DRAFT
REPORT

Amended Wastewater Treatment Plant Facility Plan

September 2014

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Executive Summary

Purpose

The City of Piqua, Ohio (City) is planning for the necessary wastewater treatment infrastructure improvements to eliminate sanitary sewer overflows (SSOs) and provide sufficient capacity for growth projections to the design year of 2030. CDM Smith developed future flow projections using available documentation to quantify population growth, land use, and redevelopment opportunities within the City. Using these flow projections, multiple wastewater treatment processes were evaluated, and then four processes were analyzed in more detail for both liquid treatment and solids processing to meet the treatment goals for the wastewater treatment plant's (WWTP's) service life, to eliminate SSOs, and provide treatment capacity for the design year. This document evaluates the alternatives and provides recommendations for infrastructure improvements to meet those goals.

Flow and Load Projections

The existing WWTP is rated for 4.5 million gallons per day (MGD) average-day/maximum-month and 8.3 MGD peak-hour treatment capacity. Based on hydraulic limitations within the plant's raw sewage pump station and some conveyances between unit processes, the plant can reliably treat a maximum flow of only 7.5 MGD. Beyond this treatment capacity, the plant has a flow equalization (EQ) basin with a storage capacity of 1 million gallons (MG), and limited additional storage within the interceptor sewer system piping for excess wet-weather volume and to help balance peak flows. Despite these treatment and storage facilities, the constructed SSO just upstream of the treatment plant on the West Interceptor activates periodically. Based on direction from Ohio EPA, it must be eliminated.

Through the planning period covered by this Amended WWTP Facility Plan, by the year 2030, the City's sewer service area is projected to develop with new residential and commercial/industrial growth that will lead to increased wastewater flows from the expanded customer base. The sewer service area is also anticipated to expand with continued development. Additionally, the City was approached by the Village of Covington for potential sanitary sewer service, which would have a significant impact on the design flows and loads tributary to the plant.

The rated average day capacity of the wastewater treatment plant is recommended to increase to 6.0 MGD to meet Piqua's future demands. The addition of the Village of Covington flow would add another 1.0 MGD average-day hydraulic capacity. The total projected average-day, maximum-month capacity of the plant is 7.0 MGD.

Table ES-1 provides the influent wastewater characteristics for the existing treatment plant and the projected characteristics at the end of the planning period.

Table ES-1: Influent Wastewater Characteristics

Parameter	Existing Average Influent Concentration (mg/L)	Existing Average Influent Loading (lbs/day)	Typical Influent Concentration (mg/L)	Projected Future Influent Loading (lbs/day)
5-day Carbonaceous Biochemical Oxygen Demand (CBOD)	140	5,300	190	9,200
Total Suspended Solids (TSS)	128	4,800	210	9,200
Ammonia (NH_3)	11.5	430	25	1,000

Evaluation and recommendation of treatment processes and capacities within this Amended WWTP Facility Plan is based on increased flow projections. The variability of flows, which are highly dependent on wet-weather influences, must also be considered. The Sanitary Sewer System Master Plan, completed in 2013, presented hydrogaphs showing future flow projections based on a 20-year planning period using continuous model simulation based on historic rainfall data. The month of April 2011 provided the worst-case conditions, with long-term precipitation documented. The worst-case rainfall and flow projections determined the required WWTP capacity.

Six combinations of additional flow equalization and treatment capacity were then considered to provide a cost-effective overall solution to treat the projected future flow volumes and eliminate the SSO. Preliminary capital costs were estimated for these treatment and storage combinations using planning-level unit costs to determine the optimum solution for the City to eliminate the SSO and provide service life for a 20-year planning period. Although calculations were provided for 6 MGD average-day treatment capacity, increasing the plant's rated capacity to 7 MGD to account for influent flow from Covington would have an equal impact for all combinations. The preliminary results from this analysis are considered order-of-magnitude cost estimates for planning purposes only; they are provided in **Table ES-2** and shown graphically in **Figure ES-1**.

Table ES-2: Planning-Level Costs for Combinations of Peak Wastewater Treatment and EQ Storage

Scenario	WWTP Max Flow (MGD)	WWTP Avg Flow (MGD)	Max/Avg Ratio	Total EQ (MG)	Additional WWTP Cost	Additional EQ Cost	Total Cost
1	10.5	6	1.75	12	$13,500,000	$14,000,000	$27,500,000
2	11	6	1.83	9	$13,500,000	$9,500,000	$23,000,000
3	12	6	2.00	7.5	$13,500,000	$7,250,000	$20,750,000
4	13	6	2.17	6	$16,500,000	$5,000,000	$21,500,000
5	17	6	2.83	3	$28,500,000	$500,000	$29,000,000
6	21.5	6	3.58	1	$42,000,000	$0	$42,000,000

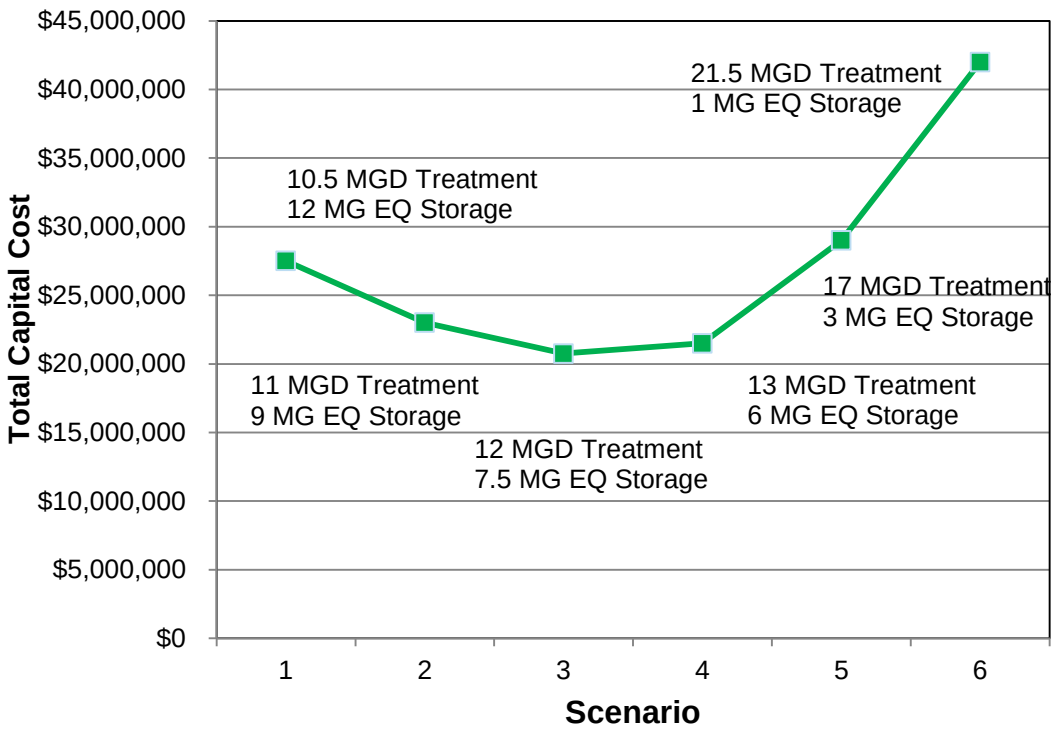


Figure ES-1: Cost Comparison of Treatment and Equalization Volume Necessary to Eliminate SSOs

The order-of-magnitude cost estimates for combinations of future treatment capacity and EQ storage shown in Table ES-2 and Figure ES-1 were used to establish a base treatment capacity on which to compare various treatment technologies. The final recommended treatment capacity is determined through a supplemental cost-optimization exercise, explained in Section 7 of this Amended WWTP Facility Plan and summarized at the end of this Executive Summary. According to Figure ES-1, the optimum combination to eliminate the SSO (and to use for comparison of treatment alternatives) includes a peak treatment capacity of 13 MGD and an EQ storage volume of 6 MG. This combination of treatment capacity and EQ storage has a comparable capital cost projection to the adjacent combination of 12 MGD treatment and 7.5 MG storage, as shown in Figure ES-1, but the13-MGD / 6 MG alternative is more consistent with the City's original plans for the expansion of EQ basin storage facilities.

Adding the projected wastewater flow from the nearby Village of Covington, the treatment capacities used to evaluate liquid treatment train alternatives within this document are 7.0 MGD average-day, maximum month (noted above) and 14 MGD peak capacity.

Existing Facility Condition

The City's WWTP currently treats influent wastewater in accordance with its NPDES permit limits. However, many components are aging and in need of upgrade or replacement to continue reliable service. This Amended WWTP Facility Plan evaluated the existing treatment processes throughout the treatment plant and identified those which should be upgraded to improve reliability, redundancy, or overall efficiency for long-term use, or until they are no longer needed.

The existing wastewater treatment facilities will continue to be used until they are upgraded or replaced. Upgrades were recommended where unit processes could be salvaged and/or included as part of a long-term solution or expansion, which is a cost-effective approach. Where processes and equipment are near the end of their useful life, or do not meet future treatment needs, replacements are recommended for those processes and equipment. Specific maintenance and/or upgrades of existing WWTP unit processes are listed under the Recommendations heading of this Executive Summary.

Unit Process Upgrades Common to All Improvement Alternatives

All of the WWTP improvement alternatives evaluated within this Amended WWTP Facility Plan, whether they feature an upgrade to the existing WWTP or an entirely new treatment plant, include certain unit process upgrades or replacements that are common to all of them. They are described briefly below:

- **Raw Sewage Pumping** – Replace the aging and undersized screw pumps and provide expanded pumping capacity as necessary to convey projected future flows.
- **Headworks** – Install fine screen(s) to remove influent solids for improved efficiency of downstream liquid treatment processes, to protect downstream liquid treatment process equipment, and to comply with regulations related to the continued beneficial reuse of biosolids through land application. Immediately downstream of the fine screens, install a vortex grit removal system for effective and efficient grit removal, including new recessed-impeller grit pumps and a grit classifier for discharge of grit into a hopper or dumpster for disposal.
- **Disinfection** – Replace the current gaseous chlorine disinfection and sulfur dioxide dechlorination systems with ultraviolet (UV) disinfection equipment installed within the existing chlorine contact tank.
- **Flow Metering** – Provide new flow metering equipment near the treatment plant outfall to monitor actual flow, instead of at the plant influent.

Liquid Treatment Train Alternatives

Ten liquid treatment train alternatives were initially considered for the treatment plant expansion. They are listed below:

- Conventional Activated Sludge (Upgrade and Expand Current Treatment Plant)
- Extended Aeration (3.0-MGD Oxidation Ditch Parallel to and Operating with Existing WWTP)
- Extended Aeration (7.0-MGD Oxidation Ditch Replacing Existing WWTP)
- Membrane Bioreactor (MBR)
- Integrated Fixed-Film Activated Sludge (IFAS)
- Sequencing Batch Reactor (SBR)
- Anaerobic/Anoxic/Oxic (A2O) Process

- BioMag Process
- Step Feed Process
- BioActiflo® Process Operating with Existing WWTP

Process Alternatives Screening Workshop #1 was conducted to discuss the above liquid treatment train alternatives. The main goals of Workshop #1 were to review the advantages and disadvantages of implementing each alternative, and then assign scores to each one in several cost and non-cost categories, finally developing a short list of four liquid treatment train alternatives to evaluate in more detail. The result of the in-depth evaluation would be the recommendation of a liquid treatment train alternative for design and construction, along with a solids treatment process that would be compatible with the liquid treatment process.

The following liquid treatment train alternatives were short-listed for more detailed analysis in this Amended WWTP Facility Plan.

- **Anaerobic/Anoxic/Oxic (A2O) Process** – providing nutrient removal with additional reactors and settling tanks and upgrading existing bioreactors and tankage for continued use within the existing WWTP
- **Extended Aeration (3.0-MGD Oxidation Ditch Parallel to and Operating with Existing WWTP)** – continuing to use the existing treatment plant, and supplementing the treatment processes with a parallel extended aeration process or oxidation ditch and associated facilities to treat the additional flows
- **Extended Aeration (7.0-MGD Oxidation Ditch Replacing Existing WWTP)** – abandoning much of the existing treatment plant, and utilizing a new extended aeration process or oxidation ditch and associated facilities to treat all wastewater flows
- **Sequencing Batch Reactor (SBR)** – abandoning much of the existing treatment plant and utilizing a new batch process to treat all wastewater flows

Each of the liquid treatment train alternatives listed above was evaluated and scored based on quantitative and qualitative criteria, including estimated relative capital cost, relative operational cost, operational requirements, treatment efficiency, ease of operation, flexibility for future expansion, ability to meet future regulations, reliability/risk, and implementation, summarized in **Table ES-3**.

Table ES-3: Liquid Treatment Train Alternatives Scoring

City of Piqua, OH Wastewater Treatment Plant - Liquid Treatment Train Alternative Ranking										
	Screening Criteria (0=lowest/worst, 5=highest/best)									
Liquid Treatment Train Alternatives	Relative Capital Cost	Relative O&M Costs	Maintenance of Plant Operations (MOPO)	Treatment Efficiency	Ease of Operation	Flexibilty for Future Expansion	Ability to Meet Future Regulations	Reliability/ Risk	Implementation	Weighted Sum
100%	18%	15%	10%	3%	12%	10%	10%	15%	7%	(1-5)
1 – A2O	4	3	2	3	2	1	3	4	2	2.84
2 - 3.0-MGD Oxidation Ditch with Existing WWTP	3	2	3	3	2	2	3	3	3	2.63
3 - 7.0-MGD Oxidation Ditch - Demolish Existing WWTP	3	4	4	3	4	4	3	5	4	3.84
4 – Sequencing Batch Reactor (SBR)	4	3	4	3	2	4	3	4	3	3.41

Alternative 3, the 7.0-MGD Oxidation Ditch, and Alternative 4, the SBR, received the highest (best) ratings based on the scoring criteria mentioned above. They both scored well in reliability, flexibility, and maintenance of plant operations (MOPO) during construction. The extended aeration or oxidation ditch process has many years and a proven track record of effective performance, and the WWTP staff from Piqua are familiar with its requirements and operation, though it is not the type of treatment plant currently existing at Piqua. Similarly, the SBR has many installations with proven performance, and advantages that make it a viable alternative for implementation. However, Piqua WWTP staff are not familiar with its operation, and it is not as conducive to future requirements for phosphorus removal, which is expected at Piqua.

Alternative 1, the A2O process, and Alternative 2, the 3.0-MGD Oxidation Ditch, would both operate parallel to an upgraded existing liquid treatment train, thereby using the existing treatment plant. This was judged to be a major drawback for both alternatives because the existing facilities would require major equipment upgrades and complex underground piping and channel modifications to allow for the increased flows, as well as modifications to improve system hydraulics. Based on the current layout, system hydraulics could be only partially improved, and would not be cost-effectively implemented throughout the existing WWTP.

As shown in **Table ES-3**, a conceptual economic analysis was conducted to evaluate the four short-listed liquid treatment train alternatives, including relative capital cost and relative ongoing O&M cost. The estimated costs covered the implementation of the liquid treatment train alternative, upgrades to portions of the existing treatment plant as applicable, and operation of the new or revised facility. These costs were then compared on a present worth basis, as shown in **Table ES-4**, under the same planning period.

Table ES-4: Present Worth Cost of Liquid Treatment Alternatives

	1	2	3	4
	A2O Process & Existing Plant Upgrade	3.0-MGD Oxidation Ditch with Upgraded Existing Plant	New 7.0-MGD Oxidation Ditch Treatment Plant	New SBR Treatment Plant
Probable Construction Cost	**$28,000,000**	**$33,000,000**	**$33,000,000**	**$31,000,000**
Annual O&M Costs				
Electricity	$240,000	$259,000	$188,000	$184,000
Labor & Maintenance	$96,000	$115,000	$86,000	$74,000
Chemicals/UV				
Total O&M Costs	$336,000	$374,000	$274,000	$258,000
Present Worth O&M Costs[1]	**$4,775,000**	**$5,315,000**	**$3,894,000**	**$3,667,000**
Total Present Worth Cost	*$32,775,000*	*$38,315,000*	*$36,894,000*	*$34,667,000*

1 Present worth cost is calculated at 3.5% interest for 20 years

Liquid Treatment Train Process Recommendation

Table ES-3 indicates that Alternative No. 3, the 7.0-MGD Oxidation Ditch treatment plant, received the highest weighted score based on a combination of cost- and non-cost-related criteria. This is an important determination because the scoring process revealed clear strengths related to this treatment alternative compared to the other three alternatives. It scored higher than the second-

ranked alternative, the SBR, in several categories, most notably 'ease of operation.' This Amended WWTP Facility Plan takes into consideration all such criteria; the weighting of each category indicates its relative importance in relation to the other categories. Considering them all, **the 7.0-MGD Oxidation Ditch treatment plant is the preferred and recommended treatment alternative.**

Alternative No. 1, the A2O process, was estimated with the lowest capital and total present-worth costs. However, its flexibility for future expansion, ease of operation and general implementation scores were poor, all related to the continued utilization of the existing WWTP and its inherent weaknesses. Thus, its weighted total score was second-lowest of the four. So in spite of favorable costs, it was not selected. Similarly, the 3.0-MGD Oxidation Ditch, which would include a process identical to the recommended alternative, would also include continued utilization of the existing WWTP facilities. Their hydraulic problems, sludge settleability issues, and other weaknesses described within this document made Alternative No. 2 the lowest-scoring option.

Alternative No. 4, the SBR process, scored medium-to-well except for ease of operation. Related to this, there was a general concern about this treatment process that is so dependent upon mechanical devices, which is true for the multiple treatment phases that occur within a single sequencing batch reactor. The layout proposed for Piqua would include four reactors, each operating relatively independently of the other. These factors contributed to the lower scoring for this process compared to the oxidation ditch process.

Solids Treatment Processes

The first task in evaluating solids treatment processes was to discuss and screen multiple process alternatives at Workshop #1A, devoted to that purpose. Six solids treatment processes were presented, discussed, and rated for their effectiveness and applicability for Piqua, listed below:

- High-Rate Anaerobic Digestion
- Aerobic Digestion
- Temperature-Phased Anaerobic Digestion (TPAD)
- Autothermal Thermophilic Aerobic Digestion (ATAD)
- Burch-Hydro Microwave (Biowave™) Process
- Thermal Drying following Digestion

It was agreed at the screening workshop that the recommended solids treatment process would have to be compatible with the selected liquid treatment process, which was being evaluated separately. With that in mind, four of the solids treatment processes were short-listed for more in-depth evaluation and future ranking to develop a final recommendation. The four short-listed processes are as follows:

- High-Rate Anaerobic Digestion
- Aerobic Digestion
- Autothermal Thermophilic Aerobic Digestion (ATAD)

- Thermal Drying following Aerobic or Anaerobic Digestion

After an evaluation period, another workshop was conducted to discuss the advantages and disadvantages of the four short-listed solids treatment processes. During this Workshop #2 the project team selected the 7.0-MGD Oxidation Ditch treatment plant as the recommended liquid treatment train alternative. This selection impacted the evaluation of the solids treatment process alternatives, because oxidation ditches are one version of the extended aeration treatment process, which does not include primary settling, and therefore does not produce primary/raw sludge. That factor generally eliminated the anaerobic digestion process from consideration as a solids treatment process at Piqua. The remaining three alternatives were evaluated and scored based on relative capital cost, relative operating cost, maintenance of plant operations (MOPO) during construction, treatment efficiency, ease of operation, flexibility for future expansion, ability to meet future regulations, reliability and risk, and general implementation. The results of the scoring and ranking exercise are shown below in **Table ES-5.**

Table ES-5: Solids Treatment Process Alternatives Scoring

City of Piqua, OH - Amended WWTP Facility Plan										
Wastewater Treatment Plant - Solids Treatment Process Alternatives Ranking										
Solids Treatment Process Alternatives	Screening Criteria (0=lowest/worst, 5=highest/best)									Weighted Sum
	Relative Capital Cost	Relative O&M Costs	Maintenance of Plant Operations (MOPO)	Treatment Efficiency	Ease of Operation	Flexibilty for Future Expansion	Ability to Meet Future Regulations	Reliability/ Risk	Implementation	
100%	18%	15%	10%	3%	12%	8%	12%	15%	7%	(1-5)
1b - Anaerobic Digestion	0	0	0	0	0	0	0	0	0	0.00
2 - Aerobic Digestion	5	5	3	3	5	5	3	4	5	4.35
3 - Autothermal Thermophilic Aerobic Digestion (ATAD)	3	3	2	5	3	2	5	5	3	3.42
4 - Digestion w/Thermal Drying	3	3	3	5	3	3	5	5	3	3.60

The four original solids treatment processes were also evaluated for their potential to produce either Class B or Exceptional Quality Biosolids. The results of that evaluation are shown in **Table ES-6**.

Table ES-6: Summary of Biosolids Processing Alternatives

Biosolids Processing Alternative	Exceptional Quality Biosolids	Class B Biosolids
High-Rate Anaerobic Digestion	No	Yes
Aerobic Digestion	No	Yes
Autothermal Thermophilic Aerobic Digestion (ATAD)	Yes	Yes
Digestion with Thermal Drying	Yes	Yes

Each solids treatment process alternative was then evaluated for its estimated capital cost and the annual O&M costs to implement the alternative, upgrade the treatment plant's existing digesters, and operate the system. These costs were then compared on a present-worth basis, shown in **Table ES-7**, to compare costs for all four processes under a 20-year planning period. It is presumed that City staffing needs would be consistent among these alternatives, and would result in no net differential labor costs.

Table ES-7 – Present Worth Cost Analysis of Sludge Digestion Alternatives

	High-Rate Anaerobic Digestion	Aerobic Digestion – Jet Air Mixing	Aerobic Digestion – Coarse-Bubble Diffusers	ATAD	Digestion with Thermal Drying
Total Probable Construction Cost	$6,800,000	$3,500,000	$3,300,000	$7,100,000	$5,900,000
Annual O&M Cost					
Electricity	$23,200	$78,000	$47,000	$184,000	$119,000
Labor					
Maintenance	$73,000	$27,000	$25,000	$64,000	$63,000
Gas					$47,000
Hauling/Land Application	$20,000	$21,000	$21,000	$17,000	$14,000
Total Annual O&M Cost	$116,200	$126,000	$93,000	$265,000	$243,000
Present Worth O&M Costs	**$1,651,000**	**$1,793,000**	**$1,323,000**	**$3,771,000**	**$3,458,000**
Total 20-year Present Worth Cost	**$8, 500,000**	**$5,300,000**	**$4,600,000**	**$10,900,000**	**$9,300,000**

Solids Treatment Process Recommendation

Drawing from **Tables ES-5, ES-6, and ES-7**, the ATAD system and the thermal drying system offer unique benefits of biosolids volume reduction by providing significant dewatering and drying capabilities or by increased destruction of volatile solids. This volume reduction results in the net effect of less biosolids material to process, store, and haul off-site for disposal. However, the capital cost and total present-worth cost are not favorable for these alternatives compared to the two versions of the aerobic digestion process for which cost estimates were developed.

Aerobic digestion would work well for the selected liquid treatment train alternative, the oxidation ditch. Based on the scoring at Workshop#2, it is the favored solids processing alternative for the new Piqua WWTP. It also has the lowest capital and operational cost, resulting in the lowest total-worth cost of the four alternatives reviewed for this Amended WWTP Facility Plan. Some of the advantages of aerobic digestion are clear – the process could be retrofitted into the existing digesters, and if additional digestion capacity is needed, the existing aeration tanks and even clarifiers are available for retrofitting after wastewater treatment is switched to the new WWTP. It is a familiar process and not complex. The main disadvantages of implementing aerobic sludge digestion are the ongoing electrical cost associated with blower operation, greater challenges related to biosolids dewatering than other sludge digestion processes, and the production of Class 'B' biosolids instead of Exceptional Quality biosolids. However, Exceptional Quality biosolids could still be produced with subsequent improvements if required by regulatory action or if desired by the City through the installation of an indirect thermal drying process or other options. Because it has been used extensively at municipal wastewater treatment plants over many years and would work well with an extended aeration / oxidation ditch treatment plant, **aerobic sludge digestion is recommended for implementation at Piqua.**

Considering other aspects of solids processing and optimization at the Piqua WWTP, one of the challenges the existing treatment plant faces with biosolids processing is the lack of thickening before sending WAS to the digesters. By delivering 1% - 2% solids to the digesters instead of 5% solids, the additional water fraction results in decreased hydraulic residence time and potential for foaming or overflow conditions in the digesters. Thickening the WAS before digestion is recommended to better operate the digesters and eliminate these operational problems.

Additional sludge cake storage will be required for the City to meet the updated biosolids regulations, specifically to store 120 days of volume to avoid land application during winter months when frozen soil prevents land application. To accommodate this storage requirement, a structure that houses dewatering equipment and provides biosolids cake storage is recommended on the south side of the existing treatment plant on City-owned property.

Cost Optimization and Saving Recommendations

Following selection of the recommended liquid treatment train and solids treatment process alternatives, another cost optimization evaluation was carried out to select the most cost-effective combination of treatment plant capacity and EQ basin storage volume. This evaluation compared the estimated construction cost for incremental increases in treatment capacity with incremental decreases in EQ basin storage volume, with the goal of determining which combination would be the most cost-effective approach to eliminate the SSO. It was determined that the most cost-effective combination of wastewater system improvements is a WWTP with a peak-flow capacity of 22.5 MGD matched with an EQ basin storage volume of 1 MG. The corresponding construction cost estimate for this alternative is shown in **Table ES-8,** along with the estimated project costs related to engineering and other services during the design and construction phases of the project.

Table ES-8 – Recommended Alternative Cost Summary

Description	Cost
Selected Liquid Treatment Alternative Alt. 3 – 7.0-MGD Oxidation Ditch Treatment Plant	$35,700,000
Selected Solids Processing Alternative Alt. 2 – Aerobic Digestion	$3,300,000
Savings in delaying construction of anaerobic tanks for phosphorus removal	-$500,000
Preliminary Opinion of Probable Construction Cost	$38,500,000
Preliminary Engineering & Detailed Design	$3,900,000
Construction-Phase Engineering Services*	$4,800,000
Total Estimated Project Cost	**$47,200,000**
*Construction Phase Engineering Services include Applications Engineering, Startup and Training, Computerized Maintenance Management System (CMMS), Electronic O&M Manual, and Resident Project Representation (RPR) Services.	

Recommendations

The preferred liquid treatment alternative (7.0-MGD Oxidation Ditch treatment plant) and the preferred solids treatment alternative (aerobic digestion) were recommended using separate criteria for non-cost and economic evaluations. These two processes are feasible together and will not interfere with the efficiency or performance of each other. As described above, additional treatment plant improvements are required to provide sufficient and reliable treatment to serve the City and its customers for a 20-year planning period. These items include new raw sewage pumping, screening, grit removal, disinfection, and flow metering and are included within the respective liquid or solids process alternative costs

Figure ES-2 provides an overall layout of the major unit processes that are recommended for implementation to provide treatment capacity for future projected flows and to eliminate the SSO.

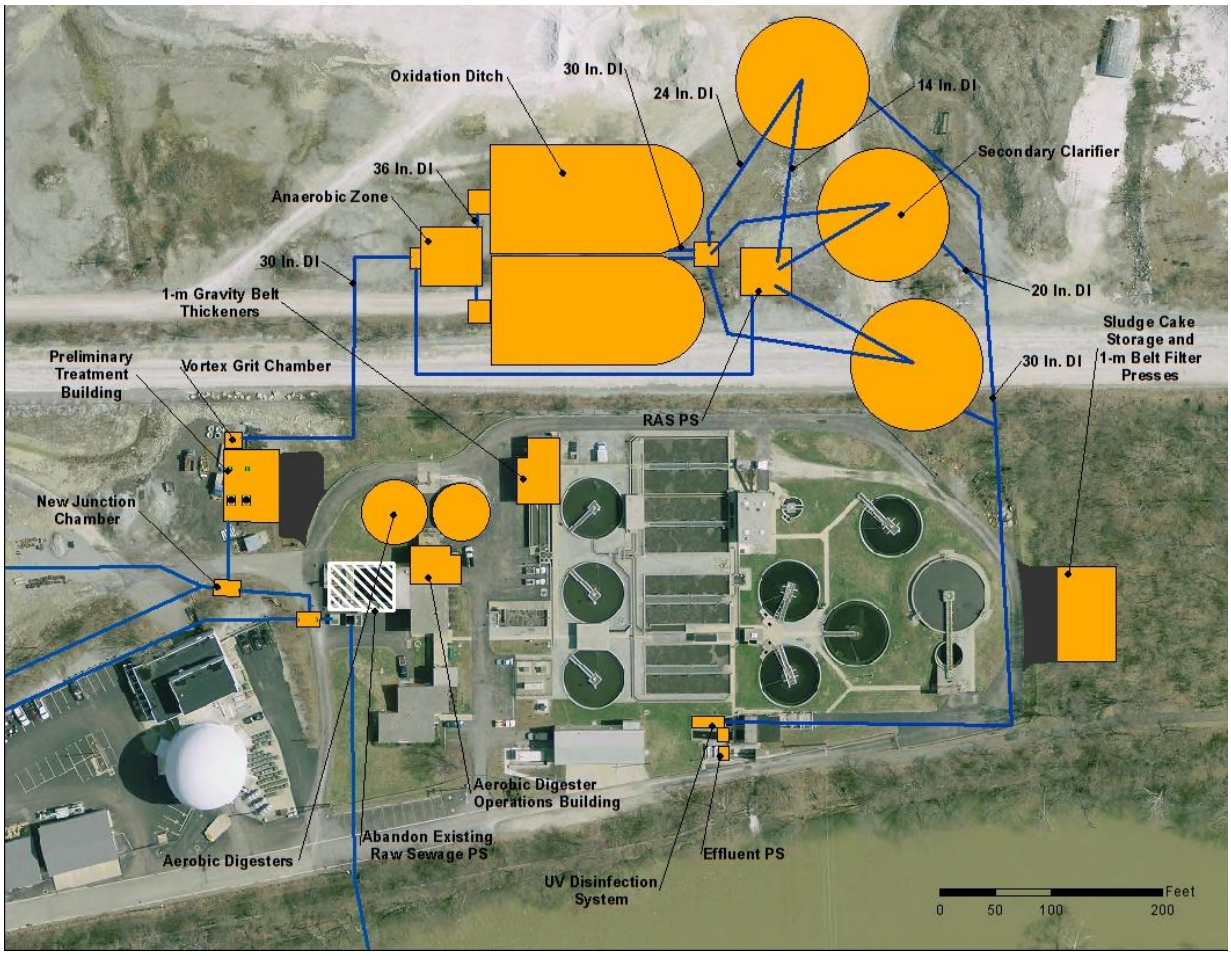


Figure ES-2: Conceptual Layout of 7-MGD Oxidation Ditch Process with Aerobic Digesters

Additional WWTP improvements and/or operational maintenance are also necessary for the existing plant to maintain its treatment performance and meet regulatory requirements as long as the existing facilities stay in service. The following unit processes should receive needed maintenance, repairs, and investment to keep the WWTP operating successfully until a new WWTP is online:

- Gear box repairs for two of the influent screw pumps (currently underway)
- Replacement of the mechanical fine screen and manual bypass screen to comply with biosolids regulations (currently underway)
- Installation of new air flow meters and DO analyzers and better control of the internal recycle mixers within the aeration tanks for improved control and efficiency in aeration, and maintenance of a distinct anoxic zone within these tanks (serving the existing WWTP only)
- Repair of the flow-control gates in the flow diversion chamber upstream of the secondary clarifiers, allowing more positive control of flow to the clarifiers and the capability to isolate each clarifier for inspection and maintenance (applicable to the existing WWTP only)
- Replacement of the effluent flow meter to pace disinfection (before and after proposed WWTP improvements) and for compliance with the City's NPDES permit

Additional details for other improvements and preliminary design criteria for the recommended unit processes will be developed as part of a Preliminary Engineering Report (PER), the next phase of this project. The Project Schedule for implementing the recommended treatment plant improvements is provided in **Table ES-9**.

Table ES-9 – Proposed Implementation Schedule

Activity/Milestone	Approximate Dates	Months
Ohio EPA Amended WWTP Facility Plan Approval	9/2014 – 12/2014	3
Preliminary Engineering Report	12/2014 – 6/2015	6
Ohio EPA Preliminary Engineering Report Approval	6/2015 – 9/2015	3
Detailed Design	10/2015 – 4/2017	18
Ohio EPA PTI Approval	5/2017 – 10/2017	5
Advertise for Bids	11/2017 – 12/2017	1
Award Construction Contract	2/2018	1
Begin Construction	3/2018	1
Construction Period	3/2018 – 2/2020	24
NPDES Milestone to Eliminate SSO	2/28/2020	

Section 1
Introduction

1.1 Project Background

In April 2013 the City of Piqua, Ohio, (City) completed the Final Report of the Wastewater Treatment Plant (WWTP) Facility Plan, which recommended WWTP improvements to enable the City to meet regulatory requirements established by the Ohio Environmental Protection Agency (Ohio EPA), and to provide treatment for increased wastewater flows that were projected as a result of residential, industrial, and commercial growth throughout the City. The most significant regulatory requirement driving the completion of the 2013 WWTP Facility Plan was the elimination of the City's constructed sanitary sewer overflow (SSO), a requirement of the City's National Pollutant Discharge Elimination System (NPDES) Permit and enforced by Ohio EPA.

The 2013 WWTP Facility Plan included evaluations of several wastewater treatment processes and solids treatment processes. After initially considering multiple treatment technologies, four treatment processes were short-listed for the liquid treatment train and four solids treatment processes were also short-listed for further evaluation. The evaluation phase of the 2013 Facility Plan project was concluded with the recommendation of upgrades to the existing WWTP to reliably treat up to 7.0 million gallons per day (MGD) of wastewater during average-day flows and some wet-weather events, and the addition of a high-rate treatment process, BioActiflo®, to treat up to an additional 6.0 MGD of wastewater during wet-weather events, contingent on successful completion of BioActiflo® pilot testing. A second flow equalization (EQ) basin and an EQ Pump Station were also recommended, which would provide total EQ storage of 6.0 million gallons (MG).

After completion of the 2013 WWTP Facility Plan and approval by Ohio EPA, the City agreed to enter a period of Pilot Testing of the BioActiflo® process. Though the non-biological Actiflo® process had been approved and implemented at many locations across the country to treat excess stormwater, Ohio EPA had not approved the BioActiflo® process to treat influent domestic wastewater. So the pilot testing phase would serve to verify the process's capability to treat domestic wastewater in an ongoing basis as the result of a high-flow event in Piqua. Piqua carried out the pilot testing in 2013.

Following the pilot testing that included varying the biomass feed rate, polymer dosage, and other process variables, the project team reached conclusions about the viability of BioActiflo® to treat wet-weather flows at the Piqua WWTP. As documented in the BioActiflo® Pilot Testing Basis of Design Report, completed in February 2014, project team members did not believe that the BioActiflo® process could be successfully implemented to meet the City's treatment goals. It was believed that the consistently weak influent wastewater did not contain adequate biomass to sustain the BioActiflo® process. Further, the chemical-laden sludge that could be returned from the BioActiflo® process to the City's existing WWTP would not aid in biological treatment, compounding the weak biomass problem. These findings resulted in a rejection of the BioActiflo® process that had been earlier recommended in the 2013 WWTP Facility Plan.

The recommendations of the BioActiflo® Pilot Testing Basis of Design Report included a re-evaluation of the liquid treatment processes that had been considered in the 2013 WWTP Master Plan, as well as consideration of other liquid treatment processes that may have merit for Piqua. Further

consideration would be given to the needs and capabilities of the existing WWTP, and how proposed upgrades to the existing WWTP could impact liquid treatment processes associated with an expansion the plant's capacity. And because some solids treatment processes are more compatible with certain liquid treatment processes than others, it was agreed to re-evaluate the solids treatment processes in order to develop a comprehensive set of recommendations for WWTP improvements at Piqua. Thus, an Amended WWTP Facility Plan was recommended within the BioActiflo® Basis of Design Report.

1.2 Purpose of the Amended WWTP Facility Plan

Following the completion of the BioActiflo® Basis of Design Report, the City of Piqua, Ohio (City) authorized the development of this Amended Wastewater Treatment Plant (WWTP) Facility Plan. The project scope remains the same as for the 2013 WWTP Facility Plan – to identify the improvements necessary to provide sufficient treatment capacity through the design period, including an expansion of treatment capacity to successfully treat flows and loads conveyed to the treatment plant and to meet current and anticipated future regulatory mandates.

The specific project goals established early in the Amended WWTP Facility Plan project included the following:

- Eliminate SSOs
  - Plan for an increase in WWTP capacity to treat flows projected for the design year, 2030.
  - Evaluate and recommend treatment process that is cost-effective, reliable, and operationally efficient.
  - Determine appropriate EQ Basin capacity to meet wet-weather flow requirements.
- Provide flexibility to meet current and potential future regulatory requirements.
- Evaluate the cost-effectiveness of 'Class B' vs. 'Exceptional Quality' biosolids production.

Recent influent flows have approached or exceeded the plant's rated capacity of 4.5 million gallons per day (MGD). Average flow rates in 2011 and the first four months of 2014 were 4.69 MGD and 5.13 MGD, respectively, though average flow rates in 2012 and 2013 were less than 3.7 MGD. More detailed analysis of historic flow data is presented in this Section 1, and projected future flow rates are presented in Section 2.

Historic rainfall of 68 inches in 2011 led to prolonged wet weather conditions that stressed both the collection system and the treatment plant. The City has one constructed SSO that must be eliminated by February 28, 2020, according to the City's recently modified NPDES permit requirements. With the regulatory driver to eliminate the SSO and planning for potential growth in the sanitary sewer system, the current treatment plant's hydraulic capacity and treatment facilities will need to be increased or replaced to enable the City to continue to provide reliable service to its current and future customers.

As was done for the 2013 WWTP Facility Plan, the Amended WWTP Facility Plan includes the evaluation of multiple liquid treatment and solids treatment technologies that could replace or supplement the existing liquid and solids treatment processes. These are being considered to reduce capital and operational costs, improve treatment efficiency, or better position the City to meet future environmental regulations.

In addition to the direct benefits of a periodic update to facilities planning, many funding agencies require a plan such as this Amended WWTP Facility Plan to be prepared and approved as a contingency of award.

1.3 Update to State 208 Plan

In the late 1970s, the State of Ohio allocated sewer system responsibility through the Section 208 Water Quality Management Plan to various municipal and governmental entities to promote efficient and comprehensive programs for controlling water pollution from point and nonpoint sources in a defined geographic area. Each of these Facility Planning Areas (FPAs) was assigned a Designated Management Agency (DMA) that has approval authority of sewer system extensions to ensure they comply with the approved plans of that DMA. The City of Piqua is the DMA for the Piqua FPA, with geographic boundary as shown in **Figure 1-1**.

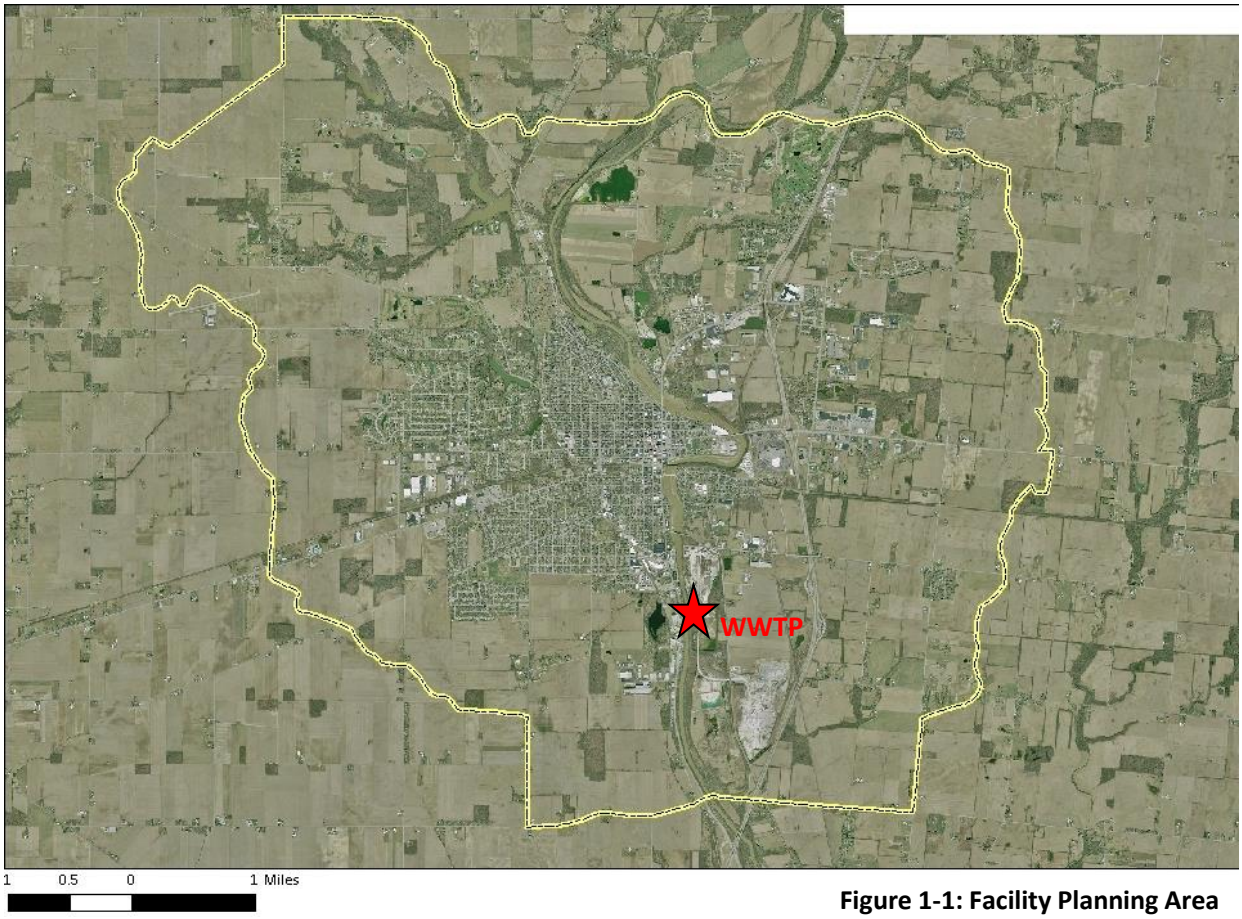


Figure 1-1: Facility Planning Area (MVRPC: Area Water Quality Management Plan, 2008)

1.3.1 Current and Future Service Areas

Although most sewer service is located within the City of Piqua's corporation limits, some unincorporated portions of Miami County are served by the City as well as the Village of Fletcher. Three main interceptors collect flow from the service area and convey flow to the Piqua WWTP near the southern boundary of the City at 121 Bridge Street.

To assess the future service areas over a 20-year planning period, CDM Smith evaluated data from several sources to understand the patterns of growth, likely system expansions, and industrial redevelopment within the City. These documents include:

- Miami Valley Regional Planning Commission (MVRPC) – GIS Data including population forecasts and future landuse shapefiles
- "Plan It Piqua – Redevelopment Analysis Report" dated April 2010
- "Plan It Piqua – Comprehensive Plan Update" dated 2007

The current and future sanitary sewer service areas are shown in **Figure 1-2**. Generally, system expansion and accompanying land use are anticipated to be commercial/industrial to the southwest; residential to the northwest; residential and commercial to the northeast; and industrial and commercial to the south. Future population projections indicate stable population within the current City corporation limits and modest growth on the periphery.

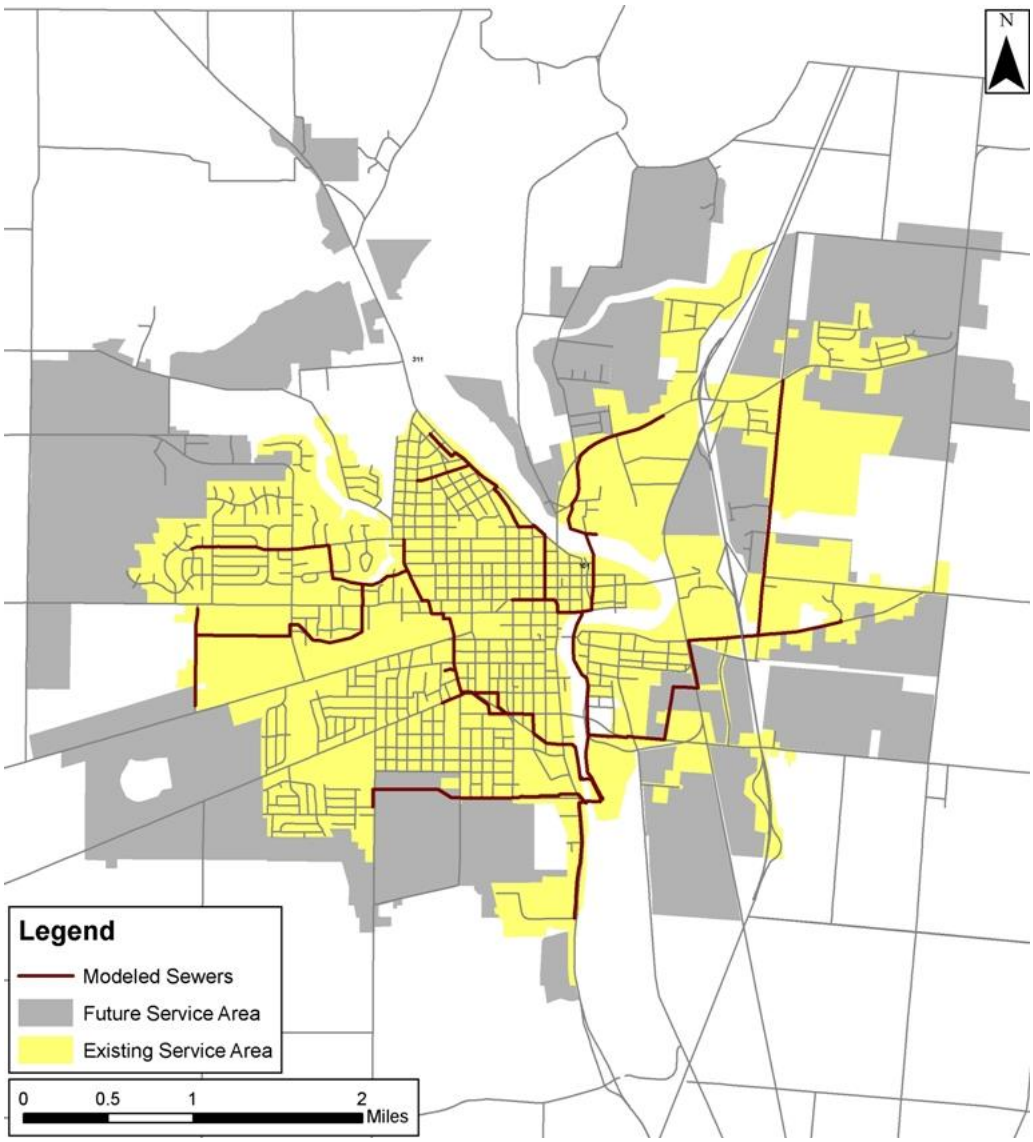


Figure 1-2: Current and Future Sanitary Sewer Service Areas

Additional sanitary sewer collection system infrastructure will be necessary to expand the system to these future customers. The City of Piqua's Sanitary Sewer System Master Plan completed in 2013 addressed the sewer system needs.

All of the contiguous planned sewer system expansions are anticipated to be within the current FPA, with some areas close to the FPA boundary in the far northeast extents and to the south of the City. These areas and any specific development plans will need to be reviewed in closer detail to ensure sewer service can be readily provided to them through the Piqua FPA.

The Village of Covington has presented the need for wastewater system improvements, with the possibility of conveying municipal wastewater to the City of Piqua's sanitary sewer and treatment system instead of upgrading its own sanitary sewer and treatment systems. The Village is facing wet weather management issues that led to SSOs and is considering options to either improve their systems or rely upon another FPA for service. This Amended WWTP Facility Plan gives consideration to potential future service to Covington and the related impacts this would have on the Piqua wastewater system. The Village is located approximately 5.5 miles west of the Piqua sewer system. Flow could be collected at a central location in Covington and discharged by pump station/force main into the City's 36-inch Hemm Road Interceptor for treatment at the Piqua WWTP. The impact of flow from Covington on the required treatment capacity of the Piqua WWTP is covered in Section 2.

1.3.2 Wastewater Treatment for Unsewered Areas

Several areas within Piqua's FPA are not currently connected to the municipal sewer collection system. These areas will continue to use home sewage treatment systems (e.g., septic tanks) to treat and dispose of their wastewater. Monitoring of these private systems will continue to be performed through the Miami County Health Department and assessed as necessary. The City does not currently accept septage for treatment and disposal, and does not plan to accept this waste in the future.

1.4 Existing WWTP Operational Assessment and Optimization

For the Amended WWTP Facility Plan, the City agreed to include an operational assessment and optimization of the existing WWTP. The goals of this task included the following:

- Gain a better understanding of the WWTP operations.
- Improve sludge settleability.
- Optimize the operation of treatment processes for continued use after the proposed WWTP upgrade, or for use until they are no longer needed.

CDM Smith provided an Operations Specialist who visited the WWTP, took part in one of the project meetings for development of this Amended WWTP Facility Plan, reviewed operational procedures and WWTP performance, and provided suggestions related to ongoing operation. The observations, suggestions, and recommendations of the operations consulting are presented in Technical Memorandum No. 1, enclosed in Addendum A of this Amended WWTP Facility Plan. Some of the issues and recommendations are re-iterated here.

Faced with a major WWTP upgrade or expansion, the City would like to maximize its investment in existing facilities to the extent practical. This approach could save capital costs related to WWTP expansion as well as minimize the time needed for the implementation of future improvements, resulting in the minimization of potential future sewer rate increases to the City's rate payers. The

assessment of the existing facilities was based on this premise, with the realization that only facilities suitable for continued use with higher flow rates would continue in service. For continued use of existing equipment and structures, the assessment included review of general structural condition, process equipment, electrical, and instrumentation components. For optimization, existing operating practices and results were analyzed and suggestions made to maintain or improve process efficiency and avoid common operational problems.

1.4.1 Initial Assessment of Existing Facilities

The project Kickoff Meeting for the Amended WWTP Facility Plan project served a dual purpose – initiating and explaining the goals of the project, as well as launching the Existing WWTP Operational Assessment and Optimization task. The CDM Smith Operations Specialist quizzed City of Piqua WWTP staff about specific operational issues, such as the methods used to promote and control aeration within the supernatant oxidation tank and the secondary aeration tanks, impacts of high flows and return sludge rates on solids retention in the secondary clarifiers, and positive flow splitting to the aeration tanks and again to the secondary clarifiers. At the end of the Kickoff Meeting, a walkthrough of each unit process followed, starting with the liquid stream process and continuing on with the solids stream process.

Based on subsequent reviews of plant operation and discussions with the plant superintendent, a draft technical memo was developed that was later expanded upon and became the basis of Technical Memorandum No. 1. The findings from the Operational Assessment and the site visit are described in Technical Memorandum No. 1 and also featured in the following pages. **Figure 1-3** presents the existing plant configuration and location of unit processes.

1.4.2 Existing Treatment Performance

The Piqua WWTP has successfully treated wastewater to meet NPDES permit requirements without violations. The figures at the end of this section show the past four years of historical conditions that the treatment plant has experienced and its treatment performance.

- Influent wastewater flow data demonstrate the seasonal variation of WWTP influent flow rates and the influence of infiltration and inflow (I/I) on the flow rate. During the monitoring period, dry weather flow ranged from approximately 2.0 to 3.0 MGD, with wet weather flows exceeding 8.0 MGD (see **Figure 1-13**).

- Influent five-day carbonaceous biochemical oxygen demand (CBOD) concentrations typically ranged from 100 to 200 mg/L (see **Figure 1-14**). A reading of 100 mg/L is relatively low for a municipal separate sanitary sewer system. Effluent CBOD generally ranged from 2 to 10 mg/L, with an average daily removal of 97.7%. Mass loading of influent CBOD is shown in **Figure 1-16**.

- Influent total suspended solids (TSS) concentrations typically ranged from 100 to 200 mg/L with an average of 130 mg/L (see **Figure 1-15**). Effluent TSS generally ranged from 2 to 15 mg/L, with an average daily removal of 96.5%. Mass loading of influent TSS is shown in **Figure 1-16**.

- Effluent nutrient concentrations for ammonia and total phosphorus demonstrate seasonal variance (see **Figure 1-17)**.

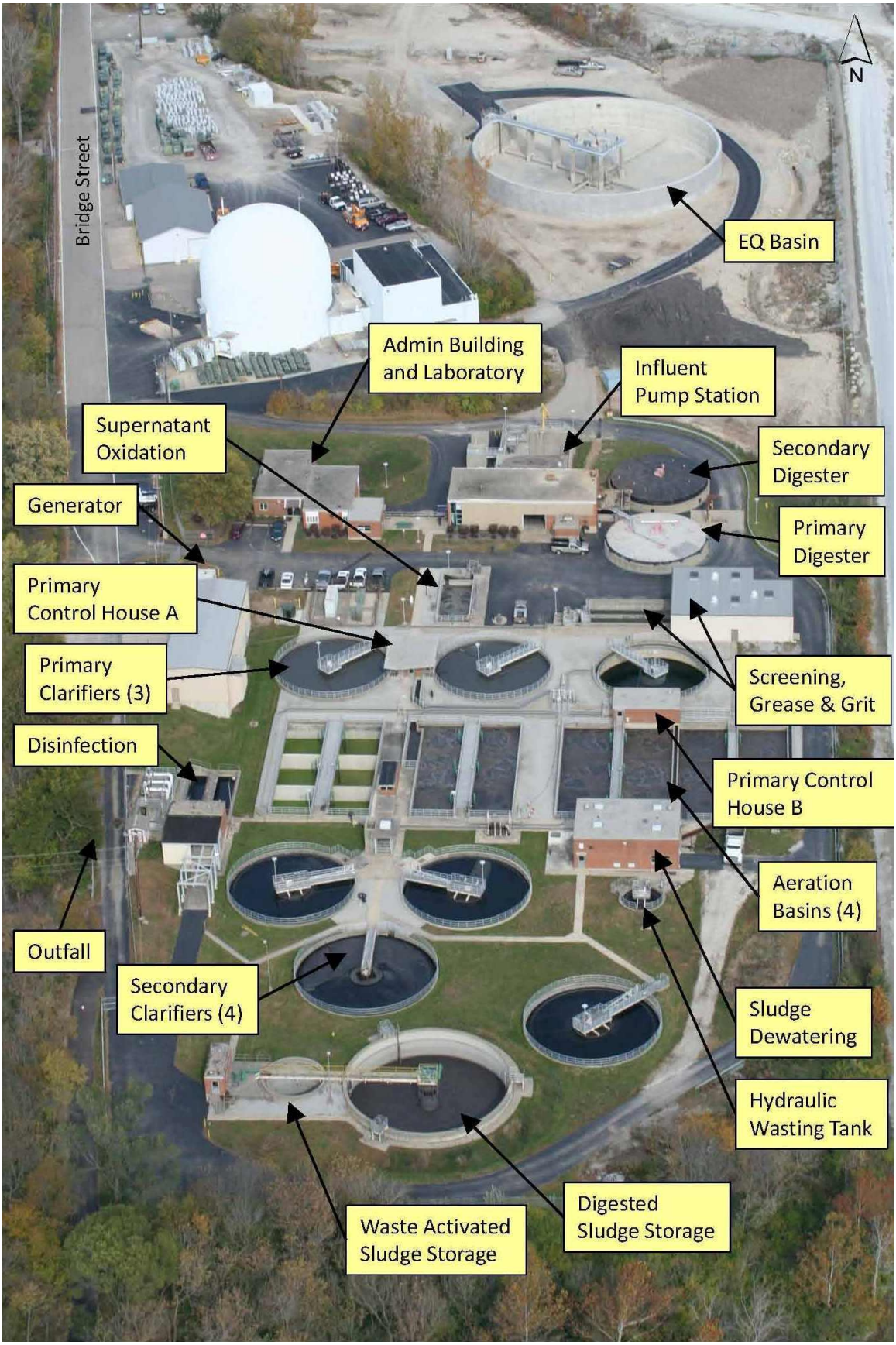


Figure 1-3: Existing Plant Configuration

1.4.2.1 Liquid Stream Process

Flow Equalization

Figure 1-4: Flow Equalization Basin

The flow equalization (EQ) basin was constructed in 2009 to store excess wastewater during wet weather events. The EQ basin stores wet weather flow for subsequent treatment, and thereby reduces the frequency and volume of SSO events. Influent wastewater enters the EQ basin from the 36-inch West Interceptor over a broad-crested weir, with a flap gate located in the diversion chamber. The influent gates at the WWTP are modulated during wet weather events to limit the flow through the WWTP to a magnitude that can be effectively treated, which raises the hydraulic grade line in the influent sewer, forcing raw sewage to flow over the weir into the EQ basin.

Design of the EQ basin allows for 1 MG of gravity-in, gravity-out storage by accepting flow from the higher-elevation West Interceptor and discharging to the lower-elevation 42-inch Miami River Interceptor. Because the basin's wall extends above the 100-year flood elevation, there is approximately 10 feet of freeboard. The EQ basin was designed to accommodate future pumping of wet-weather flow into the EQ basin, allowing the upper volume of the tank to be used for additional flow storage, and bringing the total EQ capacity to 3 MG in the future.

The EQ basin has four submersible pumps and a jet mixing header to circulate the liquid. The header includes provision for future air addition if necessary to increase mixing intensity or add dissolved oxygen into the stored wastewater to prevent it from becoming septic.

Raw Sewage Pumping

Raw sewage enters the existing plant through the raw sewage junction chamber. There are two feeds into the raw sewage junction chamber, the 42-inch Miami River Interceptor that enters from the north, and a 30-inch interceptor that enters from the west, accepting flow from the 36-inch West Interceptor and 36-inch Hemm Road Interceptor. The flow from these pipes is controlled by adjusting the position of the sluice gates with an electric motor actuator to a desired set-point in the junction chamber. As flow rates increase to the capacity of the raw sewage pump station, the sluice gates are modulated partially closed to utilize the in-pipe storage of the Miami River Interceptor.

