFT BANK
+ y-axis
+ x-axis
6.1
3.0
0.0
-3.1
-6.1
-9.2
-45.8 -7.6 30.5 68.6 106.8 144.9 183.0(m)
RIGHT BANK
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 20.33 Z:X = 15.38 ROV = 183.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS • EOD)
Module boundary (MOD)
Plume Centerline
Ready
start
CORMIX v5.0
Document1 - Microsof...
9:46 AM


CorVue - [U:\Clients\516 Lebanon\01-06-06 Walden Design\ENGR\Outfall CORMIX Modeling\Diffuser Design\CORMIX Based on DEQ Comments\12 Tideflex Valve 7Q10 3 mgd]
File Projections Transformations View Display Controls Legend Windows Tools Window Help
Lebanon
30Q5 River Flow, 11 Ports, effluent flow = 8 mgd
Time of Run: Mon Dec 17 09:44:45 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
RMZ
5.8
-7.6
30.5
68.6
106.8
144.9
183.0(m)
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 20.33 Z:X = 16.38 ROV = 183.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - CCC)
Mixing Boundary (MOD)
Plume Centerline
Ready
start
CORMIX v5.0
Document1 - Microsof...
9:48 AM

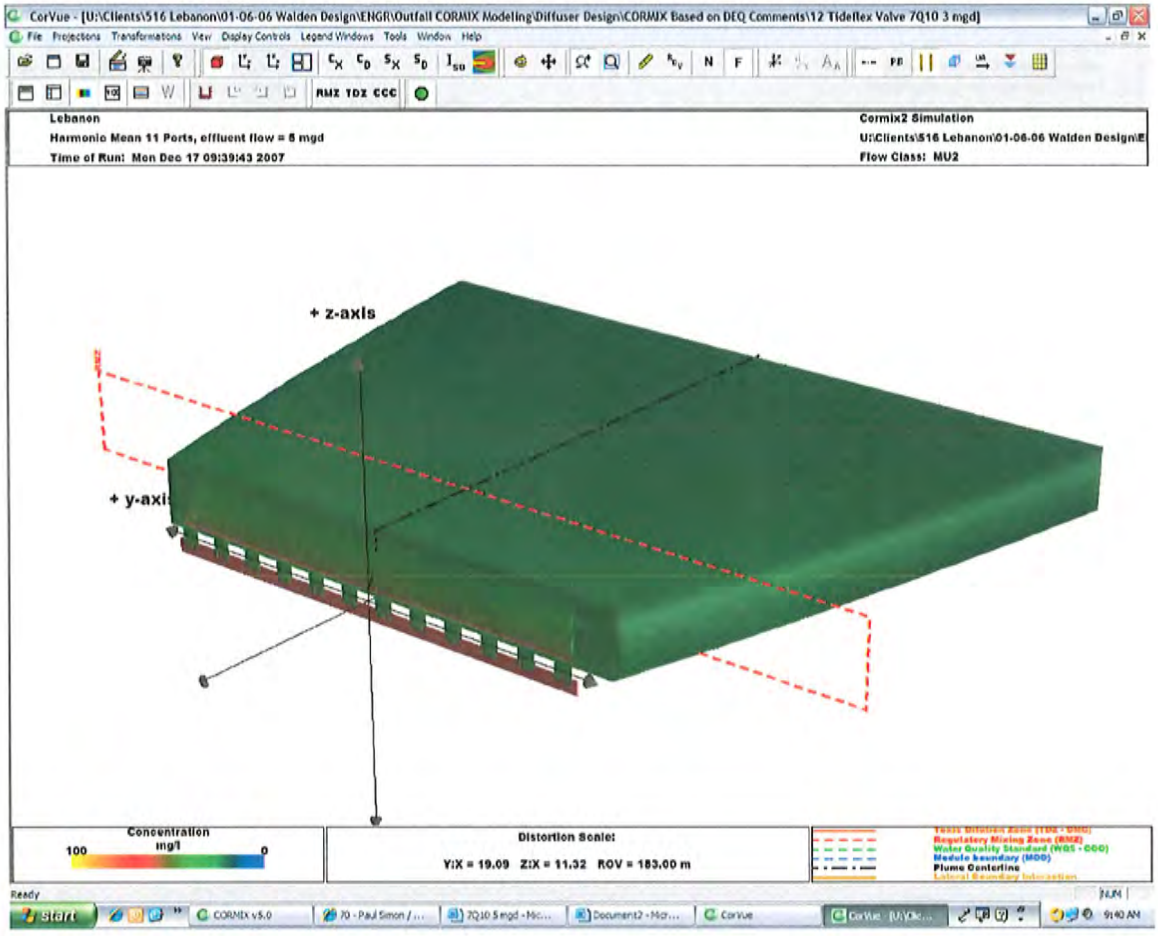
Harmonic Mean River Flow, 3 mgd Effluent Flow

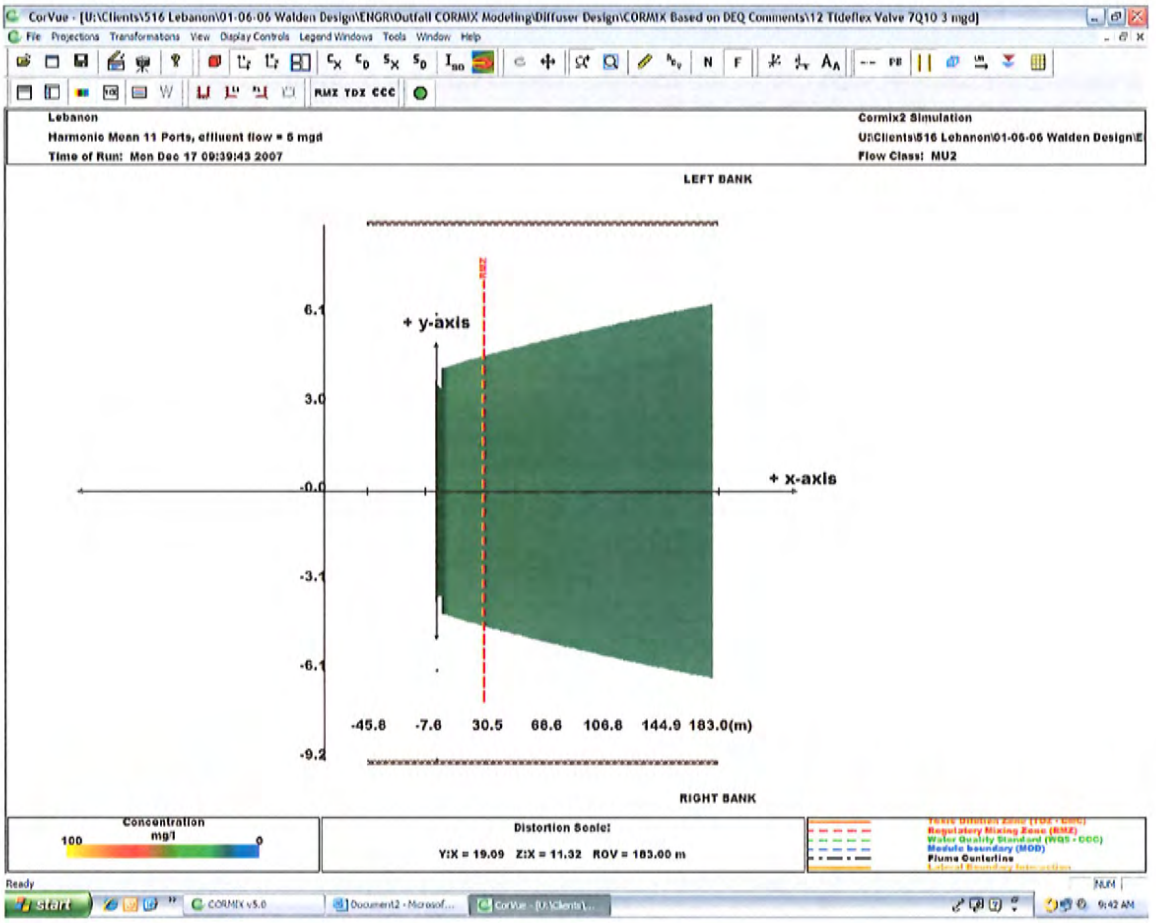


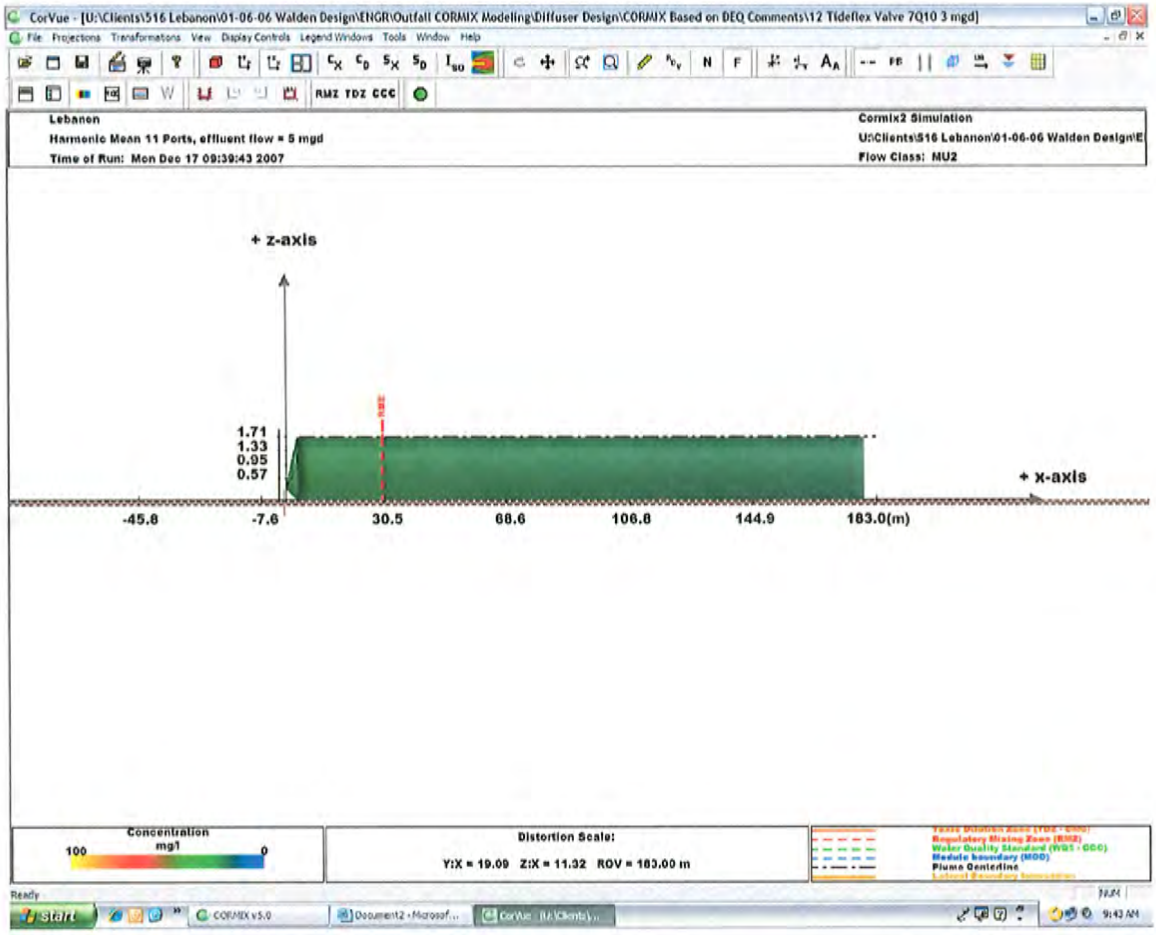

CorVue - [U:\Clients\516 Lebanon\01-06-06 Walden Design\ENGR\Outfall CORMIX Modeling\Diffuser Design\CORMIX Based on DEQ Comments\12 Tideflex Valve 7Q10 3 mgd]
File Projections Transformations View Display Controls Legend Windows Tools Window Help
Lebanon
Harmonic Mean 11 Ports, effluent flow = 3 mgd
Time of Run: Tue Dec 11 16:51:00 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
LEFT BANK
+ y-axis
+ x-axis
6.1
3.0
0.0
-3.1
-6.1
-9.2
-45.8
30.5
106.8
183.0(m)
RIGHT BANK
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 19.21 Z:X = 11.68 ROV = 183.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - CCC)
Module boundary (MOD)
Plume Centerline
Lateral Boundary Interaction
Ready
NUM SCRL
start
CORMIX v5.0
Document3 - Microsof...
CorVue - [U:\Clients\...
4:52 PM


CorVue - [U:\Clients\516 Lebanon\01-06-06 Walden Design\ENGR\Outfall CORMIX Modeling\Diffuser Design\CORMIX Based on DEQ Comments\12 Tideflex Valve 7Q10 3 mgd]
File Projections Transformations View Display Controls Legend Windows Tools Window Help
RMZ TDZ CCC
Lebanon
Harmonic Mean 11 Ports, effluent flow = 3 mgd
Time of Run: Tue Dec 11 16:51:00 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
1.71(m)
1.39
1.07
0.75
0.43
5.0
0.0
15.0
30.0
45.0
60.0(m)
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 7.93 Z:X = 3.66 ROV = 60.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS : CCC)
Module boundary (MOD)
Plume Centerline
Ready
NUM SCRL
start
CORMIX v5.0
Document3 - Microsof...
CorVue - [U:\Clients\...
4:53 PM

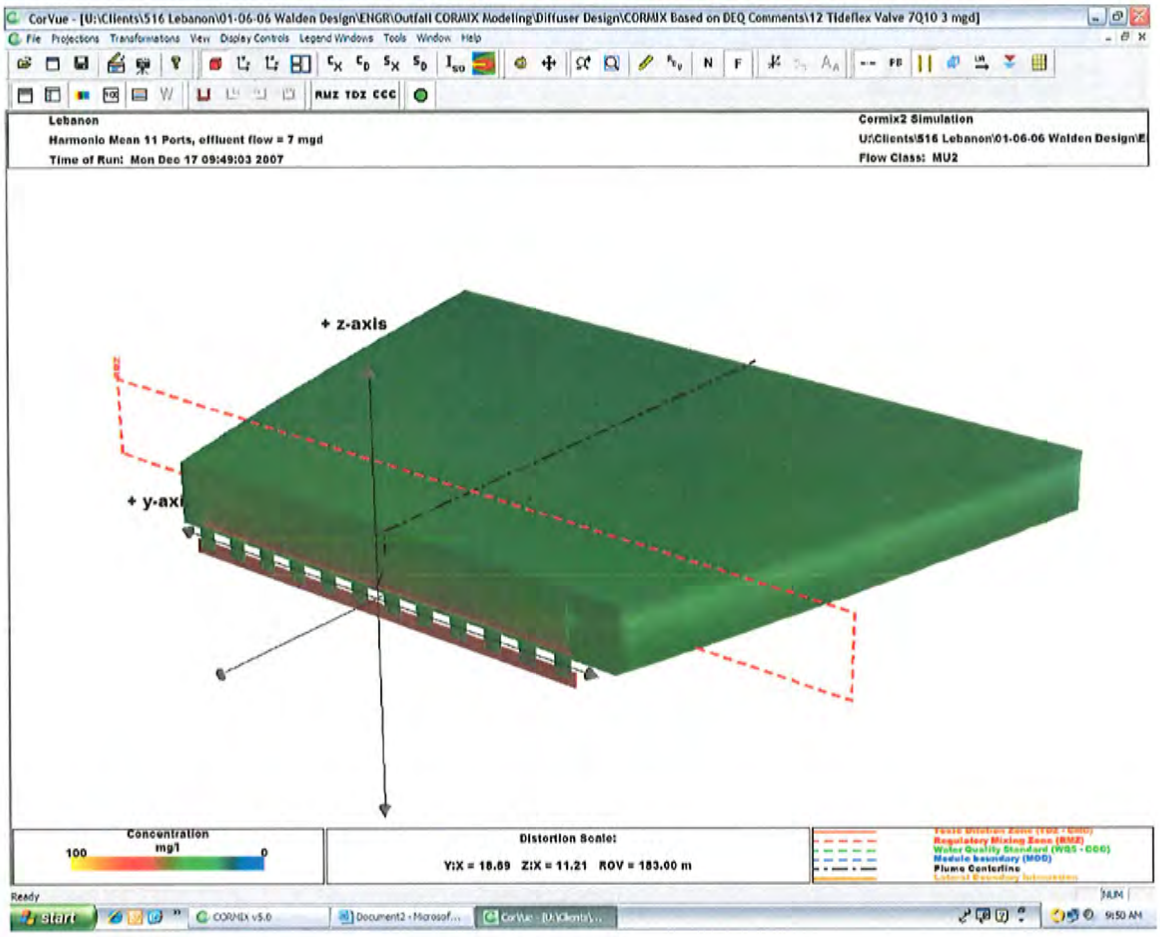
Harmonic Mean River Flow, 5 mgd Effluent Flow

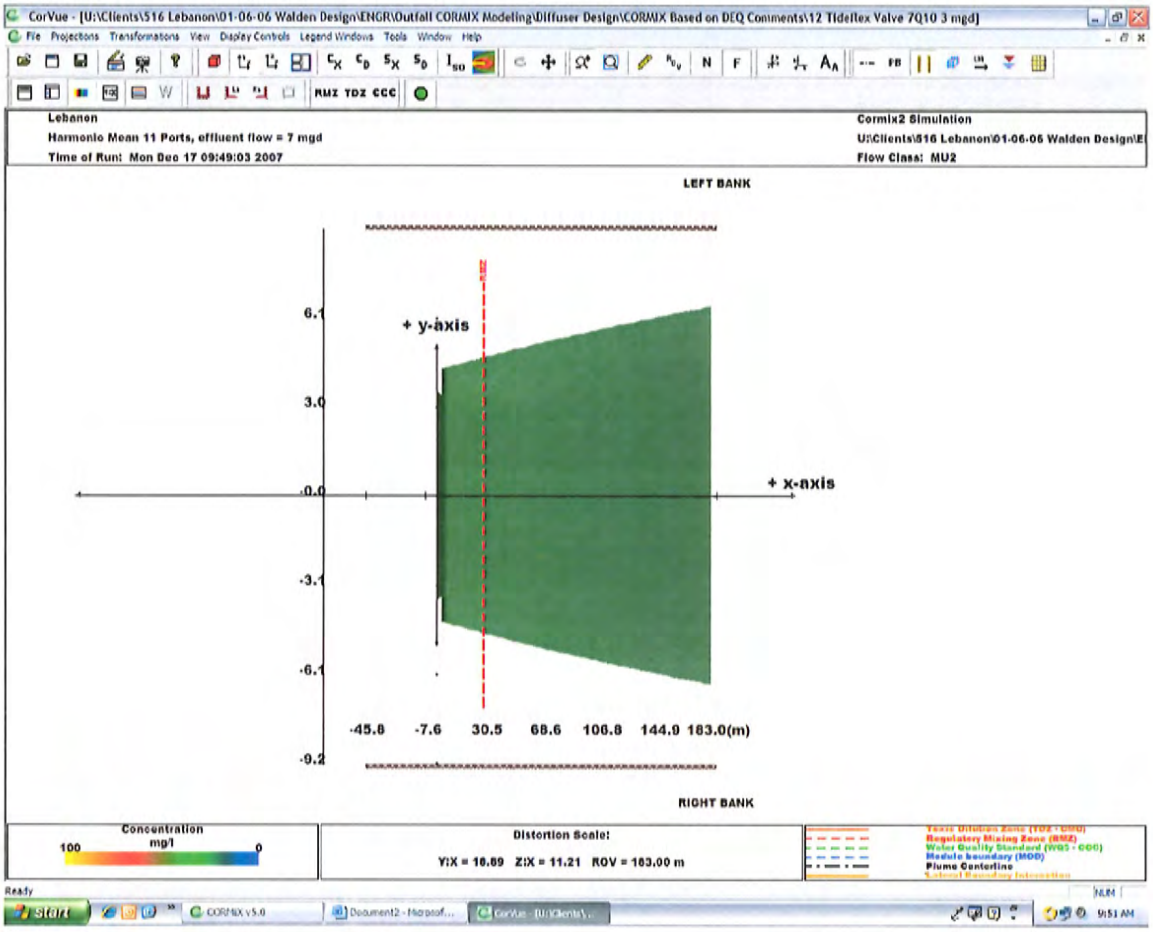


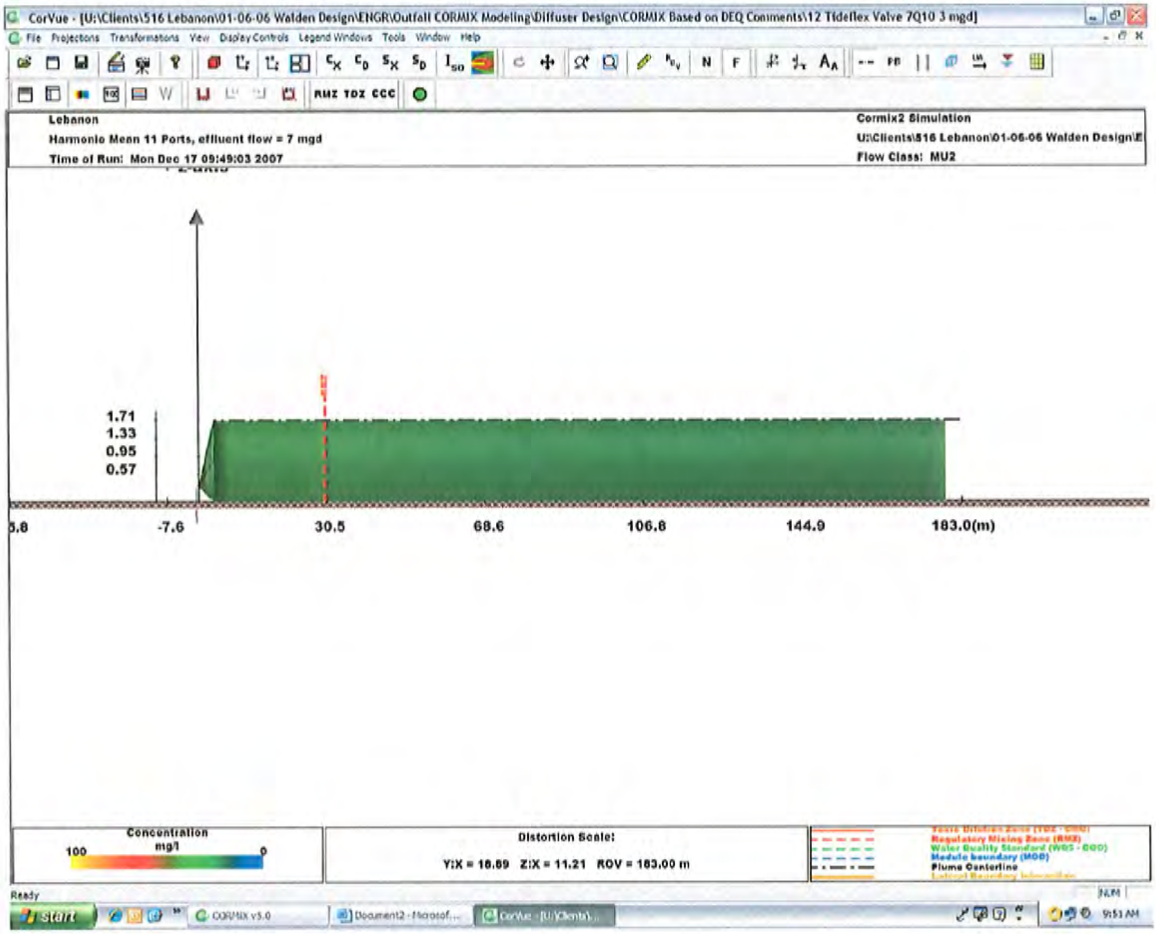

CorVue - [U:\Clients\516 Lebanon\01-06-06 Walden Design\ENGR\Outfall CORMIX Modeling\Diffuser Design\CORMIX Based on DEQ Comments\12 Tideflex Valve 7Q10 3 mgd]
File Projections Transformations View Display Controls Legend Windows Tools Window Help
RMZ TDZ CCC
Lebanon
Harmonic Mean 11 Ports, effluent flow = 5 mgd
Time of Run: Mon Dec 17 09:39:43 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
LEFT BANK
RMZ
+ y-axis
+ x-axis
6.1
3.0
-0.0
-3.1
-6.1
-9.2
-45.8 -7.6 30.5 66.6 106.8 144.9 183.0(m)
RIGHT BANK
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 19.09 Z:X = 11.32 ROV = 183.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - CCC)
Module boundary (MOD)
Plume Centerline
Ready
NUM
start
CORMIX v5.0
Document2 - Microsof...
CorVue - [U:\Clients\...
9:42 AM


CorVue - [U:\Clients\516 Lebanon\01-06-06 Walden Design\ENGR\Outfall CORMIX Modeling\Diffuser Design\CORMIX Based on DEQ Comments\12 Tideflex Valve 7Q10 3 mgd]
File Projections Transformations View Display Controls Legend Windows Tools Window Help
RMZ TDZ CCC
Lebanon
Harmonic Mean 11 Ports, effluent flow = 5 mgd
Time of Run: Mon Dec 17 09:39:43 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
+ z-axis
1.71
1.33
0.95
0.57
+ x-axis
-45.8
-7.6
30.5
68.6
106.8
144.9
183.0(m)
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 19.09 Z:X = 11.32 ROV = 183.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS : CCC)
Module boundary (MOD)
Plume Centerline
Ready
start
CORMIX v5.0
Document2 - Microsof...
CorVue - [U:\Clients\...
9:43 AM

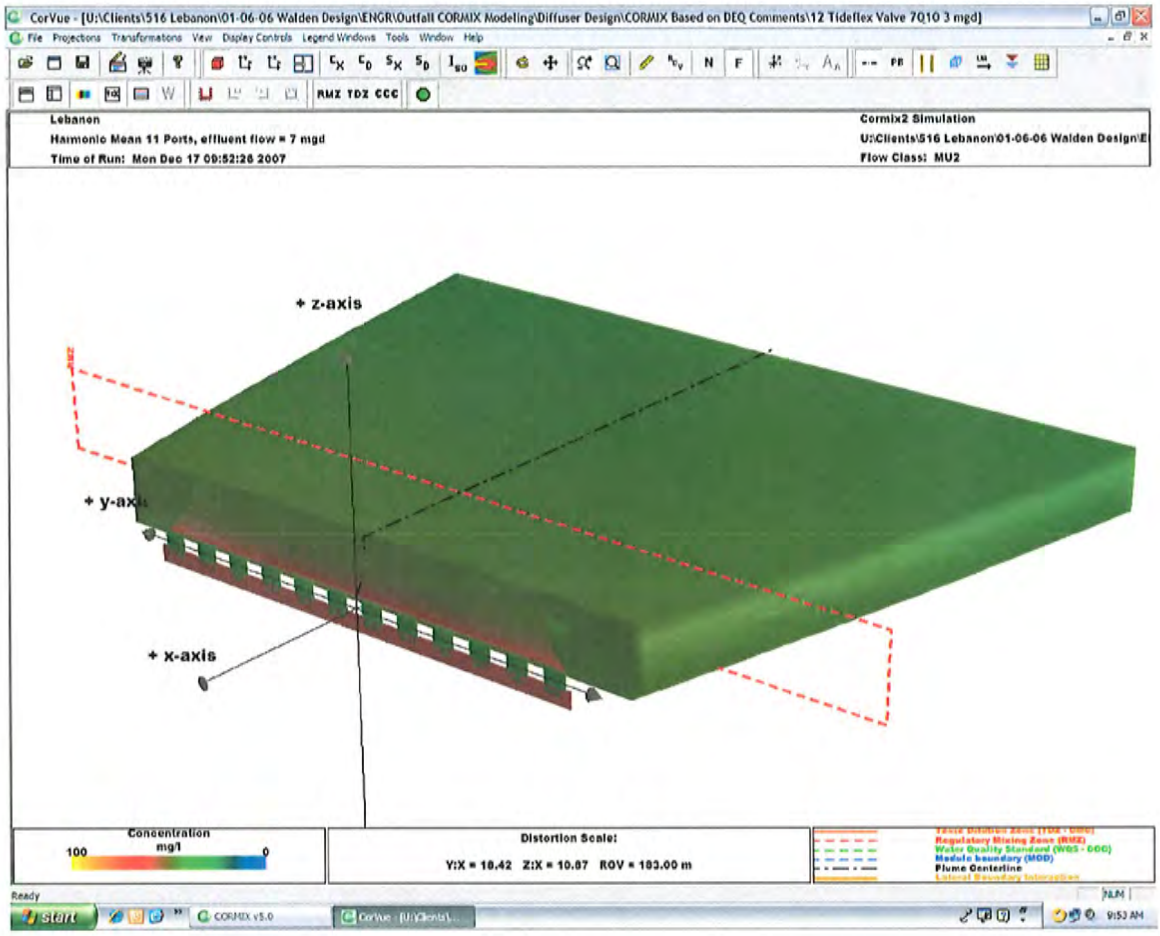
Harmonic Mean River Flow, 7 mgd Effluent Flow

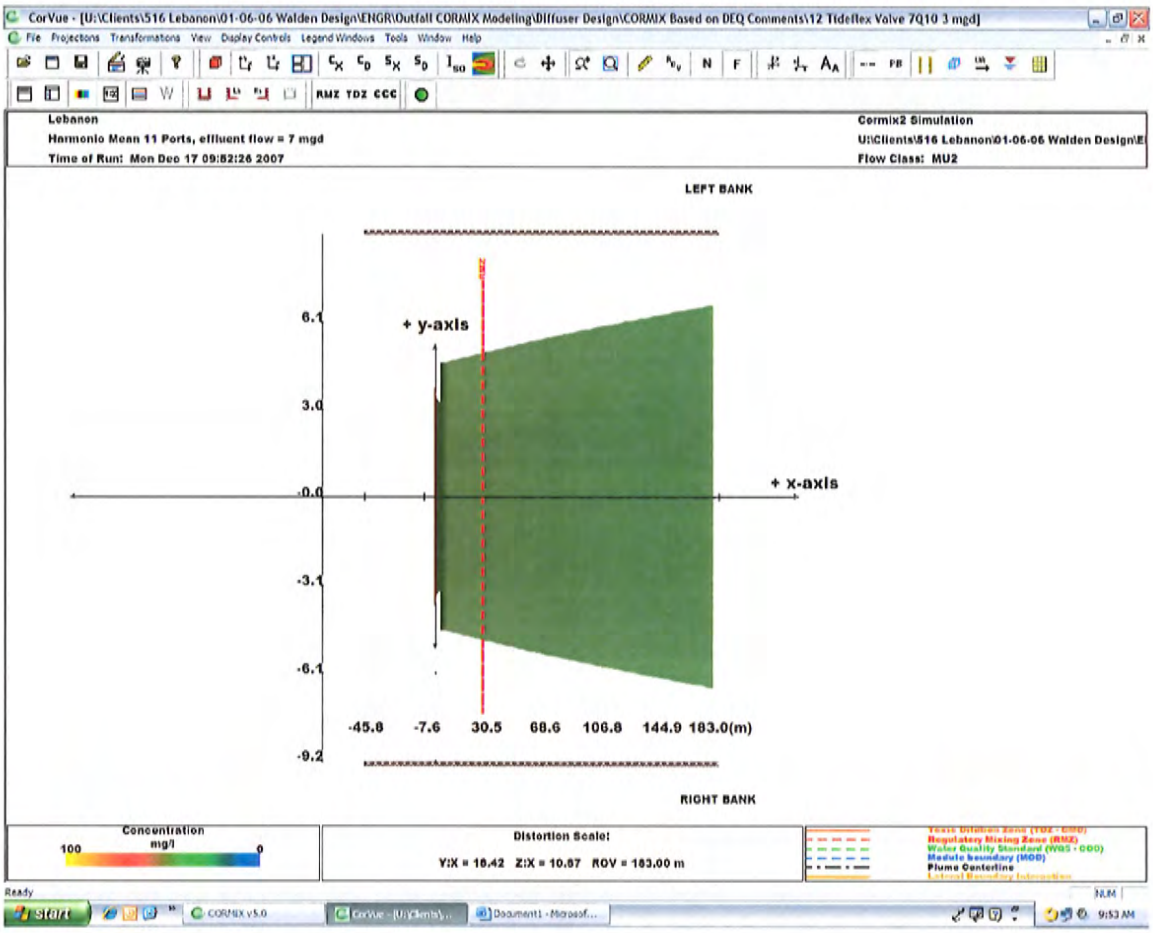


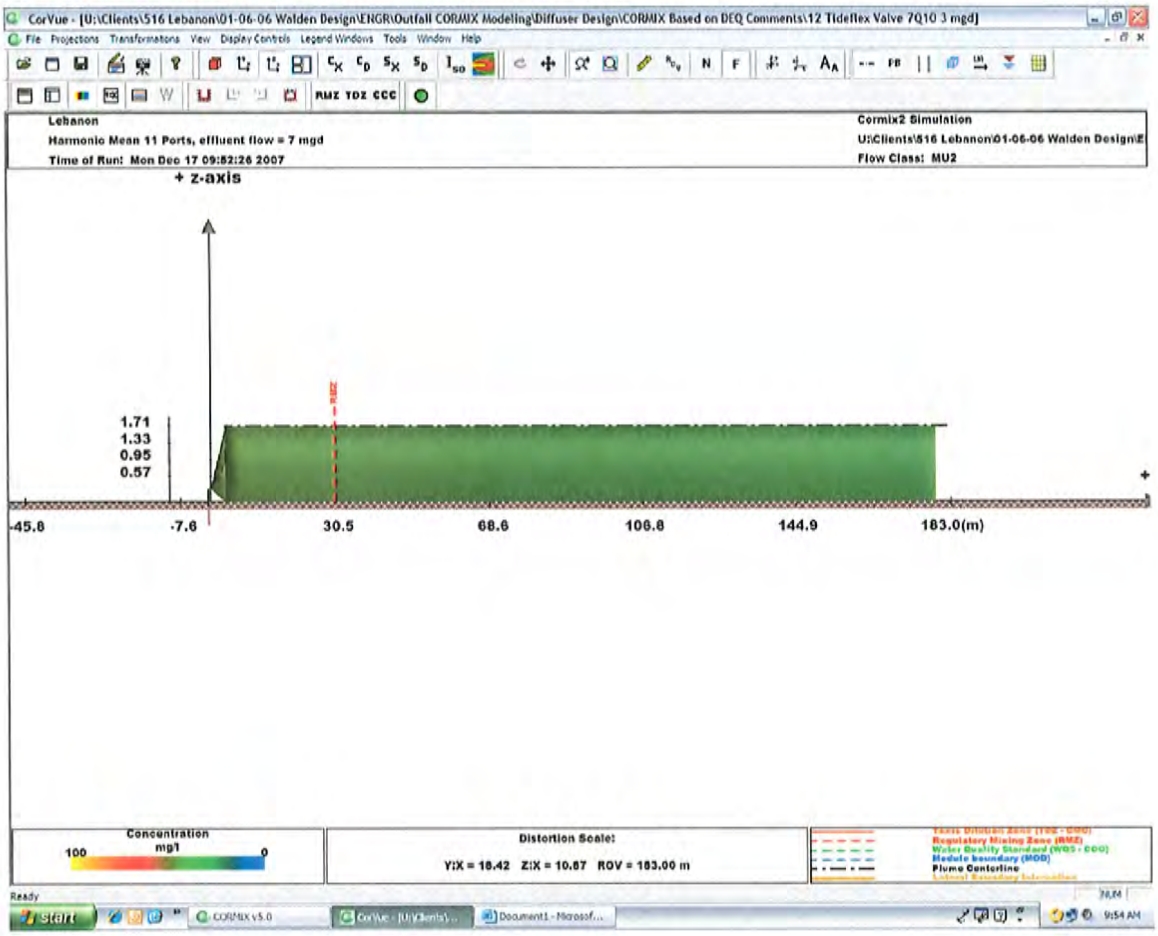

CorVue - [U:\Clients\516 Lebanon\01-06-06 Walden Design\ENGR\Outfall CORMIX Modeling\Diffuser Design\CORMIX Based on DEQ Comments\12 Tideflex Valve 7Q10 3 mgd]
File Projections Transformations View Display Controls Legend Windows Tools Window Help
Lebanon
Harmonic Mean 11 Ports, effluent flow = 7 mgd
Time of Run: Mon Dec 17 09:49:03 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
LEFT BANK
+ y-axis
+ x-axis
6.1
3.0
0.0
-3.1
-6.1
-9.2
-45.8
-7.6
30.5
68.6
106.8
144.9
183.0(m)
RIGHT BANK
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 16.89 Z:X = 11.21 ROV = 183.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - CCC)
Module boundary (MOD)
Plume Centerline
Ready
Start
CORMIX v5.0
Document2 - Microsof...
CorVue - [U:\Clients\...
9:51 AM


CorVue - [U:\Clients\516 Lebanon\01-06-06 Walden Design\ENGR\Outfall CORMIX Modeling\Diffuser Design\CORMIX Based on DEQ Comments\12 Tideflex Valve 7Q10 3 mgd]
File Projections Transformations View Display Controls Legend Windows Tools Window Help
Lebanon
Harmonic Mean 11 Ports, effluent flow = 7 mgd
Time of Run: Mon Dec 17 09:49:03 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
1.71
1.33
0.95
0.57
-7.6
30.5
68.6
106.8
144.9
183.0(m)
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 16.69 Z:X = 11.21 ROV = 183.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - EOO)
Module boundary (MOD)
Plume Centerline
Ready
start
CORMIX v5.0
Document2 - Microsof...
CorVue - [U:\Clients\...
9:51 AM

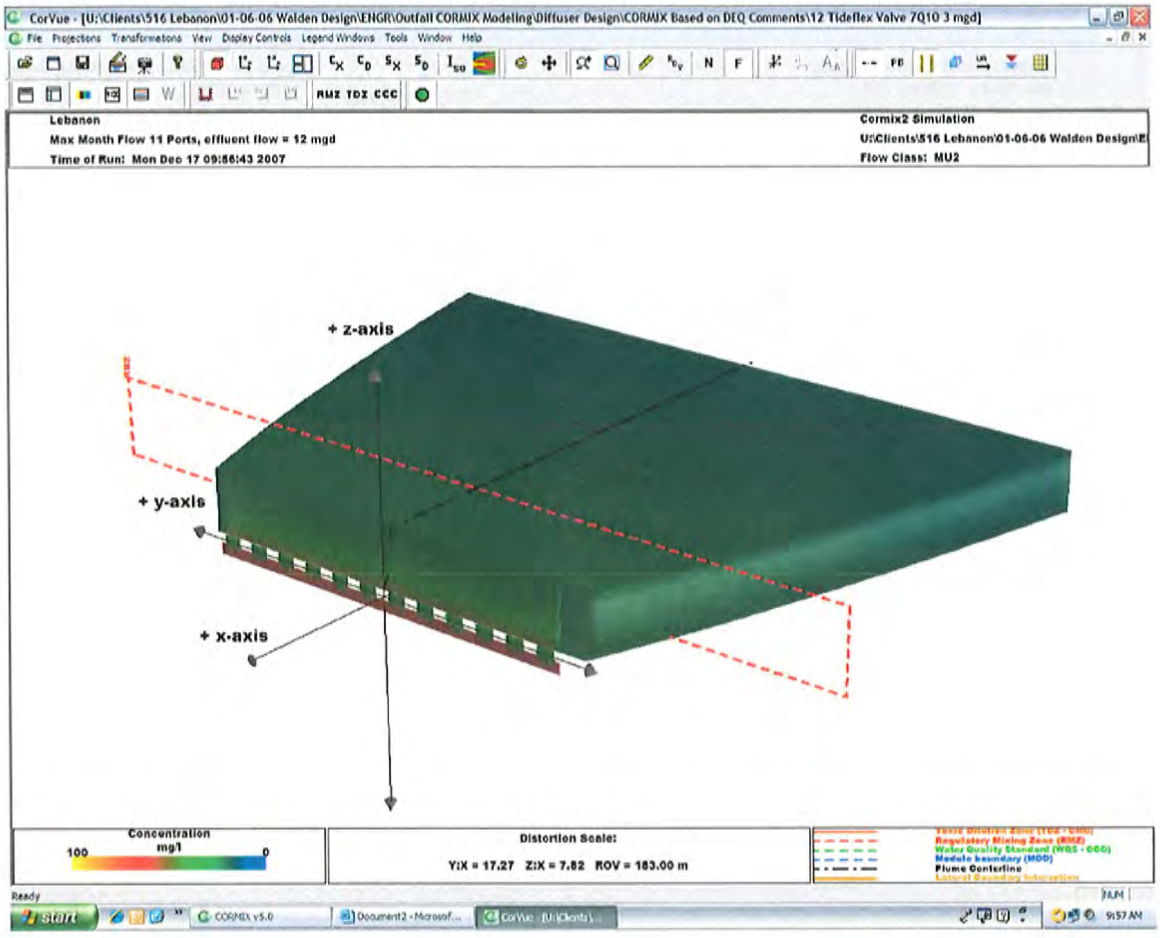
Harmonic Mean River Flow, 12 mgd Effluent Flow

