City of PIQUA Ohio

FINAL REPORT

PIQUA WATER TREATMENT PLANT PLANNING AND PRELIMINARY DESIGN

August 2012

CDM

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Table of Contents

List of Abbreviations

Executive Summary

Section 1 – Introduction and Project Background

Section 2 – Water Quality

2.1 Review of Existing Data 2-1
2.1.1 TOC 2-2
2.1.2 Atrazine 2-3
2.1.3 Hardness 2-4
2.2 Finished Water Quality 2-4
2.2.1 Finished Water Quality Goals 2-8

Section 3 – Capacity Requirements

3.1 Data Sources 3-1
3.2 Service Areas 3-1
3.3 Demand History 3-2
3.4 Population Projections 3-4
3.5 Land Use Projections 3-7
3.6 Unaccounted for Water 3-8
3.7 Future Water Demands 3-11

Section 4 – Regulatory Requirements

4.1 Current Regulations 4-1
4.1.1 SWTR 4-2
4.1.2 IESWTR 4-2
4.1.3 Lead and Copper Rule (LCR) 4-3
4.1.4 Total Coliform Rule (TRC) 4-3
4.1.5 Filter Backwash Recycling Rule (FBRR) 4-4
4.1.6 Volatile Organic Chemicals, Synthetic Organic Chemicals and Inorganic Chemicals (VOCs SOCsIOCs) 4-4
4.1.6.1 Phase 1: VOCs 4-4
4.1.6.2 Phase I and V: Inorganics, SOCs and VOCs 4-4
4.1.7 Stage 1 Disinfectant and Disinfection By-Products Rule (D/DBP) 4-5
4.1.8 Radionuclide Rule 4-5
4.1.9 Arsenic Rule 4-5
4.1.10 Source Water Protection 4-5
4.1.11 National Secondary Drinking Water Standards (NSDWS) 4-6
4.1.12 Fluoride Rule 4-6
4.1.13 LT2ESWTR 4-6
4.1.14 Stage 2 DBP Rule 4-7

4.2 Pending Regulations **4-8**
4.2.1 Total Coliform Rule Revisions (TCR) 4-8
4.2.2 Additional Chlorinated, Brominated and Iodinated DBPs 4-8
4.2.3 Perchlorate 4-8
4.2.4 Group Regulation of Carcinogenic VOCs 4-8
4.3 Future Regulations **4-9**
4.3.1 Potential Individual Contaminant Regulations: Candidate Contaminant List (CCL)3 and UCMR3 4-9
4.3.2 Six-Year Review 4-9

Section 5 – Condition Assessment of Existing Facilities

5.1 Evaluation Criteria **5-1**
5.2 River Pump Station **5-3**
5.3 Gravel Quarry Pump Station **5-7**
5.4 Raw Water Intakes and Screen/Valve Chambers at Swift Run Lake **5-8**
5.5 Sludge Disposal Force Main and Lagoon **5-11**
5.6 Filter Backwash and Sludge Disposal Facility **5-13**

Section 6 – Process Alternatives Development and Screening

6.1 Description of Process Alternatives **6-1**
6.1.1 Unit Process Alternatives 6-1
6.1.1.1 Pretreatment Alternatives 6-1
6.1.1.2 Clarification Alternatives 6-3
6.1.1.3 Softening Alternatives 6-5
6.1.1.4 Filtration Alternatives 6-7
6.1.1.5 Advanced Treatment Alternatives 6-7
6.1.1.6 Disinfection Alternatives 6-9
6.1.1.7 Summary 6-10
6.1.2 Conventional Process Train Alternatives 6-11
6.1.3 Membrane Process Train Alternatives 6-15
6.2 Alternatives Screening and Selection **6-17**
6.2.1 Ability to Meet Water Quality Goals 6-17
6.2.2 Operation and Maintenance (O&M) 6-18
6.2.3 Reliability and Risk 6-18
6.2.4 Implementation 6-18
6.2.5 Relative Cost 6-19

Section 7 – Process Alternatives Assessment

7.1 Lime Softening With Ozone (Alternative 1B) **7-5**
7.1.1 Alternative Description 7-5
7.1.2 Bench Scale Testing Results 7-25
7.2 Enhanced Coagulation and Lime Softening With Ozone (Alternative 3B) **7-26**
7.2.1 Alternative Description 7-26
7.2.2 Bench Scale Testing Results 7-37
7.3 Lime Softening With Post-Filter GAC Contactors (Alternative 1D) **7-37**
7.3.1 Alternative Description 7-38
7.3.2 Bench Scale Testing Results 7-47

7.4 Enhanced Coagulation and Lime Softening With Post-Filter GAC Contactors (Alternative 3D) 7-48
7.4.1 Alternative Description 7-48
7.4.2 Bench Scale Testing Results 7-57
7.5 Cost Summary 7-57
7.5.1 Construction Costs for Alternatives 7-57
7.5.2 Annual Operation and Maintenance Costs for Alternatives 7-62
7.6 Final Screening of Alternatives and Selection 7-63