Figure 1-5: Raw Sewage Screw Pumps

Once flow exits the raw sewage junction chamber, it passes through one of two coarse bar racks with 2.5-inch clear openings. Each bar rack is rated for a peak flow capacity of 8.3 MGD (the maximum-day capacity of the WWTP). The racks are manually raked; the raked material is placed in 55-gallon drums stored next to the bar racks and then subsequently hoisted from the pump station to the surface with a jib crane for disposal. Downstream of the bar racks, flow is routed to one of three raw sewage pumps.

The three raw sewage pumps (lead, lag and back-up) are enclosed screw pumps, each designed for a flow rate of 4.2 mgd. The number of pumps operating at a given time is determined by the flow rate entering the treatment plant. Although each pump was sized identically, each pump has exhibited different flow capacities. The pump most capable of pumping near its design rate is Pump #2, which can reportedly pump up to 4 MGD. If Pump #2 is out of service, the actual pumping capacity is approximately 7.8 MGD.

Condition Assessment

The raw sewage pumping station could benefit from numerous improvements. If the pump station is to continue in service, the first recommended improvement is to increase automation at the raw sewage junction chamber. Automation of these gates to a controlled set-point would allow the plant to better utilize storage within the system, and avoid manual control.

The coarse bar racks present several operational challenges to treatment plant staff. To manually clean the bar racks, plant staff must monitor the racks for debris accumulation and then rake off debris and place it in a container. This is challenging in the cold months because debris often freezes on the racks, which blocks flow through the racks and increases the head loss through the screens. Another challenge is removing the raked material from the pump station. Filling the 55-gallon drums with screenings and removing them with the jib crane to place the screenings in a dumpster is very labor intensive and potentially hazardous for plant staff. This process could be eliminated with mechanically-cleaned bar screens.

The raw sewage screw pumps are experiencing several issues due to the age of the pumps. The pumps have cracked barrels, and welding repairs have only been partially successful, and not addressed the interior sides of these cracks. These pumps have also been repaired several times in recent years to correct the rotating assembly at considerable cost. There are additional leaking issues at the top of pumps (in the operating building). Beyond the pumps' current structural condition, none of the pumps is capable of meeting its original design capacity and certainly will not have the capacity to meet increased flow demands with an upgraded plant. In 2014, both screw pumps number 1 and 2 had to be taken out of service to repair their gear boxes, demonstrating the ongoing maintenance cost that these aged pumps represent.

In summary, nearly all of the equipment in the raw sewage pump station is in need of improvement. The screw pumps are beyond their serviceable life and do not have the capacity to meet increased future flows. The coarse bar racks upstream of the pumps present labor intensive operations by plant staff.

In a more ideal configuration, a treatment plant would handle screenings in only one location, and currently Piqua removes screenings at two locations (one with manual raking). A consolidated screenings process is evaluated in the alternatives analysis, along with options for a new raw sewage pump station and an improved screenings process capable of meeting projected future flow rates.

Figure 1-6: Screenings and Grit

Fine Screening and Grit and Grease Removal

The screening and grit/grease removal processes are

located within a partially enclosed building. Flow from the screw pumps is passed through a single mechanical screw with ¾-inch clear openings that is rated for a peak flow of 8.3 mgd. There is a bypass channel next to the mechanical screen with a static manually-cleaned screen for passing flow when the mechanical screen is out of service. The screenings removed by the mechanical bar screen are deposited in a washer-compactor.

Downstream of the screens is an aerated grit/grease tank. The tank has a traveling bridge with a suspended grit pump to pump accumulated grit from the bottom to a de-gritting auger. A skimmer moves accumulated grease on the surface to a dumpster. The grit/grease tank has a volume of 29,330 gallons and is rated for a peak flow rate of 8.3 MGD. The dewatered screenings, grit, and grease are disposed into three 2-cubic-yard roll-off dumpsters for landfill disposal on a weekly basis.

Figure 1-7: Grease and Aerated Grit Channel

Condition Assessment

The building in which the equipment is located is not completely enclosed. The west side is open to allow the traveling bridge into the building. This leads to freezing within the screening and grit equipment, which results in to diminished capacity and operational efficiency. Attempts to shield the building opening from the westerly prevailing winds with plastic sheeting have been unsuccessful. Another issue related to the building is leaking skylights that has caused water damage on the roofing system.

The ¾-inch clear openings on the existing mechanical screen are not compliant with the updated 503B sludge regulations. The regulations require finer screens with maximum 5/8-inch clear openings for more effective solids removal that would allow biosolids to be land applied. In 2014 the City ordered a new mechanical screen with 5/8-inch openings for installation during the development of this Amended WWTP Facility Plan.

An intermittent issue with the grit equipment requires the traveling bridge in the aerated grit channel to be manually operated to return it. When the problem arises, it only travels one direction. At times, this equipment has periodic challenges with outdoor operations in freezing weather.

Alternatives for improvements to the screenings and grit/grease removal processes and the screenings building are evaluated in the alternatives analysis.

Primary Clarifiers

Flow is distributed from a diversion chamber to three circular primary clarifiers. The three primary clarifiers are each 55 feet in diameter and have a side water depth (SWD) of 12 feet. The primary clarifiers have a combined peak-flow capacity of 8.3 MGD at a surface overflow rate (SOR) of 1,165 gallons per day per square foot (gpd/sf.).

Figure 1-8: Primary Tank Distribution Box

Under normal flow conditions, one tank is taken off-line to maintain a sufficient sludge blanket in the other two

tanks. The off-line tank is typically brought on-line during wet weather events when flow ratess exceed 5.0 MGD.

Condition Assessment

Generally, the three primary clarifiers are in good condition. However, there is a hydraulic bottleneck associated with their effluent channels that causes the effluent weirs in primary clarifiers no. 1 and 2 to become submerged during period of high flow rates. A means of eliminating this bottleneck through improved hydraulic structures is needed.

Primary clarifier no. 2 was taken out of service in July 2014 for inspection and cleaning. It was found to be in generally good condition, including the sludge collector mechanism.

Operationally, primary clarifier no. 3, even though it is the newest tank, is the most difficult to operate. The plant staff report the sludge withdrawal slip tube is more difficult to use than the tubes in the other tanks and there have been issues with sludge thickening in the sludge withdrawal line.

Aeration Tanks

Flow from the primary clarifiers is routed to a diversion chamber, with gates that control flow to four rectangular aeration tanks. The elevation difference between the primary clarifiers and aeration tanks is too little to allow for positive flow splitting via fixed weirs. Flow split to each aeration tank is dictated by the hydraulics of the open channels to each tank and the inlet gate at the diversion chamber.

Figure 1-9: Aeration Tank

Each aeration tank is configured with a forward-return pass and tapered air addition through six cells. The aeration tanks are 25 feet wide and each pass is 76 feet long and 15 feet deep. The total volume under aeration is 1,645,820 gallons. The aeration is provided by fine-bubble diffusers. All the diffusers were recently retrofitted with SSI diffusers. The air is provided to the diffusers by three centrifugal blowers (designed for two in service and one standby) each rated for 2,850 SCFM. Only one blower is required to operate under current flow and loading conditions. Two of the blowers are driven by electric motors and one by a digester gas-powered engine. The engine has a complete heat recovery package (exhaust and jacket cooling water) to assist in heating the primary digester.

Condition Assessment

The air supply from the blowers is more than adequate, and serves all treatment plant air needs including aeration, aerated grit/grease tank, supernatant oxidation, and post-aeration processes. The plant actually wastes air through over-aeration (D.O. in excess of 2 mg/l in the aeration tanks). The blower runs in a throttled, or “choked”, position to reduce the amount of air supplied to better meet the overall process needs. The plant utilizes one blower that is throttled back, and has never needed to utilize a second blower.

In 2014 the blowers experienced control problems, with the automatic control loop not working properly, blower valves slamming, and blowers going into surge.

The treatment plant sees occasional elevated ammonia concentrations, although plant effluent has not exceeded NPDES permit limits. The plant does not have permanent operational dissolved oxygen meters for process control, which could be used to control blower operation and stabilize ammonia removal. This has resulted in inefficiencies and excess aeration. New air flow meters and D.O. analyzers could bring the efficiencies needed by transmitting D.O. concentrations to the SCADA system, which could control blower operation. Better control of the internal recycle (IR) mixers could also contribute to improved control of anoxic conditions in the aeration tanks.

Currently, the blowers are oversized and approaching the end of their useful lives, given their age and overall efficiency. Blower capacity and type of aeration are evaluated in the alternatives analysis section.

Secondary Clarifiers

Flow from the aeration tanks is routed to a diversion chamber with gates to control flow to four circular secondary clarifiers. Three of the tanks are 55-feet diameter by 12-feet deep and the fourth is 55-feet by 10-feet deep. The total peak-flow capacity of the secondary clarifiers is 8.3 MGD based on a SOR of 873 gpd/sf. The elevation difference between the aeration tanks and secondary clarifiers is too little to allow for positive flow splitting via fixed weirs. Flow split to each secondary clarifier is dictated by the hydraulics of the piping to each tank and the inlet gate at the diversion chamber.

Figure 1-10: Secondary Tank Diversion Chamber

Condition Assessment

Several of the gates in the diversion chamber are inoperable, which hampers their ability to provide a positive flow split to the secondary clarifiers. The gate shaft stems are bent severely on the isolation gates for secondary clarifiers no. 1 and 3, making them inoperable. The inoperable gates also make access for any repairs nearly impossible without temporary bulkheads or bypass pumping. The secondary clarifiers have not been dewatered in 10 years. It is important to repair the diversion chamber gates to provide the ability to isolate each of the secondary clarifiers for inspection and cleaning, as necessary, and to provide a positive flow split, as long as the existing secondary clarifiers are kept in service,

Secondary settling tank #3 has an outdated hydraulic sludge draw-off system and only 10-feet SWD. The other tanks are deeper with 12-feet SWD, allowing better control of the sludge blanket and prevention of solids overflowing the weirs.

The alternatives analysis section of this Amended WWTP Facility Plan evaluates options to improve the flow split into the secondary clarifiers and other improvements to enable treatment plant staff to maintain the process equipment and improve performance, as long as these secondary clarifiers remain in service. These improvements include upgrading influent flow dispersion with energy-dissipating influent baffling, Stamford baffles to further prevent solids overflowing the effluent weirs, and weir and scum baffle brush cleaning systems to prevent build-up of long stringy algae that can

plug the flushing water system or blind the UV system. These improvements are especially important for the shallowest secondary clarifier #3.

Disinfection

Effluent from the secondary clarifiers is routed to the disinfection process, where it is disinfected with gaseous chlorine, and then de-chlorinated with sulfur dioxide. The chlorine gas is dosed at a fixed rate that is manually adjusted based on the WWTP influent flow rate. The single chlorine contact basin has a volume of 87,490 gallons, providing a maximum treatment capacity of 8.3 mgd based on 15 minutes of detention time, the minimum requirement according to design standards.

Condition Assessment

The treatment plant had no issues meeting its former permit limit of 1,000 CFU of fecal coliform/100 mL. There were concerns about the existing disinfection system being able to meet the new permit limit of 126 CFU of E. coli/100mL. Throughout 2014 the plant has been able to comply with the new E. coli limit, although elevated concentrations are observed immediately following wet weather events.

There is no reliable method for flow-pacing the chlorine dosage because of a non-functioning effluent flow meter. Regardless of the disinfection system recommended in the alternatives analysis, a new effluent flow meter will be needed. Treatment plant staff have been certifying monitoring reports to Ohio EPA based on influent flow rates, which are not truly representative of effluent flow rates, considering all the internal process recycle flows and the treatment of stormwater drainage from the plant site. Effluent metering should be re-implemented for permit compliance purposes and to be able to pace disinfection.

One-ton containers of chlorine gas are used for disinfection at the plant. These containers pose a significant safety hazard to operating staff and to the general public. There is a bike path adjacent to the chlorine building. Additionally, with a single basin, there is no redundancy to allow for periodic cleaning or maintenance of the basin without bypassing the entire disinfection process. Recent upgrades were completed to the chlorination and de-chlorination feed systems because of the new E. coli-based NPDES permit limits. Compliance with the new limits has been challenging at Piqua and other treatment plants throughout Ohio, with corresponding increases in chemical feed by as much as 50%. Treatment plants utilizing a UV disinfection system are having no problems meeting the new requirements.

Post-Aeration and Outfall Discharge

Following disinfection, treated effluent passes through an aeration basin to increase the dissolved oxygen concentration prior to being discharged through the plant's outfall pipe to the Great Miami River. Effluent can also be pumped into the Post Aeration Basin prior to flowing to the river during high river levels, when the gravity outfall is surcharged. There are three vertical mixed flow pumps, each with a rated capacity of 4.2 mgd (total firm capacity of 8.3 MGD). The pumps are tested monthly but are rarely used (once every 5 to 10 years).

Figure 1-11: Post-Aeration Basin

Condition Assessment

The treatment plant gets excellent dissolved oxygen (DO) transfer by this post-aeration process. The plant staff desire better and safer access to the effluent pipe to facilitate sample collection. The current method used for sample collection is grab samples. Options to improve sampling are evaluated in the alternatives analysis.

1.4.2.2 Solids Stream Process

Anaerobic Digestion

The treatment plant has anaerobic sludge digesters – a primary and a secondary digester. The primary digester has a fixed cover with a roof-mounted gas mixing system. The secondary digester has a floating gas-holder cover. Each digester is 50 feet in diameter with a side water depth (SWD) of 22 feet. Primary sludge pumps convey sludge from the primary clarifiers. Waste activated sludge (WAS) pumps convey unthickened WAS from the secondary clarifiers to the primary digester at a constant 10 gpm.

The digesters have experienced overloading due to the limits on disposal of the sludge by the State of Ohio. The new Ohio sludge regulations have restricted land application in winter months, which has in turn increased the need for sludge storage at the plant. When the sludge storage tanks are full, the only option for the plant operators is to decrease wasting activated sludge and increase the mixed liquor suspended solids (MLSS) and the sludge age. When this old sludge is introduced into the anaerobic digesters, foaming occurs. All solids treatment process alternatives will consider expansion and upgrade of solids treatment capacity related to thickening WAS, dewatering digested sludge, and land application for disposal of the sludge.

Condition Assessment

There are several issues with the digesters. The secondary digester gas-holder cover has developed holes in the side skirt and cannot function efficiently in retaining the methane gas. The primary digester occasionally releases foam from vents in its cover. It is thought that the secondary treatment process may contribute to this problem as a result of nocardia formation or from introducing un-thickened WAS into the digester. Thickening the WAS would reduce the hydraulic loading on the digester, provide increased detention time, and minimize the foam formation within the digesters.

The primary digester bubble gun mixing system was not mixing properly because the bubble generator was not functioning correctly in two of the three mixers. This poor mixing recently resulted in decreased gas production and contributed to the primary digester discharge pipe becoming plugged with solids. The decreased gas production also resulted in operational problems with the heat exchanger due to an insufficient supply of methane. Treatment plant staff supplemented the heat produced by the heat exchanger by tapping into the hot water heating system at the plant. The cause of the poor mixing was identified as a malfunctioning float valve that was preventing the bubble guns from getting a sufficient flow of gas. The repair to the malfunctioning float valve appears to have remedied the mixing issues, restored gas production, and improved heating of the primary digester.

All the gas safety equipment is approaching the end of its service life and is recommended for replacement. The plant has four gas meters and all of them were rebuilt in 2010 and work properly.

The options for improving the digesters are thoroughly evaluated in the alternatives analysis sections of this Amended WWTP Facility Plan. It is apparent that several modifications need to be made to the digesters to increase capacity, efficiency, and safety.

Biosolids Dewatering

The City currently contracts with Burch Hydro to provide and operate a belt filter press, truck dewatered solids to off-site storage stockpiles, and manage disposal to farm fields. The City provides the building, electricity, water, and access for Burch Hydro to conduct their contract operations.

Figure 1-12: Burch Hydro's Belt Filter Press

Condition Assessment

Sludge is dewatered twice a week, running at 100 to 200 gpm to draw down the sludge storage tank volume. The filtrate from the belt filter press is pumped to the supernatant oxidation tank. The supernatant oxidation system becomes overloaded when the belt filter press filtrate is sent to the supernatant oxidation tank along with digester supernatant. To keep this process under control, digester supernatant is not drawn from the digesters when belt filter press filtrate is delivered to the supernatant oxidation tank.

Waste Activated Sludge (WAS) was previously thickened with a solid bowl-type centrifuge. The centrifuge was removed as a result of required costly repairs. Consideration should be given to implementing another WAS thickening operation to reduce the water content being pumped to the digesters.

Supernatant Oxidation

The digester supernatant is the main source of influent to this process. As noted above, the tank also receives filtrate from the belt filter press when it is in operation. A small amount of primary effluent is also pumped into the basin to feed the biomass in the tank.

Condition Assessment

There is no dedicated blower to provide oxygen transfer to the supernatant oxidation process. This process uses a sidestream from the plant's main blowers that supply air to all needs.

Flow coming into this process enters as a slug discharge and is not attenuated as a stable flow pattern. This diminishes the effectiveness of the supernatant oxidation process, with corresponding negative treatment process impacts when the belt filter press is in operation and filter press filtrate is sent to the supernatant oxidation process. A holding tank or other means to provide a steady flow regime to this process would likely increase the effectiveness of this unit process and reduce the variability in detention time.

Wastewater Treatment Plant Site Stormwater Conveyance and Treatment

All treatment plant site stormwater is collected at yard drains throughout the treatment plant site and conveyed to the drainage pump station beneath the Operations Building. From there it is pumped to the mechanical screen influent box at the WWTP headworks via the influent screw pump force main. Thus, WWTP site stormwater is currently treated with the influent raw wastewater. The pumps at the drainage pump station have exceeded their useful life.

Future stormwater management should be based on a well-conceived Stormwater Management Plan that keeps relatives clean stormwater separate from the WWTP influent sewer. The plant drainage

pump station should be evaluated and sized to coincide with projected stormwater flow rates based on stormwater volumes predicted for the WWTP site, and equipped with a flow meter that would document the volume of stormwater collected and treated and/or discharged.

Supervisory Control and Data Acquisition (SCADA) System / WWTP Automation

The treatment plant is operated using three labor shifts, but is considering additional automation to allow the City to eliminate one or two shifts. The plant staff is interested in evaluating options for increasing automation at the plant through the SCADA system. The plant uses RSView 32 for its graphical package and Operator 10 for trending, reporting, and data analysis.

Condition Assessment

Most of the plant SCADA system is configured for monitoring purposes and not automatic control of processes. An operator is required to manually start/stop most processes, although several pumps have automatic start/stop logic and a logic loop controls the RAS flow.

The pH and temperature probes (GLI – Hach product) in the influent channel have experienced corrosion issues due to the harsh environmental conditions. The City would prefer to collect influent samples downstream of the fine screens and not upstream of them as is currently done.

The plant currently has several valves and gates that must be actuated locally. There are several valves and gates that are not operable due to service beyond their useful life. Replacing inoperable valves and gates should be included in the plant upgrade, and consideration for motorized actuators that can be monitored and operated remotely through the SCADA/HMI interface.

Pumps

All of the pumps at the treatment plant are nearing the end of their service life and need to be evaluated further as alternative treatment options are investigated. Additionally, these pumps must be evaluated for their ability to meet a higher flow condition that is anticipated, if they are deemed salvageable. Pump evaluations are included in the liquid treatment train alternatives evaluation section of this Amended WWTP Master Plan.

Treatment plant staff have stated that new submersible pumps are desired for use in a new WWTP Influent Pump Station to replace the screw pumps in the existing Influent Pump Station.

Condition Assessment

The supernatant oxidation return pump discharges upstream of the mechanical screen and has experienced plugging issues. The plant staff resolved this issue by placing a stand pipe on the discharge, with discharge holes in the upper reaches of the stand pipe to prevent plugging. Any changes to this return flow configuration will take into consideration the possibility of plugging.

The two digested sludge pumps baseplates are severely corroded. The pumps have been in operation for over 20-years and should be replaced.

The automatic strainer on the flushing water system (non-potable water) leaks. Additionally, the original galvanized steel pipe has several leaks throughout the plant, including some yard hydrants. This system will be evaluated for potential replacement.

The return activated sludge (RAS) pumps have been recently rebuilt. Valve actuators on the suction side of the pumps are broken and should be repaired/rebuilt. The pumps are throttled by valves on

the discharge line to meet a desired flow, but could be automated with variable-speed drives. The plant has never operated two RAS pumps at the same time and typically only one pump is operated at 55% capacity.

Plant staff thought a level sensor to monitor the sludge blanket in the secondary clarifiers would be beneficial over their current manual methods used to detect sludge depths.

Security

The plant is currently staffed 24 hours per day, which provides a level of security. However, as more automation is added to the plant, the possibility exists that the plant may not need to be staffed 24 hours per day. Several items will be evaluated to determine what, if any, improvements may be necessary to the plant for security purposes.

Condition Assessment

The plant is located along a public bike path, which was constructed in 2009. Although most of this traffic is pedestrian and bike traffic occurs during daylight hours when the plant is staffed, there is concern about having a more public and noticeable operation that could impact security measures.

Options for including additional fencing and lock/access policy will be evaluated. Closed circuit television (CCTV) cameras will also be considered to provide surveillance at potential access points.

Miscellaneous

In addition to above equipment concerns and issues, the following miscellaneous items were mentioned for consideration during the 2013 Facility Plan development.

Condition Assessment

Concrete floors in the Operations Building are wearing and have exposed reinforcing steel in some areas. These floors should be repaired.

In the Blower Building, plastic covers are used to protect the MCC from water damage due to a leaking roof on the building. The roof should be repaired or replaced to prevent water damage to the electric gear and reduce the safety hazard.

Removal of the blowers for repairs is a difficult process. Modifications to improve access to the blowers will be evaluated. The old MCC in the Blower Building needs to be replaced. The insulation on the original wiring is cracking and crumbling off the wire, making it a safety issue.

Primary tank #3 cannot be sampled from the auto sampler. Options should be evaluated to locate a sampling device.

The City would like to remove the underground fuel oil tank located near the digesters. Using fuel oil as an energy source is expensive, and the City would like to explore other cost-effective heating options. Natural gas supply from Vectren, the local gas utility, would be preferred if the plant cannot generate enough methane gas from sludge digestion to sustain its operations. There is currently no natural gas available at the plant site, though it could be extended from further north on Bridge Street. Vectren would have to construct a new gas line to the plant.

Conclusion

The treatment plant has many process and mechanical equipment items that are approaching or are beyond their useful lives. In order for future wastewater flows to be treated reliability and efficiently, these equipment items will need to be replaced or updated. The extent of improvements necessary for unit processes will be dependent on the identified design flows and selected treatment processes, which are covered in the following sections of this Amended WWTP Facility Plan.

A summary of unit processes and equipment that is expected to be replaced or significantly updated for use in an improved treatment plant is included in **Table 1-1.**

Table 1-1: Condition Assessment Summary of Major Improvement Needs

Unit Process	Component	Improve Condition or Operations	Need to Meet Permit/Regulations
Preliminary Treatment	Raw Sewage Pumping	X	
Preliminary Treatment	Main Drain Pumping	X	
Preliminary Treatment	Coarse Bar Rack	X	
Preliminary Treatment	Screening		X
Preliminary Treatment	Grease and Grit	X	
Aeration	Blowers	X	
Secondary Clarification	Secondary Tanks Influent Junction Chamber	X	
Secondary Clarification	Secondary Tank #3 sludge withdrawal	X	
Digestion	Digesters	X	X
Digestion	Waste Gas Flare	X	
Disinfection	Chlorination	X	X
Outfall	Effluent Flow Meter	X	X
Structural Conditions	Operations Building Concrete Floor	X	
Structural Conditions	Operations Building Roof	X	
Structural Conditions	Screenings Building Roof	X	

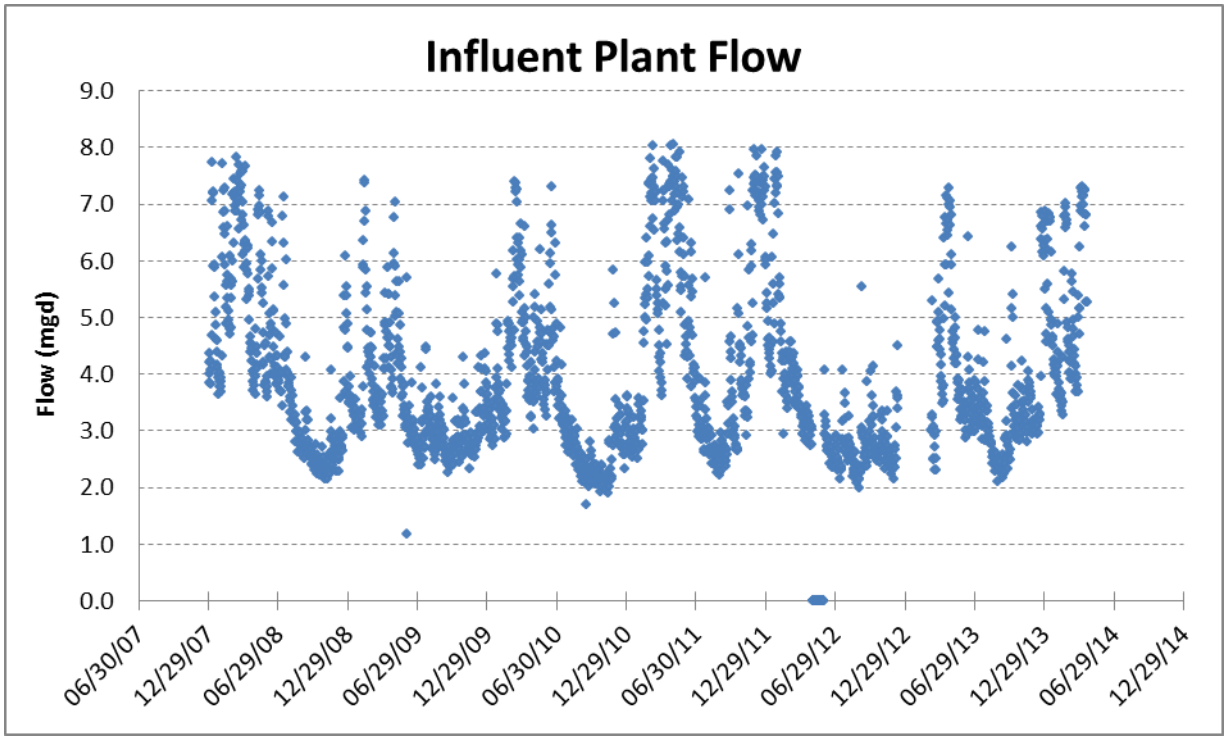
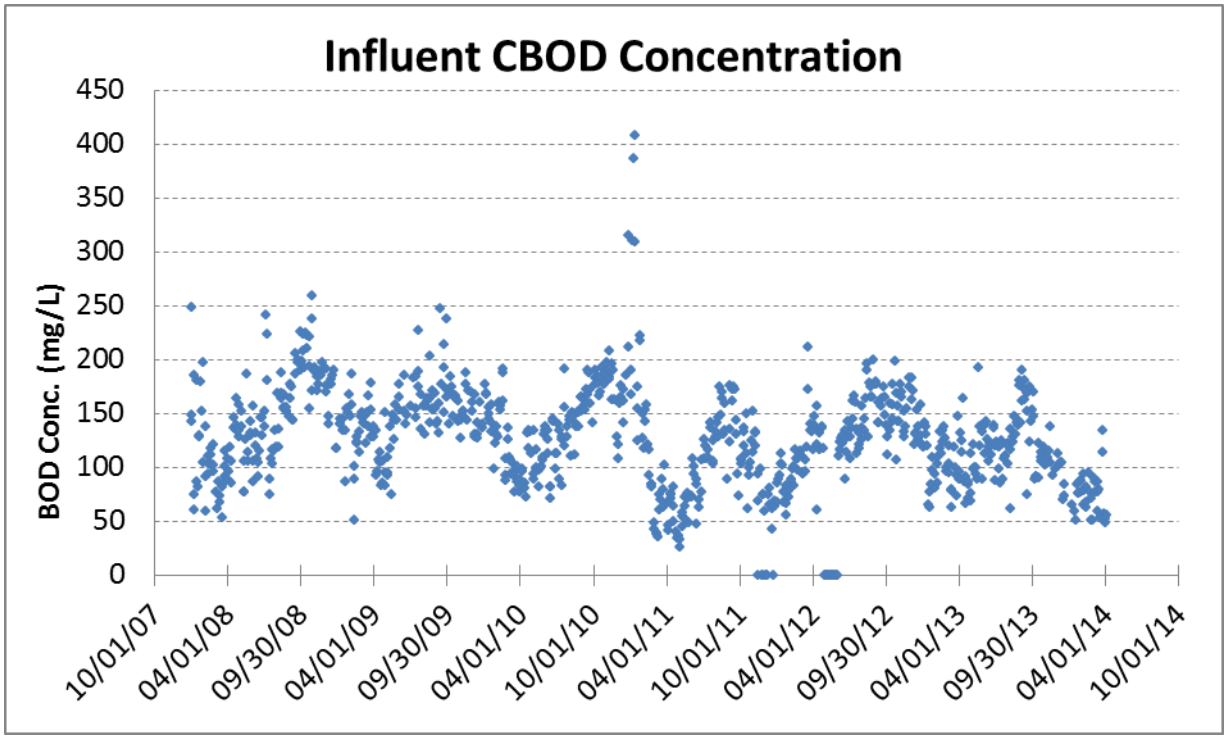


Figure 1-13: Historical Plant Influent Flow



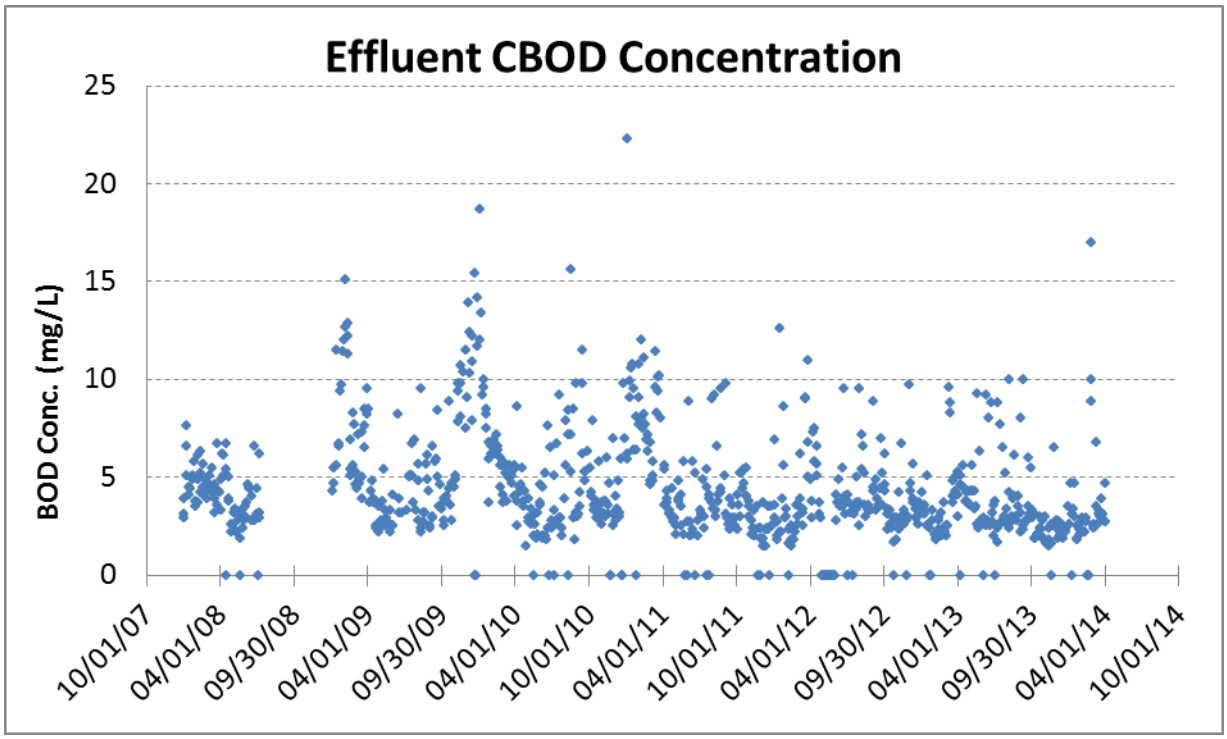
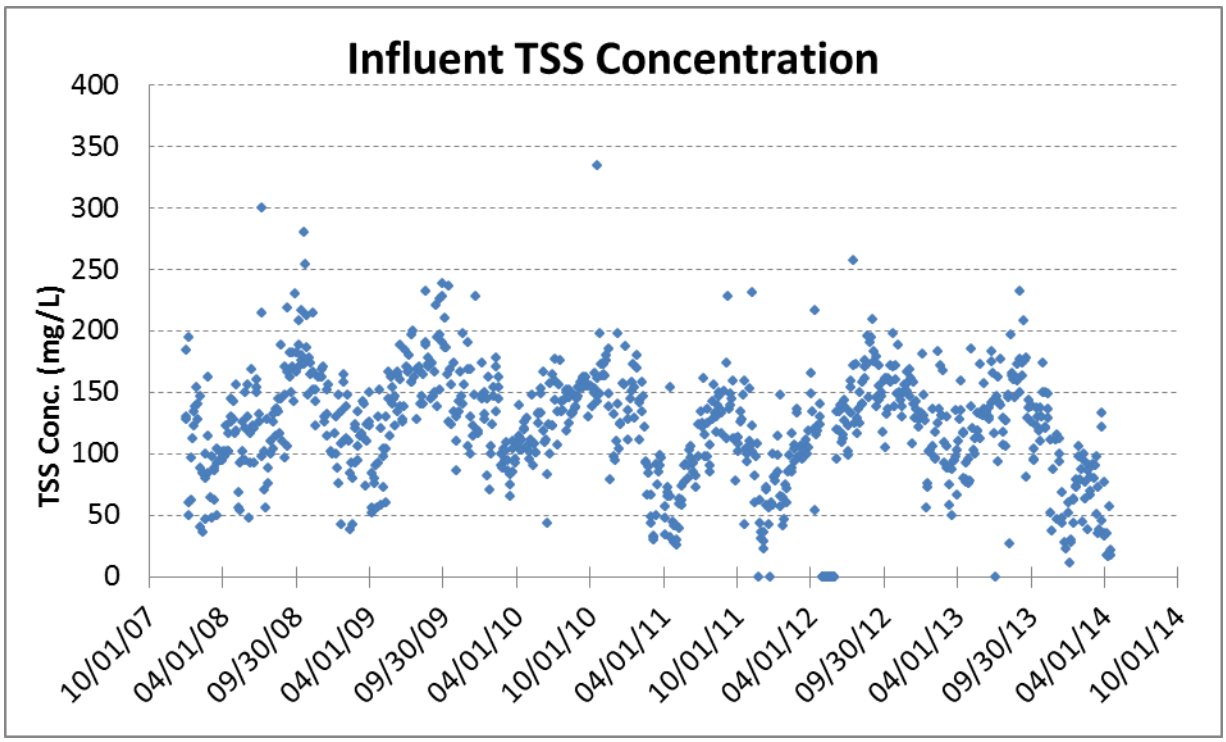


Figure 1-14: Historical Plant Influent and Effluent CBOD Concentrations



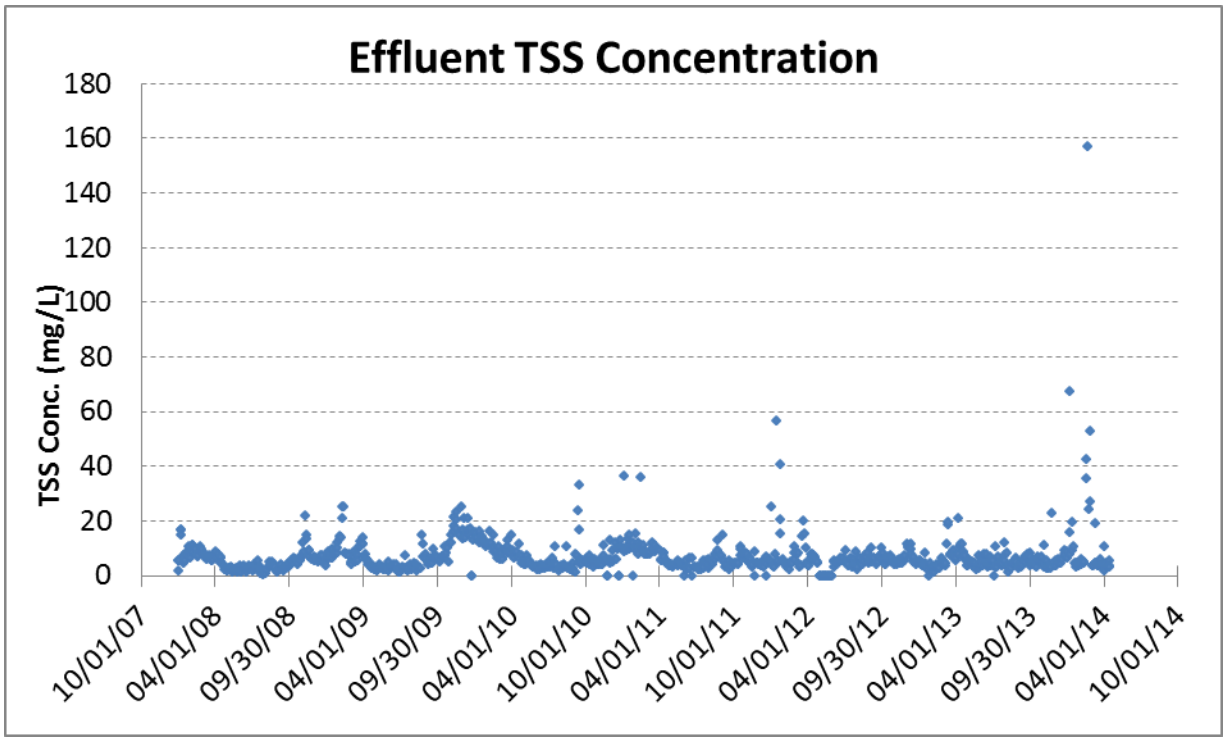
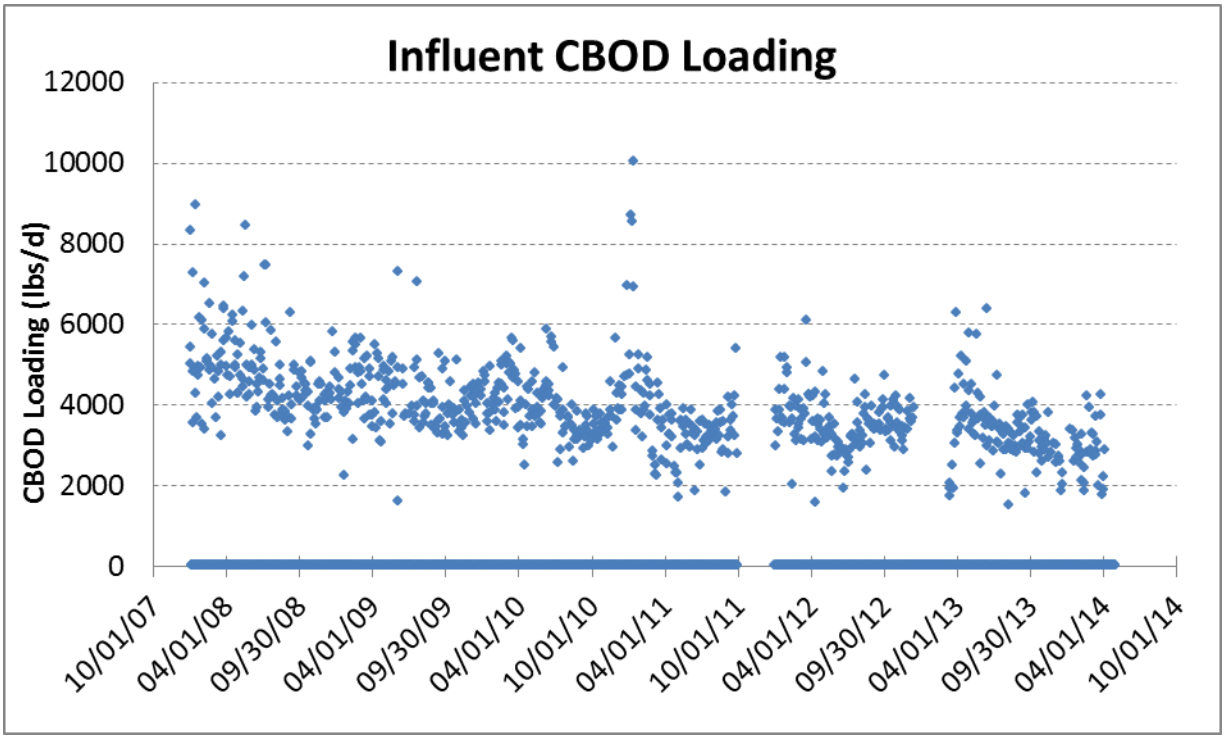


Figure 1-15: Historical Plant Influent and Effluent TSS Concentrations



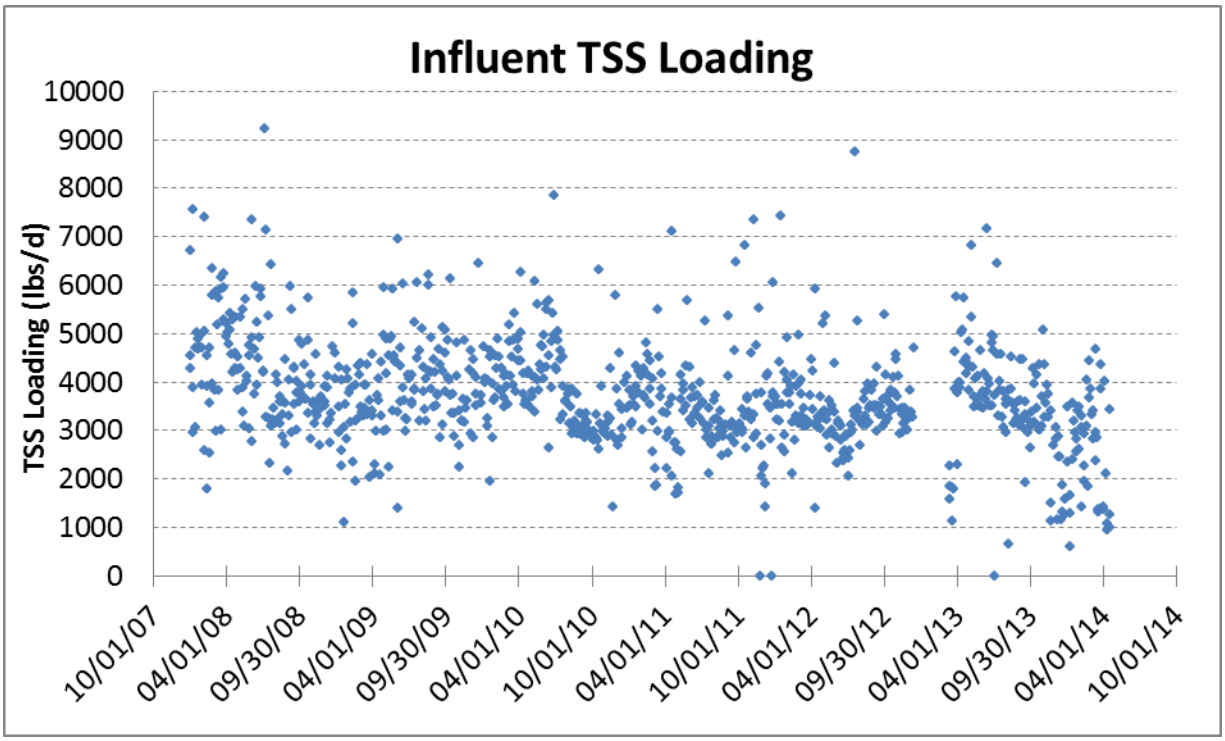
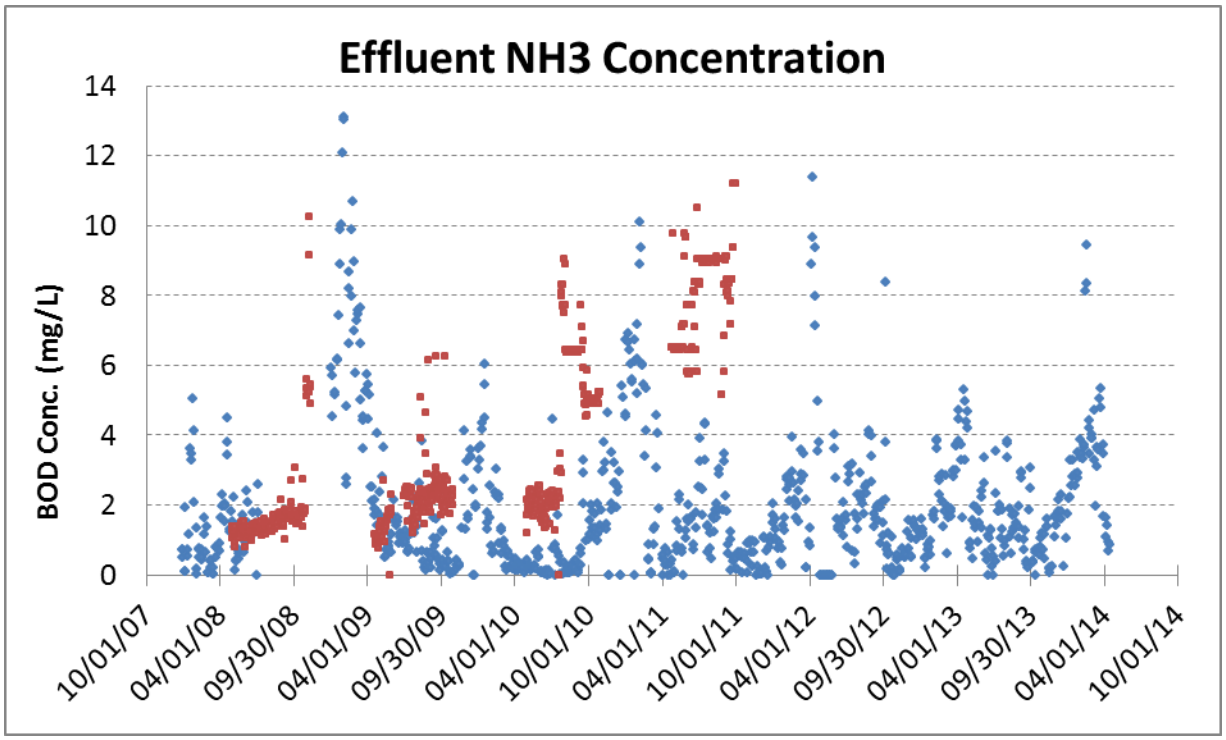


Figure 1-16: Historical Plant Influent CBOD and TSS Loadings



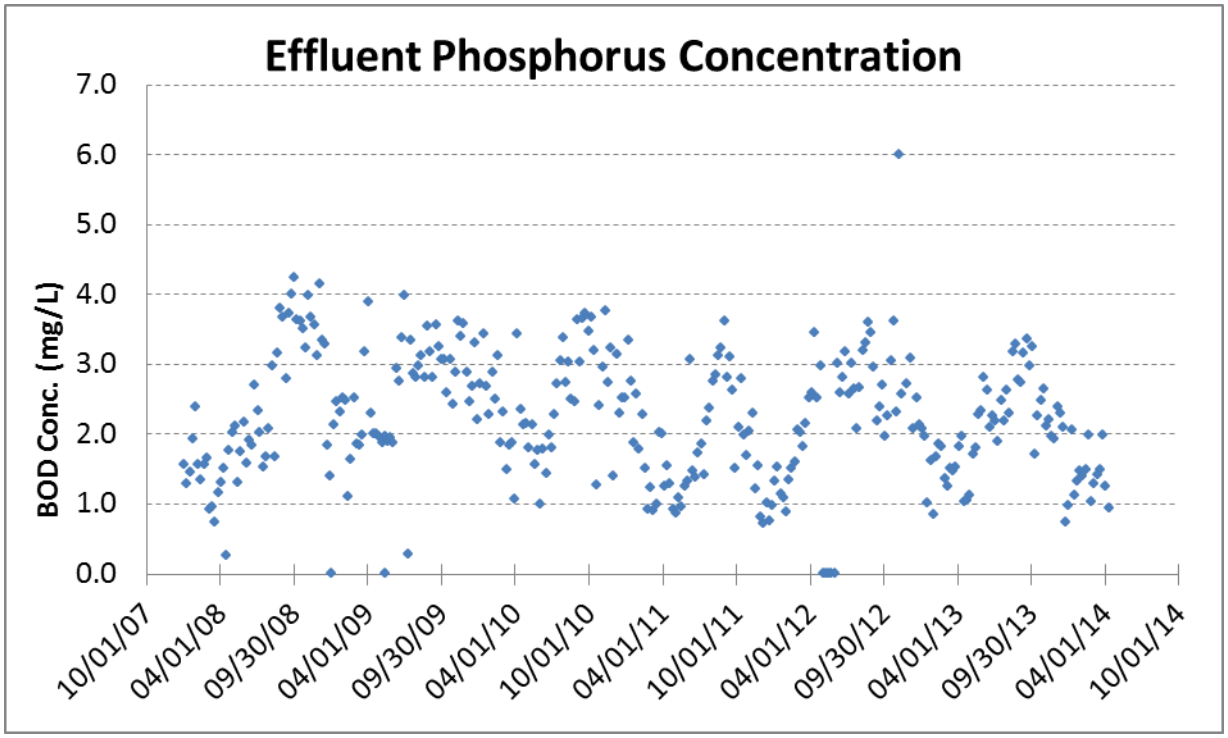


Figure 1-17: Historical Plant Ammonia and Phosphorus Effluent Concentrations

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Section 2

Future Treatment Capacity Needs

2.1 Forecasts of Population Growth

The City of Piqua (City) has had a steady population base of just over 20,000 people for the past several decades. Previous studies incorporated into the water distribution system and water treatment plant master plans identified nominal population growth within the City for the next 20 years. In addition, the City currently serves portions of unincorporated areas within Miami County that will likely expand within the 20-year planning period.

2.2 Treatment Capacity Projections

Like many communities, the City is planning for non-residential development and redevelopment of former industrial properties. These developments generally experience a higher water demand and have the potential to generate industrial wastewater with higher organic or nutrient loadings. The specific timeframe of these new and redeveloped properties is unclear; however, for the purpose of this Amended WWTP Facility Plan, it is anticipated to occur before the design year of 2030.

The Sanitary Sewer System Master Plan identified these properties and potential service area expansion, and quantified the associated additional wastewater flows that must be treated at the wastewater treatment plant. Using a ratio of projected water demands from the water treatment planning effort, water consumption is anticipated to increase 33% by 2030 throughout a similar service area as the sanitary sewer system service area.

The existing WWTP average-day design treatment capacity is 4.5 MGD. Using the same ratio as was used for the Piqua water treatment planning, the expanded wastewater treatment plant must be able to treat 6.0 MGD on an average-day, maximum-month basis.

Additional treatment capacity beyond 6.0 MGD will be needed to eliminate SSOs during wet weather, requiring a higher maximum-day plant capacity. Some of the wet-weather flow could be temporarily stored through additional flow equalization (EQ) volume, followed by treatment over a longer period of time after influent flows have declined, thereby mitigating SSOs. This Amended WWTP Facility Plant explores the cost-effectiveness of both alternatives – higher treatment capacity for wet-weather flows and additional EQ basin storage capacity for the same purpose. A third alternative, the combination of additional treatment capacity and additional EQ basin volume, may result in the optimum capital cost and lifecycle cost to the City.

As part of the Sanitary Sewer System Master Plan, CDM Smith performed continuous simulation modeling of Piqua's sanitary sewer system over a 50-year period of record rainfall data. This evaluation found that the month of April 2011, which had frequent and extensive rainfall, presented the worst-case scenario for planning purposes in terms of influent flow and volume to manage at the treatment plant. This same month of rainfall data was then simulated in the model environment under the design-year (2030) future conditions to quantify the combinations of treatment capacity and EQ basin storage that are projected to eliminate the SSO. The resulting curve of wastewater treatment capacity versus EQ basin storage volume is presented in **Figure 2-1**.

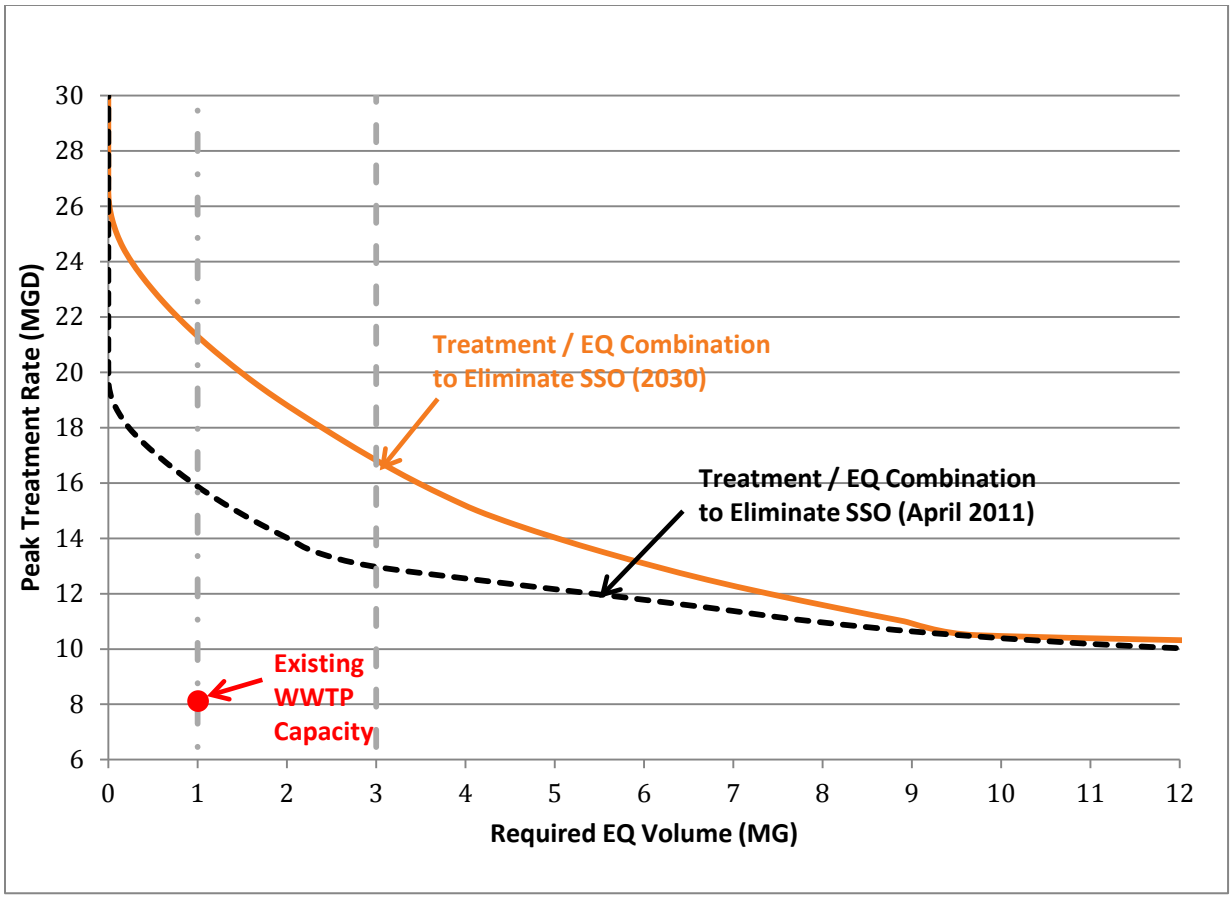

Figure 2-1: Combination of Treatment and Equalization Volume Necessary to Eliminate SSOs

From Figure 2-1, it is apparent that additional EQ basin storage capacity would not fully eliminate SSOs. Adding treatment capacity with the current 1 MG of EQ basin storage is feasible, but would require nearly triple the current maximum capacity with 21 MGD of treatment capacity necessary to eliminate SSOs in the design year, 2030. A more cost-effective combination of additional EQ basin storage and additional treatment capacity is needed. An evaluation of additional treatment capacity and EQ basin storage was then completed to find the optimum combination of peak treatment capacity and storage.

To evaluate the costs of the treatment and EQ basin expansion options, CDM Smith used the following planning level estimates.

- Capital cost of additional treatment from 4.5 MGD to 6 MGD average day = $9/gpd
- Capital cost of additional treatment beyond 12 MGD max day = $3/gpd
- Capital cost to expand EQ storage from 1 MG to 3 MG with influent pump station = $500,000
- Capital cost to expand EQ storage beyond 3 MG = $1,500,000/MG

CDM Smith then evaluated various combinations of treatment capacity and EQ basin volumes for the 20-year planning period using these unit cost factors. **Table 2-1** shows the various combinations of peak wastewater treatment capacities and EQ basin storage volumes and planning level costs. **Figure 2-2** presents the cost curve of these combinations.