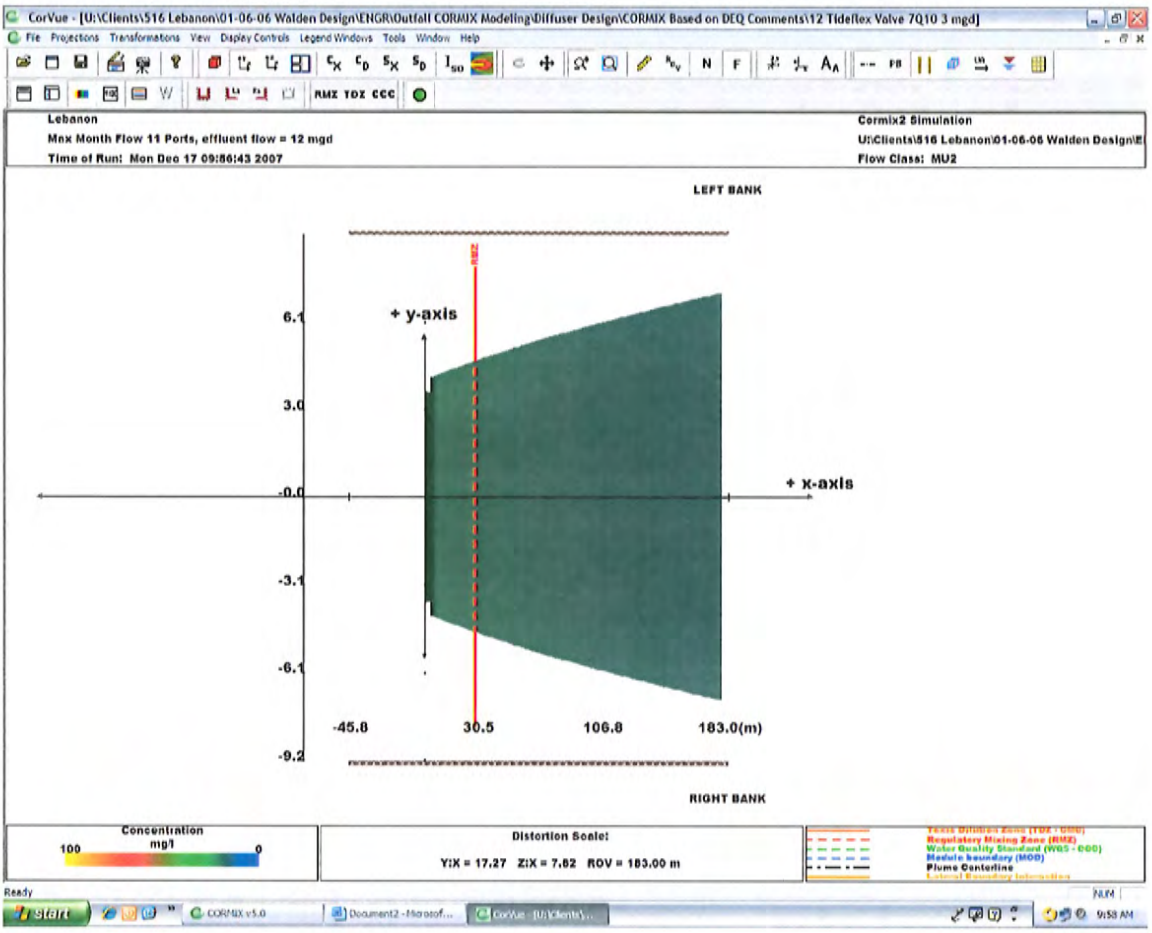


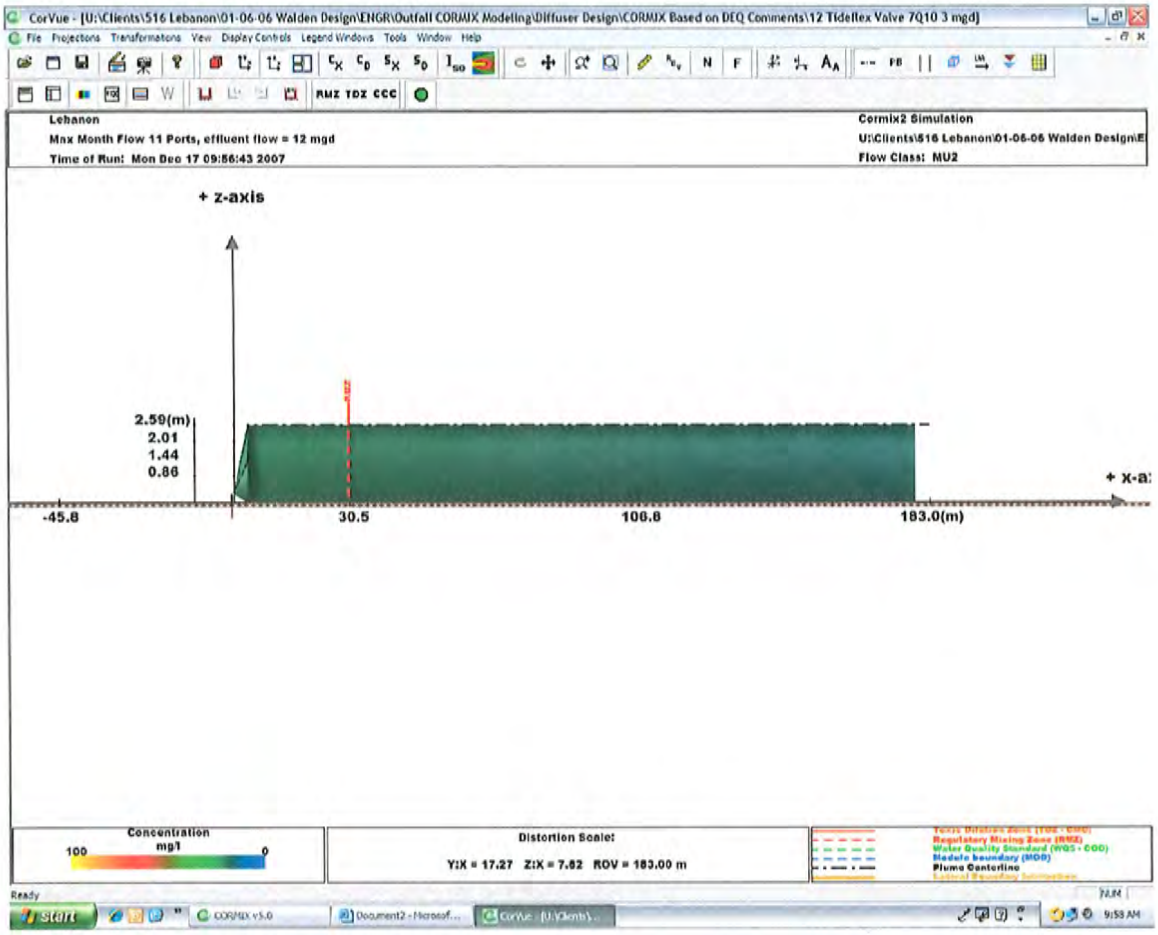

CorVue - [U:\Clients\516 Lebanon\01-06-06 Walden Design\ENGR\Outfall CORMIX Modeling\Diffuser Design\CORMIX Based on DEQ Comments\12 Tideflex Valve 7Q10 3 mgd]
File Projections Transformations View Display Controls Legend Windows Tools Window Help
Lebanon
Harmonic Mean 11 Ports, effluent flow = 7 mgd
Time of Run: Mon Dec 17 09:52:26 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
LEFT BANK
+ y-axis
+ x-axis
6.1
3.0
-0.0
-3.1
-6.1
-9.2
-45.8
-7.6
30.5
68.6
106.8
144.9
183.0(m)
RIGHT BANK
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 18.42 Z:X = 10.67 ROV = 183.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - GDO)
Module boundary (MOD)
Plume Centerline
Ready
NUM
start
CORMIX v5.0
Document1 - Microsof...
9:53 AM


CorVue - [U:\Clients\516 Lebanon\01-06-06 Walden Design\ENGR\Outfall CORMIX Modeling\Diffuser Design\CORMIX Based on DEQ Comments\12 Tideflex Valve 7Q10 3 mgd]
File Projections Transformations View Display Controls Legend Windows Tools Window Help
RMZ TDZ CCC
Lebanon
Harmonic Mean 11 Ports, effluent flow = 7 mgd
Time of Run: Mon Dec 17 09:52:26 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
+ z-axis
RMZ
1.71
1.33
0.95
0.57
-45.8
-7.6
30.5
68.6
106.8
144.9
183.0(m)
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 18.42 Z:X = 10.67 ROV = 183.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - CCC)
Module boundary (MOD)
Plume Centerline
Ready
NUM
Start
CORMIX v5.0
Document1 - Microsof...
9:54 AM

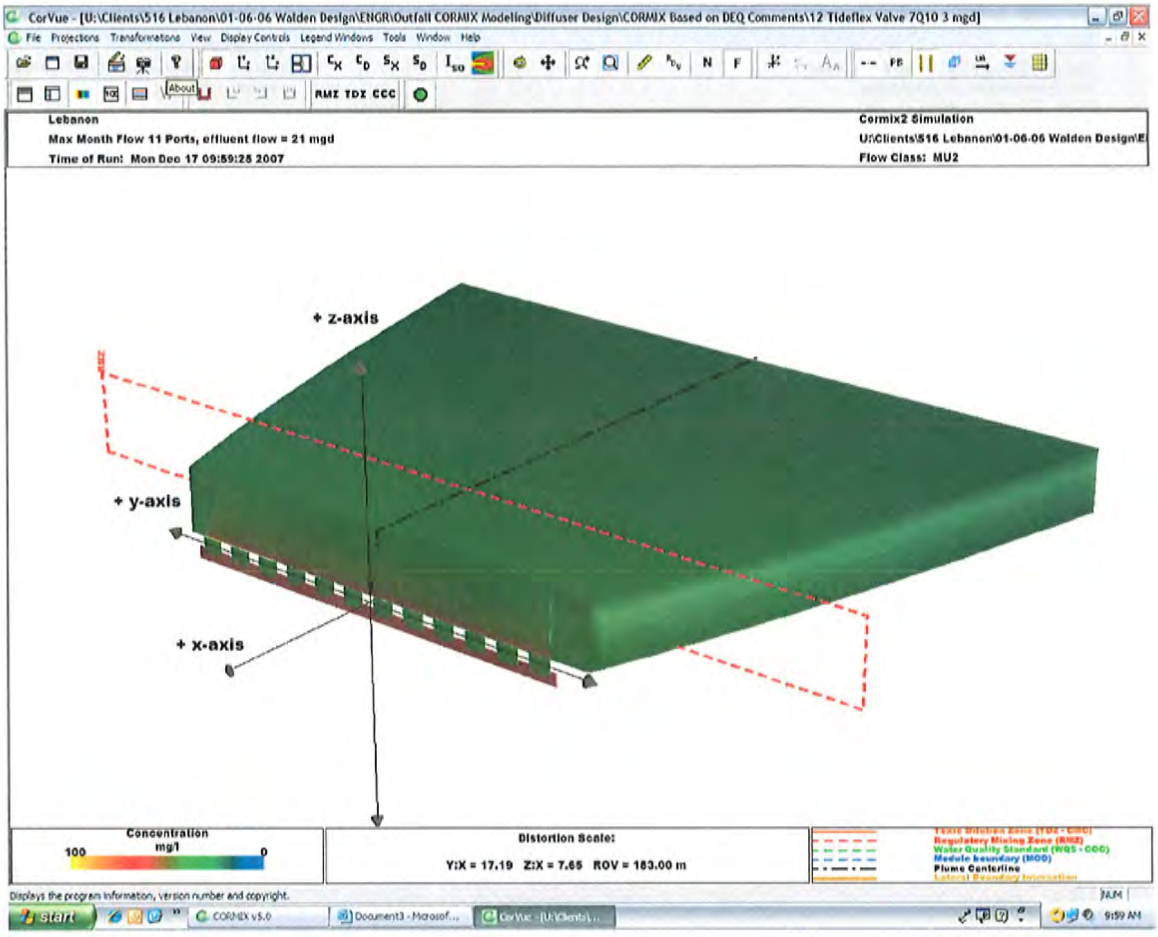
Maximum Month River Flow, 12 mgd Effluent Flow

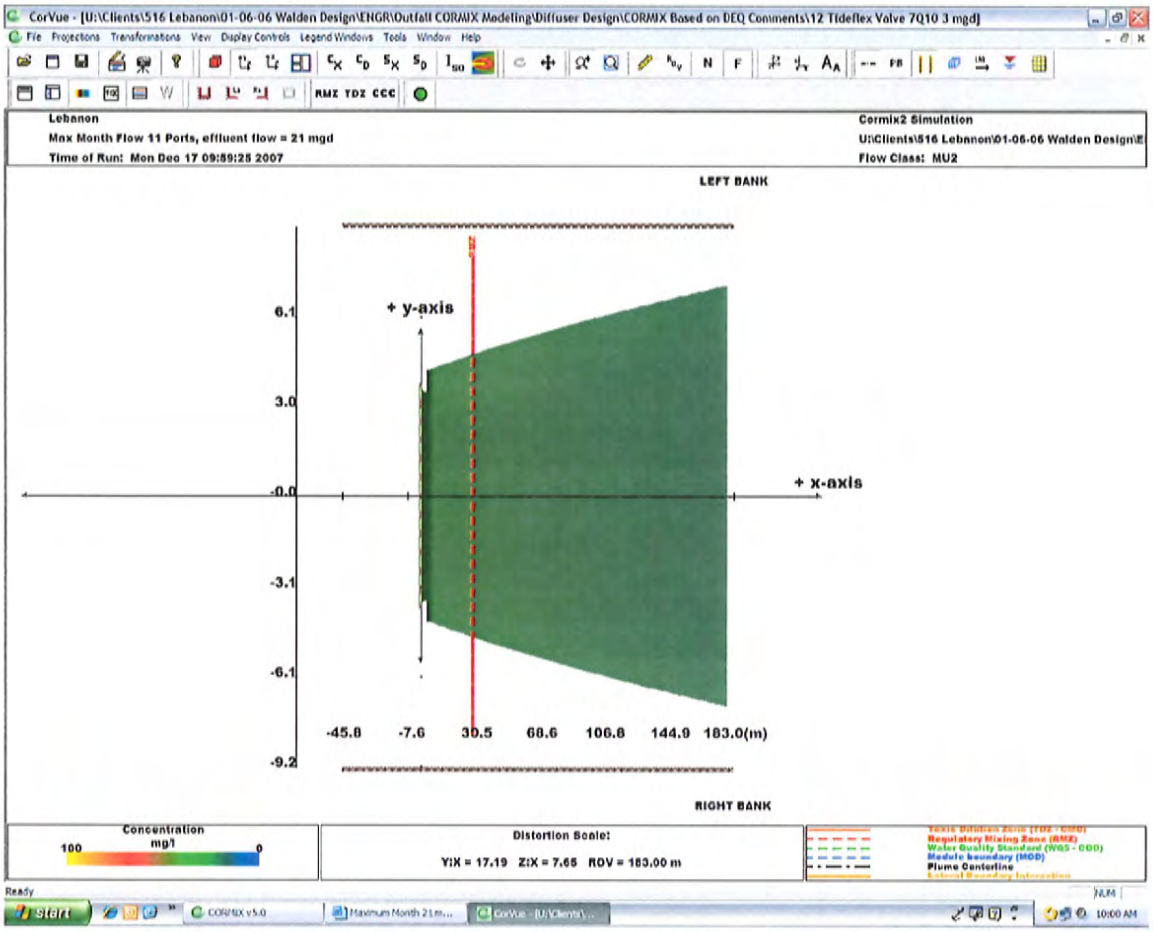


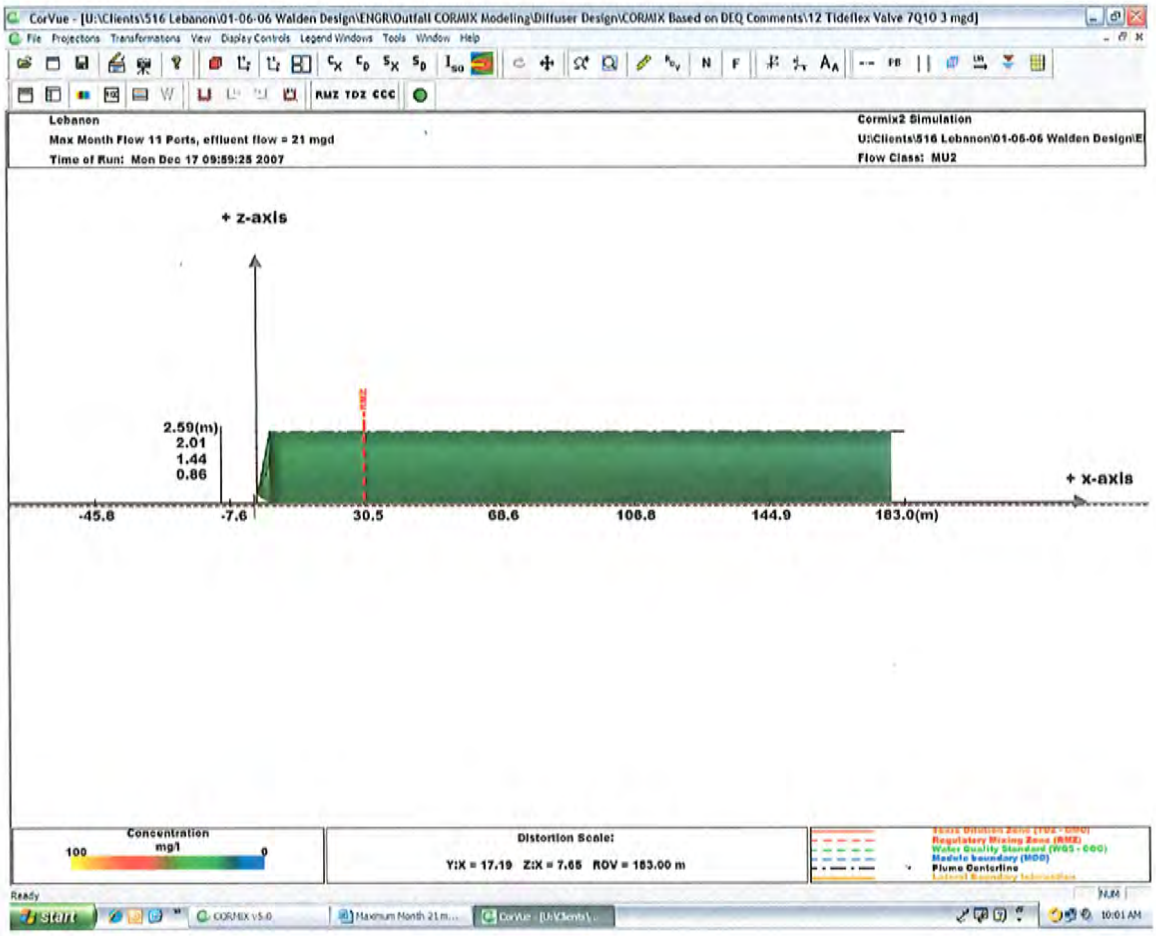

CorVue - [U:\Clients\516 Lebanon\01-06-06 Walden Design\ENGR\Outfall CORMIX Modeling\Diffuser Design\CORMIX Based on DEQ Comments\12 Tideflex Valve 7Q10 3 mgd]
File Projections Transformations View Display Controls Legend Windows Tools Window Help
Lebanon
Max Month Flow 11 Ports, effluent flow = 12 mgd
Time of Run: Mon Dec 17 09:56:43 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
LEFT BANK
RMZ
+ y-axis
+ x-axis
6.1
3.0
-0.0
-3.1
-6.1
-9.2
-45.8
30.5
106.8
183.0(m)
RIGHT BANK
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 17.27 Z:X = 7.62 ROV = 183.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - C00)
Mixing Zone Boundary (MOD)
Plume Centerline
Ready
NUM
start
CORMIX v5.0
Document2 - Microsof...
CorVue - [U:\Clients\...
9:58 AM


CorVue - [U:\Clients\516 Lebanon\01-06-06 Walden Design\ENGR\Outfall CORMIX Modeling\Diffuser Design\CORMIX Based on DEQ Comments\12 Tideflex Valve 7Q10 3 mgd]
File Projections Transformations View Display Controls Legend Windows Tools Window Help
Lebanon
Max Month Flow 11 Ports, effluent flow = 12 mgd
Time of Run: Mon Dec 17 09:56:43 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
+ z-axis
2.59(m)
2.01
1.44
0.86
+ x-a
-45.8
30.5
106.8
183.0(m)
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 17.27 Z:X = 7.62 ROV = 183.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - COO)
Module boundary (MOD)
Plume Centerline
Ready
start
CORMIX v5.0
Document2 - Microsof...
CorVue - [U:\Clients\...
9:58 AM

Maximum Month River Flow, 21 mgd Effluent Flow




CorVue - [U:\Clients\516 Lebanon\01-06-06 Walden Design\ENGR\Outfall CORMIX Modeling\Diffuser Design\CORMIX Based on DEQ Comments\12 Tideflex Valve 7Q10 3 mgd]
File Projections Transformations View Display Controls Legend Windows Tools Window Help
RMZ TDZ CCC
Lebanon
Max Month Flow 11 Ports, effluent flow = 21 mgd
Time of Run: Mon Dec 17 09:59:25 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
LEFT BANK
+ y-axis
+ x-axis
6.1
3.0
-0.0
-3.1
-6.1
-9.2
-45.8
-7.6
30.5
68.6
106.8
144.9
183.0(m)
RIGHT BANK
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 17.19 Z:X = 7.65 ROV = 183.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - 000)
Module boundary (MOD)
Plume Centerline
Ready
NUM
start
CORMIX v5.0
Maximum Month 21 m...
CorVue - [U:\Clients\...
10:00 AM


CorVue - [U:\Clients\516 Lebanon\01-06-06 Walden Design\ENGR\Outfall CORMIX Modeling\Diffuser Design\CORMIX Based on DEQ Comments\12 Tideflex Valve 7Q10 3 mgd]
File Projections Transformations View Display Controls Legend Windows Tools Window Help
Lebanon
Max Month Flow 11 Ports, effluent flow = 21 mgd
Time of Run: Mon Dec 17 09:59:25 2007
Cormix2 Simulation
U:\Clients\516 Lebanon\01-06-06 Walden Design\E
Flow Class: MU2
+ z-axis
2.59(m)
2.01
1.44
0.86
+ x-axis
-45.6
-7.6
30.5
68.6
106.8
144.9
183.0(m)
Concentration
mg/l
100
0
Distortion Scale:
Y:X = 17.19 Z:X = 7.65 ROV = 183.00 m
Regulatory Mixing Zone (RMZ)
Water Quality Standard (WQS - COC)
Module boundary (MOD)
Plume Centerline
Ready
start
CORMIX v5.0
Maximum Month 21 m...
CorVue - [U:\Clients\...
10:01 AM

ATTACHMENT 2

```
CORMIX2 PREDICTION FILE:
2222222222222222222222222222222222222222222222222222222222222222222222222222222222
                        CORMIX MIXING ZONE EXPERT SYSTEM
          Subsystem CORMIX2:  Multiport Diffuser Discharges
                              CORMIX Version 5.0GT
                           HYDRO2 Version 5.0.0.0 March 2007
------------------------------------------------------------------------------
------------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:    Lebanon
 Design case:        7Q10 River Flow, 11 Ports, effluent flow = 3 mgd
 FILE NAME:          U:\...3 mgd modeling\Length = 24 Effluent Flow 3mgd.prd
 Time stamp:         Tue Dec 11 15:28:56 2007

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS     =      18.29  AS     =      19.57  QA     =       6.85  ICHREG= 1
 HA     =       1.07  HD     =       1.07
 UA     =      0.350  F      =      0.094 USTAR =0.3793E-01
 UW     =      2.000 UWSTAR=0.2198E-02
 Uniform density environment
 STRCND=  U          RHOAM =  998.4062

DIFFUSER DISCHARGE PARAMETERS (metric units)
 Diffuser type:      DITYPE= unidirectional_perpendicular
 BANK   =  LEFT      DISTB =       9.10  YB1    =       5.40  YB2    =      12.80
 LD     =      7.42  NOPEN =    11       SPAC   =       0.74
 D0     =     0.114 A0     =      0.010 H0     =       0.31  SUB0   =       0.76
 Nozzle/port arrangement:   unidirectional_with_fanning
 GAMMA =      90.00  THETA =      10.00  SIGMA =       0.00  BETA   =      90.00
 U0     =      1.167 Q0     =      0.131        =0.1310E+00
 RHO0   =  997.8837  DRHO0 =0.5226E+00  GP0    =0.5133E-02
 C0     =0.1000E+03  CUNITS=  mg/l
 IPOLL =  1          KS     =0.0000E+00  KD     =0.0000E+00

FLUX VARIABLES - PER UNIT DIFFUSER LENGTH (metric units)
 q0     =0.1765E-01  m0     =0.2060E-01  j0     =0.9062E-04  SIGNJ0=       1.0
 Associated 2-d length scales (meters)
 lQ=B   =      0.015 lM     =      10.19  lm     =       0.17
 lmp    =  99999.00  lbp    =  99999.00  la     =   99999.00

FLUX VARIABLES - ENTIRE DIFFUSER (metric units)
 Q0     =0.1310E+00  M0     =0.1528E+00  J0     =0.6724E-03
 Associated 3-d length scales (meters)
 LQ     =      0.10  LM     =       9.43  Lm     =       1.12  Lb     =       0.02
                                          Lmp    =   99999.00  Lbp    =   99999.00

NON-DIMENSIONAL PARAMETERS
 FR0    =     132.39  FRD0  =      48.23  R      =       3.33  PL     =       3.
 (slot)               (port/nozzle)

RECOMPUTED SOURCE CONDITIONS FOR RISER GROUPS:
 Properties of riser group with  1 ports/nozzles each:
 U0     =      1.167 D0     =      0.114 A0     =      0.010 THETA =      10.00
 FR0    =     132.39  FRD0  =      48.23  R      =       3.33
 (slot)               (riser group)

FLOW CLASSIFICATION
 22222222222222222222222222222222222222
 2  Flow class (CORMIX2)       =    MU2    2
 2  Applicable layer depth HS =      1.07  2
 22222222222222222222222222222222222222
```

```
MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0    =0.1000E+03  CUNITS=  mg/l
 NTOX  =  0
 NSTD  =  0
 REGMZ =  1
 REGSPC=  1         XREG  =     30.48  WREG  =      0.00  AREG  =      0.00
 XINT  =    183.00  XMAX  =    183.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the bottom and the diffuser mid-point:
         9.10 m  from the LEFT  bank/shore.
    X-axis points downstream, Y-axis points to left, Z-axis points upward.
NSTEP =  50 display intervals per module
-------------------------------------------------------------------------------
-------------------------------------------------------------------------------
BEGIN MOD201: DIFFUSER DISCHARGE MODULE

 Due to complex near-field motions:  EQUIVALENT SLOT DIFFUSER (2-D) GEOMETRY

 Profile definitions:
   BV = Gaussian 1/e (37%) half-width, in vertical plane normal to trajectory
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic centerline dilution
   C  = centerline concentration (includes reaction effects, if any)

       X        Y        Z        S        C        BV       BH
    0.00     0.00     0.31     1.0 0.100E+03   0.01     3.71

END OF MOD201: DIFFUSER DISCHARGE MODULE
-------------------------------------------------------------------------------
-------------------------------------------------------------------------------
BEGIN MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

 Because of the FANNED-OUT HORIZONTAL ORIENTATION of the diffuser jets,
   the near-field dilution is slightly improved.

 In this laterally contracting zone the diffuser plume becomes VERTICALLY FULLY
  MIXED over the entire layer depth (HS =    1.07m).
   Full mixing is achieved after a plume distance of about five
   layer depths from the diffuser.

 Profile definitions:
   BV = layer depth (vertically mixed)
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

       X        Y        Z        S        C        BV       BH
    0.00     0.00     0.31     1.0 0.100E+03   0.01     3.71
    0.07     0.00     0.31     4.4 0.227E+02   0.03     3.70
    0.15     0.00     0.31     5.8 0.172E+02   0.04     3.69
    0.22     0.00     0.32     6.9 0.145E+02   0.06     3.67
    0.30     0.00     0.32     7.8 0.128E+02   0.09     3.66
    0.37     0.00     0.33     8.6 0.116E+02   0.11     3.65
    0.45     0.00     0.33     9.3 0.107E+02   0.13     3.64
    0.52     0.00     0.34    10.0 0.100E+02   0.15     3.64
    0.59     0.00     0.34    10.6 0.943E+01   0.17     3.63
    0.67     0.00     0.35    11.2 0.894E+01   0.19     3.62
    0.74     0.00     0.35    11.7 0.852E+01   0.21     3.61
    0.82     0.00     0.36    12.3 0.815E+01   0.24     3.60
    0.89     0.00     0.36    12.8 0.783E+01   0.26     3.60
    0.96     0.00     0.36    13.2 0.755E+01   0.28     3.59
    1.04     0.00     0.37    13.7 0.729E+01   0.30     3.58
```

1.11	0.00	0.37	14.2	0.706E+01	0.32	3.58
1.19	0.00	0.38	14.6	0.686E+01	0.34	3.57
1.26	0.00	0.38	15.0	0.666E+01	0.36	3.57
1.34	0.00	0.39	15.4	0.649E+01	0.39	3.56
1.41	0.00	0.39	15.8	0.633E+01	0.41	3.56
1.48	0.00	0.40	16.2	0.618E+01	0.43	3.55
1.56	0.00	0.40	16.6	0.604E+01	0.45	3.55
1.63	0.00	0.41	16.9	0.591E+01	0.47	3.54
1.71	0.00	0.41	17.3	0.578E+01	0.49	3.54
1.78	0.00	0.42	17.6	0.567E+01	0.51	3.54
1.86	0.00	0.42	18.0	0.556E+01	0.54	3.53
1.93	0.00	0.42	18.3	0.546E+01	0.56	3.53
2.00	0.00	0.43	18.7	0.536E+01	0.58	3.53
2.08	0.00	0.43	19.0	0.527E+01	0.60	3.52
2.15	0.00	0.44	19.3	0.518E+01	0.62	3.52
2.23	0.00	0.44	19.6	0.510E+01	0.64	3.52
2.30	0.00	0.45	19.9	0.502E+01	0.66	3.51
2.37	0.00	0.45	20.2	0.495E+01	0.68	3.51
2.45	0.00	0.46	20.5	0.487E+01	0.71	3.51
2.52	0.00	0.46	20.8	0.481E+01	0.73	3.51
2.60	0.00	0.47	21.1	0.474E+01	0.75	3.50
2.67	0.00	0.47	21.4	0.468E+01	0.77	3.50
2.75	0.00	0.48	21.7	0.462E+01	0.79	3.50
2.82	0.00	0.48	21.9	0.456E+01	0.81	3.50
2.89	0.00	0.48	22.2	0.450E+01	0.83	3.50
2.97	0.00	0.49	22.5	0.445E+01	0.86	3.50
3.04	0.00	0.49	22.7	0.440E+01	0.88	3.50
3.12	0.00	0.50	23.0	0.435E+01	0.90	3.50
3.19	0.00	0.50	23.3	0.430E+01	0.92	3.50
3.26	0.00	0.51	23.5	0.425E+01	0.94	3.49
3.34	0.00	0.51	23.8	0.420E+01	0.96	3.49
3.41	0.00	0.52	24.0	0.416E+01	0.98	3.49
3.49	0.00	0.52	24.3	0.412E+01	1.01	3.49
3.56	0.00	0.53	24.5	0.408E+01	1.03	3.49
3.64	0.00	0.53	24.8	0.404E+01	1.05	3.49
3.71	0.00	0.53	25.0	0.400E+01	1.07	3.49

Cumulative travel time = 9.3532 sec
Plume centerline may exhibit slight discontinuities in transition
to subsequent far-field module.

END OF MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

BEGIN MOD251: DIFFUSER PLUME IN CO-FLOW

Phase 1: Vertically mixed, Phase 2: Re-stratified

Phase 1: The diffuser plume is VERTICALLY FULLY MIXED over the
entire layer depth.
This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

Phase 2: The flow has RESTRATIFIED at the beginning of this zone.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

END OF MOD251: DIFFUSER PLUME IN CO-FLOW

** End of NEAR-FIELD REGION (NFR) **

The initial plume WIDTH values in the next far-field module will be
CORRECTED by a factor 1.23 to conserve the mass flux in the far-field!

BEGIN MOD241: BUOYANT AMBIENT SPREADING

Discharge is non-buoyant or weakly buoyant.
Therefore BUOYANT SPREADING REGIME is ABSENT.

END OF MOD241: BUOYANT AMBIENT SPREADING

BEGIN MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Vertical diffusivity (initial value) = 0.812E-02 m^2/s
Horizontal diffusivity (initial value) = 0.102E-01 m^2/s

Profile definitions:
BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
= or equal to layer depth, if fully mixed
BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
measured horizontally in Y-direction
ZU = upper plume boundary (Z-coordinate)
ZL = lower plume boundary (Z-coordinate)
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 1 (not bank attached):

X	Y	Z	S	C	BV	BH	ZU	ZL
3.71	0.00	1.07	25.0	0.400E+01	1.07	4.29	1.07	0.00

Plume interacts with BOTTOM.
The passive diffusion plume becomes VERTICALLY FULLY MIXED within this prediction interval.

7.30	0.00	1.07	25.2	0.396E+01	1.07	4.33	1.07	0.00
10.88	0.00	1.07	25.5	0.393E+01	1.07	4.37	1.07	0.00
14.47	0.00	1.07	25.7	0.390E+01	1.07	4.40	1.07	0.00
18.05	0.00	1.07	25.9	0.386E+01	1.07	4.44	1.07	0.00
21.64	0.00	1.07	26.1	0.383E+01	1.07	4.48	1.07	0.00
25.22	0.00	1.07	26.3	0.380E+01	1.07	4.51	1.07	0.00
28.81	0.00	1.07	26.5	0.377E+01	1.07	4.55	1.07	0.00

** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume DOWNSTREAM distance meets or exceeds the regulatory value = 30.48 m.
This is the extent of the REGULATORY MIXING ZONE.

32.40	0.00	1.07	26.7	0.374E+01	1.07	4.58	1.07	0.00
35.98	0.00	1.07	26.9	0.372E+01	1.07	4.62	1.07	0.00
39.57	0.00	1.07	27.1	0.369E+01	1.07	4.65	1.07	0.00
43.15	0.00	1.07	27.3	0.366E+01	1.07	4.69	1.07	0.00
46.74	0.00	1.07	27.5	0.364E+01	1.07	4.72	1.07	0.00
50.33	0.00	1.07	27.7	0.361E+01	1.07	4.75	1.07	0.00
53.91	0.00	1.07	27.9	0.358E+01	1.07	4.79	1.07	0.00
57.50	0.00	1.07	28.1	0.356E+01	1.07	4.82	1.07	0.00
61.08	0.00	1.07	28.3	0.354E+01	1.07	4.85	1.07	0.00
64.67	0.00	1.07	28.5	0.351E+01	1.07	4.89	1.07	0.00
68.25	0.00	1.07	28.7	0.349E+01	1.07	4.92	1.07	0.00
71.84	0.00	1.07	28.9	0.347E+01	1.07	4.95	1.07	0.00
75.43	0.00	1.07	29.0	0.344E+01	1.07	4.98	1.07	0.00
79.01	0.00	1.07	29.2	0.342E+01	1.07	5.02	1.07	0.00
82.60	0.00	1.07	29.4	0.340E+01	1.07	5.05	1.07	0.00
86.18	0.00	1.07	29.6	0.338E+01	1.07	5.08	1.07	0.00
89.77	0.00	1.07	29.8	0.336E+01	1.07	5.11	1.07	0.00
93.35	0.00	1.07	30.0	0.334E+01	1.07	5.14	1.07	0.00
96.94	0.00	1.07	30.1	0.332E+01	1.07	5.17	1.07	0.00
100.53	0.00	1.07	30.3	0.330E+01	1.07	5.20	1.07	0.00
104.11	0.00	1.07	30.5	0.328E+01	1.07	5.24	1.07	0.00
107.70	0.00	1.07	30.7	0.326E+01	1.07	5.27	1.07	0.00
111.28	0.00	1.07	30.9	0.324E+01	1.07	5.30	1.07	0.00
114.87	0.00	1.07	31.0	0.322E+01	1.07	5.33	1.07	0.00

118.46	0.00	1.07	31.2	0.320E+01	1.07	5.36	1.07	0.00
122.04	0.00	1.07	31.4	0.319E+01	1.07	5.39	1.07	0.00
125.63	0.00	1.07	31.6	0.317E+01	1.07	5.42	1.07	0.00
129.21	0.00	1.07	31.7	0.315E+01	1.07	5.45	1.07	0.00
132.80	0.00	1.07	31.9	0.313E+01	1.07	5.47	1.07	0.00
136.38	0.00	1.07	32.1	0.312E+01	1.07	5.50	1.07	0.00
139.97	0.00	1.07	32.2	0.310E+01	1.07	5.53	1.07	0.00
143.56	0.00	1.07	32.4	0.309E+01	1.07	5.56	1.07	0.00
147.14	0.00	1.07	32.6	0.307E+01	1.07	5.59	1.07	0.00
150.73	0.00	1.07	32.7	0.305E+01	1.07	5.62	1.07	0.00
154.31	0.00	1.07	32.9	0.304E+01	1.07	5.65	1.07	0.00
157.90	0.00	1.07	33.1	0.302E+01	1.07	5.68	1.07	0.00
161.49	0.00	1.07	33.2	0.301E+01	1.07	5.70	1.07	0.00
165.07	0.00	1.07	33.4	0.299E+01	1.07	5.73	1.07	0.00
168.66	0.00	1.07	33.6	0.298E+01	1.07	5.76	1.07	0.00
172.24	0.00	1.07	33.7	0.296E+01	1.07	5.79	1.07	0.00
175.83	0.00	1.07	33.9	0.295E+01	1.07	5.82	1.07	0.00
179.41	0.00	1.07	34.0	0.294E+01	1.07	5.84	1.07	0.00
183.00	0.00	1.07	34.2	0.292E+01	1.07	5.87	1.07	0.00

Cumulative travel time = 511.9688 sec

Simulation limit based on maximum specified distance = 183.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

CORMIX2: Multiport Diffuser Discharges End of Prediction File

2222222222222222222222222222222222222222222222222222222222222222222222222222

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CORMIX2 PREDICTION FILE:
2222222222222222222222222222222222222222222222222222222222222222222222222222222222
                        CORMIX MIXING ZONE EXPERT SYSTEM
          Subsystem CORMIX2:  Multiport Diffuser Discharges
                              CORMIX Version 5.0GT
                          HYDRO2 Version 5.0.0.0 March 2007
------------------------------------------------------------------------------
------------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:    Lebanon
 Design case:        7Q10 River Flow, 11 Ports, effluent flow = 5 mgd
 FILE NAME:          U:\... 7Q10 3 mgd modeling\7Q10 Effluent Flow 5 mgd.prd
 Time stamp:         Mon Dec 17 09:35:09 2007

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS     =      18.29  AS     =      19.57  QA     =       6.85  ICHREG= 1
 HA     =       1.07  HD     =       1.07
 UA     =      0.350  F      =      0.094 USTAR =0.3793E-01
 UW     =      2.000 UWSTAR=0.2198E-02
 Uniform density environment
 STRCND=  U          RHOAM =  998.4062

DIFFUSER DISCHARGE PARAMETERS (metric units)
 Diffuser type:      DITYPE= unidirectional_perpendicular
 BANK   =  LEFT      DISTB =       9.10  YB1    =       5.40  YB2    =      12.80
 LD     =      7.42  NOPEN =   11        SPAC   =       0.74
 D0     =     0.128  A0     =      0.013 H0     =       0.31  SUB0   =       0.76
 Nozzle/port arrangement:   unidirectional_with_fanning
 GAMMA =      90.00  THETA =      10.00  SIGMA =       0.00  BETA   =      90.00
 U0     =      1.542  Q0     =       0.219        =0.2190E+00
 RHO0   =  997.8837  DRHO0 =0.5226E+00  GP0    =0.5133E-02
 C0     =0.1000E+03  CUNITS=  mg/l
 IPOLL =  1          KS     =0.0000E+00  KD     =0.0000E+00

FLUX VARIABLES - PER UNIT DIFFUSER LENGTH (metric units)
 q0     =0.2951E-01  m0     =0.4552E-01  j0     =0.1515E-03  SIGNJ0=       1.0
 Associated 2-d length scales (meters)
 lQ=B   =      0.019 lM     =      15.97  lm     =       0.37
 lmp    =  99999.00  lbp    =  99999.00  la     =  99999.00

FLUX VARIABLES - ENTIRE DIFFUSER (metric units)
 Q0     =0.2190E+00  M0     =0.3378E+00  J0     =0.1124E-02
 Associated 3-d length scales (meters)
 LQ     =      0.11  LM     =      13.21  Lm     =       1.66  Lb     =       0.03
                                          Lmp    =  99999.00  Lbp    =  99999.00

NON-DIMENSIONAL PARAMETERS
 FR0    =     155.62  FRD0   =      60.13  R      =       4.41  PL     =       2.
 (slot)               (port/nozzle)

RECOMPUTED SOURCE CONDITIONS FOR RISER GROUPS:
 Properties of riser group with  1 ports/nozzles each:
 U0     =       1.542 D0     =       0.128 A0     =       0.013 THETA =      10.00
 FR0    =     155.62  FRD0   =      60.13  R      =       4.41
 (slot)               (riser group)

FLOW CLASSIFICATION
 22222222222222222222222222222222222222
 2  Flow class (CORMIX2)       =    MU2    2
 2  Applicable layer depth HS =      1.07  2
 22222222222222222222222222222222222222
```

```
MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0    =0.1000E+03  CUNITS=  mg/l
 NTOX  =  0
 NSTD  =  0
 REGMZ =  1
 REGSPC=  1         XREG  =     30.48  WREG  =      0.00  AREG  =      0.00
 XINT  =    183.00  XMAX  =    183.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the bottom and the diffuser mid-point:
         9.10 m  from the LEFT  bank/shore.
    X-axis points downstream, Y-axis points to left, Z-axis points upward.
NSTEP =  50 display intervals per module
--------------------------------------------------------------------------------
--------------------------------------------------------------------------------
BEGIN MOD201: DIFFUSER DISCHARGE MODULE

 Due to complex near-field motions:  EQUIVALENT SLOT DIFFUSER (2-D) GEOMETRY

 Profile definitions:
   BV = Gaussian 1/e (37%) half-width, in vertical plane normal to trajectory
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic centerline dilution
   C  = centerline concentration (includes reaction effects, if any)

       X        Y        Z        S        C        BV       BH
    0.00     0.00     0.31     1.0 0.100E+03   0.02     3.71

END OF MOD201: DIFFUSER DISCHARGE MODULE
--------------------------------------------------------------------------------
--------------------------------------------------------------------------------
BEGIN MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

 Because of the FANNED-OUT HORIZONTAL ORIENTATION of the diffuser jets,
   the near-field dilution is slightly improved.