Section 8 – Selected Alternative

8.1 Design Criteria for New Water Treatment Plant 8-1
8.2 Source Water Supply 8-3
8.2.1 Surface Water 8-3
8.2.2 Groundwater 8-4
8.3 Rapid Mix, Flocculation and Sedimentation 8-5
8.3.1 Rapid Mixing Basins 8-5
8.3.2 Flocculation Basins 8-5
8.3.3 Sedimentation Basins 8-6
8.4 Recarbonation Basins 8-8
8.5 Filtration 8-8
8.6 Granular Activated Carbon Contactors 8-9
8.7 Finished Water Storage 8-10
8.8 High Service Pump Station 8-10
8.9 Chemical Systems 8-17
8.9.1 Sodium Hypochlorite System 8-21
8.10 Washwater Recovery 8-22
8.10.1 Sand Filter Backwash Water 8-22
8.10.2 GAC Backwash Water 8-22
8.11 Optional Facilities 8-22
8.11.1 Pretreatment Facility 8-23
8.11.2 Residual Solids Handling Facility 8-23
8.11.3 Garage/Maintenance Facility 8-29
8.12 Civil/Site Work 8-29
8.13 Environmental Permitting 8-30
8.14 Geotechnical Investigation 8-30
8.15 Structural Design 8-33
8.16 Architectural Design 8-33
8.17 HVAC, Plumbing and Fire Protection 8-34
8.18 Electrical Design 8-34
8.19 Instrumentation and Control Design 8-37

Section 9 – Implementation Plan

9.1 CIP Schedule 9-1
9.2 Funding Options 9-2

Appendices

Appendix A Drinking Water Standards for Ohio Public Water Systems
Appendix B Overall Condition Scores for Existing Facilities
Appendix C Bench Scale Testing Results and Technical Memorandum

Tables

Table 2-1. Summary of Raw WQ Data (2005-2010) 2-2
Table 2-2. Summary of Finished WQ Data (2005-2010) 2-4
Table 2-3. Finished WQ Goals 2-10
Table 3-1. Water Production History 3-2
Table 3-2. Water Consumption History 3-4
Table 3-3. Population Trend and Projections 3-5
Table 3-4. Comparison of Current and Future Land Use Categories 3-8
Table 3-5. Historic UFW (%) 3-11
Table 3-6. Unit Water Demands 3-12
Table 3-7. Future Water Demands 3-13
Table 3-8. New Water Treatment Plant Capacity Alternatives 3-14
Table 4-1. IESWTR Requirements 4-3
Table 4-2. Ohio EPA Radionuclides Detection Limits 4-5
Table 4-3. Bin Classification For Filtered Systems 4-6
Table 6-1. Pretreatment Alternatives Comparison 6-2
Table 6-2. Clarification Alternatives Comparison 6-5
Table 6-3. Softening Alternatives Comparison 6-6
Table 6-4. Filtration Alternatives Comparison 6-7
Table 6-5. Advanced Treatment Alternatives Comparison 6-9
Table 6-6. Disinfection Alternatives Comparison 6-11
Table 6-7. WQ Goals and Unit Process Alternatives 6-11
Table 6-8. Treatment Performance Comparison for Process Train Alternatives 6-18
Table 6-9. Process Train Alternatives Scoring Matrix 6-19
Table 7-1. Pretreatment Plate Settlers Design Criteria 7-9
Table 7-2. Rapid Mix Basin Design Criteria 7-9
Table 7-3. Flocculation Basin Design Criteria 7-10
Table 7-4. Sedimentation Basin Design Criteria 7-10
Table 7-5. Recarbonation Basin Design Criteria 7-11
Table 7-6. Ozone Generation System Design Criteria 7-11
Table 7-7. Ozone Contactor Design Criteria 7-12
Table 7-8. Biological GAC Filtration Design Criteria 7-12
Table 7-9. Clearwell Design Criteria 7-13
Table 7-10. High Service Pump Station Design Criteria 7-13
Table 7-11. Alternative 1B-Chemical Doses 7-14
Table 7-12. Solids Handling Facility Design Criteria 7-14
Table 7-13. Backwash Holding Tank Design Criteria 7-15
Table 7-14. Ozone Demand/Decay Data 7-25
Table 7-15. Jar test Results for Single-Stage Treatment 7-25
Table 7-16. First Stage Conventional Treatment Design Criteria 7-27
Table 7-17. Second Stage Conventional Treatment Design Criteria 7-28