Table 2-1: Planning Level Costs for Combinations of Peak Wastewater Treatment and EQ Storage

Scenario	WWTP Max Flow (MGD)	WWTP Avg Flow (MGD)	Max/Avg Ratio	Total EQ (MG)	Additional WWTP Cost	Additional EQ Cost	Total Cost
1	10.5	6	1.75	12	$13,500,000	$14,000,000	$27,500,000
2	11	6	1.83	9	$13,500,000	$9,500,000	$23,000,000
3	12	6	2.00	7.5	$13,500,000	$7,250,000	$20,750,000
4	13	6	2.17	6	$16,500,000	$5,000,000	$21,500,000
5	17	6	2.83	3	$28,500,000	$500,000	$29,000,000
6	21.5	6	3.58	1	$42,000,000	$0	$42,000,000

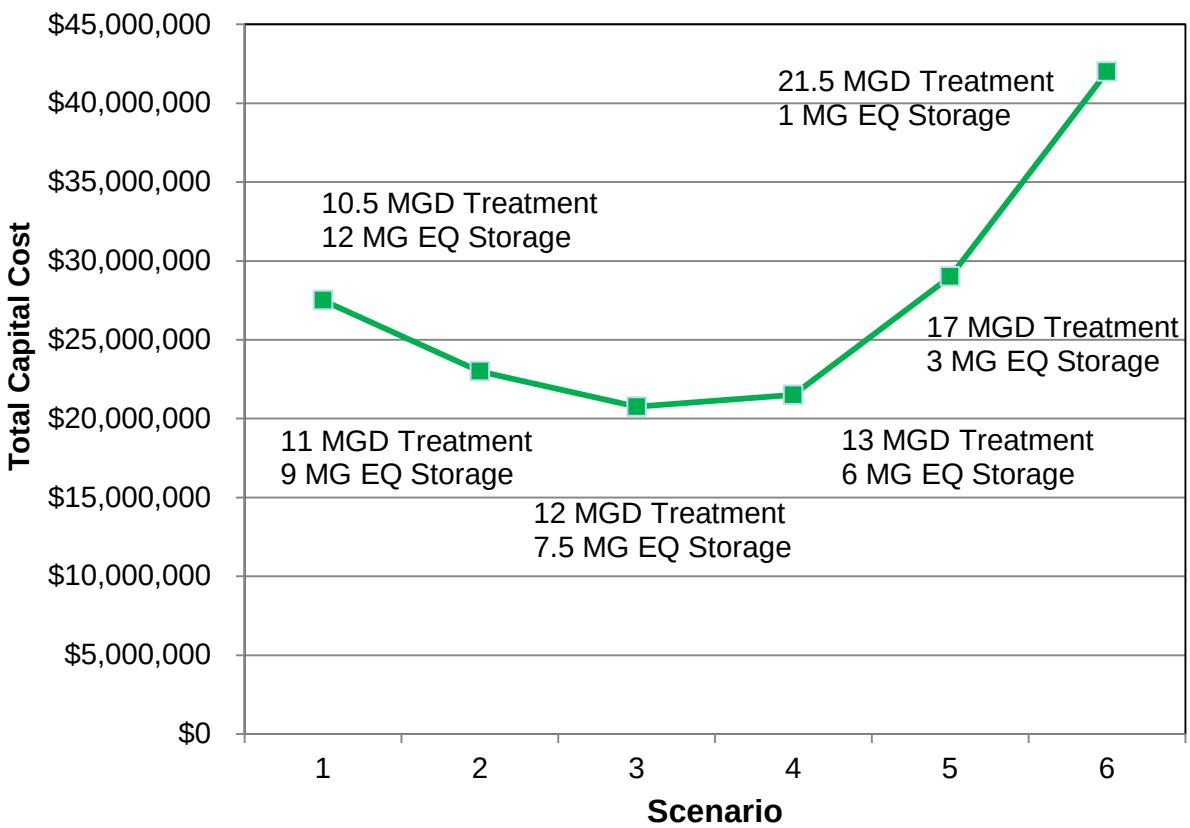

Figure 2-2: Capital Costs of Treatment and Equalization Necessary to Eliminate SSOs

The resulting cost curve indicates that a combination of treatment expansion and additional flow equalization storage provides the lowest capital costs to eliminate the SSO. At a planning-level capital cost of approximately $21M - $22M, either the 12-MGD treatment/7.5-MG EQ storage or the 13-MGD treatment/6-MG EQ storage combinations offer the most cost-effective solution.

The previous EQ basin project was planned so that a pump station and second EQ basin of the same size, 3 MG, could be constructed to provide 6-MG total storage capacity. If a second basin of 4.5 MG capacity was constructed, a total of 7.5 MG would be available, which corresponds to one of the treatment/storage combinations mentioned above. However, this option may require additional property acquisition to construct the larger basin. By comparing these two scenarios based on planning level costs, providing slightly more treatment capacity while maintaining the original concept of EQ storage volume is perceived to be more beneficial and implementable than additional incremental EQ storage volume. For these reasons, the combination of 13 MGD of peak treatment capacity and 6 MG of EQ volume is recommended.

As mentioned in Section 1, the Village of Covington has expressed interest about conveying its wastewater to Piqua's WWTP. Therefore, these future flows should also be included in the projected future flows for Piqua. Based on Covington's rated flow in the Village's NPDES operating permit, it was assumed an additional 1.0 MGD of average dry weather flow (ADF) would be conveyed to Piqua. This additional flow would affect all the treatment/storage options mentioned above equally, so this additional flow is added to the treatment flows mentioned above.

Therefore, the recommended treatment and storage capacities for the WWTP expansion are the following:

- **Average Design Flow = 7.0 MGD average day, maximum month**
- **Peak Design Flow = 14 MGD maximum day, maximum month**
- **Design EQ Storage = 6 MG total storage**

The cost optimization and WWTP sizing process described above was carried out to establish a basis for developing a treatment capacity and EQ basin storage volume to use in the evaluation of liquid treatment train alternatives for the treatment plant expansion covered by this Amended WWTP Facility Plan. The liquid treatment train alternatives are presented and evaluated in Sections 3 and 4 of this document, leading to a recommendation for design and construction.

Following the recommendation of liquid and solids treatment processes in Sections 3-6 of this Amended WWTP Facility Plan, the cost optimization described above is revisited with the recommended process as its basis. This is done to arrive at a final recommendation of treatment capacity and EQ basin storage volume that represents the lowest combined cost. The second cost evaluation, similar to the one presented in this section, is included in Section 7 of this Amended WWTP Facility Plan.

2.3 WWTP Effluent Limitations

The City maintains a NPDES permit, included in Appendix B, for wastewater discharge from the treatment plant. This permit contains effluent limitations of both concentration and mass loadings for various water quality parameters. The permit also requires associated sampling, monitoring and a reporting system to verify treatment performance.

2.3.1 Receiving Stream

Treated effluent from the wastewater treatment plant is discharged to the Great Miami River, which is a major river classified as an Exceptional Warmwater Habitat by Ohio EPA (OAC 3745-1-21). The river has recreational activity, including direct human contact.

The plant outfall operates by gravity under most conditions. During periods of high river water elevation, three vertical mixed-flow pumps are used to convey the treated effluent to the river. An auxiliary outfall located upstream of the low-head dam has been recently abandoned and discharge to this outfall is no longer allowable in the NPDES permit.

2.3.2 NPDES Permit Requirements

Ohio EPA issued a modification to the City's current NPDES permit with an effective date of August 1, 2014; it expires on January 31, 2016. Treatment performance standards and comparisons to the previous permit are presented in **Table 2-2**. Specific language is included that requires the constructed SSO to be eliminated by February 28, 2020.

Table 2-2: NPDES Permit Requirements

Parameter	Treatment Performance Requirements	Change fromPrevious NPDES Permit
SSO	None Permitted	No change
$CBOD_5$ Weekly/Monthly	Winter: 40/23 mg/L Summer: 23/15 mg/L	No change
TSS Weekly/Monthly	Winter: 45/30 mg/L Summer: 30/20 mg/L	No change
Nitrogen, Ammonia Weekly/Monthly	Fall/Spring: 13.5/9.0 mg/L Dec – Feb: 22.5/15.0 mg/L Summer: 4.4/2.9 mg/L	Summer slightly more stringent
E.Coli, Weekly/Monthly	Summer: 284/126 CFU/100mL	2,000/1,000 (Fecal Coliform)
Chlorine Residual Maximum	Summer: 0.035 mg/L	No change
Oil and Grease Maximum	Year-round: 10 mg/L	No change

2.4 Future Influent and Effluent Criteria

2.4.1 Future Influent Loads

The existing influent waste loading concentrations for CBOD, TSS, and ammonia to the WWTP have been dilute, based on industry standards. **Table 2-3** presents a comparison of the existing influent loadings to the plant from 2008 through 2011 against medium strength wastewater values (referenced from Metcalf & Eddy – Wastewater Engineering).

Table 2-3: Influent Loading Comparison

Parameter	Existing Loading Concentrations, mg/L	Typical Loading Concentrations, mg/L
CBOD	140	190
TSS	128	210
NH_3	11.5	25

The future influent loadings to the plant have been calculated using the existing dilute concentration for the current average design flow of the plant (4.5 MGD). The 2.5-MGD increase in average daily flow (1.5 MGD from Piqua, 1.0 MGD for Covington) was assumed to be loaded at typical loading concentrations. It was assumed the increase in average daily flow would be the result of industry,

which would have a higher-strength concentration than the existing loading. The calculation of the daily loading can be seen in **Appendix C**. The average daily loads are:

- Design influent $CBOD_5$ = 9,200 lb/day
- Design Influent TSS = 9,200 lb/day
- Design Influent NH_3 = 1,000 lb/day

2.4.2 Future Effluent Criteria

With the planned expansion, additional effluent flow will be discharged to the Great Miami River. The treatment plant has been assigned a Waste Load Allocation (WLA) from Ohio EPA for pollutant discharge to the Great Miami River. Higher flows at the same pollutant concentrations will lead to higher mass loadings. These future mass loadings must be compared to the plant's WLA to determine if improved treatment performance is required. An anti-degradation addendum will likely be required as part of a new NPDES permit negotiation for the higher flow requested. As an alternate to the Anti-degradation addendum, it is possible for the City to maintain the same mass loading at the higher flows by providing treatment to achieve greater removals. Based on proposed design flows of 7 MGD instead of 4.5 MGD, each parameter would need to be reduced such that the mass discharged remains the same. The discharge concentration limits would be as shown in **Table 2-4** for future conditions.

Table 2-4: Allowable Effluent Discharge Concentrations

Parameter	Concentration (mg/L)	Monitoring Timeframe
CBOD	14	Winter
CBOD	9	Summer
TSS	19	Winter
TSS	12	Summer
NH_3	5	Fall/Spring
NH_3	9	Dec.-Feb.
NH_3	1	Summer

Additional process modeling will be required during the design phase of the selected alternative to quantify the anticipated effluent concentrations under a range of flow conditions. This detailed process evaluation is outside the current facility planning scope. If additional treatment efficiency is necessary to reduce pollutant concentrations, an improved treatment process, such as tertiary treatment, may be required.

Nutrient removal may be required in future NPDES permits. The liquid treatment alternatives listed in Section 3 and evaluated in detail in Section 4 include the technology needed to meet anticipated future nutrient discharge limits. Further, space for placement of nutrient removal facilities is identified on conceptual site plans to achieve these potential future goals of total nitrogen and total phosphorus removal. Additional discharge head related to pump design would also have to be accounted for. Treatment options include biological nutrient removal and chemical precipitation. Each liquid process alternative addresses the potential of incorporating a process change to meet future nutrient removal requirements.

Beyond the anticipated nutrient removal requirements, increased treatment efficiency to achieve stricter future discharge limits for existing parameters may also be required. Examples are potential

discharge limits for $CBOD_5$ and Total Suspended Solids that are less than 10 mg/L. The Total Maximum Daily Load (TMDL) has not been calculated for the Great Miami River near Piqua, but it is expected in the future. This could impact future discharge limits for the Piqua WWTP.

Additionally, year-round disinfection limits are a potential future regulation, as well as treatment for disinfection byproducts (DBPs) of chlorinated effluent. If these parameters are regulated more strictly, other alternatives for effluent disinfection may be attractive. These are presented in Sections 3 and 4 of this Amended WWTP Facility Plan.

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Section 3

Development of Liquid Stream Alternatives

3.1 General

Having established design treatment capacities for the WWTP in Section 2, this section describes the evaluation of wastewater treatment unit processes needed for a WWTP upgrade. The unit processes that were evaluated include preliminary treatment, primary treatment, biological/secondary treatment, disinfection, and solids handling. Some of the unit processes, such as preliminary treatment and disinfection, are applicable to all of the other processes, and are included in the construction cost estimates corresponding to those processes. Beyond those unit processes that support the rest of the treatment plant, this section is divided into liquid treatment train (or liquid stream) and solids treatment process evaluations.

The development of the liquid treatment train alternatives focuses on the biological treatment process. Ten liquid treatment train alternatives were initially considered for the treatment plant expansion. They are listed below:

- Conventional Activated Sludge (Upgrade and Expand Current Treatment Plant)
- Extended Aeration (3.0-MGD Oxidation Ditch Parallel to and Operating with Existing WWTP)
- Extended Aeration (7.0-MGD Oxidation Ditch Replacing Existing WWTP)
- Membrane Bioreactor (MBR)
- Integrated Fixed-Film Activated Sludge (IFAS)
- Sequencing Batch Reactor (SBR)
- Anaerobic/Anoxic/Oxic (A2O) Process
- BioMag Process
- Step Feed Process
- BioActiflo® Process Operating with Existing WWTP

Process Alternatives Screening Workshop #1 was conducted to discuss the above liquid treatment train alternatives. The main goals of Workshop #1 were to review the advantages and disadvantages of implementing each alternative, and then assign scores to each one in several cost and non-cost categories, finally developing a short list of four liquid treatment train alternatives to evaluate in more detail. The scoring criteria were weighted according to the value the project team assigned each criterion, with the weighting percentages totaling 100%. Each treatment alternative was in turn scored in each weighted category, with scores ranging from 1 to 5, with higher numbers being more favorable. The multiplication of these scores and criteria weighting factors resulted in a weighted sum for each treatment alternative, as indicated in **Table 3-1**.

The four highest scored alternatives from the workshop comprised a short list of liquid treatment train alternatives selected for more in-depth evaluation. They are listed below:

1. A2O process, achieved by upgrading and expanding the existing plant
2. New 3.0-MGD oxidation ditch, to operate in parallel to upgraded existing plant
3. New 7.0-MGD oxidation ditch process, replacing the existing plant process
4. New 7.0-MGD sequencing batch reactor (SBR) process, replacing the existing process

The result of the in-depth evaluation of the unit processes listed above will be the recommendation of a liquid treatment train alternative for design and construction. These alternatives are evaluated in Section 4 of this Amended WWTP Facility Plan.

Common to each alternative are improvements to raw sewage pumping, headworks (screening and grit/grease removal), and disinfection. These three unit processes are presented in this Section 3 and evaluated in Section 4, separately from the four shortlisted liquid treatment train alternatives listed above.

Table 3-1 Liquid Treatment Train Alternatives Ranking Based on Workshop #1

Treatment Alternatives	Screening Criteria (0=lowest/worst, 5=highest/best)									Weighted Sum (1-5)
	Capital Cost 18%	O&M Costs 15%	Maintenance of Plant Operations (MOPO) 10%	Treatment Efficiency 3%	Ease of Operation 12%	Expandability 10%	Ability to Meet Future Nutrient Regs. 10%	Reliability 15%	Implementation 7%	
1 – Conventional	2	3	2	4	4	1	1	3	3	2.47
2 –Parallel Ox. Ditch	1	2	3	4	2	4	3	3	3	**2.50**
3 – (New Ox. Ditch	2	5	5	4	5	4	5	5	4	**4.26**
4 – MBR	1	1	2	5	3	2	4	3	3	2.30
5 – IFAS	2	2	1	4	3	1	4	3	3	2.40
6 – SBR	2	3	5	4	4	4	3	4	4	**3.49**
7 – A20	2	3	2	4	4	1	5	3	3	**2.87**
8 – BioMag	2	2	2	4	3	2	3	2	1	2.21
9 – Step Feed	2	3	2	4	3	1	2	3	3	2.45
10 – BioActiflo®	2	3	3	3	2	1	1	2	4	2.22

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3.2 Preliminary Treatment Building

It was decided that a new Preliminary Treatment Building could be evaluated for the treatment plant improvements independent of the liquid stream alternative analysis and evaluation.

The new plant headworks will be installed in a new Preliminary Treatment Building to the north and east of the existing influent screw pumps as illustrated in **Figure 3-1**. The sewerage into the plant will be modified to allow sewage to flow by gravity to the Preliminary Treatment Building. Once the plant improvements are online, the existing raw sewage pumping station will be demolished.

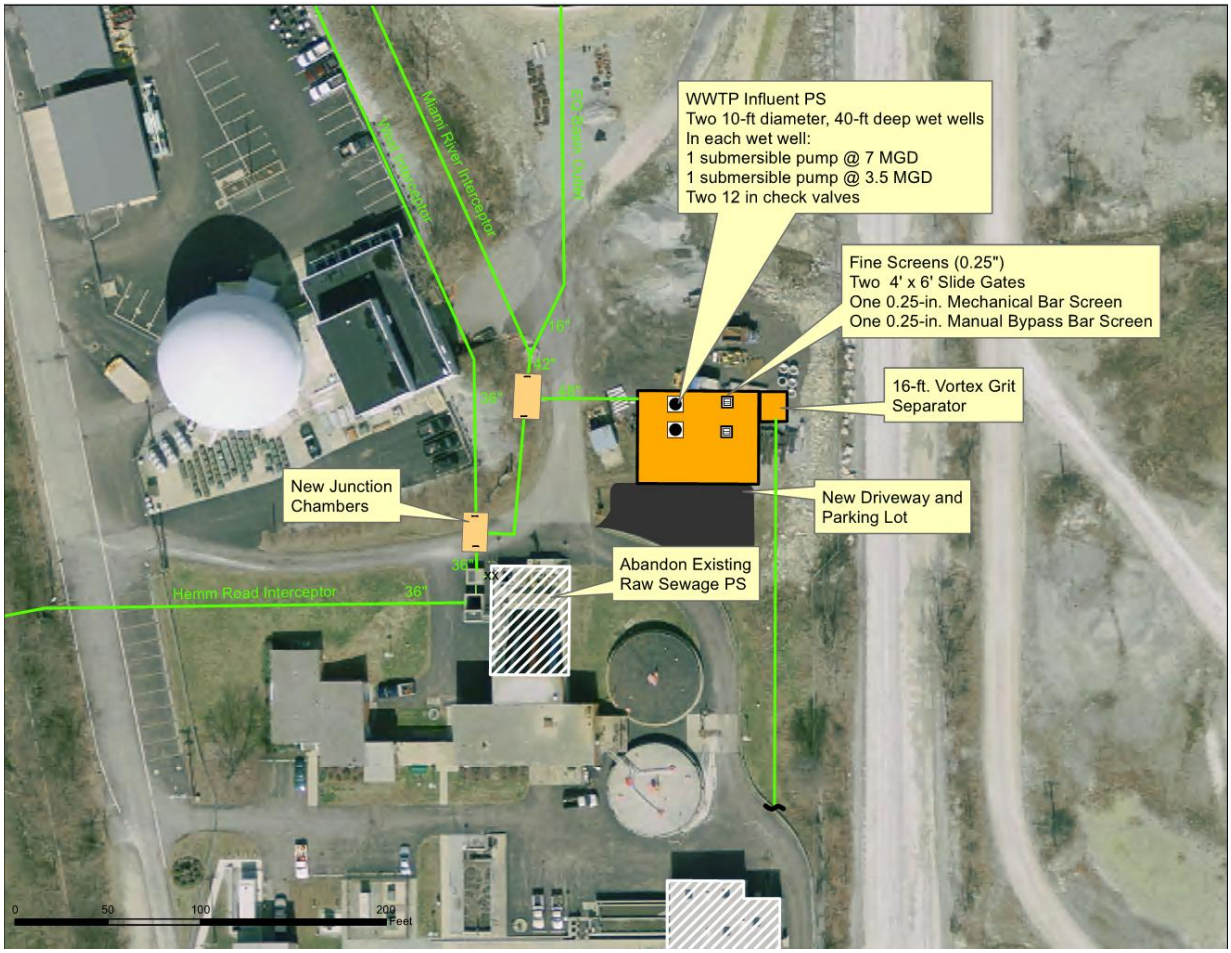

Figure 3-1 Headworks Layout

3.2.1 Raw Sewage Pumping

Wastewater collected in Piqua is conveyed to the Piqua WWTP by a system of gravity sewers, pump stations and force mains for treatment. Pumps at the beginning of the treatment process are required to provide the necessary head to convey flow through the treatment plant and ultimately to the Great Miami River.

The existing sewer system conveys wastewater to the wet well of the WWTP raw sewage pumping station. Raw sewage is then pumped by the plant's influent screw pumps to the downstream screening process. All of the following plant processes are fed by gravity.

Increasing capacity of the raw sewage pump station and subsequent processes is critical to support improvements to eliminate the existing SSO.

The existing influent screw pumps are nearing the end of their useful life. There are space limitations in the existing raw sewage pump station that prevent the addition of a fourth screw pump. There are also hydraulic restrictions in the existing downstream raw sewage channel. It was decided that modifying the existing raw sewage pump station would not be considered as an alternative improvement to meet the City's conveyance needs and eliminate reliance on the existing SSO.

The 2013 WWTP Facility Plan evaluated three alternative improvements to the raw sewage pumping process. The first alternative consisted of replacing the existing screw pumps with new higher output screw pumps to meet future flow requirements. The second alternative called for replacement of the existing pumps with submersible pumps in the existing wet well. The third alternative included abandoning the existing pump station altogether and building a new facility with a firm capacity to meet the design peak flows.

The 2013 WWTP Facility Plan presented economic and non-economic arguments that the existing pump station should be abandoned and that a new facility be installed utilizing submersible pumps as part of any plant improvement increasing plant capacity.

When considering the layout of the new raw sewage pump facility, two alternative influent pumping layouts were evaluated for this new facility.

The first option evaluated considered the installation of screens upstream of the influent pumps. Due to the depth of the sewers conveying wastewater to the plant, the installation of screens ahead of influent pumps would require the installation of these screens in a 33-ft-deep channel. The installation of screens ahead of the influent pumps would protect the pumps. The installation of screens in a 33-ft-deep channel would result in more difficult operation and maintenance of the equipment. Additionally, both the screening equipment and the channel construction associated with this option are expected to increase the cost of implementation.

The second option was to install the influent pumps ahead of the screening equipment. This option would allow for the use of a six-foot-deep screen channel. A shallower channel would generally translate to easier operation and maintenance of the equipment. The cost associated with implementing this arrangement is expected to be lower than an arrangement which places the screens ahead of the influent pumps. If this alternative was to be selected, other measures may need to be taken to protect the pumps from objects in the influent.

3.2.2 Preliminary Treatment

Existing preliminary treatment consists of automatic mechanical screening of the influent using a bar screen with a 5/8-inch opening, followed by grit and grease removal utilizing a Schreiber grit and grease removal system. Due to hydraulic limitations at the current treatment plant, each unit process of the preliminary treatment system will be replaced as part of the WWTP improvements, regardless of the liquids treatment process alternative selected. Screenings, grit and grease will be collected, classified, washed and compacted prior to being discharged to containers that will be collected by a waste collection company for disposal at a landfill.

The preliminary treatment equipment will be located in a new Preliminary Treatment Building. The 2013 WWTP Facility Plan concluded that there was a need for a new raw sewage pump station, which

could be combined with this building. The proposed location for the new building would be to the north of the existing Preliminary Treatment Building. The building and its equipment would be configured to allow solids handling trucks to gain access to dumpsters within the building by way of a roll-up garage door.

3.2.2.1 Screenings

The existing mechanical bar screen should be replaced with new fine screens that meet OAC 3745-40 sludge regulations that require a screen clear opening of <5/8-inch prior to any land application. Facilities installed in compliance with these regulations must be operational by July 1, 2015. In anticipation of these regulations, the City of Piqua has purchased mechanical 5/8-inch bar screens.

For a treatment plant upgrade or a new WWTP, the following fine screening options were evaluated as part of this planning effort:

- Multi-rake design
- Perforated plate

The 2013 WWTP Facility Plan conducted a similar alternative evaluation for screening equipment. The findings of that plan were used to develop an updated alternatives analysis, which is presented in Section 4 of this Amended WWTP Facility Plan.

3.2.2.2 Grit

The new Preliminary Treatment Building will also include grit removal and handling equipment. Grease removal currently takes place concurrently with grit removal via the Schreiber grit and grease removal system. Options that were evaluated included:

- Vortex separation
- Eutek Systems – HEADCELL™
- Grease and Grit removal using a new Schreiber system.

The 2013 WWTP Facility Plan conducted a similar alternative evaluation for grit separation equipment. The findings of that plan were used to develop an updated alternatives analysis, presented in Section 4.

3.3 Primary Treatment

As discussed below, four liquid treatment alternatives were further evaluated for the expansion and upgrading of the treatment plant. Primary clarification would be required for two of these, Alternative No. 1, A2O Process, achieved by upgrading and expanding the existing treatment plant and Alternative No. 2, New 3.0-MGD Oxidation Ditch, to operate in parallel to the upgraded existing plant.

As described in detail in Section 4, Alternative No. 1would require one additional primary clarifier (4 total), improvements to the raw sewage pump station and headworks, and changes in the Primary Control House. The new primary clarifier would be 55-feet diameter with 12-feet sidewater depth to be consistent with the existing primary clarifiers. Alternative No. 2 does not require new primary clarifiers to be added; however continued use of the three primary clarifiers would require replacement of existing sludge removal mechanisms and may require repair and upgrading to meet identified deficiencies.

3.4 Biological Treatment Options

The current biological process is a conventional activated sludge system, configured in the Modified Ludzack-Ettinger (MLE) configuration to provide nitrification and a degree of denitrification. The existing process, as operated by Piqua plant personnel, has proven to be able to provide consistent, high-quality effluent that meets the plant's permitted discharge criteria. However, the existing process does present several significant operating challenges that reduce overall flexibility and perhaps limit the plant's feasible expansion to meet future flows and loads and performance requirements.

The potential biological process options identified in the initial screening stage of the process evaluation, as presented in Table 3-1, are all evaluated for their ability to achieve future treatment goals and be feasibly implemented at the facility. The four highest-scoring alternatives are then carried forward to more detailed evaluation in Section 4.

3.4.1 Alternative 1 – Conventional Activated Sludge Treatment

The intent of this alternative would be to use the existing treatment process to the maximum extent possible. It is noted that the existing biological reactors can be configured in many ways to meet process goals, and some of these configurations are included in other options below. However, this Alternative 1 is meant to represent the possible option of essentially re-using the existing process in the existing configuration.

As indicated in Table 3-1, this alternative does not score well when compared to the other options, for the primary reason that it would not provide sufficient capacity or treatment capability to meet future nutrient removal goals. It also scored poorly on MOPO, which would affect the construction cost and schedule. Therefore, this Alternative No. 1 is not carried forward into more detailed evaluation in Section 4.

3.4.2 Alternative 2 – Extended Aeration (3.0-MGD Oxidation Ditch) Parallel to Upgraded Existing Plant

This alternative consists of constructing a new oxidation ditch process, to operate in parallel with the existing biological treatment process, recognizing the shortcomings in the existing process as discussed under Alternative 1. An oxidation ditches is a well-proven, very stable process used to provide biological nutrient removal (BNR), and a new process can feasibly be constructed to the east of the existing treatment trains.

This new parallel plant would be designed and constructed so that the new process units would handle only the required incremental capacity that cannot be reliably processed by the existing plant and retain only the existing facilities considered in acceptable condition to use with the new plant. This would result in a plant capacity of 4.0 MGD (maximum-month design average) and 6.0 MGD peak through the existing facilities and the new parallel oxidation ditch of 3.0 MGD (maximum-month design average) and 8.0 MGD peak capacity. The new unit process would be located east of the existing plant on property that would have to be acquired from the quarry. Solids from this process would be returned to the existing plant facilities to be processed with solids from the existing plant. As indicated in Table 3-1, this alternative scores well when compared to the other options, and in fact is in the top four alternatives in this screening-level step. Therefore, this Alternative No. 2 will be carried forward into more detailed evaluation in Section 4.

3.4.3 Alternative 3 – Stand-Alone 7.0-MGD Oxidation Ditch Process Replacing Existing Plant

This alternative consists of constructing a new oxidation ditch process to treat the full 7.0-mgd maximum-month design flow condition, to completely replace the existing process. The new process can feasibly be constructed to the east of the existing treatment trains, though it would require a considerable amount of new site footprint. The new plant would be located east of the existing plant and would require three new secondary clarifiers along with a new RAS/WAS pump station.

This new oxidation ditch plant would be designed and constructed so that the new process units would handle a plant capacity of 7.0 MGD (maximum-month design average) and 14.0 MGD peak. Implementing an entirely new process would have the advantage of being constructed without complicating operation of the existing treatment process, and would result in a simpler ongoing operation than utilizing the existing process tankage.

As indicated in Table 3-1, this alternative scores very well when compared to the other options, and is in the top four alternatives in this screening-level step. Therefore, this Alternative No. 3 will be carried forward into more detailed evaluation in Section 4.

3.4.4 Alternative 4 – Membrane Bioreactor (MBR)

MBRs utilize many aspects of activated sludge biological systems, but include ultrafiltration (UF) or microfiltration (MF) membranes, replacing conventional gravity clarifiers and return activated sludge (RAS) systems in conventional activated sludge biological treatment systems. The membranes are immersed directly in bioreactor tanks and the biological system can be operated at much higher mixed liquor suspended solids (MLSS) concentrations, thereby providing greater treatment capacity per unit volume. Submerged membrane assemblies are typically made up of bundles of hollow-fiber or flat sheets of microporous membranes. Clean effluent (permeate) is drawn through the membrane assemblies by means of a vacuum applied to the effluent side of the membrane by a pumping system. Turbulence on the exterior (feed side) is maintained by diffused aeration to reduce fouling.

Membranes typically have to be replaced every 7 years and have a relatively high operation and maintenance (O&M) cost. MBR membranes provide essentially complete removal of suspended solids. Thus, MBRs are generally a good fit for plants with limited space and especially strict effluent limits or the desire to produce effluent suitable for reuse. These two advantages of MBRs are not major factors at Piqua, as there is site footprint available for process tankage, and the anticipated permit limits are readily achievable without bearing the capital and O&M cost of implementing membranes. As indicated in Table 3-1, this alternative does not score as high as several other alternatives. Therefore, this Alternative No. 4 will not be carried forward into more detailed evaluation in Section 4.

3.4.5 Alternative 5 – Integrated Fixed-Film Activated Sludge (IFAS)

The IFAS process is a variation of the activated sludge process, in which a biofilm carrier media is installed within the aeration basins to increase the biological mass in the system. The media, which can be in the form of sponges, looped chords, plastic packing, or other similar configurations, is held in the aeration basin, and a biological film develops on the media. The media is held in the bioreactor by retention screens and is not allowed to reach the downstream secondary clarifiers. Since a significant portion of the biomass is held in the aeration tanks and is not loaded to the secondary clarifiers, a higher biological mass can be developed without overloading the secondary clarifiers. Nitrification and denitrification can therefore be provided with less tankage.

The primary reason for implementing IFAS at treatment plants is that it requires less site footprint than conventional treatment that uses only suspended growth for its biomass inventory. Where footprint is at a premium (or none is available) this advantage can outweigh the IFAS process primary disadvantages, which include higher energy cost (due to the recommended use of coarse-bubble diffusers and the need to maintain a higher DO concentration), and the added complexity of the process. The footprint advantage is not a major driver at Piqua, and as indicated in Table 3-1, this alternative does not score as high as several other alternatives. Therefore, this Alternative No. 5 will not be carried forward into more detailed evaluation in Section 4.

3.4.6 Alternative 6 – Sequencing Batch Reactor (SBR)

SBRs are a variation of the activated-sludge process that employs a phased-in-time approach to meeting treatment requirements, and accomplishes biological treatment and secondary clarification in the same tank. The timing of the anoxic and aerobic periods can be controlled to duplicate the conditions experienced in many continuous-flow treatment processes. In the fill phase, influent wastewater is fed to the SBR tank, which is partially filled with mixed liquor. Varying aerobic and anoxic conditions are achieved by cycling the air supply system on and off. Denitrification is achieved during the anoxic phases, and nitrification and carbon oxidation occur during the aerobic phases. After the anoxic/aerobic treatment phases are complete, a quiescent environment, like a secondary clarifier, is provided in the tank, and the mixed liquor settles. After sufficient settling time, the clear effluent is decanted from the top of the tank, and excess mixed liquor is removed. The SBR is then ready to receive and treat another batch of wastewater. Influent wastewater is fed to SBRs only during the fill cycle, and therefore multiple units must be provided to handle the continuous feed of wastewater at the plant. Operation of the multiple SBRs is coordinated such that one unit is always ready to receive wastewater.

Implementing a SBR process by retrofitting the existing process at Piqua is not feasible, because SBR tanks are deeper and require a significant differential in water surface elevation. Therefore, this alternative would be comprised of a new process located to the east of the existing process, There is sufficient footprint available, and the potential advantages of SBRs result in a high score for this option. As indicated in Table 3-1, this alternative scores very well when compared to the other options, and is in the top four alternatives in this screening-level step. Therefore, this Alternative No. 6 will be carried forward into more detailed evaluation in Section 4.

3.4.7 Alternative 7 – A2O

The A2O process is a very common configuration of the activated sludge process that is used for both nitrogen and phosphorus removal, and is therefore an appropriate process configuration to meet the goals of the Piqua WWTP. The primary components of the A2O configuration are the use of an anaerobic zone, followed by an anoxic zone, and finally an aerobic zone. The A2O process essentially is a MLE configuration, with an anaerobic zone inserted at the influent end of the reactor. The anaerobic zone enables the process to employ enhanced biological phosphorus removal (EBPR), and typically can achieve both the target nitrogen and phosphorus requirements for Piqua.

It is feasible to retrofit the existing bioreactors into the A2O configuration, though additional bioreactor volume would be required. This process configuration appears to be the best option to feasibly re-use the existing process tankage at the plant, and therefore it scores high as indicated in Table 3-1. Therefore, this Alternative No. 8 will be carried forward into more detailed evaluation in Section 4.

3.4.8 Alternative 8 – BioMag Process

The BioMag process is a unique, small-footprint biological process in that it incorporates the addition of magnetite (Fe_3O_4 iron ore) as ballast in the mixed liquor. The magnetite has an affinity for flocculating with the biological matter, and the magnetite's specific gravity of 5.2 allows the ballasted floc to settle much more rapidly than standard biological floc particles in the final settling stage. The ballasted flocculation significantly increases the capacity of the clarification process, resulting in better effluent quality, and allowing the biological system to run at a much higher mixed liquor concentration than conventional treatment. The majority of the magnetite is recovered from the WAS by means of an in-line shear mixer and recovery magnet and then is returned to the mix tank. Supplemental and recovered magnetite are blended with the RAS and returned to the process. Polymer addition aids in producing a very high-quality effluent.

The BioMag process is in the early stages of development, and has been successfully implemented at a small number of treatment facilities. These plants are all smaller in design flow than the Piqua facility and successful implementation of BioMag at larger facilities has yet to be proven. Since the primary driver for BioMag (limited footprint) is not of major concern at Piqua, and as indicated in Table 3-1, this alternative does not score as high as several other alternatives. Therefore, this Alternative No. 9 will not be carried forward into more detailed evaluation in Section 4.

3.4.9 Alternative 9 – Step Feed with Chemical Addition for P Removal

The use of step-feed configurations of activated sludge processes is employed to reduce solids loading to the secondary clarifiers while maximizing use of the bioreactor volume. By implementing step feed, it is possible to maintain an equal biomass inventory to plug-flow configurations of reactors and have a lower MLSS concentration in the inlet to the secondary clarifiers. Variations of step-feed configurations can be implemented to include alternating anoxic and aerobic volumes to achieve nitrogen removal

Though this is a feasible alternative, implementing step feed at the plant would not remove the need to expand the facility, and the process is not configured as well as the A2O process to target the Piqua WWTP's treatment goals. This disadvantage is reflected in the scoring presented in Table 3-1, and therefore, this Alternative No. 10 will not be carried forward into more detailed evaluation in Section 4.

3.4.10 Alternative 10 – BioActiflo® Supplementing the Existing Plant

The BioActiflo process is a relatively new treatment process that utilizes high-rate clarification, and a means to include biological treatment, that was developed to treat plant flow during peak wet-weather conditions. High-rate clarification (HRC) was developed primarily for treatment of combined sewer overflows to remove 60 percent of biochemical oxygen demand (CBOD) and greater than 90 percent of the total suspended solids (TSS); however, because of its inability to remove soluble CBOD, it has not been used for wet weather flows from separate sanitary sewer systems. The BioActiflo process can provide both TSS removal and soluble BOD removal at high hydraulic loading rates.

The BioActiflo process introduces RAS in a contact tank for biological treatment prior to a sand-ballasted flocculation process with coagulant and polymer doses for enhanced TSS removal, necessary to meet secondary treatment standards.

The BioActiflo process has previously been evaluated in considerable detail for implementation at the Piqua plant, including piloting. It was found that the duration and extent of wet-weather flow,

combined with the severe hydraulic restraints of the existing treatment systems at the plant, made implementation of the BioActiflo process infeasible at Piqua. This is reflected in the scoring presented in Table 3-1, and therefore, this Alternative No. 11 will not be carried forward into more detailed evaluation in Section 4.

3.4.11 Additional Alternative – MLE Biological Nutrient Removal

The MLE process is a very common configuration of the activated sludge process that is used for nitrogen removal. The plant's existing bioreactors are configured in the MLE process. The primary components of the MLE configuration are the use of an anoxic zone followed by an aerobic zone, and an internal recycle pumping system that typically recycles mixed liquor from the end of the aerobic zone back to the anoxic zone. This internal recycle pumping rate is typically up to 400% of the plant's influent flow rate, and by way of this nitrate –nitrogen recycle, the MLE process can typically remove about 80 percent of the plant's influent nitrogen.

The MLE process itself will not meet the plant's future treatment goals, which include the need to removal phosphorus to 1 mg/L. Therefore, this Alternative No. 7 will not be carried forward into more detailed evaluation in Section 4. (This process is not listed in Table 3-1 because it was considered similar to the A2O process and was not treated as a separate process at Workshop #1.)

3.5 Disinfection

The treatment plant's existing disinfection process uses gaseous chlorine for disinfection and sulfur dioxide for dechlorination of the plant effluent. The current disinfection process consistently has met permit requirements for fecal coliform (1,000 CFU/100 mL monthly and 2,000 CFU/100 mL weekly). However, the Ohio EPA has reissued more stringent disinfection requirements by switching the indicating organism from fecal coliform to *Escherichia coli (E. coli)* and establishing the monthly and weekly permit limit at 126 CFU/100 mL and 284 CFU/100 mL, respectively. As required by the Schedule of Compliance in the current NPDES permit, the City evaluated its current disinfection process. It was determined the more stringent disinfection requirements could be met with the existing disinfection process, although at a higher chlorine and sulfur dioxide feed rate. Due to the increased consumption of these dangerous gaseous chemicals, the City asked CDM Smith to evaluate other disinfection processes, including:

- Chlorination (as liquid) with sodium hypochlorite (bulk or on-site generation) followed by dechlorination with sodium bisulfite
- Ozone
- Ultraviolet (UV) light

These disinfection processes were identified in the initial screening stage of the process evaluation for their ability to achieve future treatment goals and be feasibly implemented at the facility. Only two of these processes are evaluated in more detail in Section 4.

3.5.1 Chlorination

The mechanism of disinfection with either gas chlorine or sodium hypochlorite is identical due to common chlorine chemistry which produces hypochlorous acid when either chlorine or hypochlorite is added to the water for pathogen inactivation. However, because gaseous chlorine is a highly toxic gas representing both onsite and offsite risks due to leaks and is extremely volatile and hazardous in

nature, liquid hypochlorination is a preferred alternative for disinfection due to safety and Risk Management Program (RMP) concerns.

While the chlorine disinfection process is straightforward, there are certain site-specific characteristics inherent to each wastewater plant that will affect the efficacy of sodium hypochlorite disinfection, and the required dose for effective disinfection. If ammonia (NH_3) is present in the treated effluent, hypochlorous acid will react with NH_3 to form chloramines. While free chlorine (as hypochlorous acid) is a more powerful oxidant and has faster bacterial inactivation kinetics, achieving free chlorine depends on the amount of NH_3 and organic compounds in the effluent. In some instances, high doses of chlorine may be required to form and destroy chloro-organic compounds and chloramines initially formed in order to produce free chlorine when low concentrations of NH_3 are present. This phenomenon is well documented and the breakpoint chlorination process is graphically depicted in **Figure 3-2** (Metcalf & Eddy, 2003). It is important to note that under a "breakpoint" process, no significant free chlorine residual is produced unless the breakpoint is reached.

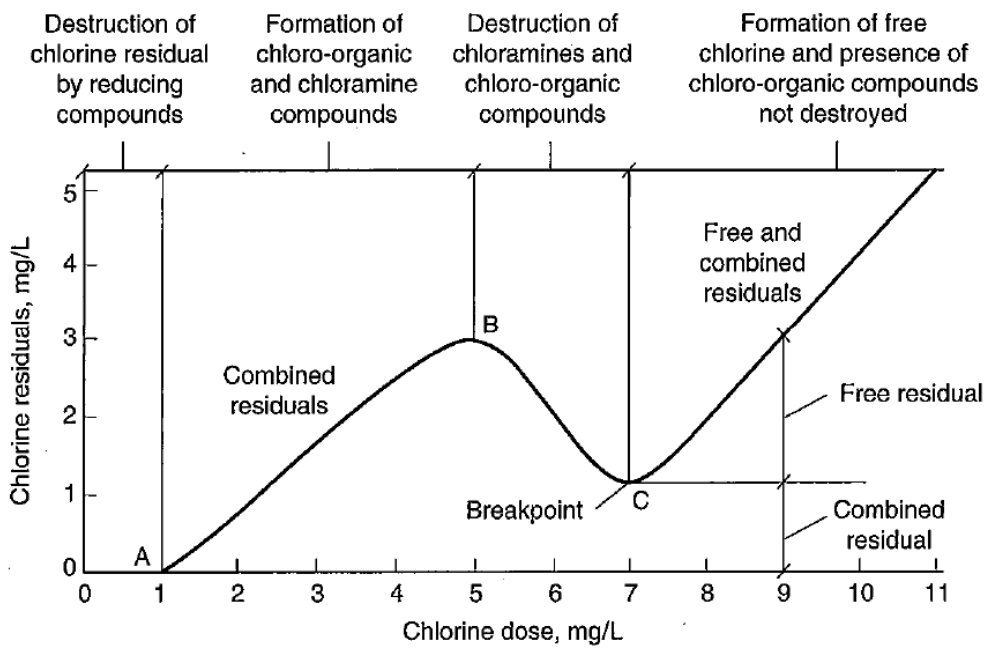

Figure 3-2: Breakpoint Chlorination Curve (Metcalf & Eddy, 2003)

While chloramines are slower reacting than free chlorine, they provide excellent pathogen inactivation. Additionally, chloramination can significantly reduce the potential for production of disinfection byproducts (DBPs). In either case, wastewater operators should be aware of the method of chlorination that is being used at the facility so that they can be diligent about approaching the process control for disinfection. Swings between the two methods can result in difficulty providing process control and potential issues with meeting bacterial discharge requirements.

As presented previously in this Report, the effluent NH_3 concentrations are generally low during the disinfection season, but there are excursions above 1.0 mg/L NH_3 in the effluent which could result in increased chlorine demands or potential swings in disinfection mechanism depending on the effluent NH_3 concentration and other constituents in the treated effluent. Regardless of the mechanism of chlorination, this is a viable alternative and will be evaluated further in Section 4.

3.5.2 Ozone

Ozonation is a mature disinfection technology that merited consideration even though only a few WWTPs in the US currently use ozone. Historically, ozone has been used as a drinking water treatment technology more than a wastewater treatment technology. Early ozone technologies were adopted by a number of publicly owned treatment works (POTWs) in the 1980s, but cost, both capital and O&M, resulted in many facilities abandoning their ozone systems. Ozone generation and application technologies have improved significantly since that time, and the technology is being re-evaluated for its applicability to wastewater disinfection, primarily because it is the only mature disinfection alternative capable of treating color and partially or completely oxidizing complex, non-degradable trace organic compounds (e.g., pharmaceuticals and personal care products [PPCPs] and endocrine disrupting compounds [EDCs]) at typical disinfection doses.

Ozone was first used for wastewater disinfection in the United States in 1975 at Indiantown, Florida. By 1985, 43 additional wastewater ozone applications were installed within the US. Because there is a long history of ozone disinfection, the mechanisms of ozone disinfection are well understood with inactivation of bacteria by ozone being attributed to the oxidation of cell membrane components and disruption of bacterial enzymatic activity. Due to operational challenges at WWTPs, by 2006 only seven facilities in the US were using ozone, as shown in Table 5-1. However, recent advances in ozone generation and dissolution technology developed by the drinking water industry have made ozone more economical in the past decade. Improved economics along with consideration of secondary benefits of ozone are resulting in increasing interest in its application at WWTPs. The secondary benefits of ozone disinfection include removal of emerging contaminants of concern such as EDCs, pharmaceutically active compounds and color.

Table 3-2: US Wastewater Treatment Plants Utilizing Ozone

Location	ADF (MGD)	Ozone Dose (mg/L)	Ozone Production (lb/d)	Ozone Treatment Objective
Mahoning County, OH	8	4	500	Disinfection
Springfield, MO	30	3	2,400	Disinfection
Frankfort, KY	40	4 – 8	1,000	Disinfection
El Paso, TX Fred Hervey Water Reuse Facility	10	5	900	Disinfection for reuse
Trion, GA	8	27	1,800	Color removal, disinfection
Gwinnett County, GA F. Wayne Hill Water Reuse Facility	50	4	4,700	Disinfection for reuse
Indianapolis, IN Belmont WWTP & Southport WWTP	110	6	12,000	Disinfection
Las Vegas, NV Clarke County Water Reuse Plant	60	8	4,000	Disinfection for reuse

(Data from Oneby et al, 2010)

Although the cost of ozone systems has decreased, without a driver for the secondary benefits, these systems are cost-prohibitive; therefore, ozone will not be evaluated further.

3.5.3 Ultraviolet (UV) Light

UV light inactivation of microorganisms is a physical or biophysical process with the germicidal wavelengths occurring in the UV-B and UV-C regions. Electromagnetic radiation in this range alters

cellular proteins and nucleic acids (i.e., DNA and RNA) through dimerization of the thymine nucleic acids on DNA molecules. Because UV light inactivates pathogens by destroying their genetic material, in order to predict the number of pathogens destroyed by a particular UV system, the required dose must be calculated. The dose is a function of the UV radiation intensity and the exposure which is a function of the time that wastewater is retained in the UV reactor.

Factors Affecting UV Disinfection

The equation used to calculate UV dose is shown below:

$$\text{UV Dose} = I \times t \qquad (1)$$

Where: I = UV intensity, in milliwatts per square centimeter (mW/cm2)

t = exposure time, in seconds (s)

UV Dose, in mW-s/cm2 or milliJoules per square centimeter (mJ/cm2)

The actual UV intensity and exposure time are complex functions of the UV system, operating parameters and water quality. For example, in order to reach pathogens, the UV radiation must travel through the quartz sleeve, wastewater and particles (if the microbes are embedded in particles). Consequently, the UV intensity actually reaching the target organisms is lower than that at the surface of the UV lamp and varies throughout the reactor.

The exposure time is ideally the average hydraulic retention time within the UV reactor (or the reactor volume divided by the flow rate). However, actual exposure times for each target microorganism are a function of reactor volume, flow rate, mixing conditions within the reactor and extent of short-circuiting. Other factors that can impact the amount of UV exposure include the distances between centers of the lamps, because even without absorption loss UV intensity decreases with increasing distance from the lamp. Also, dead space in a reactor can reduce the effective reactor volume and shortens the average hydraulic retention time. Overall, the UV dose also depends on a range of water quality and lamp condition factors. Discussion of these factors is provided in the following paragraphs.

Water Quality Parameters

Water quality affects the performance of a UV system by altering the UV intensity within the reactor and consequently, the UV dose received by the organisms within the wastewater. The most important water quality parameters are the UV transmittance (UVT) of the water and the TSS concentration and particle size. In addition, dissolved solids may foul the quartz sleeves surrounding the lamps and decrease the effective UV output, so an understanding of the water hardness, iron and other dissolved organics in the wastewater can be important to designing and evaluating a UV disinfection system.

The UVT is one of the critical water quality parameters determining the UV intensity that will act on the microorganisms. As UV rays travel through wastewater, their intensity is attenuated continuously because the substances in wastewater absorb some of the UV light. UVT is defined as the percentage of UV light at 254 nm not absorbed after passing through a 1-centimeter water sample. The relationship between intensity and transmittance is directly proportional, i.e., the higher the transmittance the higher the intensity available.

TSS will absorb and scatter UV light, thus lowering the UVT. Similarly, the higher the TSS concentration the higher UV dose required. Additionally, the size of these solids highly affects the disinfection process. Large suspended solids have the capability of screening or shading the target

microbes, preventing them from receiving their design UV dose. Although ideal, effluent filters are not a requirement for the effective application of UV disinfection at WWTPs.

All the above items were considered and it was determined that UV disinfection is a viable alternative and will be considered and further evaluated in Section 4.

Section 4

Evaluation of Liquid Stream Alternatives

4.1 Basis of Evaluation

Each of the finalist alternatives defined in Section 3 were evaluated based on economic and non-economic factors. The economic evaluation compares capital costs, operation and maintenance (O&M) costs and the resulting life-cycle cost for each alternative. O&M costs are comparative among the alternatives and include electrical power, labor, chemicals, supplies, and equipment maintenance and replacement. Non- economic factors include maintenance of plant operations (MOPO) during construction, treatment efficiency, ease of operation, flexibility for future expansion, ability to meet future regulations, reliability/risk, and implementation.

Each of the liquid-stream alternatives are documented in this section, starting with the preliminary treatment processes and systems – including influent pumping, screening and grit removal. The biological process alternatives are described next, incorporating primary treatment (where applicable) into the analysis. Finally, disinfection alternatives are addressed.

4.2 Raw Sewage Pumping

The plan for the Piqua WWTP's raw sewage pumping is a critical issue for preliminary design. Based on the evaluation performed as part of the 2013 WWTP Facility Plan, it was recommended that the existing raw sewage pumping station be replaced with new equipment in a new building. The basis of this recommendation is provided in more detail as part of Section 3 of this report. There are two general process layouts that were considered for this new raw sewage pumping and preliminary treatment building: one where screening precedes the raw sewage pumps, and a second where the pumps are the first element of the process.

The City provided constraints and priorities to be considered for this evaluation. The new building would contain raw sewage pumping facilities as well as preliminary equipment. The new building would include a roll-up garage door and concrete pad to allow a solids waste truck to access dumpsters within the building to facilitate the hauling of screenings and grit to landfill. The process would be designed such that equipment could discharge separated screenings and grit to dumpsters at an elevation accessible by the existing drives. The process would be designed such that all screening is done at one location. The raw sewage pumps would deliver the pumping head necessary to support the hydraulics of all remaining processes at the peak design flows.

4.2.1 Raw Sewage Pump Station Alternative Layouts

The first pumping layout evaluated the advantages and disadvantages of locating the screens upstream of the raw sewage pumps. The sanitary sewer system carrying wastewater to the plant is expected to have an invert elevation of approximately 820. The elevation of the drive where solid waste handling trucks will be accessing dumpsters is approximately 853 in the area where the new building is proposed. In order for the equipment to mechanically deliver screenings to a dumpster at grade, the screen channel for this layout would need to be more than 33 feet in depth. That fact about this alternative layout informs all of the layout's disadvantages. The wet well for the raw sewage pumping would also need to be more than 33 feet deep in any layout, but having the screens precede

the pumps in the process would increase the amount of excavation needed to construct this building. Much of this excavation is expected to be in rock, further increasing construction costs. Additional building material would also be necessary for a deeper channel which is also expected to drive up the cost of construction. The screening equipment necessary for this alternative layout would be more costly than equipment used in a 6-foot deep screening channel. Based on quotes provided for the two alternative layouts, the mechanical screens sized for a 33-foot deep channel were on average quoted at a price approximately 90% higher than the mechanical screens sized for a 6-foot deep channel. Equipment in a 33-foot deep channel would pose more complex operation and maintenance concerns. While the mechanically cleaned screen technology available today should minimize the need for an operator to access the bottom of the channel, it would still be necessary to design safe access measures for the deeper channel.

In spite of the drawbacks explained above, the first pumping layout has one important advantage over the alternative. The presence of screening equipment upstream of raw sewage pumps would protect pumping equipment from objects which may damage or clog the pumps. Having low-maintenance self-cleaning screens installed ahead of the influent wet well would then also serve to minimize the downtime and need for maintenance on the pumps.

An evaluation of the second pumping layout was made, noting the advantages and disadvantages of locating the raw sewage pumps upstream of all treatment processes, including the screens. Placing the screens downstream of the pumps would provide flexibility in the sizing of the screen channels. For consideration in this alternatives analysis, the screen channel depth was assumed to be 6 feet for this layout. The shallower channel depth would result in lower construction costs, lower equipment costs and less operation and maintenance concerns.

The disadvantage of placing the raw sewage pumps ahead of the screens is that the pumps would be exposed to unscreened sewage. The influent wet well would not be protected from objects that might damage or clog the pumps. The absence of screens ahead of the influent wet well may increase the amount of maintenance these pumps require.

One constraint for this process is for all screening to occur in the same location. The existing system makes use of manually cleaned screens in the raw sewage influent chamber and a mechanical screen in the Screen and Grit Building. Plant staff must physically move screenings into roll-off containers which then must be lifted to grade for disposal. This constraint precludes the consideration of an alternative layout consisting of a manually-cleaned bar screen upstream of the influent wet well.

4.2.2 Alternative Layout Evaluation

The collection system for the WWTP is a separate sewer system. Large objects that may be carried by storm water runoff are not expected to enter the system or get to the plant's headworks.

A temporary submersible pump station was in service for years on the City's Miami River Interceptor with no protective screening equipment. During that time, the submersible pumps did not experience any maintenance issues associated with unscreened sewage. Problems with the pumps were limited to nylons or rags wrapping around the impellers. The manufacturer of the pumps used at this pump station has since addressed these issues with an equipment redesign.

Other plants in Ohio dealing with similar influent sewage have been able to operate reliably without screens. Baffles or bends in the influent channel could be installed to protect the pumps without the need for screening equipment.

4.2.3 Raw Sewage Pumping Recommendations

There have been no changes in conditions since the development of the 2013 WWTP Facility Plan that impact the economic and non-economic reasons for constructing a new raw sewage pumping building with new pumping equipment. It is recommended that a new raw sewage pumping system be installed as part of any plant improvement that will increase plant capacity.

The plant's separated sewer collection system is expected to provide a level of protection necessary for reliable operation of the raw sewage pumps. The City's experience with the operation of submersible pumps in a similar application on the Miami River Interceptor support this expectation. It is recommended that the design of the raw sewage pumping system include the placement of the influent wet wells ahead of the screens.

Preliminary design for the raw sewage pumping system includes the use of four non-clog submersible pumps located in two separate wet wells.

During the design phase, it is recommended that the City consider the use of baffles or bends in the influent channel to minimize the risk of objects damaging or clogging the raw sewage pumps. Space could also be set aside for the future installation of a coarse bar screen upstream of the influent wet wells to provide for any future scenarios where the cost and effort associated with clearing screens and lifting screenings may outweigh the cost and effort associated with maintaining the raw sewage pumps.

4.3 Screening

This sub-section compares and evaluates screening technologies previously outlined in the 2013 WWTP Facility Plan. Information on manufacturers is provided for the discussed technologies. Equipment information and budget pricing has been updated based on information collected during the development of this Facility Plan. The screens were evaluated to comply with the OAC regulation related to biosolids land application, but emphasis will also be placed on:

- Screenings loads for normal and peak load operations
- Headloss
- Screening configuration
- Screening discharge

4.3.1 Screen Technology and Design Criteria

This sub-section summarizes the screen types and technologies evaluated in the 2013 WWTP Facility Plan, as well as issues to consider during screen selection. These screen types were:

- Multi-Rake
- Perforated Plate

Both screen types are capable of meeting the OAC requirements. The multi-rake has a greater screenings capacity. The multi-rake and perforated plate screens were evaluated with ¼-inch openings, which meet OAC requirements. These screens are described below.

4.3.1.1 Multi-Rake Bar Screens

Multi-rake screens are the most commonly used screen in the U.S. There are several models and manufacturers of the multi-rake screens, such as:

- Headworks Mahr® screen
- RakeMax by Huber Technologies
- Chain & Rake Monster by JWC Environmental
- FlexRake by Duperon

Multiple rake screen manufacturers offer screens down to 3-mm openings. However, equipment representatives and staff from some WWTPs with ¼-inch inch and smaller screens have indicated that grit and rocks can get lodged between bars, causing screen blinding and wear on the bars and rakes as the rake moves up the screen. Since these are bar screens, these screens have a lower screenings capture ratio, as compared to perforated plate screens, due to removal in one-dimension only. However, the screens are more rugged and more appropriate for raw sewage at treatment plants or combined sewers. These screens have lower headloss compared to perforated plate screens.

Multiple rake screens are equipped with upper and lower sprockets or guides that carry the drive chain. Multiple rakes are attached to a chain to permit quick cleaning of the bars and to reduce the amount of screen blinding. This design allows these units to have very low headroom requirements with only the motor, frame, and doctor blade mechanism located above the screen discharge point. This design does have a submerged lower sprocket and bearing, but technology innovations have greatly increased the durability of the submerged components. For example, the lower sprocket and bearing are a self-lubricating design and grease lines are not required.

4.3.1.2 Perforated Plate Screens

The perforated plate screen, sometimes called a continuous element screen, is a fine screen with a continuous band of perforated plate that rotates through the flow stream. The screen serves the dual purposes of removing debris from the flow stream and conveying it out of the channel and up to the operating floor for discharge. The debris is then usually removed from the screen by a water spray, sometimes in conjunction with a counter-rotating brush. There are two styles of continuous element screens; continuous perforated plate screens and continuous bar screens. The continuous bar screens rely on plastic media and hooks that tend to break and lead to increased maintenance. Therefore, this type of continuous element screen was not considered further.

Five perforated plate panel continuous element screens on the market in the U.S. are the Aqua Guard PF® manufactured by Parkson, the Filterscreen® manufactured by FSM, the Perforator marketed by Headworks, the Escamax® manufactured by Huber, and the Aqua-Screen 2® manufactured by Andritz.

Installation for both styles of units includes completely enclosing the screen section above the channel wall for odor reduction and safety. Common to both units, the wastewater flows through the screen and suspended particles are captured on its surface. Panels on the screens are fabricated in a step type

design to carry debris from the channel. Captured screenings are discharged to a totally enclosed chute where a counter-rotating drive brush with an integral spray bar removes solids remaining on the screen. The unit is mounted on an angle between 60 and 75 degrees to aid in material removal. This angle allows a greater screen face and a greater screenings removal at peak flows due to reduced velocity through the perforations. The perforated plates are typically attached to a drive roller-chain.

The main advantage to the continuous element perforated screens is a high screenings capture ratio. The perforations prevent thin objects from wedging into the screen, and the step design aids in lifting large debris out of the flow. The screen footprint is generally considered medium size due to the recommended angle of inclination. Another advantage to these units is that there are typically no submerged bearings.