 In this laterally contracting zone the diffuser plume becomes VERTICALLY FULLY
  MIXED over the entire layer depth (HS =    1.07m).
   Full mixing is achieved after a plume distance of about five
   layer depths from the diffuser.

 Profile definitions:
   BV = layer depth (vertically mixed)
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

       X        Y        Z        S        C        BV       BH
    0.00     0.00     0.31     1.0 0.100E+03   0.02     3.71
    0.07     0.00     0.31     3.1 0.320E+02   0.03     3.69
    0.15     0.00     0.31     4.0 0.250E+02   0.04     3.66
    0.22     0.00     0.32     4.7 0.214E+02   0.06     3.64
    0.30     0.00     0.32     5.3 0.190E+02   0.09     3.62
    0.37     0.00     0.33     5.8 0.174E+02   0.11     3.61
    0.45     0.00     0.33     6.2 0.161E+02   0.13     3.59
    0.52     0.00     0.34     6.6 0.151E+02   0.15     3.57
    0.59     0.00     0.34     7.0 0.143E+02   0.17     3.56
    0.67     0.00     0.35     7.4 0.136E+02   0.19     3.54
    0.74     0.00     0.35     7.7 0.129E+02   0.21     3.53
    0.82     0.00     0.36     8.1 0.124E+02   0.24     3.52
    0.89     0.00     0.36     8.4 0.120E+02   0.26     3.50
    0.96     0.00     0.36     8.7 0.115E+02   0.28     3.49
    1.04     0.00     0.37     9.0 0.112E+02   0.30     3.48
```

1.11	0.00	0.37	9.2	0.108E+02	0.32	3.47
1.19	0.00	0.38	9.5	0.105E+02	0.34	3.46
1.26	0.00	0.38	9.8	0.102E+02	0.36	3.45
1.34	0.00	0.39	10.0	0.998E+01	0.39	3.44
1.41	0.00	0.39	10.3	0.974E+01	0.41	3.43
1.48	0.00	0.40	10.5	0.952E+01	0.43	3.42
1.56	0.00	0.40	10.7	0.931E+01	0.45	3.41
1.63	0.00	0.41	11.0	0.911E+01	0.47	3.41
1.71	0.00	0.41	11.2	0.893E+01	0.49	3.40
1.78	0.00	0.42	11.4	0.876E+01	0.51	3.39
1.86	0.00	0.42	11.6	0.860E+01	0.54	3.38
1.93	0.00	0.42	11.8	0.844E+01	0.56	3.38
2.00	0.00	0.43	12.0	0.830E+01	0.58	3.37
2.08	0.00	0.43	12.3	0.816E+01	0.60	3.37
2.15	0.00	0.44	12.5	0.803E+01	0.62	3.36
2.23	0.00	0.44	12.6	0.791E+01	0.64	3.35
2.30	0.00	0.45	12.8	0.779E+01	0.66	3.35
2.37	0.00	0.45	13.0	0.768E+01	0.68	3.35
2.45	0.00	0.46	13.2	0.757E+01	0.71	3.34
2.52	0.00	0.46	13.4	0.746E+01	0.73	3.34
2.60	0.00	0.47	13.6	0.736E+01	0.75	3.33
2.67	0.00	0.47	13.8	0.727E+01	0.77	3.33
2.75	0.00	0.48	13.9	0.718E+01	0.79	3.33
2.82	0.00	0.48	14.1	0.709E+01	0.81	3.33
2.89	0.00	0.48	14.3	0.700E+01	0.83	3.32
2.97	0.00	0.49	14.4	0.692E+01	0.86	3.32
3.04	0.00	0.49	14.6	0.684E+01	0.88	3.32
3.12	0.00	0.50	14.8	0.677E+01	0.90	3.32
3.19	0.00	0.50	14.9	0.669E+01	0.92	3.32
3.26	0.00	0.51	15.1	0.662E+01	0.94	3.31
3.34	0.00	0.51	15.3	0.655E+01	0.96	3.31
3.41	0.00	0.52	15.4	0.648E+01	0.98	3.31
3.49	0.00	0.52	15.6	0.642E+01	1.01	3.31
3.56	0.00	0.53	15.7	0.636E+01	1.03	3.31
3.64	0.00	0.53	15.9	0.630E+01	1.05	3.31
3.71	0.00	0.53	16.0	0.624E+01	1.07	3.31

Cumulative travel time = 8.3146 sec
Plume centerline may exhibit slight discontinuities in transition
to subsequent far-field module.

END OF MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

BEGIN MOD251: DIFFUSER PLUME IN CO-FLOW

Phase 1: Vertically mixed, Phase 2: Re-stratified

Phase 1: The diffuser plume is VERTICALLY FULLY MIXED over the
entire layer depth.
This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

Phase 2: The flow has RESTRATIFIED at the beginning of this zone.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

END OF MOD251: DIFFUSER PLUME IN CO-FLOW

** End of NEAR-FIELD REGION (NFR) **

The initial plume WIDTH values in the next far-field module will be
CORRECTED by a factor 1.37 to conserve the mass flux in the far-field!

BEGIN MOD241: BUOYANT AMBIENT SPREADING

Discharge is non-buoyant or weakly buoyant.
Therefore BUOYANT SPREADING REGIME is ABSENT.

END OF MOD241: BUOYANT AMBIENT SPREADING

BEGIN MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Vertical diffusivity (initial value) = 0.812E-02 m^2/s
Horizontal diffusivity (initial value) = 0.102E-01 m^2/s

Profile definitions:
BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
= or equal to layer depth, if fully mixed
BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
measured horizontally in Y-direction
ZU = upper plume boundary (Z-coordinate)
ZL = lower plume boundary (Z-coordinate)
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 1 (not bank attached):

X	Y	Z	S	C	BV	BH	ZU	ZL
3.71	0.00	1.07	16.0	0.624E+01	1.07	4.54	1.07	0.00

Plume interacts with BOTTOM.
The passive diffusion plume becomes VERTICALLY FULLY MIXED within this prediction interval.

7.30	0.00	1.07	16.2	0.619E+01	1.07	4.58	1.07	0.00
10.88	0.00	1.07	16.3	0.614E+01	1.07	4.61	1.07	0.00
14.47	0.00	1.07	16.4	0.610E+01	1.07	4.65	1.07	0.00
18.05	0.00	1.07	16.5	0.605E+01	1.07	4.68	1.07	0.00
21.64	0.00	1.07	16.6	0.601E+01	1.07	4.71	1.07	0.00
25.22	0.00	1.07	16.8	0.597E+01	1.07	4.75	1.07	0.00
28.81	0.00	1.07	16.9	0.593E+01	1.07	4.78	1.07	0.00

** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume DOWNSTREAM distance meets or exceeds the regulatory value = 30.48 m.
This is the extent of the REGULATORY MIXING ZONE.

32.40	0.00	1.07	17.0	0.589E+01	1.07	4.81	1.07	0.00
35.98	0.00	1.07	17.1	0.585E+01	1.07	4.85	1.07	0.00
39.57	0.00	1.07	17.2	0.581E+01	1.07	4.88	1.07	0.00
43.15	0.00	1.07	17.3	0.577E+01	1.07	4.91	1.07	0.00
46.74	0.00	1.07	17.4	0.573E+01	1.07	4.94	1.07	0.00
50.33	0.00	1.07	17.6	0.569E+01	1.07	4.98	1.07	0.00
53.91	0.00	1.07	17.7	0.566E+01	1.07	5.01	1.07	0.00
57.50	0.00	1.07	17.8	0.562E+01	1.07	5.04	1.07	0.00
61.08	0.00	1.07	17.9	0.559E+01	1.07	5.07	1.07	0.00
64.67	0.00	1.07	18.0	0.555E+01	1.07	5.10	1.07	0.00
68.25	0.00	1.07	18.1	0.552E+01	1.07	5.13	1.07	0.00
71.84	0.00	1.07	18.2	0.549E+01	1.07	5.16	1.07	0.00
75.43	0.00	1.07	18.3	0.546E+01	1.07	5.19	1.07	0.00
79.01	0.00	1.07	18.4	0.542E+01	1.07	5.22	1.07	0.00
82.60	0.00	1.07	18.5	0.539E+01	1.07	5.25	1.07	0.00
86.18	0.00	1.07	18.7	0.536E+01	1.07	5.28	1.07	0.00
89.77	0.00	1.07	18.8	0.533E+01	1.07	5.31	1.07	0.00
93.35	0.00	1.07	18.9	0.530E+01	1.07	5.34	1.07	0.00
96.94	0.00	1.07	19.0	0.527E+01	1.07	5.37	1.07	0.00
100.53	0.00	1.07	19.1	0.524E+01	1.07	5.40	1.07	0.00
104.11	0.00	1.07	19.2	0.522E+01	1.07	5.43	1.07	0.00
107.70	0.00	1.07	19.3	0.519E+01	1.07	5.46	1.07	0.00
111.28	0.00	1.07	19.4	0.516E+01	1.07	5.49	1.07	0.00
114.87	0.00	1.07	19.5	0.513E+01	1.07	5.52	1.07	0.00

118.46	0.00	1.07	19.6	0.511E+01	1.07	5.55	1.07	0.00
122.04	0.00	1.07	19.7	0.508E+01	1.07	5.58	1.07	0.00
125.63	0.00	1.07	19.8	0.506E+01	1.07	5.60	1.07	0.00
129.21	0.00	1.07	19.9	0.503E+01	1.07	5.63	1.07	0.00
132.80	0.00	1.07	20.0	0.501E+01	1.07	5.66	1.07	0.00
136.38	0.00	1.07	20.1	0.498E+01	1.07	5.69	1.07	0.00
139.97	0.00	1.07	20.2	0.496E+01	1.07	5.72	1.07	0.00
143.56	0.00	1.07	20.3	0.493E+01	1.07	5.74	1.07	0.00
147.14	0.00	1.07	20.4	0.491E+01	1.07	5.77	1.07	0.00
150.73	0.00	1.07	20.5	0.489E+01	1.07	5.80	1.07	0.00
154.31	0.00	1.07	20.6	0.486E+01	1.07	5.83	1.07	0.00
157.90	0.00	1.07	20.7	0.484E+01	1.07	5.85	1.07	0.00
161.49	0.00	1.07	20.8	0.482E+01	1.07	5.88	1.07	0.00
165.07	0.00	1.07	20.8	0.480E+01	1.07	5.91	1.07	0.00
168.66	0.00	1.07	20.9	0.478E+01	1.07	5.93	1.07	0.00
172.24	0.00	1.07	21.0	0.475E+01	1.07	5.96	1.07	0.00
175.83	0.00	1.07	21.1	0.473E+01	1.07	5.99	1.07	0.00
179.41	0.00	1.07	21.2	0.471E+01	1.07	6.01	1.07	0.00
183.00	0.00	1.07	21.3	0.469E+01	1.07	6.04	1.07	0.00

Cumulative travel time = 504.6732 sec

Simulation limit based on maximum specified distance = 183.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

CORMIX2: Multiport Diffuser Discharges End of Prediction File

22222222222222222222222222222222222222222222222222222222222222222222222222

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CORMIX2 PREDICTION FILE:
2222222222222222222222222222222222222222222222222222222222222222222222222222222
                      CORMIX MIXING ZONE EXPERT SYSTEM
         Subsystem CORMIX2:  Multiport Diffuser Discharges
                          CORMIX Version 5.0GT
                       HYDRO2 Version 5.0.0.0 March 2007
------------------------------------------------------------------------------
------------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:    Lebanon
 Design case:        30Q5 River Flow, 11 Ports, effluent flow = 3 mgd
 FILE NAME:          U:\...e 7Q10 3 mgd modeling\30Q5 Effluent Flow 3mgd.prd
 Time stamp:         Tue Dec 11 16:46:16 2007

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS    =      18.29  AS    =      20.85  QA    =       7.95  ICHREG= 1
 HA    =       1.14  HD    =       1.14
 UA    =      0.381  F     =      0.092 USTAR =0.4087E-01
 UW    =      2.000 UWSTAR=0.2198E-02
 Uniform density environment
 STRCND=  U          RHOAM =  998.4062

DIFFUSER DISCHARGE PARAMETERS (metric units)
 Diffuser type:      DITYPE= unidirectional_perpendicular
 BANK  =  LEFT       DISTB =       9.10  YB1   =       5.40  YB2   =      12.80
 LD    =       7.42  NOPEN =   11        SPAC  =       0.74
 D0    =      0.114  A0    =      0.010 H0    =       0.31  SUB0  =       0.83
 Nozzle/port arrangement:   unidirectional_with_fanning
 GAMMA =      90.00  THETA =      10.00  SIGMA =       0.00  BETA  =      90.00
 U0    =      1.167  Q0    =      0.131        =0.1310E+00
 RHO0  =  997.8837  DRHO0 =0.5226E+00  GP0   =0.5133E-02
 C0    =0.1000E+03  CUNITS=  mg/l
 IPOLL =  1         KS    =0.0000E+00  KD    =0.0000E+00

FLUX VARIABLES - PER UNIT DIFFUSER LENGTH (metric units)
 q0    =0.1765E-01  m0    =0.2060E-01  j0    =0.9062E-04  SIGNJ0=       1.0
 Associated 2-d length scales (meters)
 lQ=B  =      0.015 lM    =     10.19  lm    =      0.14
 lmp   =  99999.00  lbp   =  99999.00  la    =  99999.00

FLUX VARIABLES - ENTIRE DIFFUSER (metric units)
 Q0    =0.1310E+00  M0    =0.1528E+00  J0    =0.6724E-03
 Associated 3-d length scales (meters)
 LQ    =      0.10  LM    =      9.43  Lm    =      1.03  Lb    =      0.01
                                       Lmp   =  99999.00  Lbp   =  99999.00

NON-DIMENSIONAL PARAMETERS
 FR0   =    132.39  FRD0  =     48.23  R     =      3.06  PL    =       3.
 (slot)             (port/nozzle)

RECOMPUTED SOURCE CONDITIONS FOR RISER GROUPS:
 Properties of riser group with  1 ports/nozzles each:
 U0    =      1.167 D0    =      0.114 A0    =      0.010 THETA =      10.00
 FR0   =    132.39  FRD0  =     48.23  R     =      3.06
 (slot)             (riser group)

FLOW CLASSIFICATION
 22222222222222222222222222222222222222
 2  Flow class (CORMIX2)      =    MU2    2
 2  Applicable layer depth HS =     1.14  2
 22222222222222222222222222222222222222
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MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0    =0.1000E+03  CUNITS=  mg/l
 NTOX  =  0
 NSTD  =  0
 REGMZ =  1
 REGSPC=  1          XREG  =     30.48  WREG  =      0.00  AREG  =      0.00
 XINT  =    183.00  XMAX  =    183.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the bottom and the diffuser mid-point:
         9.10 m  from the LEFT  bank/shore.
    X-axis points downstream, Y-axis points to left, Z-axis points upward.
NSTEP =  50 display intervals per module
--------------------------------------------------------------------------------
--------------------------------------------------------------------------------
BEGIN MOD201: DIFFUSER DISCHARGE MODULE

 Due to complex near-field motions:  EQUIVALENT SLOT DIFFUSER (2-D) GEOMETRY

 Profile definitions:
   BV = Gaussian 1/e (37%) half-width, in vertical plane normal to trajectory
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic centerline dilution
   C  = centerline concentration (includes reaction effects, if any)

       X        Y       Z        S       C        BV       BH
    0.00     0.00    0.31     1.0 0.100E+03   0.01     3.71

END OF MOD201: DIFFUSER DISCHARGE MODULE
--------------------------------------------------------------------------------
--------------------------------------------------------------------------------
BEGIN MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

 Because of the FANNED-OUT HORIZONTAL ORIENTATION of the diffuser jets,
   the near-field dilution is slightly improved.

 In this laterally contracting zone the diffuser plume becomes VERTICALLY FULLY
  MIXED over the entire layer depth (HS =    1.14m).
   Full mixing is achieved after a plume distance of about five
   layer depths from the diffuser.

 Profile definitions:
   BV = layer depth (vertically mixed)
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

       X        Y       Z        S       C        BV       BH
    0.00     0.00    0.31     1.0 0.100E+03   0.01     3.71
    0.07     0.00    0.31     4.9 0.204E+02   0.03     3.70
    0.15     0.00    0.32     6.5 0.153E+02   0.05     3.69
    0.22     0.00    0.32     7.8 0.129E+02   0.07     3.68
    0.30     0.00    0.33     8.8 0.113E+02   0.09     3.67
    0.37     0.00    0.33     9.7 0.103E+02   0.11     3.66
    0.45     0.00    0.34    10.6 0.946E+01   0.14     3.66
    0.52     0.00    0.34    11.3 0.882E+01   0.16     3.65
    0.59     0.00    0.35    12.1 0.830E+01   0.18     3.64
    0.67     0.00    0.35    12.7 0.786E+01   0.21     3.63
    0.74     0.00    0.36    13.4 0.749E+01   0.23     3.63
    0.82     0.00    0.36    14.0 0.716E+01   0.25     3.62
    0.89     0.00    0.37    14.5 0.688E+01   0.27     3.62
    0.96     0.00    0.37    15.1 0.663E+01   0.30     3.61
    1.04     0.00    0.38    15.6 0.640E+01   0.32     3.61
```

1.11	0.00	0.38	16.1	0.620E+01	0.34	3.60
1.19	0.00	0.39	16.6	0.601E+01	0.36	3.60
1.26	0.00	0.40	17.1	0.584E+01	0.39	3.59
1.34	0.00	0.40	17.6	0.569E+01	0.41	3.59
1.41	0.00	0.41	18.0	0.554E+01	0.43	3.58
1.48	0.00	0.41	18.5	0.541E+01	0.46	3.58
1.56	0.00	0.42	18.9	0.529E+01	0.48	3.58
1.63	0.00	0.42	19.3	0.517E+01	0.50	3.57
1.71	0.00	0.43	19.7	0.506E+01	0.52	3.57
1.78	0.00	0.43	20.1	0.496E+01	0.55	3.57
1.86	0.00	0.44	20.5	0.487E+01	0.57	3.56
1.93	0.00	0.44	20.9	0.478E+01	0.59	3.56
2.00	0.00	0.45	21.3	0.469E+01	0.62	3.56
2.08	0.00	0.45	21.7	0.461E+01	0.64	3.56
2.15	0.00	0.46	22.0	0.454E+01	0.66	3.55
2.23	0.00	0.46	22.4	0.446E+01	0.68	3.55
2.30	0.00	0.47	22.8	0.439E+01	0.71	3.55
2.37	0.00	0.47	23.1	0.433E+01	0.73	3.55
2.45	0.00	0.48	23.5	0.426E+01	0.75	3.54
2.52	0.00	0.49	23.8	0.420E+01	0.78	3.54
2.60	0.00	0.49	24.1	0.415E+01	0.80	3.54
2.67	0.00	0.50	24.5	0.409E+01	0.82	3.54
2.75	0.00	0.50	24.8	0.404E+01	0.84	3.54
2.82	0.00	0.51	25.1	0.399E+01	0.87	3.54
2.89	0.00	0.51	25.4	0.394E+01	0.89	3.54
2.97	0.00	0.52	25.7	0.389E+01	0.91	3.54
3.04	0.00	0.52	26.0	0.384E+01	0.93	3.53
3.12	0.00	0.53	26.3	0.380E+01	0.96	3.53
3.19	0.00	0.53	26.6	0.376E+01	0.98	3.53
3.26	0.00	0.54	26.9	0.371E+01	1.00	3.53
3.34	0.00	0.54	27.2	0.367E+01	1.03	3.53
3.41	0.00	0.55	27.5	0.364E+01	1.05	3.53
3.49	0.00	0.55	27.8	0.360E+01	1.07	3.53
3.56	0.00	0.56	28.1	0.356E+01	1.09	3.53
3.64	0.00	0.56	28.4	0.353E+01	1.12	3.53
3.71	0.00	0.57	28.6	0.349E+01	1.14	3.53

Cumulative travel time = 8.7947 sec
Plume centerline may exhibit slight discontinuities in transition
to subsequent far-field module.

END OF MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

BEGIN MOD251: DIFFUSER PLUME IN CO-FLOW

Phase 1: Vertically mixed, Phase 2: Re-stratified

Phase 1: The diffuser plume is VERTICALLY FULLY MIXED over the
entire layer depth.
This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

Phase 2: The flow has RESTRATIFIED at the beginning of this zone.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

END OF MOD251: DIFFUSER PLUME IN CO-FLOW

** End of NEAR-FIELD REGION (NFR) **

The initial plume WIDTH values in the next far-field module will be
CORRECTED by a factor 1.20 to conserve the mass flux in the far-field!

BEGIN MOD241: BUOYANT AMBIENT SPREADING

Discharge is non-buoyant or weakly buoyant.
Therefore BUOYANT SPREADING REGIME is ABSENT.

END OF MOD241: BUOYANT AMBIENT SPREADING

BEGIN MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Vertical diffusivity (initial value) = 0.932E-02 m^2/s
Horizontal diffusivity (initial value) = 0.117E-01 m^2/s

Profile definitions:
BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
= or equal to layer depth, if fully mixed
BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
measured horizontally in Y-direction
ZU = upper plume boundary (Z-coordinate)
ZL = lower plume boundary (Z-coordinate)
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 1 (not bank attached):

X	Y	Z	S	C	BV	BH	ZU	ZL
3.71	0.00	1.14	28.6	0.349E+01	1.14	4.25	1.14	0.00

Plume interacts with BOTTOM.
The passive diffusion plume becomes VERTICALLY FULLY MIXED within this prediction interval.

7.30	0.00	1.14	28.9	0.346E+01	1.14	4.29	1.14	0.00
10.88	0.00	1.14	29.2	0.343E+01	1.14	4.33	1.14	0.00
14.47	0.00	1.14	29.4	0.340E+01	1.14	4.37	1.14	0.00
18.05	0.00	1.14	29.7	0.337E+01	1.14	4.40	1.14	0.00
21.64	0.00	1.14	30.0	0.334E+01	1.14	4.44	1.14	0.00
25.22	0.00	1.14	30.2	0.331E+01	1.14	4.48	1.14	0.00
28.81	0.00	1.14	30.5	0.328E+01	1.14	4.52	1.14	0.00

** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume DOWNSTREAM distance meets or exceeds the regulatory value = 30.48 m.
This is the extent of the REGULATORY MIXING ZONE.

32.40	0.00	1.14	30.7	0.326E+01	1.14	4.56	1.14	0.00
35.98	0.00	1.14	31.0	0.323E+01	1.14	4.59	1.14	0.00
39.57	0.00	1.14	31.2	0.320E+01	1.14	4.63	1.14	0.00
43.15	0.00	1.14	31.5	0.318E+01	1.14	4.67	1.14	0.00
46.74	0.00	1.14	31.7	0.315E+01	1.14	4.70	1.14	0.00
50.33	0.00	1.14	31.9	0.313E+01	1.14	4.74	1.14	0.00
53.91	0.00	1.14	32.2	0.311E+01	1.14	4.77	1.14	0.00
57.50	0.00	1.14	32.4	0.308E+01	1.14	4.81	1.14	0.00
61.08	0.00	1.14	32.7	0.306E+01	1.14	4.84	1.14	0.00
64.67	0.00	1.14	32.9	0.304E+01	1.14	4.88	1.14	0.00
68.25	0.00	1.14	33.1	0.302E+01	1.14	4.91	1.14	0.00
71.84	0.00	1.14	33.4	0.300E+01	1.14	4.95	1.14	0.00
75.43	0.00	1.14	33.6	0.298E+01	1.14	4.98	1.14	0.00
79.01	0.00	1.14	33.8	0.296E+01	1.14	5.02	1.14	0.00
82.60	0.00	1.14	34.0	0.294E+01	1.14	5.05	1.14	0.00
86.18	0.00	1.14	34.3	0.292E+01	1.14	5.08	1.14	0.00
89.77	0.00	1.14	34.5	0.290E+01	1.14	5.12	1.14	0.00
93.35	0.00	1.14	34.7	0.288E+01	1.14	5.15	1.14	0.00
96.94	0.00	1.14	34.9	0.286E+01	1.14	5.18	1.14	0.00
100.53	0.00	1.14	35.2	0.284E+01	1.14	5.21	1.14	0.00
104.11	0.00	1.14	35.4	0.283E+01	1.14	5.25	1.14	0.00
107.70	0.00	1.14	35.6	0.281E+01	1.14	5.28	1.14	0.00
111.28	0.00	1.14	35.8	0.279E+01	1.14	5.31	1.14	0.00
114.87	0.00	1.14	36.0	0.278E+01	1.14	5.34	1.14	0.00

118.46	0.00	1.14	36.2 0.276E+01	1.14	5.37	1.14	0.00
122.04	0.00	1.14	36.4 0.274E+01	1.14	5.41	1.14	0.00
125.63	0.00	1.14	36.7 0.273E+01	1.14	5.44	1.14	0.00
129.21	0.00	1.14	36.9 0.271E+01	1.14	5.47	1.14	0.00
132.80	0.00	1.14	37.1 0.270E+01	1.14	5.50	1.14	0.00
136.38	0.00	1.14	37.3 0.268E+01	1.14	5.53	1.14	0.00
139.97	0.00	1.14	37.5 0.267E+01	1.14	5.56	1.14	0.00
143.56	0.00	1.14	37.7 0.265E+01	1.14	5.59	1.14	0.00
147.14	0.00	1.14	37.9 0.264E+01	1.14	5.62	1.14	0.00
150.73	0.00	1.14	38.1 0.262E+01	1.14	5.65	1.14	0.00
154.31	0.00	1.14	38.3 0.261E+01	1.14	5.68	1.14	0.00
157.90	0.00	1.14	38.5 0.260E+01	1.14	5.71	1.14	0.00
161.49	0.00	1.14	38.7 0.258E+01	1.14	5.74	1.14	0.00
165.07	0.00	1.14	38.9 0.257E+01	1.14	5.77	1.14	0.00
168.66	0.00	1.14	39.1 0.256E+01	1.14	5.80	1.14	0.00
172.24	0.00	1.14	39.3 0.254E+01	1.14	5.83	1.14	0.00
175.83	0.00	1.14	39.5 0.253E+01	1.14	5.86	1.14	0.00
179.41	0.00	1.14	39.7 0.252E+01	1.14	5.89	1.14	0.00
183.00	0.00	1.14	39.9 0.251E+01	1.14	5.91	1.14	0.00

Cumulative travel time = 471.6186 sec

Simulation limit based on maximum specified distance = 183.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

CORMIX2: Multiport Diffuser Discharges End of Prediction File

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CORMIX2 PREDICTION FILE:
2222222222222222222222222222222222222222222222222222222222222222222222222222222222
                        CORMIX MIXING ZONE EXPERT SYSTEM
          Subsystem CORMIX2:  Multiport Diffuser Discharges
                              CORMIX Version 5.0GT
                           HYDRO2 Version 5.0.0.0 March 2007
------------------------------------------------------------------------------
------------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:    Lebanon
 Design case:        30Q5 River Flow, 11 Ports, effluent flow = 5 mgd
 FILE NAME:          U:\... 7Q10 3 mgd modeling\30Q5 Effluent Flow 5 mgd.prd
 Time stamp:         Mon Dec 17 09:44:45 2007

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS     =      18.29  AS     =      20.85  QA     =       7.95  ICHREG= 1
 HA     =       1.14  HD     =       1.14
 UA     =      0.381  F      =      0.092 USTAR =0.4087E-01
 UW     =      2.000 UWSTAR=0.2198E-02
 Uniform density environment
 STRCND=  U          RHOAM =  998.4062

DIFFUSER DISCHARGE PARAMETERS (metric units)
 Diffuser type:      DITYPE= unidirectional_perpendicular
 BANK   =  LEFT      DISTB =       9.10  YB1    =       5.40  YB2    =      12.80
 LD     =       7.42  NOPEN =   11        SPAC   =       0.74
 D0     =      0.128 A0     =      0.013 H0     =       0.31  SUB0   =       0.83
 Nozzle/port arrangement:   unidirectional_with_fanning
 GAMMA =      90.00  THETA =      10.00  SIGMA =       0.00  BETA   =      90.00
 U0     =      1.542 Q0     =      0.219        =0.2190E+00
 RHO0   =  997.8837  DRHO0 =0.5226E+00  GP0    =0.5133E-02
 C0     =0.1000E+03  CUNITS=  mg/l
 IPOLL =  1          KS     =0.0000E+00  KD     =0.0000E+00

FLUX VARIABLES - PER UNIT DIFFUSER LENGTH (metric units)
 q0     =0.2951E-01  m0     =0.4552E-01  j0     =0.1515E-03  SIGNJ0=       1.0
 Associated 2-d length scales (meters)
 lQ=B   =      0.019 lM     =      15.97  lm     =       0.31
 lmp    =  99999.00  lbp    =  99999.00  la     =   99999.00

FLUX VARIABLES - ENTIRE DIFFUSER (metric units)
 Q0     =0.2190E+00  M0     =0.3378E+00  J0     =0.1124E-02
 Associated 3-d length scales (meters)
 LQ     =      0.11  LM     =      13.21  Lm     =       1.53  Lb     =       0.02
                                          Lmp    =   99999.00  Lbp    =   99999.00

NON-DIMENSIONAL PARAMETERS
 FR0    =     155.62  FRD0  =      60.13  R      =       4.05  PL     =       2.
 (slot)               (port/nozzle)

RECOMPUTED SOURCE CONDITIONS FOR RISER GROUPS:
 Properties of riser group with  1 ports/nozzles each:
 U0     =       1.542 D0     =       0.128 A0     =       0.013 THETA =      10.00
 FR0    =     155.62  FRD0  =      60.13  R      =       4.05
 (slot)               (riser group)

FLOW CLASSIFICATION
 22222222222222222222222222222222222222
 2  Flow class (CORMIX2)       =    MU2    2
 2  Applicable layer depth HS =      1.14  2
 22222222222222222222222222222222222222
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MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0    =0.1000E+03  CUNITS=  mg/l
 NTOX  =  0
 NSTD  =  0
 REGMZ =  1
 REGSPC=  1          XREG  =     30.48  WREG  =      0.00  AREG  =      0.00
 XINT  =    183.00  XMAX  =    183.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the bottom and the diffuser mid-point:
         9.10 m  from the LEFT  bank/shore.
    X-axis points downstream, Y-axis points to left, Z-axis points upward.
NSTEP =  50 display intervals per module
--------------------------------------------------------------------------------
--------------------------------------------------------------------------------
BEGIN MOD201: DIFFUSER DISCHARGE MODULE

 Due to complex near-field motions:  EQUIVALENT SLOT DIFFUSER (2-D) GEOMETRY

 Profile definitions:
   BV = Gaussian 1/e (37%) half-width, in vertical plane normal to trajectory
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic centerline dilution
   C  = centerline concentration (includes reaction effects, if any)

      X        Y        Z        S        C        BV       BH
   0.00     0.00     0.31     1.0 0.100E+03   0.02     3.71

END OF MOD201: DIFFUSER DISCHARGE MODULE
--------------------------------------------------------------------------------
--------------------------------------------------------------------------------
BEGIN MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

 Because of the FANNED-OUT HORIZONTAL ORIENTATION of the diffuser jets,
   the near-field dilution is slightly improved.

 In this laterally contracting zone the diffuser plume becomes VERTICALLY FULLY
  MIXED over the entire layer depth (HS =    1.14m).
   Full mixing is achieved after a plume distance of about five
   layer depths from the diffuser.