A disadvantage of the continuous element screens is high headloss because of the low percentage of open area and flow having to pass through the screen twice. Headloss through these types of screens can further be aggravated if a mat forms along the face of the screen. This problem can be countered by increasing the rotational speed of the screen. Whether or not the speed can be increased to the point that prevents mat formation during peak screen loading is a critical evaluation factor for this type of screen.

Maintenance issues noted with the continuous element screens include plugging of openings with hair and other stringy material unable to be removed by the cleaning brush or spray water. This is referred to as "stapling". This problem is a key consideration for O&M differences between perforated plate and multi-rake screens. Another potential problem is the brush or spray water not fully removing the screened material. When this happens, material removed on the upstream side of the unit is carried over and deposited in the downstream flow, partially reducing the capture of the screen. The brush itself is also commonly found to be a messy and a high maintenance item.

4.3.2 Screenings Production

Design guidelines for the amount of screenings to be anticipated from separate and combined sewer systems are published by the Water Environment Federation in its Manual of Practice No. 8 (MOP 8). Average volumes range from 0.5 cubic feet/million gallons (ft^3/MG) for coarse screens (nominal 2½-inch openings) to approximately 14.0 ft^3/MG for fine screens (nominal ¼-inch openings) for bar style configuration. Peak hourly volumes can range from 2 to 20 times these values. Typically, the peak volumes are produced during wet weather periods with the increased screenings volumes predominantly consisting of coarser material gathered from the storm water influences or washed out sediment from the sewer system.

In general, as the opening between bars decreases from 1-inch, the quantity of screenings removed increases rapidly. It was reported the plant currently generates approximately 1.5 cy of screenings per week. This screenings generation aligns with the values reported in MOP 8 for a ¾-inch screen. For ¼-inch spacing, the average quantity of screenings removed is approximately 14 ft^3/MG. During design a more thorough analysis is needed to quantify the increased generation of screenings and the best way to handle them. It is important to note that these estimates are for vertical bar screens. Perforated screens will remove more screenings from the flow because they have smaller open flow area.

4.3.3 Hydraulics

The hydraulics constraints are typically a key design parameter for fine screens. The improvements to the raw sewage pump station will slightly reduce this impact, provided the headloss through the screen is not so great that it significantly changes the pumping requirements by requiring significantly higher-head pumps. As mentioned previously, the headloss through a perforated plate screen is significantly greater than that of a multi-rake screen as shown in **Table 4-1**.

Table 4-1: Hydraulic Performance of Screens

Screen Type	Headloss w/o Blinding	Headloss w/30% Blinding
Multi-Rake	1.3 in	7.2 in
Perforated Plate	~12 in	19 in

4.3.4 Screen Location

The influent wet well, raw sewage pumps and preliminary treatment equipment will be installed in a new Preliminary Treatment Building.

The conceptual layout for the new Preliminary Treatment Building will include two deep hydraulically-interconnected wet wells to which the City's sanitary collection system will convey influent wastewater. These wet wells will include two raw sewage pumps each. Two of the raw sewage pumps will be sized to convey one-quarter of the upsized plant's peak capacity. Two of the raw sewage pumps will be sized to convey one-half of the upsized plant's peak capacity. In the peak influent condition, the two smaller pumps and one larger pump will deliver flow to the treatment system, while the fourth pump will be available as a back-up of the largest pump. These pumps will be operated by variable frequency drives (VFDs), which will allow the pumps to increase and decrease flow to the plant to respond to variable inflow.

Based on the evaluation documented in Section 4.2, it is recommended that the screens follow the raw sewage pumps and be located at the elevation required by the plant hydraulics of the selected liquid process train.

4.3.5 Capital Cost of Screening Alternatives

Given the design criteria, planning level costs were requested from screen manufacturers for perforated plate and multi-rake screens in a 6-ft deep channel and a 33-ft deep channel. Each screen was sized to handle the PHF of 14 MGD. There was little variation from the perforated plate and multi-rake screens layouts, so the conceptual layouts were assumed to be identical for the different screen technologies.

Budget proposals collected from sales representatives for manufacturers of mechanical bar screens estimated equipment cost very close to those developed by the sales representatives for perforated plate screens. The total estimated cost for equipment associated with the proposed screening process applicable to this stage of planning is $650,000. This cost does not include an estimate for the construction of the Preliminary Treatment Building or ancillary processes.

4.3.6 Operation and Maintenance Cost of Screening Alternatives

The O&M cost for screening operations is rarely done for conceptual comparisons due to the difficulty in predicting screenings production and the variability in screenings related to wet weather events. As a result, O&M comparison is done qualitatively.

As discussed earlier, the perforated plate screens are more capable at removing screening material from wastewater. This is due to the clear opening being the same size in all orientations. This will result in a significant increase in the screenings in comparison to a multi-rake design. The increased screenings production has a direct result on the O&M cost. The more screenings collected results in additional cleaning and dewatering. However, debris passing through a multi-rake design may require increased maintenance in a downstream process. The WWTP has not had any issues with screening debris downstream or in the digesters with the existing ¾-inch mechanical screen. If a new biosolids process is selected, such as ATAD, the higher screen capture may prove to be more beneficial to that process by eliminating or minimizing plugging of mixing nozzles.

4.4 Grit and Grease Removal

4.4.1 General

Grit removal is an important part of the wastewater treatment process to protect downstream equipment and biological processes. The removal of grit reduces unnecessary abrasion and wear of mechanical equipment, such as primary clarifier sludge pumps, digester recirculation pumps, and sludge dewatering equipment, particularly centrifuges. Additionally, grit removal prevents grit deposition in other unit process, such as primary clarifiers, aeration basins, or digesters, which can cause operational issues to the aforementioned processes.

Quantification of grit loading through a study is the preferred method to ensure proper sizing of the grit dewatering/cleaning processes and conveyors. However, without the benefit of such a study, grit loadings of 2-5 cubic feet per million gallons are typically used. Typically, grit loading numbers vary widely, and will be highly dependent on the type and age of the collection system and degree of grit washing provided. A very efficient system with excellent grit washing can actually result in a fairly low grit quantity due to the complete lack of organic material.

In addition to the quantity of the grit, the grit density is also a critical design criterion. The density is determined by settling velocity and applying Stoke's law. The settling velocity should be determined in a large diameter cylinder to avoid errors due to wall effects. Grease coating of grit particles and the dispersion effect of detergent (or what Eutek refers to as the "froth effect") are likely reasons why traditional grit removal systems have generally not performed according to expectations. The density measurement will provide an evaluation of the severity of the froth effect. This data is used to refine methods for sizing the primary grit removal process. If a large percentage of low density grit is found, the grit system sizing should be more conservative, and methods of lowering the density of the grit such as vigorous aeration, or returning waste activated sludge to the grit chamber, should be considered.

4.4.2 Existing System

The existing grit removal system is a Schreiber Grit and Grease removal system. This system is a unique system designed to remove both grit and grease in a common structure. The system consists of a trapezoidal-shaped concrete channel that has two separate zones. Combined, these zones separate and collect both grit from the bottom and grease for removal from the quiescent zone. One zone is

designed to settle grit particles for removal and the other collects grease for removal. Grit removal is accomplished by a rotating spiral flow pattern which scours and washes organics from the grit. The grit is then deposited in a trough at the bottom of the channel. A grit pump mounted to a traveling bridge pumps the collected grit to an elevated trough sloped at one end of the structure to transfer the grit slurry to a grit classifier for further washing and dewatering.

Floating grease and scum are transported to one end of the channel by a grease skimmer blade and basket. The grease is directed to a screw conveyor. As the screw conveyor rotates, lifting the grease for disposal in a collection container, the water content is reduced, thus reducing the overall volume of material being transferred for disposal.

The existing grit and grease removal system was sized to treat a PHF of 8.3 MGD, with a hydraulic detention time of 5.2 minutes. The existing system does not have sufficient capacity to handle the future peak flow of 14.0 MGD. Therefore, this sub-section will evaluate various alternatives for providing grit removal at the plant for the higher design flows.

4.4.3 Initial Screening Process

During a workshop with the City, numerous technologies for removing grit were discussed. These technologies consisted of aerated grit chambers and vortex grit basins. An additional technology not discussed during the workshop was a plate settler, such as Eutek's HeadCell unit. There is considerable controversy as to the preferred method of grit removal. There is a roughly equal split among WWTP operators and owners in the preference for aerated vs. vortex basins. Aerated basins are still largely preferred in Europe, whereas vortex basins have gained a broader acceptance in the US. European plants tend to be smaller and the aerated basins are more affordable, whereas in the larger US plants, considerable cost savings can be realized by using vortex basins. Plate settlers used for grit removal represent a very small portion of grit removal application, so currently, less is known about this technology. However, it appears to be a promising technology. To meet future demands for grit removal, the following grit removal alternatives were evaluated:

- Additional aerated grit chamber and upgrades to the existing aerated grit system
- Replacement of aerated grit chamber with vortex grit basin
- Replacement of aerated grit chamber with grit plate settler

It was assumed that grit removal would be accomplished by a singular technology, i.e. the existing additional aerated grit chamber (AGC) would be expanded with an additional AGC or the existing AGC would be replaced with a new technology (vortex or plate settler).

4.4.4 Technology Overview and Design Criteria

4.4.4.1 Aerated Grit Chamber

The additional AGC would be located adjacent to the existing system. This location would allow common dumpsters to be used for both the grit and grease. It was reported that cold weather has caused operational issues for the grit removal equipment. As a result, the headworks building should be reconfigured, so that the equipment is no longer exposed to the elements. It is anticipated that the existing steel structure would be demolished, salvaged, and rebuilt with a new structure that is tolerant of the corrosive environment associated with wastewater treatment headworks. It is likely the building would be CMU block with concrete roof. Ventilation would be in accordance with

applicable laws, regulations, and guidelines. Heating would be provided in the winter to keep the building at a temperate level (about 50° F).

Table 4-2: Design Criteria for Aerated Grit Chamber

GRIT SEPARATION FACILITY	
Grit Facility Traveling Bridge	
Motor	Constant Speed
Motor Size, hp	0.25
Grit Blowers[1]	
Number	2
Motor Size, hp	15
Grit Screw Classifier	
Type	Screw conveyor w/classifier & washer
Number	1
Classifier Size, in	12
Motor Size, hp	1.0
Grease Screw Conveyor	
Type	shafted
Number	1
Motor Size, hp	1.5
Grit Pump	
Number	1
Motor Size, hp	2.4
Headloss	<6”

1. Air could be supplied from the aeration system blowers as is the current practice.

4.4.4.2 Vortex Grit System

Vortex grit basins first began to be commonly used in the 1980’s. Vortex grit basins are subcategorized into two types; forced vortex or free vortex. Free vortex grit basins use centrifugal force to throw the grit particles against the side walls of the grit basin, and the particles travel down and out the bottom of the tank. Forced vortex grit removal basins use a much slower circular flow pattern to create a quiescent zone at the center of the basin where the grit migrates and is then removed. Forced vortex basins use stirring paddles to control the velocity in the chamber and lift out any organics that also might migrate to the quiescent zone. A forced vortex basin is used as the basis of consideration for this alternative.

A vortex grit removal system would consist of one basin and a bypass channel, in accordance with the Ten State Standards Section 63.3. An enclosure would be constructed over the grit pump and motor, which is mounted in the center of the grit basin. The grit would be pumped to the grit washer/classifier in the new grit handling building, adjacent to the new screening facility. This location would allow for consolidation of screening and grit handling into one building.

Table 4-3: Design Criteria for Vortex Grit Basin

GRIT SEPARATION FACILITY	
Grit Facility Common Mixer	
Type	Vertical Shaft
Motor	Constant Speed
Motor Size, hp	25
Grit Vortex Unit	
Type	Vortex
Number	1
Capacity (each), MGD	16
Motor Size, hp	1
Grit Screw Classifier	
Type	Screw conveyor w/classifier & washer
Number	1
Capacity, gpm	250
Motor Size, hp	3
Headloss	<4”

4.4.4.3 Plate Settler System

The plate settling unit used for this evaluation was the Eutek HeadCell®. This system is an all hydraulic grit concentrator, which uses vortex flow and a stacked plate (or tray) design to efficiently capture and settle fine grit via large surface area and short settling distances. The unit is typically installed into the process flow, downstream of screening. The unit requires no external power source, has no internal moving parts, is self-cleaning, and has a compact modular construction. Wide turndown ratios can be accommodated in this system. An illustration of a typical unit has been provided in **Figure 4-1** to assist the City in evaluating this option.

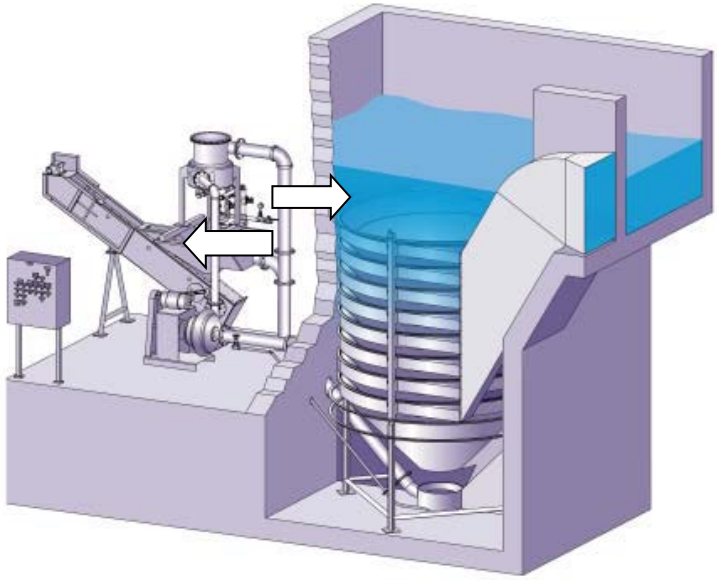

Figure 4-1 Typical HeadCell® Plate Settler Arrangement

At Piqua, the HeadCell installation would consist of one unit. The unit would have seven settling plates 12-feet in diameter. With a loading rate of 11.4 gpm/ft^2, the unit would be capable of removing 95% of all grit (specific gravity of 2.65) ≥ 106 microns at peak flow conditions. Additionally, the unit would be capable of removing 95% of all grit (specific gravity of 2.65) ≥ 5 microns at average flow conditions. The grit is collected at the bottom of the unit. The grit slurry is then removed by a pump and discharged to the grit washer/classifier. The washed grit is removed and deposited in a dumpster for disposal. The water from the washing process is put back in the wastewater for removal of organics.

Table 4-4: Design Criteria for Plate Settler

GRIT SEPARATION FACILITY	
Grit Facility Common Mixer	
Type	Vertical Shaft
Motor	Constant Speed
Motor Size, hp	25
Grit Vortex Unit	
Type	Vortex
Number	1
Capacity (each), MGD	16
Motor Size, hp	1
Grit Screw Classifier	
Type	Screw conveyor w/classifier & washer
Number	1
Capacity, gpm	250
Motor Size, hp	3
Headloss	<12”

4.4.5 Construction Cost

4.4.5.1 Aerated Grit Chamber

The estimated total construction cost to construct an additional aerated grit chamber is $1,200,000. The key cost components of this alternative consist of the equipment, concrete, and building to house the grit handling facility. This alternative has an identical construction cost as the plate settling unit, but removes grease in addition to grit. The AGC is the only process discussed in this Amended WWTP Facility Plan that is capable of removing both grit and grease. The concentration of grease in the influent should be considered when making a decision on the preferred grit removal alternative. Adding a second AGC in parallel with the existing unit may create some piping and hydraulic problems for several of the biological process alternatives because of existing hydraulic limitations of the downstream channels.

4.4.5.2 Vortex Grit Removal

The estimated total construction cost to construct a vortex grit removal system is $870,000. This is the lowest cost alternative to remove grit. This system also has the lowest headloss, which would help minimize pumping costs to convey flows through this process. This alternative is not capable of removing grease. The importance of grease removal must be considered when making a decision on the preferred grit removal alternative.

4.4.5.3 Plate Settler System

The estimated total construction cost to construct a plate settler grit removal system is $1,200,000. This alternative is tied for the highest cost alternative. However, based on manufacturers' data, this process has the best grit removal performance. This high removal efficiency comes at a "cost" of the highest headloss, which is approximately triple the headloss of a vortex unit.

4.4.6 Operation and Maintenance Cost of Grit Removal Alternatives

The O&M cost for grit removal operation is difficult to predict. This is due to the highly variable grit concentrations seen during typical average dry weather flows compared to peak flow events. During the peak flow a massive surge, or plug, of grit is introduced to the treatment plant. Because there is not site-specific data for grit concentrations, O&M costs were not developed during this stage of the planning process.

4.5 Biological Processes

The following finalist alternative biological processes were selected for evaluation as described in Section 3:

- **Alternative 1** – A2O process, achieved by upgrading and expanding the existing plant
- **Alternative 2** – New 3.0-MGD oxidation ditch, to operate in parallel to upgraded existing plant
- **Alternative 3** –New 7.0-MGD oxidation ditch process, replacing the existing plant process
- **Alternative 4** – New 7.0-MGD sequencing batch reactor (SBR) process, replacing the existing process

4.5.1 Alternate No. 1 – A2O Process

Description

Figure 4-2 shows the existing process flow schematic for the Piqua WWTP. In order to implement the A2O process by making use of the existing process facilities, the plant's primary clarifiers, bioreactors, secondary clarifiers and all supporting process-mechanical systems would require a significant amount of upgrading. A conceptual layout for Alternative 1 is provided in **Figure 4-3** to assist in the description and evaluation of this alternative.

The plant's existing three 55-ft diameter primary clarifiers do not have sufficient capacity to successfully treat the design peak flow of 14 MGD, so a fourth clarifier would be required. The new preliminary treatment facility described earlier in this section would be constructed at a sufficient elevation to enable adequate hydraulic capacity and flow split capability to the four primary clarifiers. The existing hydraulic constraints downstream of the existing primary clarifiers would need to be addressed, by construction of a new primary effluent pump station which would receive primary effluent from each clarifier, and lift it to a new splitter box that would feed the upgraded biological process. As part of this alternative, the clarifier mechanisms in each of the three existing clarifiers would be replaced due to their age and condition.

The existing four bioreactors are each configured as a Modified Ludzack-Ettinger (MLE) process, consisting of a small anoxic zone followed by an aeration zone, with an internal mixed-liquor recycle pumping system. In order to achieve an A2O process configuration at the plant to meet both the projected flows and loads and the nutrient removal process goals (total nitrogen concentration of 8

mg/L and total phosphorus concentration of 1 mg/L) , the four bioreactors would require significant expansion and upgrade.

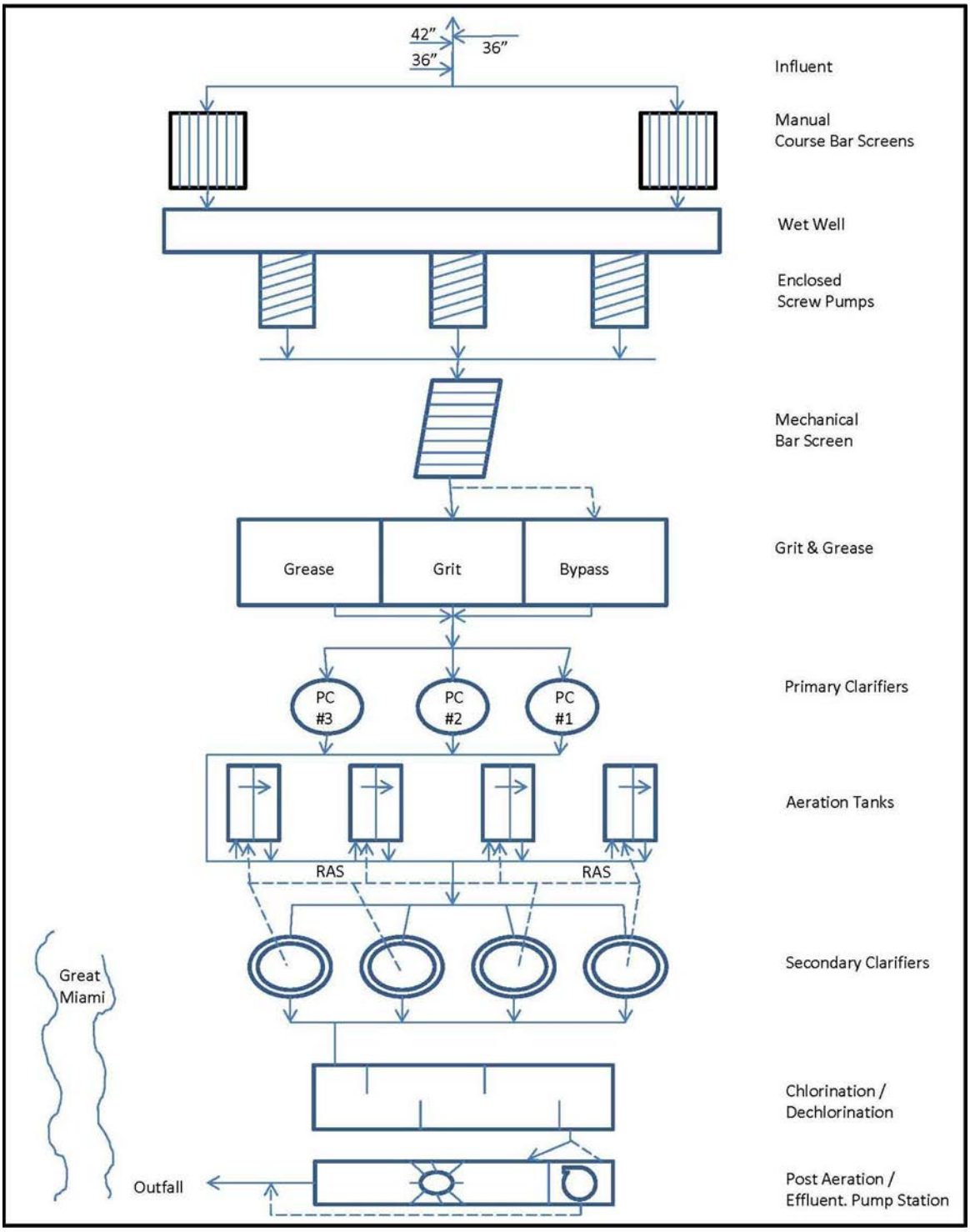

Figure 4-2: Existing Flow Schematic Wastewater Treatment Plant Facility Plan Update

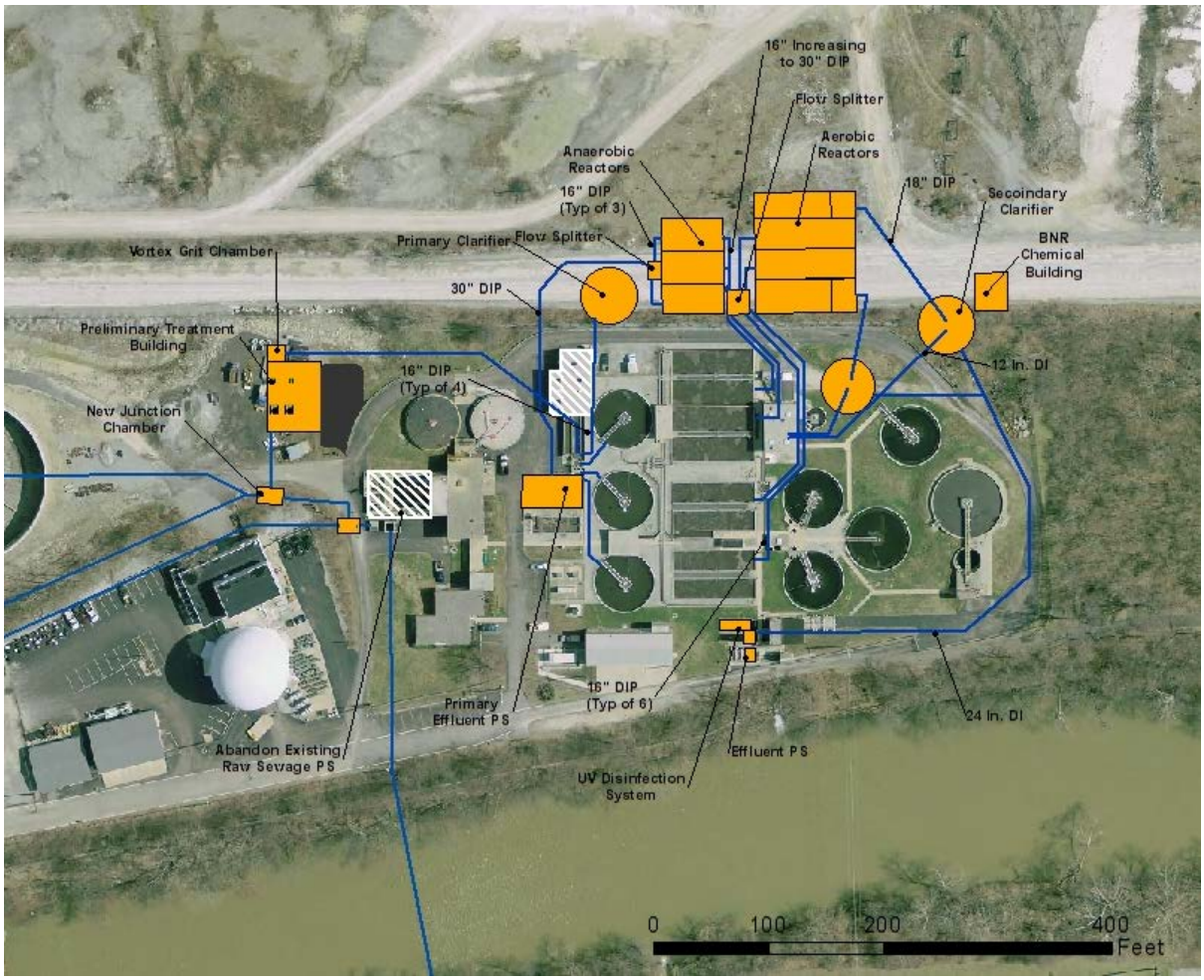

Figure 4-3: Alternative No. 1 Conceptual Layout

Due to the existing tankage configuration and capacity, the most feasible way of implementing the A2O process is to construct a new anaerobic reactor process upstream of the existing bioreactors. This anaerobic process would be built to the east of the existing treatment plant, and would receive primary effluent and return activated sludge (RAS). Three parallel anaerobic reactors are anticipated in this alternative. Flow from the anaerobic basins would then be combined and conveyed to a new flow splitter box.

Flow from this box would be split into six anoxic-aerobic bioreactors – the four existing tanks and two additional trains required to provide sufficient process tankage to meet the treatment goals. Each of the six bioreactors would include an anoxic volume, approximately twice as large as the anoxic zones in the current tank layout, with the remaining tankage used for aerobic volume. The tanks would be equipped with new anoxic mixing, air diffuser systems and other equipment necessary to operate and monitor the process. New aeration blowers would be included to provide sufficient capacity and to replace the existing, aged equipment.

Two new secondary clarifiers would also be required, for a total of six. This option's configuration does not lend itself to improving the existing flow split constraints downstream of the aeration basins; instead, the most feasible approach is to dedicate the two new clarifiers to the two new bioreactors

and "live with" the flow split challenges that the plant has experienced in the past. An upgraded RAS pumping system would be necessary to return settled sludge to the anaerobic tanks. As part of this alternative, the clarifier mechanisms in each of the four existing clarifiers would be replaced due to their age and condition.

The process sizing and anticipated treatment accomplished was developed using BioWin 4.0, a wastewater process simulation software package commonly utilized in process design practice. Among other benefits of using this tool for design is that it is able to balance and predict the impacts of competing biological reactions. As an example, the process simulation modeling predicts that during maximum-month loading conditions, there will be a need to add supplemental carbon to facilitate both enhanced biological phosphorus removal and denitrification. A new chemical building is included to house the required storage and feed facilities.

Table 4-5 summarizes the design criteria for this alternative.

Table 4-5: Design Criteria for Alternative No. 1 – A2O Process

Primary Clarifiers	
No.	4
Diameter (ft)	55
Surface Area, total (sf)	9500
Surface overflow rate (gpd/sf)	
- Design average	740
- Peak	1470
Anaerobic Volume	
Number of Trains	3
Total volume (MG)	0.426
Anoxic/Aerobic Bioreactors	
Number of Trains	6
Anoxic volume, total (MG)	0.872
Aerobic volume, total (MG)	1.704
Design MLSS, mg/L	2800
Design aerobic SRT, days	10.5
Secondary Clarifiers	
No.	6
Diameter (ft)	55
Surface Area, total (sf)	14300
Surface overflow rate (gpd/sf)	
- Average	490
- Peak	980
Solids Loading rate (ppd/sf)	
- Average	20 (RAS ratio 0.75)
- Peak	34 (RAS ratio 0.5)

Advantages and Disadvantages

The main advantage of this alternative at Piqua is that it makes use of the existing treatment tankage, therefore minimizing the overall project's new footprint and tankage requirements (and therefore the capital cost to construct tankage). Most of the existing concrete tanks are structurally sound and are likely to require relatively minor repairs. From a process perspective, this option is a proven configuration to provide enhanced biological phosphorus removal, and would represent the least amount of change from the current operation.

There are several disadvantages to this alternative, however:

- *Needed facilities to address hydraulic constraints* – Because this alternative makes use of the existing primary clarifiers, bioreactors and secondary clarifiers, it must address the hydraulic limitations of the existing plant layout. This necessitates the need for the primary effluent lift station described above, as well as the new splitter boxes. In addition, as noted, this alternative does not fix the existing flow split difficulty between the bioreactors and the secondary clarifiers.

- *Intricate construction sequence to maintain plant operations* – The treatment plant will be required to maintain operations throughout the construction phase of the project, and therefore it will be necessary to upgrade one train at a time – i.e., one primary clarifier, one bioreactor, one secondary clarifier. This will extend the overall construction duration substantially, and will require a complex interim operations plan as upgraded tanks are placed in service side-by-side with non-upgraded tanks.

- *High component count* – As described, this alternative would require a fourth primary clarifier, a 5th and 6th bioreactor and a 5th and 6th secondary clarifier. Six biological treatment trains is a lot for a plant the size of Piqua's. In addition, there would need to be a new primary effluent pump station and two new splitter boxes to accommodate the high number of process trains. All of these new facilities will require operations and maintenance attention for the service duration of the WWTP.

- *Limited flexibility for future expansion or additional peak flow* – As described in Chapter 2, the alternatives are being evaluated for their ability to treat a peak flow of 14 MGD, based on an influent equalization (EQ) volume of 6 million gallons; however, it may be preferable to treat at a higher rate than 14 MGD and reduce EQ volume. This alternative would not allow for an increase in peak flow rate, and further expansion beyond the site configuration shown in Figure 4-4 would be infeasible.

4.5.2 Alternate No. 2 – 3.0-MGD Oxidation Ditch, Parallel to Existing Process

Description

Alternative No. 2 includes construction of a new 3.0-MGD oxidation ditch and support facilities to the east of the existing treatment plant, and then operating the new oxidation ditch in parallel with the existing plant, which would be upgraded to provide a treatment capacity of 4.0 MGD. A conceptual layout for Alternative 2 is provided in **Figure 4-4** to assist in the description and evaluation of this alternative.

The new 3.0-MGD oxidation ditch process would be fed by gravity from the new preliminary treatment facility described earlier in this section. The new biological process would consist of one train consisting of an anaerobic zone followed by an oxidation ditch configured to provide both anoxic

and aerobic process volume. Therefore, the biological configuration of the oxidation ditch would be A2O, similar schematically to Alternative No. 1. Mixed liquor effluent from the single oxidation ditch would be split to two new secondary clarifiers. A new RAS pump station would serve the two new clarifiers and pump RAS back to the anaerobic zone. As is typical for oxidation ditch processes, the flow treated in the ditch would be raw influent, not primary effluent.

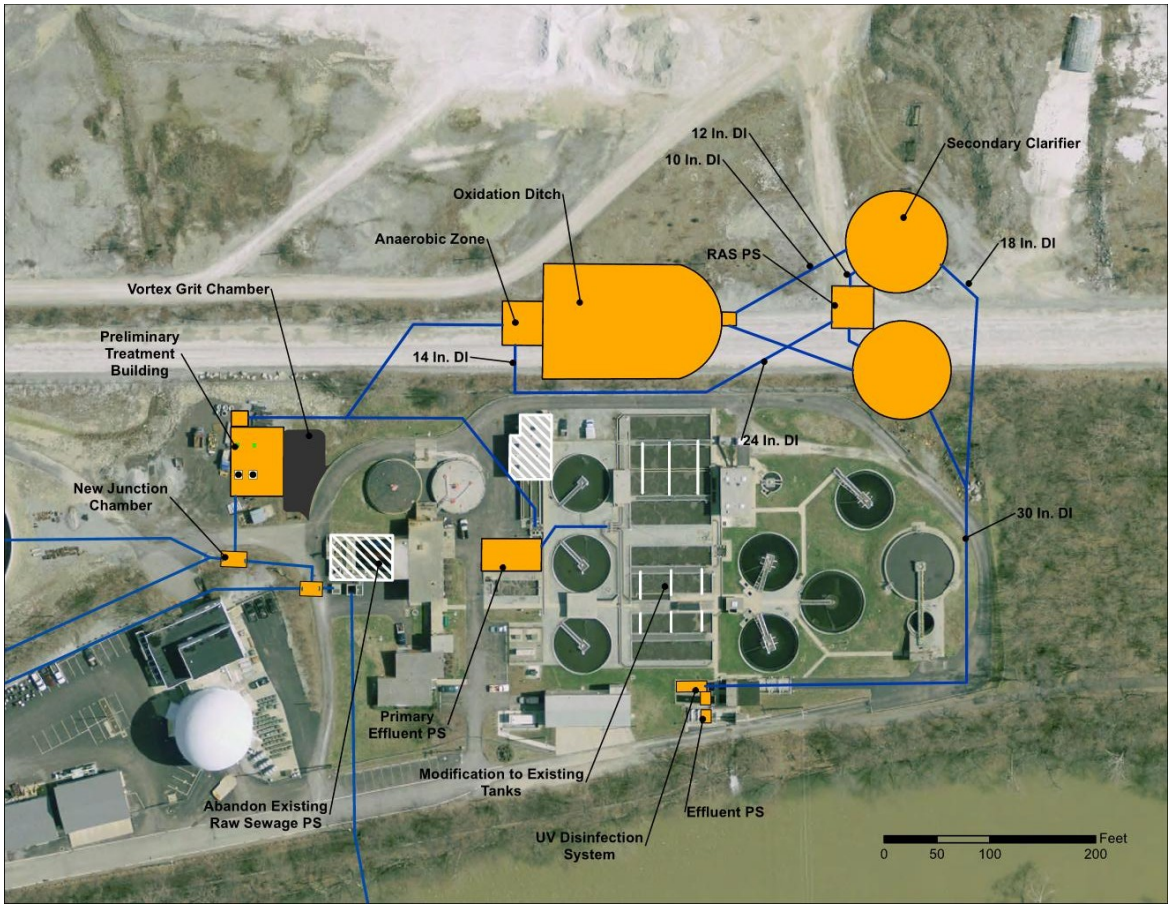

Figure 4-4: Alternative No. 2 Conceptual Layout

However, the plant's existing three 55-ft diameter primary clarifiers would continue to provide primary treatment for flow to be treated in the plant's existing bioreactors. The existing hydraulic constraints downstream of the existing primary clarifiers would need to be addressed, by construction of a new primary effluent pump station which would receive primary effluent from each clarifier, and lift it to a new splitter box that would feed the four existing bioreactors. As part of this alternative, the clarifier mechanisms in each of the three existing clarifiers would be replaced due to their age and condition.

In order to achieve an A2O process configuration within the existing four bioreactors, a significant expansion and upgrade work would be required. In the overall plant layout of Alternative No. 2, the most feasible way of implementing the A2O process is to construct a new anaerobic reactor zone within each of the existing bioreactors. Each of the four bioreactors would include an anaerobic volume, anoxic volume, and aerobic volume. The tanks would be equipped with new anaerobic and anoxic mixing, air diffuser systems and other equipment necessary to operate and monitor the

process. New aeration blowers would be included to provide sufficient capacity and to replace the existing, aged equipment.

This option continues use of the existing four secondary clarifiers, and the configuration does not solve the difficult existing flow split constraints downstream of the bioreactors. An upgraded RAS pumping system would be necessary to return settled sludge to the bioreactors. As part of this alternative, the clarifier mechanisms in each of the four existing clarifiers would be replaced due to their age and condition. **Table 4-6** summarizes the design criteria for this alternative.

Table 4-6: Design Criteria for Alternative No. 2 – 3.0-MGD Oxidation Ditch

Primary Clarifiers	
No.	3
Diameter (ft)	55
Surface Area, total (sf)	7130
Surface overflow rate (gpd/sf) - Average - Peak	 560 1120
Anaerobic Volume (new)	
Number of Trains	1
Total volume (MG)	0.141
Oxidation Ditch (new)	
Number of Trains	1
Anoxic volume, total (MG)	0.438
Aerobic volume, total (MG)	1.12
Design MLSS, mg/L	4000
Design aerobic SRT, days	10.5
A2O Bioreactors (existing)	
Number of Trains	4
Anaerobic volume, total (MG)	0.14
Anoxic volume, total (MG)	0.41
Aerobic volume, total (MG)	1.10
Design MLSS, mg/L	2800
Design aerobic SRT, days	10.5
Secondary Clarifiers	
No.	6
Diameter (ft)	4 at 55; 2 at 95
Surface Area, total (sf)	9500 (existing); 14,200 (new)
Surface overflow rate (gpd/sf) - Average - Peak	 420 (existing); 210 (new) 840 (existing); 420 (new)
Solids Loading rate (ppd/sf) - Average - Peak	 17 (existing); 12 (new) 29 (existing); 21 (new)

Advantages and Disadvantages

The main advantage of this alternative, similar to Alternative No. 2, is that it makes use of the existing treatment tankage, therefore minimizing the overall project's new footprint and tankage requirements. The oxidation ditch process is well-proven and reliable, is relatively simple to operate, and it also represents a slight reduction in the component count required for Alternative No. 1.

There are several disadvantages to this alternative:

- *New, significant process tankage required* – This alternative includes construction of considerable new facilities as shown on Figure 4-5.There is a bigger site impact (a larger expansion of plant property would be required) as well as cost.
- *Needed facilities to address hydraulic constraints* – Similar to Alternative No. 1, this alternative makes use of the existing primary clarifiers, bioreactors and secondary clarifiers, and it must address the hydraulic limitations of the existing plant layout. This necessitates the need for the primary effluent lift station described above, as well as a new splitter box. In addition, as noted, this alternative does not fix the existing flow split difficulty between the bioreactors and the secondary clarifiers.
- *Intricate construction sequence to maintain plant operations* – Although this alternative requires a slightly less intricate construction sequence than Alternative No. 1, it will still be necessary to upgrade one existing train at a time. This will extend the overall construction duration, and will require a complex interim operations plan as upgraded tanks are placed in service side-by-side with non-upgraded tanks.
- *Operation of two parallel, separate treatment processes* – Perhaps the most significant disadvantage of this alternative is that it would require ongoing parallel operation of two separate biological treatment processes, with different loading, solids generation, operating MLSS concentrations, etc., essentially doubling the amount of operator attention required to monitor the process.

4.5.3 Alternate No. 3 – New 7.0-MGD Oxidation Ditch Process

Description

Alternative No. 3 includes construction of a new 7.0-MGD oxidation ditch process and support facilities to the east of the existing treatment plant, replacing the existing biological treatment process in its entirety. After construction of the new facilities, the existing process facilities will be abandoned, with demolition of mechanical systems that are no longer needed. A conceptual layout for Alternative 3 is provided in **Figure 4-5** to assist in the description and evaluation of this alternative.

The new 7.0-MGD oxidation ditch process would be fed by gravity from the new preliminary treatment facility described earlier in this section. The new biological process would be comprised of two trains, each consisting of an anaerobic zone followed by an oxidation ditch configured to provide both anoxic and aerobic process volume. Similar to Alternative Nos. 1 and 2, the biological configuration of the oxidation ditches would be A2O. Mixed liquor effluent from the oxidation ditches would be split to three new 120-ft diameter secondary clarifiers. A new RAS pump station would serve the three new clarifiers and pump RAS back to the anaerobic zones. As is typical for oxidation ditch processes, the flow treated in the ditch would be raw influent, not primary effluent. Like the existing biological process tankage and facilities, the plant's existing three 55-ft diameter primary clarifiers would be abandoned after commissioning of the new oxidation ditch process.

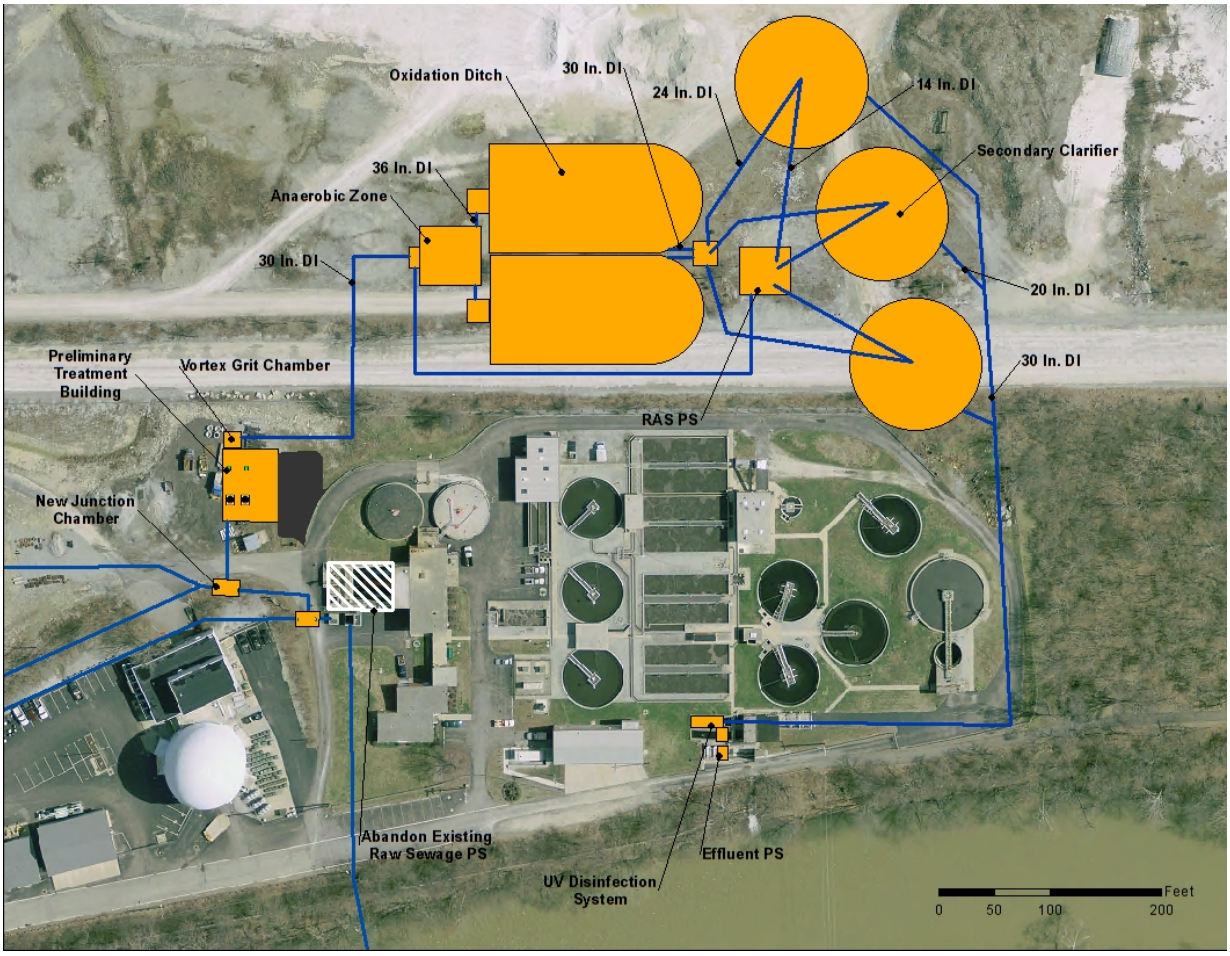

Figure 4-5: Alternative No. 3 Conceptual Layout

Table 4-7 summarizes the design criteria for this alternative.

Table 4-7: Design Criteria for Alternative No. 3 – 7.0-MGD Oxidation Ditch

Anaerobic Volume	
Number of Trains	2
Total volume (MG)	0.33
Oxidation Ditches	
Number of Trains	2
Anoxic volume, total (MG)	1.04
Aerobic volume, total (MG)	2.61
Design MLSS, mg/L	4000
Design aerobic SRT, days	10.5
Secondary Clarifiers	
No.	3
Diameter (ft)	120
Surface Area, total (sf)	33900
Surface overflow rate (gpd/sf) - Average - Peak	 210 410
Solids Loading rate (ppd/sf) - Average - Peak	 12 21

Advantages and Disadvantages

The oxidation ditch process is well-proven and reliable, and is relatively simple to operate. Personnel from the City of Piqua have conducted site visits to existing oxidation ditch facilities and fully understand and are very comfortable with the process. Implementation of Alternative No. 3 would also result in a significant reduction in the component count required for Alternative Nos. 1 and 2. The ditches themselves have minimal mechanical equipment (the aerators), and there would be only two biological trains to operate and maintain.

The primary disadvantage to this alternative is that it would require a significant amount of new site footprint to the east of the existing process, more than the other alternatives.

4.5.4 Alternate No. 4 – New 7.0-MGD Sequencing Batch Reactors (SBRs)

Description

Alternative No. 4 includes construction of a new 7.0-MGD sequencing batch reactor (SBR) process and support facilities to the east of the existing treatment plant, replacing the existing biological treatment process in its entirety. Similar to Alternative No. 3, after construction of the new facilities, the existing process facilities will be abandoned, with demolition of mechanical systems that are no longer needed. A conceptual layout for Alternative 4 is provided in **Figure 4-6** to assist in the description and evaluation of this alternative.

The new 7.0-MGD SBR process would be fed by gravity from the new preliminary treatment facility described earlier in this section. The new SBR process would be comprised of four separate reactors, each equipped with mechanical mixing, aeration diffusers and a decanter mechanism to enable the SBR to phase through anoxic, aerobic, settling and decanting periods during each treatment cycle.

Though some SBR installations have reported a degree of enhanced biological phosphorus removal (EBPR), the SBR process is not conducive to reliable EBPR. Therefore, implementation of the SBR process would include construction and operation of a new coagulant feed system, to achieve chemical phosphorus removal. There are no secondary clarifiers or RAS pumping systems required for SBRs, as the reactors themselves accomplish solids separation in the settling phase. The SBRs would be fed process air from a new aeration blower system, to be installed in an adjacent building.

The decant rate from SBRs exceeds the influent flow rate, because of the necessary restrictions on available decant time in each cycle. This alternative includes a post-SBR equalization tank to absorb the peak decant rates and provide a more reasonable flow rate to the downstream disinfection process.

As is typical for SBR processes, and again similar to oxidation ditches, the flow treated in the SBR would be raw influent, not primary effluent. Like the existing biological process tankage and facilities, the plant's existing three 55-ft diameter primary clarifiers would be abandoned after commissioning of the new oxidation ditch process.

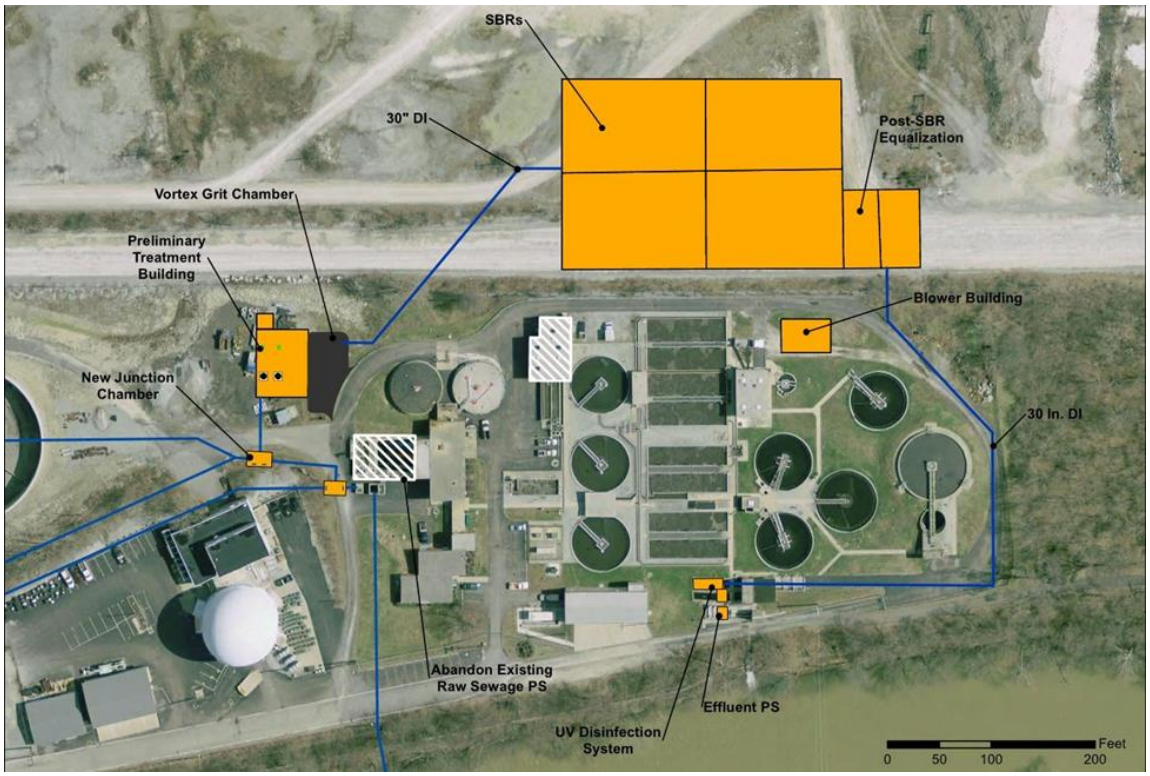

Figure 4-6: Alternative No. 4 Conceptual Layout

Table 4-8 summarizes the design criteria for this alternative.

Table 4-8: Design Criteria for Alternative No. 4 – 7.0-MGD SBR

SBRs	
Number of Trains	4
Total volume (MG)	6.20 (at AWL)
Design MLSS, mg/L	4500 (at AWL)
Design aerobic SRT, days	10.5
% of total cycle time aerated	50
Post-SBR Equalization	
Number of Tanks	1
Total Volume (MG)	0.29

Advantages and Disadvantages

The SBR process is well-proven and reliable, although it is somewhat different and requires more intricate controls than continuous-flow activated sludge processes and therefore would require a "learning curve" for proper operation and maintenance. Implementation of Alternative No. 4 would result in a significant reduction in the component count required, a similar advantage as that provided by Alternative No. 3.

There are several disadvantages to this alternative:

- *New, significant process tankage required* – This alternative includes construction of considerable new facilities as shown on Figure 4-7.The overall site impact is similar, though not as extensive, to that of Alternative No. 3.
- *Unfamiliar Process to Piqua Personnel* – The SBR process requires a different operating approach than the continuous-flow processes described in Alternative Nos. 1-3. Though City staff is capable and qualified to operate SBRs, there is an unfamiliarity with the process and therefore confidence in the process is not as high as the other options.
- *Each SBR is Its Own Process* – Similar to a noted disadvantage of Alternative No. 2, implementation of the SBR process would require ongoing parallel operation of four separate biological treatment processes, with different loading, solids generation, operating MLSS concentrations, etc., substantially increasing the amount of operator attention required to monitor the process.
- *Hydraulic Grade Line* – SBRs require a much higher water surface differential from the inlet to the discharge of the process than the continuous-flow options discussed. This is because the SBR must be able to accommodate a range of water levels in the tank (which typical may be up to 6-7 feet), plus the additional range in the post-SBR equalization tank (which can be several more feet). In order to implement SBRs at Piqua, the new preliminary treatment building would have to be built at a higher elevation, which would increase the cost of construction plus influent pumping costs among other options.

4.6 Disinfection

As discussed in Section 3, the existing disinfection process is able to meet the more stringent disinfection requirements outlined in the WWTP's operating permit. However, with the increased plant capacity to handle future flows, the City wanted to evaluate different disinfection treatment alternatives to meet the more stringent standards, while providing a safer work environment for its staff and the community. The two selected disinfection processes to further evaluate were chlorination-dechlorination and ultra-violet (UV) disinfection.

4.6.1 Chlorination – Dechlorination

Chlorination is a well-established disinfection technology that is utilized at WWTPs throughout the United States. Dechlorination is the process of removing the residual chlorine from disinfected wastewater prior to discharge to the environment. This evaluation will focus on the use of sodium hypochlorite (NaOCl) as the primary disinfectant chemical.

Bulk NaOCl solution is produced off-site using the chlor-alkali process. The NaOCl solution is delivered to the site typically using tanker trucks (4,500 to 5,000 gals). At a concentration of 12.5 percent, the NaOCl solution is corrosive and can cause severe burns to skin. This concentration of NaOCl also tends to degrade rapidly in the presence of sunlight, heat, and iron, copper, nickel, or cobalt. Bulk NaOCl does off-gas oxygen, which can cause problems with the vapor locking of feed pumps and choking off chemical piping. Storing the bulk solution in an enclosed ventilated building, establishing NaOCl quality specifications, storing the solution for short periods of time, and diluting the high-strength NaOCl solution all help to mitigate the degradation factor. Finally, the storage tanks, metering pumps, contaminant structure, and unloading area would be housed inside a new chemical storage and feed building. The process schematic for bulk NaOCl would be identical to the existing configuration, i.e. NaOCl injection at the upstream end of the chlorine contact basin and sodium bisulfite injection at the end of the contact basin.

Effectiveness

The effectiveness of NaOCl was discussed in Section 3 and it is reasonable to expect very effective disinfection results with NaOCl.

The dechlorination process is a rapid reaction accomplished by a reducing agent, e.g. sodium bisulfite. Although others, such as gaseous sulfur dioxide, sodium sulfite, sodium metabisulfite, or sodium thiosulfate have also been used, sodium bisulfite ($NaHSO_3$) solution is the preferred chemical for dechlorination over sulfur dioxide for safety reasons.

Design Criteria

The general design criteria for evaluating bulk NaOCl as disinfectants at the City's WWTP is presented in **Table 4-9**.

Table 4-9: NaOCl Design Criteria

Process Criteria	Units	Value
ADF	MGD	7
PDF	MGD	14
Contact basin detention time at PDF	minutes	15
Chlorine design dose	mg/L	4
Percent Solution of Bulk Hypochlorite	%	12.5
Specific density of bulk hypochlorite	lbs/gal	9.9
Delivered Hypochlorite	Gal/yr	40,000
Sodium bisulfite dose, mg/L	mg/L	2.5
Percent solution sodium bisulfite	%	38
Specific density of sodium bisulfite	lbs/gal	11.1
Delivered Hypochlorite	Gal/yr	7,000

Implementing Chlorine Disinfection

Chlorine has been successfully used for disinfection at WWTPs around the United States. An expanded chlorine contact basin would need to be constructed in order to achieve the necessary 15 minute contact time at PDF, per the Ten State Standards. A chemical storage and feed building would be required for the bulk NaOCl and sodium bisulfite storage tanks and pumping equipment. The historical chemical feed rates for chlorine and sulfur dioxide were used to estimate NaOCl and sodium bisulfite feed rates. A conceptual markup of the contact basin expansion and new chemical storage and feed building is provided in Appendix C.

Process control and disinfection efficiency can be readily monitored and maintained via total residual chlorine (TRC), although the dechlorination process can be difficult to control when near zero levels of residual chlorine are required. The ratio of sodium bisulfite applied to TRC removed (expressed as Cl_2) typically ranges from 1.4 to 1.6 on a mass basis. Proper dosage is critical to reduce chlorine residuals to non-detectible levels.

Dechlorination with sodium bisulfite is a rapid reaction completed within 10 seconds. However, the EPA considers that contact time of one to five minutes is sufficient, presumably to provide time for complete mixing. To achieve effective dechlorination, complete chemical blending within a few seconds at the point of application is required (CDM Disinfection Report for NYC WPCP).

The chlorination/dechlorination process uses corrosive chemicals (sodium hypochlorite and sodium bisulfite) that can be a threat to treatment plant personnel and the public, thus strict safety measures must be employed.

Capital and Operational Costs

A planning-level cost estimate to expand the chlorine contact basin and construct a new chemical building is $1,200,000. Preliminary estimates from area chemical suppliers indicate that the delivered chemical costs are $1.06/gallon for bulk NaOCl (12.5 percent solution in 4,500 gallon shipments) and $1.55/gallon for sodium bisulfite (38 percent solution).

Table 4-10 – NaOCL Disinfection Operation and Maintenance Cost Estimate

Cost Component	O&M Cost
Annual chemical costs for NaOCl	$ 42,000
Annual chemical costs for Sodium bisulfite	$ 11,000
Annual maintenance costs	$ 7,000
Present Value of Annual Costs	$ 1,070,000

4.6.2 Ultraviolet (UV) Light

The use of UV for disinfecting treated wastewater is widespread in the United States, and is popular in wastewater treatment because of its effectiveness, ease of use, and no chemicals to handle. There are reportedly over 3,500 UV wastewater disinfection systems currently operating in North America, treating flows of up to 300 MGD (CDM Smith Disinfection Report for NYC WPCP). As alluded to, UV disinfection eliminates the operational and environmental hazards associated with the use of chlorine compounds (and sulfite compounds when dechlorination is required), and does not produce harmful disinfection byproducts.

4.6.2.1 Effectiveness of UV

UV is a physical process, relying on the transfer of electromagnetic energy to a microbe's DNA. When absorbed in sufficient quantity (the "dose"), the energy damages the DNA strands by causing specific thymine monomers to combine, which in turn prevents the cell from replicating. This inability to reproduce is the lethal effect of UV. DNA absorbs UV light in the spectral region between 200 and 300 nm, with maximum absorption, and germicidal impact, between 240 and 280 nm. The optimal germicidal wavelength for UV disinfection is 254 nm.

4.6.2.2 UV Configurations

There are several manufacturers of UV systems. These are commercially available in "low-pressure" and "medium-pressure" lamp configurations, driven by electronic ballasts. Medium-pressure lamps are polychromatic and exhibit a continuous spectral UV output between 200 and 400 nm, and have several significant output lines between 240 and 290 nm. With the higher mercury pressures, the lamps are driven at substantially higher input power levels (greater than 1 kW, and as high as 20 kW per lamp) and temperatures (600 to 800 degrees C). They are not as efficient as the monochromatic low-pressure lamps, with conversion of about 7 to 9 percent of their input power to 254 nm output, and 10 to 15 percent total output in the germicidal region. Overall, the medium-pressure lamps require about 4 to 5 times the power than the low-pressure lamps to deliver an equivalent germicidal energy. However, because of their much higher absolute output levels, fewer lamps are needed, often resulting in a smaller footprint for the UV system.

Low-pressure lamp output is optimized via mercury vapor pressure and electric current control, and is effectively monochromatic about the resonance line for mercury, or 253.7 nm, which is very near the optimum germicidal wavelengths for UV disinfection. These low-pressure lamps are highly efficient, converting nearly half of their input energy to light, with 85 percent of this light at 254 nm. The original low-pressure systems' absolute outputs were relatively low, with typical UV ratings of 30 to 50 Watts per lamp at 254 nm, for 80-110 W input lamps. These systems were known as low-pressure low-output technology (LPLO). Advances in these low-pressure lamps, using mercury amalgams and driving the lamps at a higher input power (300 to 500 W) have resulted in higher UV outputs (100 to 150 W), while retaining their highly efficient energy conversion characteristic, known as low-pressure high-output technology (LPHO).The higher input power levels of medium-pressure

systems would be less cost effective than a LPHO system. Therefore, a LPHO system was used for this evaluation.

The lamps of a LPHO system are sheathed in quartz sleeves (highly transmissible in the UV region), and submerged in the flowing wastewater. The lamp/quartz assemblies are typically arranged in modules, with several modules comprising a bank of lamps. The banks of lamps are typically placed in open channels, either horizontally or vertically oriented, with level control devices that maintain water levels above the submergence level of the lamps.

Pressure units, using closed-vessel reactors, are also used for wastewater, although far less frequently than the open-channel designs. Many LPHO systems employ automatic cleaning systems which are integral to the lamp banks, to remove fouling and maintain the transparency of the quartz surfaces. Depending on the manufacturer of the LPHO system, periodically dipping the UV modules into a weak acidic solution is required in addition to the automatic cleaning system.

4.6.2.3 Design Considerations

There are several factors that affect the design of a UV system for wastewater disinfection. These factors will affect the required design dose, defined as the product of the intensity of UV energy (the rate at which it is being delivered) and the time to which the organism is exposed to this intensity. Ideally, these factors can be applied such that all of the wastewater receives the same dose as it passes through the UV unit. But the practical application of UV is not ideal; there is a variable intensity field within the unit and a distribution of exposure times, resulting in a dose distribution. Effective design optimizes this dose distribution and avoids any semblance of hydraulic short-circuiting through the UV unit.

Exposure time is dependent on the hydraulic characteristics of the unit, reflecting the spacing of the quartz/lamp assemblies, inlet and outlet conditions, and hydraulic loading rates. Intensity is affected by the output energy of the lamps, the transmissivity of the quartz sleeves, and the transmittance of the wastewater itself. The loss of energy due to the aging of lamps and degradation of the quartz sleeve transparency must be accommodated in the design and sizing of the UV units.

Generally, the lamp output will decrease to between 50 and 90 % of the nominal output at its end life (typically warranted at 12,000 hours for low-pressure lamps and 5,000 hours for medium-pressure lamps). Quartz fouling will typically account for a 20 to 30 percent decrease in transparency through the life of the quartz sleeve, assuming that the quartz sleeves are routinely cleaned of materials adhering to the surface. The transmittance of treated wastewater effluents generally ranges between 50 and 75 percent and preliminary results indicate that Piqua's WWTP effluent is on the high end of that range. The dose requirement is a key parameter. Typically, a dose of 30,000 to 40,000 µWatts-sec per square centimeter ($\mu W\text{-}s/cm^2$) is specified for treated wastewater disinfection.

The dose requirement is determined by directly testing the response of the targeted organisms to UV dose. This is accomplished via specific laboratory test protocol using a collimated beam apparatus which allows the intensity and time of exposure to be measured precisely, unlike the inability to do so with a flow-through UV unit. This testing will be done with the assistance of UV manufacturers during the preliminary phases of design.

The key parameters that comprise the design basis for a UV system include:

- UV transmittance
- Inlet bacterial densities
- Suspended solids
- Particle densities and size distribution
- Flow rates
- Fouling factors, e.g. hardness and iron concentrations
- Hydraulics

Knowledge of these parameters is essential to meet the NPDES permit discharge limit of 126 CFU/100 mL of *E. coli* and will be further defined during design, particularly the hydraulics. Although these parameters will require thorough evaluation during design, the design criteria used to evaluate UV as a disinfection technology at Piqua is presented in **Table 4-11**.

Table 4-11 – UV Design Criteria

Process Criteria	Units	Value
ADF	MGD	7
PDF	MGD	14
UV design dose	$\mu Ws/cm^2$	30,000
UV system type	-	LPHO
UV channels	No.	2
UV transmittance (minimum)	%	70
Headloss per channel	inches	<12

4.6.2.4 UV Disinfection at Piqua

The UV disinfection system would be located inside the existing chlorine contact basin. Therefore, there would be no change to the current process flow at the WWTP as shown schematically in **Figure 4-7.**

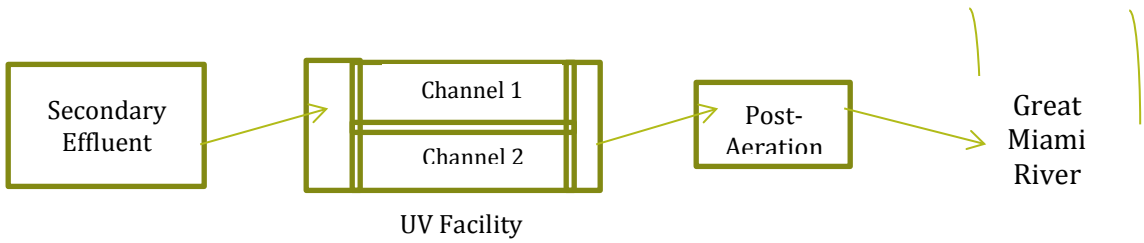

Figure 4-7 – UV Process Flow Schematic

The UV system would consist of two UV channels. Each channel would be able to treat up to 7 MGD of flow (for a total capacity of 14 MGD with both channels). A two-channel configuration allows for increased energy efficiency and ease of operations and maintenance. Keeping the lamps of the UV system submerged is also essential to ensure efficient and proper operation of the UV system. Submergence of the lamps would be provided by effluent weirs located at the end of each UV channel. A potential layout of a UV system is shown in **Figure 4-8**. Note that a design approach is shown that would allow both UV channels to be constructed within one run of the existing chlorine contact basin, allowing the other to be used for flow through.

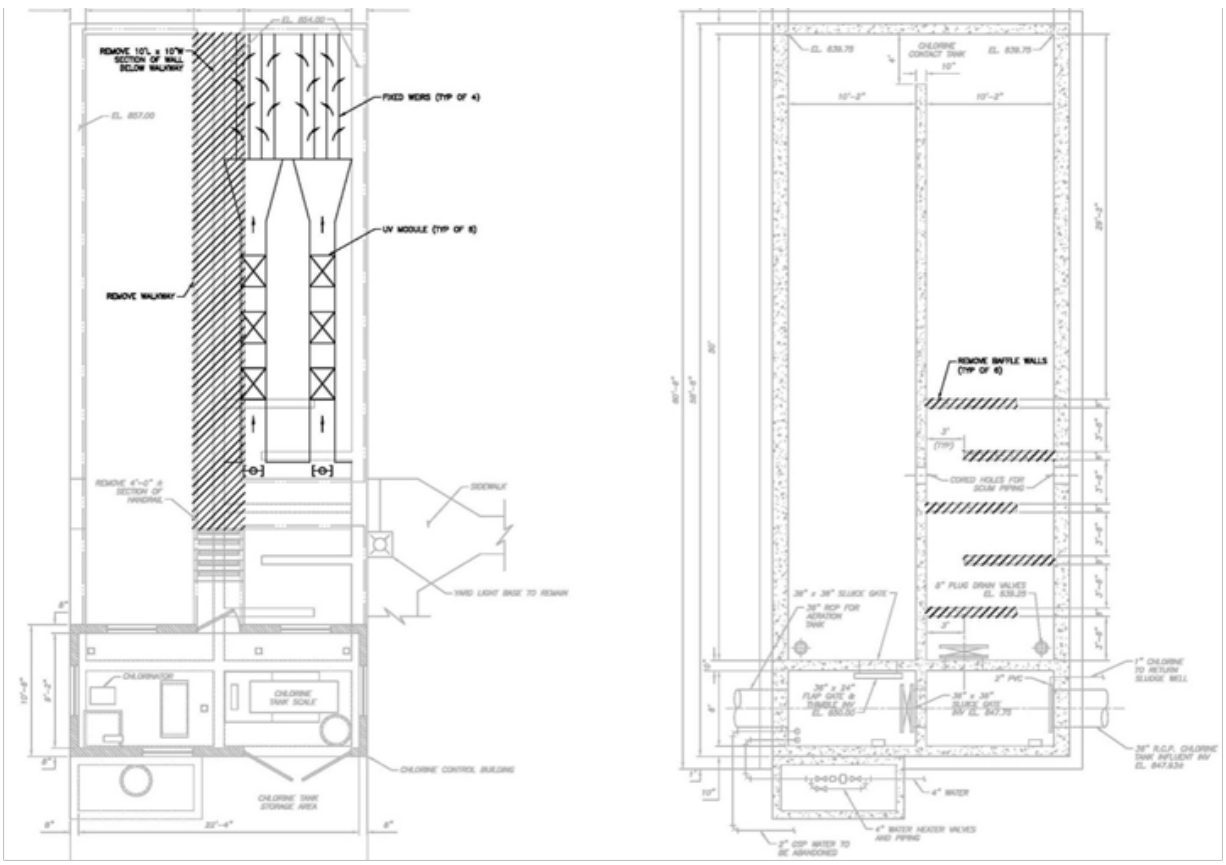

Figure 4-8 – Conceptual UV Layout and Modifications

With UV as the primary plant disinfectant, supplemental chlorine addition would still be necessary to meet the needs of the non-potable water (NPW) system and to continue to control filamentous bacteria through the RAS chlorination system. The chlorine dosage is infrequent and minimal for these purposes, but must still be included in the capital cost estimate. System design requirements and basis of analysis will require more consideration and coordination with the City staff to come to a sustainable long-term basis for design. As such, this component of the analysis will be evaluated further during preliminary design.

4.6.2.5 UV Disinfection Cost

A planning-level cost estimate to construct a UV system within the existing chlorine contact chamber was estimated to be $1,200,000. An inclined UV system was used for this planning level-cost estimate. However, during design, a thorough evaluation of vertical, horizontal, and inclined UV systems should be conducted to determine the best possible configuration for Piqua.

Table 4-12 – UV Operation and Maintenance Cost Estimate

Cost Component	O&M Cost
Annual electricity costs for UV	$ 5,100
Annual costs for lamp replacement	$ 2,500
Annual costs for ballast replacement	$ 800
Annual maintenance costs	$ 15,000
Present Value of Annual Costs	$ 440,000

4.7 Summary of Liquid Stream Process Evaluation

4.7.1 Construction Costs

This section presents budgetary cost estimates for each of the four biological process alternatives. The estimates involve a significant amount of judgment at this stage of planning and should be considered only approximate but relative to each other in accuracy. Generally, planning level estimates are considered to have an accuracy of +/- 30 percent. More refined estimates will be developed at each subsequent design phase of the project.

An Engineer's Opinion of Probable Construction cost was developed for each alternative using supplier quotations and historic cost data and published data. A comparison of the alternative capital costs follows:

- Alternative No. 1 – A2O Process - $28,000,000
- Alternative 2 – 3.0-MGD Oxidation Ditch/Existing Process - $33,000,000
- Alternative 3 – 7.0-MGD Oxidation Ditch - $33,000,000
- Alternative 4 – 7.0-MGD SBR - $31,000,000

Contractor insurance, bonds, general conditions and overhead and profit were assumed to be 15% of the total construction cost including contingency.

A construction cost contingency of 30% is added to the construction cost of each alternative. A 30% contingency is appropriate at a planning level to allow for unforeseen and undefined cost items.

Cost estimates were increased an additional 9.5% to adjust the value from present dollars to future dollars. The adjustment accounts for an escalation to the mid-point of construction using an assumed 2% per year inflation rate over that period.

It is important to note that the cost estimates are preliminary planning level costs based on information available at the time of the estimates and are considered to be "order of magnitude". The actual cost of the recommended alternative will depend on actual labor and material costs, competitive market conditions, final project scope, implementation schedule, and other factors. As a result, the final costs will most likely vary from the estimates presented herein.

4.7.2 Operation and Maintenance Costs

Annual operation and maintenance costs were estimated for each of the four liquid process alternatives. The expected electricity usage and maintenance costs were evaluated. Labor costs were not taken into consideration as part of this evaluation. It was not expected that the staffing of the plant would be impacted as a result of advancing WWTP improvements associated with any of the alternative processes.

Electrical costs were estimated for each alternative. The anticipated equipment horsepower, and expected operation duration were used to estimate the total amount of Kw-hours used over a year. Using a unit price of $0.06/Kw-hour, the total annual electrical cost for each alternative was estimated

To develop an estimate for operation and maintenance cost, independent of labor, a flat estimate of 0.02% of the total equipment cost was used. This percentage should cover the annual cost of spare parts and materials for repairs.

4.7.3 Present Worth Analysis

The cost analysis of the alternatives includes the development of total present worth costs based on construction and annual O&M costs. The cost figures developed not only facilitate the direct comparison between alternatives but also indicate the magnitude of the cost for implementing each Alternative.

The cost estimates are based on the planning level design of each alternative to determine the equipment, land area, process building, structure requirements, electrical utility, maintenance, and staffing requirements. Construction and annual O&M costs of similar facilities constructed were considered in the cost analysis as well as information provided by manufacturers of the various processes and past budgets for operation of the Piqua plant.

The construction and O&M costs are compared using a 20-year life and an interest rate of 3.5 percent. The present worth cost includes both construction and O&M costs over the next 20 years. The analysis assumes that the facilities are constructed at one time and the constant O&M costs start at the same time and continue over the 20-year period. This procedure converts these costs over the project life into an equivalent cost that represents the current investment that would be required to satisfy all of the identified project costs for the planning period.

The cost analysis of the alternatives is based on the following specific parameters:

- Project design period (useful life of the facilities) = 20 years
- Interest rate = 3.5 percent
- Present worth factor = 14.23 (O&M cost x 14.23)
- Electricity cost = $0.06 per Kw-hr
- Gas cost = $7.35 per mmBTU
- Maintenance cost = 0.02% of total equipment costs

Engineering design, construction management, and legal are not included in the costs, but presumed to be similar between the alternatives. The O&M cost estimates are based on the average daily flows and peak hourly flows anticipated during the 20-year design period. Administration and laboratory costs are included in the annual O&M cost estimates.

Table 4-13 provides a summary of the present worth cost analysis for the four alternatives.

Table 4-13: Present Worth Cost of Liquid Treatment Alternatives

	1	2	3	4
	A2O Process & Existing Plant Upgrade	3.0-MGD Oxidation Ditch with Upgraded Existing Plant	New 7.0-MGD Oxidation Ditch Treatment Plant	New SBR Treatment Plant
Probable Construction Cost	**$28,000,000**	**$33,000,000**	**$33,000,000**	**$31,000,000**
Annual O&M Costs				
Electricity	$240,000	$259,000	$188,000	$184,000
Maintenance	$96,000	$115,000	$86,000	$74,000
Total O&M Costs	$336,000	$374,000	$274,000	$258,000
Present Worth O&M Costs	**$ 4,775,000**	**$5,315,000**	**$3,894,000**	**$3,667,000**
Total Present Worth Cost[1]	$32,775,000	$38,315,000	$36,894,000	$34,667,000

[1] Present worth cost is calculated at 3.5% interest for 20 years

4.7.4 Non-Economic Evaluation

In addition to evaluating relative capital cost and relative operation and maintenance cost, each alternative was evaluated on non-economic criteria:

- **Maintenance of Plant Opeartions (MOPO):** Ease of operating the plant during construction of the proposed process.
- **Treatment Efficiency:** Relative level of treatment the proposed process is able to provide.
- **Ease of Operation:** Level of effort required to operate and maintain the proposed process.
- **Flexibility for Expansion:** Expandability, including use of modular construction so that the facility can be expanded as future demand increases.

- **Ability to Meet Future Regulations:** The proposed process's ability to meet treatment requirements that may be expected under future permits.
- **Reliability and Risk:** How established the proposed process is in wastewater treatment plants in a similar climate with similar capacity and similar wastewater characteristics.
- **Implementation:** The relative ease with which the proposed process could be constructed.

The non-economic comparison of alternatives is presented in **Table 4-14**. Numerical ratings from 1 to 5 were assigned to each factor. A rating of 1 is poor and a rating of 5 is excellent.

Table 4-14: Weighted Ranking of Liquid Treatment Alternatives

Liquid Process Alternative	Relative Capital Cost	Relative O&M Costs	MOPO	Treatment Efficiency	Ease of Operation	Flexibility for Expansion	Ability to Meet Future Regulations	Reliability and Risk	Implementation	Weighted Sum
Weighting	18%	15%	10%	3%	12%	10%	10%	15%	7%	
1 A2O	4	3	2	3	2	1	3	4	2	**2.84**
2 3.0-MGD Oxidation Ditch with Existing WWTP	3	2	3	3	2	2	3	3	3	**2.63**
3 7.0-MGD Oxidation Ditch – Abandon Existing WWTP	3	4	4	3	4	4	3	5	4	**3.84**
4 Sequencing Batch Reactor (SBR)	4	3	4	3	2	4	3	4	3	**3.41**

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Alternatives 3 and 4 received the highest (best) weighted sums for the four alternatives evaluated. Alternative 3 received the highest overall rating. That alternative's reliability, ease of implementation and ease of operation set it apart as the highest-rated alternative.

4.8 Conclusions and Recommendations

Four biological liquid stream alternatives were developed and compared in terms of maintenance of plant operations (MOPO) during construction, treatment efficiency, ease of operation, flexibility for future expansion, ability to meet future regulations, reliability/risk, and implementation., annual O&M, and present worth/life cycle costs. Advantages and disadvantages of each alternative related to cost and non-cost parameters were identified. This information developed and analyzed assisted in arriving at the recommended liquid stream process for the Piqua WWTP.

Based on the economic and non-economic analysis of the four alternatives, Alternative No. 3 – a new 7.0-MGD oxidation ditch process – was selected for implementation at the Piqua WWTP. Though not the lowest cost alternative, both the capital and ongoing O&M costs for this alternative do not vary significantly from the other alternatives evaluated. When the non-economic advantages of this alternative are included in the evaluation, the overall score of Alternative No. 3 indicates that it is the preferred alternative for implementation at Piqua.

Section 5

Development of Solids Stream Alternatives

5.1 General

The Piqua Wastewater Treatment Plant (WWTP) is a conventional activated sludge plant rated for 4.5 MGD. The liquid treatment scheme at the WWTP consists of screening, grease and grit removal, primary settling, activated sludge aeration, secondary settling, chlorination, dechlorination, and post aeration. Plant effluent is discharged to the Great Miami River. The solids treatment scheme involves separate primary and waste activated sludge pumping, anaerobic digestion, sludge dewatering, and land application.

A key to a reliable wastewater treatment plant operation is the effective management of the wastewater solids generated at the plant. Wastewater solids are removed from the wastewater stream by physical unit processes and produced by biological processes during sewage treatment. These solids include screenings, grit, scum, and sludge. The Federal Part 503 Standards for the use and disposal of sewage sludge define sewage sludge as a solid, semi-solid, or liquid residue generated during treatment of sewage in a treatment plant. Sewage sludge includes scum or solids removed in primary, secondary, or advanced wastewater treatment processes but does not include grit and screenings. Organic residuals from primary and secondary treatment constitute most of the sludge. Piqua employs three circular primary settling tanks and four circular secondary settling tanks to remove sewage sludge from the liquid wastewater stream.

At a certain point in the solids processing scheme the sludge is referred to as biosolids. Biosolids are primarily organic solids produced by WWTP stabilization processes that can be beneficially reused or recycled. The term biosolids is used only after the beneficial use criteria have been achieved through a sludge stabilization process. The United States Environmental Protection Agency (USEPA) and the State of Ohio Environmental Protection Agency (OEPA) are very supportive of the beneficial use of biosolids in their sewage sludge regulations and emphasize the beneficial nature of this valuable, recyclable resource.

The current biosolids management plan at Piqua consists of anaerobic digestion and land application of the biosolids. Sludge treatment/stabilization processes convert sewage sludge to a stable end product by reducing pathogen (disease-causing organism) levels in the sludge and offensive odors. Anaerobic digestion of wastewater sludge is approved by the OEPA and USEPA as a Process to Significantly Reduce Pathogens (PSRP) and a Process to Further Reduce Pathogens (PFRP). PSRP and PFRP are the criteria for alternate levels of pathogen reduction (i.e., Class B and Exceptional Quality (EQ) biosolids, respectively) as required by the federal and state regulations prior to land application and/or distribution and marketing. Anaerobic digestion also meets vector (e.g., insects, birds, rodents, etc.) attraction reduction (VAR) requirements set by the USEPA and OEPA. The Piqua anaerobic digestion process is a PSRP producing Class B biosolids and meeting the vector attraction reduction requirements. The digested biosolids are stored in a tank prior to dewatering via a belt filter press. A private contractor (Burch Hydro) operates the belt filter press and then transports and land applies the biosolids on nearby farmlands.

The Piqua plant currently utilizes a two-stage, primary-secondary, mesophilic anaerobic digestion system. Primary sludge and unthickened waste activated sludge (WAS) is sent first to the Primary Digester that is used to provide active mixing, heating, and digestion. Next, the sludge is transferred to the Secondary Digester that serves as a solid-liquid separator; it provides settling and separation of the sludge solids from the excess water (supernatant). The methane gas which is produced during the digestion process is used to run the gas engine driven aeration blower or burned to heat the digesting sludge. The concentrated digested sludge is removed from the Secondary Digester and pumped to either the Sludge Truck Loading Station or the Digested Sludge Storage Tank. The supernatant, which is drawn from the top portion of the Secondary Digester, flows by gravity to either the plant main drain for additional treatment or to the Supernatant Pump Station from which it is pumped to the Supernatant Oxidation Tank for additional treatment.

The two-tank system is comprised of the following major components:

- A 50-foot diameter Primary Digester with sidewater depth (SWD) of 20 feet, a fixed cover, an external heating system, and a gas mixing system.
- A 50-foot diameter Secondary Digester with SWD of 20 feet and a gas holding, floating cover held up by the pressure of the sludge gas produced mainly from the Primary Digester. There are no heating or mixing systems for the Secondary Digester.

The Primary Digester heating system consists of process hot water piping, sludge circulation piping, digester sludge recirculation pumps, a process water pump, an engine jacket water pump, a sludge heater, a sludge heat exchanger, a process heat exchanger, and an excess heat exchanger.

The mixing system in the Primary Digester consists of three 18-inch diameter by 17-feet long mixing guns that are symmetrically positioned on top of the digester cover. Each mixing gun assembly includes a mixing tube and a gas distributor which generates intermittent gas piston-like bubbles at a controlled frequency to produce a continuous sludge flow through the gun stack. Two liquid ring-type mixing compressors are used to provide the compressed gas flow of 60 SCFM required for the operation of the mixing guns, which provide a scouring velocity along the bottom of the digester to prevent the deposition of heavy solids and organic materials.

The anaerobic system uses two sludge recirculation pumps, which withdraw sludge from the Primary Digester and pump it though the sludge heating system then back into the Primary Digester to maintain a constant temperature, or could be used to circulate supernatant to the Secondary Digester. Digested sludge pumps are used to pump digested sludge from the Digested Sludge Draw-off Well to either the Sludge Truck Loading Station or to the Digested Sludge Storage Tank.

The two-stage digester gas handling system consists of meters, filters, a waste gas burner, and piping. The digester gas produced in the Primary Digester can either be burned in the sludge heater for the digester heating system or used to run the gas engine driven aeration blower. Any unused gas can be burned in the waste gas burner.

5.1.1 EQ Biosolids versus Class B Biosolids – Regulatory Outlook

Since the most recent revisions to the sewage sludge regulations in the State (effective July 1, 2011) make them stricter, the City of Piqua is concerned with the long-term viability of their Class B sludge digestion system. Ohio WWTPs must meet the sewage sludge regulations in the Part 503 (including all amendments) as well as Ohio's sewage sludge regulations enforced by the OEPA. As a delegate, OEPA

has the exclusive authority to revise the current sludge disposal regulations as long as the Federal regulations are met. That is, OEPA can make the sludge regulations more stringent. In fact, Ohio's sewage sludge regulations are considered to be stricter than other states in the country. Over the years, OEPA has made revisions to the regulations mainly with the permitting and management of Class B biosolids. The most recent changes now regulate the management practices of bulk EQ biosolids and impose stricter land application requirements, such as the prohibition of surface application of Class B biosolids and bulk EQ biosolids from December 15 to March 31. Only injection or incorporation within 24 hours of production are not banned. This requirement reinforces the need for 120-day sludge storage at plants and causes WWTPs that do not have adequate sludge storage to look at other means to dispose of their sludge. The regulations now include precipitation restrictions for Class B and bulk EQ biosolids. For example, beneficial reuse is not permitted when the forecast predicts a 50% chance that ½ inch of rain will occur within 24 hours of beneficial use application. Another recent change to the regulations involves screening at the head of the plant. By July 1, 2015, any treatment plant operator who plans on practicing beneficial reuse of biosolids must include fine screening (5/8" maximum aperture or finer) in the liquid treatment train to remove manufactured inert solids from influent sewage, septage, or sewage sludge. Moreover, there is always the recurring threat to ban land application of Class B biosolids, but the likelihood of it being enforced within the next twenty years is highly doubtful. Replacing Class B systems with EQ systems would require additional treatment and have a significant impact on the capital and operation/maintenance costs associated with biosolids management.

According to OEPA records for the Year 2009, only 30 treatment plants in Ohio land apply or distribute and market EQ biosolids, while 336 Ohio plants land apply Class B biosolids. Furthermore, 47% of the biosolids produced in 2009 was either land applied or distributed and marketed, with the remaining 53% disposed of via landfilling and incineration. Any changes in the current regulations to require the production of EQ biosolids would affect many communities, including Piqua. OEPA has indicated that a requirement to produce EQ biosolids is not likely in the foreseeable future. However, it is prudent as part of a comprehensive WWTP plan to examine alternatives to produce EQ biosolids in the event the regulatory climate changes.

5.1.2 Piqua Biosolids Management Plan

Although the Class B anaerobic digestion system at the Piqua treatment plant has generated a very useful product for nearby farmers for several years, the City is concerned with the long-term viability of the current process. Due to age and inadequate performance, replacement of existing equipment is warranted. For instance, the bubble gun mixers are inefficient, the gas holder cover is defective, and gas collection and safety system needs to be upgraded. In addition, the OEPA's most recent ban on surface application during the winter months necessitates additional long-term biosolids storage. The City realizes that upgrades to the current Class B biosolids management system is required. Wastewater treatment staff have also expressed interest in a system that could produce Exceptional Quality biosolids. With Exceptional Quality biosolids sludge application rates would be safe no matter how much biosolids were applied to the land, whereas Class B biosolids have additional restrictions. The goal of this part of the Amended WWTP Facility Plan is to develop a long-term biosolids management program that is environmentally sound, cost-effective, and more importantly meets the needs of the community and is publicly accepted.

5.2 Biosolids Management Options and Initial Screening Process

Numerous technologies can be applied to sludge removed from wastewater for volume reduction, treatment, and stabilization. Common sludge treatment/stabilization technologies include anaerobic digestion, aerobic digestion, autothermal thermophilic aerobic digestion (ATAD), composting, and lime stabilization. Sludge treatment/stabilization processes convert sewage sludge to a stable end product (biosolids). Such processes are the key to an effective, reliable WWTP operation. These treatment processes are used so that various disposal or utilization methods can be undertaken. Essentially, the selection of a stabilization method depends on the utilization/disposal procedure to be used. Biosolids disposal methods include landfilling and incineration. Common biosolids utilization practices include land application to agricultural and non-agricultural lands and distribution and marketing.

In an effort to streamline the comprehensive study process and to involve City staff directly in the decision-making process, CDM Smith conducted a workshop (Solids Treatment Alternatives Screening Workshop #1A) that assessed the existing wastewater solids processing facilities at the Piqua WWTP, and several sludge stabilization and biosolids management alternatives considered as viable options for the City were identified, evaluated, and screened.

In order to develop a Biosolids Management Plan (BMP) for the City of Piqua, the integration of several combinations of treatment/stabilization technologies and disposal/utilization methods with the overall treatment process at the Piqua plant were reviewed and screened. The following BMP alternatives were considered:

- Aerobic Digestion/Land Application
- Mesophilic Anaerobic Digestion/Land Application
- Thermophilic Anaerobic Digestion/Land Application
- Autothermal Thermophilic Aerobic Digestion/Land Application and/or Distribution and Marketing
- Exceptional Quality and Class B Alkaline Stabilization/Land Application and/or Distribution and Marketing
- Composting/Distribution and Marketing
- Indirect Thermal Drying/Land Application and/or Distribution and Marketing
- Burch-Hydro microwave process (BioWave™ Process)/Land Application and/or Distribution and Marketing

Lime stabilization and composting were quickly eliminated. The digestion, thermal drying, and microwave options remained. Aerobic digestion, high-rate anaerobic digestion, TPAD, ATAD, the BioWave™ Process, and indirect thermal drying were assessed based on facility and regulatory requirements and advantages/disadvantages at Workshop #1A. In addition, the solids treatment options were ranked and scored based on weighted evaluation criteria. These alternatives are discussed on the following pages.

The purpose of Section 5 of this report is to develop the remaining solids treatment alternatives considered for installation at the Piqua WWTP.

5.3 Evaluation Criteria

Each alternative is presented through a process description and evaluation of the process. Evaluation criteria are applied to the development and comparison of the alternatives. Facility requirements and regulatory requirements are considered in the screening process. Construction, annual operation and maintenance (O&M) and present worth costs, and other non-cost parameters are established and analyzed in Section 6 of this report.

Facility Requirements

All alternatives are evaluated along with auxiliary equipment or operations that would be required to make a fair comparison of the alternatives. The role of each existing unit treatment process and operational practice in conjunction with the potential new process alternatives in achieving the overall sludge stabilization and process objectives are assessed. In some cases, existing treatment processes would be replaced or upgraded, and in other cases existing processes would be abandoned.

The sizing requirements for the BMP options were established. The facilities were sized and designed for a useful life of 20 years as dictated by the OEPA. CDM Smith reviewed the treatment plant monthly operating reports (MORs) (2008 through 2014). The MORs were collected, compiled, and analyzed to identify current plant flows, sludge production quantities, sludge characteristics, and sludge peaking factors. Current plant flow and sludge production values provided the basis for estimating future flow and sludge production rates. The average influent wastewater flow at the Piqua plant for the examined period was 3.90 MGD. The average sludge production rate for the period was 3,800 dry pounds per day (dppd). A future design flow of 7.0 MGD was projected for the 20-year planning period.

The new sludge stabilization facilities must be sized to handle both the estimated future average sludge production rate and a future maximum quantity – the maximum month sludge production rate. Based on peaking factors obtained from the existing treatment plant data and other facilities similar to the Piqua plant, future sludge loads (and based on future liquid process alternatives) were projected and are summarized in **Table 5-1**.

TABLE 5-1: Maximum Month Sludge Production Rates

Liquid Process Alternative	Maximum Month Sludge Production Rate (dppd)
A_2O Process	9,500
3.0-MGD Oxidation Ditch with Existing WWTP	9,000
New 7.0-MGD Oxidation Ditch	8,300
New SBR Plant	8,300

The following assumptions were made in sizing the facilities:

- Thickened sludge percent solids = 5.0%
- Blended sludge volatile solids = 74%
- Volatile solids reduction = aerobic digestion (40%); anaerobic digestion (50%); TPAD (55%); and ATAD (60%)
- Digested sludge percent solids = 2.0%
- Dewatered digested biosolids percent solids = aerobic (18%); high-rate anaerobic (20%); TPAD (20%); and ATAD (25%)
- Dewatered/dried biosolids percent solids = microwave (70%); indirect thermal drying (90%)

In summary, the facility requirements presented in this section for each of the digestion and drying alternatives are determined based on a projected future maximum monthly digested biosolids production rate ranging from 8,300 to 9,500 dppd. Furthermore, the various treatment processes are sized according to the USEPA and OEPA sludge regulatory requirements for pathogen reduction (for an Exceptional Quality or Class B end product) and vector attraction reduction. Design criteria described in the 10-State Standards, or other widely accepted design parameters, the validity of which have been proven historically, are also used to size the facilities. Variations in these assumptions may be experienced with different liquid process alternates; however, these planning level assumptions are considered appropriate for development of each option. Applicability of each biosolids process to the liquid process alternates will be addressed as appropriate.

5.4 Biosolids Management Alternatives

5.4.1 Alternative No. 1 –Aerobic Digestion

Process Description

Aerobic digestion is a biological sludge treatment process that stabilizes the sludge by aerobic bacterial breakdown of the volatile and biodegradable organic constituents (e.g., proteins, carbohydrates, fats, and grease) of sludge in the presence of free oxygen. Aerobic digestion is based on the principle that, in the absence of an adequate external food source, microorganisms will consume their own cellular mass to obtain energy which in turn reduces the sludge volume. This biological process is commonly referred to as "endogenous respiration".

Aerobic digestion can treat primary, biological, and waste activated sludges. The digestion process normally takes place in tanks or reactors. This process requires aeration equipment, solid-liquid separation capability, and pumps with their associated valves and piping. In general, sludge solids are aerated in a tank for a period of several weeks to ensure the solids are thoroughly stabilized. Aeration is then stopped, and the solids are allowed to settle. Clarified liquid is decanted, and the thickened solids are removed from the bottom of the tank for dewatering and ultimate disposal. The most critical operating parameter is maintaining sufficient oxygen levels which allows the biological process to occur and helps control nuisance odors.

Aerobic digestion of wastewater sludge has been practiced since the 1950s and has generally been performed at smaller wastewater treatment facilities (less than 5 MGD) because of the simplicity of

the process operation. It is classified as a PSRP. Sewage sludge that is applied to land or incorporated must be treated by a PSRP prior to land application or incorporation. Thus, aerobically digested sludge is a Class B sludge that can be utilized in land application.

Facility Requirements

The aerobic digestion system as shown in **Figure 5-1** consists of two 50-foot diameter tanks each with a 20-foot sidewater depth. The Operations Building is situated adjacent to the two tanks. In order to convert the existing process into an aerobic digestion system, several changes would need to be made.

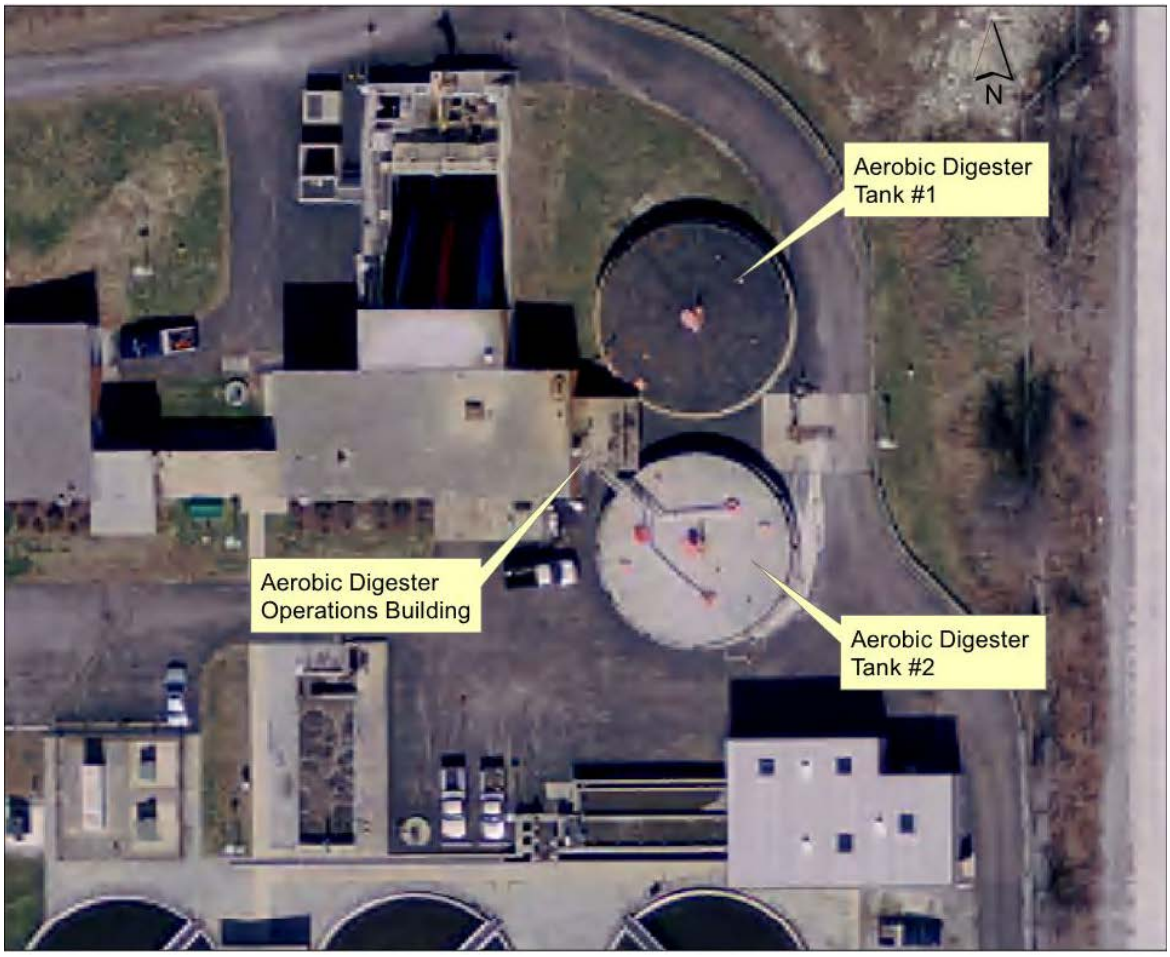
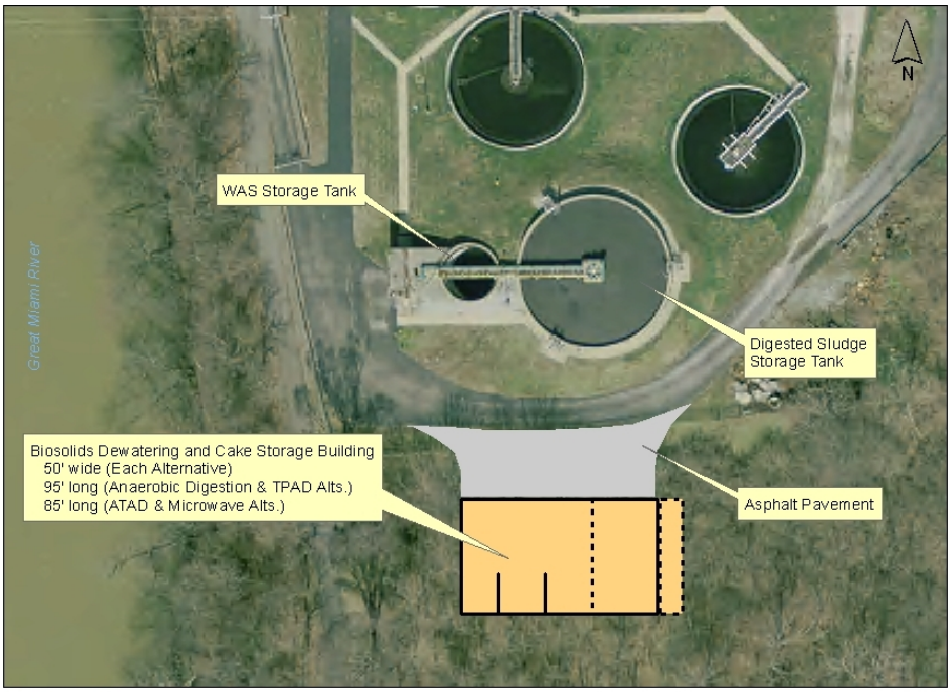


Figure 5-1: Aerobic Digestion Layout



CDM Smith PIQUA 0 25 50 100 Feet

Biosolids Processing and Storage Facilities Location Plan
Figure 5-2

Demolition work for this option would involve removing the fixed and floating covers on the primary and secondary digesters, respectively; removing the Atara bubble gun mixing system in the primary tank; and removing the gas collection and safety equipment. The two tanks would be converted to aerobic digesters with either coarse bubble diffusers or a jet-air mix system. Site work would be minimal for this option.

Assuming a thickened feed solids concentration of 5 percent, both existing digesters are required to provide a solids retention time (SRT) of 40 days at 20°C or 60 days at 15°C.

Coarse bubble aeration and jet air mix systems are two viable options for aeration of the digester tanks. For coarse bubble aeration, each tank would include a snap cap floor mounted grid air diffuser system, two feed pumps, three blowers (one per tank plus one backup), piping, and valves. For the jet air mix system, each tank would include a packaged jet aeration system that includes aerators with jet nozzles, two recirculation pumps (one per tank), two blowers (one per tank) with VFD drives, two feed pumps, piping, and valves.

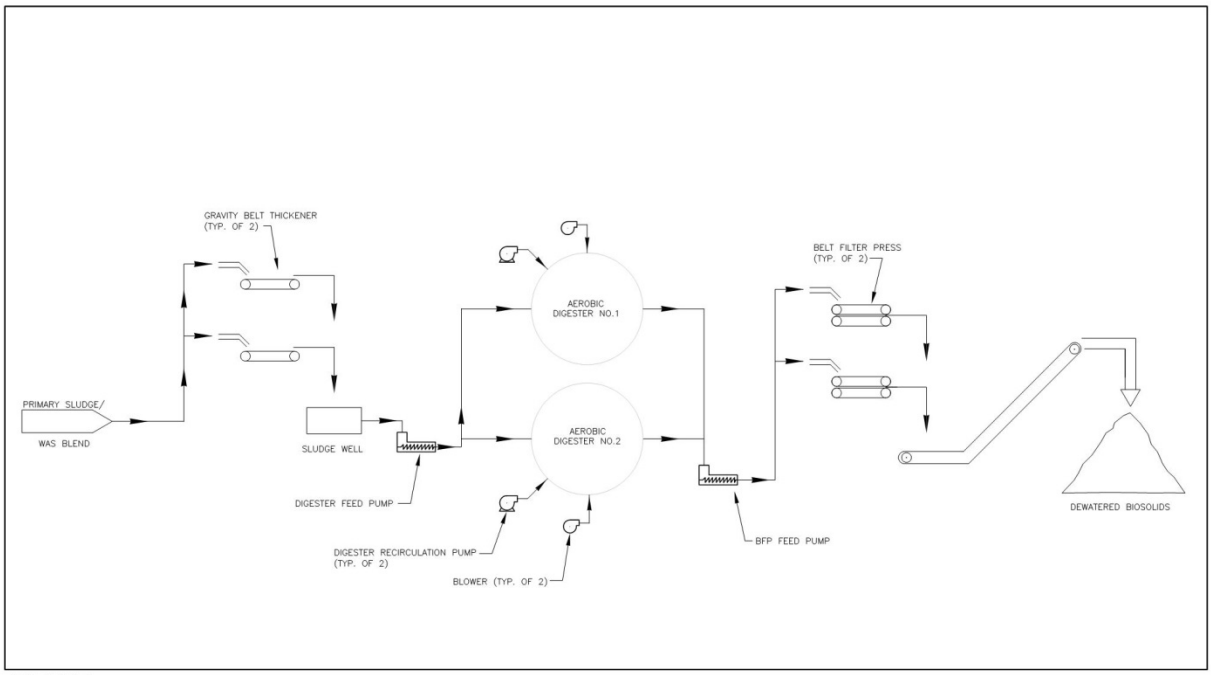
It is assumed that existing sludge feed piping and valves to the existing belt filter press (BFPs) would be reused. The Operations Building would house all of the digestion equipment. Two gravity belt thickeners (GBTs) would be needed to thicken the sludge to 5% prior to entering the digesters. These GBTs would be housed in the existing Secondary Control Building. A new Dewatering/Biosolids Storage Building would also need to be constructed. The new building would house two BFPs and associated processing equipment and controls, and a biosolids storage area capable of 120-day storage. The building would be a 4,750-square foot pre-engineered type structure with concrete floors and push walls. The size of this building varies with each alternative. The building would be constructed just south of the Digested Sludge Storage Tank. See **Figure 5-2** for a location plan. Note that the location for this building is the same for all six BMP alternatives being evaluated.

Along with the GBTs, GBT feed pumps, transfer pumps, digested sludge feed pumps (to pump digested sludge to the dewatering facilities), piping (in-tank and out-of-tank), and valves would need to be installed.

Regulatory Requirements

Aerobic digestion is a PSRP. Under the 40 CFR Part 503 standards, sewage sludge meeting the requirements of a PSRP is considered Class B with respect to pathogens. According to the regulations, aerobic digestion can be classified as a PSRP if the SRT under aerobic conditions (sewage sludge treated with air) is at least 40 days at 20^{0}C (68^{0}F) or 60 days at 15^{0}C (59^{0}F). Aerobic digestion achieves the vector attraction reduction requirements by reducing the volatile solids in the sludge by at least 38%. Class B aerobic digested biosolids can be land applied as long as the pollutant limits and vector attraction requirements are achieved. These criteria are easily met by a properly designed and operated digestion system.

See **Figure 5-3** for the process flow schematic of the Aerobic Digestion alternative.



CDM Smith

Alternative No. 1
Aerobic Digestion Schematic
Figure 5-3

5.4.2 Alternative No. 2 – High-Rate Anaerobic Digestion

Process Description

Anaerobic digestion is the most widely used method of sludge stabilization in the wastewater industry. Moreover, it is the most common stabilization process in Ohio. Anaerobic digestion has been used in virtually all sizes of wastewater treatment plants. The Piqua WWTP currently employs a primary-secondary anaerobic digestion system.

The anaerobic digestion system considered for this alternative is a high-rate digestion system. A high-rate system is characterized by each tank/reactor having auxiliary heating and mixing, and a controlled, elevated temperature to increase the rate of volatile solids destruction. High-rate digesters

are operated in mesophilic temperature ranges (approximately 95 to 110°F) and thermophilic temperature ranges (approximately 131 to 140°F). The amount of volatile solids destroyed is a function of both temperature and solids retention time (SRT). Digester sizing in this study is based on a 15-day SRT to achieve a reduction of the volatile solids content by 50%.

Facility Requirements

The existing anaerobic digestion system as shown in **Figure 5-4** consists of two 50-foot diameter tanks each with a 20-foot sidewater depth. The Operations Building is situated adjacent to the two tanks. Associated pumps, piping, heat exchangers, boilers, gas mix system, and other auxiliary equipment are located in the Operations Building. In order to convert the existing process into a high-rate anaerobic digestion system, several changes will need to be made.

Figure 5-4: Anaerobic Digestion Layout

Demolition work for this option would involve removing the fixed and floating covers on the primary and secondary digesters, respectively; removing the Atara bubble gun mixing system in the primary tank; and removing the gas collection and safety equipment. The two tanks would be converted to high-rate digesters with mixing and heating in each tank. Site work would be minimal for this option.

Assuming a feed solids concentration of 5 percent, both existing digesters are required to provide a SRT of 15 days. Both digesters would be heated and mixed to mesophilic conditions. New external

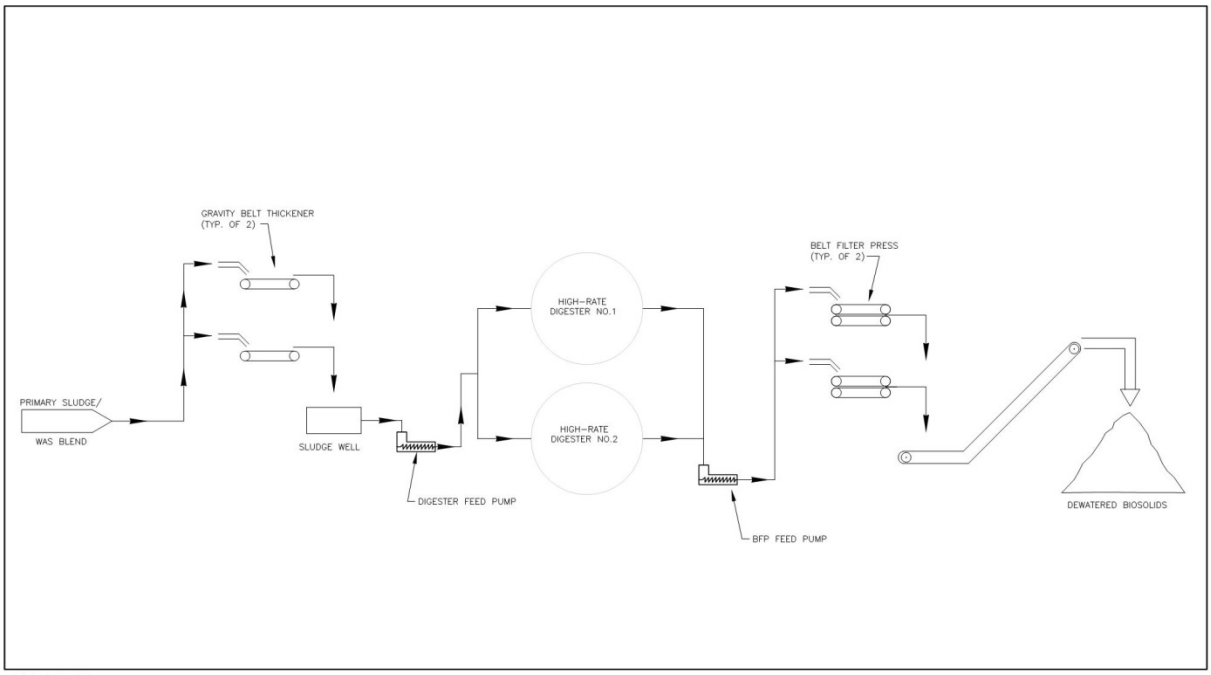
draft tube mechanical mixers (two per tank), two combination heater and heat exchangers, two recirculation pumps, two transfer pumps, piping, and valves would need to be installed, along with gas collection, handling, and safety equipment. The digesters would be equipped with membrane-type gas holders to accommodate gas storage. Digester gas would be utilized to heat the incoming sludge. Note that various mixing systems were investigated including draft tube mixers, jet mixers, and vertical liner motion mixers. It is assumed that existing sludge feed piping and valves to the existing belt filter press (BFPs) would be reused. The Operations Building would house all of the digestion equipment. Two gravity belt thickeners (GBTs) would be needed to thicken the sludge to 5% prior to entering the digesters. These GBTs would be housed in the existing Secondary Control Building. A new Dewatering/Biosolids Storage Building would also need to be constructed. The new building would house two BFPs and associated processing equipment and controls, and a biosolids storage area capable of 120-day storage. The building would be a 4,750-square foot pre-engineered type structure with concrete floors and push walls. The size of this building varies with each alternative. The building would be constructed just south of the Digested Sludge Storage Tank. See **Figure 5-2** for a location plan. Note that the location for this building is the same for all six BMP alternatives being evaluated.

Along with the GBTs, GBT feed pumps, transfer pumps, digested sludge feed pumps (to pump digested sludge to the dewatering facilities), piping (in-tank and out-of-tank), and valves would need to be installed.

Regulatory Requirements

Anaerobic digestion is a PSRP. Under the 40 CFR Part 503 standards, sewage sludge meeting the requirements of a PSRP is considered Class B with respect to pathogens. According to the regulations, anaerobic digestion can be classified as a PSRP if the SRT under anaerobic conditions (sewage sludge treated in the absence of air) is at least 15 days at 35^0C to 55^0C (95^0F to 131^0F). Anaerobic digestion achieves the vector attraction reduction requirements by reducing the volatile solids in the sludge by at least 38%. Class B anaerobic digested biosolids can be land applied as long as the pollutant limits and vector attraction requirements are achieved. These criteria are easily met by a properly designed and operated digestion system.

See **Figure 5-5** for the process flow schematic of the High-Rate Anaerobic Digestion alternative.



CDM Smith

Alternative No. 2
High-Rate Anaerobic Digestion Schematic
Figure 5-5

5.4.3 Alternative No. 3 – Temperature-Phased Anaerobic Digestion (TPAD)

Process Description

TPAD is a two-phase digestion process with the first phase operating in the thermophilic temperature range (131 to 140 °F) and second phase in mesophilic temperature range (95 to 110 °F). Sludge can meet one of the Exceptional Quality biosolids pathogen reduction criteria when it is heated to 131 °F and held at that temperature for at least one day. After having met the Exceptional Quality biosolids pathogen reduction criteria, sludge can then be digested in the mesophilic phase to further destruct volatile solids to meet the VAR criteria. The existing Primary Digester would be converted into the thermophilic tank, and the existing Secondary Digester would be converted into the mesophilic tank.

The TPAD process is designed to take advantage of the thermophilic digestion rates, which are estimated to be four times faster than mesophilic digestion. The thermophilic digester would provide about 2 to 5 days of SRT. This tank would also need to operate in a fill-hold-draw batch mode to make sure that the digester content is held at the specified temperature for at least 24 hours to meet EPA's Exceptional Quality biosolids pathogen reduction criteria. Testing would be conducted to verify that all sludge particles in the thermophilic phase have maintained a temperature of 131 °F or higher for at least 24 hours. The mesophilic digester would provide 10 days of SRT.

Digester sizing in this study is based on the SRTs stated and a reduction in the volatile solids content by 55%. A major challenge in modifying a conventional mesophilic process to a thermophilic, sequential-batch system is meeting the heating loads that are about twice that of mesophilic digestion at the same feed rate. Heat exchanger selection is a critical step in this design.

Facility Requirements

Since the plant has two existing anaerobic digesters, each tank can achieve the recommended SRT; no new tanks are needed. **Figure 5-6** presents a layout of this alternative.

Figure 5-6: Temperature-Phased Anaerobic Digestion (TPAD) Layout

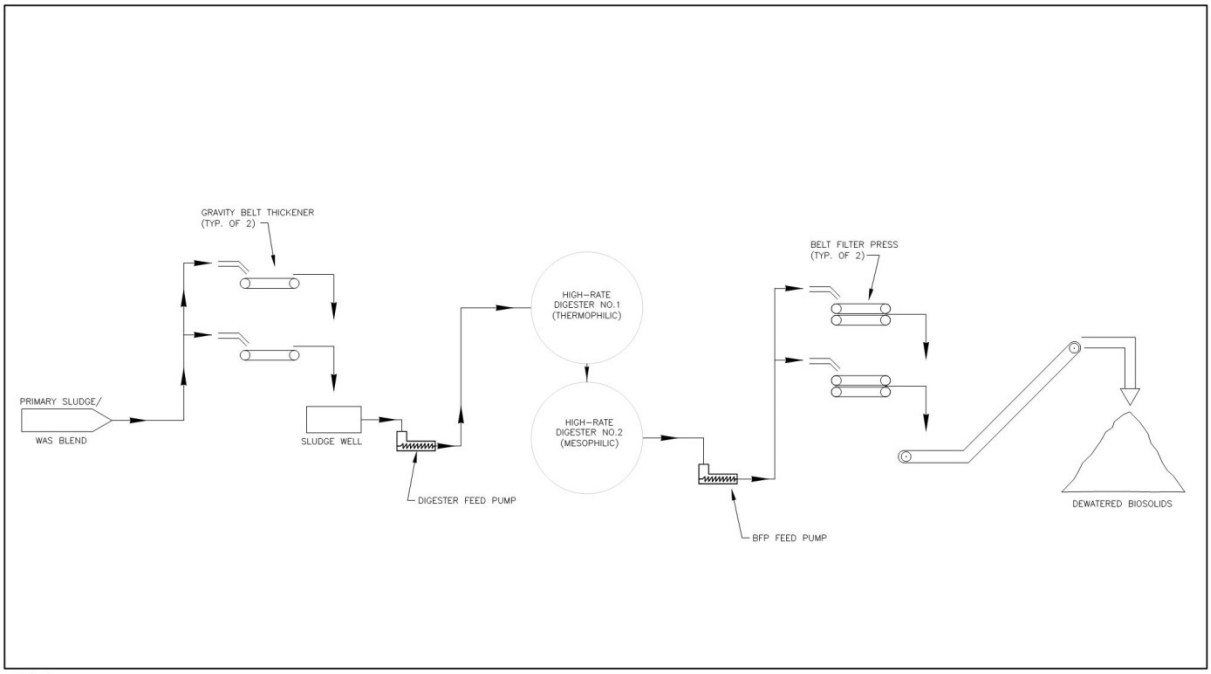
To accommodate a TPAD system several existing systems would need to be removed or replaced. Similar to the high-rate anaerobic digestion system option, demolition work for this option would also involve removing both digester covers, the bubble gun mixing system, the recirculation pumps, and the heat exchanger system. Major new equipment would include gas-holder covers for both digesters, a mixing system for each digester, two heater/heat exchangers (one each for mesophilic conditions and thermophilic conditions), recirculation pumps, and transfer pumps. Two gravity belt thickeners (GBTs) would be needed to thicken the sludge to 5% prior to entering the thermophilic digester. These GBTs would be housed in the existing Secondary Control Building. A new Dewatering/Biosolids Storage Building would also need to be constructed. The new building would house two BFPs and associated processing equipment and controls, and a biosolids storage area capable of 120-day storage. The building would be a 4,750-square foot pre-engineered type structure with concrete floors

and push walls. The building would be constructed just south of the Digested Sludge Storage Tank. See **Figure 5-2** for a location plan.

Along with the GBTs, GBT feed pumps, transfer pumps, digested sludge feed pumps (to pump digested sludge to the dewatering facilities), piping (in-tank and out-of-tank), and valves would need to be installed.