 Profile definitions:
   BV = layer depth (vertically mixed)
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

      X        Y        Z        S        C        BV       BH
   0.00     0.00     0.31     1.0 0.100E+03   0.02     3.71
   0.07     0.00     0.31     3.4 0.292E+02   0.03     3.69
   0.15     0.00     0.32     4.4 0.226E+02   0.05     3.67
   0.22     0.00     0.32     5.2 0.192E+02   0.07     3.65
   0.30     0.00     0.33     5.9 0.171E+02   0.09     3.64
   0.37     0.00     0.33     6.4 0.156E+02   0.11     3.62
   0.45     0.00     0.34     6.9 0.144E+02   0.14     3.61
   0.52     0.00     0.34     7.4 0.135E+02   0.16     3.59
   0.59     0.00     0.35     7.9 0.127E+02   0.18     3.58
   0.67     0.00     0.35     8.3 0.121E+02   0.21     3.57
   0.74     0.00     0.36     8.7 0.115E+02   0.23     3.56
   0.82     0.00     0.36     9.0 0.111E+02   0.25     3.55
   0.89     0.00     0.37     9.4 0.106E+02   0.27     3.53
   0.96     0.00     0.37     9.7 0.103E+02   0.30     3.52
   1.04     0.00     0.38    10.1 0.993E+01   0.32     3.52
```

1.11	0.00	0.38	10.4	0.962E+01	0.34	3.51
1.19	0.00	0.39	10.7	0.935E+01	0.36	3.50
1.26	0.00	0.40	11.0	0.909E+01	0.39	3.49
1.34	0.00	0.40	11.3	0.886E+01	0.41	3.48
1.41	0.00	0.41	11.6	0.864E+01	0.43	3.47
1.48	0.00	0.41	11.8	0.844E+01	0.46	3.47
1.56	0.00	0.42	12.1	0.826E+01	0.48	3.46
1.63	0.00	0.42	12.4	0.808E+01	0.50	3.45
1.71	0.00	0.43	12.6	0.792E+01	0.52	3.45
1.78	0.00	0.43	12.9	0.776E+01	0.55	3.44
1.86	0.00	0.44	13.1	0.762E+01	0.57	3.43
1.93	0.00	0.44	13.4	0.748E+01	0.59	3.43
2.00	0.00	0.45	13.6	0.735E+01	0.62	3.42
2.08	0.00	0.45	13.8	0.723E+01	0.64	3.42
2.15	0.00	0.46	14.1	0.711E+01	0.66	3.41
2.23	0.00	0.46	14.3	0.700E+01	0.68	3.41
2.30	0.00	0.47	14.5	0.690E+01	0.71	3.41
2.37	0.00	0.47	14.7	0.679E+01	0.73	3.40
2.45	0.00	0.48	14.9	0.670E+01	0.75	3.40
2.52	0.00	0.49	15.1	0.660E+01	0.78	3.39
2.60	0.00	0.49	15.3	0.652E+01	0.80	3.39
2.67	0.00	0.50	15.6	0.643E+01	0.82	3.39
2.75	0.00	0.50	15.8	0.635E+01	0.84	3.39
2.82	0.00	0.51	15.9	0.627E+01	0.87	3.38
2.89	0.00	0.51	16.1	0.619E+01	0.89	3.38
2.97	0.00	0.52	16.3	0.612E+01	0.91	3.38
3.04	0.00	0.52	16.5	0.605E+01	0.93	3.38
3.12	0.00	0.53	16.7	0.598E+01	0.96	3.38
3.19	0.00	0.53	16.9	0.592E+01	0.98	3.38
3.26	0.00	0.54	17.1	0.585E+01	1.00	3.38
3.34	0.00	0.54	17.3	0.579E+01	1.03	3.37
3.41	0.00	0.55	17.4	0.573E+01	1.05	3.37
3.49	0.00	0.55	17.6	0.567E+01	1.07	3.37
3.56	0.00	0.56	17.8	0.562E+01	1.09	3.37
3.64	0.00	0.56	18.0	0.556E+01	1.12	3.37
3.71	0.00	0.57	18.1	0.551E+01	1.14	3.37

Cumulative travel time = 7.9598 sec
Plume centerline may exhibit slight discontinuities in transition to subsequent far-field module.

END OF MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

BEGIN MOD251: DIFFUSER PLUME IN CO-FLOW

Phase 1: Vertically mixed, Phase 2: Re-stratified

Phase 1: The diffuser plume is VERTICALLY FULLY MIXED over the entire layer depth.
This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

Phase 2: The flow has RESTRATIFIED at the beginning of this zone.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

END OF MOD251: DIFFUSER PLUME IN CO-FLOW

** End of NEAR-FIELD REGION (NFR) **

The initial plume WIDTH values in the next far-field module will be CORRECTED by a factor 1.32 to conserve the mass flux in the far-field!

BEGIN MOD241: BUOYANT AMBIENT SPREADING

Discharge is non-buoyant or weakly buoyant.
Therefore BUOYANT SPREADING REGIME is ABSENT.

END OF MOD241: BUOYANT AMBIENT SPREADING

BEGIN MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Vertical diffusivity (initial value) = 0.932E-02 m^2/s
Horizontal diffusivity (initial value) = 0.117E-01 m^2/s

Profile definitions:
BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
= or equal to layer depth, if fully mixed
BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
measured horizontally in Y-direction
ZU = upper plume boundary (Z-coordinate)
ZL = lower plume boundary (Z-coordinate)
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 1 (not bank attached):

X	Y	Z	S	C	BV	BH	ZU	ZL
3.71	0.00	1.14	18.1	0.551E+01	1.14	4.45	1.14	0.00

Plume interacts with BOTTOM.
The passive diffusion plume becomes VERTICALLY FULLY MIXED within this prediction interval.

7.30	0.00	1.14	18.3	0.546E+01	1.14	4.49	1.14	0.00
10.88	0.00	1.14	18.5	0.542E+01	1.14	4.53	1.14	0.00
14.47	0.00	1.14	18.6	0.538E+01	1.14	4.56	1.14	0.00
18.05	0.00	1.14	18.8	0.533E+01	1.14	4.60	1.14	0.00
21.64	0.00	1.14	18.9	0.529E+01	1.14	4.64	1.14	0.00
25.22	0.00	1.14	19.0	0.525E+01	1.14	4.67	1.14	0.00
28.81	0.00	1.14	19.2	0.521E+01	1.14	4.71	1.14	0.00

** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume DOWNSTREAM distance meets or exceeds the regulatory value = 30.48 m.
This is the extent of the REGULATORY MIXING ZONE.

32.40	0.00	1.14	19.3	0.517E+01	1.14	4.74	1.14	0.00
35.98	0.00	1.14	19.5	0.513E+01	1.14	4.78	1.14	0.00
39.57	0.00	1.14	19.6	0.510E+01	1.14	4.81	1.14	0.00
43.15	0.00	1.14	19.8	0.506E+01	1.14	4.85	1.14	0.00
46.74	0.00	1.14	19.9	0.502E+01	1.14	4.88	1.14	0.00
50.33	0.00	1.14	20.0	0.499E+01	1.14	4.92	1.14	0.00
53.91	0.00	1.14	20.2	0.495E+01	1.14	4.95	1.14	0.00
57.50	0.00	1.14	20.3	0.492E+01	1.14	4.98	1.14	0.00
61.08	0.00	1.14	20.5	0.489E+01	1.14	5.02	1.14	0.00
64.67	0.00	1.14	20.6	0.486E+01	1.14	5.05	1.14	0.00
68.25	0.00	1.14	20.7	0.482E+01	1.14	5.08	1.14	0.00
71.84	0.00	1.14	20.9	0.479E+01	1.14	5.12	1.14	0.00
75.43	0.00	1.14	21.0	0.476E+01	1.14	5.15	1.14	0.00
79.01	0.00	1.14	21.1	0.473E+01	1.14	5.18	1.14	0.00
82.60	0.00	1.14	21.3	0.470E+01	1.14	5.21	1.14	0.00
86.18	0.00	1.14	21.4	0.468E+01	1.14	5.25	1.14	0.00
89.77	0.00	1.14	21.5	0.465E+01	1.14	5.28	1.14	0.00
93.35	0.00	1.14	21.6	0.462E+01	1.14	5.31	1.14	0.00
96.94	0.00	1.14	21.8	0.459E+01	1.14	5.34	1.14	0.00
100.53	0.00	1.14	21.9	0.457E+01	1.14	5.37	1.14	0.00
104.11	0.00	1.14	22.0	0.454E+01	1.14	5.40	1.14	0.00
107.70	0.00	1.14	22.2	0.451E+01	1.14	5.43	1.14	0.00
111.28	0.00	1.14	22.3	0.449E+01	1.14	5.47	1.14	0.00
114.87	0.00	1.14	22.4	0.446E+01	1.14	5.50	1.14	0.00

118.46	0.00	1.14	22.5 0.444E+01	1.14	5.53	1.14	0.00
122.04	0.00	1.14	22.7 0.441E+01	1.14	5.56	1.14	0.00
125.63	0.00	1.14	22.8 0.439E+01	1.14	5.59	1.14	0.00
129.21	0.00	1.14	22.9 0.437E+01	1.14	5.62	1.14	0.00
132.80	0.00	1.14	23.0 0.434E+01	1.14	5.65	1.14	0.00
136.38	0.00	1.14	23.1 0.432E+01	1.14	5.68	1.14	0.00
139.97	0.00	1.14	23.3 0.430E+01	1.14	5.71	1.14	0.00
143.56	0.00	1.14	23.4 0.428E+01	1.14	5.73	1.14	0.00
147.14	0.00	1.14	23.5 0.426E+01	1.14	5.76	1.14	0.00
150.73	0.00	1.14	23.6 0.423E+01	1.14	5.79	1.14	0.00
154.31	0.00	1.14	23.7 0.421E+01	1.14	5.82	1.14	0.00
157.90	0.00	1.14	23.9 0.419E+01	1.14	5.85	1.14	0.00
161.49	0.00	1.14	24.0 0.417E+01	1.14	5.88	1.14	0.00
165.07	0.00	1.14	24.1 0.415E+01	1.14	5.91	1.14	0.00
168.66	0.00	1.14	24.2 0.413E+01	1.14	5.94	1.14	0.00
172.24	0.00	1.14	24.3 0.411E+01	1.14	5.96	1.14	0.00
175.83	0.00	1.14	24.4 0.409E+01	1.14	5.99	1.14	0.00
179.41	0.00	1.14	24.5 0.407E+01	1.14	6.02	1.14	0.00
183.00	0.00	1.14	24.7 0.406E+01	1.14	6.05	1.14	0.00

Cumulative travel time = 465.7956 sec

Simulation limit based on maximum specified distance = 183.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

CORMIX2: Multiport Diffuser Discharges End of Prediction File

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CORMIX2 PREDICTION FILE:
2222222222222222222222222222222222222222222222222222222222222222222222222222222
                      CORMIX MIXING ZONE EXPERT SYSTEM
         Subsystem CORMIX2:  Multiport Diffuser Discharges
                          CORMIX Version 5.0GT
                       HYDRO2 Version 5.0.0.0 March 2007
------------------------------------------------------------------------------
------------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:   Lebanon
 Design case:       Harmonic Mean 11 Ports, effluent flow = 3 mgd
 FILE NAME:         U:\...gd modeling\Harmonic Mean Effluent Flow 3 mgd.prd
 Time stamp:        Tue Dec 11 16:51:00 2007

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS    =     18.29  AS    =     31.28  QA    =     16.21  ICHREG= 1
 HA    =      1.71  HD    =      1.71
 UA    =     0.518  F     =     0.080 USTAR =0.5195E-01
 UW    =     2.000 UWSTAR=0.2198E-02
 Uniform density environment
 STRCND=  U         RHOAM =  998.4062

DIFFUSER DISCHARGE PARAMETERS (metric units)
 Diffuser type:     DITYPE= unidirectional_perpendicular
 BANK  =  LEFT      DISTB =     9.10  YB1   =     5.40  YB2   =     12.80
 LD    =     7.42  NOPEN =   11       SPAC  =     0.74
 D0    =    0.114  A0    =    0.010  H0    =     0.31  SUB0  =      1.40
 Nozzle/port arrangement:   unidirectional_with_fanning
 GAMMA =    90.00  THETA =    10.00  SIGMA =     0.00  BETA  =     90.00
 U0    =    1.167  Q0    =    0.131       =0.1310E+00
 RHO0  =  997.8837  DRHO0 =0.5226E+00  GP0   =0.5133E-02
 C0    =0.1000E+03  CUNITS=  mg/l
 IPOLL =  1         KS    =0.0000E+00  KD    =0.0000E+00

FLUX VARIABLES - PER UNIT DIFFUSER LENGTH (metric units)
 q0    =0.1765E-01  m0    =0.2060E-01  j0    =0.9062E-04  SIGNJ0=      1.0
 Associated 2-d length scales (meters)
 lQ=B  =     0.015  lM    =    10.19  lm    =     0.08
 lmp   =  99999.00  lbp   =  99999.00  la    =  99999.00

FLUX VARIABLES - ENTIRE DIFFUSER (metric units)
 Q0    =0.1310E+00  M0    =0.1528E+00  J0    =0.6724E-03
 Associated 3-d length scales (meters)
 LQ    =     0.10  LM    =     9.43  Lm    =     0.75  Lb    =      0.00
                                     Lmp   =  99999.00  Lbp   =  99999.00

NON-DIMENSIONAL PARAMETERS
 FR0   =    132.39  FRD0  =    48.23  R     =     2.25  PL    =      3.
 (slot)            (port/nozzle)

RECOMPUTED SOURCE CONDITIONS FOR RISER GROUPS:
 Properties of riser group with  1 ports/nozzles each:
 U0    =    1.167  D0    =    0.114  A0    =    0.010  THETA =     10.00
 FR0   =    132.39  FRD0  =    48.23  R     =     2.25
 (slot)            (riser group)

FLOW CLASSIFICATION
 22222222222222222222222222222222222222
 2  Flow class (CORMIX2)      =    MU2    2
 2  Applicable layer depth HS =     1.71  2
 22222222222222222222222222222222222222
```

MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0 =0.1000E+03 CUNITS= mg/l
 NTOX = 0
 NSTD = 0
 REGMZ = 1
 REGSPC= 1 XREG = 30.48 WREG = 0.00 AREG = 0.00
 XINT = 183.00 XMAX = 183.00

X-Y-Z COORDINATE SYSTEM:
 ORIGIN is located at the bottom and the diffuser mid-point:
 9.10 m from the LEFT bank/shore.
 X-axis points downstream, Y-axis points to left, Z-axis points upward.
NSTEP = 50 display intervals per module

BEGIN MOD201: DIFFUSER DISCHARGE MODULE

Due to complex near-field motions: EQUIVALENT SLOT DIFFUSER (2-D) GEOMETRY

Profile definitions:
 BV = Gaussian 1/e (37%) half-width, in vertical plane normal to trajectory
 BH = top-hat half-width, in horizontal plane normal to trajectory
 S = hydrodynamic centerline dilution
 C = centerline concentration (includes reaction effects, if any)

X	Y	Z	S	C	BV	BH
0.00	0.00	0.31	1.0	0.100E+03	0.01	3.71

END OF MOD201: DIFFUSER DISCHARGE MODULE

BEGIN MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

Because of the FANNED-OUT HORIZONTAL ORIENTATION of the diffuser jets,
 the near-field dilution is slightly improved.

In this laterally contracting zone the diffuser plume becomes VERTICALLY FULLY
 MIXED over the entire layer depth (HS = 1.71m).
 Full mixing is achieved after a plume distance of about five
 layer depths from the diffuser.

Profile definitions:
 BV = layer depth (vertically mixed)
 BH = top-hat half-width, in horizontal plane normal to trajectory
 S = hydrodynamic average (bulk) dilution
 C = average (bulk) concentration (includes reaction effects, if any)

X	Y	Z	S	C	BV	BH
0.00	0.00	0.31	1.0	0.100E+03	0.01	3.71
0.07	0.00	0.32	8.8	0.113E+02	0.03	3.71
0.15	0.00	0.33	12.1	0.828E+01	0.07	3.70
0.22	0.00	0.34	14.6	0.686E+01	0.10	3.70
0.30	0.00	0.35	16.7	0.600E+01	0.14	3.69
0.37	0.00	0.36	18.5	0.540E+01	0.17	3.69
0.45	0.00	0.37	20.2	0.495E+01	0.21	3.69
0.52	0.00	0.38	21.7	0.460E+01	0.24	3.69
0.59	0.00	0.39	23.2	0.432E+01	0.27	3.68
0.67	0.00	0.40	24.5	0.408E+01	0.31	3.68
0.74	0.00	0.42	25.8	0.388E+01	0.34	3.68
0.82	0.00	0.43	27.0	0.370E+01	0.38	3.68
0.89	0.00	0.44	28.1	0.355E+01	0.41	3.67
0.96	0.00	0.45	29.3	0.342E+01	0.44	3.67
1.04	0.00	0.46	30.3	0.330E+01	0.48	3.67

1.11	0.00	0.47	31.4	0.319E+01	0.51	3.67
1.19	0.00	0.48	32.3	0.309E+01	0.55	3.67
1.26	0.00	0.49	33.3	0.300E+01	0.58	3.66
1.34	0.00	0.50	34.2	0.292E+01	0.62	3.66
1.41	0.00	0.51	35.2	0.284E+01	0.65	3.66
1.48	0.00	0.53	36.0	0.277E+01	0.68	3.66
1.56	0.00	0.54	36.9	0.271E+01	0.72	3.66
1.63	0.00	0.55	37.8	0.265E+01	0.75	3.66
1.71	0.00	0.56	38.6	0.259E+01	0.79	3.65
1.78	0.00	0.57	39.4	0.254E+01	0.82	3.65
1.86	0.00	0.58	40.2	0.249E+01	0.86	3.65
1.93	0.00	0.59	41.0	0.244E+01	0.89	3.65
2.00	0.00	0.60	41.7	0.240E+01	0.92	3.65
2.08	0.00	0.61	42.5	0.235E+01	0.96	3.65
2.15	0.00	0.62	43.2	0.231E+01	0.99	3.65
2.23	0.00	0.64	43.9	0.228E+01	1.03	3.65
2.30	0.00	0.65	44.6	0.224E+01	1.06	3.65
2.37	0.00	0.66	45.3	0.221E+01	1.09	3.65
2.45	0.00	0.67	46.0	0.217E+01	1.13	3.64
2.52	0.00	0.68	46.7	0.214E+01	1.16	3.64
2.60	0.00	0.69	47.4	0.211E+01	1.20	3.64
2.67	0.00	0.70	48.0	0.208E+01	1.23	3.64
2.75	0.00	0.71	48.7	0.205E+01	1.27	3.64
2.82	0.00	0.72	49.3	0.203E+01	1.30	3.64
2.89	0.00	0.73	49.9	0.200E+01	1.33	3.64
2.97	0.00	0.74	50.6	0.198E+01	1.37	3.64
3.04	0.00	0.76	51.2	0.195E+01	1.40	3.64
3.12	0.00	0.77	51.8	0.193E+01	1.44	3.64
3.19	0.00	0.78	52.4	0.191E+01	1.47	3.64
3.26	0.00	0.79	53.0	0.189E+01	1.50	3.64
3.34	0.00	0.80	53.6	0.187E+01	1.54	3.64
3.41	0.00	0.81	54.2	0.185E+01	1.57	3.64
3.49	0.00	0.82	54.7	0.183E+01	1.61	3.64
3.56	0.00	0.83	55.3	0.181E+01	1.64	3.64
3.64	0.00	0.84	55.9	0.179E+01	1.68	3.64
3.71	0.00	0.85	56.4	0.177E+01	1.71	3.64

Cumulative travel time = 6.8833 sec
Plume centerline may exhibit slight discontinuities in transition
to subsequent far-field module.

END OF MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

BEGIN MOD251: DIFFUSER PLUME IN CO-FLOW

Phase 1: Vertically mixed, Phase 2: Re-stratified

Phase 1: The diffuser plume is VERTICALLY FULLY MIXED over the
entire layer depth.
This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

Phase 2: The flow has RESTRATIFIED at the beginning of this zone.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

END OF MOD251: DIFFUSER PLUME IN CO-FLOW

** End of NEAR-FIELD REGION (NFR) **

The initial plume WIDTH values in the next far-field module will be
CORRECTED by a factor 1.14 to conserve the mass flux in the far-field!

BEGIN MOD241: BUOYANT AMBIENT SPREADING

Discharge is non-buoyant or weakly buoyant.
Therefore BUOYANT SPREADING REGIME is ABSENT.

END OF MOD241: BUOYANT AMBIENT SPREADING

BEGIN MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Vertical diffusivity (initial value) = 0.178E-01 m^2/s
Horizontal diffusivity (initial value) = 0.222E-01 m^2/s

Profile definitions:
BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
= or equal to layer depth, if fully mixed
BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
measured horizontally in Y-direction
ZU = upper plume boundary (Z-coordinate)
ZL = lower plume boundary (Z-coordinate)
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 1 (not bank attached):

X	Y	Z	S	C	BV	BH	ZU	ZL
3.71	0.00	1.71	56.4	0.177E+01	1.71	4.14	1.71	0.00

Plume interacts with BOTTOM.
The passive diffusion plume becomes VERTICALLY FULLY MIXED within this prediction interval.

7.30	0.00	1.71	57.2	0.175E+01	1.71	4.19	1.71	0.00
10.88	0.00	1.71	58.0	0.172E+01	1.71	4.25	1.71	0.00
14.47	0.00	1.71	58.7	0.170E+01	1.71	4.31	1.71	0.00
18.05	0.00	1.71	59.5	0.168E+01	1.71	4.36	1.71	0.00
21.64	0.00	1.71	60.2	0.166E+01	1.71	4.42	1.71	0.00
25.22	0.00	1.71	61.0	0.164E+01	1.71	4.47	1.71	0.00
28.81	0.00	1.71	61.7	0.162E+01	1.71	4.52	1.71	0.00

** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume DOWNSTREAM distance meets or exceeds the regulatory value = 30.48 m.
This is the extent of the REGULATORY MIXING ZONE.

32.40	0.00	1.71	62.4	0.160E+01	1.71	4.58	1.71	0.00
35.98	0.00	1.71	63.1	0.158E+01	1.71	4.63	1.71	0.00
39.57	0.00	1.71	63.8	0.157E+01	1.71	4.68	1.71	0.00
43.15	0.00	1.71	64.5	0.155E+01	1.71	4.73	1.71	0.00
46.74	0.00	1.71	65.2	0.153E+01	1.71	4.78	1.71	0.00
50.33	0.00	1.71	65.9	0.152E+01	1.71	4.83	1.71	0.00
53.91	0.00	1.71	66.6	0.150E+01	1.71	4.88	1.71	0.00
57.50	0.00	1.71	67.2	0.149E+01	1.71	4.93	1.71	0.00
61.08	0.00	1.71	67.9	0.147E+01	1.71	4.98	1.71	0.00
64.67	0.00	1.71	68.5	0.146E+01	1.71	5.02	1.71	0.00
68.25	0.00	1.71	69.2	0.145E+01	1.71	5.07	1.71	0.00
71.84	0.00	1.71	69.8	0.143E+01	1.71	5.12	1.71	0.00
75.43	0.00	1.71	70.5	0.142E+01	1.71	5.17	1.71	0.00
79.01	0.00	1.71	71.1	0.141E+01	1.71	5.21	1.71	0.00
82.60	0.00	1.71	71.7	0.139E+01	1.71	5.26	1.71	0.00
86.18	0.00	1.71	72.3	0.138E+01	1.71	5.30	1.71	0.00
89.77	0.00	1.71	72.9	0.137E+01	1.71	5.35	1.71	0.00
93.35	0.00	1.71	73.6	0.136E+01	1.71	5.39	1.71	0.00
96.94	0.00	1.71	74.2	0.135E+01	1.71	5.44	1.71	0.00
100.53	0.00	1.71	74.8	0.134E+01	1.71	5.48	1.71	0.00
104.11	0.00	1.71	75.4	0.133E+01	1.71	5.52	1.71	0.00
107.70	0.00	1.71	75.9	0.132E+01	1.71	5.57	1.71	0.00
111.28	0.00	1.71	76.5	0.131E+01	1.71	5.61	1.71	0.00
114.87	0.00	1.71	77.1	0.130E+01	1.71	5.65	1.71	0.00

118.46	0.00	1.71	77.7	0.129E+01	1.71	5.70	1.71	0.00
122.04	0.00	1.71	78.3	0.128E+01	1.71	5.74	1.71	0.00
125.63	0.00	1.71	78.8	0.127E+01	1.71	5.78	1.71	0.00
129.21	0.00	1.71	79.4	0.126E+01	1.71	5.82	1.71	0.00
132.80	0.00	1.71	79.9	0.125E+01	1.71	5.86	1.71	0.00
136.38	0.00	1.71	80.5	0.124E+01	1.71	5.90	1.71	0.00
139.97	0.00	1.71	81.1	0.123E+01	1.71	5.94	1.71	0.00
143.56	0.00	1.71	81.6	0.123E+01	1.71	5.98	1.71	0.00
147.14	0.00	1.71	82.1	0.122E+01	1.71	6.02	1.71	0.00
150.73	0.00	1.71	82.7	0.121E+01	1.71	6.06	1.71	0.00
154.31	0.00	1.71	83.2	0.120E+01	1.71	6.10	1.71	0.00
157.90	0.00	1.71	83.8	0.119E+01	1.71	6.14	1.71	0.00
161.49	0.00	1.71	84.3	0.119E+01	1.71	6.18	1.71	0.00
165.07	0.00	1.71	84.8	0.118E+01	1.71	6.22	1.71	0.00
168.66	0.00	1.71	85.3	0.117E+01	1.71	6.26	1.71	0.00
172.24	0.00	1.71	85.9	0.116E+01	1.71	6.29	1.71	0.00
175.83	0.00	1.71	86.4	0.116E+01	1.71	6.33	1.71	0.00
179.41	0.00	1.71	86.9	0.115E+01	1.71	6.37	1.71	0.00
183.00	0.00	1.71	87.4	0.114E+01	1.71	6.41	1.71	0.00

Cumulative travel time = 350.0296 sec

Simulation limit based on maximum specified distance = 183.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

CORMIX2: Multiport Diffuser Discharges End of Prediction File

22222222222222222222222222222222222222222222222222222222222222222222222222

Harmonic Mean Effluent Flow 5 mgd.txt

```
CORMIX2 PREDICTION FILE:
2222222222222222222222222222222222222222222222222222222222222222222222222222
                      CORMIX MIXING ZONE EXPERT SYSTEM
         Subsystem CORMIX2:  Multiport Diffuser Discharges
                          CORMIX Version 5.0GT
                      HYDRO2 Version 5.0.0.0 March 2007
------------------------------------------------------------------------------
------------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:    Lebanon
 Design case:        Harmonic Mean 11 Ports, effluent flow = 5 mgd
 FILE NAME:          U:\...gd modeling\Harmonic Mean Effluent Flow 5 mgd.prd
 Time stamp:         Mon Dec 17 09:39:43 2007

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS     =      18.29  AS     =      31.28  QA     =      16.21  ICHREG= 1
 HA     =       1.71  HD     =       1.71
 UA     =      0.518  F      =      0.080 USTAR =0.5195E-01
 UW     =      2.000 UWSTAR=0.2198E-02
 Uniform density environment
 STRCND=  U          RHOAM =  998.4062

DIFFUSER DISCHARGE PARAMETERS (metric units)
 Diffuser type:      DITYPE= unidirectional_perpendicular
 BANK   =  LEFT      DISTB =       9.10  YB1    =       5.40  YB2    =      12.80
 LD     =      7.42  NOPEN =   11        SPAC   =       0.74
 D0     =     0.128  A0     =      0.013 H0     =       0.31  SUB0   =       1.40
 Nozzle/port arrangement:   unidirectional_with_fanning
 GAMMA  =     90.00  THETA  =      10.00 SIGMA  =       0.00  BETA   =      90.00
 U0     =     1.542  Q0     =      0.219        =0.2190E+00
 RHO0   =  997.8837  DRHO0 =0.5226E+00  GP0    =0.5133E-02
 C0     =0.1000E+03  CUNITS=  mg/l
 IPOLL  =  1         KS     =0.0000E+00  KD     =0.0000E+00

FLUX VARIABLES - PER UNIT DIFFUSER LENGTH (metric units)
 q0     =0.2951E-01  m0     =0.4552E-01  j0     =0.1515E-03  SIGNJ0=       1.0
 Associated 2-d length scales (meters)
 lQ=B   =      0.019 lM     =      15.97  lm     =      0.17
 lmp    =  99999.00  lbp    =  99999.00  la     =  99999.00

FLUX VARIABLES - ENTIRE DIFFUSER (metric units)
 Q0     =0.2190E+00  M0     =0.3378E+00  J0     =0.1124E-02
 Associated 3-d length scales (meters)
 LQ     =      0.11  LM     =      13.21  Lm     =      1.12  Lb     =       0.01
                                          Lmp    =  99999.00  Lbp    =   99999.00

NON-DIMENSIONAL PARAMETERS
 FR0    =    155.62  FRD0   =      60.13  R      =       2.98  PL     =       2.
 (slot)              (port/nozzle)

RECOMPUTED SOURCE CONDITIONS FOR RISER GROUPS:
 Properties of riser group with  1 ports/nozzles each:
 U0     =     1.542  D0     =      0.128  A0     =      0.013  THETA  =      10.00
 FR0    =    155.62  FRD0   =      60.13  R      =       2.98
 (slot)              (riser group)

FLOW CLASSIFICATION
 22222222222222222222222222222222222222
 2  Flow class (CORMIX2)       =    MU2    2
 2  Applicable layer depth HS  =     1.71  2
 22222222222222222222222222222222222222
```

```
MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0    =0.1000E+03  CUNITS=  mg/l
 NTOX  =  0
 NSTD  =  0
 REGMZ =  1
 REGSPC=  1          XREG  =     30.48  WREG  =      0.00  AREG  =      0.00
 XINT  =    183.00  XMAX  =    183.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the bottom and the diffuser mid-point:
         9.10 m  from the LEFT  bank/shore.
    X-axis points downstream, Y-axis points to left, Z-axis points upward.
NSTEP =  50 display intervals per module
-------------------------------------------------------------------------------
-------------------------------------------------------------------------------
BEGIN MOD201: DIFFUSER DISCHARGE MODULE

 Due to complex near-field motions:  EQUIVALENT SLOT DIFFUSER (2-D) GEOMETRY

 Profile definitions:
   BV = Gaussian 1/e (37%) half-width, in vertical plane normal to trajectory
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic centerline dilution
   C  = centerline concentration (includes reaction effects, if any)

       X        Y        Z        S        C        BV       BH
    0.00     0.00     0.31      1.0 0.100E+03   0.02     3.71

END OF MOD201: DIFFUSER DISCHARGE MODULE
-------------------------------------------------------------------------------
-------------------------------------------------------------------------------
BEGIN MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

 Because of the FANNED-OUT HORIZONTAL ORIENTATION of the diffuser jets,
   the near-field dilution is slightly improved.

 In this laterally contracting zone the diffuser plume becomes VERTICALLY FULLY
  MIXED over the entire layer depth (HS =    1.71m).
   Full mixing is achieved after a plume distance of about five
   layer depths from the diffuser.

 Profile definitions:
   BV = layer depth (vertically mixed)
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

       X        Y        Z        S        C        BV       BH
    0.00     0.00     0.31      1.0 0.100E+03   0.02     3.71
    0.07     0.00     0.32      5.7 0.174E+02   0.03     3.70
    0.15     0.00     0.33      7.7 0.130E+02   0.07     3.69
    0.22     0.00     0.34      9.2 0.108E+02   0.10     3.69
    0.30     0.00     0.35     10.5 0.953E+01   0.14     3.68
    0.37     0.00     0.36     11.6 0.861E+01   0.17     3.67
    0.45     0.00     0.37     12.6 0.792E+01   0.21     3.67
    0.52     0.00     0.38     13.6 0.737E+01   0.24     3.66
    0.59     0.00     0.39     14.4 0.693E+01   0.27     3.65
    0.67     0.00     0.40     15.2 0.656E+01   0.31     3.65
    0.74     0.00     0.42     16.0 0.624E+01   0.34     3.64
    0.82     0.00     0.43     16.7 0.597E+01   0.38     3.64
    0.89     0.00     0.44     17.4 0.573E+01   0.41     3.63
    0.96     0.00     0.45     18.1 0.552E+01   0.44     3.63
    1.04     0.00     0.46     18.8 0.533E+01   0.48     3.63
```

1.11	0.00	0.47	19.4	0.516E+01	0.51	3.62
1.19	0.00	0.48	20.0	0.500E+01	0.55	3.62
1.26	0.00	0.49	20.6	0.486E+01	0.58	3.61
1.34	0.00	0.50	21.1	0.473E+01	0.62	3.61
1.41	0.00	0.51	21.7	0.461E+01	0.65	3.61
1.48	0.00	0.53	22.2	0.450E+01	0.68	3.60
1.56	0.00	0.54	22.8	0.439E+01	0.72	3.60
1.63	0.00	0.55	23.3	0.430E+01	0.75	3.60
1.71	0.00	0.56	23.8	0.421E+01	0.79	3.60
1.78	0.00	0.57	24.3	0.412E+01	0.82	3.59
1.86	0.00	0.58	24.7	0.404E+01	0.86	3.59
1.93	0.00	0.59	25.2	0.397E+01	0.89	3.59
2.00	0.00	0.60	25.7	0.390E+01	0.92	3.59
2.08	0.00	0.61	26.1	0.383E+01	0.96	3.58
2.15	0.00	0.62	26.6	0.376E+01	0.99	3.58
2.23	0.00	0.64	27.0	0.370E+01	1.03	3.58
2.30	0.00	0.65	27.4	0.365E+01	1.06	3.58
2.37	0.00	0.66	27.9	0.359E+01	1.09	3.58
2.45	0.00	0.67	28.3	0.354E+01	1.13	3.57
2.52	0.00	0.68	28.7	0.349E+01	1.16	3.57
2.60	0.00	0.69	29.1	0.344E+01	1.20	3.57
2.67	0.00	0.70	29.5	0.339E+01	1.23	3.57
2.75	0.00	0.71	29.9	0.335E+01	1.27	3.57
2.82	0.00	0.72	30.3	0.330E+01	1.30	3.57
2.89	0.00	0.73	30.6	0.326E+01	1.33	3.57
2.97	0.00	0.74	31.0	0.322E+01	1.37	3.57
3.04	0.00	0.76	31.4	0.318E+01	1.40	3.57
3.12	0.00	0.77	31.8	0.315E+01	1.44	3.57
3.19	0.00	0.78	32.1	0.311E+01	1.47	3.57
3.26	0.00	0.79	32.5	0.308E+01	1.50	3.56
3.34	0.00	0.80	32.8	0.304E+01	1.54	3.56
3.41	0.00	0.81	33.2	0.301E+01	1.57	3.56
3.49	0.00	0.82	33.5	0.298E+01	1.61	3.56
3.56	0.00	0.83	33.9	0.295E+01	1.64	3.56
3.64	0.00	0.84	34.2	0.292E+01	1.68	3.56
3.71	0.00	0.85	34.6	0.289E+01	1.71	3.56

Cumulative travel time = 6.5907 sec
Plume centerline may exhibit slight discontinuities in transition
to subsequent far-field module.

END OF MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

BEGIN MOD251: DIFFUSER PLUME IN CO-FLOW

Phase 1: Vertically mixed, Phase 2: Re-stratified

Phase 1: The diffuser plume is VERTICALLY FULLY MIXED over the
entire layer depth.
This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

Phase 2: The flow has RESTRATIFIED at the beginning of this zone.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

END OF MOD251: DIFFUSER PLUME IN CO-FLOW

** End of NEAR-FIELD REGION (NFR) **

The initial plume WIDTH values in the next far-field module will be
CORRECTED by a factor 1.18 to conserve the mass flux in the far-field!

BEGIN MOD241: BUOYANT AMBIENT SPREADING

Discharge is non-buoyant or weakly buoyant.
Therefore BUOYANT SPREADING REGIME is ABSENT.

END OF MOD241: BUOYANT AMBIENT SPREADING

BEGIN MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Vertical diffusivity (initial value) = 0.178E-01 m^2/s
Horizontal diffusivity (initial value) = 0.222E-01 m^2/s

Profile definitions:
BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
= or equal to layer depth, if fully mixed
BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
measured horizontally in Y-direction
ZU = upper plume boundary (Z-coordinate)
ZL = lower plume boundary (Z-coordinate)
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 1 (not bank attached):

X	Y	Z	S	C	BV	BH	ZU	ZL
3.71	0.00	1.71	34.6	0.289E+01	1.71	4.21	1.71	0.00

Plume interacts with BOTTOM.
The passive diffusion plume becomes VERTICALLY FULLY MIXED within this prediction interval.

7.30	0.00	1.71	35.0	0.285E+01	1.71	4.27	1.71	0.00
10.88	0.00	1.71	35.5	0.282E+01	1.71	4.33	1.71	0.00
14.47	0.00	1.71	35.9	0.278E+01	1.71	4.38	1.71	0.00
18.05	0.00	1.71	36.4	0.275E+01	1.71	4.43	1.71	0.00
21.64	0.00	1.71	36.8	0.272E+01	1.71	4.49	1.71	0.00
25.22	0.00	1.71	37.3	0.268E+01	1.71	4.54	1.71	0.00
28.81	0.00	1.71	37.7	0.265E+01	1.71	4.59	1.71	0.00

** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume DOWNSTREAM distance meets or exceeds the regulatory value = 30.48 m.
This is the extent of the REGULATORY MIXING ZONE.

32.40	0.00	1.71	38.1	0.262E+01	1.71	4.64	1.71	0.00
35.98	0.00	1.71	38.5	0.260E+01	1.71	4.70	1.71	0.00
39.57	0.00	1.71	38.9	0.257E+01	1.71	4.75	1.71	0.00
43.15	0.00	1.71	39.3	0.254E+01	1.71	4.80	1.71	0.00
46.74	0.00	1.71	39.7	0.252E+01	1.71	4.85	1.71	0.00
50.33	0.00	1.71	40.1	0.249E+01	1.71	4.89	1.71	0.00
53.91	0.00	1.71	40.5	0.247E+01	1.71	4.94	1.71	0.00
57.50	0.00	1.71	40.9	0.244E+01	1.71	4.99	1.71	0.00
61.08	0.00	1.71	41.3	0.242E+01	1.71	5.04	1.71	0.00
64.67	0.00	1.71	41.7	0.240E+01	1.71	5.09	1.71	0.00
68.25	0.00	1.71	42.1	0.238E+01	1.71	5.13	1.71	0.00
71.84	0.00	1.71	42.5	0.235E+01	1.71	5.18	1.71	0.00
75.43	0.00	1.71	42.9	0.233E+01	1.71	5.22	1.71	0.00
79.01	0.00	1.71	43.2	0.231E+01	1.71	5.27	1.71	0.00
82.60	0.00	1.71	43.6	0.229E+01	1.71	5.31	1.71	0.00
86.18	0.00	1.71	44.0	0.227E+01	1.71	5.36	1.71	0.00
89.77	0.00	1.71	44.3	0.226E+01	1.71	5.40	1.71	0.00
93.35	0.00	1.71	44.7	0.224E+01	1.71	5.45	1.71	0.00
96.94	0.00	1.71	45.0	0.222E+01	1.71	5.49	1.71	0.00
100.53	0.00	1.71	45.4	0.220E+01	1.71	5.53	1.71	0.00
104.11	0.00	1.71	45.7	0.219E+01	1.71	5.58	1.71	0.00
107.70	0.00	1.71	46.1	0.217E+01	1.71	5.62	1.71	0.00
111.28	0.00	1.71	46.4	0.215E+01	1.71	5.66	1.71	0.00
114.87	0.00	1.71	46.8	0.214E+01	1.71	5.70	1.71	0.00

118.46	0.00	1.71	47.1	0.212E+01	1.71	5.75	1.71	0.00
122.04	0.00	1.71	47.5	0.211E+01	1.71	5.79	1.71	0.00
125.63	0.00	1.71	47.8	0.209E+01	1.71	5.83	1.71	0.00
129.21	0.00	1.71	48.1	0.208E+01	1.71	5.87	1.71	0.00
132.80	0.00	1.71	48.5	0.206E+01	1.71	5.91	1.71	0.00
136.38	0.00	1.71	48.8	0.205E+01	1.71	5.95	1.71	0.00
139.97	0.00	1.71	49.1	0.204E+01	1.71	5.99	1.71	0.00
143.56	0.00	1.71	49.5	0.202E+01	1.71	6.03	1.71	0.00
147.14	0.00	1.71	49.8	0.201E+01	1.71	6.07	1.71	0.00
150.73	0.00	1.71	50.1	0.200E+01	1.71	6.11	1.71	0.00
154.31	0.00	1.71	50.4	0.198E+01	1.71	6.15	1.71	0.00
157.90	0.00	1.71	50.7	0.197E+01	1.71	6.18	1.71	0.00
161.49	0.00	1.71	51.0	0.196E+01	1.71	6.22	1.71	0.00
165.07	0.00	1.71	51.4	0.195E+01	1.71	6.26	1.71	0.00
168.66	0.00	1.71	51.7	0.194E+01	1.71	6.30	1.71	0.00
172.24	0.00	1.71	52.0	0.192E+01	1.71	6.34	1.71	0.00
175.83	0.00	1.71	52.3	0.191E+01	1.71	6.37	1.71	0.00
179.41	0.00	1.71	52.6	0.190E+01	1.71	6.41	1.71	0.00
183.00	0.00	1.71	52.9	0.189E+01	1.71	6.45	1.71	0.00

Cumulative travel time = 347.8990 sec

Simulation limit based on maximum specified distance = 183.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

CORMIX2: Multiport Diffuser Discharges End of Prediction File

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CORMIX2 PREDICTION FILE:
2222222222222222222222222222222222222222222222222222222222222222222222222222
                      CORMIX MIXING ZONE EXPERT SYSTEM
         Subsystem CORMIX2:  Multiport Diffuser Discharges
                          CORMIX Version 5.0GT
                      HYDRO2 Version 5.0.0.0 March 2007
------------------------------------------------------------------------------
------------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:   Lebanon
 Design case:       Harmonic Mean 11 Ports, effluent flow = 7 mgd
 FILE NAME:         U:\...mgd modeling\Harmonic mean Effluent Flow 7mgd.prd
 Time stamp:        Mon Dec 17 09:49:03 2007

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS    =     18.29  AS    =     31.28  QA    =     16.21  ICHREG= 1
 HA    =      1.71  HD    =      1.71
 UA    =     0.518  F     =     0.080 USTAR =0.5195E-01
 UW    =     2.000 UWSTAR=0.2198E-02
 Uniform density environment
 STRCND=  U         RHOAM =  998.4062

DIFFUSER DISCHARGE PARAMETERS (metric units)
 Diffuser type:     DITYPE= unidirectional_perpendicular
 BANK  =  LEFT      DISTB =      9.10  YB1   =      5.40  YB2   =     12.80
 LD    =     7.42  NOPEN =   11       SPAC  =      0.74
 D0    =    0.134  A0    =     0.014  H0    =      0.31  SUB0  =      1.40
 Nozzle/port arrangement:   unidirectional_with_fanning
 GAMMA =     90.00  THETA =     10.00  SIGMA =      0.00  BETA  =     90.00
 U0    =     1.962  Q0    =     0.307        =0.3066E+00
 RHO0  =  997.8837  DRHO0 =0.5226E+00  GP0   =0.5133E-02
 C0    =0.1000E+03  CUNITS=  mg/l
 IPOLL =  1         KS    =0.0000E+00  KD    =0.0000E+00

FLUX VARIABLES - PER UNIT DIFFUSER LENGTH (metric units)
 q0    =0.4132E-01  m0    =0.8106E-01  j0    =0.2121E-03  SIGNJ0=      1.0
 Associated 2-d length scales (meters)
 lQ=B  =     0.021  lM    =     22.72  lm    =      0.30
 lmp   =  99999.00  lbp   =  99999.00  la    =  99999.00

FLUX VARIABLES - ENTIRE DIFFUSER (metric units)
 Q0    =0.3066E+00  M0    =0.6015E+00  J0    =0.1574E-02
 Associated 3-d length scales (meters)
 LQ    =     0.12  LM    =     17.22  Lm    =      1.50  Lb    =      0.01
                                      Lmp   =  99999.00  Lbp   =  99999.00

NON-DIMENSIONAL PARAMETERS
 FR0   =    188.67  FRD0  =     74.66  R     =      3.78  PL    =      2.
 (slot)             (port/nozzle)

RECOMPUTED SOURCE CONDITIONS FOR RISER GROUPS:
 Properties of riser group with  1 ports/nozzles each:
 U0    =     1.962 D0    =     0.134 A0    =     0.014 THETA =     10.00
 FR0   =    188.67  FRD0  =     74.66  R     =      3.78
 (slot)             (riser group)

FLOW CLASSIFICATION
 22222222222222222222222222222222222222
 2  Flow class (CORMIX2)      =    MU2    2
 2  Applicable layer depth HS =     1.71  2
 22222222222222222222222222222222222222
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MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0    =0.1000E+03  CUNITS=  mg/l
 NTOX  =  0
 NSTD  =  0
 REGMZ =  1
 REGSPC=  1          XREG  =     30.48  WREG  =      0.00  AREG  =      0.00
 XINT  =    183.00  XMAX  =    183.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the bottom and the diffuser mid-point:
         9.10 m  from the LEFT  bank/shore.
    X-axis points downstream, Y-axis points to left, Z-axis points upward.
NSTEP =  50 display intervals per module
--------------------------------------------------------------------------------
--------------------------------------------------------------------------------
BEGIN MOD201: DIFFUSER DISCHARGE MODULE

 Due to complex near-field motions:  EQUIVALENT SLOT DIFFUSER (2-D) GEOMETRY

 Profile definitions:
   BV = Gaussian 1/e (37%) half-width, in vertical plane normal to trajectory
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic centerline dilution
   C  = centerline concentration (includes reaction effects, if any)

       X        Y        Z        S        C        BV       BH
    0.00     0.00     0.31     1.0 0.100E+03   0.02     3.71

END OF MOD201: DIFFUSER DISCHARGE MODULE
--------------------------------------------------------------------------------
--------------------------------------------------------------------------------
BEGIN MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

 Because of the FANNED-OUT HORIZONTAL ORIENTATION of the diffuser jets,
   the near-field dilution is slightly improved.