Regulatory Requirements

TPAD can be a PSRP or PFRP. Under the 40 CFR Part 503 standards, sewage sludge meeting the requirements of a PSRP is considered Class B with respect to pathogens. According to the regulations, TPAD can be classified as a PSRP if the SRT under anaerobic conditions (sewage sludge treated in the absence of air) is at least 15 days at 35^{0}C to 55^{0}C (95^{0}F to 131^{0}F). TPAD achieves the vector attraction reduction requirements by reducing the volatile solids in the sludge by at least 38%. Class B anaerobic digested biosolids can be land applied as long as the pollutant limits and vector attraction requirements are achieved. These criteria are easily met by a properly designed and operated digestion system. TPAD can achieve PFRP status (Exceptional Quality biosolids) under the time-temperature regime of the Part 503 standards with respect to pathogens. That is, sewage sludge must be operated at thermophilic temperatures in a sequential batch mode such that every particle is subjected to time and temperature conditions. The time and temperature requirement is 24 hours at 55^{0}C, with additional time needed at lower temperatures and less time at higher temperatures. Pathogen destruction (pasteurization) must precede or be accomplished concurrently with vector attraction reduction. TPAD achieves the vector attraction reduction requirements by reducing the volatile solids in the sludge by at least 38%. These criteria can be met by a properly designed and operated TPAD system. Exceptional Quality biosolids can be utilized via land application and distribution and marketing. See **Figure 5-7** for the process flow schematic of the TPAD alternative.



CDM Smith

Alternative No. 3
Temperature-Phased Anaerobic Digestion (TPAD) Schematic
Figure 5-7

5.4.4 Alternative No. 4 – Autothermal Thermophilic Aerobic Digestion (ATAD)

Process Description

The Autothermal Thermophilic Aerobic Digestion (ATAD) process is an aerobic digestion technology that operates at thermophilic temperatures by utilizing the heat produced by the process. Autothermal conditions result from an adequately thickened sludge feed, a suitably insulated reactor, good mixing, and an efficient aeration device that keeps the latent heat loss to an acceptable level. Heat generated by the sludge decomposition is sufficient to warm the incoming sludge without an external heat source.

ATAD is a refinement of the conventional aerobic digestion process that achieves thermophilic operating temperatures without supplemental heat (autothermal) beyond that supplied by mixing energy. In this process the feed sewage sludge is pre-thickened and an efficient aerator is used. Because of the severe odor problems associated with the off-gases expelled from the 1st generation ATAD systems, a 2nd generation ATAD process by Thermal Process Systems (patented ThermAer™ ATAD system) was considered. Compared to the 1st generation ATAD units, the 2nd generation ATAD units provide less complex reactor schemes, higher SRT levels, and improved high-efficiency aeration and mixing systems. Moreover, the biofiltration system included with the process has proven to be very efficient in treating odors. ATAD reactor sizing in this study is based on a 10 to 12-day SRT in the reactors to achieve a reduction of the volatile solids content by 60% and an SRT of a minimum of 5 days in the storage tank.

Facility Requirements

As with the anaerobic digestion options, demolition work for this option would involve removing the bubble gun mixing system, primary fixed cover, secondary gas holder cover, associated pumps, heat exchanger system, boiler, and gas equipment in the tanks and Operations Building. The primary tank would be converted to an ATAD Thermaer reactor, and the existing secondary tank would become the Storage/Nitrification/Denitrification (SNDR) tank. The Thermaer reactor tank would have a concrete cover and be insulated. The SNDR tank would have an aluminum dome cover. Site work for this option would include a biofiltration odor control unit approximately 50 ft x 25 ft x 10 ft. From previous ATAD facilities in Ohio, the installation of fine screens with 1/4-inch to 3/8-inch screen opening sizes at the headworks would be necessary.

The existing anaerobic digesters would be retrofitted into a 2nd generation ATAD facility. Assuming a feed solids concentration of 5 percent, one reactor (Thermaer™ reactor) and one storage tank (SNDR tank) would be needed to provide the required SRTs. One reactor tank would be required to provide a detention time of 10-12 days for the design solids loading. One jet-motive pump aeration system (ThermAer™ liquid and air jet header and nozzle systems) and one hydraulic foam control system would be installed in the reactor. The jet manifold would be comprised of integrally fabricated air and liquid headers equipped with jet nozzles. The jet manifold would have a dedicated 75-horsepower, variable speed, dry pit, end suction, centrifugal pump for the liquid recirculation component and two 40-horsepower, variable speed, positive displacement blowers to provide the airflow component. One blower would serve as a spare for the reactor and SNDR. The jet aeration system would be equipped with a pneumatic flushout system and foam control jet motive pump.

Downstream of the reactor, sludge storage for a period of 5 days would be required to allow the biosolids to cool to a mesophilic temperature. This cooling step is critical for efficient dewatering of

the biosolids downstream. A heat exchanger would also aid in the cooling process. The Secondary Digester would be converted into a storage tank equipped with a jet-motive pump aeration system. An aluminum cover would replace the existing floating gas-holder cover.

A two-stage odor control system consisting of a humidification system and biofilter would be used to control odorous emissions from the reactors and storage tank. A biofilter is an odor control technology that uses a biologically active media bed to adsorb and absorb contaminants from the air stream passing upward through the bed and retain them for subsequent microbial degradation and oxidation. The microorganisms that reside in the media feed on the odorous compounds releasing non-odorous air to the atmosphere. A biofilter fan would draw air from the tanks via collection piping and discharge the odorous air up through the biofilter media bed.

The first stage of the odor control system would be the humidification/scrubber unit. This humidification/scrubber unit removes a large amount of ammonia from the influent foul air stream, controls the temperature of the air assuring that it is conducive to biological activity, and raises the humidity of the foul air for further treatment by the biofilter downstream, the second stage of the odor control system. The biofilter would be a 50-foot by 25-foot aboveground unit consisting of a concrete tank, a plastic aeration plenum, and biofilter media (placed within the tank walls approximately 10 feet high) supported by the plenum below. Alternately, a packaged system could be installed, potentially with a smaller footprint.

In addition, this alternative would include two GBTs housed in the existing Secondary Control Building to increase the solids feed to 5% enroute to the reactor. Higher solids feed results in greater reduction of the volatile solids. Along with the GBTs, GBT feed pumps, transfer pumps, digested sludge feed pumps (to pump digested sludge to the dewatering facilities), piping (in-tank and out-of-tank), and valves would need to be installed.

The Operations Building would house all of the ATAD equipment, pumps, piping, valves, and instrumentation and controls. A new Dewatering/Biosolids Storage Building would also need to be constructed. The new building would house two BFPs and associated processing equipment and controls, and a biosolids storage area capable of 120-day storage. The building would be a 4,250-square foot pre-engineered type structure with concrete floors and push walls. The building would be constructed just south of the Digested Sludge Storage Tank. See **Figure 5-2** for a location plan.

A layout of this alternative is presented in **Figure 5-8**.

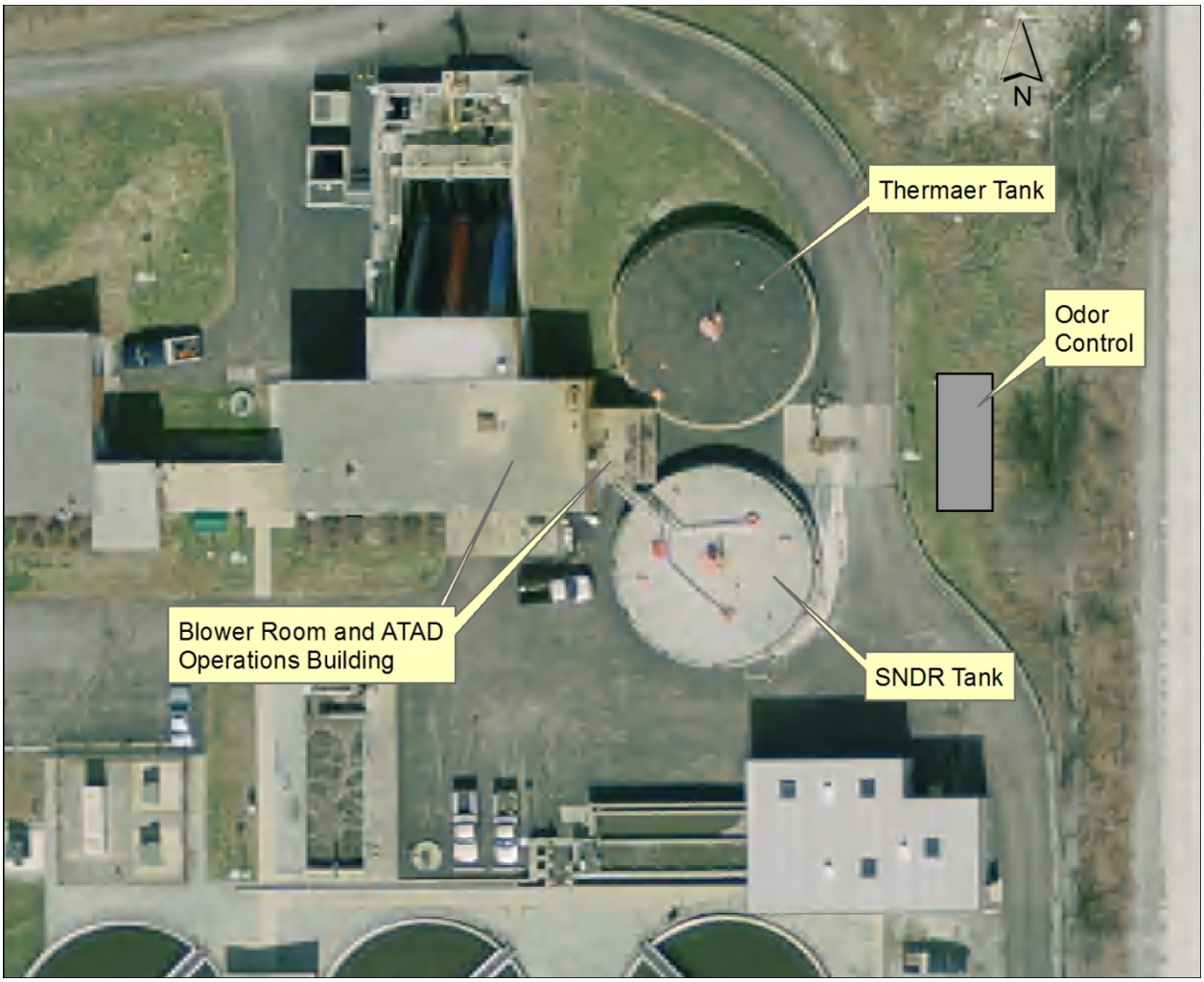
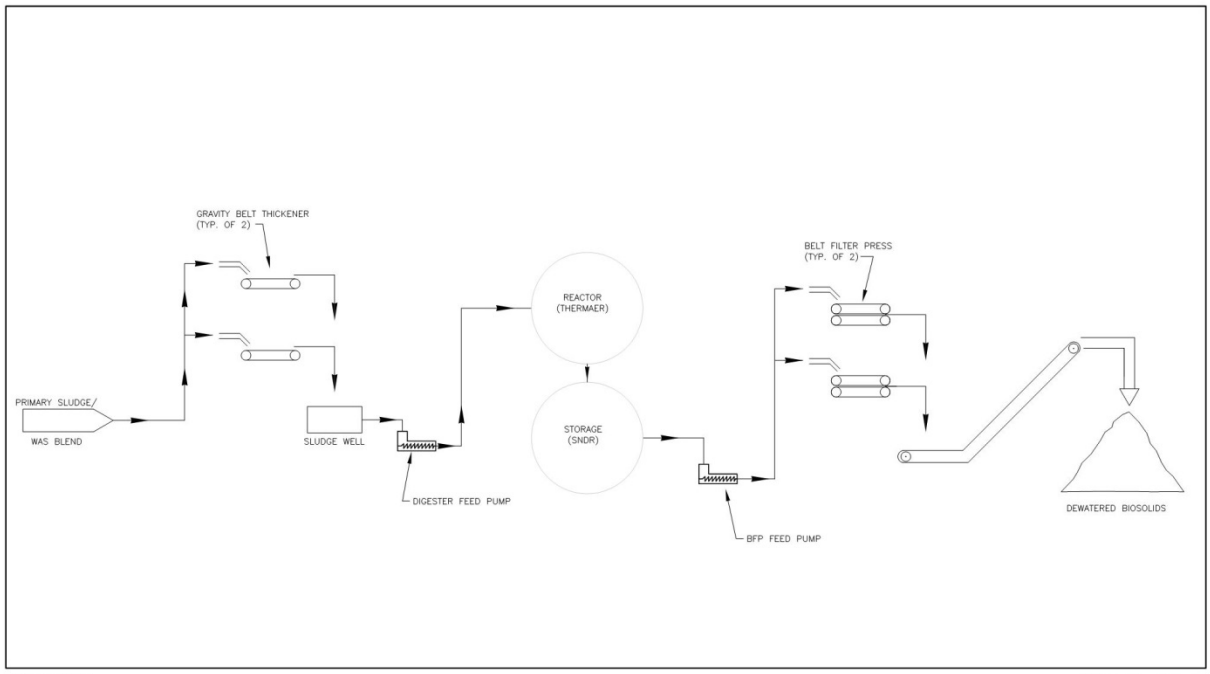


Figure 5-8: Autothermal Thermophilic Aerobic Digestion (ATAD) Layout

Regulatory Requirements

ATAD is a PFRP. Under the Part 503 standards, sewage sludge meeting the requirements of a PFRP is considered Exceptional Quality with respect to pathogens. According to the regulations, ATAD can be classified as a PFRP if the SRT under aerobic conditions (sewage sludge agitated with air or oxygen) is 10 consecutive days at 55°C to 60°C (131°F to 140°F). ATAD achieves the vector attraction reduction requirements by reducing the volatile solids in the sludge by at least 38%. These criteria are easily met by a properly designed and operated ATAD system. Exceptional Quality biosolids can be utilized via land application and distribution and marketing.

See **Figure 5-9** for the process flow schematic of the ATAD alternative.



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Alternative No. 4
Autothermal Thermophilic Aerobic Digestion (ATAD) Schematic
Figure 5-9

5.4.5 Alternative No. 5 – Burch-Hydro BioWave™ Process

Process Description

The objective of the microwave system is to remove water from the biosolids, producing biosolids with relatively high percent solids, which in turn reduces the weight and volume of the biosolids. The drying process is flexible and can produce marketable products that meet Class B or Exceptional Quality standards. The reduction in volume and weight also reduces transportation costs; however, in Piqua's case there would not be a significant cost reduction since there is eligible farm land in close proximity to the plant.

The ideal percent solids produced via the microwave process is about 60 to 70%. At existing facilities this is accomplished by drying to 50% using the microwave dryer and then the biosolids will lose another 10% while in the storage pile before disposal. The 60% solids is desired because it kills all pathogens (they do not regenerate) and reduces ammonia odors (which helps with public acceptance). Since the drying only removes water, the product retains the beneficial nutrients, and it is very close to what farmers are used to handling with their spreaders, so they have a high acceptance of the product. The BioWave™ process uses electromagnetic waves or microwaves to thermally heat the biosolids and then with supplemental gas and fans, drive off the moisture as safe steam to an odor control system. The unit has four major components: the control center, the transmitters, the waveguides, and the applicator oven. Two of the components, the transmitters and the control center, should be in a climate controlled room. The waveguides and applicator oven should be in a building, but climate control is not required. However, for the comfort of the operators some temperature control is advised.

The control center is a programmable logic controller that can monitor and control various functions such as belt speed, burner temperature, magnetron power, and air flow. The panel includes a touch screen which can also monitor and control belt filter press controls.

There would be four transmitters, each housing one magnetron. Each magnetron converts 0-100 KW of electrical energy into microwaves. The microwaves are then transmitted through special ducts called waveguides to the applicator oven, which is an open stainless steel shell approximately 2-meters wide and 50-feet long with a belt running through it. The belt can be run continuously because each end of the applicator oven is equipped with choke pins which trap the microwaves from escaping to the outside. The belt speed and the microwave power are both adjustable so that the percent solids of the product can be increased or decreased by either slowing or speeding up the belt, or increasing or decreasing the power to the magnetrons.

The system has been tested for municipal sludge treatment and no air pollution permits are required.

The microwave dryer's major advantage is that it will reduce the volume and weight of Piqua's biosolids by 60 to 70%.

Facility Requirements

The existing Secondary Control Building houses a one-meter belt filter press along with a polymer feed system that includes a polymer feed pump and a Seepex progressive cavity sludge feed pump. The solids handling is contracted to Burch Hydro, a contractor that dewaters, hauls, and land applies the digested cake. They currently run the belt filter press on average 2-3 days per week and 8 hours per day. They process 17 dry tons per month .A major advantage of the microwave system is that it can be set to match the output of the belt filter press, and one person can operate both the press and the microwave. However, to do this the two systems need to be next to each other. The existing Secondary Control Building cannot accommodate adding the microwave dryer into the building. Additional electrical power would need to be run to the new microwave building to supply the large demand of the microwave. In addition, upgrades to the existing digestion system would still have to be carried out with this alternative. A combination of high-rate digestion upgrades and a microwave drying process makes up Alternative No. 5.

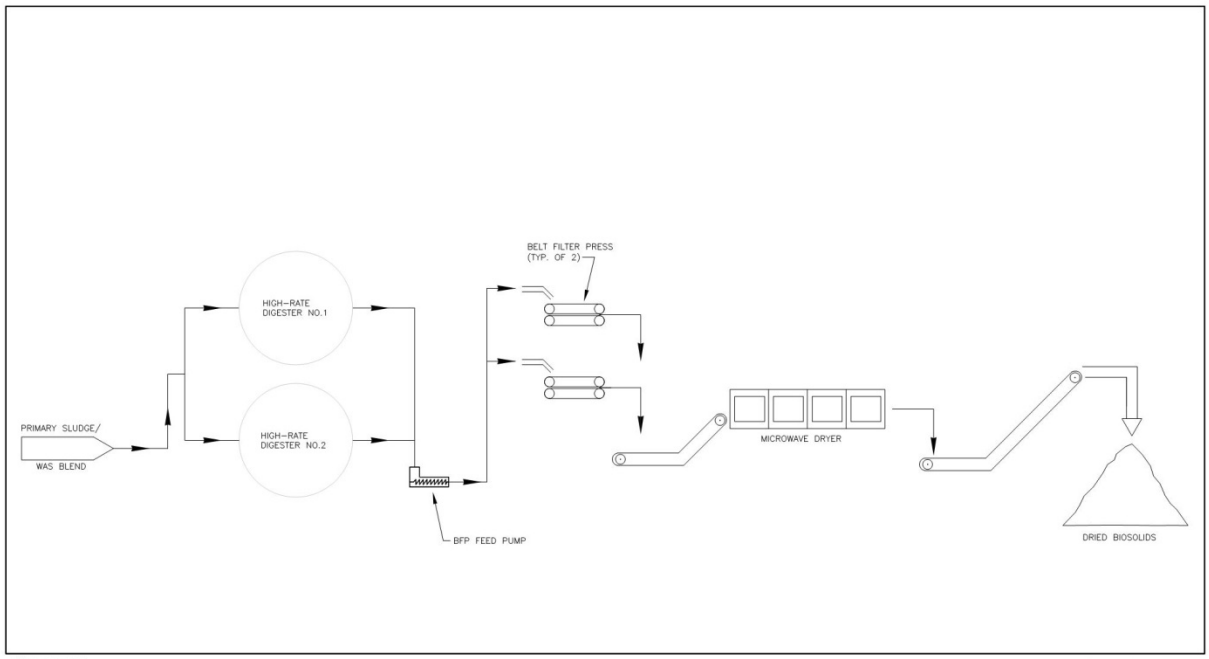
Unlike the other options, sludge thickening facilities are not required, since the actual stabilization step is the microwave. However, digested sludge feed pumps (to pump digested sludge to the dewatering facilities), piping (in-tank and out-of-tank), and valves would still need to be installed.

A new Dewatering/Microwave Drying/Biosolids Storage Building would also need to be constructed. The new building would house two BFPs and associated processing equipment and controls, a 400-kW microwave system, and a biosolids storage area capable of 120-day storage. The building would be a 4,250-square foot pre-engineered type structure with concrete floors and push walls. The building would be constructed just south of the Digested Sludge Storage Tank. See **Figure 5-2** for a location plan.

Regulatory Requirements

The BioWave™ Process is an approved US EPA Exceptional Quality process and listed by the Agency as an emerging technology. Of the six pathogen reduction alternatives, the BioWave™ process qualifies as an Exceptional Quality process through Alternative 1, Regime B of the 503 regulations – Thermally Treated Biosolids, which means it dries the biosolids to 8% or more and raises the temperature of the biosolids to 50°C for more than 15 seconds. It can meet either Option 7 or Option 8 of the vector attraction reduction alternatives, depending on the level of treatment the biosolids have undergone before entering the system. Option 7 requires drying to 75% when the biosolids are digested and Option 8 requires drying to 90% solids for undigested sludge. Since the treatment plant will continue to provide sludge digestion, the microwave system would be sized to produce EQ biosolids with a solids content of 70%.

See **Figure 5-10** for the process flow schematic of the Microwave Drying alternative.



CDM Smith

Alternative No. 5
Microwave Drying Schematic
Figure 5-10

5.4.6 Alternative No. 6 – Indirect Thermal Drying Systems

Process Description

Thermal drying is a process to further reduce pathogens. Heat-dried material meets the requirements of the Part 503 Standards for pathogen and vector attraction reduction generating a Class A product. There are two types of thermal dryers: direct and indirect. For this study, CDM Smith evaluated indirect dryers.

With indirect thermal dryers, solid metal walls separate the wet sludge from the heat transfer medium (steam, hot water, or oil). Thermal energy is transferred from hot transfer medium into the metal wall and then from the metal wall into the cold sludge. The sludge temperature is elevated by contact with

hot metal surfaces, and the sludge never comes in direct contact with the primary heating medium. The predominant method of heat transfer is conduction. The analysis of the indirect drying system was based on the implementation of a Komline-Sanderson (KS) Paddle Drying System.

The KS Paddle Drying System includes an indirect, twin-auger dryer. The dryer consists of hollow, rotating twin augers that are filled with either steam or hot oil to dry the sludge. The augers counter-rotate with a very small tolerance between paddles, creating maximum surface area contact with the sludge. The system can also be run in either a continuous mode or a batch operation. The sludge enters one end of the enclosed unit and is transferred to the discharge end while it is dried to a hard, Class A pellet with dry solids content up to 99%. The sludge is not exposed to the heating medium within the enclosed unit, and no air is allowed inside the dryer.

Facility Requirements

The existing Secondary Control Building houses a one-meter belt filter press along with a polymer feed system with a polymer feed pump and a Seepex progressive cavity sludge feed pump. The solids handling is contracted to Burch Hydro. Burch Hydro dewaters, hauls, and land applies the digested cake. They currently run the belt filter press on average 2-3 days per week and 8 hours per day. They process 17 dry tons per month.

Two centrifuges would replace the belt filter press and precede the thermal dryer. The existing Secondary Control Building cannot accommodate adding the centrifuges and the thermal dryer system into the building. Additional electrical power as well as a new gas line would need to be run to the new dryer building to supply the large energy demand of the dryer. In addition, upgrades to the existing digestion system would still have to be carried out with this alternative. A combination of aerobic digestion upgrades and an indirect drying process makes up Alternative No. 6.

Sludge thickening facilities would be required and digested sludge feed pumps (to pump digested sludge to the dewatering facilities), piping (in-tank and out-of-tank), and valves would still need to be installed.

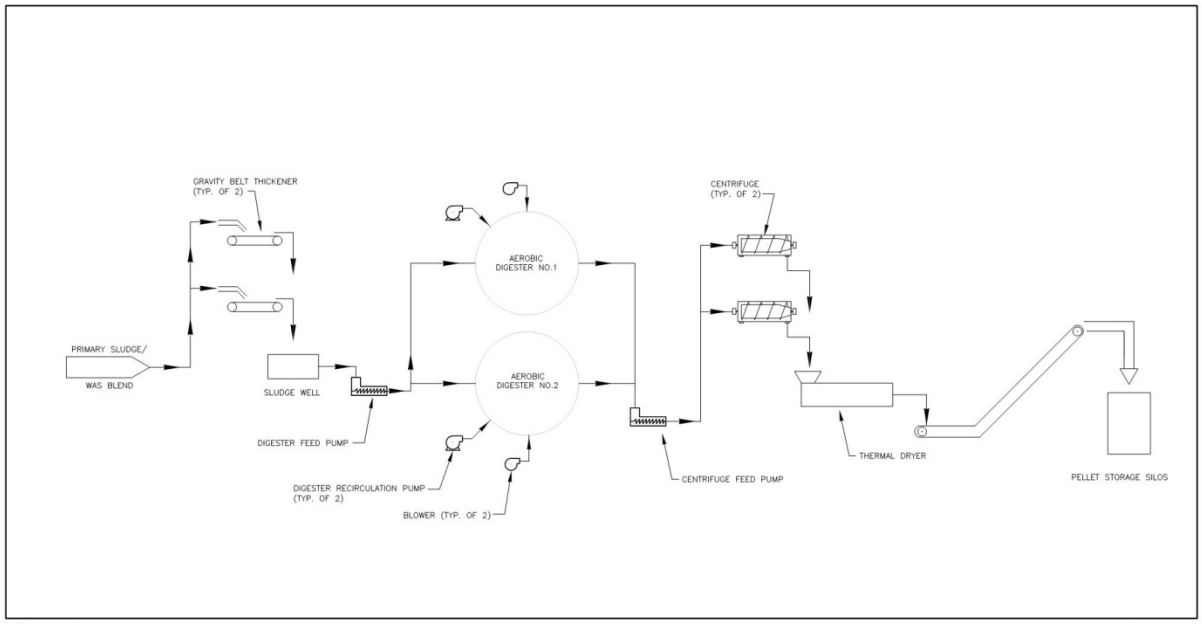
A new Dewatering/Indirect Thermal Drying Building would also need to be constructed. The new building would house two centrifuges and associated processing equipment and controls, and the drying system.

A 7,200-square foot by 30-foot high, pre-engineered metal Process Building would be erected to house the two centrifuges and the indirect drying equipment – feed hopper, feed pumps, Paddle Dryer dryer unit, gas burners, hot oil system, condenser system, discharge and product cooling conveyors, compressor, and instrumentation and controls. The building would be constructed just south of the Digested Sludge Storage Tank. Four silos to store the dried biosolids/pellets (total 120-day storage) would be constructed outside. See **Figure 5-2** for a location plan.

Regulatory Requirements

According to the Part 503 Standards, thermal drying can be classified as a PFRP if the sewage sludge is dried by direct contact with hot gases to reduce the moisture content to 10% or lower and either the temperature of the sewage sludge particles exceeds 80 ^{0}C (176 ^{0}F) or the wet bulb temperature of the gas in contact with the sewage sludge as it leaves the dryer exceeds 80 ^{0}C (176 ^{0}F).

See **Figure 5-11** for the process flow schematic of the Indirect Thermal Drying alternative.



CDM Smith

Alternative No. 6
Indirect Thermal Drying with Aerobic Digestion Schematic
Figure 5-11

5.5 Sludge Thickening

Sludge thickening is required to reduce volumetric loading on the digestion process, produce a relatively solids-free supernatant, and increase the efficiency of subsequent solids-processing steps. The feed sludge (primary, WAS, or primary/WAS blend) will need to be thickened from 3% to 5% prior to entering the digesters. Doing so will increase volatile solids destruction which improves operation and reduces the costs for storage.

Three common sludge thickening methods are gravity thickening, gravity belt thickening, or centrifugal thickening. For this study, CDM Smith assumed the use of gravity belt thickeners.

A gravity belt thickener (GBT) is a belt filter press with a modified upper gravity drainage zone that allows water to drain through the moving, fabric-mesh belt while coagulating and flocculating solids. GBTs typically capture 95% solids and can thicken up to 6% solids. Because of the efficient space requirement, lower power use, and moderate capital costs, GBTs are a popular technology. It is assumed two 1-meter gravity belt thickeners would be installed to thicken the feed entering the digesters.

5.6 Biosolids Dewatering

A dewatering system will be needed after digestion to further remove water from solids to reduce the volume and produce a biosolids cake material suitable for land application. The dewatering process will produce a liquid stream, which can be recycled to the supernatant oxidation process. Typical dewatering systems include belt filter presses, centrifuges, and rotary presses. For this study, CDM Smith assumed the use of belt filter presses for all alternatives except the thermal drying system , which would use centrifuges.

Belt Filter Press

Gravity drainage and compression aids the filter belt in separating water from solids. With low energy consumption per volume of solids dewatered, the BFP can produce a cake containing 15 to 20% solids when dewatering anaerobically digested material (14 to 18% solids when dewatering aerobically digested material) and has solids capture rate range of 85 to 95%. It is assumed two 1-meter belt filter presses would be installed to dewater the digested sludge prior to land application or further microwave drying.

Centrifuges

Centrifuge dewatering is a process that uses the force developed by the rotational movement of a bowl to separate the sludge solids from the liquids. Sludge is pumped through a central pipe into a bowl rotating at speeds of approximately 3,000 revolutions per minute (rpm). Centrifuges rotate at a high speed to apply a centrifugal force to the sludge slurry, forcing the heavier sludge to separate from the water fraction and collect along the bowl wall. The liquid overflows centrate discharge weirs located at one end of the unit. A screw conveyor inside the centrifuge moves the sludge cake from the bowl up the conical section (the "beach") where the cake is discharged at the other end of the unit. The centrifuge can produce a cake containing 22 to 26% solids when dewatering anaerobically digested material (20 to 22% solids when dewatering aerobically digested material) and has solids capture rate range of 95% or better.

5.7 Summary

Table 5-2 provides a summary of each biosolids processing alternative evaluated.

Table 5-2: Summary of Biosolids Processing Alternatives

Biosolids Processing Alternative	Exceptional Quality Biosolids	Class B Biosolids
Aerobic Digestion	No	Yes
High-Rate Anaerobic Digestion	No	Yes
Temperature-Phased Anaerobic Digestion	Yes	Yes
Autothermal Thermophilic Aerobic Digestion (ATAD)	Yes	Yes
Burch-Hydro BioWave™	Yes	Yes
Indirect Thermal Drying System	Yes	Yes

At workshop #1A, aerobic digestion, high-rate anaerobic digestion, ATAD, and aerobic digestion with indirect thermal drying were shortlisted to be further evaluated based on cost and non-cost parameters. Construction, annual operation and maintenance (O&M) and present worth costs, and other non-cost parameters are established and analyzed in Section 6 of this report.

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Section 6

Evaluation of Solids Stream Alternatives

6.1 Basis of Evaluation

The cost analysis of the alternatives includes the development of total present worth costs based on construction and annual operation and maintenance (O&M) costs. The cost figures developed not only facilitate the direct comparison between alternatives but also indicate the magnitude of the cost for implementing each biosolids management plan (BMP).

The cost estimates are based on the preliminary design of each alternative to determine the equipment, land area, process building, storage, odor control, utility, maintenance, and staffing requirements. Construction and annual O&M costs of similar facilities constructed were considered in the cost analysis as well as information provided by manufacturers of the various processes.

The construction and O&M costs are compared using a 20-year life and an interest rate of 3.5 percent. The present worth cost includes both construction and O&M costs for the next 20 years. The analysis assumes that the facilities are constructed at one time and the constant O&M costs start at the same time and continue over the 20-year period. This procedure converts these costs over the project life into an equivalent cost that represents the current investment that would be required to satisfy all of the identified project costs for the planning period.

The cost analysis of the alternatives is based on the following specific parameters:

- Project design period (useful life of the facilities) = 20 years
- Interest rate = 3.5 percent
- Present worth factor = 14.23 (O&M cost x 14.23)
- Labor cost = $35.00 per hour (includes fringe benefits)
- Electricity cost = $0.06 per Kw-hr
- Gas cost = $7.35 mmBTU per hour
- Maintenance cost = 2.0% x equipment cost

Engineering design, construction management, and legal fees are not included in the costs, but presumed to be similar between the alternatives. The O&M cost estimates are based on the average daily solids production of 3.2 dry tons per day anticipated during the 20-year design period. Administration and laboratory costs are not included in the annual O&M cost estimates.

A contingency of 30% is added to the construction cost of each alternative. A contingency is appropriate at a planning level to allow for unforeseen and undefined cost items. It is important to note that the cost estimates are preliminary planning-level costs based on information available at the time of the estimates and are considered to be "order of magnitude". The actual cost of the recommended alternative will depend on actual labor and material costs, competitive market conditions, final project

scope, implementation schedule, and other factors. As a result, the final costs will most likely vary from the estimates presented herein.

6.2 Construction Costs

See **Table 6-1** for a cost breakdown of each alternative. Differences in construction costs for each of the alternatives are described below.

6.2.1 Alternative No. 1 – Aerobic Digestion

The construction cost for the Aerobic Digestion Jet Air Mix System option is estimated at $3,500,000 and the Coarse Bubble Diffuser System option is estimated at $3,300,000. Major equipment included in this cost consists of two gravity belt thickeners (GBT), two belt filter presses (BFP), two coarse bubble aeration systems including blowers or two jet air mix systems including recirculation pumps and blowers, and biosolids storage. The Aerobic Digestion alternative has both the lowest construction cost and present worth cost of all alternatives.

6.2.2 Alternative No. 2 – High-Rate Anaerobic Digestion

The construction cost for the High-Rate Anaerobic Digestion option is estimated at $6,800,000. Major equipment included in this cost consists of two gravity belt thickeners (GBT), two belt filter presses (BFP), two gas-holding membrane covers, two mixers, two heat exchangers (to maintain mesophilic temperatures), and biosolids storage.

6.2.3 Alternative No. 3 – Autothermal Thermophilic Aerobic Digestion (ATAD)

The construction cost for the ATAD option is estimated at $7,100,000. This cost is slightly higher than the high-rate anaerobic digestion option since the ATAD system will require more equipment and site work. The ATAD system includes the following equipment: two GBTs, two BFPs, a concrete cover for the reactor, an aluminum cover for the SNDR tank (storage), jet mixing systems, transfer pumps, foam control systems, biofilter odor control unit, and biosolids storage.

6.2.4 Alternative No. 4 – Indirect Thermal Drying

The construction cost for the thermal drying option is estimated at $5,900,000. This is the second lowest construction cost of the four options evaluated mainly due to the fact that improvements to the digesters (switching to aerobic digestion) must still be accomplished. Major equipment included in this cost consists of two gravity belt thickeners (GBT), two centrifuges, two coarse bubble aeration systems or two jet air mix systems, an indirect thermal dryer, and biosolids storage. The new dewatering/storage building not only would house the centrifuges and biosolids storage but would also include an area for the thermal dryer system.

6.2.5 Maintenance of Plant Operations (MOPO)

Maintenance of plant operations (MOPO) was considered in the evaluation of the four biosolids management alternatives. Since all four options would require upgrades to the existing anaerobic digesters, the digestion operation at the Piqua plant would have to be taken off-line in order to construct each alternative. In order to minimize negative impacts to the existing plant operations during construction, a contract for temporary dewatering, hauling, and landfilling must be retained. Primary and waste activated sludge would need to be dewatered via the existing belt filter press or a mobile unit. The sludge cake would then need to be hauled and landfilled at a facility that accepts raw sludge. The cost for this temporary operation is included in **Table 6-1** for each alternative being evaluated. It was assumed that the temporary operation would need to be on-line for a 12-month period.

6.3 Operation and Maintenance Costs

See **Table 6-1** for a cost breakdown of each alternative. Assumptions and operating conditions for each of the alternatives are described below.

6.3.1 Alternative No. 1 – Aerobic Digestion

The total annual O&M cost for the jet air mix system is estimated at $126,000. The total present worth cost equates to $5,300,000.

The total annual O&M cost for the coarse bubble diffuser system is estimated at $93,000. The total present worth cost equates to $4,600,000.

It is expected that the digestion system would be operated 24 hours a day, seven days a week, 52 weeks per year with no additional operator or maintenance person at the plant. Annual operation and maintenance costs include the cost for electricity and maintenance (e.g., lubricants and replacement parts). The O&M costs include costs for hauling and applying the digested biosolids at a land application site.

6.3.2 Alternative No. 2 – High-Rate Anaerobic Digestion

The total annual O&M cost is estimated at $116,000. The total present worth cost equates to $8,500,000.

It is expected that the digestion system would be operated 24 hours a day, seven days a week, 52 weeks per year with no additional operator or maintenance person at the plant. Annual operation and maintenance costs include the cost for electricity and maintenance (e.g., lubricants and replacement parts). It is assumed that the digester gas will fuel the heaters. The O&M costs include costs for hauling and applying the digested biosolids at a land application site.

6.3.3 Alternative No. 3 – Autothermal Thermophilic Aerobic Digestion (ATAD)

The total annual O&M cost is estimated at $265,000. The total present worth cost equates to $10,900,000.

It is expected that the ATAD system would be operated 24 hours a day, seven days a week, 52 weeks per year with no additional operator or maintenance person at the plant. Annual operation and maintenance costs include the cost for electricity and maintenance (e.g., lubricants and replacement parts). The O&M costs include costs for hauling and applying the digested biosolids at a land application site. The ATAD option has a higher energy cost than all the other digestion options.

6.3.4 Alternative No. 4 – Indirect Thermal Drying Process

The total annual O&M cost is estimated at $243,000. The total present worth cost equates to $9,300,000.

For the purpose of estimating the O&M cost for indirect thermal drying the O&M cost for one of the above digestion alternatives must be included. In this case aerobic digestion (Alternative No. 1) is paired with drying to estimate the total O&M cost. It is expected that the thermal drying would be operated 24 hours a day, five days a week, 52 weeks per year with no additional operator or maintenance person at the plant. Annual operation and maintenance costs include the cost for electricity, gas, and maintenance (e.g., lubricants and replacement parts). The O&M costs include costs for hauling and applying the dried biosolids at a land application site. The thermal dryer process has the second highest energy cost of the options evaluated. On the other hand, its greater volume reduction results in a cost savings with regard to land application.

6.4 Non-Economic Evaluation

Many non-cost parameters and constraints affect either positively or negatively the ranking of the alternatives under evaluation. These criteria refer to such issues as ease of implementing the alternatives, operability, and space impacts. These issues may not have a cost associated but may impact the operation of the facility. In order for the alternatives to be acceptable for implementation, these parameters must be satisfied, and their negative impacts must be minimized. The non-cost parameters considered in this evaluation of each alternative are public acceptance/potential for odor, long-term viability/regulatory requirements, constructability and space constraints, ease of use/maintainability, flexibility/adaptability, reliability/performance, safety impacts, and final product end use.

6.4.1 Alternative No. 1 – Aerobic Digestion

The advantages and disadvantages of the Aerobic Digestion alternative include the following:

Advantages/Disadvantages

Advantages:

- Lowest construction and present worth costs of the alternatives evaluated.
- The operation of aerobic systems is relatively easy compared to that of anaerobic systems.
- Proven process. Widely used in the industry. Simple operational control
- Low odors.
- Good volatile solids destruction (35 to 50%) – reduces total sludge mass.
- PSRP process (Class B). Biosolids suitable for agricultural use containing nutrients and organic matter that can improve the fertility and texture of soils.
- Safety impacts are minimal.
- Tanks and Operations Building are already existing; therefore, relatively limited concrete construction work is needed.
- Supernatant is of a better quality than that from anaerobic processes.
- Aerobic digestion is effective in reducing the quantity of grease and oil in the sludge mass.

Disadvantages:

- Biosolids typically are difficult to dewater by mechanical means.
- Learning curve is required.
- The process requires large amounts of energy to produce a stabilized end product.
- Performance is very much affected by the sludge temperature and the ambient air temperature. Cold temperatures adversely affect performance.
- The process requires long detention times to meet the definition of a PSRP.
- The process is strictly a biological process – beyond operator control.

- Lower volatile suspended solids destruction than anaerobic.
- May experience foaming.

6.4.2 Alternative No. 2 – High-Rate Anaerobic Digestion

The advantages and disadvantages of the High-Rate Anaerobic Digestion alternative include the following:

Advantages/Disadvantages

Advantages:

- Low net energy requirements.
- Ability to use existing Operations Building for digester heating and mixing equipment.
- The process produces a recoverable energy by-product, methane gas, which can be burned to provide energy for sustaining the process. Surplus methane can be used for other purposes within the treatment plant including heating, fuel for an engine-driven aeration blower, or generation of electricity. Net operational cost can be low if methane gas is used.
- Reduces the total sludge mass requiring disposal. Typically 25-45% of the raw sludge solids and 40-50% of the volatile solids are destroyed during the digestion process.
- Reduces the odor potential and opportunity for rodents and insects to be attracted to the resulting sludge product.
- Inactivates pathogens during its lengthy processing time.
- Tanks and Operations Building are already existing; therefore, relatively limited concrete construction work is needed.
- Proven technology with proven system equipment - most widely used stabilization process in the wastewater industry. Process reliability is high.
- Existing tanks can handle future plant capacity if WAS is thickened prior to introducing to digesters.
- No adverse environmental impacts are anticipated.
- Plant staff is familiar with the operation and maintenance of an anaerobic digestion system since it is currently used at the plant – no real learning curve.
- Biosolids suitable for agricultural use containing nutrients and organic matter that can improve the fertility and texture of soils.

Disadvantages:

- Produces a strong recycle stream (supernatant) that can have a high oxygen demand and concentrations of nitrogen and suspended solids. These streams can impact the overall plant treatment process.
- Can generate nuisance odors resulting from anaerobic nature of the process.

- The production of methane gas raises safety issues concerning flammability of the gas. High capital cost for gas handling and safety equipment.
- Requires a significant amount of mechanical equipment. The complexity of the equipment requires a qualified operating staff. Requires skilled operators for process control. Digester cleaning is difficult (scum and grit).
- Process is susceptible to upsets because methane formers (principle microorganisms involved in the decomposition process) are sensitive to small changes in their environment. Anaerobic bacteria are slow-growing and typically recover slowly from any upset.
- May continue to experience foaming.

6.4.3 Alternative No. 3 – Autothermal Thermophilic Aerobic Digestion (ATAD)

The advantages and disadvantages of the ATAD alternative include the following:

Advantages/Disadvantages

Advantages:

- Achieves good volatile solids destruction (55 to 60%). Reduces total sludge mass requiring disposal.
- ATAD is a PFRP – an EQ biosolids digestion process. Biosolids are suitable for land application and/or distribution and marketing. Product can serve as a soil amendment enriching the soil with essential nutrients and organic matter. Product can also be blended with other organic materials such as yard waste compost.
- Reduced hydraulic retention time compared with conventional aerobic digestion.
- The existing digester tanks can handle future plant buildout capacity.
- No adverse environmental impacts are anticipated.
- Safety impacts are minimal.
- ATAD is a proven and reliable technology with several full-scale systems operational in the United States, including four in Ohio.
- Tanks and Operations Building are already existing; therefore, relatively limited concrete construction work is needed.

Disadvantages:

- Highest construction and present worth costs of the options evaluated.
- Significantly higher energy consumption and cost than the other anaerobic digestion alternatives evaluated.
- Can generate nuisance odors.
- Although not as complex as anaerobic digestion equipment, the equipment requires a qualified operating staff. Requires skilled operators for process control. Learning curve is required.

- May experience foaming.
- Cooling step (SNDR) is required for efficient dewatering.
- Thickening to 5% solids is required.

6.4.4 Alternative No. 4 – Indirect Thermal Drying

The advantages and disadvantages of the Indirect Drying System alternative include the following:

Advantages/Disadvantages

Advantages:

- Thermal drying reduces the volume and weight of biosolids produced at the plant. This results in reduced transportation costs and storage, and less biosolids to dispose of/utilize.
- Because indirect dryers generate limited quantities of non-condensable gas, little odor control treatment is required. Odors arising from the process can be contained and controlled.
- Dust problems are reduced because of the small volume of carrier or sweep gases used in indirect drying.
- Heat-dried biosolids are EQ biosolids.
- Smallest footprint required as compared to all of the drying alternatives evaluated in this study. Site work is minimal. Facility layout is simple.
- Can be combined with other processes
- Thermal drying has a high potential for public acceptance. Odors can be contained and controlled.
- The KS Paddle Dryer system is proven process with over 100 systems installed worldwide, including over 40 in the U.S.
- Not a biological process so it can be started quickly.

Disadvantages:

- Energy intensive.
- High gas prices are a concern.
- Complexity of drying equipment requires qualified operating staff.
- Maintenance requirements are typically high. A learning curve would be required. Erosion of conveying equipment and dryer shells by abrasive dried sludge can be a major maintenance problem.
- Safety hazards are an issue with sludge dryers and product storage. Safety concerns of thermal drying include the explosivity of dust and potential for product overheating and fires. However, indirect dryers allow operation under a vacuum or closely controlled atmosphere. Therefore, fire and explosion hazards are reduced within such units.
- Indirect drying system has a potential for dust creation.

- Air emissions are produced at any thermal drying facility. Air permitting and air pollution control may be required.
- End product is inconsistent in size (poorly graded) and contains a significant amount of dust. Lesser quality product than the direct dryer options. The dustier product may limit marketing options, increase handling and storage costs, and necessitate further processing of the product by compaction and/or screening to create a product acceptable to the fertilizer industry.
- Dust control is a major concern with trucking the material to its destination.

6.5 Summary of Evaluation

TABLE 6-1 – Present Worth Cost Analysis of Biosolids Management Plan Alternatives

	Aerobic Digestion (Jet mix)	Aerobic Digestion (Diffuser)	High-Rate Anaerobic Digestion	ATAD	Indirect Thermal Drying
Construction Cost					
Equipment	$1,573,000	$1,420,000	$4,182,000	$4,487,000	$3,620,000
Demolition	$100,000	$100,000	$100,000	$125,000	$180,000
Site Work	$25,000	$25,000	$25,000	$60,000	$60,000
Operations Building Work	$50,000	$50,000	$50,000	$50,000	$50,000
New Dewatering/ Storage Building	$830,000	$830,000	$795,000	$655,000	$480,000
Temporary Dewatering and Landfilling	$110,500	$110,500	$110,500	$110,500	$110,500
Subtotal	$2,688,500	$2,535,500	$5,262,500	$5,487,500	$4,500,500
Contingency @ 30%	$811,500	$764,500	$1,537,500	$1,612,500	$1,399,500
Total Construction Cost	$3,500,000	$3,300,000	$6,800,000	$7,100,000	$5,900,000
Annual O&M Cost					
Electricity	$78,000	$47,000	$23,200	$184,000	$119,000
Labor	$0	$0	$0	$0	$0
Maintenance	$27,000	$25,000	$73,000	$64,000	$63,000
Gas	$0	$0	$0	$0	$47,000
Hauling/Land Application	$21,000	$21,000	$20,000	$17,000	$14,000
Total Annual O&M Cost	$126,000	$93,000	$116,000	$265,000	$243,000
Present Worth O&M Costs	**$1,793,000**	**$1,323,000**	**$1,651,000**	**$3,771,000**	**$3,458,000**
Total Present Worth Cost	**$5,300,000**	**$4,600,000**	**$8, 500,000**	**$10,900,000**	**$9,300,000**

6.6 Conclusions and Recommendation

Four biosolids management alternatives – three digestion and one indirect thermal dryer combined with digestion – have been developed. Each alternative was developed and compared in terms of facility requirements, regulatory requirements, construction cost, annual O&M cost, and present worth/life cycle costs. In addition, advantages and disadvantages of implementing each option related to cost and non-cost parameters were identified. This information developed and analyzed assisted in arriving at the recommended Biosolids Management Plan for the Piqua WWTP.

The aerobic digestion alternative has both the lowest construction cost and lowest present worth cost of the four options evaluated. On the other hand, the ATAD process has both the highest construction cost and the highest present worth cost.

The aerobic digestion option would be easy to construct and has a track record of being easy to operate; however, there would be a learning curve for the plant staff. The high-rate anaerobic digestion alternative would include upgrades to the existing digesters so that they would run at mesophilic and thermophilic temperatures. Although Class B biosolids (mesophilic) would still be the end result, increased volatile solids destruction and volume reduction would be benefits of high-rating the digesters. Anaerobic digestion is the current sludge stabilization process at the Piqua WWTP, so the learning curve for this option would be minimal.

ATAD is a proven process that has been used for years at numerous treatment plants throughout the US, including four successful operations in Ohio (Middletown, Delphos, Bowling Green, and Portsmouth). The ATAD option would not only provide the biological stabilization benefits of digestion at a similar cost but also offer the added benefit of producing EQ biosolids. The product could also be blended with the end product produced at the City's yard waste composting facility. The ATAD system does not have the inherent digester gas handling and safety issues as with the high-rate anaerobic digestion alternative. Although it would require a learning curve to operate and maintain the system, the equipment is not as complex as anaerobic digestion equipment. Converting the existing digestion system to an ATAD system would be straightforward. The construction cost is slightly higher than the high-rate anaerobic digestion alternative, but more importantly, this system is an EQ process.

As stated above, the aerobic digestion alternative has both the lowest construction cost of the four options evaluated, and the lowest present worth cost. Although this system would not produce EQ biosolids, it offers the City a low cost upgrade option initially that has the flexibility to add indirect thermal drying in the future to achieve an EQ end product.

Based on the advantages and disadvantages presented and evaluated, the recommended Biosolids Management Plan to be implemented at the Piqua WWTP is the aerobic digestion system with utilization of the end product via land application.

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Section 7

WWTP Recommendations and Cost Optimization

7.1 WWTP Process Recommendations

Four liquid treatment train alternatives and four solids treatment process alternatives were short-listed from an initial group of WWTP improvement alternatives for in-depth evaluation to meet the future wastewater treatment needs at Piqua. Additional treatment capacity is needed to eliminate the SSO and provide for future system growth and development within the municipal sewer service area. The in-depth evaluations within this Amended WWTP Facility Plan were all based on hydraulic and process treatment capacities combined with equalization storage volumes derived in Section 2 of this Amended WWTP Facility Plan. The design WWTP capacities were proposed at 7.0 MGD average-day, maximum-month flow and 14 MGD peak flow, with 6 MG flow equalization storage volume.

The evaluation process resulted in a recommendation for the 7.0 MGD Oxidation Ditch treatment plant as the preferred liquid treatment train process. Aerobic digestion was recommended to complement the extended aeration oxidation ditch process and provide the City with the lowest capital cost solids treatment alternative.

Preliminary construction cost estimates were developed based on the WWTP capacities and EQ storage volume mentioned above. **Table 7-1** shows the Engineer's Preliminary Opinion of the Probable Construction Cost for the recommended liquid treatment train and solids treatment process improvements, as well as the estimated construction cost of an additional EQ basin and pump station needed to provide 6 MG of equalization storage.

Table 7-1: Engineer's Preliminary Opinion of Probable Construction Cost

Wastewater Treatment or Storage Improvement	Construction Cost Estimate
7.0-MGD Oxidation Ditch WWTP – Liquid Train	$33,000,000
Aerobic Digestion Facilities – Solids Treatment Process	$3,300,000
Additional 3-MG EQ Basin and Pump Station	$5,000,000
Combined Treatment and Storage Capital Cost	$41,300,000

Based on the process evaluations in the preceding sections of this document, the recommended treatment components for the liquid treatment train and solids treatment process are shown in **Figure 7-1** on the following page.

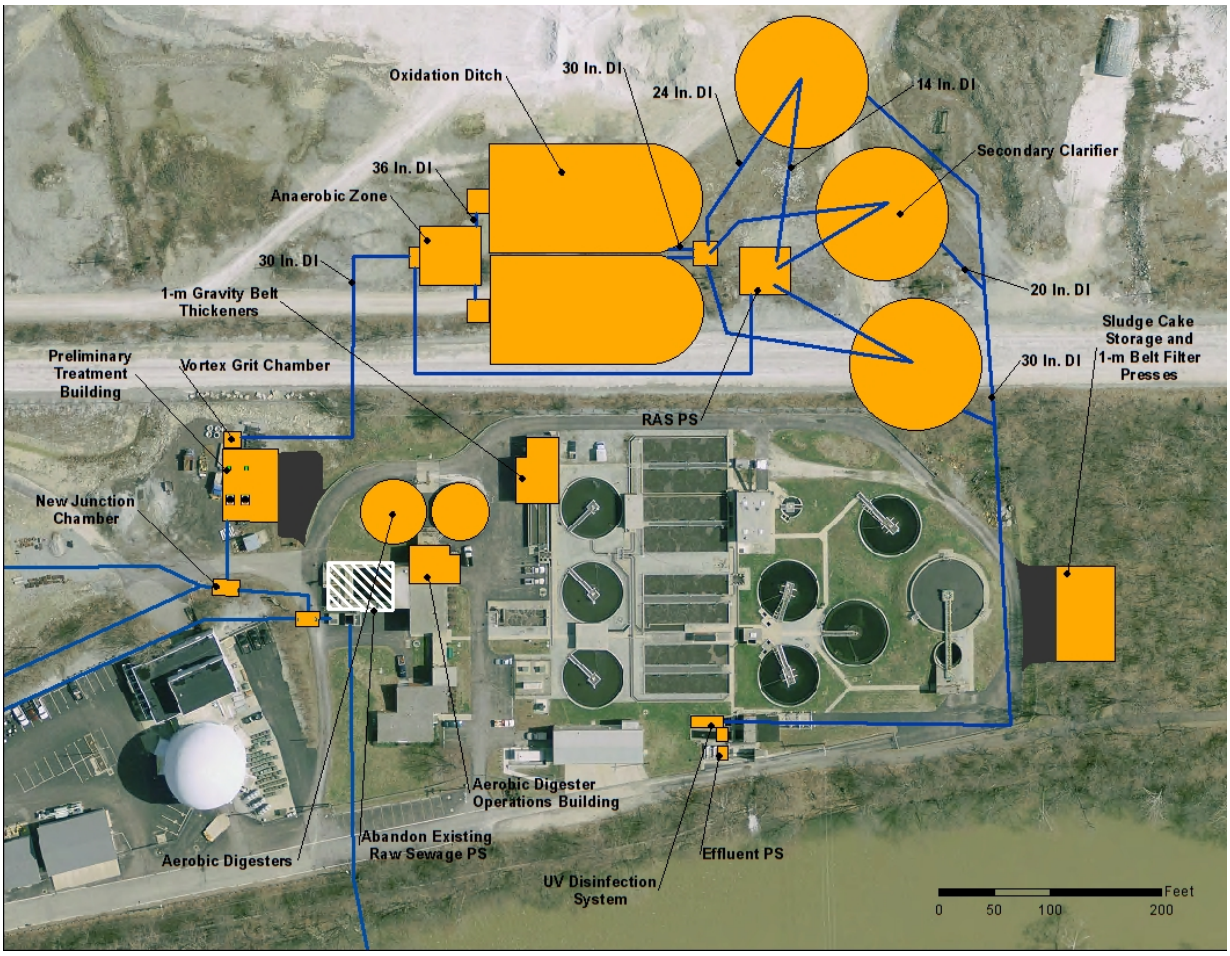


Figure 7-1: Recommended WWTP Improvements

Additional WWTP improvements and/or operational maintenance are also necessary for the existing plant to maintain its treatment performance and meet regulatory requirements as long as the existing facilities stay in service. As detailed in Section 1 of this Amended WWTP Facility Plan, the following unit processes should receive needed maintenance, repairs, and investment to keep the WWTP operating successfully until a new WWTP is online:

- Gear box repairs for two of the influent screw pumps
- Replacement of the mechanical fine screen to comply with biosolids regulations
- Installation of new air flow meters and DO analyzers and better control of the internal recycle mixers within the aeration tanks for improved control and efficiency in aeration, and maintenance of a distinct anoxic zone within these tanks
- Repair of the flow-control gates in the flow diversion chamber upstream of the secondary clarifiers, allowing more positive control of flow to the clarifiers and the capability to isolate each clarifier for inspection and maintenance
- Replacement of the effluent flow meter to pace disinfection (before and after proposed WWTP improvements) and for compliance with the City's NPDES permit

7.2 Cost Optimization

Section 2 of this Amended WWTP Facility Plan included an explanation of how the required treatment capacity and EQ basin storage capacity were derived for the evaluation of liquid treatment train alternatives. The design capacity for the WWTP was based on future population and flow projections; including potential flow from the Village of Covington, this capacity was set at 7.0 MGD. The maximum-day/peak capacity was based on sewer system modeling to determine what would be necessary to eliminate the SSO. The modeling indicated a range of peak treatment capacities would be adequate for eliminating the SSO, depending on the volume of EQ basin storage provided. The cost optimization curve in Section 2 predicted that 14 MGD peak treatment capacity matched with 6 MG of EQ basin storage would be the most cost-effective arrangement.

The cost optimization approach described above and in Section 2 was based on arbitrary unit costs of construction for planning-level accuracy. In this Section 7 the cost optimization analysis is taken a step further, based on the Engineer's Opinion of Probable Construction Cost developed for the liquid treatment train alternative and the solids treatment process alternative recommended from the preceding sections of this document. These are noted in **Table 7-1** above.

7.2.1 Potential Cost Savings – Most Cost-Effective Project Combination

The recommended wastewater system improvements described within this document represent a major investment by the City of Piqua. It is important to analyze that investment to determine whether there are opportunities to decrease the overall cost. This additional level of cost optimization is focused on the most cost-effective combination of improvements that would meet regulatory requirements and optimize the investment, getting the "most bang for the buck" of the City's wastewater system improvements. The following paragraphs describe the cost optimization.

Similar to the approach in Section 2, a range of treatment capacities and EQ basin storage volume was re-evaluated to determine the most cost-effective combination of improvements. The most significant capital cost being considered is the amount needed to design and construct the 7.0-MGD oxidation ditch treatment plant. With a peak capacity of 14 MGD, this proposed treatment plant would be built along with a new 3-MG EQ basin and EQ pump station (total EQ basin storage capacity – 6 MG) – the capacities projected in Section 2 that would meet the requirements for SSO elimination.

The capital cost for the 7.0-MGD WWTP is estimated to be $36,500,000, including both liquid and solid treatment trains. The capital cost for the EQ basin and pump station is estimated to be $5,000,000, which includes an estimated $3,000,000 for the new EQ basin and $2,000,000 for the pump station. This cost optimization process asks the question, "Is the additional $5,000,000 for the EQ basin and pump station needed for SSO elimination, or could an incremental increase in treatment plant capacity meet the requirement for SSO elimination at a lower capital cost?"