 In this laterally contracting zone the diffuser plume becomes VERTICALLY FULLY
  MIXED over the entire layer depth (HS =    1.71m).
   Full mixing is achieved after a plume distance of about five
   layer depths from the diffuser.

 Profile definitions:
   BV = layer depth (vertically mixed)
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

       X        Y        Z        S        C        BV       BH
    0.00     0.00     0.31     1.0 0.100E+03   0.02     3.71
    0.07     0.00     0.32     4.5 0.224E+02   0.03     3.70
    0.15     0.00     0.33     5.9 0.170E+02   0.07     3.68
    0.22     0.00     0.34     7.0 0.143E+02   0.10     3.67
    0.30     0.00     0.35     7.9 0.126E+02   0.14     3.66
    0.37     0.00     0.36     8.7 0.114E+02   0.17     3.65
    0.45     0.00     0.37     9.5 0.105E+02   0.21     3.64
    0.52     0.00     0.38    10.2 0.984E+01   0.24     3.63
    0.59     0.00     0.39    10.8 0.926E+01   0.27     3.62
    0.67     0.00     0.40    11.4 0.878E+01   0.31     3.61
    0.74     0.00     0.42    12.0 0.837E+01   0.34     3.60
    0.82     0.00     0.43    12.5 0.801E+01   0.38     3.59
    0.89     0.00     0.44    13.0 0.769E+01   0.41     3.59
    0.96     0.00     0.45    13.5 0.741E+01   0.44     3.58
    1.04     0.00     0.46    14.0 0.716E+01   0.48     3.57
```

1.11	0.00	0.47	14.4	0.694E+01	0.51	3.57
1.19	0.00	0.48	14.9	0.673E+01	0.55	3.56
1.26	0.00	0.49	15.3	0.654E+01	0.58	3.55
1.34	0.00	0.50	15.7	0.637E+01	0.62	3.55
1.41	0.00	0.51	16.1	0.621E+01	0.65	3.54
1.48	0.00	0.53	16.5	0.606E+01	0.68	3.54
1.56	0.00	0.54	16.9	0.593E+01	0.72	3.53
1.63	0.00	0.55	17.2	0.580E+01	0.75	3.53
1.71	0.00	0.56	17.6	0.568E+01	0.79	3.52
1.78	0.00	0.57	18.0	0.557E+01	0.82	3.52
1.86	0.00	0.58	18.3	0.546E+01	0.86	3.51
1.93	0.00	0.59	18.7	0.536E+01	0.89	3.51
2.00	0.00	0.60	19.0	0.526E+01	0.92	3.51
2.08	0.00	0.61	19.3	0.517E+01	0.96	3.50
2.15	0.00	0.62	19.7	0.509E+01	0.99	3.50
2.23	0.00	0.64	20.0	0.501E+01	1.03	3.50
2.30	0.00	0.65	20.3	0.493E+01	1.06	3.49
2.37	0.00	0.66	20.6	0.486E+01	1.09	3.49
2.45	0.00	0.67	20.9	0.479E+01	1.13	3.49
2.52	0.00	0.68	21.2	0.472E+01	1.16	3.49
2.60	0.00	0.69	21.5	0.465E+01	1.20	3.48
2.67	0.00	0.70	21.8	0.459E+01	1.23	3.48
2.75	0.00	0.71	22.1	0.453E+01	1.27	3.48
2.82	0.00	0.72	22.4	0.447E+01	1.30	3.48
2.89	0.00	0.73	22.6	0.442E+01	1.33	3.48
2.97	0.00	0.74	22.9	0.437E+01	1.37	3.48
3.04	0.00	0.76	23.2	0.431E+01	1.40	3.48
3.12	0.00	0.77	23.4	0.426E+01	1.44	3.47
3.19	0.00	0.78	23.7	0.422E+01	1.47	3.47
3.26	0.00	0.79	24.0	0.417E+01	1.50	3.47
3.34	0.00	0.80	24.2	0.413E+01	1.54	3.47
3.41	0.00	0.81	24.5	0.408E+01	1.57	3.47
3.49	0.00	0.82	24.7	0.404E+01	1.61	3.47
3.56	0.00	0.83	25.0	0.400E+01	1.64	3.47
3.64	0.00	0.84	25.2	0.396E+01	1.68	3.47
3.71	0.00	0.85	25.5	0.392E+01	1.71	3.47

Cumulative travel time = 6.2326 sec
Plume centerline may exhibit slight discontinuities in transition
to subsequent far-field module.

END OF MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

BEGIN MOD251: DIFFUSER PLUME IN CO-FLOW

Phase 1: Vertically mixed, Phase 2: Re-stratified

Phase 1: The diffuser plume is VERTICALLY FULLY MIXED over the
entire layer depth.
This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

Phase 2: The flow has RESTRATIFIED at the beginning of this zone.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

END OF MOD251: DIFFUSER PLUME IN CO-FLOW

** End of NEAR-FIELD REGION (NFR) **

The initial plume WIDTH values in the next far-field module will be
CORRECTED by a factor 1.25 to conserve the mass flux in the far-field!

BEGIN MOD241: BUOYANT AMBIENT SPREADING

Discharge is non-buoyant or weakly buoyant.
Therefore BUOYANT SPREADING REGIME is ABSENT.

END OF MOD241: BUOYANT AMBIENT SPREADING

BEGIN MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Vertical diffusivity (initial value) = 0.178E-01 m^2/s
Horizontal diffusivity (initial value) = 0.222E-01 m^2/s

Profile definitions:
BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
= or equal to layer depth, if fully mixed
BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
measured horizontally in Y-direction
ZU = upper plume boundary (Z-coordinate)
ZL = lower plume boundary (Z-coordinate)
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 1 (not bank attached):

X	Y	Z	S	C	BV	BH	ZU	ZL
3.71	0.00	1.71	25.5	0.392E+01	1.71	4.33	1.71	0.00

Plume interacts with BOTTOM.
The passive diffusion plume becomes VERTICALLY FULLY MIXED within this prediction interval.

7.30	0.00	1.71	25.8	0.387E+01	1.71	4.38	1.71	0.00
10.88	0.00	1.71	26.1	0.383E+01	1.71	4.44	1.71	0.00
14.47	0.00	1.71	26.4	0.378E+01	1.71	4.49	1.71	0.00
18.05	0.00	1.71	26.8	0.374E+01	1.71	4.54	1.71	0.00
21.64	0.00	1.71	27.1	0.370E+01	1.71	4.59	1.71	0.00
25.22	0.00	1.71	27.4	0.366E+01	1.71	4.64	1.71	0.00
28.81	0.00	1.71	27.7	0.362E+01	1.71	4.70	1.71	0.00

** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume DOWNSTREAM distance meets or exceeds the regulatory value = 30.48 m.
This is the extent of the REGULATORY MIXING ZONE.

32.40	0.00	1.71	28.0	0.358E+01	1.71	4.75	1.71	0.00
35.98	0.00	1.71	28.2	0.354E+01	1.71	4.79	1.71	0.00
39.57	0.00	1.71	28.5	0.350E+01	1.71	4.84	1.71	0.00
43.15	0.00	1.71	28.8	0.347E+01	1.71	4.89	1.71	0.00
46.74	0.00	1.71	29.1	0.344E+01	1.71	4.94	1.71	0.00
50.33	0.00	1.71	29.4	0.340E+01	1.71	4.99	1.71	0.00
53.91	0.00	1.71	29.7	0.337E+01	1.71	5.04	1.71	0.00
57.50	0.00	1.71	29.9	0.334E+01	1.71	5.08	1.71	0.00
61.08	0.00	1.71	30.2	0.331E+01	1.71	5.13	1.71	0.00
64.67	0.00	1.71	30.5	0.328E+01	1.71	5.18	1.71	0.00
68.25	0.00	1.71	30.8	0.325E+01	1.71	5.22	1.71	0.00
71.84	0.00	1.71	31.0	0.322E+01	1.71	5.27	1.71	0.00
75.43	0.00	1.71	31.3	0.320E+01	1.71	5.31	1.71	0.00
79.01	0.00	1.71	31.5	0.317E+01	1.71	5.36	1.71	0.00
82.60	0.00	1.71	31.8	0.314E+01	1.71	5.40	1.71	0.00
86.18	0.00	1.71	32.1	0.312E+01	1.71	5.44	1.71	0.00
89.77	0.00	1.71	32.3	0.309E+01	1.71	5.49	1.71	0.00
93.35	0.00	1.71	32.6	0.307E+01	1.71	5.53	1.71	0.00
96.94	0.00	1.71	32.8	0.305E+01	1.71	5.57	1.71	0.00
100.53	0.00	1.71	33.1	0.302E+01	1.71	5.61	1.71	0.00
104.11	0.00	1.71	33.3	0.300E+01	1.71	5.66	1.71	0.00
107.70	0.00	1.71	33.6	0.298E+01	1.71	5.70	1.71	0.00
111.28	0.00	1.71	33.8	0.296E+01	1.71	5.74	1.71	0.00
114.87	0.00	1.71	34.1	0.294E+01	1.71	5.78	1.71	0.00

118.46	0.00	1.71	34.3	0.292E+01	1.71	5.82	1.71	0.00
122.04	0.00	1.71	34.5	0.290E+01	1.71	5.86	1.71	0.00
125.63	0.00	1.71	34.8	0.288E+01	1.71	5.90	1.71	0.00
129.21	0.00	1.71	35.0	0.286E+01	1.71	5.94	1.71	0.00
132.80	0.00	1.71	35.2	0.284E+01	1.71	5.98	1.71	0.00
136.38	0.00	1.71	35.5	0.282E+01	1.71	6.02	1.71	0.00
139.97	0.00	1.71	35.7	0.280E+01	1.71	6.06	1.71	0.00
143.56	0.00	1.71	35.9	0.278E+01	1.71	6.10	1.71	0.00
147.14	0.00	1.71	36.2	0.277E+01	1.71	6.14	1.71	0.00
150.73	0.00	1.71	36.4	0.275E+01	1.71	6.18	1.71	0.00
154.31	0.00	1.71	36.6	0.273E+01	1.71	6.22	1.71	0.00
157.90	0.00	1.71	36.8	0.271E+01	1.71	6.25	1.71	0.00
161.49	0.00	1.71	37.1	0.270E+01	1.71	6.29	1.71	0.00
165.07	0.00	1.71	37.3	0.268E+01	1.71	6.33	1.71	0.00
168.66	0.00	1.71	37.5	0.267E+01	1.71	6.37	1.71	0.00
172.24	0.00	1.71	37.7	0.265E+01	1.71	6.40	1.71	0.00
175.83	0.00	1.71	37.9	0.264E+01	1.71	6.44	1.71	0.00
179.41	0.00	1.71	38.2	0.262E+01	1.71	6.48	1.71	0.00
183.00	0.00	1.71	38.4	0.261E+01	1.71	6.51	1.71	0.00

Cumulative travel time = 345.7307 sec

Simulation limit based on maximum specified distance = 183.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

CORMIX2: Multiport Diffuser Discharges End of Prediction File

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CORMIX2 PREDICTION FILE:
2222222222222222222222222222222222222222222222222222222222222222222222222222
                      CORMIX MIXING ZONE EXPERT SYSTEM
          Subsystem CORMIX2:  Multiport Diffuser Discharges
                          CORMIX Version 5.0GT
                       HYDRO2 Version 5.0.0.0 March 2007
------------------------------------------------------------------------------
------------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:    Lebanon
 Design case:        Harmonic Mean 11 Ports, effluent flow = 12 mgd
 FILE NAME:          U:\...d modeling\Harmonic mean Effluent Flow 12 mgd.prd
 Time stamp:         Mon Dec 17 09:52:26 2007

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS    =     18.29  AS    =     31.28  QA    =     16.21  ICHREG= 1
 HA    =      1.71  HD    =      1.71
 UA    =     0.518  F     =     0.080 USTAR =0.5195E-01
 UW    =     2.000 UWSTAR=0.2198E-02
 Uniform density environment
 STRCND=  U          RHOAM =  998.4062

DIFFUSER DISCHARGE PARAMETERS (metric units)
 Diffuser type:      DITYPE= unidirectional_perpendicular
 BANK  =  LEFT       DISTB =      9.10  YB1   =      5.40  YB2   =     12.80
 LD    =     7.42   NOPEN =   11        SPAC  =      0.74
 D0    =    0.154   A0    =     0.019  H0    =      0.31  SUB0  =      1.40
 Nozzle/port arrangement:   unidirectional_with_fanning
 GAMMA =     90.00  THETA =     10.00  SIGMA =      0.00  BETA  =     90.00
 U0    =     2.577  Q0    =     0.528        =0.5280E+00
 RHO0  =  997.8837  DRHO0 =0.5226E+00  GP0   =0.5133E-02
 C0    =0.1000E+03  CUNITS=  mg/l
 IPOLL =  1         KS    =0.0000E+00  KD    =0.0000E+00

FLUX VARIABLES - PER UNIT DIFFUSER LENGTH (metric units)
 q0    =0.7116E-01  m0    =0.1834E+00  j0    =0.3653E-03  SIGNJ0=      1.0
 Associated 2-d length scales (meters)
 lQ=B  =     0.028  lM    =     35.76  lm    =      0.68
 lmp   =  99999.00  lbp   =  99999.00  la    =  99999.00

FLUX VARIABLES - ENTIRE DIFFUSER (metric units)
 Q0    =0.5280E+00  M0    =0.1361E+01  J0    =0.2710E-02
 Associated 3-d length scales (meters)
 LQ    =     0.14   LM    =     24.20  Lm    =      2.25  Lb    =      0.02
                                       Lmp   =  99999.00  Lbp   =  99999.00

NON-DIMENSIONAL PARAMETERS
 FR0   =    216.45  FRD0  =     91.66  R     =      4.97  PL    =      1.
 (slot)             (port/nozzle)

RECOMPUTED SOURCE CONDITIONS FOR RISER GROUPS:
 Properties of riser group with  1 ports/nozzles each:
 U0    =     2.577  D0    =     0.154  A0    =     0.019  THETA =     10.00
 FR0   =    216.45  FRD0  =     91.66  R     =      4.97
 (slot)             (riser group)

FLOW CLASSIFICATION
 22222222222222222222222222222222222222
 2  Flow class (CORMIX2)      =    MU2    2
 2  Applicable layer depth HS =     1.71  2
 22222222222222222222222222222222222222
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MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0    =0.1000E+03  CUNITS=  mg/l
 NTOX  =  0
 NSTD  =  0
 REGMZ =  1
 REGSPC=  1          XREG  =     30.48  WREG  =      0.00  AREG  =      0.00
 XINT  =    183.00  XMAX  =    183.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the bottom and the diffuser mid-point:
         9.10 m  from the LEFT  bank/shore.
    X-axis points downstream, Y-axis points to left, Z-axis points upward.
NSTEP =  50 display intervals per module
-------------------------------------------------------------------------------
-------------------------------------------------------------------------------
BEGIN MOD201: DIFFUSER DISCHARGE MODULE

 Due to complex near-field motions:  EQUIVALENT SLOT DIFFUSER (2-D) GEOMETRY

 Profile definitions:
   BV = Gaussian 1/e (37%) half-width, in vertical plane normal to trajectory
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic centerline dilution
   C  = centerline concentration (includes reaction effects, if any)

       X        Y        Z        S        C        BV       BH
    0.00     0.00     0.31     1.0 0.100E+03   0.02     3.71

END OF MOD201: DIFFUSER DISCHARGE MODULE
-------------------------------------------------------------------------------
-------------------------------------------------------------------------------
BEGIN MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

 Because of the FANNED-OUT HORIZONTAL ORIENTATION of the diffuser jets,
   the near-field dilution is slightly improved.

 In this laterally contracting zone the diffuser plume becomes VERTICALLY FULLY
  MIXED over the entire layer depth (HS =    1.71m).
   Full mixing is achieved after a plume distance of about five
   layer depths from the diffuser.

 Profile definitions:
   BV = layer depth (vertically mixed)
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

       X        Y        Z        S        C        BV       BH
    0.00     0.00     0.31     1.0 0.100E+03   0.02     3.71
    0.07     0.00     0.32     3.1 0.320E+02   0.04     3.68
    0.15     0.00     0.33     4.0 0.250E+02   0.07     3.66
    0.22     0.00     0.34     4.7 0.214E+02   0.10     3.64
    0.30     0.00     0.35     5.2 0.191E+02   0.14     3.62
    0.37     0.00     0.36     5.7 0.174E+02   0.17     3.60
    0.45     0.00     0.37     6.2 0.161E+02   0.21     3.58
    0.52     0.00     0.38     6.6 0.151E+02   0.24     3.56
    0.59     0.00     0.39     7.0 0.143E+02   0.27     3.54
    0.67     0.00     0.40     7.4 0.136E+02   0.31     3.53
    0.74     0.00     0.42     7.7 0.130E+02   0.34     3.51
    0.82     0.00     0.43     8.0 0.124E+02   0.38     3.50
    0.89     0.00     0.44     8.4 0.120E+02   0.41     3.48
    0.96     0.00     0.45     8.7 0.116E+02   0.44     3.47
    1.04     0.00     0.46     8.9 0.112E+02   0.48     3.46
```

1.11	0.00	0.47	9.2	0.108E+02	0.51	3.44
1.19	0.00	0.48	9.5	0.105E+02	0.55	3.43
1.26	0.00	0.49	9.7	0.103E+02	0.58	3.42
1.34	0.00	0.50	10.0	0.100E+02	0.62	3.41
1.41	0.00	0.51	10.3	0.976E+01	0.65	3.40
1.48	0.00	0.53	10.5	0.953E+01	0.68	3.39
1.56	0.00	0.54	10.7	0.932E+01	0.72	3.38
1.63	0.00	0.55	11.0	0.913E+01	0.75	3.38
1.71	0.00	0.56	11.2	0.895E+01	0.79	3.37
1.78	0.00	0.57	11.4	0.877E+01	0.82	3.36
1.86	0.00	0.58	11.6	0.861E+01	0.86	3.35
1.93	0.00	0.59	11.8	0.846E+01	0.89	3.34
2.00	0.00	0.60	12.0	0.831E+01	0.92	3.34
2.08	0.00	0.61	12.2	0.818E+01	0.96	3.33
2.15	0.00	0.62	12.4	0.805E+01	0.99	3.33
2.23	0.00	0.64	12.6	0.792E+01	1.03	3.32
2.30	0.00	0.65	12.8	0.780E+01	1.06	3.31
2.37	0.00	0.66	13.0	0.769E+01	1.09	3.31
2.45	0.00	0.67	13.2	0.758E+01	1.13	3.30
2.52	0.00	0.68	13.4	0.748E+01	1.16	3.30
2.60	0.00	0.69	13.6	0.738E+01	1.20	3.30
2.67	0.00	0.70	13.7	0.728E+01	1.23	3.29
2.75	0.00	0.71	13.9	0.719E+01	1.27	3.29
2.82	0.00	0.72	14.1	0.710E+01	1.30	3.29
2.89	0.00	0.73	14.3	0.702E+01	1.33	3.28
2.97	0.00	0.74	14.4	0.693E+01	1.37	3.28
3.04	0.00	0.76	14.6	0.685E+01	1.40	3.28
3.12	0.00	0.77	14.8	0.678E+01	1.44	3.28
3.19	0.00	0.78	14.9	0.670E+01	1.47	3.28
3.26	0.00	0.79	15.1	0.663E+01	1.50	3.28
3.34	0.00	0.80	15.2	0.656E+01	1.54	3.27
3.41	0.00	0.81	15.4	0.650E+01	1.57	3.27
3.49	0.00	0.82	15.5	0.643E+01	1.61	3.27
3.56	0.00	0.83	15.7	0.637E+01	1.64	3.27
3.64	0.00	0.84	15.9	0.631E+01	1.68	3.27
3.71	0.00	0.85	16.0	0.625E+01	1.71	3.27

Cumulative travel time = 5.4615 sec

Plume centerline may exhibit slight discontinuities in transition to subsequent far-field module.

END OF MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

BEGIN MOD251: DIFFUSER PLUME IN CO-FLOW

Phase 1: Vertically mixed, Phase 2: Re-stratified

Phase 1: The diffuser plume is VERTICALLY FULLY MIXED over the entire layer depth.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

Phase 2: The flow has RESTRATIFIED at the beginning of this zone.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

END OF MOD251: DIFFUSER PLUME IN CO-FLOW

** End of NEAR-FIELD REGION (NFR) **

The initial plume WIDTH values in the next far-field module will be CORRECTED by a factor 1.41 to conserve the mass flux in the far-field!

BEGIN MOD241: BUOYANT AMBIENT SPREADING

Discharge is non-buoyant or weakly buoyant.
Therefore BUOYANT SPREADING REGIME is ABSENT.

END OF MOD241: BUOYANT AMBIENT SPREADING

BEGIN MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Vertical diffusivity (initial value) = 0.178E-01 m^2/s
Horizontal diffusivity (initial value) = 0.222E-01 m^2/s

Profile definitions:
BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
= or equal to layer depth, if fully mixed
BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
measured horizontally in Y-direction
ZU = upper plume boundary (Z-coordinate)
ZL = lower plume boundary (Z-coordinate)
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 1 (not bank attached):

X	Y	Z	S	C	BV	BH	ZU	ZL
3.71	0.00	1.71	16.0	0.625E+01	1.71	4.62	1.71	0.00

Plume interacts with BOTTOM.
The passive diffusion plume becomes VERTICALLY FULLY MIXED within this prediction interval.

7.30	0.00	1.71	16.2	0.618E+01	1.71	4.67	1.71	0.00
10.88	0.00	1.71	16.4	0.612E+01	1.71	4.72	1.71	0.00
14.47	0.00	1.71	16.5	0.605E+01	1.71	4.77	1.71	0.00
18.05	0.00	1.71	16.7	0.599E+01	1.71	4.82	1.71	0.00
21.64	0.00	1.71	16.9	0.593E+01	1.71	4.86	1.71	0.00
25.22	0.00	1.71	17.0	0.587E+01	1.71	4.91	1.71	0.00
28.81	0.00	1.71	17.2	0.582E+01	1.71	4.96	1.71	0.00

** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume DOWNSTREAM distance meets or exceeds the regulatory value = 30.48 m.
This is the extent of the REGULATORY MIXING ZONE.

32.40	0.00	1.71	17.4	0.576E+01	1.71	5.01	1.71	0.00
35.98	0.00	1.71	17.5	0.571E+01	1.71	5.05	1.71	0.00
39.57	0.00	1.71	17.7	0.566E+01	1.71	5.10	1.71	0.00
43.15	0.00	1.71	17.8	0.561E+01	1.71	5.14	1.71	0.00
46.74	0.00	1.71	18.0	0.556E+01	1.71	5.19	1.71	0.00
50.33	0.00	1.71	18.1	0.551E+01	1.71	5.23	1.71	0.00
53.91	0.00	1.71	18.3	0.546E+01	1.71	5.28	1.71	0.00
57.50	0.00	1.71	18.5	0.542E+01	1.71	5.32	1.71	0.00
61.08	0.00	1.71	18.6	0.538E+01	1.71	5.37	1.71	0.00
64.67	0.00	1.71	18.8	0.533E+01	1.71	5.41	1.71	0.00
68.25	0.00	1.71	18.9	0.529E+01	1.71	5.45	1.71	0.00
71.84	0.00	1.71	19.1	0.525E+01	1.71	5.50	1.71	0.00
75.43	0.00	1.71	19.2	0.521E+01	1.71	5.54	1.71	0.00
79.01	0.00	1.71	19.3	0.517E+01	1.71	5.58	1.71	0.00
82.60	0.00	1.71	19.5	0.513E+01	1.71	5.62	1.71	0.00
86.18	0.00	1.71	19.6	0.509E+01	1.71	5.66	1.71	0.00
89.77	0.00	1.71	19.8	0.506E+01	1.71	5.70	1.71	0.00
93.35	0.00	1.71	19.9	0.502E+01	1.71	5.75	1.71	0.00
96.94	0.00	1.71	20.1	0.499E+01	1.71	5.79	1.71	0.00
100.53	0.00	1.71	20.2	0.495E+01	1.71	5.83	1.71	0.00
104.11	0.00	1.71	20.3	0.492E+01	1.71	5.87	1.71	0.00
107.70	0.00	1.71	20.5	0.488E+01	1.71	5.91	1.71	0.00
111.28	0.00	1.71	20.6	0.485E+01	1.71	5.95	1.71	0.00
114.87	0.00	1.71	20.7	0.482E+01	1.71	5.98	1.71	0.00

Harmonic mean Effluent Flow 12 mgd.txt

118.46	0.00	1.71	20.9	0.479E+01	1.71	6.02	1.71	0.00
122.04	0.00	1.71	21.0	0.476E+01	1.71	6.06	1.71	0.00
125.63	0.00	1.71	21.1	0.473E+01	1.71	6.10	1.71	0.00
129.21	0.00	1.71	21.3	0.470E+01	1.71	6.14	1.71	0.00
132.80	0.00	1.71	21.4	0.467E+01	1.71	6.18	1.71	0.00
136.38	0.00	1.71	21.5	0.464E+01	1.71	6.21	1.71	0.00
139.97	0.00	1.71	21.7	0.461E+01	1.71	6.25	1.71	0.00
143.56	0.00	1.71	21.8	0.459E+01	1.71	6.29	1.71	0.00
147.14	0.00	1.71	21.9	0.456E+01	1.71	6.33	1.71	0.00
150.73	0.00	1.71	22.1	0.453E+01	1.71	6.36	1.71	0.00
154.31	0.00	1.71	22.2	0.451E+01	1.71	6.40	1.71	0.00
157.90	0.00	1.71	22.3	0.448E+01	1.71	6.44	1.71	0.00
161.49	0.00	1.71	22.4	0.446E+01	1.71	6.47	1.71	0.00
165.07	0.00	1.71	22.6	0.443E+01	1.71	6.51	1.71	0.00
168.66	0.00	1.71	22.7	0.441E+01	1.71	6.54	1.71	0.00
172.24	0.00	1.71	22.8	0.438E+01	1.71	6.58	1.71	0.00
175.83	0.00	1.71	22.9	0.436E+01	1.71	6.62	1.71	0.00
179.41	0.00	1.71	23.1	0.434E+01	1.71	6.65	1.71	0.00
183.00	0.00	1.71	23.2	0.431E+01	1.71	6.69	1.71	0.00

Cumulative travel time = 340.4690 sec

Simulation limit based on maximum specified distance = 183.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

CORMIX2: Multiport Diffuser Discharges End of Prediction File

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CORMIX2 PREDICTION FILE:
2222222222222222222222222222222222222222222222222222222222222222222222222222
                      CORMIX MIXING ZONE EXPERT SYSTEM
         Subsystem CORMIX2:  Multiport Diffuser Discharges
                          CORMIX Version 5.0GT
                     HYDRO2 Version 5.0.0.0 March 2007
------------------------------------------------------------------------------
------------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:    Lebanon
 Design case:        Max Month Flow 11 Ports, effluent flow = 12 mgd
 FILE NAME:          U:\...Q10 3 mgd modeling\MMWWF Effluent Flow 12 mgd.prd
 Time stamp:         Mon Dec 17 09:56:43 2007

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS     =      18.29  AS     =      47.37  QA     =      56.37  ICHREG= 1
 HA     =       2.59  HD     =       2.59
 UA     =      1.190  F      =      0.070 USTAR =0.1113E+00
 UW     =      2.000 UWSTAR=0.2198E-02
 Uniform density environment
 STRCND=  U          RHOAM =  998.4062

DIFFUSER DISCHARGE PARAMETERS (metric units)
 Diffuser type:      DITYPE= unidirectional_perpendicular
 BANK   =  LEFT      DISTB =       9.10  YB1    =       5.40  YB2    =      12.80
 LD     =       7.42  NOPEN =   11        SPAC   =       0.74
 D0     =      0.154 A0     =      0.019 H0     =       0.31  SUB0   =       2.29
 Nozzle/port arrangement:   unidirectional_with_fanning
 GAMMA  =      90.00  THETA  =      10.00  SIGMA  =       0.00  BETA   =      90.00
 U0     =      2.564  Q0     =      0.528        =0.5280E+00
 RHO0   =  997.8837  DRHO0 =0.5226E+00  GP0    =0.5133E-02
 C0     =0.1000E+03  CUNITS=  mg/l
 IPOLL  =  1         KS     =0.0000E+00  KD     =0.0000E+00

FLUX VARIABLES - PER UNIT DIFFUSER LENGTH (metric units)
 q0     =0.7116E-01  m0     =0.1824E+00  j0     =0.3653E-03  SIGNJ0=       1.0
 Associated 2-d length scales (meters)
 lQ=B   =      0.028 lM     =      35.57  lm     =       0.13
 lmp    =   99999.00  lbp    =   99999.00  la     =   99999.00

FLUX VARIABLES - ENTIRE DIFFUSER (metric units)
 Q0     =0.5280E+00  M0     =0.1354E+01  J0     =0.2710E-02
 Associated 3-d length scales (meters)
 LQ     =       0.14  LM     =      24.11  Lm     =       0.98  Lb     =       0.00
                                          Lmp    =   99999.00  Lbp    =   99999.00

NON-DIMENSIONAL PARAMETERS
 FR0    =     214.78  FRD0   =      91.06  R      =       2.15  PL     =       1.
 (slot)               (port/nozzle)

RECOMPUTED SOURCE CONDITIONS FOR RISER GROUPS:
 Properties of riser group with  1 ports/nozzles each:
 U0     =      2.564 D0     =      0.154 A0     =      0.019 THETA =      10.00
 FR0    =     214.78  FRD0   =      91.06  R      =       2.15
 (slot)               (riser group)

FLOW CLASSIFICATION
 22222222222222222222222222222222222222
 2  Flow class (CORMIX2)       =    MU2    2
 2  Applicable layer depth HS =      2.59  2
 22222222222222222222222222222222222222
```

```
MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0    =0.1000E+03  CUNITS=  mg/l
 NTOX  =  0
 NSTD  =  0
 REGMZ =  1
 REGSPC=  1          XREG  =     30.48  WREG  =      0.00  AREG  =      0.00
 XINT  =    183.00  XMAX  =    183.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the bottom and the diffuser mid-point:
         9.10 m  from the LEFT  bank/shore.
    X-axis points downstream, Y-axis points to left, Z-axis points upward.
NSTEP =  50 display intervals per module
--------------------------------------------------------------------------------
--------------------------------------------------------------------------------
BEGIN MOD201: DIFFUSER DISCHARGE MODULE

 Due to complex near-field motions:  EQUIVALENT SLOT DIFFUSER (2-D) GEOMETRY

 Profile definitions:
   BV = Gaussian 1/e (37%) half-width, in vertical plane normal to trajectory
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic centerline dilution
   C  = centerline concentration (includes reaction effects, if any)

       X        Y       Z        S       C        BV       BH
    0.00     0.00    0.31     1.0 0.100E+03   0.02     3.71

END OF MOD201: DIFFUSER DISCHARGE MODULE
--------------------------------------------------------------------------------
--------------------------------------------------------------------------------
BEGIN MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

 Because of the FANNED-OUT HORIZONTAL ORIENTATION of the diffuser jets,
   the near-field dilution is slightly improved.

 In this laterally contracting zone the diffuser plume becomes VERTICALLY FULLY
  MIXED over the entire layer depth (HS =    2.59m).
   Full mixing is achieved after a plume distance of about five
   layer depths from the diffuser.

 Profile definitions:
   BV = layer depth (vertically mixed)
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

       X        Y       Z        S       C        BV       BH
    0.00     0.00    0.31     1.0 0.100E+03   0.02     3.71
    0.07     0.00    0.32     7.8 0.129E+02   0.05     3.71
    0.15     0.00    0.34    10.6 0.947E+01   0.10     3.70
    0.22     0.00    0.36    12.7 0.787E+01   0.16     3.70
    0.30     0.00    0.38    14.5 0.689E+01   0.21     3.69
    0.37     0.00    0.40    16.1 0.621E+01   0.26     3.69
    0.45     0.00    0.42    17.6 0.570E+01   0.31     3.69
    0.52     0.00    0.44    18.9 0.530E+01   0.36     3.68
    0.59     0.00    0.46    20.1 0.497E+01   0.41     3.68
    0.67     0.00    0.48    21.3 0.470E+01   0.47     3.68
    0.74     0.00    0.50    22.4 0.447E+01   0.52     3.67
    0.82     0.00    0.52    23.4 0.427E+01   0.57     3.67
    0.89     0.00    0.54    24.4 0.410E+01   0.62     3.67
    0.96     0.00    0.56    25.4 0.394E+01   0.67     3.67
    1.04     0.00    0.58    26.3 0.380E+01   0.73     3.66
```

1.11	0.00	0.60	27.2	0.368E+01	0.78	3.66
1.19	0.00	0.62	28.0	0.357E+01	0.83	3.66
1.26	0.00	0.64	28.9	0.346E+01	0.88	3.66
1.34	0.00	0.66	29.7	0.337E+01	0.93	3.66
1.41	0.00	0.68	30.5	0.328E+01	0.98	3.66
1.48	0.00	0.70	31.2	0.320E+01	1.04	3.65
1.56	0.00	0.72	32.0	0.313E+01	1.09	3.65
1.63	0.00	0.74	32.7	0.306E+01	1.14	3.65
1.71	0.00	0.76	33.4	0.299E+01	1.19	3.65
1.78	0.00	0.78	34.1	0.293E+01	1.24	3.65
1.86	0.00	0.80	34.8	0.287E+01	1.30	3.65
1.93	0.00	0.82	35.5	0.282E+01	1.35	3.64
2.00	0.00	0.84	36.1	0.277E+01	1.40	3.64
2.08	0.00	0.86	36.8	0.272E+01	1.45	3.64
2.15	0.00	0.88	37.4	0.267E+01	1.50	3.64
2.23	0.00	0.90	38.0	0.263E+01	1.55	3.64
2.30	0.00	0.92	38.6	0.259E+01	1.61	3.64
2.37	0.00	0.94	39.2	0.255E+01	1.66	3.64
2.45	0.00	0.96	39.8	0.251E+01	1.71	3.64
2.52	0.00	0.98	40.4	0.248E+01	1.76	3.64
2.60	0.00	1.00	41.0	0.244E+01	1.81	3.64
2.67	0.00	1.02	41.5	0.241E+01	1.86	3.64
2.75	0.00	1.04	42.1	0.238E+01	1.92	3.64
2.82	0.00	1.06	42.7	0.234E+01	1.97	3.63
2.89	0.00	1.08	43.2	0.231E+01	2.02	3.63
2.97	0.00	1.10	43.7	0.229E+01	2.07	3.63
3.04	0.00	1.12	44.3	0.226E+01	2.12	3.63
3.12	0.00	1.14	44.8	0.223E+01	2.18	3.63
3.19	0.00	1.16	45.3	0.221E+01	2.23	3.63
3.26	0.00	1.18	45.8	0.218E+01	2.28	3.63
3.34	0.00	1.20	46.3	0.216E+01	2.33	3.63
3.41	0.00	1.22	46.8	0.214E+01	2.38	3.63
3.49	0.00	1.24	47.3	0.211E+01	2.43	3.63
3.56	0.00	1.26	47.8	0.209E+01	2.49	3.63
3.64	0.00	1.28	48.3	0.207E+01	2.54	3.63
3.71	0.00	1.29	48.8	0.205E+01	2.59	3.63

Cumulative travel time = 2.9858 sec
Plume centerline may exhibit slight discontinuities in transition
to subsequent far-field module.

END OF MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

BEGIN MOD251: DIFFUSER PLUME IN CO-FLOW

Phase 1: Vertically mixed, Phase 2: Re-stratified

Phase 1: The diffuser plume is VERTICALLY FULLY MIXED over the
entire layer depth.
This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

Phase 2: The flow has RESTRATIFIED at the beginning of this zone.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

END OF MOD251: DIFFUSER PLUME IN CO-FLOW

** End of NEAR-FIELD REGION (NFR) **

The initial plume WIDTH values in the next far-field module will be
CORRECTED by a factor 1.14 to conserve the mass flux in the far-field!

BEGIN MOD241: BUOYANT AMBIENT SPREADING

Discharge is non-buoyant or weakly buoyant.
Therefore BUOYANT SPREADING REGIME is ABSENT.

END OF MOD241: BUOYANT AMBIENT SPREADING

BEGIN MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Vertical diffusivity (initial value) = 0.577E-01 m^2/s
Horizontal diffusivity (initial value) = 0.721E-01 m^2/s

Profile definitions:
BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
= or equal to layer depth, if fully mixed
BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
measured horizontally in Y-direction
ZU = upper plume boundary (Z-coordinate)
ZL = lower plume boundary (Z-coordinate)
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 1 (not bank attached):

X	Y	Z	S	C	BV	BH	ZU	ZL
3.71	0.00	2.59	48.8	0.205E+01	2.59	4.14	2.59	0.00

Plume interacts with BOTTOM.
The passive diffusion plume becomes VERTICALLY FULLY MIXED within this prediction interval.

7.30	0.00	2.59	49.7	0.201E+01	2.59	4.22	2.59	0.00
10.88	0.00	2.59	50.7	0.197E+01	2.59	4.30	2.59	0.00
14.47	0.00	2.59	51.6	0.194E+01	2.59	4.38	2.59	0.00
18.05	0.00	2.59	52.5	0.191E+01	2.59	4.45	2.59	0.00
21.64	0.00	2.59	53.4	0.187E+01	2.59	4.53	2.59	0.00
25.22	0.00	2.59	54.3	0.184E+01	2.59	4.60	2.59	0.00
28.81	0.00	2.59	55.1	0.181E+01	2.59	4.68	2.59	0.00

** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume DOWNSTREAM distance meets or exceeds the regulatory value = 30.48 m.
This is the extent of the REGULATORY MIXING ZONE.

32.40	0.00	2.59	56.0	0.179E+01	2.59	4.75	2.59	0.00
35.98	0.00	2.59	56.8	0.176E+01	2.59	4.82	2.59	0.00
39.57	0.00	2.59	57.6	0.174E+01	2.59	4.89	2.59	0.00
43.15	0.00	2.59	58.4	0.171E+01	2.59	4.96	2.59	0.00
46.74	0.00	2.59	59.2	0.169E+01	2.59	5.03	2.59	0.00
50.33	0.00	2.59	60.0	0.167E+01	2.59	5.09	2.59	0.00
53.91	0.00	2.59	60.8	0.165E+01	2.59	5.16	2.59	0.00
57.50	0.00	2.59	61.5	0.162E+01	2.59	5.22	2.59	0.00
61.08	0.00	2.59	62.3	0.160E+01	2.59	5.29	2.59	0.00
64.67	0.00	2.59	63.1	0.159E+01	2.59	5.35	2.59	0.00
68.25	0.00	2.59	63.8	0.157E+01	2.59	5.41	2.59	0.00
71.84	0.00	2.59	64.5	0.155E+01	2.59	5.48	2.59	0.00
75.43	0.00	2.59	65.3	0.153E+01	2.59	5.54	2.59	0.00
79.01	0.00	2.59	66.0	0.152E+01	2.59	5.60	2.59	0.00
82.60	0.00	2.59	66.7	0.150E+01	2.59	5.66	2.59	0.00
86.18	0.00	2.59	67.4	0.148E+01	2.59	5.72	2.59	0.00
89.77	0.00	2.59	68.1	0.147E+01	2.59	5.78	2.59	0.00
93.35	0.00	2.59	68.8	0.145E+01	2.59	5.83	2.59	0.00
96.94	0.00	2.59	69.4	0.144E+01	2.59	5.89	2.59	0.00
100.53	0.00	2.59	70.1	0.143E+01	2.59	5.95	2.59	0.00
104.11	0.00	2.59	70.8	0.141E+01	2.59	6.01	2.59	0.00
107.70	0.00	2.59	71.4	0.140E+01	2.59	6.06	2.59	0.00
111.28	0.00	2.59	72.1	0.139E+01	2.59	6.12	2.59	0.00
114.87	0.00	2.59	72.7	0.137E+01	2.59	6.17	2.59	0.00

118.46	0.00	2.59	73.4	0.136E+01	2.59	6.23	2.59	0.00
122.04	0.00	2.59	74.0	0.135E+01	2.59	6.28	2.59	0.00
125.63	0.00	2.59	74.6	0.134E+01	2.59	6.33	2.59	0.00
129.21	0.00	2.59	75.3	0.133E+01	2.59	6.39	2.59	0.00
132.80	0.00	2.59	75.9	0.132E+01	2.59	6.44	2.59	0.00
136.38	0.00	2.59	76.5	0.131E+01	2.59	6.49	2.59	0.00
139.97	0.00	2.59	77.1	0.130E+01	2.59	6.54	2.59	0.00
143.56	0.00	2.59	77.7	0.129E+01	2.59	6.60	2.59	0.00
147.14	0.00	2.59	78.3	0.128E+01	2.59	6.65	2.59	0.00
150.73	0.00	2.59	78.9	0.127E+01	2.59	6.70	2.59	0.00
154.31	0.00	2.59	79.5	0.126E+01	2.59	6.75	2.59	0.00
157.90	0.00	2.59	80.1	0.125E+01	2.59	6.80	2.59	0.00
161.49	0.00	2.59	80.7	0.124E+01	2.59	6.85	2.59	0.00
165.07	0.00	2.59	81.3	0.123E+01	2.59	6.90	2.59	0.00
168.66	0.00	2.59	81.8	0.122E+01	2.59	6.95	2.59	0.00
172.24	0.00	2.59	82.4	0.121E+01	2.59	6.99	2.59	0.00
175.83	0.00	2.59	83.0	0.121E+01	2.59	7.04	2.59	0.00
179.41	0.00	2.59	83.5	0.120E+01	2.59	7.09	2.59	0.00
183.00	0.00	2.59	84.1	0.119E+01	2.59	7.14	2.59	0.00

Cumulative travel time = 152.2516 sec

Simulation limit based on maximum specified distance = 183.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

CORMIX2: Multiport Diffuser Discharges End of Prediction File

22222222222222222222222222222222222222222222222222222222222222222222222222

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CORMIX2 PREDICTION FILE:
2222222222222222222222222222222222222222222222222222222222222222222222222222222222
                      CORMIX MIXING ZONE EXPERT SYSTEM
          Subsystem CORMIX2:  Multiport Diffuser Discharges
                          CORMIX Version 5.0GT
                     HYDRO2 Version 5.0.0.0 March 2007
------------------------------------------------------------------------------
------------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:    Lebanon
 Design case:        Max Month Flow 11 Ports, effluent flow = 21 mgd
 FILE NAME:          U:\...Q10 3 mgd modeling\MMWWF Effluent Flow 21 mgd.prd
 Time stamp:         Mon Dec 17 09:59:25 2007

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS     =      18.29  AS     =      47.37  QA     =      56.37  ICHREG= 1
 HA     =       2.59  HD     =       2.59
 UA     =      1.190  F      =      0.070 USTAR =0.1113E+00
 UW     =      2.000 UWSTAR=0.2198E-02
 Uniform density environment
 STRCND=  U          RHOAM =  998.4062

DIFFUSER DISCHARGE PARAMETERS (metric units)
 Diffuser type:      DITYPE= unidirectional_perpendicular
 BANK   =  LEFT      DISTB =       9.10  YB1    =       5.40  YB2    =      12.80
 LD     =       7.42 NOPEN =   11        SPAC   =       0.74
 D0     =      0.174 A0     =      0.024 H0     =       0.31  SUB0   =       2.29
 Nozzle/port arrangement:   unidirectional_with_fanning
 GAMMA  =      90.00  THETA =      10.00  SIGMA  =       0.00  BETA   =      90.00
 U0     =      3.501  Q0     =      0.920        =0.9200E+00
 RHO0   =  997.8837  DRHO0 =0.5226E+00  GP0    =0.5133E-02
 C0     =0.1000E+03  CUNITS=  mg/l
 IPOLL  =  1         KS     =0.0000E+00  KD     =0.0000E+00

FLUX VARIABLES - PER UNIT DIFFUSER LENGTH (metric units)
 q0     =0.1240E+00  m0     =0.4341E+00  j0     =0.6364E-03  SIGNJ0=       1.0
 Associated 2-d length scales (meters)
 lQ=B   =      0.035 lM     =      58.43  lm     =       0.31
 lmp    =   99999.00 lbp    =   99999.00  la     =   99999.00

FLUX VARIABLES - ENTIRE DIFFUSER (metric units)
 Q0     =0.9200E+00  M0     =0.3221E+01  J0     =0.4722E-02
 Associated 3-d length scales (meters)
 LQ     =       0.15 LM     =      34.99  Lm     =       1.51  Lb     =       0.00
                                          Lmp    =   99999.00  Lbp    =   99999.00

NON-DIMENSIONAL PARAMETERS
 FR0    =     259.68  FRD0   =     117.02  R      =       2.94  PL     =       1.
 (slot)               (port/nozzle)

RECOMPUTED SOURCE CONDITIONS FOR RISER GROUPS:
 Properties of riser group with  1 ports/nozzles each:
 U0     =      3.501 D0     =      0.174 A0     =      0.024 THETA =      10.00
 FR0    =     259.68  FRD0   =     117.02  R      =       2.94
 (slot)               (riser group)

FLOW CLASSIFICATION
 22222222222222222222222222222222222222
 2  Flow class (CORMIX2)       =    MU2    2
 2  Applicable layer depth HS =      2.59  2
 22222222222222222222222222222222222222
```

MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0   =0.1000E+03 CUNITS= mg/l
 NTOX = 0
 NSTD = 0
 REGMZ = 1
 REGSPC= 1   XREG =   30.48 WREG =   0.00 AREG =   0.00
 XINT =   183.00 XMAX =   183.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the bottom and the diffuser mid-point:
         9.10 m from the LEFT bank/shore.
    X-axis points downstream, Y-axis points to left, Z-axis points upward.
NSTEP = 50 display intervals per module

BEGIN MOD201: DIFFUSER DISCHARGE MODULE

 Due to complex near-field motions: EQUIVALENT SLOT DIFFUSER (2-D) GEOMETRY

 Profile definitions:
   BV = Gaussian 1/e (37%) half-width, in vertical plane normal to trajectory
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S = hydrodynamic centerline dilution
   C = centerline concentration (includes reaction effects, if any)

X	Y	Z	S	C	BV	BH
0.00	0.00	0.31	1.0	0.100E+03	0.03	3.71

END OF MOD201: DIFFUSER DISCHARGE MODULE

BEGIN MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

 Because of the FANNED-OUT HORIZONTAL ORIENTATION of the diffuser jets,
   the near-field dilution is slightly improved.

 In this laterally contracting zone the diffuser plume becomes VERTICALLY FULLY
  MIXED over the entire layer depth (HS =   2.59m).
   Full mixing is achieved after a plume distance of about five
   layer depths from the diffuser.

 Profile definitions:
   BV = layer depth (vertically mixed)
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S = hydrodynamic average (bulk) dilution
   C = average (bulk) concentration (includes reaction effects, if any)

X	Y	Z	S	C	BV	BH
0.00	0.00	0.31	1.0	0.100E+03	0.03	3.71
0.07	0.00	0.32	4.9	0.202E+02	0.05	3.70
0.15	0.00	0.34	6.6	0.152E+02	0.10	3.69
0.22	0.00	0.36	7.8	0.128E+02	0.16	3.68
0.30	0.00	0.38	8.9	0.113E+02	0.21	3.67
0.37	0.00	0.40	9.8	0.102E+02	0.26	3.67
0.45	0.00	0.42	10.6	0.939E+01	0.31	3.66
0.52	0.00	0.44	11.4	0.876E+01	0.36	3.65
0.59	0.00	0.46	12.1	0.824E+01	0.41	3.64
0.67	0.00	0.48	12.8	0.780E+01	0.47	3.64
0.74	0.00	0.50	13.5	0.743E+01	0.52	3.63
0.82	0.00	0.52	14.1	0.711E+01	0.57	3.63
0.89	0.00	0.54	14.6	0.683E+01	0.62	3.62
0.96	0.00	0.56	15.2	0.658E+01	0.67	3.62
1.04	0.00	0.58	15.7	0.635E+01	0.73	3.61

1.11	0.00	0.60	16.3	0.615E+01	0.78	3.61
1.19	0.00	0.62	16.8	0.597E+01	0.83	3.60
1.26	0.00	0.64	17.2	0.580E+01	0.88	3.60
1.34	0.00	0.66	17.7	0.565E+01	0.93	3.59
1.41	0.00	0.68	18.2	0.550E+01	0.98	3.59
1.48	0.00	0.70	18.6	0.537E+01	1.04	3.59
1.56	0.00	0.72	19.1	0.525E+01	1.09	3.58
1.63	0.00	0.74	19.5	0.513E+01	1.14	3.58
1.71	0.00	0.76	19.9	0.503E+01	1.19	3.58
1.78	0.00	0.78	20.3	0.493E+01	1.24	3.57
1.86	0.00	0.80	20.7	0.483E+01	1.30	3.57
1.93	0.00	0.82	21.1	0.474E+01	1.35	3.57
2.00	0.00	0.84	21.5	0.466E+01	1.40	3.56
2.08	0.00	0.86	21.8	0.458E+01	1.45	3.56
2.15	0.00	0.88	22.2	0.450E+01	1.50	3.56
2.23	0.00	0.90	22.6	0.443E+01	1.55	3.56
2.30	0.00	0.92	22.9	0.436E+01	1.61	3.56
2.37	0.00	0.94	23.3	0.429E+01	1.66	3.55
2.45	0.00	0.96	23.6	0.423E+01	1.71	3.55
2.52	0.00	0.98	24.0	0.417E+01	1.76	3.55
2.60	0.00	1.00	24.3	0.411E+01	1.81	3.55
2.67	0.00	1.02	24.6	0.406E+01	1.86	3.55
2.75	0.00	1.04	25.0	0.401E+01	1.92	3.55
2.82	0.00	1.06	25.3	0.396E+01	1.97	3.54
2.89	0.00	1.08	25.6	0.391E+01	2.02	3.54
2.97	0.00	1.10	25.9	0.386E+01	2.07	3.54
3.04	0.00	1.12	26.2	0.381E+01	2.12	3.54
3.12	0.00	1.14	26.5	0.377E+01	2.18	3.54
3.19	0.00	1.16	26.8	0.373E+01	2.23	3.54
3.26	0.00	1.18	27.1	0.369E+01	2.28	3.54
3.34	0.00	1.20	27.4	0.365E+01	2.33	3.54
3.41	0.00	1.22	27.7	0.361E+01	2.38	3.54
3.49	0.00	1.24	28.0	0.357E+01	2.43	3.54
3.56	0.00	1.26	28.3	0.353E+01	2.49	3.54
3.64	0.00	1.28	28.6	0.350E+01	2.54	3.54
3.71	0.00	1.29	28.9	0.347E+01	2.59	3.54

Cumulative travel time = 2.8292 sec
Plume centerline may exhibit slight discontinuities in transition
to subsequent far-field module.

END OF MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

BEGIN MOD251: DIFFUSER PLUME IN CO-FLOW

Phase 1: Vertically mixed, Phase 2: Re-stratified

Phase 1: The diffuser plume is VERTICALLY FULLY MIXED over the
entire layer depth.
This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

Phase 2: The flow has RESTRATIFIED at the beginning of this zone.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

END OF MOD251: DIFFUSER PLUME IN CO-FLOW

** End of NEAR-FIELD REGION (NFR) **

The initial plume WIDTH values in the next far-field module will be
CORRECTED by a factor 1.20 to conserve the mass flux in the far-field!

BEGIN MOD241: BUOYANT AMBIENT SPREADING

Discharge is non-buoyant or weakly buoyant.
Therefore BUOYANT SPREADING REGIME is ABSENT.

END OF MOD241: BUOYANT AMBIENT SPREADING

BEGIN MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Vertical diffusivity (initial value) = 0.577E-01 m^2/s
Horizontal diffusivity (initial value) = 0.721E-01 m^2/s

Profile definitions:
BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
= or equal to layer depth, if fully mixed
BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
measured horizontally in Y-direction
ZU = upper plume boundary (Z-coordinate)
ZL = lower plume boundary (Z-coordinate)
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 1 (not bank attached):

X	Y	Z	S	C	BV	BH	ZU	ZL
3.71	0.00	2.59	28.9	0.347E+01	2.59	4.24	2.59	0.00

Plume interacts with BOTTOM.
The passive diffusion plume becomes VERTICALLY FULLY MIXED within this prediction interval.

7.30	0.00	2.59	29.4	0.340E+01	2.59	4.32	2.59	0.00
10.88	0.00	2.59	29.9	0.334E+01	2.59	4.39	2.59	0.00
14.47	0.00	2.59	30.4	0.329E+01	2.59	4.47	2.59	0.00
18.05	0.00	2.59	30.9	0.323E+01	2.59	4.54	2.59	0.00
21.64	0.00	2.59	31.4	0.318E+01	2.59	4.62	2.59	0.00
25.22	0.00	2.59	31.9	0.313E+01	2.59	4.69	2.59	0.00
28.81	0.00	2.59	32.4	0.309E+01	2.59	4.76	2.59	0.00

** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume DOWNSTREAM distance meets or exceeds the regulatory value = 30.48 m.
This is the extent of the REGULATORY MIXING ZONE.

32.40	0.00	2.59	32.9	0.304E+01	2.59	4.83	2.59	0.00
35.98	0.00	2.59	33.4	0.300E+01	2.59	4.90	2.59	0.00
39.57	0.00	2.59	33.8	0.296E+01	2.59	4.97	2.59	0.00
43.15	0.00	2.59	34.3	0.292E+01	2.59	5.03	2.59	0.00
46.74	0.00	2.59	34.7	0.288E+01	2.59	5.10	2.59	0.00
50.33	0.00	2.59	35.2	0.284E+01	2.59	5.17	2.59	0.00
53.91	0.00	2.59	35.6	0.281E+01	2.59	5.23	2.59	0.00
57.50	0.00	2.59	36.1	0.277E+01	2.59	5.29	2.59	0.00
61.08	0.00	2.59	36.5	0.274E+01	2.59	5.36	2.59	0.00
64.67	0.00	2.59	36.9	0.271E+01	2.59	5.42	2.59	0.00
68.25	0.00	2.59	37.3	0.268E+01	2.59	5.48	2.59	0.00
71.84	0.00	2.59	37.7	0.265E+01	2.59	5.54	2.59	0.00
75.43	0.00	2.59	38.1	0.262E+01	2.59	5.60	2.59	0.00
79.01	0.00	2.59	38.6	0.259E+01	2.59	5.66	2.59	0.00
82.60	0.00	2.59	39.0	0.257E+01	2.59	5.72	2.59	0.00
86.18	0.00	2.59	39.4	0.254E+01	2.59	5.78	2.59	0.00
89.77	0.00	2.59	39.7	0.252E+01	2.59	5.84	2.59	0.00
93.35	0.00	2.59	40.1	0.249E+01	2.59	5.89	2.59	0.00
96.94	0.00	2.59	40.5	0.247E+01	2.59	5.95	2.59	0.00
100.53	0.00	2.59	40.9	0.244E+01	2.59	6.01	2.59	0.00
104.11	0.00	2.59	41.3	0.242E+01	2.59	6.06	2.59	0.00
107.70	0.00	2.59	41.7	0.240E+01	2.59	6.12	2.59	0.00
111.28	0.00	2.59	42.0	0.238E+01	2.59	6.17	2.59	0.00
114.87	0.00	2.59	42.4	0.236E+01	2.59	6.23	2.59	0.00

118.46	0.00	2.59	42.8	0.234E+01	2.59	6.28	2.59	0.00
122.04	0.00	2.59	43.1	0.232E+01	2.59	6.33	2.59	0.00
125.63	0.00	2.59	43.5	0.230E+01	2.59	6.39	2.59	0.00
129.21	0.00	2.59	43.8	0.228E+01	2.59	6.44	2.59	0.00
132.80	0.00	2.59	44.2	0.226E+01	2.59	6.49	2.59	0.00
136.38	0.00	2.59	44.5	0.224E+01	2.59	6.54	2.59	0.00
139.97	0.00	2.59	44.9	0.223E+01	2.59	6.59	2.59	0.00
143.56	0.00	2.59	45.2	0.221E+01	2.59	6.64	2.59	0.00
147.14	0.00	2.59	45.6	0.219E+01	2.59	6.69	2.59	0.00
150.73	0.00	2.59	45.9	0.218E+01	2.59	6.74	2.59	0.00
154.31	0.00	2.59	46.3	0.216E+01	2.59	6.79	2.59	0.00
157.90	0.00	2.59	46.6	0.215E+01	2.59	6.84	2.59	0.00
161.49	0.00	2.59	46.9	0.213E+01	2.59	6.89	2.59	0.00
165.07	0.00	2.59	47.3	0.212E+01	2.59	6.94	2.59	0.00
168.66	0.00	2.59	47.6	0.210E+01	2.59	6.99	2.59	0.00
172.24	0.00	2.59	47.9	0.209E+01	2.59	7.04	2.59	0.00
175.83	0.00	2.59	48.2	0.207E+01	2.59	7.08	2.59	0.00
179.41	0.00	2.59	48.6	0.206E+01	2.59	7.13	2.59	0.00
183.00	0.00	2.59	48.9	0.205E+01	2.59	7.18	2.59	0.00

Cumulative travel time = 151.0736 sec

Simulation limit based on maximum specified distance = 183.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

CORMIX2: Multiport Diffuser Discharges End of Prediction File

2222222222222222222222222222222222222222222222222222222222222222222222222222

Appendix G: Biosolids Management Plan

Kennedy Jenks

1500 NE Irving Street, Suite 200
Portland, Oregon 97232
503-423-4000

City of Lebanon Biosolids Management Plan

11 August 2025

Prepared for

City of Lebanon
33110 Tennessee Road
Lebanon, Oregon 97355

KJ Project No. 2576005*00

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List of Acronyms and Abbreviations

%	percent
°C	degrees Celsius
°F	degrees Fahrenheit
ADF	average daily flow
ADWDF	average dry weather design flow
AWWDF	average wet weather design flow
BFP	belt filter press
BSMP	Biosolids Management Plan
CCB	chlorine contact basins
CFR	Code of Federal Regulations
CFU	colony forming units
City	City of Lebanon
DEQ	Oregon Department of Environmental Quality
DO	dissolved oxygen
e.g.	for example
ESI	East Side Interceptor
EPA	United States Environmental Protection Agency
EQ	Exceptional Quality
gal	gallon
GBT	gravity belt thickener
gpm	gallons per minute
hp	horsepower
i.e.	that is
ICP	Inductively Coupled Plasma spectrometer
IMD	Internal Management Directive
MMDWF	maximum month dry weather flow
MMWWF	maximum month wet weather flow
MGD	million gallons per day
MPN	most probable number
N/A	not applicable
ND	not detected
NH_4	ammonium-nitrogen
PNW	Pacific Northwest
TKN	total Kjeldahl nitrogen
NO_3	nitrate-nitrogen
NPDES	National Pollutant Discharge Elimination System
OAR	Oregon Administrative Rules
OERS	Oregon Emergency Response System
OSU	Oregon State University
PDF	peak day flow
PIF	peak instantaneous flow
RAS	return activated sludge
SM	standard method

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SOUR	Specific Oxygen Uptake Rate
TS	total solids
UGB	Urban Growth Boundary
UV	ultraviolet
UW	University of Washington
VAR	vector attraction reduction
WAS	waste activated sludge
WSI	West Side Interceptor
WIPS	West Side Interceptor Pump Station
WSU	Washington State University
WWTP	Wastewater Treatment Plant

Executive Summary

The City of Lebanon (City) owns and operates a municipal wastewater collection and treatment system, and manages a biosolids land application program. The City took over operations of their wastewater treatment plant (WWTP) from a contract operator in September 2020. Wastewater processed by the treatment works is primarily of domestic origin, and no formal pretreatment program is required to be implemented under our National Pollutant Discharge Elimination System (NPDES) permit. The City has three sewage lift stations (two in the collection system and one at the WWTP), 66 miles of gravity lines and currently 3.5 state certified operators and budgeted for 5.5.

The City's primary method of managing biosolids includes beneficial use to local agricultural crops. The City periodically hauls dewatered biosolids to Coffin Butte Landfill or liquid biosolids to the City of Albany for disposal into their wastewater treatment plant.

This Biosolids Management Plan (BSMP) provides information about how the City's biosolids program is operated, as well as monitoring and testing data to document product quality. The BSMP addresses the requirements set forth in Oregon Administrative Rules (OAR) Chapter 340 Division 50 and Title 40 Code of Federal Regulations (CFR) Part 503.

Section 1: Introduction

Biosolids are the nutrient-rich organic materials resulting from the treatment of domestic sewage at domestic WWTPs. Through biosolids management, solid residue from the wastewater treatment process is treated to reduce or eliminate pathogens and minimize odors, forming a safe, beneficial product. Biosolids can be applied as a soil amendment to improve and maintain productive soils and stimulate plant growth. They are also used to fertilize gardens and parks and reclaim mining sites. Biosolids are classified into two general categories, Class A and Class B.

Class A biosolids contain insignificant levels of pathogens. To achieve Class A, biosolids must undergo heating, composting, digestion or lime addition that reduces pathogens to below detectable levels. Biosolids must also meet or exceed federal standards for nine regulated pollutants in accordance with 40 CFR Part 503.13(b)(1) or (3). If the biosolids achieve the low pollutant concentrations specified in 40 CFR Part 503.13(b)(3) and Class A with respect to pathogens, they are deemed Class A Exceptional Quality (EQ). Class A EQ biosolids can be land applied without any pathogen-related restrictions at a site or bagged and marketed to the public for application to lawns, gardens and multiple other landscape uses.

Class B biosolids have less stringent standards for treatment and contain small amounts of pathogens. Class B regulatory requirements ensure pathogens in biosolids have been reduced to levels that protect public health and the environment and include certain restrictions for crop harvesting, grazing animals and public contact for all forms of Class B biosolids. As is true of their Class A counterpart, Class B biosolids are treated in a wastewater treatment facility and undergo heating, digestion or increased pH processes before leaving the WWTP. Unlike Class A, Class B biosolids cannot be bagged or marketed to the public for application to gardens, landscapes or other public uses. Rather, Class B biosolids are typically land applied to agricultural sites under permits and with restricted public access. When managed in this way, biosolids land application provides beneficial reuse or recycling of nutrients and organic matter. This is one of the many ways WWTP staff contributes to a better environment for Oregon. The City manages its biosolids in two ways: (1) hauling liquid or dewatered Class B biosolids to Oregon Department of Environmental Quality (DEQ) authorized biosolids land applications sites and (2) hauling dewatered solids to Coffin Butte Landfill, as needed.

Biosolids are regulated by both the United States Environmental Protection Agency (EPA) and DEQ to ensure quality standards are met. EPA's regulations can be found in 40 CFR Part 503 and DEQ's Chapter 340 Division 50 of the OARs. Regulations address pollutant concentrations, pathogen content, odor potential and basic operational practices. Beneficial reuse of biosolids has long been preferred over historical disposal practices such as incineration or landfilling. Land application practices and marketable biosolids products are encouraged, as an alternative to disposal, by state and federal regulatory authorities. Numerous publications from the EPA and regional academic institutions such as Oregon State University (OSU), Washington State University (WSU), and University of Washington (UW) provide valuable information regarding biosolids management practices. DEQ's December 2005 biosolids guidance titled, "Implementing Oregon's Biosolids Program Internal Management Directive" (IMD) provides very useful information for permit writers and the public regarding how Oregon administers state and federal biosolids regulations.

This BSMP outlines treatment practices, monitoring, and methods of operation for the City's biosolids program following DEQ's BSMP template contained within the December 2005 Biosolids IMD.

The BSMP is intended to satisfy the National Pollutant Discharge Elimination System (NPDES) requirements for the City's WWTP:

- NPDES Permit for the WWTP
  - NPDES Permit Number: 101771
  - File Number: 49764
  - Contact Name: Jason Williams
  - Contact Title: Public Works Director
  - Contact Telephone Number: 541-258-4280

Section 2: Biosolids Program Contact Information

The following table describes the City's biosolids program contact information including position title, responsibilities, and phone numbers.

Table 2-1: Biosolids Program Contact Information

Title	Role	Phone Numbers
Steve Haney, Water/Wastewater Treatment Plant Manager, City of Lebanon	• Overall direction and responsibility for program • Questions regarding the BSMP • Daily biosolids management	541-258-4991
Jason Williams, Public Works Director, City of Lebanon	• City public works administration and oversight	541-258-4280

Section 3: Wastewater Collection System

3.1 Collection System

The City maintains and operates a sewage collection system within its Urban Growth Boundary (UGB) that totals over 300,000 feet of pipe, six pump stations, and multiple overflow structures. The collection system consists of a gravity-fed system with pipe diameters ranging from 6 to 54 inches, as summarized in **Table 3-1**.

Table 3-1: Collection System Inventory

Sewer Pipe Diameter, Inches	Pipe Length, Feet[(a)]
6	6,990
8	202,410
10	31,770
12	41084
14	720
15	70
16	3,510
18	11,830
20	1,340
21	3,230
24	4,500
27	8,700
30	3,930
36	7,900
42	4,900
54	7,900

Notes:
(a) Values rounded to the nearest 100 ft.

As shown in **Figure 3-1**, there are two main sanitary sewer interceptor systems: the East Side Interceptor (ESI) system and the West Side Interceptor (WSI) system. The ESI system generally serves the area bounded by River Drive in the south, W. Mary Street to the north, 6th Street to the west and the South Santiam River on the east. The WSI system generally serves the area to the west of 6th Street and extending north and south to the edges of the UGB.

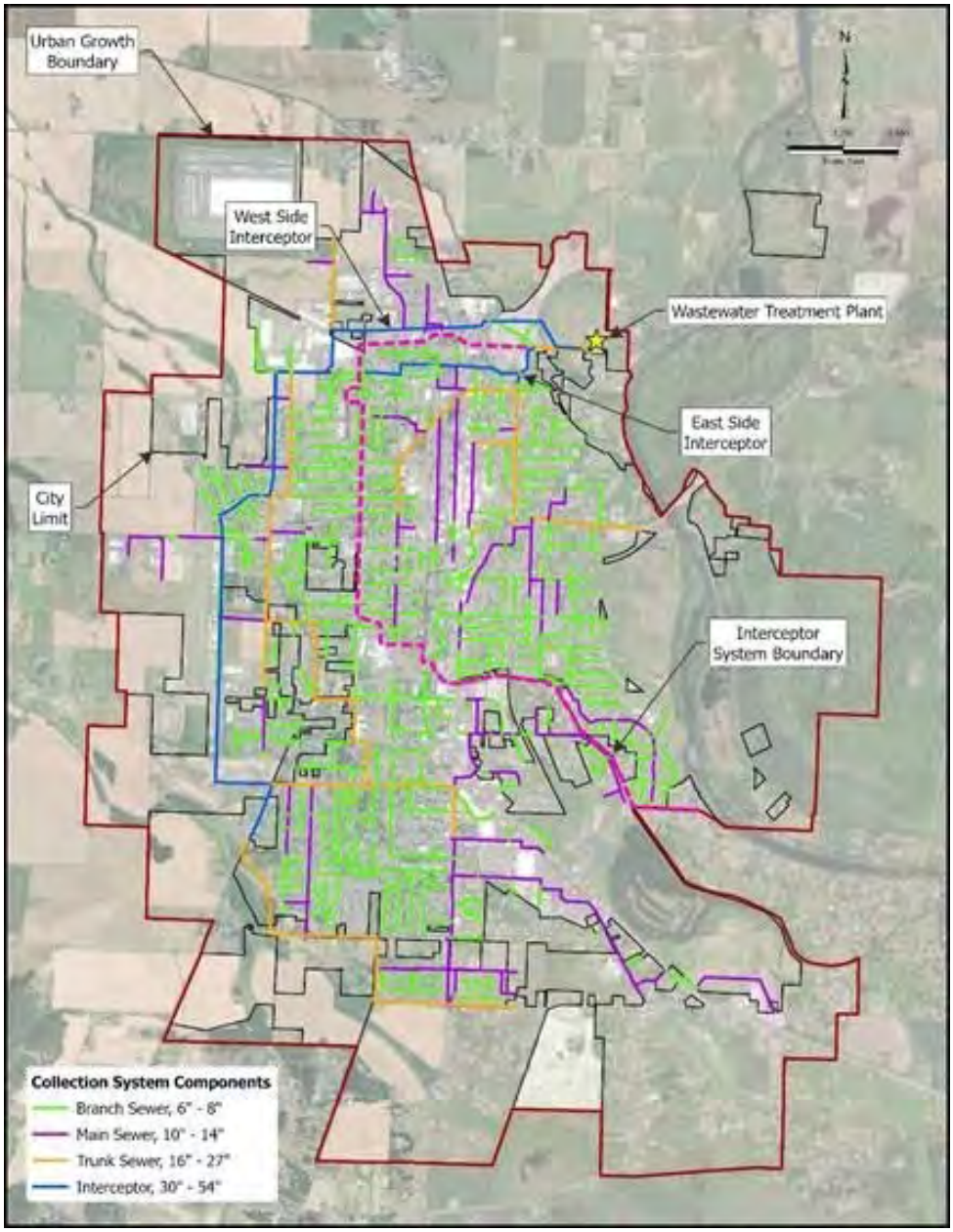

Figure 3-1: Collection System

Section 4: Wastewater Treatment Plant

The WWTP is located to the northeast of the City at the intersection of Beaton Lane and Tennessee Road (see **Figure 4-1**). It was originally constructed in 1954 and consisted of a grit channel, comminutor, primary clarifier, trickling filter, secondary clarifier, chlorine disinfection, aerobic digester, and sludge drying beds. Structures from the original WWTP still in service include the trickling filter, primary clarifier, and secondary clarifier. In 1976, the trickling filter was converted to an aerobic digester. The primary clarifier was converted to a secondary clarifier.

Figure 4-1: Lebanon WWTP Location Map

Definitions of the various flow conditions are shown below.

- **Average dry weather flow (ADWF)** is the average flow at the WWTP during the dry weather season, defined as May through October.
- **Average wet weather flow (AWWF)** is the average flow at the WWTP during the wet weather season, defined as November through April.
- **Average daily flow (ADF)** is the annual average daily flow.
- **Maximum month dry weather flow (MMDWF)** is the dry-weather, maximum month flow rate estimated to occur once every 10 years (10 percent (%) probability of occurrence). In western Oregon, this value normally occurs during May and is associated with the monthly rainfall accumulation with a 10% probability of occurrence.
- **Maximum month wet weather flow (MMWWF**) is the wet-weather, maximum month flow estimated to occur once every 5 years (20% probability of occurrence). This value occurs during the wet weather season when groundwater levels are high (typically from January through April). The value typically occurs in January and is associated with the monthly rainfall accumulation that has a 20% probability of occurrence.
- **Peak day flow (PDF)** is the flow rate corresponding to a 1-in-5 year, 24-hour storm event that occurs during a period of high groundwater and saturated soils.
- **Peak instantaneous flow (PIF)** is the peak hour flow rate expected to occur during the peak day flow.

The ADWDF is 2.3 MGD with an AWWDF of 6.4 MGD. Of this flow approximately 90% is domestic and 10% is from commercial or industrial sources (e.g., hotels and breweries). **Table 4-1** below shows the various flows for the WWTP.

Table 4-1: Wastewater Treatment Plant Flows

Design Flow Type	Current (2022), MGD[(a)]	Design Year (2045)[(b)]
ADWF	2.3	2.7
AWWF	6.4	7.1
ADF	4.3	5.0
MMDWF	5.4	6.0
MMWWF	9.2	10.1
PDF	18.1	19.4
PIF	29.2	31.1

Notes:

(a) MGD = million gallons per day
(b) Data from ongoing facility planning update in 2025

There have been numerous modifications to the WWTP since its original construction. The most significant of these changes are summarized in **Table 4-2**, below.

Table 4-2: **Wastewater Treatment Plant Modifications**

Year	Project Name	Project Elements
1976	Wastewater Treatment Plant Upgrades	• Raw Sewage Pump Station • Headworks • Storm Screens • Aeration Basins • Convert Primary Clarifier to Secondary Clarifier • Tertiary Filtration • Chlorine Contact Basins with Gaseous Chlorine • Sludge Pump Station • Convert Trickling Filter to Aerobic Digester • Convert Digester to Sludge Storage Tank • Control Building
1992	Sludge Thickening Facility	• Sludge Thickening Building • Gravity Belt Thickener
1999	Sodium Hypochlorite Conversion Plan	• Sodium Hypochlorite Storage and Metering
2003	West Side Interceptor Pump Station	• Submersible Influent Pump Station • New engine driven standby generator
2008	Effluent Discharge System Improvements	• 42-inch Outfall and River Diffuser • Sodium Bisulfite Dechlorination Storage and Metering
2008	Cannibal® Solids Reduction System Project	• Convert to Cannibal® Solids Reduction System • Convert Aerobic Digester to Interchange Reactor • Aeration Basin Upgrades
2012	Clarifier Expansion Improvements	• 130-ft Secondary Clarifier • RAS/WAS/Scum Pumping
2015	Effluent Pump Station Improvements	• New Submersible Pumps • Electrical Enclosure
2020	City Resumes Operation of Plant	• Eliminate Cannibal® Solids Reduction System process • Convert Interchange Reactor to Aerobic Digester • New Belt Filter Press

The existing WWTP facilities are shown in **Figure 4-2** below. They are described in detail in the subsequent sections of this BSMP.

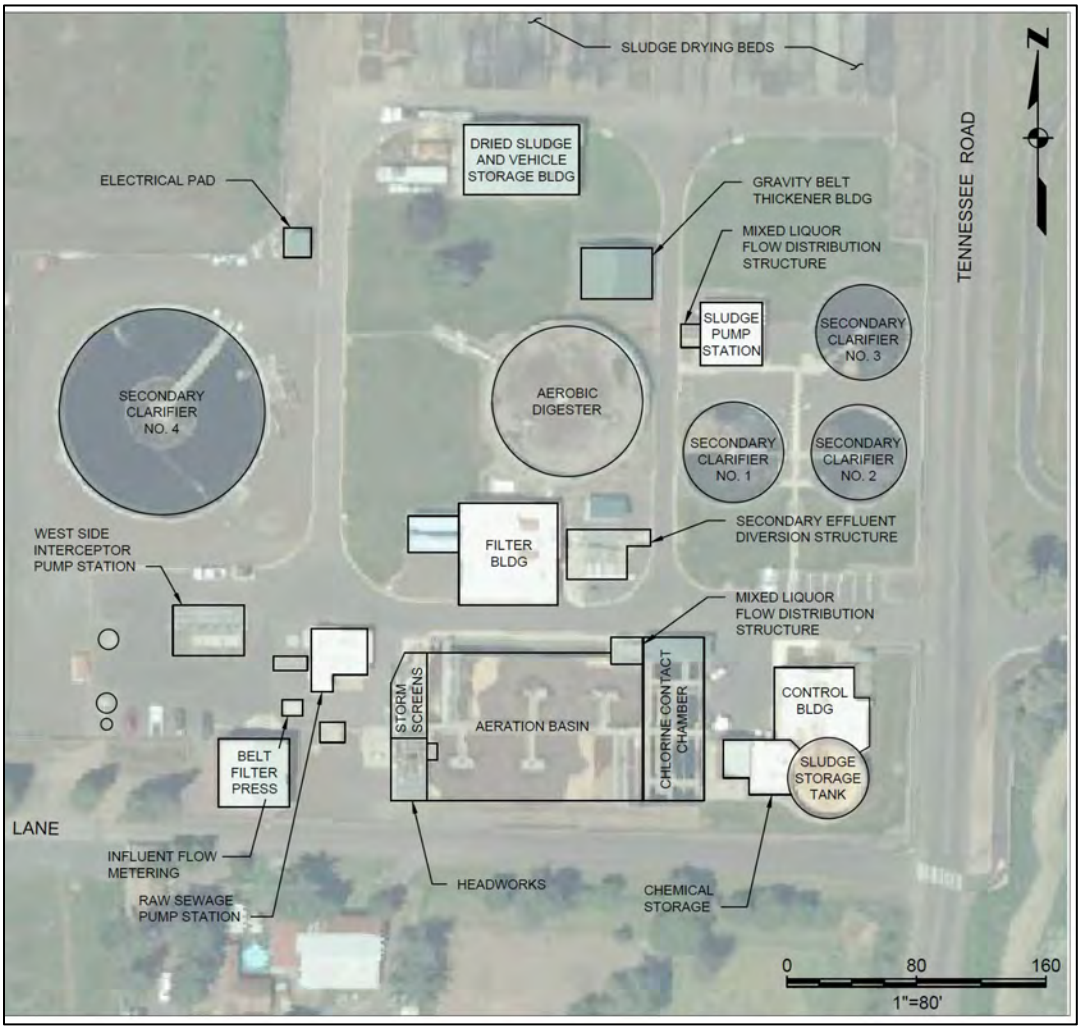

Figure 4-2: Existing Wastewater Treatment Plant Facilities

4.1 Liquid Stream Treatment Process

The following section describes the liquid stream processes currently active at the WWTP. A flow schematic of the existing liquid stream treatment processes is presented in **Figure 4-3** with descriptions of each unit process following.