New cost quotes from equipment and treatment process vendors were obtained for every major treatment process related to the proposed 14.0-MGD peak-capacity oxidation ditch treatment plant, corresponding to the following revised combinations of treatment capacity and EQ basin storage (similar to Section 2 of this document):

- 14-MGD treatment/6-MG storage (the original recommended plan; requires a new 3-MG EQ basin and EQ pump station)
- 18-MGD treatment/3-MG storage (requires an EQ pump station, but no new EQ basin)

- 20-MGD treatment/2-MG storage (requires a new EQ basin, but no EQ pump station)
- 22.5-MGD treatment/1-MG storage (requires no additional EQ storage or pump station)

The Engineer's Opinion of the Probable Construction Cost was then developed for each of the above scenarios to determine the most cost-effective combination. **Table 7-2** shows the results of the revised construction cost estimating.

Table 7-2: Evaluation of Revised Engineer's Opinion of Probable Construction Cost

Wastewater Treatment or Storage Improvement	Construction Cost Estimate			
	14-MGD WWTP; 6-MG EQ Basin	18-MGD WWTP; 3-MG EQ Basin	20-MGD WWTP; 2-MG EQ Basin	22.5-MGD WWTP; 1-MG EQ Basin
7.0-MGD Oxidation Ditch WWTP – Liquid Treatment Train	$33,000,000	$34,400,000	$35,000,000	$35,700,000
Aerobic Digestion Facilities – Solids Treatment Process	$3,300,000	$3,300,000	$3,300,000	$3,300,000
Additional EQ Basin and Pump Station	$5,000,000	$2,000,000	$3,000,000	$0
Combined Treatment and Storage Capital Cost	$41,300,000	$39,700,000	$41,300,000	$39,000,000

Based on the cost optimization demonstrated in **Table 7-2**, the **22.5-MGD peak-capacity WWTP combined with 1 MG of EQ basin storage** is the most cost-effective combination of treatment capacity and EQ basin storage. This means there would be no revisions to the existing EQ basin and no new EQ pump station built. **This is the recommended combination for WWTP design and construction for Piqua.**

7.2.2 Potential Additional Cost Savings – Revised Design Parameters

Another focus of cost optimization relates to the area of regulatory compliance. Are there any treatment plant components or processes that are currently not needed for regulatory compliance, but could be added later if future regulations from Ohio EPA require them? Three treatment unit processes were considered for this evaluation.

1. The oxidation ditch treatment plant recommended for implementation includes large tanks to provide an anaerobic zone for phosphorus removal in anticipation of future phosphorus discharge limits in Piqua's NPDES permit. These discharge limits are not yet required, and the anaerobic tanks are therefore not yet needed. Extended aeration treatment could be provided without this additional unit process until regulatory requirements are changed.

2. Though the existing anaerobic digesters are in need of repairs, an evaluation of the conversion to aerobic digesters was made to assess whether there could be cost savings in delaying such a conversion. It was determined that the anaerobic digesters were not well-suited for sludge digestion of solids from an extended aeration process. Further, the condition of the existing anaerobic digesters is such that extensive repairs would be needed even if they were kept in service. No change is proposed in this recommendation.

3. An assessment of the disinfection process was made to determine whether continued reliance on the existing chlorine contact tanks and chlorination/dechlorination equipment could result in cost savings of converting to UV disinfection. It was quickly determined that the chlorine

contact tanks have inadequate capacity for the revised treatment plant flow rates corresponding to the proposed improvements. Therefore, as covered earlier in this document, the recommendation for conversion to UV disinfection was kept.

Of the above three unit processes, it was determined that design and construction of the phosphorus removal facilities could be delayed until they are actually needed, resulting in immediate savings in the estimated capital cost of those facilities. **Table 7-3** illustrates the impact of this revised design parameter on the Engineer's Preliminary Opinion of the Probable Construction Cost.

Table 7-3: Revised Engineer's Preliminary Opinion of Probable Construction Cost

Wastewater Treatment or Storage Improvement	Construction Cost Estimate
7.0-MGD Average Daily Flow / 22.5-MGD Peak Flow Oxidation Ditch WWTP – Liquid Treatment Train	$35,700,000
Aerobic Digestion Facilities – Solids Treatment Process	$3,300,000
Savings in delaying construction of anaerobic tanks for phosphorus removal	-$500,000
Combined Treatment and Storage Capital Cost	$38,500,000

7.3 Final Recommendations

Based on the treatment process evaluations described in Sections 3-6 of this Amended WWTP Facility Plan, and based on the cost optimization described above, ***the 7.0-MGD Oxidation Ditch treatment plant with a peak capacity of 22.5 MGD is the recommended liquid treatment train alternative for upcoming WWTP improvements. It is also recommended that the existing anaerobic digesters be converted to aerobic digesters.*** This recommendation does not include an expansion of the EQ basin facilities.

City officials should review, comment on, and ultimately accept this Amended WWTP Facility Plan, authorizing the preliminary design phase and development of the Preliminary Engineering Report (PER) for the 7.0-MGD Oxidation Ditch treatment plant and aerobic digestion facilities.

Additional ultraviolet transmittance (UVT) data should be collected on the existing treatment plant's effluent to support the UV disinfection system design. Initial testing for average and high flows during two events in December 2011 provided insight that the existing treatment plant produces effluent quality with high enough UVT to utilize this new disinfection system. However, more data should be collected to support design and system sizing needs and provide more confidence in the equipment selection to continue to meet the NPDES disinfection requirements.

The City has several financing options that can be considered for the design and construction of the recommended improvements. These are described in Section 8. Dividing the project into separate phases and loans can stagger the construction costs in different years to fall below the $25 million threshold to achieve optimal interest rate discounts through OWDA.

The recommended alternative cost summary for construction costs and overall project cost is provided in **Table 7-4**. The cost of property acquisition is not included in these costs.

Table 7-4: Recommended Alternative Cost Summary

Description	Cost
Selected Liquid Treatment Alternative Alt. 3 – 7.0-MGD Oxidation Ditch Treatment Plant	$35,700,000
Selected Solids Processing Alternative Alt. 2 – Aerobic Digestion	$3,300,000
Savings in delaying construction of anaerobic tanks for phosphorus removal	-$500,000
Preliminary Opinion of Probable Construction Cost	$38,500,000
Preliminary Engineering & Detailed Design	$3,900,000
Construction Phase Engineering Services*	$4,800,000
Total Estimated Project Cost	**$47,200,000**
*Construction Phase Engineering Services include Applications Engineering, Startup and Training, Computerized Maintenance Management System (CMMS), Electronic O&M Manual, and Resident Project Representation (RPR) Services.	

7.4 Project Schedule

Implementing the recommended improvements should follow a normal progression of design and construction, and include necessary time periods for regulatory review and plan approvals. The anticipated schedule of activities and milestones is presented in **Table 7-5**.

Table 7-5: Project Implementation Schedule

Activity/Milestone	Approximate Dates	Months
Ohio EPA Amended WWTP Facility Plan Approval	9/2014 – 12/2014	3
Preliminary Engineering Report	12/2014 – 6/2015	6
Ohio EPA Preliminary Engineering Report Approval	6/2015 – 9/2015	3
Detailed Design	10/2015 – 4/2017	18
Ohio EPA PTI Approval	5/2017 – 10/2017	5
Advertise for Bids	11/2017 – 12/2017	1
Award Construction Contract	2/2018	1
Begin Construction	3/2018	1
Construction Period	3/2018 – 2/2020	24
NPDES Milestone to Eliminate SSO	2/28/2020	

The above Project Implementation Schedule includes key dates contained in the City's NPDES Permit Compliance Schedule related to SSO elimination, as well as other project milestones intended to keep the project on track for NPDES Permit compliance. The City's NPDES permit was modified with an effective date of August 1, 2014, and expiration date of January 31, 2016. This permit modification includes some of the same project milestones shown above that occur after the permit expiration, but will be carried forth in the next NPDES permit.

Three of the project tasks listed in **Table 7-4** address Ohio EPA approval of the Amended WWTP Facility, the upcoming Preliminary Engineering Report, and the detailed design of proposed improvements. In each case, the City does not directly control the Ohio EPA approval process, but the Project Implementation Schedule allots a reasonable amount of time to complete each approval task.

Section 8

Project Financing Options

Piqua has multiple options to finance the capital cost necessary to implement the WWTP improvements. Loans will be required to fund the planning, design and construction efforts. Piqua already has a loan from the Ohio Water Development Authority (OWDA) in the amount of $3,345,100 for the design of proposed wastewater system improvements to eliminate the SSO. Available funding sources for other financing needs such as the construction of wastewater system improvements include multiple State of Ohio programs and municipal revenue bonds that are described below. Different options carry different requirements for approvals as well as payment terms and interest rates.

- **Water Pollution Control Loan Fund (WPCLF)** is administered through the Ohio EPA Department of Environmental and Financial Assistance (DEFA). Funds are available through this agency for planning, design, and construction phases of the project. Discussions with Ohio EPA/DEFA staff indicate that their funding is aimed at addressing existing problems instead of funding growth. Standard loan rates through WPCLF are at 3.06% for September 2014, for 20 years for construction loans and for 5 years for planning and design loans, which can be rolled into the construction loan. There is a loan application fee of 0.35% of the project amount. A discount of up to 0.2% on the interest rate is available for conversion from Class B Biosolids to Exceptional Quality Biosolids processes. The value of savings on the loan amount is limited to the cost of the facilities needed to accomplish the sludge processing enhancement. It is necessary for Ohio EPA/DEFA to review the facilities planning and complete an environmental review similar to the one prepared for the EQ basin project. The environmental review will require the issuance of a Finding of No Significant Impact (FONSI) prior to the award of the design loan.

- **Ohio Water Development Authority (OWDA)** provides a variety of funding for planning, design, and construction of wastewater facilities under their Freshwater Program without regard for whether the project is funding growth or is addressing existing problems. Rates are currently 4.29% for communities the size of Piqua (over 5,000 population) for both design and construction loans; with a potential discount of 0.5% for communities with prior borrowing experience with OWDA (which Piqua has) for construction projects up to $25 million in any one calendar year. No discounts are available for construction projects over $25 million. There is a loan application fee of 0.35% of the project amount. Unlike with WPCLF funding, this source of funds has the advantage of ready availability and funding payment terms of up to 30 years are available. Like with WPCLF, this funding source is also without the issuance and coverage costs of conventional revenue bonding.

  As mentioned above, Piqua has a loan in the amount of $3,345,100 from OWDA for the design of improvements for SSO elimination. This loan has an interest rate of 2.44% and a term of 5 years.

- **Ohio Department of Development** has funds available to assist communities and encourage industrial and commercial development based on formulas linked to the number of new jobs created. These programs are generally aimed at specific large-scale employment opportunities,

but may be available for limited funding of infrastructure for smaller projects. To qualify for these funds, it is essential to have specific economic development plans with demonstrable economic impact. At this time, this is not a viable source for the current improvements, but should be considered for future industrial developments to be located in Piqua.

- **Community Development Block Grant (CDBG)** funds are generally limited and are available for low- and moderate-income areas. The funds are usually restricted to addressing existing problems. Seeking these funds would be most appropriate for wastewater collection system improvements specifically directed at economically disadvantaged portions of the community.
- **Ohio Public Works Commission (OPWC)** provides a grant/loan program for which interest rates and the mix of grant and loan percentages will vary. Project awards are competitive with other projects in Ohio, and an award would require significant effort to secure. Competition is likely from other projects needing financial support in a multi-county district. A request for participation on a single project element may be advantageous; e.g., implementation of a part of the off-site piping work (siphon improvements) could be submitted for consideration.
- **Conventional bonding** involves variable rates depending on market conditions and community bond rating and possibly requiring bond insurance. Current interest rates for AA-rated, 20-year maturity municipal (general obligation) bonds found on internet listings are approximately 0.5 to 4.0% and revenue bonds would typically be higher. More detailed information on current bond market funding as it relates to Piqua should be obtained from the City's financial advisor. Note that use of general obligation bonds may adversely affect the City's ability to borrow for other necessary projects, as the total general obligation indebtedness is limited.

Table 8-1: Most Viable State Funding Loan Programs

Funding Source	Availability	Loan Admin. Fee	Current Interest Rates[1]	Loan Period	Interest Rate Discounts
WPCLF	Planning Design	0.35 % of Total	3.06%	5 years	▪ 0.2% for upgrade to Exceptional Quality Biosolids production
	Construction	0.35 % of Total	3.06%	20 years	
OWDA	Planning Design	0.35 % of Total	4.29%	5 years	▪ 0.5% for prior Ohio EPA customers ▪ 0.5% for Ohio EPA Findings and Orders or documented health risk ▪ Cap or reduction 1.0% for borrowing in any one year for loans up to $15M and 0.5% for loans $15M - $25M. No discounts available for loans > $25M
	Construction	0.35 % of Total	4.29%	Up to 30 years	

[1] Interest rates are subject to change on a monthly basis, and are anticipated to be significantly changed for OWDA funding mechanisms.

Draft Technical Memorandum No. 1
City of Piqua, Ohio
Amended WWTP Facility Plan and Preliminary Engineering Report

To: *David Burtner, Dave Davis, Amy Havenar, Chris Melvin, Greg Peltier*

From: *CDM Smith*

Date: *August 5, 2014*

Subject: ***Existing Wastewater Treatment Plant Operational Assessment and Optimization***

Purpose

Amendment No. 1 to the Agreement between CDM Smith and the City of Piqua, Ohio, was signed on April 25, 2014, authorizing development of the Amended Wastewater Treatment Plant (WWTP) Facility Plan and Preliminary Engineering Report (PER). One of the first tasks of this project is the Existing Facility Operational Assessment and Optimization, documented in this Technical Memorandum No. 1. The scope of engineering services for the Existing Facility Operational Assessment and Optimization is summarized as follows:

- Focus on improving the operations of the secondary treatment processes at the WWTP to promote better sludge settleability.
- Meet with WWTP staff to gain a better understanding of WWTP operations.
- Determine how each of the existing secondary treatment processes can be optimized for incorporation into a long-term plan for the WWTP.
- Update the BioWin process model to assess impacts of varying influent loadings and flow rates and treatment approach variations, and to simulate adjustments in sludge return rates and aeration, leading to recommendations for process modifications.
- Provide an Operations Specialist onsite periodically to review WWTP operational procedures, results, and performance, including adjustments to aeration rate, return sludge rate, and preserving and re-establishing the anoxic zone in the biological selector. The Operations Specialist will provide input to the Owner regarding data collection and record-keeping for the facility assessment. The Operations Specialist will also work with the Owner to make agreed-upon process modifications for improved WWTP performance.

- Develop Technical Memorandum #1 that will include recommendations for improvements to secondary treatment processes.

Project Kickoff Meeting and WWTP Tour on 4/17/14

The Kickoff Meeting for the Piqua Amended WWTP Facility Plan and PER project was held on April 17, 2014, and is documented in meeting minutes separate from this Technical Memorandum. During the meeting it was noted that there will be a specific emphasis placed on review of the WWTP's current treatment processes and performance.

The current average influent flow rate is 3.9-4.0 million gallons per day (MGD), based on the past 5 years of WWTP performance records. Wet-weather flow treatment capability is 8.0-8.3 MGD.

Currently the WWTP is in general compliance with Ohio EPA regulations and the City's NPDES permit, meeting discharge limits, including effective removal of CBOD5, suspended solids (TSS), and ammonia (NH3). The major area of periodic noncompliance is the existence and operation of a sanitary sewer overflow (SSO) in the sanitary collection system near the treatment plant. One of the main purposes of this Amended Facility Plan is to recommend WWTP improvements that will enable the City of Piqua eliminate the SSO and thereby achieve full compliance with the City's NPDES permit. These matters were re-iterated at the Project Kickoff Meeting on April 17, 2014.

Current sludge production is approximately 230 dry tons a year.

For the past two years the NPDES permit has had E coli limits. Dave Davis said that the WWTP needs to meet an effluent ammonia concentration of 1.0 mg/L or less to create chloramine to meet the E coli limit.

Existing WWTP Condition and Performance

General WWTP condition and influent and effluent water quality were discussed during a meeting between CDM Smith's Operations Specialist and WWTP staff on April 18, 2014. The following paragraphs document the discussions during that meeting.

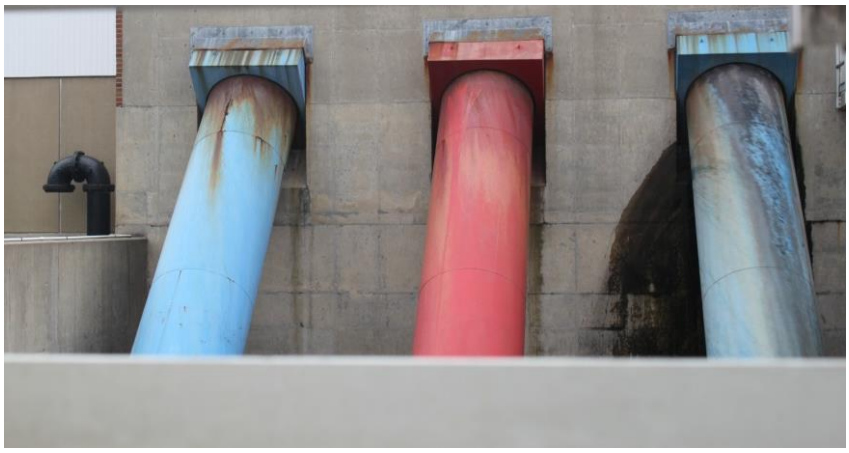
WWTP Influent Water Quality and Equalization (EQ) Basin

The City has an industrial pretreatment program (IPP). There are 4 regulated industries and 13 non-Significant Industrial Users (SIUs). WWTP influent is estimated to have 10% industrial flow and 15% flow from restaurants. There is no septage accepted at the WWTP.

The WWTP receives industrial flow from Hartzell Propeller. Hartzell has a chromium, cadmium and cyanide (CN) treatment system. The WWTP also gets waste from D&D Briteworks, an auto frame and parts stripping and refinishing company.

The WWTP receives weak flow with low BOD concentration. The EQ basin can currently store a little over 1 million gallons (MG), with influent and tank drains flowing by gravity. If a pump station is recommended, designed and built, the existing tank can store 3 MG. The EQ basin uses circulation mixers and has water cannons for cleaning the tank during drawdown.

Headworks



Three screw pumps convey WWTP influent to downstream treatment processes. Pumps No. 1 and 2 have been rebuilt. On April 18, 2014, the gearbox for Pump No. 2 was noisy and hot to the touch. (As documented later in this Tech Memo in telephone consultations in July 2014, both of these screw pumps had to be taken out of service for repair.) Dave Davis explained that the oil in the gear reducers for the screw pumps and other equipment throughout the WWTP is analyzed annually for wear, presence of particles that could cause deterioration, and general condition.

Screenings are removed with a screenings rake and washer/compacter equipped with a dewatering screw. Grit is removed within an aerated grit chamber and a grit classifier. Grease is removed in a chamber parallel to the aerated grit tank.

The Facility Plan project goal related to the Headworks is to evaluate and follow through with repairs that may be necessary to keep the equipment in successful operation until a WWTP upgrade is completed. It is recommended that in addition to oil analyses, the condition of the motors and gear reducers be checked routinely for excess temperature and noise. A heat sensing gun can be used to take temperature readings each week and the readings should be recorded.

Supernatant Oxidation Tank

The Supernatant Oxidation Tank receives digester supernatant and belt filter press filtrate. The aeration process within this tank is carried out using membrane diffusers. Following aeration, the settled supernatant pumps discharge to the headworks facility upstream of the mechanical screen.

When it is operated successfully, the Supernatant Oxidation Tank typically decreases the ammonia concentration in the side streams from approximately 700 mg/L to around 100 mg/L. The Supernatant Oxidation Tank gets overloaded when it receives filtrate from the belt filter press. The tank has a very light sludge that doesn't settle well. It has experienced filamentous bacteria and foaming problems in the past.

The Facility Plan project goal related to the Supernatant Oxidation Tank is to improve the oxygen transfer through the aeration process and promote a more consistent supernatant feed back to the main WWTP. It is recommended to routinely monitor the dissolved oxygen (DO) levels, especially when the belt filter press is in service.

Flow Splitter Box and Primary Clarifiers

There are three circular primary clarifiers. The flow splitter box upstream of the primary clarifiers is the only place where a positive flow split is provided throughout the plant. Flow is divided evenly among the three clarifiers.

The primary clarifier effluent channels are a hydraulic bottleneck. When influent flow is near the peak capacity of the plant, around 8.0 MGD, the primary clarifier weirs become submerged in primary clarifiers no. 1 and 2.

There are sludge slip tubes for each primary clarifier. Wemco recessed-impeller pumps withdraw primary sludge and convey it to the digesters at approximately 6% solids. There is no primary sludge flow metering; sludge withdrawal control is manual.

The Facility Plan project goal related to the primary clarifiers is to eliminate the hydraulic bottleneck downstream of the primary clarifiers and to add metering capability for the primary sludge flow. This goal should be implemented only if a future WWTP improvement or expansion will include continued utilization of the existing primary clarifiers. If this is not the case, the hydraulic bottleneck associated with the primary clarifiers will be eliminated when they are no longer in service.

Aeration Tanks

When the influent flow is around 8.0 MGD, the aeration tank baffles are submerged. This was the case on 4/17/14 and 4/18/14.

The mixed liquor suspended solids (MLSS) concentration is typically 1,500-2,000 mg/L. Though this concentration is lower than at some other treatment plants, Piqua experiences solids loss over the secondary clarifier weirs when the MLSS concentration exceeds 2,500 mg/L.

The aeration basins are supplied air by three 300-HP Hoffman centrifugal blowers. Normally only one is in service. Blowers no. 1 and 2 are driven by electric motors; blower no. 3 is driven by a biogas (methane)-driven engine. Blower no. 1 was rebuilt. The blower automatic valves are not used. Blower no. 3 runs for only 10-12 hours at a time due to low biogas production.

The automatic control loop for the blowers did not work. Blower valves slammed and blowers went into surge.

The aeration basin dissolved oxygen (DO) probes do not work. The operators use a YSI handheld meter to check DO within the basins. The DO in the no. 1 cell of the tanks is kept at 0.5 mg/L or less. The internal recycle (IR) mixer is set and the operators do not rotate the position, nor experiment with it, due to interference with the tank walls. The air is tapered upstream of the IR zone to avoid oxygen carryover into the anoxic zone. The aeration basin diffusers have been replaced or supplemented with membrane diffusers. The diffuser grids are not all at the same elevation.

The aeration tank grid dropleg pitot inserts have not been used to check the air flow to each grid because connection couplings are incomplete and there appear to be missing gauges. Dave Davis said that operators do not know how to use the pitot tubes. The pitot is missing the flat flange and union that couples it to each sampling point as well as the magnehelic gauge that the high and low connection ports would connect to.

The Facility Plan project goals related to the aeration tanks include the following:

- Experiment with the IR mixer orientation to determine best position for anoxic conditions in cell 1 of each tank.
- Install permanent air flow (SCFM) meters on the droplegs and DO analyzers that transmit DO concentration to the SCADA system for DO trending and optimization of the anoxic zone.

Secondary Clarifiers and Sludge Settleability

All four secondary clarifiers are in service. Recent MLSS settleometer readings have been in the range of 450 mL/L, though in July 2014 they were down to approximately 200 mL/L. A sludge judge core taken from secondary clarifier no. 4 on 4/18/14 indicated a fluffy sludge blanket depth of 9 ft. See the photo, below right. The sludge blanket had been reduced to around 1 ft. by July 2014. Probably because of the high flow rates and relatively high MLSS concentration on 4/18/14, the clarifiers were experiencing hydraulic bulking. Secondary clarifiers no. 1, 2, and 4 have a 12-ft. side water depth. Clarifier no. 3 has a 10-ft. side water depth.

Only secondary clarifiers no. 2 and 4 can be isolated and taken out of service. However, it is not easy to operate the clarifier isolation gates. The gate shaft stems are bent severely on the isolation gates for clarifiers no. 1 and 3, making them inoperable. The secondary clarifiers have not been dewatered in 10 years.

The ducking skimmer is problematic. Ashing was observed in the clarifiers.

The piping and a blind flange are installed to connect to a fifth secondary clarifier to the Mixed Liquor Distribution Box.

Three return activated sludge (RAS) pumps convey settled mixed liquor (or RAS) from the final clarifiers to the aeration tank splitter box. RAS flow is metered.

The Facility Plan goals related to the secondary clarifiers are (1) repair the isolation gates in the splitter chamber upstream of the clarifiers, and (2) take each clarifier out of service individually for cleaning and internal inspection.

Disinfection

NPDES Permit requires chlorination and dechlorination from May 1st to October 31st each year. Currently, gaseous chlorine in 1-ton containers is used for disinfection and sulfur dioxide is used for dechlorination. Cascade aeration is provided downstream of chlorination. DO in the effluent has been as high as 9.0 mg/L.

Effluent Pumps

The effluent pumps have been used only four times since the 1980s. See photo, right.

Hydraulic Wasting

Waste sludge is conveyed by the waste mixed liquor pumps from the Mixed Liquor Distribution Box to the dedicated hydraulic wasting tank at a flow rate of approximately 60 gpm, depending on the SRT that

operators are trying to achieve. Sludge is thickened in the hydraulic wasting tank. Two thickened sludge pumps convey sludge from the hydraulic wasting tank to the digester. Supernatant from the hydraulic wasting tank flows by gravity to the plant effluent. See photo, right.

Anaerobic Digesters

The anaerobic digesters are operated as mesophilic digesters. The primary digester has a volume of 325,000 gallons, a fixed cover, and is heated to 98°-102° F. The secondary digester also has a volume of 325,000 gallons, has a gas-holder cover, and is not heated. Together they provide a solids retention time (SRT) of 19-25 days, but typical operation is 17-18 days.

The digesters have not been cleaned since 1984. The winter of 2014 is the first winter that the primary digester did not foam. Typically it foams around Thanksgiving to early January every year. Mixing is accomplished with Degremont bubble cannons. Digester gas (methane) produced in the digesters is used to fuel the engine that heats the heat exchanger; waste gas is flared. A fuel oil boiler provides building heat. There is currently no natural gas at the WWTP site, though installation of a natural gas line is under consideration.

Digested Sludge Holding Tank

The digested sludge holding tank is at the south end of the treatment plant; it was formerly an aerobic digester. Wilo mixers with variable frequency drives (VFDs) serve to mix the sludge, operating at approximately 50% of full speed. The tank has been operated with sludge levels close to the overflow point, and the tank has overflowed in the past.

Sludge Dewatering

The City has a contract with Burch Hydro for dewatering and hauling biosolids. The belt filter press, owned by Burch Hydro, processes sludge conveyed from the digested sludge storage tank. From approximately 4% influent solids, the belt filter press produces a sludge cake with 14-15% solids. One Seepex belt filter press feed pump conveys sludge from the sludge holding tank to the belt filter press; a second pump is not in service. Two filtrate pumps convey belt filter press filtrate to the supernatant oxidation tank. Though the City benefits in its sludge hauling contract by paying for the total dry tonnage of biosolids hauled, improved dewatering and less sludge

volume could be realized with a more efficient dewatering device, such as a new belt filter press or centrifuge.

Site Stormwater

All site stormwater flows to the drainage pump station beneath the Operating Building. From there it is pumped to the mechanical screen influent box at the WWTP headworks via the influent screw pump force main. Thus, WWTP site stormwater is currently treated with the influent raw wastewater. The pumps at the drainage pump station have exceeded their useful life. There are yard drains located in the lawn areas of the plant.

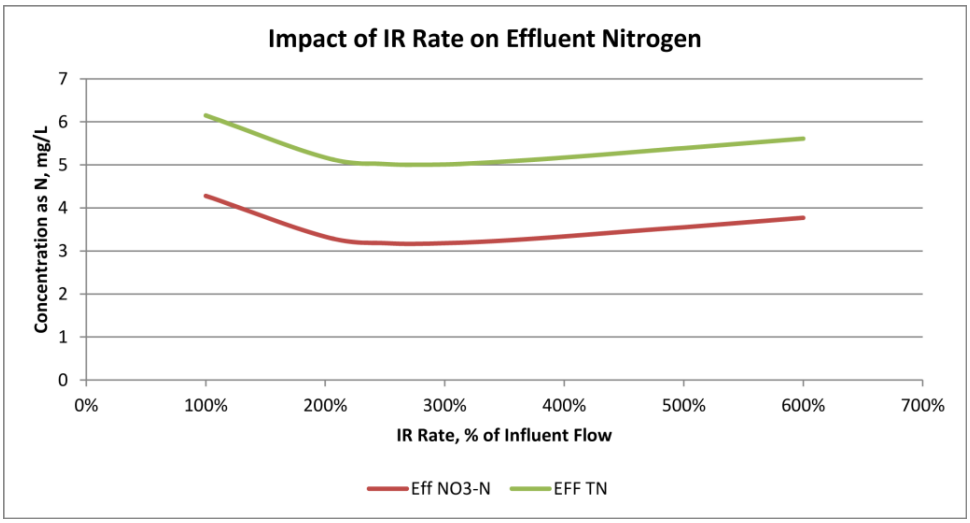
The Facility Plan goals related to the WWTP stormwater drainage system are (1) replace the pumps at the drainage pump station with pumps sized to convey tank drainage and stormwater flow anticipated throughout the WWTP, and (2) develop a stormwater management system that will prevent stormwater from being treated with the sanitary wastewater.

Treatment Chemicals

Polymer is used to improve solids separation at the belt filter press, gaseous chlorine is used for effluent disinfection and sulfur dioxide for dechlorination. In the past they have used a little bit of polymer in the primary clarifiers when they co-settled.

Biowin Modeling Results for WWTP Optimization

CDM Smith performed simulations of WWTP performance using the Biowin model, resulting in projections of total effluent nitrogen (TN) and ammonia (NH_3) based on adjusting the internal recycle rate within the aeration tanks. As shown in the graph below, the estimated optimum IR rate is 300% of WWTP influent flow rate. This illustrates the usefulness of the Biowin model in developing performance strategies to optimize treatment.



General Condition and Performance Overview

A copy of the two-volume plant Operations and Maintenance Manual is located in Dave Davis's office as well as in the operations room.

The laboratory is well-organized. The WWTP staff appears to be an experienced, cohesive and professional group that takes pride in their plant.

The City uses Orion probes for quick NH_3 checks.

Operations Changes Implemented Since September 2013

A major operational change that was implemented after September 2013 was increasing the aeration tank MLSS concentration from approximately 1,500 mg/L to approximately 2,500 mg/L. On April 18, 2014, the plant was operating at a 17-day SRT. Typical SRT for the summer months is expected to be 14-15 days. Plant staff also increased the RAS pumping rates. These operational changes were later further revised, as noted in the paragraphs below.

CDM Smith Operations Specialist Telephone Consultations

May 14, 2014

Discussions between CDM Smith's Operations Specialist, Georgine Grissop, and Dave Davis focused on recent treatment plant performance. The EQ basin had been utilized immediately prior to this call to prevent a sanitary sewer overflow, and the wastewater temporarily stored in the EQ basin was being released to WWTP the day following the rain event.

The condition of the secondary clarifiers was discussed, with a reference to clarifier sludge removal mechanisms at another treatment plant experiencing structural failure due to exceeding their life expectancy and lack of inspection and repair. Piqua's secondary clarifier sludge removal mechanisms cannot be inspected nor maintained unless special measures are taken to isolate the clarifiers from the flow distribution box, where slide gate stems are currently broken. CDM Smith advised that this step should be taken.

In the aeration tanks, the operation of the anoxic zone mixers has not been optimal, as their speed can be reduced to 50% of full speed, but no lower. Treatment plant operators sometimes take

them out of service. Adjusting their positions has not been successful in promoting anoxic conditions.

The MLSS concentration, which was maintained at elevated levels through the winter of 2014, was adjusted downward to 1,800-2,000 mg/L in May 2014 through increased wasting of sludge. This was intended to decrease the fluffy sludge blankets that persisted within the secondary clarifiers.

July 8, 2014

Another follow-up telephone consultation between Dave Davis and Georgine Grissop was held on July 8, 2014. Dave described the impact of recent heavy rain on the SSO and on WWTP influent flow rates. More than 7 inches of rain in June, including one day with 3.04 inches of rain, produced an SSO event lasting 8 days, followed by another SSO event because of another rain event. At the treatment plant, stormwater flow collected on the plant site was pumped to the front of the plant, immediately elevating influent flow rates and forcing the plant to treat unwanted stormwater.

DO levels in the aeration tanks were being maintained to optimize treatment (0.5-2.0 mg/L, depending on location in the tanks). Internal recycle (IR) mixers were operating at 60%-75% of full speed. Sludge wasting reduced the SRT to approximately 13.5 days and the MLSS concentration was down to 1,500-1,700 mg/L. As a result, sludge blankets in the secondary clarifiers were down to around one foot, and more dense than they were over the winter months. The SVI readings had been reduced from an average of 170 to 115.

Due to the wet weather, no aeration tanks or secondary clarifiers had been taken out of service for inspection or repair. Hoping for drier weather in July or August, a window of approximately one week would be needed to perform this task. Dave Davis was especially interested in reviewing the condition of aeration tank no. 3 because it was believed that a diffuser membrane was missing.

The pitot tube fittings had not been purchased to bring them into service, but Dave Davis was considering installing two DO probes per year that could enable monitoring DO in the tanks via the SCADA system.

July 22, 2014

A follow-up telephone consultation was conducted on July 22, 2014. Following the heavy rain in June was a relatively drier period that allowed treatment plant staff to take primary settling tank no. 2 out of service for inspection and maintenance. There were plans to take aeration tank no. 3 out of service to replace a diffuser membrane.

A more serious problem had occurred with two of the influent screw pumps. Screw pumps no. 1 and 2 were both out of service due to excessive noise during operation, indicating internal

problems. Only screw pump no. 3 was available and in service, along with the plant drain pump station, which is the former influent pump station. Flow cannot be measured from the plant drain pump station, however. Dave Davis notified Ohio EPA about this issue.

In spite of the above, the WWTP was performing well, with the SRT at approximately 13 days and well-settling sludge and good clarity in the plant effluent.

July 28, 2014

A telephone consultation on July 28, 2014, confirmed that the gear boxes for screw pumps no. 1 and 2 must be rebuilt. WWTP staff sent the gear box for pump no. 2 to a shop to be rebuilt on July 2nd, and the gear box for pump no. 1 will also be sent out to be rebuilt. Once they are repaired and replaced, both screw pumps are expected to perform well for at least five years. Screw pump no. 3 is performing well.

Primary clarifier no. 2 was taken out of service for inspection and cleaning. It was found to be in good condition, including the sludge collector mechanism. There are still plans to take aeration tank no. 3 out of service for inspection and potential replacement of a diffuser membrane(s).

The WWTP was still performing well, with a SRT of approximately 13 days.

Applicability of Recommendations to Operation of the Piqua WWTP

The operational recommendations contained within this Technical Memorandum No. 1 can improve the performance of the Piqua WWTP if implemented in a timely fashion. The WWTP as currently laid out will continue in operation for 5 years or more before improvements recommended in the Amended Facility Plan are constructed, so performance optimization could be advantageous from a cost standpoint and for compliance with environmental regulations. Therefore, *short-term* improvement or optimization of WWTP performance can be realized if the following recommendations are implemented:

- Headworks: Evaluate and follow through with repairs that may be necessary to keep the equipment in successful operation until a WWTP upgrade is completed. The condition of the motors and gear reducers should be checked routinely for excess temperature and noise. (As of the completion of this Technical Memorandum, the gear reducers for screw pumps no. 1 and 2 were being repaired for ongoing service.)

- Improve the oxygen transfer through the aeration process within the Supernatant Oxidation Tank to promote a more consistent supernatant feed back to the main WWTP. Routinely monitor the dissolved oxygen (DO) levels, especially when the belt filter press is in service.

- Aeration tanks:
    - Experiment with the IR mixer orientation to determine best position for anoxic conditions in cell 1 of each tank.
    - Install permanent air flow (SCFM) meters on the droplegs and DO analyzers that transmit DO concentration to the SCADA system for DO trending and optimization of the anoxic zone.
- Secondary clarifiers:
    - Repair the isolation gates in the flow distribution box upstream of the clarifiers.
    - Take each clarifier out of service individually for internal inspection, repair if necessary, and cleaning.
- Take the anaerobic digesters out of service one-at-a-time for inspection, repair if necessary, and cleaning.
- Take the digested sludge storage tank out of service temporarily for inspection, repair if necessary, and cleaning.

Long-term optimization could mean the completion of permanent improvements, which are currently proposed as a new WWTP to replace the existing facilities. Construction completion for the new WWTP is scheduled for no later than February 2020, according to the most recent (proposed) modification to the City's NPDES Permit Compliance Schedule. This Amended Facility Plan and Preliminary Engineering Report project is based on that milestone. Because of the significant cost and investment in the proposed WWTP, the *long-term* application of the recommendations contained herein may be tempered by the cost-effectiveness of the recommendations and the ability to afford them along with the permanent improvements represented by the new WWTP. Following are long-term recommendations:

- Eliminate the hydraulic bottleneck downstream of the primary clarifiers. This could be accomplished through design and construction of a new primary effluent pump station and flow distribution box. This goal should be implemented only if a future WWTP improvement or expansion will include continued utilization of the existing primary clarifiers. If this is not the case, the hydraulic bottleneck associated with the primary clarifiers will be eliminated when they are no longer in service.
- Add metering capability for the primary sludge flow. The need and timing of this recommendation is in the same context as the elimination of the hydraulic bottleneck mentioned above.

- Install a more efficient sludge dewatering device, such as a new belt filter press or centrifuge. Though the City benefits in its sludge hauling contract by paying for the total dry tonnage of biosolids hauled, improved dewatering and less sludge volume could be realized.

- Stormwater management:

    - Replace the pumps at the drainage pump station with pumps sized to convey tank drainage and stormwater flow anticipated throughout the WWTP.

    - Develop a stormwater management system that will prevent stormwater from being treated with the sanitary wastewater.

Application No. OH0027049

Modification Issue Date: May 30, 2014

Modification Effective Date: August 1, 2014

Expiration Date: January 31, 2016

Ohio Environmental Protection Agency
Authorization to Discharge Under the
National Pollutant Discharge Elimination System

In compliance with the provisions of the Federal Water Pollution Control Act, as amended (33 U.S.C. 1251 et. seq., hereinafter referred to as the "Act"), and the Ohio Water Pollution Control Act (Ohio Revised Code Section 6111),

City of Piqua

is authorized by the Ohio Environmental Protection Agency, hereinafter referred to as "Ohio EPA," to discharge from the City of Piqua wastewater treatment works located at 121 Bridge Street, Piqua, Ohio, Miami County and discharging to the Great Miami River in accordance with the conditions specified in Parts I, II, and III of this permit.

This modified permit is conditioned upon payment of applicable fees as required by Section 3745.11 of the Ohio Revised Code.

This modified permit and the authorization to discharge shall expire at midnight on the expiration date shown above. In order to receive authorization to discharge beyond the above date of expiration, the permittee shall submit such information and forms as are required by the Ohio EPA no later than 180 days prior to the above date of expiration.

_____
Craig W. Butler
Director

Total Pages: 43

Part I, A. - FINAL EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

1. During the period beginning from the effective date of the modified permit and lasting until the expiration date of the modified permit, the permittee is authorized to discharge in accordance with the following limitations and monitoring requirements from the following outfall: 1PD000008001. See Part II, OTHER REQUIREMENTS, for locations of effluent sampling.

Table - Final Outfall - 001 - Final

Effluent Characteristic	Discharge Limitations							Monitoring Requirements		
	Concentration Specified Units				Loading* kg/day					
Parameter	Maximum	Minimum	Weekly	Monthly	Daily	Weekly	Monthly	Measuring Frequency	Sampling Type	Monitoring Months
00010 - Water Temperature - C	-	-	-	-	-	-	-	1/Day	Maximum Indicating Thermometer	All
00300 - Dissolved Oxygen - mg/l	-	5.0	-	-	-	-	-	1/Day	Multiple Grab	All
00530 - Total Suspended Solids - mg/l	-	-	30	20	-	512	341	3/Week	24hr Composite	Summer
00530 - Total Suspended Solids - mg/l	-	-	45	30	-	768	512	3/Week	24hr Composite	Winter
00552 - Oil and Grease, Hexane Extr Method - mg/l	10	-	-	-	-	-	-	1/Month	Grab	All
00610 - Nitrogen, Ammonia (NH3) - mg/l	-	-	13.5	9.0	-	230	153	3/Week	24hr Composite	Fall/Spring
00610 - Nitrogen, Ammonia (NH3) - mg/l	-	-	22.5	15	-	383	255	3/Week	24hr Composite	Dec. - Feb.
00610 - Nitrogen, Ammonia (NH3) - mg/l	-	-	4.4	2.9	-	74.9	49.4	3/Week	24hr Composite	Summer
00625 - Nitrogen Kjeldahl, Total - mg/l	-	-	-	-	-	-	-	1/Month	24hr Composite	All
00630 - Nitrite Plus Nitrate, Total - mg/l	-	-	-	-	-	-	-	1/Month	24hr Composite	All
00665 - Phosphorus, Total (P) - mg/l	-	-	-	-	-	-	-	1/Week	24hr Composite	All
00719 - Cyanide, Free - mg/l	-	-	-	-	-	-	-	1/Quarter	Grab	Quarterly
01009 - Barium, Total Recoverable - ug/l	-	-	-	-	-	-	-	1/Quarter	24hr Composite	Quarterly
01074 - Nickel, Total Recoverable - ug/l	-	-	-	-	-	-	-	1/Quarter	24hr Composite	Quarterly
01079 - Silver, Total Recoverable - ug/l	-	-	-	-	-	-	-	1/Quarter	24hr Composite	Quarterly
01094 - Zinc, Total Recoverable - ug/l	-	-	-	-	-	-	-	1/Quarter	24hr Composite	Quarterly
01113 - Cadmium, Total Recoverable - ug/l	-	-	-	-	-	-	-	1/Quarter	24hr Composite	Quarterly
01114 - Lead, Total Recoverable - ug/l	-	-	-	-	-	-	-	1/Quarter	24hr Composite	Quarterly

Effluent Characteristic	Discharge Limitations							Monitoring Requirements		
	Concentration Specified Units				Loading* kg/day					
Parameter	Maximum	Minimum	Weekly	Monthly	Daily	Weekly	Monthly	Measuring Frequency	Sampling Type	Monitoring Months
01118 - Chromium, Total Recoverable - ug/l	-	-	-	-	-	-	-	1/Quarter	24hr Composite	Quarterly
01119 - Copper, Total Recoverable - ug/l	-	-	-	-	-	-	-	1/Month	24hr Composite	All
01220 - Chromium, Dissolved Hexavalent - ug/l	-	-	-	-	-	-	-	1/Quarter	Grab	Quarterly
31648 - E. coli - #/100 ml	-	-	284	126	-	-	-	3/Week	Grab	Summer
50050 - Flow Rate - MGD	-	-	-	-	-	-	-	1/Day	Continuous	All
50060 - Chlorine, Total Residual - mg/l	0.035	-	-	-	-	-	-	1/Day	Multiple Grab	Summer
50092 - Mercury, Total (Low Level) - ng/l	-	-	-	-	-	-	-	1/Month	Grab	All
61425 - Acute Toxicity, Ceriodaphnia dubia - TUa	-	-	-	-	-	-	-	1/Year	24hr Composite	September
61426 - Chronic Toxicity, Ceriodaphnia dubia - TUc	-	-	-	-	-	-	-	1/Year	24hr Composite	September
61427 - Acute Toxicity, Pimephales promelas - TUa	-	-	-	-	-	-	-	1/Year	24hr Composite	September
61428 - Chronic Toxicity, Pimephales promelas - TUc	-	-	-	-	-	-	-	1/Year	24hr Composite	September
61941 - pH, Maximum - S.U.	9.0	-	-	-	-	-	-	1/Day	Multiple Grab	All
61942 - pH, Minimum - S.U.	-	6.5	-	-	-	-	-	1/Day	Multiple Grab	All
70300 - Residue, Total Filterable - mg/l	-	-	-	-	-	-	-	1/Quarter	24hr Composite	Quarterly
80082 - CBOD 5 day - mg/l	-	-	40	23	-	682	392	3/Week	24hr Composite	Winter
80082 - CBOD 5 day - mg/l	-	-	23	15	-	392	256	3/Week	24hr Composite	Summer

Notes for station 1PD00008001:

* Effluent loadings based on average design flow of 4.5 MGD.

- Total residual chlorine - See Part II, Item J.

- Nickel, silver, zinc, cadmium, lead, total chromium, and copper - See Part II, Item M.

- Dissolved hexavalent chromium - See Part II, Item N.

- Mercury - See Part II, Items N and U.

- Free cyanide - See Part II, Items N and T.

- Dissolved oxygen and pH (minimum) - Report critical low value.

- pH (maximum) - Report critical maximum value.

- Whole effluent toxicity - See Part II, Item W.

Part I, B. - DOWNSTREAM-NEARFIELD MONITORING REQUIREMENTS

1. Downstream-Nearfield Monitoring. During the period beginning on the effective date of the modified permit and lasting until the expiration date of the modified permit, the permittee shall monitor the receiving stream, downstream of the point of discharge, at Station Number 1PD00008901, and report to the Ohio EPA in accordance with the following table. See Part II, OTHER REQUIREMENTS, for location of sampling.

Table - Downstream-Nearfield Monitoring - Final

Effluent Characteristic	Discharge Limitations							Monitoring Requirements		
	Concentration Specified Units				Loading* kg/day					
Parameter	Maximum	Minimum	Weekly	Monthly	Daily	Weekly	Monthly	Measuring Frequency	Sampling Type	Monitoring Months
00010 - Water Temperature - C	-	-	-	-	-	-	-	1/Month	Grab	All
00300 - Dissolved Oxygen - mg/l	-	-	-	-	-	-	-	1/Month	Grab	All
00400 - pH - S.U.	-	-	-	-	-	-	-	1/Month	Grab	All
00610 - Nitrogen, Ammonia (NH3) - mg/l	-	-	-	-	-	-	-	1/Month	Grab	All
00630 - Nitrite Plus Nitrate, Total - mg/l	-	-	-	-	-	-	-	1/Month	Grab	All
00665 - Phosphorus, Total (P) - mg/l	-	-	-	-	-	-	-	1/Month	Grab	All
00900 - Hardness, Total (CaCO3) - mg/l	-	-	-	-	-	-	-	1/Month	Grab	All
01119 - Copper, Total Recoverable - ug/l	-	-	-	-	-	-	-	1/Quarter	Grab	Quarterly
31648 - E. coli - #/100 ml	-	-	-	-	-	-	-	1/Month	Grab	Summer

NOTES for Station Number 1PD00008901:
- Nitrite plus nitrate, phosphorus and copper - See Part II, Item M.

Part I, B. - SSO MONITORING EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

2. SSO Monitoring. During the period beginning on the effective date of the modified permit and lasting until the expiration date of the modified permit, the permittee shall monitor at Station Number 1PD00008300, and report to the Ohio EPA in accordance with the following table. See Part II, OTHER REQUIREMENTS, for location of sampling.

Table - SSO Monitoring - 300 - Final

Effluent Characteristic	Discharge Limitations							Monitoring Requirements		
	Concentration Specified Units				Loading* kg/day					
Parameter	Maximum	Minimum	Weekly	Monthly	Daily	Weekly	Monthly	Measuring Frequency	Sampling Type	Monitoring Months
74062 - Overflow Occurrence - No./Month	-	-	-	-	-	-	-	1/Month	Total	All

NOTES for Station Number 1PD00008300:

- A sanitary sewer overflow is an overflow, spill, release, or diversion of wastewater from a sanitary sewer system. These overflows shall be monitored when they discharge. Only sanitary sewer overflows that enter waters of the state, either directly or through a storm sewer or other conveyance, must be reported under this monitoring station.

- For the purpose of counting occurrences, each location on the sanitary sewer system where there is an overflow, spill, release, or diversion of wastewater on a given day that enters waters of the state is counted as one occurrence. For example, if on a given day overflows occur from a manhole at one location and from a damaged pipe at another location and they both enter waters of the state, record two occurrences for that day. If overflows from both locations continue on the following day, record two occurrences for the following day. At the end of the month, total the daily occurrences and report this number in the first column of the first day of the month on the 4500 form. If there are no overflows during the entire month, report "zero" (0).

- All sanitary sewer overflows are prohibited.

- See Part II, Items D and E.

Part I, B. - SLUDGE MONITORING REQUIREMENTS

3. Sludge Monitoring. During the period beginning on the effective date of the modified permit and lasting until the expiration date of the modified permit, the permittee shall monitor the treatment works' final sludge at Station Number 1PD00008581, and report to the Ohio EPA in accordance with the following table. See Part II, OTHER REQUIREMENTS, for location of sludge sampling.

Table - Sludge Monitoring - 581 - Final

Effluent Characteristic	Discharge Limitations							Monitoring Requirements		
	Concentration Specified Units				Loading* kg/day					
Parameter	Maximum	Minimum	Weekly	Monthly	Daily	Weekly	Monthly	Measuring Frequency	Sampling Type	Monitoring Months
00611 - Ammonia (NH3) In Sludge - mg/kg	-	-	-	-	-	-	-	2/Year	Composite	Semi-annual
00627 - Nitrogen Kjeldahl, Total In Sludge - mg/kg	-	-	-	-	-	-	-	2/Year	Composite	Semi-annual
00668 - Phosphorus, Total In Sludge - mg/kg	-	-	-	-	-	-	-	2/Year	Composite	Semi-annual
00938 - Potassium In Sludge - mg/kg	-	-	-	-	-	-	-	2/Year	Composite	Semi-annual
01003 - Arsenic, Total In Sludge - mg/kg	75	-	-	-	-	-	-	2/Year	Composite	Semi-annual
01028 - Cadmium, Total In Sludge - mg/kg	85	-	-	-	-	-	-	2/Year	Composite	Semi-annual
01043 - Copper, Total In Sludge - mg/kg	4300	-	-	-	-	-	-	2/Year	Composite	Semi-annual
01052 - Lead, Total In Sludge - mg/kg	840	-	-	-	-	-	-	2/Year	Composite	Semi-annual
01068 - Nickel, Total In Sludge - mg/kg	420	-	-	-	-	-	-	2/Year	Composite	Semi-annual
01093 - Zinc, Total In Sludge - mg/kg	7500	-	-	-	-	-	-	2/Year	Composite	Semi-annual
01148 - Selenium, Total In Sludge - mg/kg	100	-	-	-	-	-	-	2/Year	Composite	Semi-annual
51129 - Sludge Fee Weight - dry tons	-	-	-	-	-	-	-	2/Year	Total	Semi-annual
70316 - Sludge Weight - Dry Tons	-	-	-	-	-	-	-	2/Year	Total	Semi-annual
71921 - Mercury, Total In Sludge - mg/kg	57	-	-	-	-	-	-	2/Year	Composite	Semi-annual
78465 - Molybdenum In Sludge - mg/kg	75	-	-	-	-	-	-	2/Year	Composite	Semi-annual

NOTES for Station Number 1PD00008581:
- Monitoring is required when sewage sludge is removed from the permittee's facility for application to the land. The monitoring data shall be reported on the June and December Discharge Monitoring Report (DMR). The monitoring data can be collected at any time during the reporting period.

- Metal pollutant analysis must be completed during each reporting period, whether sewage sludge is removed from the facility or not, or the number of composite samples collected and reported shall be increased prior to the next land application event to account for the reporting period(s) in which land application did not occur, unless all previously accumulated sewage sludge has been removed and disposed of via a landfill, through incineration or by transfer to another treatment works.

- If no sewage sludge is removed from the facility during the reporting period, enter the results for the metal analysis in eDMR or on the 4500 report and enter "0" for sludge weight and sludge fee weight.

- If no sewage sludge is removed from the facility during the reporting period and no metal analysis is completed during the reporting period, the permittee shall report under station 581 in the following manner: select the "No Discharge" check box on the data entry form. PIN the eDMR.

- If metal analysis has not been completed previously during each reporting period: when sewage sludge is removed from the facility all metal analysis results shall be reported on the applicable DMR by entering the separate results on different days within the DMR. For example, if no sewage sludge has been removed from the facility for a full calendar year, and quarterly monitoring is required by the permit, then five (four from the previous year and one for the current monitoring period) separate composite samples of the sewage sludge are required to be collected and analyzed for metals prior to removal from the facility. The first sample result may be entered on the first day of the DMR, the second result on the second day of the DMR, and so on. A note may then be added to indicate the actual day(s) when the samples were collected.

- It is recommended that composite samples of the sewage sludge be collected and analyzed close enough to the time of land application to be reflective of the sludge's current quality, but not so close that the results of the analysis are not available prior to land applying the sludge.

- The permittee shall maintain the appropriate records on site to verify that the requirements of Pathogen Reduction and Vector Attraction Reduction have been met.

- Units of mg/kg are on a dry weight basis.

- Sludge weight is a calculated total for the year. To convert from gallons of liquid sewage sludge to dry tons of sewage sludge: dry tons= gallons x 8.34 (lbs/gallon) x 0.0005 (tons/lb) x decimal fraction total solids.

- Sludge fee weight means sludge weight, in dry U.S. tons, excluding any admixtures such as liming material or bulking agents.

- See Part II, Items P, Q, R, and S.

Part I, B. - SLUDGE MONITORING REQUIREMENTS

4. Sludge Monitoring. During the period beginning on the effective date of the modified permit and lasting until the expiration date of the modified permit, the permittee shall monitor the treatment works' final sludge at Station Number 1PD00008586, and report to the Ohio EPA in accordance with the following table. See Part II, OTHER REQUIREMENTS, for location of sludge sampling.

Table - Sludge Monitoring - 586 - Final

Effluent Characteristic	Discharge Limitations							Monitoring Requirements		
	Concentration Specified Units				Loading* kg/day					
Parameter	Maximum	Minimum	Weekly	Monthly	Daily	Weekly	Monthly	Measuring Frequency	Sampling Type	Monitoring Months
51129 - Sludge Fee Weight - dry tons	-	-	-	-	-	-	-	1/Year	Total	December

NOTES for Station Number 1PD00008586-

- Monitoring is required when sewage sludge is removed from the permittee's facility for disposal in a mixed solid waste landfill. The total Sludge Fee Weight of sewage sludge disposed of in a mixed solid waste landfill for the entire year shall be reported on the December Discharge Monitoring Report (DMR).

- If no sewage sludge is removed from the Permittee's facility for disposal in a mixed solid waste landfill during the year select the "No Discharge" check box on the data entry form. PIN the eDMR.

- Sludge fee weight means sludge weight, in dry U.S. tons, excluding any admixtures such as liming material or bulking agents.

- See Part II, Items P, R, and S.

Part I, B. - INFLUENT MONITORING REQUIREMENTS

5. Influent Monitoring. During the period beginning on the effective date of the modified permit and lasting until the expiration date of the modified permit, the permittee shall monitor the treatment works' influent wastewater at Station Number 1PD00008601, and report to the Ohio EPA in accordance with the following table. Samples of influent used for determination of net values or percent removal must be taken the same day as those samples of effluent used for that determination. See Part II, OTHER REQUIREMENTS, for location of influent sampling.

Table - Influent Monitoring - 601 - Final

Effluent Characteristic	Discharge Limitations							Monitoring Requirements		
	Concentration Specified Units				Loading* kg/day					
Parameter	Maximum	Minimum	Weekly	Monthly	Daily	Weekly	Monthly	Measuring Frequency	Sampling Type	Monitoring Months
00400 - pH - S.U.	-	-	-	-	-	-	-	1/Day	Grab	All
00530 - Total Suspended Solids - mg/l	-	-	-	-	-	-	-	3/Week	24hr Composite	All
00719 - Cyanide, Free - mg/l	-	-	-	-	-	-	-	1/Quarter	Grab	Quarterly
01074 - Nickel, Total Recoverable - ug/l	-	-	-	-	-	-	-	1/Quarter	24hr Composite	Quarterly
01079 - Silver, Total Recoverable - ug/l	-	-	-	-	-	-	-	1/Quarter	24hr Composite	Quarterly
01094 - Zinc, Total Recoverable - ug/l	-	-	-	-	-	-	-	1/Quarter	24hr Composite	Quarterly
01113 - Cadmium, Total Recoverable - ug/l	-	-	-	-	-	-	-	1/Quarter	24hr Composite	Quarterly
01114 - Lead, Total Recoverable - ug/l	-	-	-	-	-	-	-	1/Quarter	24hr Composite	Quarterly
01118 - Chromium, Total Recoverable - ug/l	-	-	-	-	-	-	-	1/Quarter	24hr Composite	Quarterly
01119 - Copper, Total Recoverable - ug/l	-	-	-	-	-	-	-	1/Month	24hr Composite	All
01220 - Chromium, Dissolved Hexavalent - ug/l	-	-	-	-	-	-	-	1/Quarter	Grab	Quarterly
50092 - Mercury, Total (Low Level) - ng/l	-	-	-	-	-	-	-	1/Month	Grab	All
80082 - CBOD 5 day - mg/l	-	-	-	-	-	-	-	3/Week	24hr Composite	All

NOTES for Station Number 1PD00008601:

- Nickel, silver, zinc, cadmium, lead, total chromium and copper - See Part II, Item N.
- Dissolved hexavalent chromium - See Part II, Item O.
- Free cyanide - See Part II, Item O and T.
- Mercury - See Part II, Items O and U.

Part I, B. - UPSTREAM MONITORING REQUIREMENTS

6. Upstream Monitoring. During the period beginning on the effective date of the modified permit and lasting until the expiration date of the modified permit, the permittee shall monitor the receiving stream, upstream of the point of discharge at Station Number 1PD00008801, and report to the Ohio EPA in accordance with the following table. See Part II, OTHER REQUIREMENTS, for location of sampling.