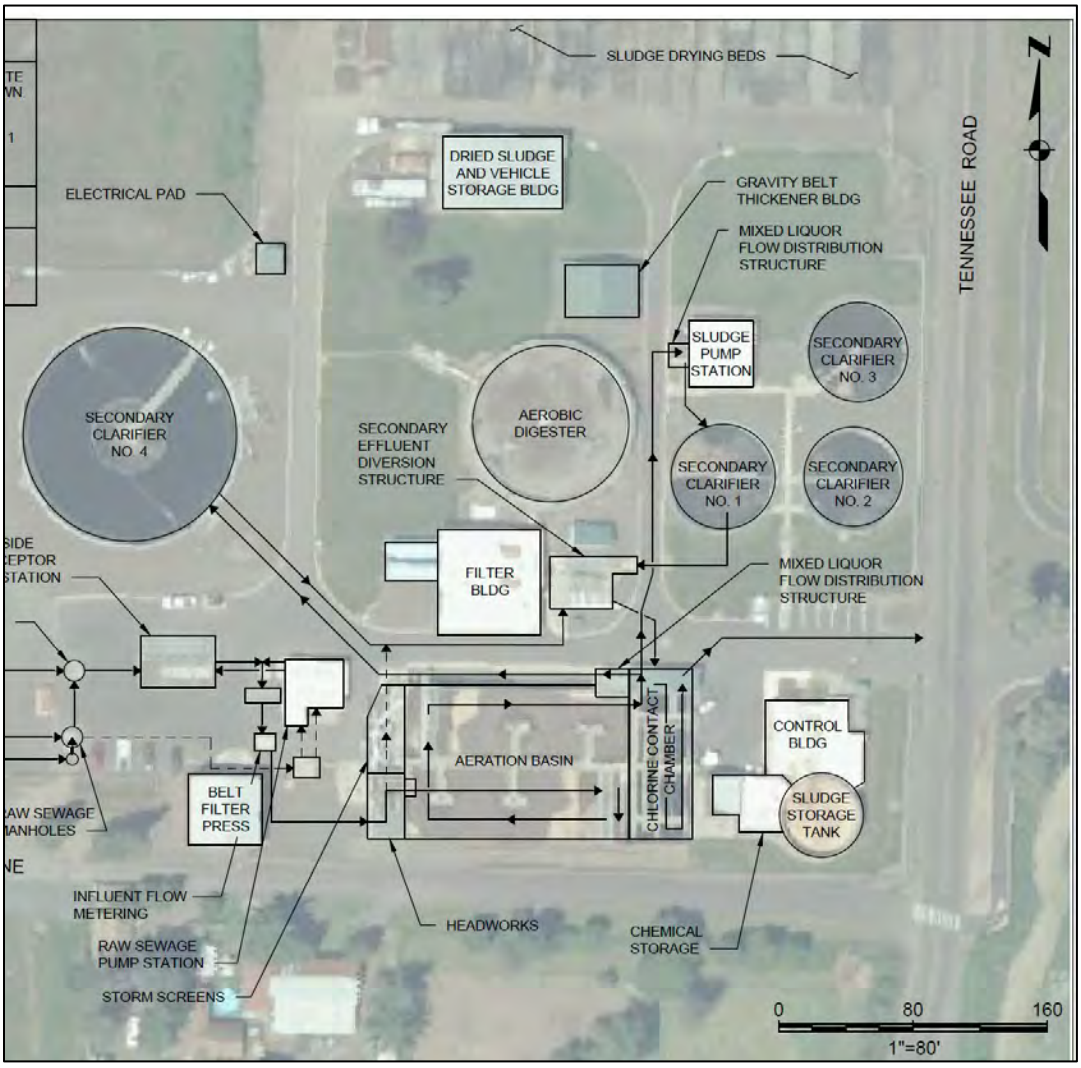

Figure 4-3: Liquid Stream Treatment Process Flow Schematic

4.1.1 Influent Pumping

Raw sewage from the WSI system enters the WWTP from the west via a 54-inch gravity pipeline and discharges to the 120-inch diameter WSI manhole. Flow continues to the west via another 54-inch line and into the WIPS. Raw sewage from the ESI system enters the WWTP from the west via parallel 24-inch and 27-inch pipelines that discharge to 60-inch and 120-inch diameter manholes, respectively. A 24-inch line connects the two manholes. Flow from the manholes is directed north via a 36-inch line into the WSI manhole and then to the WIPS.

The WIPS which was completed in 2003 and shown in **Figure 4-4**, is a variable speed, submersible, self-cleaning wet well-type pumping station with five available pump slots. The station design anticipated phased expansion to increase capacity. The current configuration installed as part of the 2003 project, includes three large (two 85-hp and one 75-hp) pumps and one smaller (25-hp) pump. The smaller pump functions as a jockey pump operating under low flow conditions. The station has a capacity of 16 MGD with two large pumps running. The station was designed for a maximum capacity of 37 MGD with the future installation of up to five larger (150-hp) pumps.

Figure 4-4: West Side Interceptor Pump Station

WIPS pumps discharge to a common 24-inch discharge header before being directed through one of two magnetic flow meters (10-inch and 20-inch). Meter selection is made by opening and closing 10-inch and 20-inch plug valves in an upstream vault. The 10-inch meter is used for flows up to 8 MGD. The 20-inch meter is used for flows greater than 8 MGD. The 10-inch and 20-inch lines combine into a 24-inch line that conveys flow to the headworks.

The older Raw Sewage pump station was constructed in 1976. It is a wet well/dry well type station, contains four pumps, and had an original design capacity of 26.8 MGD. The station has

two, 75-hp variable speed pumps; one, 25-hp variable speed pump, and one 50-hp constant speed pump. The pumps are located within the dry well on the bottom level of the pump station. Drive shafts connect the pumps to their respective motors that are located on the ground floor along with other station electrical equipment. The Raw Sewage Pump Station was meant to serve a backup for the WIPS during peak flow events exceeding the capacity of the WIPS. However, WWTP staff report that discharge piping from the station appears to have been plugged and the station is no longer operational.

4.1.2 Preliminary Treatment

The headworks was constructed in 1976 as part of a large common wall structure that included the headworks, aeration basins, storm flow diversion, and chlorine contact basins. Raw sewage enters the headworks through a 24-inch pipe and flows by gravity through one of three parallel channels. Originally one of the channels contained a comminutor with a manual bar screen in another. The third channel was provided for future expansion. As shown in **Figure 4-5**, the headworks now includes two mechanically cleaned bar screens that discharge to a screw auger. The first mechanical screen was installed in 1999 and replaced the comminutor. The second mechanical screen was installed in 2021. A manually raked bar screen provides additional redundancy. The headworks contains no grit removal.

Figure 4-5: Existing Headworks Screening Equipment

The flow rate through the headworks is limited to approximately 14 MGD. When flows exceed this amount, excess flow is diverted around the headworks via an overflow weir upstream of the bar screens and is directed through two rotary storm screens. The existing storm screens are

shown in **Figure 4-6**. Effluent from the storm screens is blended with secondary effluent before entering the chlorine contact basin for disinfection.

Figure 4-6: Existing Rotary Storm Screens

An automatic composite sampler draws from the influent channel immediately prior to screening. This is the sampling point for compliance with NPDES permit plant influent wastewater sampling.

4.1.3 Secondary Treatment

The WWTP's existing activated sludge treatment system was constructed in 1976 as part of a large common wall structure that included the two parallel aeration basins, headworks, storm flow diversion, and chlorine contact basins. The basins as originally constructed were only able to operate in parallel. Alternate operating modes (e.g., series operation, step feed, sludge reaeration, etc.) were not possible. Aeration was provided by six surface aerators, three per basin. The basins were operated as aerobic, complete-mix reactors.

The aeration basins were modified as part of the 2008 Cannibal® Project. The current configuration of the basins is shown in **Figure 4-7**. A series of baffle walls and weirs were added with vertical shaft mixers to create two unaerated selector zones in each aeration basin. Isolation gates were provided to allow multiple operating modes including series, parallel, sludge reaeration, and step feed. WWTP staff select the appropriate operating mode to maximize performance of the secondary treatment process in response to seasonal changes in influent flows and to facilitate routine maintenance. Four new, variable speed surface aerators replaced the original six aerators. Dissolved oxygen (DO) probes are used to measure DO in the two basins. DO is controlled by varying aerator speed and the number of aerators in operation.

Figure 4-7: Existing Aeration Basins

Following biological treatment, mixed liquor spills over a weir and into a common mixed liquor channel along the east end of the aeration basins before entering one of two flow control structures. The first flow structure is located at the east end of the north aeration basin and directs mixed liquor to Clarifier 4 using two electrically actuated weir gates. The second flow splitting structure is located on the west side of the Sludge Pump Station. Manually actuated weir gates are used to split and control flow between Clarifiers 1, 2, and 3. Coagulant (i.e., aluminum chlorohydrate) can be added to the mixed liquor channel to enhance performance of the secondary clarifiers.

The WWTP has four secondary clarifiers. Clarifier 1 was originally a 60-ft diameter, primary clarifier that was part of the original facility constructed in 1954. It was converted to a secondary clarifier during the 1976 upgrades project at which time two additional 55-ft clarifiers (Clarifier 2 and 3) were constructed. The three clarifiers shown in **Figure 4-8** have sidewall depths of 10 ft and were originally rated for a peak overflow rate of 8 MGD. They are equipped with scraper type sludge collector mechanisms that move settled sludge to central sumps where it is drawn off by the RAS pumps. There are four, recessed impeller-type, RAS pumps located in the Sludge Pump Station that serve Clarifiers 1, 2, and 3. Each clarifier has a dedicated RAS pump with the fourth pump providing redundancy. The pumps are operated at variable speed to control RAS return rates.

Figure 4-8: Secondary Clarifiers 1, 2, and 3

Clarifier 4 shown in **Figure 4-9** was constructed in 2012 and has an inside diameter of 130 feet with a sidewall depth of 16 feet. Clarifier 4 was designed for a peak wet weather flow of 17 MGD and is equipped with a suction-type sludge collector mechanism with two suction arms. RAS is pulled towards the center column and is conveyed beneath the clarifier floor through a 20-inch RAS pipe to the adjacent RAS Pump Station that contains three variable speed, submersible pumps. The WAS and scum pumping are provided by dedicated constant speed, submersible pumps located in separate wet wells located on the outer wall of the clarifier. Because of the age, condition, and overall poor performance of Clarifiers 1, 2, and 3, Clarifier 4 operates year-round with the smaller clarifiers being kept offline except during higher flow periods of the year.

Figure 4-9: Secondary Clarifier 4

4.1.4 Filtration

Tertiary filtration facilities were constructed as part of the 1976 upgrades project and consisted of two pressure type filter vessels, filter feed pumps, backwash pump, chemical feed systems, and the associated Filter Building. The filtration system was originally designed to operate between June 1 and October 31 to reduce effluent suspended solids. However, the system has not been operated for more than twenty years and was officially removed from service in 2020.

4.1.5 Secondary Effluent Pumping

Secondary effluent from the secondary clarifiers as well as flow diverted through the storm screens during high flow events are pumped to the chlorine contact basins by the Effluent Pump Station. The station was originally constructed in 1976 and later upgraded in 2015. Secondary effluent from Clarifiers 1 through 3 combine and flow through a Parshall flume before entering the station's wet well. Flow from the Storm Screens and secondary effluent from Clarifier 4 combine in a 30-inch diameter line before also discharging to the wet well. The wet well contains a dividing wall with weir that when the filters were operational allowed secondary effluent to be directed to the tertiary filters.

The Effluent Pump Station shown in **Figure 4-10** currently contains four, variable speed, submersible pumps that were installed in 2015. Electrical equipment for the pump station is housed in the small structure shown immediately to the north of the station. The pumps consist of two larger capacity pumps and two smaller capacity pumps. The pumps are installed within separate sections of the wet well that can be isolated from one another by manually operated

slide gates. Pump speed and number of pumps running are controlled to maintain preset water level within the station's wet well. The discharge piping from each pump combines into a single 24-inch pipe that conveys flow to the chlorine contact basin.

Figure 4-10: Effluent Pump Station

4.1.6 Disinfection

Sodium hypochlorite is used to disinfect secondary effluent prior to discharge to the South Santiam River. Secondary effluent from the Effluent Pump Station is distributed to the existing chlorine contact basins (CCBs) via a common 36-inch pipe header. There are two parallel CCBs that can be operated in series or in parallel. Sluice gates in the common header allow secondary effluent to be directed to either of the two existing CCBs. Additional sluice gates allow the CCBs to be operated in series or in parallel. Each basin can be independently removed from service for routine maintenance. The CCBs empty to a common discharge chamber with flow leaving the structure over a fixed weir. The CCBs are shown in **Figure 4-11**.

Figure 4-11: Chlorine Contact Basins

The original chlorine gas disinfection system was replaced with a sodium hypochlorite system in 1999. A single polyethylene sodium hypochlorite solution storage tank is located adjacent to the effluent pump station wet well. Sodium hypochlorite is pumped from the tank by two, flow-paced, peristaltic metering pumps. The sodium hypochlorite solution is injected into the Effluent Pump Station wet well. An in-line total chlorine residual analyzer measures chlorine residual within the Chlorine Contact Basins (CCB).

Sodium bisulfite is added to dechlorinate the final effluent downstream of the CCB at the fixed weir prior to discharge. The system is housed in the dichlorination building constructed as part of the 2008 effluent discharge system improvements. The dichlorination system consists of a polyethylene storage tank and two, flow-paced peristaltic metering pumps.

4.1.7 Final Effluent Discharge

After the final effluent weir, dechlorinated effluent enters a common 36-inch header and flow split structure where effluent sampling is taken for NPDES permit compliance. Dechlorinated effluent is conveyed to the South Santiam River through a 1,500 ft long, 42-inch diameter outfall constructed in 2008. Discharge to the river is via a multi-port diffuser buried beneath the bottom of the riverbed. The diffuser is approximately 24 ft long and consists of eleven, evenly spaced, 12-inch duckbill diffusers on 12-inch diameter HDPE risers and is located at river mile 17.4.

4.2 Solids Stream Treatment Process

The following section describes the solids treatment process at the WWTP which consists of WAS pumping, sludge thickening, aerobic digestion, dewatering, sludge drying beds, and Class B biosolids land application. A flow schematic of the existing solids stream treatment processes is presented in **Figure 4-12** with descriptions of each unit process following.

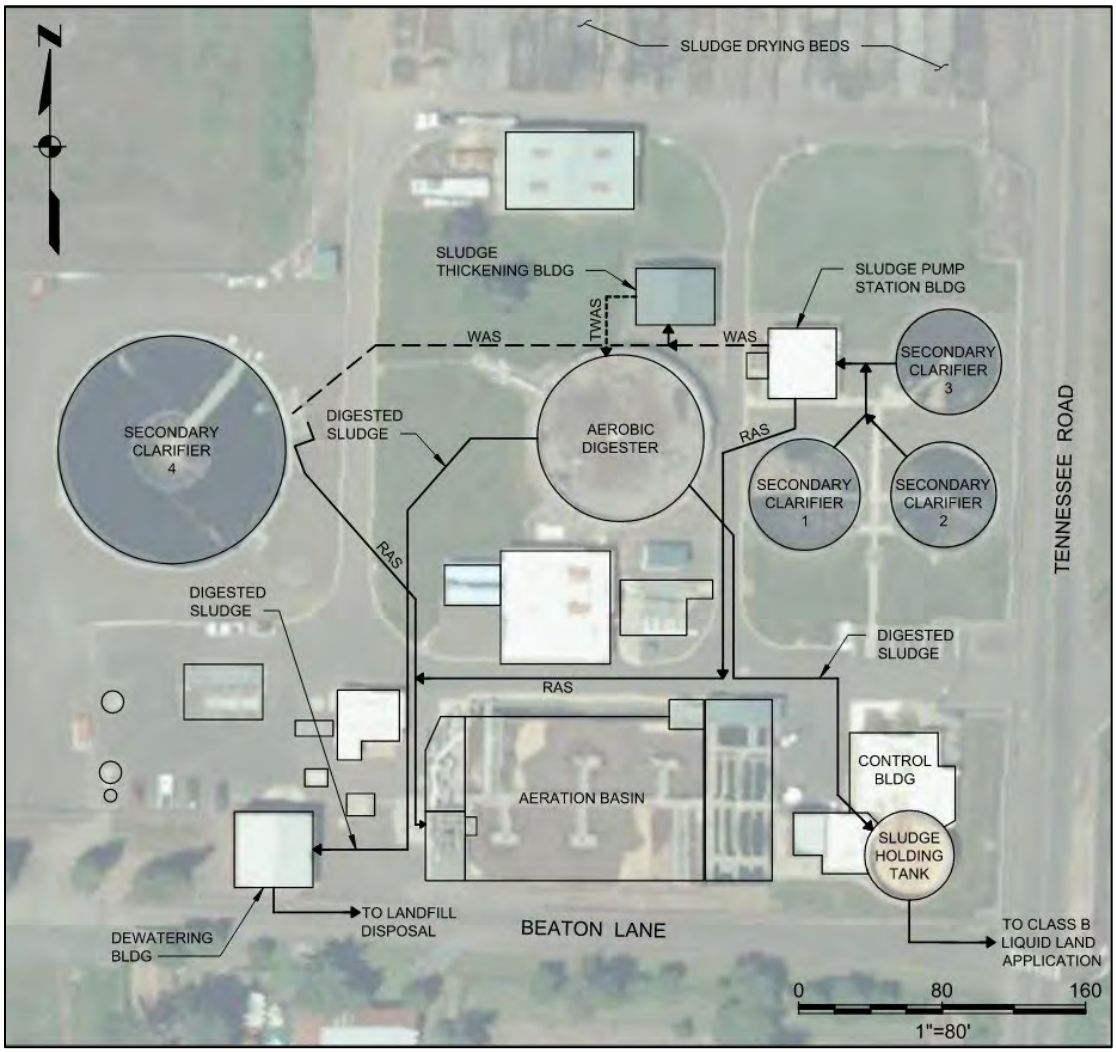

Figure 4-12: Solids Steam Schematic

4.2.1 Waste Activated Sludge Pumping

The WAS pumping facilities consist of one, horizontal, recessed impeller type pump located inside the Sludge Pump Station Building originally constructed in 1976. The pump, shown in **Figure 4-13**, draws from the RAS pump suction header serving secondary Clarifiers 1, 2, and 3. Valving allows the pump to direct WAS to the Sludge Thickening Building, sludge truck pump, or aerobic digester.

Secondary Clarifier 4 has one, submersible solids handling WAS pump within a wet well immediately adjacent to the clarifier. The WAS from Secondary Clarifier 4 is pumped directly to the Gravity Belt Thickener (GBT).

Figure 4-13: WAS Pump for Clarifiers 1,2, and 3

4.2.2 Sludge Thickening

Sludge thickening facilities were constructed in 1992 and consist of a GBT, sludge feed pump, sludge grinder, thickened sludge pump, and polymer feed equipment. Sludge thickening facilities are contained within the 1,400 square foot Sludge Thickening Building. The existing GBT is shown in **Figure 4-14.**

Figure 4-14: Gravity Belt Thickener

The WAS from the existing secondary clarifiers is pumped to the sludge feed pump inside the Sludge Thickening Building. The feed pump, which is a vertical, screw-type, centrifugal pump. conveys the WAS through a sludge grinder and then to the GBT. A liquid emulsion polymer feed system installed in 2020 injects polymer immediately downstream of the sludge grinder. Injecting polymer increases the efficiency of the GBT which although capable of producing sludge concentrations ranging from 3 to 8 percent, normally thickens sludge to between 4 and 5 percent. A progressive cavity pump pulls thickened sludge from the thickened sludge hopper and pumps it to the aerobic digester for stabilization. Filtrate from the GBT drains by gravity to the Drainage Pumping Station where it is pumped to the headworks. Aerobic Digestion

Aerobic digestion is used to stabilize wastewater solids from the liquid stream secondary treatment process. The WWTP's 530,000-gal aerobic digester was originally constructed as a trickling filter but was converted to an aerobic digester in 1976. In 2008, it was modified for use as an interchange reactor as part of the conversion to the Cannibal® solids reduction system. Because of poor performance, the Cannibal® system was decommissioned in 2020 and the interchange reactor was returned to use as aerobic digester when City staff took over operation of the WWTP.

The aerobic digester is shown in **Figure 4-15.** Aeration and mixing of digester contents is provided by three, floating aerators. The floating mixer shown in the center of the digester is no longer in service and has been removed.

Figure 4-15: Aerobic Digester

When hauled offsite for liquid land application, digested sludge is temporarily stored in a 265,000-gal sludge holding tank. The sludge holding tank is part of the Control Building structure and was originally an anerobic digester. It was converted for use as a sludge holding tank in 1976. A positive displacement blower provides mixing and aeration air. The sludge holding tank and Control Building are shown in **Figure 4-16**.

Figure 4-16: Sludge Holding Tank and Control Building

The existing aerobic digester does not provide adequate detention time to meet regulatory requirements for Class B biosolids. Because of this, lime stabilization is used by WWTP staff to produce Class B biosolids for land application. This is done by combining lime slurry with the partially stabilized biosolids in the sludge holding tank. After holding for at least the regulatory required minimum detention time, the resulting lime stabilized Class B liquid biosolids are pumped by a self-priming pump to the City's sludge truck for land application. The pump used for truck loading is also used as a sludge circulation pump.

4.2.3 Biosolids Dewatering

Biosolids dewatering facilities consist of a belt filter press (BFP), sludge feed pump, polymer addition system, wash water pump, and dewatered cake conveyor. Dewatering facilities were installed in 2022 inside a pre-engineered metal building originally constructed as part of the Cannibal® solids reduction project to house RAS screening facilities. When the Cannibal® process was decommissioned because of poor performance, the RAS screening equipment was removed from service and later replaced by the current dewatering equipment.

When operating, digested sludge flows by gravity from the aerobic digester to the Dewatering Building. The sludge is pumped into the flocculation tank and then into the gravity dewatering section of the BFP (see **Figure 4-17**) with the partially dewatered cake then being further dewatered within the pressure zone of the BFP. The dewatered cake drops into a shaftless

screw conveyor that deposits the sludge onto the floor of the Dewatering Building. City staff use a front-end loader to move the dewater cake into 20-cubic yard roll-off bins for disposal.

Liquid emulsion polymer is injected upstream of the flocculation tank resulting in the production of dewatered cake with typical solids concentration between 16 and 17%. A multi-stage vertical type pump provides high pressure wash water for belt cleaning. Filtrate from the BFP drains by gravity to the Drainage Pumping Station where it is pumped to the headworks.

Figure 4-17: Biosolids Dewatering Belt Filter Press

4.2.4 Sludge Drying Beds

The WWTP has 20 sludge drying beds (see **Figure 3-18**) with a total area of 37,000 square feet. They were originally constructed in 1976 and are in poor condition but do not leak. Because of their condition, the City uses the drying beds only for vactor waste. The vactor waste is disposed of in the drying beds and hauled to Coffin Butte or other sanitary landfill approximately every 2-3 years. The drying beds are evaporative and do not drain back to the WWTP.

Figure 4-18: Sludge Drying Beds

4.2.5 Beneficial Use and Disposal

The City produces aerobically, partially digested, lime stabilized Class B biosolids that are liquid land applied to five DEQ authorized application sites with a total combined area of approximately 368 acres. **Table 10-3** "Biosolids Land Application Site Inventory" in section 10.6.6 of this BSMP details authorized land application sites. However, of these five authorized sites, only two are currently in use both of which produce grass hay. One of these sites (Sommers) was originally authorized for year-round biosolids application and has a total area of 106 acres. However, this site is now authorized for dry season application and the usable acreage is approximately 40-60 acres. The City's 4,800-gal liquid haul truck transports biosolids to the field where a self-driven irrigation hose reel system applies the liquid biosolids or fitted for splash plate application.

The second site (Morgan) currently in use is only authorized for summertime application and has a total area of 68 acres with approximately 40 acres that are usable. Land application at the site typically begins in June or July and must be completed prior to the end of October unless specifically authorized by the DEQ. The City's haul truck is fitted with a splash plate and used to apply the liquid Class B biosolids directly to the field, as shown in **Figure 4-19.**

Figure 4-19: Class B Biosolids Liquid Land Application

In 2024, all 228 dry tons of biosolids produced by the WWTP were land applied (Lebanon 2024 Biosolids Annual Report). Because the City is currently only using two of the five authorized sites, the amount of biosolids that will be land applied is anticipated to decrease. The City may seek authorization for new sites as well as haul to the landfill. Any new land application sites would be in Linn County, OR.

Excess biosolids that cannot be land applied are dewatered and hauled for disposal at the Coffin Butte Landfill. The landfill is about 30 miles northwest of the WWTP in Benton County. It should be noted that the landfill operators, although seeking to expand, report there to be approximately 12.9 years of remaining capacity as of January 2023 (Coffin Butte Landfill and Pacific Region Compost Annual Report, 2023).

4.2.6 Pretreatment Program

The City implements a comprehensive pretreatment program codified in the City's, "Code of Ordinance, Title 13, Public Services, Chapter 13.05, Pretreatment." The ordinance includes prohibited discharge standards, categorical pretreatment standards, incorporation of State of Oregon pretreatment standards, a structure for local limits, pretreatment requirements for wastewater discharged or transported to the WWTP, a wastewater discharge permitting process, reporting, compliance tracking and an enforcement program.

Section 5: Biosolids Treatment Processes

In accordance with 40 CFR Part 503 and OAR 340-050, pathogen reduction and vector attraction reduction (VAR) for biosolids must be met prior to land application. VAR requirements can also be met at the time of land application if biosolids are injected below the surface of the land or incorporated into the soil within six hours after application to the land. Biosolids are categorized as Class A or Class B depending on the method used to determine pathogen reduction. Biosolids may also be classified as EQ if the product meets: pollutant concentration limits in 40 CFR Part 503, one of the Class A pathogen reduction alternatives in 40 CFR Part 503.32(a), and one of the vector attraction reduction options in 40 CFR Part 503.33(b)(1) through (8). To meet regulatory requirements, pathogen reduction must be met before or at the same time that vector attraction reduction is achieved.

The City produces aerobically, partially digested, lime stabilized Class B biosolids that are liquid land applied to five DEQ authorized application sites. The City's authorized land application sites have a total combined area of approximately 368 acres. The City's 4,800-gal liquid haul truck transports biosolids to the field where a self-driven irrigation hose reel system applies the liquid biosolids. Excess biosolids that cannot be land applied are dewatered and hauled for disposal at the Coffin Butte Landfill.

The City will notify DEQ in writing and get approval prior to any process change that would utilize pathogen reduction or VAR alternatives not specified in this BSMP. The City will also certify the alternatives used are EPA approved, and sampling and monitoring conforms to the 40 CFR 503 and OAR 340-050 regulations. This section describes the processes implemented by the City to achieve Class B biosolids criteria.

5.1 Class B Biosolids

For Class B biosolids production for land application, the City must comply with 40 CFR 503.32(b)(2), (b)(3), or (b)(4) with respect to pathogens. Additionally, one of the vector attraction reduction requirements must be met under 40 CFR 503.33(b)(1) through (b)(10) at the time of use. Pathogen reduction must occur at the same time or before VAR requirements are met. The City meets pathogen reduction and VAR requirements at the same time.

5.1.1 Pathogen Reduction

The City's WWTP meets pathogen reduction requirements for Class B biosolids by meeting the Process to Significantly Reduce Pathogens (PSRP) in accordance with 40 CFR Part 503.32(b)(3), Alternative 2. This process involves adding sufficient lime to the sewage sludge to raise the pH to 12 for ≥2 hours of contact.

When biosolids are applied to the land, the site restrictions in 40 CFR Part 503.32(b)(5) are also met. Additionally, the requirements of OAR 340-050-0060, 0065, 0070 and 0080, Best Management Practices for Site Selection and the Use and Application of Bulk Biosolids and Domestic Septage, are followed.

5.1.2 Vector Attraction Reduction

The City's WWTP meets the VAR requirements for Class B biosolids by following the method in accordance with 40 CFR Part 503.33(b)(6), Option 6. This process involves adding sufficient lime to raise the pH to at least 12 at 25ºC (77ºF) and maintain a pH ≥ 12 for 2 hours and a pH ≥ 11.5 for 22 more hours.

Section 6 of the BSMP describes the monitoring and sampling procedures used by the City to comply with requirements for pathogen reduction, VAR reduction, nutrients, pollutants (metals), and other parameters.

Section 6: Biosolids Monitoring and Characteristics

6.1 Biosolids Monitoring

The following describes the monitoring that occurs for the City's biosolids beneficial use program.

6.1.1 Monitoring and Sampling

The City has developed and implements a biosolids monitoring and sampling plan. Samples collected and analyzed will be representative of the biosolids to be land applied. Quality control measures and procedures will be implemented for microbiological tests to verify precision and accuracy. Sampling location(s) stated will demonstrate how vector attraction reduction option (40 CFR Part 503.33(b)(6), Option 6) is met. All monitoring and reporting will be conducted in accordance with the City's NPDES permit. The monitoring frequency is based on the amount of biosolids generated that are applied to the land. Based on 40 CFR §503.16, Table 1 and the amount of biosolids generated and used during 2024, the City is required to sample biosolids annually. However, the City may choose to sample more frequently for quality control purposes. The following monitoring and sampling plan describes the City's procedures for ensuring compliance for pathogen reduction, VAR reduction, nutrients, pollutants (metals), and other parameters:

- Lime slurry is delivered by Gilmour Lime Company in a tanker truck. The lime slurry consists of 10 tons of hydrated lime and mixed with 5,000 gallons of water.
- The sludge holding tank (see **Figure 4-16**) is prefilled with approximately 250,000 gallons of partially digested liquid solids from the aerobic digester. The truck pumps lime slurry directly into the sludge holding tank. The sludge holding tank has both a pump mix system and an aeration system to ensure complete mixing.
- After approximately 30 minutes of mixing, a sample of the limed biosolids is taken from a sample port on the discharge of the pump mix system.
- Assuming the pH is at least 12.0, this would be our zero reading and recorded (see **Table 6-1** below). If a pH of 12.0 is not obtained, the City waits an additional 30 minutes and repeats the sampling procedure. To date, the pH has always been at least 12.0 after 30 minutes.
- After 2 hours sampling is repeated and recorded. If the pH is not at least 12.0, the material is reprocessed through the aerobic digester. To date, the pH has always been at least 12.0 after 2 hours.
- After an additional 22 hours sampling is repeated and recorded. If the pH is not at least 11.5, the material is reprocessed through the aerobic digester. To date, the pH has always been at least 11.5 after 22 hours.

- If lime stabilization has failed, the City would order more lime (reduced calculated amount based on results of initial attempt) and reprocess to meet Pathogen and VAR reduction requirements.
- After achieving lime stabilization following the 22-hour process, samples are preserved in coolers/containers as required by a certified laboratory and transported to a certified laboratory for analysis of the required parameters, nutrients, and pollutants (metals).
- Following confirmation from the laboratory that the biosolids meet all Class B requirements, they are hauled to authorized sites for land application.
- No biosolids are removed from the sludge holding tank for land application until they have been lime stabilized and all pH readings are adjusted for temperature compensation to confirm that regulations have been met.

Table 6-1: Biosolids Lime Stabilization Sheet

Biosolids Lime Stabilization Worksheet

Date _____ Biosolids Source _____ Biosolids %TS _____

Biosolids Gal. _____ Lime Slurry Gal. _____ Lime Lbs _____

Hour "ZERO"				Hour "TWO"				Hour "TWENTY-FOUR"				
Time	Temp	pH	Operator	Time	Temp	pH	Operator	Date	Temp	Time	pH	Operator
	>77	>12			>77	>12			>77		>11.5	

Notes:

pH must be above 12 before the 2 hour timer starts

40 CFR Part 503.33 Vector attraction reduction Alternative 6 - (503.33(b)(6)) The pH of sewage sludge shall be raised to 12 or higher by alkali addition and, without the addition of more alkali, shall remain at 12 or higher for two hours and then at 11.5 or higher for an additional 22 hours.

Meet pH and time criteria for Vector Attraction Reduction, also meet (PSRP) Process to Significantly Reduce Pahtogens

If temperature is below 77 ^{0}F (25 ^{0}C), use Correction Factor (see below) to adjust pH for the lower tempature.

$$Correction\ Factor = \frac{0.03\ pH\ units\ X\ (Tmeas - 25°C)}{1.0°C}$$

Where T_{meas} = the measured temperature in degrees centigrade

Actual pH = Measured pH +/- the Correction factor

Table 6-2 lists the various analytical methodologies used by the certified laboratories for the City's biosolids parameters. The methodologies are subject to change based on DEQ or EPA published requirements.

Table 6-2: Biosolids Analytical Methodologies

Parameter [(a)(b)]	Analytical Method [(c)]
Total Solids	EPA 160.3
Volatile Solids	EPA 160.3
Fecal Coliform	SM 9221 E
pH	EPA 9040C
TKN	EPA 351.3
NH_4-N	EPA 350.2
NO_3-N	EPA 300.0
P	EPA 365.3
K	EPA 6010 B ICP
As	EPA 6010 B ICP
Cd	EPA 6010 B ICP
Cu	EPA 6010 B ICP
Pb	EPA 6010 B ICP
Hg	EPA 7471 A
Mo	EPA 6010 B ICP
Ni	EPA 6010 B ICP
Se	EPA 6010 B ICP
Zn	EPA 6010 B ICP

Notes:

(a) EPA = Environmental Protection Agency
(b) SM = Standard Method
(c) ICP = Inductively Coupled Plasma spectrometer

The frequency of monitoring is based on 40 CFR Part 503.16, Table 1 and depends on the quantity of biosolids generated, prepared, or land applied by a wastewater treatment facility or land applier [OAR 340-050-0035(2)(c)]. Table 1 of 40 CFR Part 503.16 contains requirements for monitoring frequency of pollutants (metals) listed in Tables 1 and 2 of Part 503.13; the pathogen density requirements in Part 503.32(b)(2); and the vector attraction reduction requirements in Part 503.33 (b)(1), (b)(4), (b)(9) or (b)(10). The minimum monitoring frequency is based on the amount of biosolids generated and used (see **Table 6-1**).

Table 6-4: Biosolids Monitoring Frequency 40 CFR Part 503.16, Table 1

Amount of Biosolids Generated and Used		Monitoring Frequency
Dry weight - metric tons/year	**Tons/year**	
> zero but < 290	> zero but < 319	Annually
≥ 290 but <1,500	≥ 319 but < 1,650	Quarterly
≥ 1,500 but <15,000	≥ 1,650 but < 16,500	Bimonthly
≥ 15,000	≥ 16,500	Monthly

6.2 Biosolids Quality

Table 6-5 summarizes pollutants, nutrients, and physical parameters from the City's 2024 Biosolids Annual Report. Biosolids from the WWTP met the concentration limits for pollutants set forth in Table 3 of 40 CFR Part 503.13(b). These are commonly referred to as "low metals" limits and are the values that must be met for Class B biosolids to be beneficially used.

Table 6-5: Biosolids Quality Data from 2024

Parameter [(a)(b)(c)(d)]	Q1	Q2	Q3	Q4	Table 3 of 40 CFR Part 503.13
Solids, %	3.4	4.1	4.3	No Biosolids Land Application in Q4 of 2024	---[(e)]
pH	12	12	13		---
TKN, %	4.3	4.6	5		---
NH_4-N, %	0.99	0.12	0.21		---
NO_3-N, %	0.91	ND	ND		---
P, %	2.2	ND	2		---
K, %	1	0.8	1		---
As, mg/kg	4.56	3.67	3.44		41
Cd, mg/kg	0.853	0.731	0.591		39
Cu, mg/kg	249	243	264		1500
Pb, mg/kg	8.73	7.11	8.22		300
Hg, mg/kg	0.146	0.224	0.0917		17
Mo, mg/kg	13.8	8.97	12.3		75[(f)]
Ni, mg/kg	10.5	10.2	10		420
Se, mg/kg	6.37	3.79	5.93		100
Zn, mg/kg	539	394	387		2800

Notes:

(a) All values are arithmetic means
(b) All concentrations are on a dry weight basis
(c) ND = concentration was not detected above the indicated method detection limit.
(d) Federal limits for Fecal Coliform are not contained in Table 3 of 40 CFR Part 503.13(b)
(e) --- = no federal or state regulatory limits for this parameter
(f) Molybdenum value is the Ceiling Limit per Table 1 of 40 CFR Part 503.13(b)

Section 7: Biosolids Storage

All aerobically, partially digested, lime stabilized Class B biosolids are transported to the City's DEQ-authorized land application sites for beneficial use. Additionally, dewatered biosolids are hauled to the Coffin Butte Landfill for disposal when land application is not feasible (e.g., emergency or bad weather).

The City land applied approximately 228 dry tons (683,040 gallons) to two sites totaling 80 acres of grass seed fields in 2024. The liquid biosolids averaged 3.9% solids content.

7.1 Treatment Facility

The City can haul liquid or dewatered Class B biosolids to five DEQ-authorized land application sites for dry season land application. The primary method of land application is for liquid Class B biosolids.

The aerobic digester is designed with a total volume of 530,000 gallons to accommodate approximately 17 days of storage at any given period during the year. This is based on 2024 biosolids production rates. During periods that land application is not permitted, partially digested biosolids are dewatered from the aerobic digester and hauled to the landfill for disposal.

7.2 Staging

The unloading and placement of dewatered biosolids in one area at a land application site may occur on a limited time basis. If staging of biosolids occurs, the requirements outlined in the site authorization letters for each site will be followed.

7.3 Field Storage

Field storage is not authorized for any of the City's land application sites.

Section 8: Transportation

The City of Lebanon owns a 2019 Kenworth haul truck with 4,800-gal capacity to transport liquid biosolids from the WWTP to DEQ-authorized land application sites for beneficial use. The haul truck transports biosolids to the field where a self-driven irrigation hose reel system applies the liquid biosolids or fitted for splash plate application.

Section 9: Remedial Procedures

The following section describes procedures for biosolids spills and solids process failures.

9.1 Solids Treatment Process Failure or Modification

If a mechanical problem occurs with the digesters or other biosolids treatment components and replacement parts are not in stock at the treatment facility, an emergency parts order is placed. During this period, treatment, storage, handling, and transportation activities are modified to ensure that pathogen reduction, vector attraction reduction, and pollutant concentration limits do not result in non-compliance with the requirements of CFR Part 503 and OAR 340-050.

Procedures for properly maintaining, operating, and addressing repairs of the City's treatment processes are outlined in operation and maintenance manuals located at the WWTP.

9.2 Odor Response

In the event of odor complaints at the WWTP, City staff speak or meet with complainants to understand the specific date, location, and nature of the incident. Odor complaints rarely occur at the WWTP and the following procedure is implemented to address these complaints.

City staff speak with the complainant by phone or in person. Information is gathered from the complainant such as their name and address; date, time and location of the odor event; and any other pertinent site information. If the odor complaint is found to be the result of an acute issue at the WWTP (e.g., loading of biosolids) then staff determine if mitigation measures can be employed (e.g., timing of process) to reduce the potential for an odor event. The incident is documented in writing and maintained at the City's WWTP.

9.3 Spill Response Plan

In the event biosolids or dewatered sludge or are spilled between the treatment facility and the land application or disposal sites the driver notifies the City. The City assists the contracted hauling company to contain the spill, lime, absorb (using sand) and remove the spilled dewatered sludge or biosolids with a front-end loader or shovels and dispose of the spillage at an authorized land application or disposal site. All spills into waters of the State or spills on the ground surface that are likely to enter waters of the State shall be reported immediately to the Oregon Emergency Response System (OERS) at 1-800-452-0311 and the regional DEQ biosolids coordinator at 541-686-7866.

Any amount of dewatered sludge or biosolids spilled during transport will be immediately reported to the transporting company project manager and the City of Lebanon WWTP.

Wastewater Treatment Plant Office:	541-258-4991
City of Lebanon Public Works:	541-258-4918
City of Lebanon, City Hall:	541-258-4900
City of Lebanon Police Department:	541-451-1751

9.3.1 Spill Response Plan Process

If the spill can be contained and properly cleaned-up by the driver:

Driver's Responsibilities:

Step 1 Clean-up spill.

Step 2 Immediately report to transport company project manager.

Step 3 Continue on trip if possible, without spilling more dewatered sludge or biosolids.

Step 4 Complete an incident report upon the completion of the truck trip and submit the completed report the City of Lebanon WWTP.

If the spill is greater than is practical for the driver to clean-up:

Drivers Responsibilities:

Step 1 Immediately report spill and amount to transport company project manager and their office.

Step 2 Remain with vehicle and warn all pedestrians and motorists to stay away from the spill area. Indicate traffic hazard involved and call 911 to notify police and/or fire department.

Step 3 Upon arrival of police or fire department, the driver will inform them of the nature of material spilled and request the area be blocked-off to both pedestrians and vehicles to prevent property damage or any serious personal injury.

Step 4 Driver will request police or fire department personnel to protect area while driver reports to the transport company's office and the City of Lebanon.

Step 5 Complete an incident report and submit to the City of Lebanon WWTP.

Transport Company's Responsibilities:

Step 1 Transport company will notify their project manager.

Step 2 Contact a "Spill Response Company" if necessary.

Step 3 Contact the City of Lebanon WWTP.

Step 4 Contact DEQ within an hour of spill incident, if unable to contact the City of Lebanon within 30 minutes of incident at DEQ Regional Office and OERS.

Step 5 Submit completed spill report to the City of Lebanon WWTP.

City of Lebanon Responsibilities:

Step 1 City of Lebanon WWTP is contacted

Step 2 Contact DEQ Regional Office and/or OERS

Step 3 Contact Public Works Director and/or City Manager

Step 4 In most cases City of Lebanon WWTP personnel will report to spill location to inspect incident and ensure proper cleanup

Step 5 Completed spill report is shared with affected departments

City of Lebanon Responder's Responsibilities:

Step 1 Contain spill if it has not been done.

Step 2 Cleanup: If the spill has been contained on an impervious paved surface, material should be scrapped from the surface and loaded with flat edged shovel or backhoe.

Step 3 Any material remaining on paved surfaces should be absorbed onto compatible material (i.e., sand or dirt). If the spillage has reached earth, all affected soil should be collected and/or placed into dump truck for return to plant or taken to farm site.

Step 4 If material is contaminated with fuels, oils, or other debris which precludes land application, biosolids are to be hauled to a DEQ approved site for disposal.

9.4 Hauler Spill Response Form

A. SPILL DESCRIPTION			
Date spill occurred:		Time spill occurred:	
Who discovered the spill:			
Description of spill incident:			
Factors that may have contributed to spill (e.g., equipment failure, road conditions):			
Estimated Amount of Material Spilled:			
B. NOTIFICATIONS			
Agency	Time Notified	Contact Person's Name	Who Performed Notification
911			
Transport Company			
City of Lebanon			
DEQ			
OERS			
WWTP			
Other			
Return completed form to: Steve Haney, Water/Wastewater Treatment Plant Manager, City of Lebanon, 33110 Tennessee Road, Lebanon, Oregon 97355			

9.5 Lebanon Staff Spill Response Form

C. POTENTIAL IMPACTS OF SPILL
Injuries:
Environmental:
Health:
Other:
D. CLEAN-UP ACTIONS
Who performed the clean-up?
Describe clean-up activities and results:
Where was spill material disposed:
Date/Time clean-up was completed:
Name of person completing this report form:
Date/Time this spill report form was completed:
Return completed form to: Steve Haney, Water/Wastewater Treatment Plant Manager, City of Lebanon, 33110 Tennessee Road, Lebanon, Oregon 97355

9.6 Guidelines for Driver Protection

The City's Class B biosolids present little threat to hauler health and safety, but the potential exists for disease-causing microorganisms to remain in solids transported from the WWTP to land application sites or the landfill.

The following safety practices shall be observed to minimize exposure:

- Wash hands before eating, drinking, or smoking.
- Use waterless disinfectant soap for washing at sites where water is not available.
- Avoid rubbing eyes, nose, and mouth after handling.
- Do not smoke, eat, or drink in loading and unloading areas.
- Wear gloves during loading and off-loading operations.
- Wear protective clothing when there will be more than casual contact with material (e.g., during the clean-up of spills).
- When clothing or body parts are exposed to the material, shower and change into clean clothes before leaving work.
- Immediately clean and disinfect cuts or scrapes.
- Keep all wounds protected from contamination.

Section 10: Biosolids Utilization Program

The following describes the City's biosolids utilization program.

10.1 Biosolids Land Application Plan

Class B biosolids will be land applied on DEQ-authorized agricultural fields in accordance with 40 CFR Part 503 and OAR 340-050. The beneficial use practices are governed under the City's DEQ approved Biosolids Management Plan, Biosolids Site Authorizations and NPDES permit. The City will transport Class B biosolids from the WWTP and land apply liquid or dewatered cake biosolids on the DEQ authorized sites.

The City currently land applies Class B biosolids to the DEQ authorized sites listed **Table 10-1** below. The City currently has 368 acres that are authorized by DEQ for biosolids land application.

Table 10-1: Biosolids Land Application Site Inventory

Site Name/Identifier	Type of Crop/Acres	lb. N/acre	Time of year applied	Harvest Cycle	DEQ Authorized?
Sommers, R1W, Sec. 11, TL13S Linn County	Grass Seed / 106	100	May - October	Once per Year	Yes
Morgan, T11S, R1W, Sec. 19, TL 3000, W.M, Linn County	Grass Hay / 68	100	May - October	Once per Year	Yes
Koos Field 12, ,Township 12S, R3W, Koos 12 Sec. 1 Albany	Grass Seed / 58	100	May - October	Once per Year	Yes
Koos Field 14, Township 12S, R2W, Sec. 8, TL 600	Grass Seed / 66	100	May - October	Once per Year	Yes
Koos Field 15, Township 12S, R2W, Sec. 8, TL 600	Grass Seed / 70	100	May - October	Once per Year	Yes

The City's biosolids land application program consists of several key elements, including:

- Producing biosolids that meet pathogen reduction, vector attraction reduction, and pollutant concentrations in accordance with 40 CFR Part 503 and OAR 340-50 (these elements are explained in detail in Section 5 of this BSMP).
- Obtaining biosolids site authorizations from DEQ. This includes selecting sites using, at a minimum, criteria specified in OAR 340-050.
- Determining appropriate agronomic rate calculations based on crop nitrogen requirements. This includes implementing a soil sampling protocol to assist with these calculations (agronomic rate calculation methodology is explained in detail within this section of the BSMP).
- Implementing site management practices to control potential biosolids run-off, buffers to wells, public access restriction, and appropriate public notification.
- Management of the Class B biosolids application process shall conform to the requirements of 40 CFR 503.14, which are summarized as follows:
    - Land application shall not adversely affect threatened or endangered species.
    - Land application shall not take place on agricultural land that is flooded, frozen so that the biosolids enters a wetland or other water.
    - Land application shall not be applied to agricultural land that is 10 meters or less from water.
    - Land application shall be equal to or less than the agronomic rate for biosolids.
- Site restrictions associated with the land application of Class B biosolids shall conform to 40 CFR 503.32(b)5. Turf crops (if City ever applies to this type of crop) shall not be harvested for one year after application when the harvested turf is place on land with a high potential for public exposure or a lawn.
- The volume of biosolids, solids concentration, nutrient and pollutant data shall be reported for each hauling event in the DEQ Biosolids Annual Report.

10.2 Site Authorizations

A DEQ Site Authorization Letter outlines site management conditions based on the characteristics of the biosolids being land applied and the specific location where land application will occur. The letter is considered an integral part of the BSMP, and the provisions stated in the letter are enforceable conditions under the City's NPDES permit.

When the City wants to apply Class B biosolids to a new site, the following steps are followed prior to commencing land application:

1) Submission of a site authorization request with supporting documentation specified in DEQ's, "Site Authorization Documentation Checklist for the Land Application of Biosolids," and data is to be provided to the appropriate DEQ regional biosolids specialist.
2) Documentation that all newly proposed land application sites meet relevant site selection criteria that have been identified in the approved biosolids management plan.

3) Documentation the public notification process for proposed biosolids land application activity is in place. The process includes to whom notification was made and when.
4) Documentation regarding notification and the process is kept on file and available upon request from the DEQ.
5) Receipt of written site authorization from the DEQ.

10.3 Site Selection Criteria for New Land Application Sites

At a minimum and in accordance with OAR 340-050, the following site selection criteria will be used by the City when selecting new biosolids land application sites:

a) Sites will be on a stable geologic formation not subject to flooding or excessive runoff from adjacent land. If periodic flooding cannot be avoided, the period of application will be restricted, and soil incorporation may be implemented.

b) The minimum depth to permanent groundwater will be four feet and the minimum depth to temporary groundwater will be one foot.

c) Topography of the site will be suitable to allow normal agricultural operations. Where needed, runoff and erosion control measures will be constructed. In general, liquid biosolids will not be surface applied on bare soils where the ground slope exceeds 12% unless otherwise approved by DEQ. Well vegetated sites with slopes up to 30% may be used for dewatered biosolids, or for liquid biosolids application with appropriate management to prevent runoff.

d) Soil will have a minimum rooting depth of 24 inches. The underlying substratum to at least 24 inches will not be rapidly draining so that leachate will not be short circuited to groundwater.

e) Sites with saline and/or sodic soils will be avoided.

f) Buffer strips large enough to prevent nuisance odors or wind drift will be implemented. Approval of buffer strip sizes will be determined by DEQ through the site authorization process and depend upon the method of application used, total solids content, and proximity to sensitive areas, for example:

- Direct injection: no limit required
- Truck spreading (liquid): 0 to 200 feet
- Spray irrigation: 50 to 500 feet
- Cake or dried solids: 0 to 50 feet.
- Buffer strips will be provided along well traveled highways.

g) No Class B biosolids will be spread at a land application site closer than 50 feet to any ditch, channel, pond or waterway or within 200 feet of a domestic water source or well.

10.4 Public Notification

The City is required to notify the public of a proposed Class B biosolids land application activity in accordance with 40 CFR Part 122. Prior to land application of biosolids, the City will notify neighbors adjacent to existing authorized land application sites regarding expected biosolids application activities to occur at the authorized sites. The purpose of this notification is to ensure that any new residents are aware of biosolids land application activity near their property. The City will also conduct this notification process for any new biosolids land application sites during the DEQ site authorization process.

10.5 Site Management Practices

Site access restrictions and setbacks will be followed as outlined in DEQ site authorization letters. The City will ensure that access is restricted by appropriate means, as necessary, such as fencing or posting of signs at the land application site. However, in accordance with OAR 340-050-0065, access control is assumed on rural private land. Biosolids land application will not occur in those areas designated as buffer strips and will be achieved through accurate measurement of the buffer area prior to commencing land application.

Odor complaints addressed to the City will be responded to within 48 hours. DEQ will be copied on any associated correspondence or communications.

10.6 Agronomic Application Rates and Site Crops

10.6.1 Introduction

Class B biosolids are required to be land applied to a site at a rate that is equal to or less than the agronomic rate for the site. An agronomic rate is the whole biosolids application rate (dry weight basis) designed to provide the annual total amount of nitrogen needed by a crop and to minimize the amount of nitrogen passing below the root zone of the crop or vegetation to groundwater.

10.6.2 Nitrogen

Biosolids nitrogen is present in two forms, inorganic and organic. Inorganic or mineral nitrogen (as ammonium (NH_4) and nitrate (NO_3) is typically immediately available for plant uptake, while organic nitrogen must first be biologically transformed (mineralized). The concentration of various nitrogen forms in biosolids, plus the rate of organic nitrogen mineralization, is used to determine the annual agronomic loading of biosolids. Mineralization rates vary depending on the stabilization process and duration, application method, soil temperature and moisture conditions, and season.

Table 10-2 shows estimated organic nitrogen mineralization rates for the first year after biosolids application, *[(Worksheet for Calculating Biosolids Application Rates in Agriculture, Revised 2023 (WSU, OSU, and University of Idaho extension agencies, PNW 511)].* The range of values listed allows the City flexibility to decide on a mineralization rate based on WWTP specific data. Mineralization rates for lagoon stabilization will be used for agronomic rate calculations implemented by the City.

Table 10-2: Mineralization Rate Estimates for Organic Nitrogen in Biosolids

Biosolids Processing	First-year organic N mineralization rate (%)
Fresh[(a)]	
Anaerobic digestion, liquid or dewatered	30–40
Aerobic digestion, liquid or dewatered	30–40
Drying bed	30–40
Heat-dried	30–40
Lagoon	
Less than 6 months	30–40
6 months to 2 years	20–25
2 to 10 years	10–20
More than 10 years	5–10
Composting	0–10
Blends and soil products	(b)

Notes:

(a) Fresh includes all biosolids that have not been stabilized by long-term storage in lagoons or composting.
(b) Because blends (with woody materials) and soil products that contain biosolids vary widely in composition and age depending on intended use, available N may vary widely among products. For blends, available N can be estimated through laboratory incubation studies.

Ammonia volatilization and denitrification are active processes affecting nitrogen availability. Ammonia loss through volatilization is affected by soil pH, soil moisture, and the land application method and rate. For example, the amount of ammonium-nitrogen (NH_4) retained through surface application of dewatered biosolids not immediately tilled into the soil is estimated to be 50% *[(Worksheet for Calculating Biosolids Application Rates in Agriculture, Revised 2023 (WSU, OSU, and University of Idaho extension agencies, PNW 511)].* If biosolids are immediately tilled in (e.g., through soil incorporation or injection) then the maximum amount of NH_4 is retained. Estimate values of NH_4 retained after land application may be found in Table 2 *[(Worksheet for Calculating Biosolids Application Rates in Agriculture, Revised 2023 (WSU, OSU, and University of Idaho extension agencies, PNW 511)].* Denitrification losses may vary widely depending on soil temperature and moisture conditions. These losses are rarely factored into agricultural application rate calculations due to variability, although they may factor into biosolids application on forest sites.

10.6.3 Soil Sampling

Prior to the initiation of biosolids application to a site (pre- or post-harvest), a representative soil sample will be collected across the entire site and analyzed by an independent commercial laboratory. The benefits of conducting soil tests for agricultural programs are to:

- Assess pre-planting soil nutrient levels
- Monitor during the growing season for optimum crop yields

- Verify plant uptake and low residual soil nitrate-nitrogen (NO_3) levels after the growing season.
- Samples will be collected in the soil profile between 0-12 inches and 12-24 inches (the plant rooting zone) for these parameters: NH_4, NO_3, Total Kjeldahl Nitrogen (TKN), P and pH.

Available nitrogen concentrations in soil for a given season can be affected by several elements, including the following: other sources of nitrogen; soil pH; rainfall; and carry-over from previous year's applications or crop residuals. Existing nitrogen levels in the soil profile are subtracted from the OSU Fertilizer Guides application rates for the crop and the biosolids application rate is adjusted.

In the event of annual biosolids application to the same field for three consecutive years, annual sampling and testing of application site soils for NO_3 and NH_4 will be conducted prior to biosolids application [OAR 340-050-0080(5)(a)]. Application rates must be adjusted to account for available nitrogen carried over from previous biosolids applications. This helps to utilize residual nitrogen for crop uptake and also protect groundwater from migration of nitrogen through the soil profile.

Table 10-3 is an *example* soil sample testing schedule based on land application of Class B biosolids to a single agricultural site. If biosolids application events become more frequent, the City will incorporate available nitrogen carryover from previous applications in the updated BSMP.

Table 10-3: Example Soil Sample Testing Schedule

Year[(a)]	Biosolids Applied	Soil Testing Required
Year 1	No	No
Year 2	Yes	Yes (summer of Year 2)
Year 3	No	No
Year 4	No	No
Year 5	No	No
Year 6	Yes	Yes (summer of Year 6)
Year 7	No	No
Year 8	No	No

Notes:

(a) Adapted from DEQ's Implementing Oregon's Biosolids Program IMD, December 2005.

The following regional university and, federal and state agency references are used by the City to assist with soil sampling procedures (including the most recent additions):

- *[(Worksheet for Calculating Biosolids Application Rates in Agriculture, Revised 2023 (WSU, OSU, and University of Idaho extension agencies, PNW 511)].*

https://extension.oregonstate.edu/catalog/pub/pnw-511-worksheet-calculating-biosolids-application-rates-agriculture

- *Fertilizing with Biosolids,* Revised April 2022, Pacific Northwest Extension publication PNW 508. https://extension.oregonstate.edu/catalog/pub/pnw-508-fertilizing-biosolids

- *Monitoring Soil Nutrients Using a Management Unit Approach*, October 2003, Pacific Northwest Extension publication PNW 570. https://extension.oregonstate.edu/catalog/pub/pnw-570-monitoring-soil-nutrients-using-management-unit-approach

- *Postharvest Soil Nitrate Testing for Manured Grass and Silage Corn (West of the Cascades)* - fourth section on detailed suggestions for soil sampling and planning, Revised March 2023, OSU Extension Service publication EM 8832~~-E~~. https://extension.oregonstate.edu/catalog/pub/em-8832-postharvest-soil-nitrate-testing-manured-grass-silage-corn-west-cascades

- *Get Actionable Results from a Soil, Plant or Environmental Testing Lab* – Find an accredited laboratory, Revised May 2024, OSU Extension Service publication EM 8677. https://catalog.extension.oregonstate.edu/em8677

- *Soil Test Interpretation Guide*, Revised July 2019, OSU Extension Service publication EC 1478. https://catalog.extension.oregonstate.edu/ec1478/html

10.6.4 Agronomic Application Rate Methodology

Biosolids agronomic application rates for the City's sites will be developed based on OSU Extension Service Fertilizer Guides for the specific crops receiving biosolids. The agronomic rate is typically not based on the maximum crop yield for the site. The City's agronomic rate calculation method will be based on, *Worksheet for Calculating Biosolids Application Rates in Agriculture, Revised 2023 (WSU, OSU, and University of Idaho extension agencies, PNW 511).* Specific site agronomic loading rates will be stated in the DEQ issued site authorization letters.

The following regional university and, federal and state agency references are used by the City to assist with agronomic application rate calculations (including the most recent additions):

- *Worksheet for Calculating Biosolids Application Rates in Agriculture*, Revised 2023, WSU, OSU, and University of Idaho extension agencies, Pacific Northwest Extension publication PNW 511. https://extension.oregonstate.edu/catalog/pub/pnw-511-worksheet-calculating-biosolids-application-rates-agriculture

- *Fertilizing with Biosolids*, Revised April 2022. WSU, OSU, and University of Idaho extension agencies, Pacific Northwest Extension publication PNW 508. https://extension.oregonstate.edu/catalog/pub/pnw-508-fertilizing-biosolids

10.6.5 Site Crops

The City will land apply to crops that benefit from the nutrient and organic matter content of the Class B biosolids. Crops may typically consist of grass pasture, grass hay, and small grains. Other crops may receive biosolids based on customer and City staff approval and in accordance with 40 CFR Part 503 and OAR 340-050 requirements.

Crop nitrogen fertilizer requirements will vary from field to field and year to year depending on soil fertility, crop productivity, cropping methods and history, fertilizer applications, weather, and crop management. Based on OSU fertilizer guides requirements (pounds per acre) for the most predominant crops receiving the City's biosolids include: Pasture and grass hay, 60-100 pounds per acre.

Section 11: Recordkeeping and Reporting Procedures

11.1 Recordkeeping

The City as the preparer and land applier of biosolids is required to maintain records to demonstrate that federal and state biosolids requirements are met. Records will be kept on file by the City and will be available upon request by DEQ. Monitoring and sampling records are retained for no less than five years, unless otherwise required by the NPDES permit or a DEQ site authorization letter. The minimum required records include the following information:

- Pollutant concentrations of each parameter stated in the NPDES permit.
- Pathogen requirements as stated in the permit for Class B biosolids.
- Description of how one of the vector attraction reduction requirements in 40 CFR Part 503.33(b)(1) through (8) are met.
- Description of how the management practices in 40 CFR Part 503.14 and site restrictions in 40 CFR Part 503.32(b)(5) are met for each biosolids land application site.
- Certification that the information submitted is accurate to determine compliance with pathogen and vector attraction reduction requirements, and site restriction/management requirements.

11.2 Annual Reporting

The DEQ Annual Biosolids Report is due February 19 of each year for the previous year's biosolids production. A copy of this report is submitted to the DEQ. The annual report describes the quality and quantity of dewatered sludge or biosolids land applied or disposed. The City uses the DEQ's biosolids report form. The annual biosolids report includes, at a minimum, the following:

- Description of how biosolids were produced, stabilized, and tested.
- The amount of biosolids produced.
- Information characterizing the quality of the biosolids.
- Description of how pathogen reduction and vector attraction reduction requirements were met.
- Description of how and where the biosolids were managed.
- Necessary signed certifications for pathogen reduction, vector attraction reduction, and site management practices.
- Description of any major modifications to solids handling or land application site management practices.
- Description of any violation of state or federal rules pertaining to biosolids and remedial actions taken.

- The volume of biosolids and solids concentration shall be reported for each hauling event in the annual DEQ report. Nutrient and pollutant data will be in the annual report.
- Daily site logs for Class B biosolids, which have the date, time, and quantity land applied. Site logs will have a scaled map showing the site and the land application location that coincides with the daily site loading methods (i.e., truck splash plate).

In addition to the above information, the following is provided on the land application sites:

- Site names and locations
- Crops grown
- Amount of acreage amended
- Amount of biosolids applied (dry tons/acre), including date and location
- Amount of available nitrogen supplied by the biosolids
- Soil test(s) if required by a site authorization letter.

The Biosolids Annual Report will have a signed copy of the certification statements for pathogen reduction, vector attraction reduction and the biosolids that have been land applied at approved agronomic loading rates. The person signing the statements will be the operator of record at the WWTP. The operator will show how the vector attraction reduction was met for biosolids generated at the City's WWTP. Certification of pathogen reduction is required and is satisfied by submittal of test results in the DEQ Annual Biosolids Report. All the previous year's biosolids sampling and analysis that is required by the permit will be included in each year's annual report appendix.

The dates, volumes, and locations of any Class B biosolids land applications will be recorded in a log that can be reviewed by DEQ and maintained for a period of at least five years.

11.3 Certification Statements

The City's WWTP will be capable of meeting alternatives for achieving biosolids pathogen and vector attraction reduction criteria. Signed Class B biosolids and vector attraction certification statements are maintained at the City and are available upon request. As required in accordance with 40 CFR Part 503.17, the City must retain a certification statement indicating whether compliance with pathogen reduction, vector attraction reduction, and certain site restrictions has been met. The certification statement must be retained for a period of five years and must be submitted with the annual report that is due 19 February or as required by the permit. The City will retain the certification statement and it will be signed by a principal executive officer or ranking elected official or their duly authorized representative (e.g., individual or position having responsibility for the overall operation of the system, such as the position of plant manager, supervisor, superintendent or equivalent responsibility).

The following certification statements are applicable to the City's biosolids management program:

The City is required as the biosolids preparer to certify that pathogen reduction and VAR requirements in 40 CFR Part 503.32 and 503.33 are being met.

Class B Biosolids Pathogen and Vector Attraction Reduction Requirements

> *"I certify, under penalty of law, that the information that will be used to determine compliance with the Class B pathogen requirements in §503.32(b)(4) and the vector attraction reduction requirement in §503.33(b)(6) was prepared under my direction and supervision in accordance with the system designed to ensure that qualified personnel properly gather and evaluate this information. I am aware that there are significant penalties for false certification including the possibility of fine and imprisonment."*

Signature…… Date……

The City is also required as the land applier to certify that the management practices in 40 CFR Part 503.14, 503.32 and 503.33 are being met. This certification includes that biosolids are being land applied at approved agronomic loading rates as specified in DEQ issued site authorization letters.

Class B Biosolids Site Management Requirements

> *"I certify, under penalty of law that the management practices in 40 CFR §503.14 and 40 CFR §503.32(b) (5) have been met for each site on which bulk biosolids is applied. This determination has been made under my direction and supervision in accordance with the system designed to ensure that qualified personnel properly gather and evaluate the information used to determine that the management practices have been met. I am aware that there are significant penalties for false certification, including the possibility of fine and imprisonment."*

Signature…… Date……

> *"I certify, under penalty of law, that the information that will be used to determine compliance with the management practices in § 503.14 and the vector attraction reduction requirement in § 503.33(b)(10) was prepared under my direction and supervision in accordance with the system designed to ensure that qualified personnel properly gather and evaluate this information. I am aware that there are significant penalties for false certification including the possibility of fine and imprisonment."*

Signature…… Date……

Appendix H: WWTP Performance Summary

WWTP Effluent $CBOD_5$ (2020 – 2024)

Month	Influent Flow, MGD		Effluent Concentration, mg/L		Effluent Mass, lbs/day			Percent Removal
	Avg.	Max Day	Avg.	Max Week	Avg.	Max Week	Max Day	Avg.
Jan, 2020	8.3	14.4	8.3	9.1	509	668	1067	81
Feb, 2020	5.8	8.8	3.1	4.6	145	181	223	91
Mar, 2020	3.8	7.2	3.9	6.1	133	276	596	94
Apr, 2020	4.0	7.2	2.2	2.4	81	110	154	96
May, 2020	3.6	5.0	2.1	3.0	65	73	103	96
Jun, 2020	3.3	4.2	2.0	2.2	57	70	71	96
Jul, 2020	1.9	2.7	2.5	5.2	37	52	69	99
Aug, 2020	1.5	1.7	3.2	6.7	41	83	83	98
Sep, 2020	1.8	3.0	3.1	3.7	43	57	63	98
Oct, 2020	2.0	3.1	2.5	3.0	45	65	66	98
Nov, 2020	4.1	7.5	4.5	5.5	137	220	230	93
Dec, 2020	6.9	16.6	4.0	4.8	252	534	638	90
Jan, 2021	8.0	13.5	2.8	3.7	222	349	386	91
Feb, 2021	7.0	10.5	2.8	3.1	167	193	249	93
Mar, 2021	4.2	5.4	3.9	4.2	138	154	166	94
Apr, 2021	2.5	3.2	5.1	5.9	107	128	132	96
May, 2021	2.2	3.0	4.6	5.5	85	94	96	96
Jun, 2021	2.2	3.8	3.7	4.5	68	94	98	97
Jul, 2021	1.6	1.8	3.8	4.6	50	59	66	98
Aug, 2021	1.4	1.5	5.0	6.0	56	67	74	98
Sep, 2021	1.8	3.1	4.6	6.0	72	93	106	96
Oct, 2021	2.6	4.2	4.3	4.6	84	80	109	96
Nov, 2021	5.2	9.9	3.6	4.6	142	225	258	96
Dec, 2021	8.6	15.1	3.3	4.1	279	446	475	90
Jan, 2022	7.4	13.9	2.5	3.9	175	370	598	93
Feb, 2022	3.4	5.3	2.6	3.2	80	84	138	97
Mar, 2022	5.2	8.6	2.3	3.1	96	116	128	96
Apr, 2022	4.6	6.2	2.8	3	102	119	122	96
May, 2022	6.0	9.1	3.5	4.9	166	262	279	93
Jun, 2022	4.6	6.9	4.8	5.6	184	264	296	91
Jul, 2022	2.4	2.8	4.3	4.6	89	103	106	96

Month	Influent Flow, MGD		Effluent Concentration, mg/L		Effluent Mass, lbs/day			Percent Removal
	Avg.	Max Day	Avg.	Max Week	Avg.	Max Week	Max Day	Avg.
Aug, 2022	1.7	1.9	4.7	6.9	70	105	106	97
Sep, 2022	1.6	2.0	4.6	6.1	63	92	107	98
Oct, 2022	1.5	2.0	3.0	4.1	39	49	52	98
Nov, 2022	4.0	8.0	2.1	2.6	66	106	126	97
Dec, 2022	5.1	9.7	3.7	7.5	150	342	505	94
Jan 2023	5.7	9.1	2	3	109	134	138	95
Feb 2023	3.9	4.6	4	7	132	132	235	94
Mar 2023	5.0	8.6	4	5	178	369	446	91
Apr 2023	6.9	11.6	9	13	603	1,011	1,250	79
May 2023	3.4	4.7	3	5	98	151	169	97
Jun 2023	2.1	2.4	3	4	50	80	85	98
Jul 2023	1.5	2.4	**13**	**39**	187	**599**	**1,108**	92
Aug 2023	1.3	1.4	3	4	29	37	42	99
Sep 2023	1.4	1.9	2	5	26	55	55	98
Oct 2023	2.1	3.3	5	7	85	120	130	94
Nov 2023	3.8	7.1	5	7	180	290	337	93
Dec 2023	6.7	12.0	5	5	304	419	602	89
Jan 2024	8.83	15.2	6	8	492	725	924	80
Feb 2024	6.38	8.40	5	7	252	732	337	88
Mar 2024	6.77	13.4	4	6	214	339	407	90
Apr 2024	4.04	5.3	3	4	104	139	169	96
May 2024	4.9	10.9	3	4	128	192	220	97
Jun 2024	2.7	4.61	3	4	70	120	120	98
Jul 2024	1.8	2.1	3	3	40	51	61	99
Aug 2024	1.7	2.3	4	6	59	99	100	98
Sep 2024	1.7	2.1	3	4	48	52	53	98
Oct 2024	2.2	4.2	3	3	56	65	87	98
Nov 2024	6.2	9.8	3	3	131	184	188	95
Dec 2024	6.7	14.3	3	3	184	191	677	96

Note:

(a) Values in bold and underlined denote permit exceedances.

WWTP Effluent TSS (2020 – 2024)

Month	Influent Flow, MGD		Effluent Concentration,		Effluent Mass, lbs/day			Percent Removal
	Avg.	Max Day	Avg.	Max Week	Avg.	Max Week	Max Day	Avg.
Jan, 2020	8.3	14.4	**41**	**81**	**3,021**	**7009**	**10257**	**49**
Feb, 2020	5.8	8.8	6.6	15.7	333	453	1139	88
Mar, 2020	3.8	7.2	3.8	13.6	271	637	2907	92
Apr, 2020	4.0	7.2	7.2	10	284	465	658	90
May, 2020	3.6	5.0	4.9	6.0	148	218	218	96
Jun, 2020	3.3	4.2	4.6	5.0	127	143	165	94
Jul, 2020	1.9	2.7	5.0	7.6	74	102	132	97
Aug, 2020	1.5	1.7	8.6	11.9	111	153	498	98
Sep, 2020	1.8	3.0	5.7	9.8	76	103	127	95
Oct, 2020	2.0	3.1	5.9	8.3	105	148	161	96
Nov, 2020	4.1	7.5	16.2	19.4	477	613	771	84
Dec, 2020	6.9	16.6	11.5	14.2	756	1573	1824	75
Jan, 2021	8.0	13.5	6.6	9.1	513	844	991	88
Feb, 2021	7.0	10.5	5.0	7.0	316	388	548	89
Mar, 2021	4.2	5.4	6.4	6.4	231	272	362	92
Apr, 2021	2.5	3.2	7.0	8.0	146	204	204	95
May, 2021	2.2	3.0	7.3	10.2	134	149	163	96
Jun, 2021	2.2	3.8	2.9	6.4	51	68	105	98
Jul, 2021	1.6	1.8	3.3	4.4	43	56	61	98
Aug, 2021	1.4	1.5	3.0	4.0	37	49	49	98
Sep, 2021	1.8	3.1	5.7	8.0	95	144	175	96
Oct, 2021	2.6	4.2	8.3	11.7	173	209	262	94
Nov, 2021	5.2	9.9	13	22	526	1049	1624	83
Dec, 2021	8.6	15.1	8.5	9.0	780	932	1187	81
Jan, 2022	7.4	13.9	6.6	9.9	460	915	1412	87
Feb, 2022	3.4	5.3	4.0	4.0	123	142	178	96
Mar, 2022	5.2	8.6	4.8	6	198	240	271	93
Apr, 2022	4.6	6.2	4.6	5.2	168	202	230	94
May, 2022	6.0	9.1	7.0	8.4	332	453	557	89
Jun, 2022	4.6	6.9	8.3	10.2	341	530	680	87
Jul, 2022	2.4	2.8	4.1	5.2	86	116	118	97

Month	Influent Flow, MGD		Effluent Concentration,		Effluent Mass, lbs/day			Percent Removal
	Avg.	Max Day	Avg.	Max Week	Avg.	Max Week	Max Day	Avg.
Aug, 2022	1.7	1.9	5.1	7.5	75	114	114	97
Sep, 2022	1.6	2.0	2.8	5.4	38	80	82	99
Oct, 2022	1.5	2.0	3.3	4.2	43	58	64	99
Nov, 2022	4.0	8.0	4.2	5.2	169	211	319	95
Dec, 2022	5.1	9.7	35.2	**152**	**1724**	**7432**	**14305**	**11**
Jan 2023	5.7	9.1	11	15	516	817	1,123	85
Feb 2023	3.9	4.6	11	17	365	542	635	89
Mar 2023	5	8.6	13	29	540	1,353	1,708	82
Apr 2023	6.9	11.6	14	34	981	2,326	2,633	75
May 2023	3.4	4.7	4	7	132	235	236	96
Jun 2023	2.1	2.4	3	4	49	68	68	99
Jul 2023	1.5	2.4	**102**	**394**	**1,540**	**5,990**	**11,781**	**57**
Aug 2023	1.3	1.4	3	4	30	43	53	99
Sep 2023	1.4	1.9	4	8	48	97	115	97
Oct 2023	2.1	3.3	10	12	184	238	330	92
Nov 2023	3.8	7.1	12	13	402	617	760	89
Dec 2023	6.7	12	10	12	649	970	1,196	83
Jan 2024	8.8	15.2	18	30	1,443	2,447	3,466	**68**
Feb 2024	6.4	8.4	8	19	440	1,936	708	89
Mar 2024	6.8	13.4	6	8	360	598	673	91
Apr 2024	4.0	5.3	4	5	130	177	205	99
May 2024	4.9	10.9	5	6	199	346	356	98
Jun 2024	2.7	4.6	4	6	103	178	231	99
Jul 2024	1.8	2.1	2	3	30	51	66	100
Aug 2024	1.7	2.3	6	14	92	246	261	99
Sep 2024	1.7	2.1	3	3	37	42	44	99
Oct 2024	2.2	4.2	3	3	55	60	117	99
Nov 2024	6.2	9.8	3	4	174	263	277	96
Dec 2024	6.7	14.3	4	5	228	303	735	98

Note:

(a) Values in bold and underlined denote permit exceedances.

WWTP Effluent E. coli, Ammonia, and Chlorine (2020 – 2024)

Month	Influent Flow, MGD		*E. coli* Bacteria, MPN		Ammonia, mg/L		Chlorine Residual, mg/L	
	Avg.	Max Day	Avg.	Max Sample	Avg.	Max Day	Avg.	Max Day
Jan, 2020	8.3	14.4	113	325	0.59	1.00	0.003	0.009
Feb, 2020	5.8	8.8	55	239	0.50	1.20	0.003	0.009
Mar, 2020	3.8	7.2	19.4	115	0.20	0.72	0.005	0.012
Apr, 2020	4	7.2	15.8	71	0.39	0.90	0.003	0.009
May, 2020	3.6	5.0	5.4	9.2	0.06	0.12	0.005	0.012
Jun, 2020	3.3	4.2	36.7	209.8	0.05	0.10	0.004	0.008
Jul, 2020	1.9	2.7	34.7	60	0.10	0.17	0.003	0.012
Aug, 2020	1.5	1.7	36	243	0.20	0.20	0.004	0.015
Sep, 2020	1.8	3.0	19.7	116.9	0.10	0.16	0.003	0.008
Oct, 2020	2.0	3.1	20.2	37	0.08	0.12	0.005	0.012
Nov, 2020	4.1	7.5	99.4	310	0.06	0.11	0.007	0.014
Dec, 2020	6.9	16.6	17.5	26.8	0.11	0.25	0.008	0.02
Jan, 2021	8.0	13.5	14.2	163	0.12	0.23	0.007	0.019
Feb, 2021	7.0	10.5	7.0	261	0.1	0.17	0.004	0.012
Mar, 2021	4.2	5.4	7.4	18.5	0.06	0.08	0.004	0.015
Apr, 2021	2.5	3.2	7.1	200	0.10	0.13	0.005	**0.054**
May, 2021	2.2	3.0	1.0	1.0	0.12	0.20	0.004	0.019
Jun, 2021	2.2	3.8	1.3	3.1	0.09	0.20	0.004	0.19
Jul, 2021	1.6	1.8	1.0	1.0	0.13	0.24	0.003	0.02
Aug, 2021	1.4	1.5	1	2	0.13	0.22	0.002	0.004
Sep, 2021	1.8	3.1	1.6	4.1	0.17	0.28	0.002	0.002
Oct, 2021	2.6	4.2	1.4	3	0.20	0.25	0.002	0.002
Nov, 2021	5.2	9.9	1	1	0.09	0.19	0.002	0.002
Dec, 2021	8.6	15.1	1.8	3	0.21	0.25	0.002	0.004
Jan, 2022	7.4	13.9	1.2	3	0.27	0.53	0.002	0.005
Feb, 2022	3.4	5.3	1	1	0.2	0.2	0.003	0.008
Mar, 2022	5.2	8.6	1	1	0.2	0.2	0.006	0.018
Apr, 2022	4.6	6.2	1.2	3.1	0.11	0.2	0.003	0.009
May, 2022	6.0	9.1	1	1	0.03	0.04	0.006	0.02
Jun, 2022	4.6	6.9	1.2	3.1	0.04	0.04	0.005	0.015
Jul, 2022	2.4	2.8	1.5	4.1	0.04	0.05	0.003	0.006

Month	Influent Flow, MGD		*E. coli* Bacteria, MPN		Ammonia, mg/L		Chlorine Residual, mg/L	
	Avg.	Max Day	Avg.	Max Sample	Avg.	Max Day	Avg.	Max Day
Aug, 2022	1.7	1.9	1.3	7.5	0.40	0.90	0.002	0.010
Sep, 2022	1.6	2.0	3.1	28.5	0.20	0.66	0.003	0.014
Oct, 2022	1.5	2.0	4.9	38.4	0.05	0.09	0.003	0.009
Nov, 2022	4.0	8.0	1.3	4.1	0.04	0.14	0.005	0.019
Dec, 2022	5.1	9.7	2.1	6.3	0.08	0.18	0.003	0.007
Jan 2023	5.7	9.1	3.5	12	0.02	0.04	0.00	0.01
Feb 2023	3.9	4.6	6.1	12	0.07	0.09	0.00	0.01
Mar 2023	5.0	8.6	2.5	8	0.05	0.08	**0.02**	**0.51**
Apr 2023	6.9	11.6	2.5	5	0.18	0.71	0.00	0.01
May 2023	3.4	4.7	2.3	7	0.11	0.21	0.01	0.02
Jun 2023	2.1	2.4	5.6	16	0.13	0.39	0.01	0.02
Jul 2023	1.5	2.4	8.2	**1,987**	0.29	0.85	**0.02**	**0.36**
Aug 2023	1.3	1.4	2.3	5	0.13	0.22	0.00	0.02
Sep 2023	1.4	1.9	1.1	2	0.26	0.97	0.00	0.01
Oct 2023	2.1	3.3	1.1	1	0.54	1.02	0.01	0.02
Nov 2023	3.8	7.1	17.2	93	0.35	0.55	0.00	0.01
Dec 2023	6.7	12.0	3.0	22	0.33	0.62	0.00	0.01
Jan 2024	8.8	15.2	5.7	24	0.41	0.56	0.00	0.01
Feb 2024	6.4	8.4	3.2	11	0.41	1.69	0.00	0.00
Mar 2024	6.8	13.4	2.1	5	0.09	0.20	0.00	0.02
Apr 2024	4.0	5.3	1.3	3	0.08	0.21	0.00	0.01
May 2024	4.9	10.9	1.9	4	0.13	0.61	0.01	0.02
Jun 2024	2.7	4.6	1.9	3	0.05	0.06	0.01	0.01
Jul 2024	1.8	2.1	1.9	6	0.08	0.20	0.00	0.01
Aug 2024	1.7	2.3	3.1	24	0.41	1.81	0.00	0.02
Sep 2024	1.7	2.1	1.5	4	0.07	0.11	0.00	0.00
Oct 2024	2.2	4.2	1.4	3	0.21	0.50	0.00	0.02
Nov 2024	6.2	9.8	1.4	3	0.16	0.27	0.00	0.00
Dec 2024	6.7	14.3	1.3	8	0.34	1.11	0.00	0.01

Note:

(b) Values in bold and underlined denote permit exceedances.