Table - Upstream Monitoring - 801 - Final

Effluent Characteristic	Discharge Limitations							Monitoring Requirements		
	Concentration Specified Units				Loading* kg/day					
Parameter	Maximum	Minimum	Weekly	Monthly	Daily	Weekly	Monthly	Measuring Frequency	Sampling Type	Monitoring Months
00010 - Water Temperature - C	-	-	-	-	-	-	-	1/Month	Grab	All
00300 - Dissolved Oxygen - mg/l	-	-	-	-	-	-	-	1/Month	Grab	All
00400 - pH - S.U.	-	-	-	-	-	-	-	1/Month	Grab	All
00610 - Nitrogen, Ammonia (NH3) - mg/l	-	-	-	-	-	-	-	1/Month	Grab	All
00630 - Nitrite Plus Nitrate, Total - mg/l	-	-	-	-	-	-	-	1/Month	Grab	All
00665 - Phosphorus, Total (P) - mg/l	-	-	-	-	-	-	-	1/Month	Grab	All
31648 - E. coli - #/100 ml	-	-	-	-	-	-	-	1/Month	Grab	Summer
61432 - 48-Hr. Acute Toxicity Ceriodaphnia dubia - % Affected	-	-	-	-	-	-	-	1/Year	Grab	September
61435 - 96-Hr. Acute Toxicity Pimephales promela - % Affected	-	-	-	-	-	-	-	1/Year	Grab	September
61438 - 7-Day Chronic Toxicity Ceriodaphnia dubia - % Affected	-	-	-	-	-	-	-	1/Year	Grab	September
61441 - 7-Day Chronic Toxicity Pimephales promelas - % Affected	-	-	-	-	-	-	-	1/Year	Grab	September

NOTES for Station Number 1PD00008801:
- Whole effluent toxicity - See Part II, Item W.

Part I, C - Schedule of Compliance

1. E. coli and Summer Ammonia Limits Schedule

The permittee shall achieve compliance with the final effluent limits for Escherichia coli and ammonia-nitrogen during the months of June - September as soon as possible, but not later than the dates developed in accordance with the following schedule:

a. The permittee shall evaluate the ability of its existing treatment facilities to meet the final effluent limits for E.coli and ammonia-N (June - September) at outfall 1PD00008001. (ITEM COMPLETED)

b. Not later than 6 months from the effective date of this permit, the permittee shall submit to the Ohio EPA Southwest District Office a brief status report on the ability of its existing treatment facilities to meet the final effluent limits for E.coli and ammonia-N (June - September) or on plant improvements necessary to meet the final effluent limits. (ITEM COMPLETED)

c. If the permittee determines that its existing treatment facilities are not capable of meeting the final effluent limits for E. coli and ammonia-N (June - September), not later than 6 months from the effective date of this permit, the permittee shall submit an approvable Permit To Install, if necessary, for plant improvements necessary to meet the final effluent limits. (ITEM COMPLETED)

d. Not later than 9 months from the effective date of this permit, the permittee shall commence construction, if necessary, for plant improvements to meet the final effluent limits for e.coli and ammonia-N (June - September). (ITEM COMPLETED)

e. Not later than 12 months from the effective date of this permit, the permittee shall achieve the final effluent limits for E. coli and ammonia-N (June - September) at outfall 1PD0008001. (ITEM COMPLETED)

f. The permittee shall notify the Ohio EPA Southwest District Office in writing within 7-days of achieving compliance with the final effluent limits for E. coli and ammonia-N (June - September). (ITEM COMPLETED)

2. Municipal Sanitary Sewer Overflow (SSO) Schedule

Sanitary sewer overflows on the permittee's collection system are not authorized by this permit, including the provisions in this schedule of compliance.

The permittee shall complete the actions described below as soon as possible, but not later than the dates included in the following schedule:

a. The permittee shall evaluate the impacts that construction of the equalization basin and other improvements have had on the West Interceptor Sewer SSO located just upstream of the wastewater plant. From February through October 2011 the City shall complete flow monitoring, update its interceptor model, recalibrate the interceptor model and evaluate alternatives to eliminate the SSO. (ITEM COMPLETED)

b. The permittee shall expand its interceptor-only model to include its major trunk sewers. From February through August 2011, the City shall complete the necessary flow monitoring, update its interceptor model and complete model calibration. (ITEM COMPLETED)

c. Not later than June 30 2012, the permittee shall submit two copies of a collection system master plan to the Ohio EPA Southwest District Office. The master plan shall include a prioritized list of projects that the City must complete to eliminate the West Interceptor Sewer sanitary sewer overflow. (ITEM COMPLETED)

d. Not later than December 31, 2014, the permittee shall submit a Amended Facility Plan for the wastewater treatment plant (WWTP) to eliminate the SSO. This Amended Facility Plan is to incorporate any findings and/or determinations not available at the time of the previously submitted Facility Plan for the WWTP.

e. Not later than October 1, 2017, the permittee shall submit an approvable Permit-to-Install application(s) and detailed plans, if necessary, for the projects to eliminate the SSO.

f. Not later than March 31, 2018, the permittee shall begin construction, if necessary, of projects to eliminate the SSO.

g. Not later than February 28, 2020, the permittee shall complete all work identified as necessary to eliminate the West Interceptor Sewer SSO.

h. The permittee shall notify the Ohio EPA Southwest District Office within 7 days of completing all work identified as necessary to eliminate the sanitary sewer overflow.

i. Beginning on June 1, 2013 and annually thereafter, the permittee shall submit to the Ohio EPA Southwest District Office a written status report on all work completed during the previous 12 months to eliminate the West Interceptor Sewer SSO. (Event Code 03599)

This NPDES permit, Ohio EPA permit number 1PD00008*TD, will expire before the compliance schedule is completed. This Schedule of Compliance includes items that extends beyond the term of the permit. The requirements of Schedule of Compliance items 2(d), 2(e). and 2(f). including the compliance dates, will be included in permit 1PD00008*UD when it is renewed.

3. Municipal Pretreatment Schedule

a. The permittee shall evaluate the adequacy of local industrial user limitations to attain compliance with final table limits. A technical justification for revising local industrial user limitations to attain compliance with final table limits, along with a pretreatment program modification request, or technical justification for retaining existing local industrial user limitations shall be submit to Ohio EPA, Central Office Pretreatment Unit, in duplicate,as soon as possible, but no later than 6 months after the effective date of this permit. (ITEM COMPLETED)

Technical justification is required for arsenic, cadmium, total chromium, dissolved hexavalent chromium, copper, free cyanide, lead, molybdenum, nickel, selenium, silver and zinc unless screening of wastewater and sludge indicate these pollutants are not present in significant amounts. Furthermore, technical justification is required for any other pollutants where a local limit may be necessary to protect against pass through and interference.

b. To demonstrate technical justification for new local industrial user limits or justification for retaining existing limits, the following information must be submitted to Ohio EPA:

i. Treatment plant flow, domestic/background concentrations, and industrial flows to which local limits will be applied.

ii. Treatment plant removal efficiencies.

iii. A comparison of maximum allowable headworks loadings based on all applicable criteria. Criteria may include sludge disposal, NPDES permit limits, waste load allocation values, and interference with biological processes such as activated sludge, sludge digestion, nitrification, etc.

iv. If revised industrial user discharge limits are proposed, the method of allocating available pollutant loads to industrial users.

v. Supporting data, assumptions, and methodologies used in establishing the information in item a.i through iv above.

b. If revisions to local industrial user limitations including best management practices are determined to be necessary, no later than 2 months after the date of Ohio EPA's approval, the permittee shall incorporate revised local industrial user limitations in all industrial user control documents.

c. The permittee shall evaluate the adequacy of local industrial user limitations for mercury. A technical justification for revising local industrial user limitations, along with a pretreatment program modification request, or technical justification for retaining existing local industrial user limitations shall be submitted to Ohio EPA, Central Office Pretreatment Unit, in duplicate, as soon as possible, but no later than 6 months from the effective date of this permit. (ITEM COMPLETED)

To demonstrate technical justification for new local industrial user limits or justification for retaining existing limits, the following information must be submitted to Ohio EPA:

i. Treatment plant flow, domestic/background concentrations, and industrial flows to which local limits will be applied. When representative sampling of the collection system and industrial pollutant contributors conducted using EPA Method 245.1 or 245.2 shows mercury concentrations that are below detection, EPA Method 1631 or 245.7 shall be used to quantify domestic/background and industrial pollutant contributions of mercury.

ii. Treatment plant removal efficiencies. When representative sampling of the influent and effluent conducted using EPA Method 245.1 or 245.2 shows mercury concentrations that are below detection, EPA Method 1631 or 245.7 shall be used to quantify influent and effluent mercury concentrations.

iii. A comparison of maximum allowable headworks loadings based on all applicable criteria. Criteria may include sludge disposal, NPDES permit limits, waste load allocation values, and interference with biological processes such as activated sludge, sludge digestion, nitrification, etc.

iv. If industrial user discharge limits are proposed, the method of allocating available pollutant loads to industrial users. When appropriate, industrial user discharge limits may include narrative local limits requiring industrial users to develop and implement best management practices for mercury. These narrative local limits may be used either alone or as a supplement to a numeric limit.

v. Supporting data, assumptions, and methodologies used in establishing the information in Item c.i. through iv above.

d. If revisions to local industrial user limitations for mercury are required, no later than 2 months after the date of Ohio EPA's approval, the permittee shall incorporate revised local industrial user limitations in all industrial user control documents.

Part II, Other Requirements

A. Operator Certification Requirements

1. Classification

a. In accordance with Ohio Administrative Code 3745-7-04, the sewage treatment facility at this facility shall be classified as a Class III facility.

b. All sewerage (collection) systems that are tributary to this treatment works are Class II sewerage systems in accordance with paragraph (B)(1)(a) of rule 3745-7-04 of the Ohio Administrative Code.

2. Operator of Record

a. The permittee shall designate one or more operator of record to oversee the technical operation of the treatment works and sewerage (collection) system in accordance with paragraph (A)(2) of rule 3745-7-02 of the Ohio Administrative Code.

b. Each operator of record shall have a valid certification of a class equal to or greater than the classification of the treatment works as defined in Part II, Item A.1 of this NPDES permit.

c. Within three days of a change in an operator of record, the permittee shall notify the Director of the Ohio EPA of any such change on a form acceptable to Ohio EPA. The appropriate form can be found at the following website:

http://www.epa.ohio.gov/portals/28/Documents/opcert/Operator%20of%20Record%20Notification%20Form.pdf

d. Within 60 days of the effective date of this modified permit, the permittee shall notify the Director of Ohio EPA of the operators of record on a form acceptable to Ohio EPA.

e. The operator of record for a class II, III, or IV treatment works or class II sewerage system may be replaced by a backup operator with a certificate one classification lower than the treatment works or sewerage system for a period of up to thirty consecutive days. The use of this provision does not require notification to the agency.

f. Upon proper justification, such as military leave or long term illness, the director may authorize the replacement of the operator of record for a class II, III, or IV treatment works or class II sewerage system by a backup operator with a certificate one classification lower than the facility for a period of greater than thirty consecutive days. Such requests shall be made in writing to the appropriate district office.

3. Minimum Staffing Requirements

a. The permittee shall ensure that the treatment works operator of record is physically present at the facility in accordance with the minimum staffing requirements per paragraph (C)(1) of rule 3745-7-04 of the Ohio Administrative Code or the requirements from an approved 3745-7-04(C) minimum staffing hour reduction plan.

b. Sewerage (collection) system Operators of Record are not required to meet minimum staffing requirements in paragraph (C)(1) of rule 3745-7-04 of the Ohio Administrative Code.

c. If Ohio EPA approves a reduction in minimum staffing requirements based upon a facility operating plan, any change in the criteria under which the operating plan was approved (such as enforcement status, history of noncompliance, or provisions included in the plan) will require that the treatment works immediately return to the minimum staffing requirements included in paragraph (C)(1) of rule 3745-7-04 of the Ohio Administrative Code.

B. Description of the location of the required sampling stations are as follows:

Sampling Station	Description of Location
1PD00008001	Final effluent prior to discharge to the Great Miami River
.	(Lat: 40 N 07' 49"; Long: 84 W 14' 06")
1PD00008581	Sludge disposal by land application at agronomic rates
1PD00008586	Sludge disposal by hauling to mixed solid waste landfill
1PD00008300	System-wide sanitary sewer overflow occurrences
1PD00008601	Plant influent
1PD00008801	Upstream of wastewater plant at the Main Street bridge in
.	the Great Miami River
1PD00008901	Downstream of wastewater plant at the Farrington Road bridge
.	in the Great Miami River

C. All parameters, except flow, need not be monitored on days when the plant is not normally staffed (Saturdays, Sundays, and Holidays). On those days, report "AN" on the monthly report form.

D. Sanitary Sewer Overflow (SSO) Reporting Requirements

A sanitary sewer overflow is an overflow, spill, release, or diversion of wastewater from a sanitary sewer system. SSOs do not include wet weather discharges from combined sewer overflows specifically listed in Part II of this NPDES permit (if any). All SSOs are prohibited.

1. Reporting for SSOs That Imminently and Substantially Endanger Human Health

a) Immediate Notification

You must notify Ohio EPA (1-800-282-9378) and the appropriate Board of Health (i.e., city or county) within 24 hours of learning of any SSO from your sewers or from your maintenance contract areas that may imminently and substantially endanger human health. The telephone report must identify the location, estimated volume and receiving water, if any, of the overflow. An SSO that may imminently and substantially endanger human health includes dry weather overflows, major line breaks, overflow events that result in fish kills or other significant harm, overflows that expose the general public to contact with raw sewage, and overflow events that occur in sensitive waters and high exposure areas such as protection areas for public drinking water intakes and waters where primary contact recreation occurs.

b) Follow-Up Written Report

Within 5 days of the time you become aware of any SSO that may imminently and substantially endanger human health, you must provide the appropriate Ohio EPA district office a written report that includes:

(i) the estimated date and time when the overflow began and stopped or will be stopped (if known);
(ii) the location of the SSO including an identification number or designation if one exists;
(iii) the receiving water (if there is one);
(iv) an estimate of the volume of the SSO (if known);
(v) a description of the sewer system component from which the release occurred (e.g., manhole, constructed overflow pipe, crack in pipe);
(vi) the cause or suspected cause of the overflow;
(vii) steps taken or planned to reduce, eliminate, and prevent reoccurrence of the overflow and a schedule of major milestones for those steps; and
(viii) steps taken or planned to mitigate the impact(s) of the overflow and a schedule of major milestones for those steps.

An acceptable 5-day follow-up written report can be filled-in or downloaded from the Ohio EPA Division of Surface Water Permits Program Technical Assistance Web page at http://www.epa.ohio.gov/dsw/permits/technical_assistance.aspx .

2. Reporting for All SSOs, Including Those That Imminently and Substantially Endanger Human Health

a) Monthly Operating Reports

Sanitary sewer overflows that enter waters of the state, either directly or through a storm sewer or other conveyance, shall be reported on your monthly operating reports. You must report the system-wide number of occurrences for SSOs that enter waters of the state in accordance with the requirements for station number 300. A monitoring table for this station is included in Part I, B of this NPDES permit. For the purpose of counting occurrences, each location on the sanitary sewer system where there is an overflow, spill, release, or diversion of wastewater on a given day is counted as one occurrence. For example, if on a given day overflows occur from a manhole at one location and from a damaged pipe at another location and they both enter waters of the state, you should record two occurrences for that day. If overflows from both locations continue on the following day, you should record two occurrences for the following day. At the end of the month, total the daily occurrences from all locations on your system and report this number using reporting code 74062 (Overflow Occurrence, No./Month) on the 4500 form for station number 300.

b) Annual Report

You must prepare an annual report of all SSOs in your collection system, including those that do not enter waters of the state. The annual report must be in an acceptable format (see below) and must include:

(i) A table that lists an identification number, a location description, and the receiving water (if any) for each existing SSO. If an SSO previously included in the list has been eliminated, this shall be noted. Assign each SSO location a unique identification by numbering them consecutively, beginning with 301.

(ii) A table that lists the date that an overflow occurred, the unique ID of the overflow, the name of affected receiving waters (if any), and the estimated volume of the overflow (in millions of gallons). The annual report may summarize information regarding overflows of less than approximately 1,000 gallons.

(iii) A table that summarizes the occurrence of water in basements (WIBs) by total number and by sewershed. The report shall include a narrative analysis of WIB patterns by location, frequency and cause. Only WIBs caused by a problem in the publicly-owned collection system must be included.

Not later than March 31 of each year, you must submit one copy of the annual report for the previous calendar year to the appropriate Ohio EPA district office and one copy to: Ohio EPA; Division of Surface Water; NPDES Permit Unit; P.O. Box 1049; Columbus, OH 43216-1049. You also must provide adequate notice to the public of the availability of the report.

Systems serving fewer than 10,000 people are not required to prepare an annual report if all monthly operating reports for the preceding calendar year show no discharge from overflows.

An acceptable annual SSO report can be filled-in or downloaded from the Ohio EPA Division of Surface Water Permits Program Technical Assistance Web page at http://www.epa.ohio.gov/dsw/permits/technical_assistance.aspx .

E. The permittee shall maintain in good working order and operate as efficiently as possible the "treatment works" and "sewerage system" as defined in ORC 6111.01 to achieve compliance with the terms and conditions of this permit and to prevent discharges to the waters of the state, surface of the ground, basements, homes, buildings, etc.

F. Composite samples shall be comprised of a series of grab samples collected over a 24-hour period and proportionate in volume to the sewage flow rate at the time of sampling. Such samples shall be collected at such times and locations, and in such a fashion, as to be representative of the facility's overall performance.

G. Grab samples shall be collected at such times and locations, and in such fashion, as to be representative of the facility's performance.

H. Multiple grab samples shall be comprised of at least three grab samples collected at intervals of at least three hours during the period that the plant is staffed on each day for sampling. Samples shall be collected at such times and locations, and in such fashion, as to be representative of the facility's overall performance. The critical value shall be reported.

I. The treatment works must obtain at least 85 percent removal of carbonaceous biochemical oxygen demand (five-day) and suspended solids (see Part III, Item 1).

J. The parameters below have had effluent limitations established that are below the Ohio EPA Quantification Level (OEPA QL) for the approved analytical procedure promulgated at 40 CFR 136. OEPA QLs may be expressed as Practical Quantification Levels (PQL) or Minimum Levels (ML).

Compliance with an effluent limit that is below the OEPA QL is determined in accordance with ORC Section 6111.13 and OAC Rule 3745-33-07(C). For maximum effluent limits, any value reported below the OEPA QL shall be considered in compliance with the effluent limit. For average effluent limits, compliance shall be determined by taking the arithmetic mean of values reported for a specified averaging period, using zero (0) for any value reported at a concentration less than the OEPA QL, and comparing that mean to the appropriate average effluent limit. An arithmetic mean that is less than or equal to the average effluent limit shall be considered in compliance with that limit.

The permittee must utilize the lowest available detection method currently approved under 40 CFR Part 136 for monitoring these parameters.

REPORTING:

All analytical results, even those below the OEPA QL (listed below), shall be reported. Analytical results are to be reported as follows:

1. Results above the QL: Report the analytical result for the parameter of concern.

2. Results above the MDL, but below the QL: Report the analytical result, even though it is below the QL.

3. Results below the MDL: Analytical results below the method detection limit shall be reported as "below detection" using the reporting code "AA".

The following table of quantification levels will be used to determine compliance with NPDES permit limits:

Parameter	PQL	ML
Chlorine, tot. res.	0.050 mg/l	--

This permit may be modified, or, alternatively, revoked and reissued, to include more stringent effluent limits or conditions if information generated as a result of the conditions of this permit indicate the presence of these pollutants in the discharge at levels above the water quality based effluent limit (WQBEL).

K. POTWs that accept hazardous wastes by truck, rail, or dedicated pipeline are considered to be hazardous waste treatment, storage, and disposal facilities (TSDFs) and are subject to regulation under the Resource Conservation and Recovery Act (RCRA). Under the "permit-by-rule" regulation found at 40 CFR 270.60(c), a POTW must:

1) comply with all conditions of its NPDES permit,
2) obtain a RCRA ID number and comply with certain manifest and reporting requirements under RCRA,
3) satisfy corrective action requirements, and
4) meet all federal, state, and local pretreatment requirements.

L. Final permit limitations based on preliminary or approved waste load allocations are subject to change based on modifications to or finalization of the allocation or report or changes to Water Quality Standards. Monitoring requirements and/or special conditions of this permit are subject to change based on regulatory or policy changes.

M. Sampling for these parameters at station 1PD00008001, 1PD00008601, and 1PD00008901 shall occur the same day.

N. Sampling at station 1PD000008001 for these parameters shall occur one detention time (the time it takes for a volume of water to travel through the treatment plant) after sampling at station 1PD00008601 for the same parameters on the same day.

O. Sampling at station 1PD00008601 for these parameters shall occur one detention time (the time it takes for a volume of water to travel through the treatment plant) prior to sampling at station 1PD00008001 for the same parameters on the same day.

P. All disposal, use, storage, or treatment of sewage sludge by the Permittee shall comply with Chapter 6111. of the Ohio Revised Code, Chapter 3745-40 of the Ohio Administrative Code and any future revisions thereof, any further requirements specified in this NPDES permit, and any other actions of the Director that pertain to the disposal, use, storage, or treatment of sewage sludge by the Permittee.

Q. Sewage sludge composite samples shall consist of a minimum of six grab samples collected at such times and locations, and in such fashion, as to be representative of the facility's sewage sludge.

R. No later than January 31 of each calendar year the Permittee shall submit two (2) copies of a report summarizing the sewage sludge disposal, use, storage, or treatment activities of the Permittee during the previous calendar year. One copy of the report shall be sent to the Ohio EPA, Division of Surface Water, P.O. Box 1049, Columbus, Ohio 43216-1049, and one copy of the report shall be sent to the Ohio EPA Southwest District Office. The report shall be submitted on Ohio EPA Form 4229.

S. Each day when sewage sludge is removed from the wastewater treatment plant for use or disposal, a representative sample of sewage sludge shall be collected and analyzed for percent total solids. This value of percent total solids shall be used to calculate the total Sewage Sludge Weight (Discharge Monitoring Report code 70316) and/or total Sewage Sludge Fee Weight (Discharge Monitoring Report code 51129) removed from the treatment plant on that day. The results of the daily monitoring, and the weight calculations, shall be maintained on site for a minimum of five years. The test methodology used shall be from the latest edition, Part 2540 G of Standard Methods for the Examination of Water and Wastewater American Public Health Association, American Water Works Association, and Water Environment Federation. To convert from gallons of liquid sewage sludge to dry tons of sewage sludge: dry tons = gallons x 8.34 (lbs/gallon) x 0.0005 (tons/lb) x decimal fraction total solids.

T. It is understood by Ohio EPA that at the time permit 1PD00008 becomes effective, an analytical method is not approved under 40 CFR 136 to comply with the free cyanide monitoring requirements included in the permit. The permittee shall utilize method 4500-CN I in the 18th, 19th, or 20th edition of Standard Methods.

U. The permittee shall use either EPA Method 1631 or EPA Method 245.7 promulgated under 40 CFR 136 to comply with the influent and effluent mercury monitoring requirements of this permit.

V. The permittee shall post and maintain a permanent marker on the stream bank at each outfall that is regulated under this NPDES permit and discharges to the Great Miami River. This includes final outfalls, bypasses, and combined sewer overflows. The marker shall consist at a minimum of the name of the establishment to which the permit was issued, the Ohio EPA permit number, and the outfall number and a contact telephone number. The information shall be printed in letters not less than two inches in height. The marker shall be a minimum of 2 feet by 2 feet and shall be a minimum of 3 feet above ground level. The sign shall be not be obstructed such that persons in boats or persons swimming on the river or someone fishing or walking along the shore cannot read the sign. Vegetation shall be periodically removed to keep the sign visible. If the outfall is normally submerged the sign shall indicate that. If the outfall is a combined sewer outfall, the sign shall indicate that untreated human sewage may be discharged from the outfall during wet weather and that harmful bacteria may be present in the water.

W. Biomonitoring Program Requirements

General Requirements

All toxicity testing conducted as required by this permit shall be done in accordance with "Reporting and Testing Guidance for Biomonitoring Required by the Ohio Environmental Protection Agency" (hereinafter, the "biomonitoring guidance"), Ohio EPA, July 1998 (or current revision). The Standard Operating Procedures (SOP) or verification of SOP submittal, as described in Section 1.B. of the biomonitoring guidance shall be submitted no later than three months after the effective date of this permit. If the laboratory performing the testing has modified its protocols, a new SOP is required.

Testing Requirements

1. Chronic Bioassays

The permittee shall conduct annual chronic toxicity tests using Ceriodaphnia dubia and fathead minnows (Pimephales promelas) on effluent samples from outfall 1PD00008001. These tests shall be conducted as specified in Section 3 of the biomonitoring guidance.

2. Acute Bioassays

Acute endpoints, as described in Section 2.H. of the biomonitoring guidance, shall be derived from the chronic test.

3. Testing of Ambient Water

In conjunction with the chronic toxicity tests, upstream control water shall be collected at a point outside the zone of effluent and receiving water interaction at station 1PD00008801. Testing of ambient waters shall be done in accordance with Section 3 of the biomonitoring guidance.

4. Data Review

a. Reporting

Following completion of each annual bioassay requirement, the permittee shall report results of the tests in accordance with Sections 3.H.1., and 3.H.2.a. of the biomonitoring guidance, including reporting the results on the monthly DMR and submitting a copy of the complete test report to Ohio EPA, Division of Surface Water, NPDES Permit Unit, P.O. Box 1049, Columbus, OH, 43216-1049.

Based on Ohio EPA's evaluation of the results, this permit may be modified to require additional biomonitoring, require a toxicity reduction evaluation, and/or contain whole effluent toxicity limits.

b. Definitions

TUa = Acute Toxicity Units = 100/LC50

TUc = Chronic Toxicity Units = 100/IC25

This equation for chronic toxicity units applies outside the mixing zone for warmwater, modified warmwater, exceptional warmwater, coldwater, and seasonal salmonid use designations except when the following equation is more restrictive (Ceriodaphnia dubia only):

TUc = Chronic Toxic Units = 100/square root of (NOEC x LOEC)

X. Pretreatment Program Requirements

The permittee's pretreatment program initially approved on February 8, 1985 and all subsequent modifications approved before the effective date of this permit, shall be an enforceable term and condition of this permit.

To ensure that the approved program is implemented in accordance with 40 CFR 403, Chapter 3745-3 of Ohio Administrative Code and Chapter 6111 of the Ohio Revised Code, the permittee shall comply with the following conditions:

1. Legal Authority

The permittee shall adopt and maintain legal authority which enables it to fully implement and enforce all aspects of its approved pretreatment program including the identification and characterization of industrial sources, issuance of control documents, compliance monitoring and reporting, and enforcement.

The permittee shall establish agreements with all contributing jurisdictions, as necessary, to enable the permittee to fulfill its requirements with respect to industrial users discharging to its system.

2. Industrial User Inventory

The permittee shall identify all industrial users subject to pretreatment standards and requirements and characterize the nature and volume of pollutants in their wastewater. Dischargers determined to be Significant Industrial Users according to OAC 3745-3-01(FF) must be notified of applicable pretreatment standards and requirements within 30 days of making such a determination. This inventory shall be updated at a frequency to ensure proper identification and characterization of industrial users.

3. Slug Load Control Plans for Significant Industrial Users

The permittee shall evaluate the need for a plan, device or structure to control a potential slug discharge at least once during the term of each significant industrial user¿s control mechanism. Existing significant industrial users shall be evaluated within one year of the effective date of this permit if the users have never been evaluated. New industrial users identified as significant industrial users shall be evaluated within one year of being identified as a significant industrial user.

4. Local Limits

The permittee shall develop and enforce technically based local limits to prevent the introduction of pollutants into the POTW which will interfere with the operation of the POTW, pass through the treatment works, be incompatible with the treatment works, or limit wastewater or sludge use options.

The permittee shall use the following waste load allocation values when evaluating local limits for the following pollutants for which a final effluent limit has not been established:

Arsenic 59 ug/l
Cadmium 13 ug/l
Chromium, hexavalent 25 ug/l
Chromium, total 482 ug/l
Copper 53 ug/l
Free Cyanide 0.042 mg/l
Lead 59 ug/l
Mercury 12 ng/l
Molybdenum 45950 ug/l
Nickel 296 ug/l
Selenium 11 ug/l
Silver 3.0 ug/l
Zinc 610 ug/l

For the purpose of periodically reevaluating local limits, the permittee shall implement and maintain a sampling program to characterize pollutant contribution to the POTW from industrial and residential sources and to determine pollutant removal efficiencies through the POTW. The permittee shall continue to review and develop local limits as necessary.

5. Control Mechanisms

The permittee shall issue control mechanisms to all industries determined to be Significant Industrial Users as define in OAC 3745-3-01(FF). Control mechanisms must meet at least the minimum requirements of OAC-3745-3-03(C)(1)(c).

6. Industrial Compliance Monitoring

The permittee shall sample and inspect industrial users in accordance with the approved program or approved modifications, including inspection and sampling of all significant industrial users at least annually. Sample collection, preservation and analysis must be performed in accordance with procedures in 40 CFR 136 and with sufficient care to produce evidence admissible in judicial enforcement proceedings.

The permittee shall also require, receive, and review self-monitoring and other industrial user reports when necessary to determine compliance with pretreatment standards and requirements. If the permittee performs sampling and analysis in lieu of an industrial user¿s self-monitoring, the permittee shall perform repeat sampling and analysis within 30 days of becoming aware of a permit violation, unless the permittee notifies the user of the violation and requires the user to perform the repeat analysis and reporting.

7. POTW Priority Pollutant Monitoring

The permittee shall annually monitor priority pollutants, as defined by U.S. EPA, in the POTW's influent, effluent and sludge. Sample collection, preservation, and analysis shall be performed using U.S. EPA approved methods.

a. A sample of the influent and the effluent shall be collected when industrial discharges are occurring at normal to maximum levels. Sampling of the influent shall be done prior to any recycle streams and sampling of the effluent shall be after disinfection. Both samples shall be collected on the same day or, alternately, the effluent sample may be collected following the influent sample by approximately the retention time of the POTW.

Sampling of sludge shall be representative of sludge removed to final disposal. A minimum of one grab sample shall be taken during actual sludge removal and disposal unless the POTW uses more than one disposal option. If multiple disposal options are used, the POTW shall collect a composite of grab samples from all disposal practices which are proportional to the annual flows to each type of disposal.

b. A reasonable attempt shall be made to identify and quantify additional constituents (excluding priority pollutants and unsubstituted aliphatic compounds) at each sample location. Identification of additional peaks more than ten times higher than the adjacent background noise on the total ion plots (reconstructed gas chromatograms) shall be attempted through the use of U.S. EPA/NIH computerized library of mass spectra, with visual confirmation by an experienced analyst. Quantification may be based on an order of magnitude estimate compared with an internal standard.

The results of these samples must be submitted on Ohio EPA Form 4221 with the permittee's annual pretreatment report. Samples may be collected at any time during the 12 months preceding the due date of the annual report and may be used to fulfill other NPDES monitoring requirements where applicable.

8. Enforcement

The permittee shall investigate all instances of noncompliance with pretreatment standards and requirements and take timely, appropriate, and effective enforcement action to resolve the noncompliance in accordance with the permittee's approved enforcement response plan.

On or prior to April 15th of each year, the permittee shall publish, in the largest daily newspaper within the permittee's service area, a list of industrial users which, during the previous period of April 1st through March 31st, have been in Significant Noncompliance [OAC 3745-3-03(C)(2)(h)] with applicable pretreatment standards or requirements.

9. Reporting

All reports required under this section shall be submitted to the following address in duplicate:

Ohio Environmental Protection Agency
Division of Surface Water
Pretreatment Unit
P.O. Box 1049
Columbus, OH 43216-1049

a. Quarterly Industrial User Violation Report

On or prior to the 15th day of each February, May, August, and November, the permittee shall report the industrial users that are in violation of applicable pretreatment standards during the previous corresponding periods of November through January, February through April, May through July and August through October.

The report shall be prepared in accordance with guidance provided by Ohio EPA and shall include a description of all industrial user violations and corrective actions taken to resolve the violations.

b. Annual Pretreatment Report

On or prior to April 15th of each year, the permittee shall submit an annual report on the effectiveness of the pretreatment program for the previous twelve-month period of April 1st through March 31st.

The report shall be prepared in accordance with guidance provided by Ohio EPA and include, but not be limited to: a discussion of program effectiveness; and industrial user inventory; a description of the permittee's monitoring program; a description of any pass through or interference incidents; a copy of the annual publication of industries in Significant Noncompliance; and, priority pollutant monitoring results.

10. Record Keeping

All records of pretreatment activities including, but not limited to, industrial inventory data, monitoring results, enforcement actions, and reports submitted by industrial users must be maintained for a minimum of three (3) years. This period of retention shall be extended during the course of any unresolved litigation. Records must be made available to Ohio EPA and U.S. EPA upon request.

11. Program Modifications

Any proposed modifications of the approved pretreatment program must be submitted to Ohio EPA for review, on forms available from Ohio EPA and consistent with guidance provided by Ohio EPA. If the modification is deemed to be substantial, prior approval must be obtained before implementation; otherwise, the modification is considered to be effective 45 days after the date of application. Substantial program modifications include, among other things, changes to the POTW's legal authority, industrial user control mechanisms, local limits, confidentiality procedures, or monitoring frequencies.

PART III - GENERAL CONDITIONS

1. DEFINITIONS

"Daily discharge" means the discharge of a pollutant measured during a calendar day or any 24-hour period that reasonably represents the calendar day for purposes of sampling. For pollutants with limitations expressed in units of mass, the "daily discharge" is calculated as the total mass of the pollutant discharged over the day. For pollutants with limitations expressed in other units of measurement, the "daily discharge" is calculated as the average measurement of the pollutant over the day.

"Average weekly" discharge limitation means the highest allowable average of "daily discharges" over a calendar week, calculated as the sum of all "daily discharges" measured during a calendar week divided by the number of "daily discharges" measured during that week. Each of the following 7-day periods is defined as a calendar week: Week 1 is Days 1 - 7 of the month; Week 2 is Days 8 - 14; Week 3 is Days 15 - 21; and Week 4 is Days 22 - 28. If the "daily discharge" on days 29, 30 or 31 exceeds the "average weekly" discharge limitation, Ohio EPA may elect to evaluate the last 7 days of the month as Week 4 instead of Days 22 - 28. Compliance with fecal coliform bacteria or E coli bacteria limitations shall be determined using the geometric mean.

"Average monthly" discharge limitation means the highest allowable average of "daily discharges" over a calendar month, calculated as the sum of all "daily discharges" measured during a calendar month divided by the number of "daily discharges" measured during that month. Compliance with fecal coliform bacteria or E coli bacteria limitations shall be determined using the geometric mean.

"85 percent removal" means the arithmetic mean of the values for effluent samples collected in a period of 30 consecutive days shall not exceed 15 percent of the arithmetic mean of the values for influent samples collected at approximately the same times during the same period.

"Absolute Limitations" Compliance with limitations having descriptions of "shall not be less than," "nor greater than," "shall not exceed," "minimum," or "maximum" shall be determined from any single value for effluent samples and/or measurements collected.

"Net concentration" shall mean the difference between the concentration of a given substance in a sample taken of the discharge and the concentration of the same substances in a sample taken at the intake which supplies water to the given process. For the purpose of this definition, samples that are taken to determine the net concentration shall always be 24-hour composite samples made up of at least six increments taken at regular intervals throughout the plant day.

"Net Load" shall mean the difference between the load of a given substance as calculated from a sample taken of the discharge and the load of the same substance in a sample taken at the intake which supplies water to given process. For purposes of this definition, samples that are taken to determine the net loading shall always be 24-hour composite samples made up of at least six increments taken at regular intervals throughout the plant day.

"MGD" means million gallons per day.

"mg/l" means milligrams per liter.

"ug/l" means micrograms per liter.

"ng/l" means nanograms per liter.

"S.U." means standard pH unit.

"kg/day" means kilograms per day.

"Reporting Code" is a five digit number used by the Ohio EPA in processing reported data. The reporting code does not imply the type of analysis used nor the sampling techniques employed.

"Quarterly (1/Quarter) sampling frequency" means the sampling shall be done in the months of March, June, August, and December, unless specifically identified otherwise in the Effluent Limitations and Monitoring Requirements table.

"Yearly (1/Year) sampling frequency" means the sampling shall be done in the month of September, unless specifically identified otherwise in the effluent limitations and monitoring requirements table.

"Semi-annual (2/Year) sampling frequency" means the sampling shall be done during the months of June and December, unless specifically identified otherwise.

"Winter" shall be considered to be the period from November 1 through April 30.

"Bypass" means the intentional diversion of waste streams from any portion of the treatment facility.

"Summer" shall be considered to be the period from May 1 through October 31.

"Severe property damage" means substantial physical damage to property, damage to the treatment facilities which would cause 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.

"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 operational error, improperly designed treatment facilities, inadequate treatment facilities, lack of preventive maintenance, or careless or improper operation.

"Sewage sludge" means a solid, semi-solid, or liquid residue generated during the treatment of domestic sewage in a treatment works as defined in section 6111.01 of the Revised Code. "Sewage sludge" includes, but is not limited to, scum or solids removed in primary, secondary, or advanced wastewater treatment processes. "Sewage sludge" does not include ash generated during the firing of sewage sludge in a sewage sludge incinerator, grit and screenings generated during preliminary treatment of domestic sewage in a treatment works, animal manure, residue generated during treatment of animal manure, or domestic septage.

"Sewage sludge weight" means the weight of sewage sludge, in dry U.S. tons, including admixtures such as liming materials or bulking agents. Monitoring frequencies for sewage sludge parameters are based on the reported sludge weight generated in a calendar year (use the most recent calendar year data when the NPDES permit is up for renewal).

"Sewage sludge fee weight" means the weight of sewage sludge, in dry U.S. tons, excluding admixtures such as liming materials or bulking agents. Annual sewage sludge fees, as per section 3745.11(Y) of the Ohio Revised Code, are based on the reported sludge fee weight for the most recent calendar year.

2. GENERAL EFFLUENT LIMITATIONS

The effluent shall, at all times, be free of substances:

A. In amounts that will settle to form putrescent, or otherwise objectionable, sludge deposits; or that will adversely affect aquatic life or water fowl;

B. Of an oily, greasy, or surface-active nature, and of other floating debris, in amounts that will form noticeable accumulations of scum, foam or sheen;

C. In amounts that will alter the natural color or odor of the receiving water to such degree as to create a nuisance;

D. In amounts that either singly or in combination with other substances are toxic to human, animal, or aquatic life;

E. In amounts that are conducive to the growth of aquatic weeds or algae to the extent that such growths become inimical to more desirable forms of aquatic life, or create conditions that are unsightly, or constitute a nuisance in any other fashion;

F. In amounts that will impair designated instream or downstream water uses.

3. FACILITY OPERATION AND QUALITY CONTROL

All wastewater treatment works shall be operated in a manner consistent with the following:

A. At all times, the permittee shall maintain in good working order and operate as efficiently as possible all treatment or control facilities or systems installed or used by the permittee necessary to achieve compliance with the terms and 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 conditions of the permit.

B. The permittee shall effectively monitor the operation and efficiency of treatment and control facilities and the quantity and quality of the treated discharge.

C. Maintenance of wastewater treatment works that results in degradation of effluent quality shall be scheduled during non-critical water quality periods and shall be carried out in a manner approved by Ohio EPA as specified in the Paragraph in the PART III entitled, "UNAUTHORIZED DISCHARGES".

4. REPORTING

A. Monitoring data required by this permit shall be submitted monthly on Ohio EPA 4500 Discharge Monitoring Report (DMR) forms using the electronic DMR (e-DMR) internet application. e-DMR allows permitted facilities to enter, sign, and submit DMRs on the internet. e-DMR information is found on the following web page:

http://www.epa.ohio.gov/dsw/edmr/eDMR.aspx

Alternatively, if you are unable to use e-DMR due to a demonstrated hardship, monitoring data may be submitted on paper DMR forms provided by Ohio EPA. Monitoring data shall be typed on the forms. Please contact Ohio EPA, Division of Surface Water at (614) 644-2050 if you wish to receive paper DMR forms.

B. DMRs shall be signed by a facility's Responsible Official or a Delegated Responsible Official (i.e. a person delegated by the Responsible Official). The Responsible Official of a facility is defined as:

1. For corporations - a president, secretary, treasurer, or vice-president of the corporation in charge of a principal business function, or any other person who performs similar policy or decision making functions for the corporation; or the manager of one or more manufacturing, production or operating facilities, provided the manager is authorized to make management decisions which govern the operation of the regulated facility including having explicit or implicit duty of making major capital investment recommendations, and initiating and directing other comprehensive measures to assure long-term environmental compliance with environmental laws and regulations; the manager can ensure that the necessary systems are established or actions taken to gather complete and accurate information for permit application requirements; and where authority to sign documents has been assigned or delegated to the manager in accordance with corporate procedures;

2. For partnerships - a general partner;

3. For a sole proprietorship - the proprietor; or,

4. For a municipality, state or other public facility - a principal executive officer, a ranking elected official or other duly authorized employee.

For e-DMR, the person signing and submitting the DMR will need to obtain an eBusiness Center account and Personal Identification Number (PIN). Additionally, Delegated Responsible Officials must be delegated by the Responsible Official, either on-line using the eBusiness Center's delegation function, or on a paper delegation form provided by Ohio EPA. For more information on the PIN and delegation processes, please view the following web page:

http://epa.ohio.gov/dsw/edmr/eDMR.aspx

C. DMRs submitted using e-DMR shall be submitted to Ohio EPA by the 20th day of the month following the month-of-interest. DMRs submitted on paper must include the original signed DMR form and shall be mailed to Ohio EPA at the following address so that they are received no later than the 15th day of the month following the month-of-interest:

Ohio Environmental Protection Agency
Lazarus Government Center
Division of Surface Water - PCU
P.O. Box 1049
Columbus, Ohio 43216-1049

D. Regardless of the submission method, a paper copy of the submitted Ohio EPA 4500 DMR shall be maintained onsite for records retention purposes (see Section 7. RECORDS RETENTION). For e-DMR users, view and print the DMR from the Submission Report Information page after each original or revised DMR is submitted. For submittals on paper, make a copy of the completed paper form after it is signed by a Responsible Official or a Delegated Responsible Official.

E. If the permittee monitors any pollutant at the location(s) designated herein more frequently than required by this permit, using approved analytical methods as specified in Section 5. SAMPLING AND ANALYTICAL METHODS, the results of such monitoring shall be included in the calculation and reporting of the values required in the reports specified above.

F. Analyses of pollutants not required by this permit, except as noted in the preceding paragraph, shall not be reported to the Ohio EPA, but records shall be retained as specified in Section 7. RECORDS RETENTION.

5. SAMPLING AND ANALYTICAL METHOD

Samples and measurements taken as required herein shall be representative of the volume and nature of the monitored flow. Test procedures for the analysis of pollutants shall conform to regulation 40 CFR 136, "Test Procedures For The Analysis of Pollutants" unless other test procedures have been specified in this permit. The permittee shall periodically calibrate and perform maintenance procedures on all monitoring and analytical instrumentation at intervals to insure accuracy of measurements.

6. RECORDING OF RESULTS

For each measurement or sample taken pursuant to the requirements of this permit, the permittee shall record the following information:

A. The exact place and date of sampling; (time of sampling not required on EPA 4500)

B. The person(s) who performed the sampling or measurements;

C. The date the analyses were performed on those samples;

D. The person(s) who performed the analyses;

E. The analytical techniques or methods used; and

F. The results of all analyses and measurements.

7. RECORDS RETENTION

The permittee shall retain all of the following records for the wastewater treatment works for a minimum of three years except those records that pertain to sewage sludge disposal, use, storage, or treatment, which shall be kept for a minimum of five years, including:

A. All sampling and analytical records (including internal sampling data not reported);

B. All original recordings for any continuous monitoring instrumentation;

C. All instrumentation, calibration and maintenance records;

D. All plant operation and maintenance records;

E. All reports required by this permit; and

F. Records of all data used to complete the application for this permit for a period of at least three years, or five years for sewage sludge, from the date of the sample, measurement, report, or application.

These periods will be extended during the course of any unresolved litigation, or when requested by the Regional Administrator or the Ohio EPA. The three year period, or five year period for sewage sludge, for retention of records shall start from the date of sample, measurement, report, or application.

8. AVAILABILITY OF REPORTS

Except for data determined by the Ohio EPA to be entitled to confidential status, all reports prepared in accordance with the terms of this permit shall be available for public inspection at the appropriate district offices of the Ohio EPA. Both the Clean Water Act and Section 6111.05 Ohio Revised Code state that effluent data and receiving water quality data shall not be considered confidential.

9. DUTY TO PROVIDE INFORMATION

The permittee shall furnish to the Director, within a reasonable time, any information which the Director may request to determine whether cause exists for modifying, revoking, and reissuing, or terminating the permit, or to determine compliance with this permit. The permittee shall also furnish to the Director, upon request, copies of records required to be kept by this permit.

10. RIGHT OF ENTRY

The permittee shall allow the Director or an authorized representative upon presentation of credentials and other documents as may be required by law 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 the permit.

C. Inspect at reasonable times any facilities, equipment (including monitoring and control equipment), practices, or operations regulated or required under this permit.

D. Sample or monitor at reasonable times, for the purposes of assuring permit compliance or as otherwise authorized by the Clean Water Act, any substances or parameters at any location.

11. UNAUTHORIZED DISCHARGES

A. Bypass Not Exceeding Limitations - The permittee may allow any bypass to occur which does not cause effluent limitations to be exceeded, but only if it also is for essential maintenance to assure efficient operation. These bypasses are not subject to the provisions of paragraphs 11.B and 11.C.

B. Notice

1. Anticipated Bypass - If the permittee knows in advance of the need for a bypass, it shall submit prior 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 paragraph 12.B (24 hour notice).

C. Prohibition of Bypass

1. Bypass is prohibited, and the Director may take enforcement action against a permittee for bypass, unless:

a. Bypass was unavoidable 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 back-up equipment should have been installed in the exercise of reasonable engineering judgment to prevent a bypass which occurred during normal periods of equipment downtime or preventive maintenance; and

c. The permittee submitted notices as required under paragraph 11.B.

2. The Director may approve an anticipated bypass, after considering its adverse effects, if the Director determines that it will meet the three conditions listed above in paragraph 11.C.1.

12. NONCOMPLIANCE NOTIFICATION

A. Exceedance of a Daily Maximum Discharge Limit

1. The permittee shall report noncompliance that is the result of any violation of a daily maximum discharge limit for any of the pollutants listed by the Director in the permit by e-mail or telephone within twenty-four (24) hours of discovery.

The permittee may report to the appropriate Ohio EPA district office e-mail account as follows (this method is preferred):

Southeast District Office: sedo24hournpdes@epa.state.oh.us
Southwest District Office: swdo24hournpdes@epa.state.oh.us
Northwest District Office: nwdo24hournpdes@epa.state.oh.us
Northeast District Office: nedo24hournpdes@epa.state.oh.us
Central District Office: cdo24hournpdes@epa.state.oh.us
Central Office: co24hournpdes@epa.state.oh.us

The permittee shall attach a noncompliance report to the e-mail. A noncompliance report form is available on the following web site under the Monitoring and Reporting - Non-Compliance Notification section:

http://epa.ohio.gov/dsw/permits/individuals.aspx

Or, the permittee may report to the appropriate Ohio EPA district office by telephone toll-free between 8:00 AM and 5:00 PM as follows:

Southeast District Office: (800) 686-7330
Southwest District Office: (800) 686-8930
Northwest District Office: (800) 686-6930
Northeast District Office: (800) 686-6330
Central District Office: (800) 686-2330
Central Office: (614) 644-2001

The permittee shall include the following information in the telephone noncompliance report:

a. The name of the permittee, and a contact name and telephone number;

b. The limit(s) that has been exceeded;

c. The extent of the exceedance(s);

d. The cause of the exceedance(s);

e. The period of the exceedance(s) including exact dates and times;

f. If uncorrected, the anticipated time the exceedance(s) is expected to continue; and,

g. Steps taken to reduce, eliminate or prevent occurrence of the exceedance(s).

B. Other Permit Violations

1. The permittee shall report noncompliance that is the result of any unanticipated bypass resulting in an exceedance of any effluent limit in the permit or any upset resulting in an exceedance of any effluent limit in the permit by e-mail or telephone within twenty-four (24) hours of discovery.

The permittee may report to the appropriate Ohio EPA district office e-mail account as follows (this method is preferred):

Southeast District Office: sedo24hournpdes@epa.state.oh.us
Southwest District Office: swdo24hournpdes@epa.state.oh.us
Northwest District Office: nwdo24hournpdes@epa.state.oh.us
Northeast District Office: nedo24hournpdes@epa.state.oh.us
Central District Office: cdo24hournpdes@epa.state.oh.us
Central Office: co24hournpdes@epa.state.oh.us

The permittee shall attach a noncompliance report to the e-mail. A noncompliance report form is available on the following web site:

http://www.epa.ohio.gov/dsw/permits/permits.aspx

Or, the permittee may report to the appropriate Ohio EPA district office by telephone toll-free between 8:00 AM and 5:00 PM as follows:

Southeast District Office: (800) 686-7330
Southwest District Office: (800) 686-8930
Northwest District Office: (800) 686-6930
Northeast District Office: (800) 686-6330
Central District Office: (800) 686-2330
Central Office: (614) 644-2001

The permittee shall include the following information in the telephone noncompliance report:

a. The name of the permittee, and a contact name and telephone number;

b. The time(s) at which the discharge occurred, and was discovered;

c. The approximate amount and the characteristics of the discharge;

d. The stream(s) affected by the discharge;

e. The circumstances which created the discharge;

f. The name and telephone number of the person(s) who have knowledge of these circumstances;

g. What remedial steps are being taken; and,

h. The name and telephone number of the person(s) responsible for such remedial steps.

2. The permittee shall report noncompliance that is the result of any spill or discharge which may endanger human health or the environment within thirty (30) minutes of discovery by calling the 24-Hour Emergency Hotline toll-free at (800) 282-9378. The permittee shall also report the spill or discharge by e-mail or telephone within twenty-four (24) hours of discovery in accordance with B.1 above.

C. When the telephone option is used for the noncompliance reports required by A and B, the permittee shall submit to the appropriate Ohio EPA district office a confirmation letter and a completed noncompliance report within five (5) days of the discovery of the noncompliance. This follow up report is not necessary for the e-mail option which already includes a completed noncompliance report.

D. If the permittee is unable to meet any date for achieving an event, as specified in a schedule of compliance in their permit, the permittee shall submit a written report to the appropriate Ohio EPA district office within fourteen (14) days of becoming aware of such a situation. The report shall include the following:

1. The compliance event which has been or will be violated;

2. The cause of the violation;

3. The remedial action being taken;

4. The probable date by which compliance will occur; and,

5. The probability of complying with subsequent and final events as scheduled.

E. The permittee shall report all other instances of permit noncompliance not reported under paragraphs A or B of this section on their monthly DMR submission. The DMR shall contain comments that include the information listed in paragraphs A or B as appropriate.

F. If the permittee becomes aware that it failed to submit an application, or submitted incorrect information in an application or in any report to the director, it shall promptly submit such facts or information.

13. RESERVED

14. DUTY TO MITIGATE

The permittee shall take all reasonable steps to minimize or prevent any discharge in violation of this permit which has a reasonable likelihood of adversely affecting human health or the environment.

15. AUTHORIZED DISCHARGES

All discharges authorized herein shall be consistent with the terms and conditions of this permit. The discharge of any pollutant identified in this permit more frequently than, or at a level in excess of, that authorized by this permit shall constitute a violation of the terms and conditions of this permit. Such violations may result in the imposition of civil and/or criminal penalties as provided for in Section 309 of the Act and Ohio Revised Code Sections 6111.09 and 6111.99.

16. DISCHARGE CHANGES

The following changes must be reported to the appropriate Ohio EPA district office as soon as practicable:

A. For all treatment works, any significant change in character of the discharge which the permittee knows or has reason to believe has occurred or will occur which would constitute cause for modification or revocation and reissuance. 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. Notification of permit changes or anticipated noncompliance does not stay any permit condition.

B. For publicly owned treatment works:

1. Any proposed plant modification, addition, and/or expansion that will change the capacity or efficiency of the plant;

2. The addition of any new significant industrial discharge; and

3. Changes in the quantity or quality of the wastes from existing tributary industrial discharges which will result in significant new or increased discharges of pollutants.

C. For non-publicly owned treatment works, any proposed facility expansions, production increases, or process modifications, which will result in new, different, or increased discharges of pollutants.

Following this notice, modifications to the permit may be made to reflect any necessary changes in permit conditions, including any necessary effluent limitations for any pollutants not identified and limited herein. A determination will also be made as to whether a National Environmental Policy Act (NEPA) review will be required. Sections 6111.44 and 6111.45, Ohio Revised Code, require that plans for treatment works or improvements to such works be approved by the Director of the Ohio EPA prior to initiation of construction.

D. In addition to the reporting requirements under 40 CFR 122.41(l) and per 40 CFR 122.42(a), all existing manufacturing, commercial, mining, and silvicultural dischargers must notify the Director as soon as they know or have reason to believe:

1. 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 "notification levels" specified in 40 CFR Sections 122.42(a)(1)(i) through 122.42(a)(1)(iv).

2. 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 "notification levels" specified in 122.42(a)(2)(i) through 122.42(a)(2)(iv).

17. TOXIC POLLUTANTS

The permittee shall comply with 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 these standards or prohibitions, even if the permit has not yet been modified to incorporate the requirement. Following establishment of such standards or prohibitions, the Director shall modify this permit and so notify the permittee.

18. PERMIT MODIFICATION OR REVOCATION

A. After notice and opportunity for a hearing, this permit may be modified or revoked, by the Ohio EPA, in whole or in part during its term for cause including, but not limited to, the following:

1. Violation of any terms or conditions of this permit;

2. Obtaining this permit by misrepresentation or failure to disclose fully all relevant facts; or

3. Change in any condition that requires either a temporary or permanent reduction or elimination of the permitted discharge.

B. Pursuant to rule 3745-33-04, Ohio Administrative Code, the permittee may at any time apply to the Ohio EPA for modification of any part of this permit. The filing of a request by the permittee for a permit modification or revocation does not stay any permit condition. The application for modification should be received by the appropriate Ohio EPA district office at least ninety days before the date on which it is desired that the modification become effective. The application shall be made only on forms approved by the Ohio EPA.

19. TRANSFER OF OWNERSHIP OR CONTROL

This permit may be transferred or assigned and a new owner or successor can be authorized to discharge from this facility, provided the following requirements are met:

A. The permittee shall notify the succeeding owner or successor of the existence of this permit by a letter, a copy of which shall be forwarded to the appropriate Ohio EPA district office. The copy of that letter will serve as the permittee's notice to the Director of the proposed transfer. The copy of that letter shall be received by the appropriate Ohio EPA district office sixty (60) days prior to the proposed date of transfer;

B. A written agreement containing a specific date for transfer of permit responsibility and coverage between the current and new permittee (including acknowledgement that the existing permittee is liable for violations up to that date, and that the new permittee is liable for violations from that date on) shall be submitted to the appropriate Ohio EPA district office within sixty days after receipt by the district office of the copy of the letter from the permittee to the succeeding owner;

At anytime during the sixty (60) day period between notification of the proposed transfer and the effective date of the transfer, the Director may prevent the transfer if he concludes that such transfer will jeopardize compliance with the terms and conditions of the permit. If the Director does not prevent transfer, he will modify the permit to reflect the new owner.

20. OIL AND HAZARDOUS SUBSTANCE LIABILITY

Nothing in this permit shall be construed to preclude the institution of any legal action or relieve the permittee from any responsibilities, liabilities, or penalties to which the permittee is or may be subject under Section 311 of the Clean Water Act.

21. SOLIDS DISPOSAL

Collected grit and screenings, and other solids other than sewage sludge, shall be disposed of in such a manner as to prevent entry of those wastes into waters of the state, and in accordance with all applicable laws and rules.

22. CONSTRUCTION AFFECTING NAVIGABLE WATERS

This permit does not authorize or approve the construction of any onshore or offshore physical structures or facilities or the undertaking of any work in any navigable waters.

23. CIVIL AND CRIMINAL LIABILITY

Except as exempted in the permit conditions on UNAUTHORIZED DISCHARGES or UPSETS, nothing in this permit shall be construed to relieve the permittee from civil or criminal penalties for noncompliance.

24. STATE LAWS AND REGULATIONS

Nothing in this permit shall be construed to preclude the institution of any legal action or relieve the permittee from any responsibilities, liabilities, or penalties established pursuant to any applicable state law or regulation under authority preserved by Section 510 of the Clean Water Act.

25. PROPERTY RIGHTS

The issuance of this permit does not convey any property rights in either real or personal property, or any exclusive privileges, nor does it authorize any injury to private property or any invasion of personal rights, nor any infringement of federal, state, or local laws or regulations.

26. UPSET

The provisions of 40 CFR Section 122.41(n), relating to "Upset," are specifically incorporated herein by reference in their entirety. For definition of "upset," see Part III, Paragraph 1, DEFINITIONS.

27. SEVERABILITY

The provisions of this permit are severable, and if any provision of this permit, or the application of any provision of this permit to any circumstance, is held invalid, the application of such provision to other circumstances, and the remainder of this permit, shall not be affected thereby.

28. SIGNATORY REQUIREMENTS

All applications submitted to the Director shall be signed and certified in accordance with the requirements of 40 CFR 122.22.

All reports submitted to the Director shall be signed and certified in accordance with the requirements of 40 CFR Section 122.22.

29. OTHER INFORMATION

A. Where 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 in any report to the Director, it shall promptly submit such facts or information.

B. ORC 6111.99 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 $25,000 per violation.

C. ORC 6111.99 states that any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit including monitoring reports or reports of compliance or noncompliance shall, upon conviction, be punished by a fine of not more than $25,000 per violation.

D. ORC 6111.99 provides that any person who violates Sections 6111.04, 6111.042, 6111.05, or division (A) of Section 6111.07 of the Revised Code shall be fined not more than $25,000 or imprisoned not more than one year, or both.

30. NEED TO HALT OR REDUCE ACTIVITY

40 CFR 122.41(c) states that 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 conditions of this permit.

31. APPLICABLE FEDERAL RULES

All references to 40 CFR in this permit mean the version of 40 CFR which is effective as of the effective date of this permit.

32. AVAILABILITY OF PUBLIC SEWERS

Not withstanding the issuance or non-issuance of an NPDES permit to a semi-public disposal system, whenever the sewage system of a publicly owned treatment works becomes available and accessible, the permittee operating any semi-public disposal system shall abandon the semi-public disposal system and connect it into the publicly owned treatment works.

CDM Smith