Table 7-18. Jar Test Results for Two-Stage Treatment 7-37
Table 7-19. Conventional Filtration Design Criteria 7-39
Table 7-20. GAC Feed Pumps Design Criteria 7-39
Table 7-21. Post-Filter GAC Contactors Design Criteria 7-40
Table 7-22. Backwash Holding Tank Design Criteria 7-40
Table 7-23. Potassium Permanganate Demand/Decay Data 7-47
Table 7-24. Chlorine Dioxide Demand/Decay Data 7-47
Table 7-25. Alternative 1B Preliminary Opinion of Probable Construction Cost 7-58
Table 7-26. Alternative 3B Preliminary Opinion of Probable Construction Cost 7-59
Table 7-27. Alternative 1D Preliminary Opinion of Probable Construction Cost 7-60
Table 7-28. Alternative 3D Preliminary Opinion of Probable Construction Cost 7-61
Table 7-29. Annual O&M Cost Summary 7-63
Table 7-30. Present Worth Cost Summary 7-63
Table 7-31. Process Train Alterative Final Scoring Matrix 7-64
Table 8-1. Design Criteria for New WTP 8-1
Table 8-2. Design Criteria for Water Supply Sources 8-3
Table 8-3. Chemical Bulk Storage Summary 8-18
Table 8-4. Estimated Sludge Production 8-23
Table 8-5. Estimated Electrical Load 8-35

Figures

Figure 2-1. Raw Water TOC 2-2
Figure 2-2. Raw Water Atrazine 2-3
Figure 2-3. TOC Removal 2-5
Figure 2-4. TTHM RAA 2-6
Figure 2-5. HAA RAA 2-6
Figure 2-6. TTHM LRAA at Compliance Monitoring Locations 2-7
Figure 2-7. HAA LRAA at Compliance Monitoring Locations 2-8
Figure 3-1. Planning Area 3-3
Figure 3-2. Water Production History 3-4
Figure 3-3. Population Change Forecast 3-6
Figure 3-4. Future Land Use Distribution (Total Area = 13,240 acres) 3-7
Figure 3-5. Current Land Use for Planning Area 3-9
Figure 3-6. Future Land Use for Planning Area 3-10
Figure 3-7. Future Water Demands Range 3-13
Figure 3-8. Service Life from 6.0 MGD and 6.75 MGD Initial Capacities 3-15
Figure 5-1. River Pump Station Exterior Walls 5-3
Figure 5-2. River Pump Station Interior Ceiling 5-4
Figure 5-3. River Pump Station Piping 5-5
Figure 5-4. Flood Map 5-6
Figure 5-5. Quarry Pump Station 5-7
Figure 5-6. Screen Chamber No. 1 5-9
Figure 5-7. Screen Chamber No. 1 Exhaust Hood 5-9

Figure 5-8. Screen Chamber No. 1 Gate Operator 5-10
Figure 5-9. Screen Chamber No. 2 5-10
Figure 5-10. Sludge Disposal Lagoon 5-12
Figure 5-11. Sludge Disposal Lagoon Decant Structure 5-12
Figure 5-12. Filter Backwash Holding Basin 5-14
Figure 5-13. Gilter Backwash Sludge Pumps 5-14
Figure 5-14. Filter Backwash and Sludge Disposal Facility 5-15
Figure 6-1. Plate Settler 6-4
Figure 6-2. Solids Contact Clarifier 6-6
Figure 6-3. GAC Adsorber 6-8
Figure 6-4. Ozonation System 6-9
Figure 6-5. Alternative 1A – Lime Softening 6-12
Figure 6-6. Alternative 1B – Lime Softening with Ozone 6-12
Figure 6-7. Alternative 1D – Lime Softening with Post-Filter GAC Contactors 6-13
Figure 6-8. Alternative 3A – Enhanced Coagulation and Lime Softening 6-13
Figure 6-9. Alternative 3B – Enhanced Coagulation and Lime Softening With Ozone 6-14
Figure 6-10. Alternative 3D – Enhanced Coagulation and Lime Softening with Post-Filter GAC Contactors 6-14
Figure 6-11. Alternative 1C – Lime Softening with MF/UF Filtration 6-15
Figure 6-12. Alternative 2A – Enhanced Coagulation and Membrane Softening 6-16
Figure 6-13. Alternative 2B – Enhanced Coagulation and Membrane Softening with MF/UF 6-16
Figure 6-14. Alternative 3C – Enhanced Coagulation and Lime Softening with MF/UF Filtration 6-17
Figure 7-1. New WTP Location 7-3
Figure 7-2. Alternative 1B – Lime Softening with Ozone 7-5
Figure 7-3. Proposed Raw Water Pump Station Location 7-7
Figure 7-4. Offsite Improvements 7-17
Figure 7-5. Hydraulic Profile - Alternative 1B 7-19
Figure 7-6. Site plan - Alternative 1B 7-21
Figure 7-7. P&ID - Alternative 1B 7-23
Figure 7-8. Alternative 3B - Enhanced Coagulating and Lime Softening with Ozone 7-26
Figure 7-9. Hydraulic Profile - Alternative 3B 7-31
Figure 7-10. Site Plan - Alternative 3B 7-33
Figure 7-11. P&ID - Alternative 3B 7-35
Figure 7-12. Alternative 1D - Lime Softening with Post-Filter GAC Contactors 7-37
Figure 7-13. Hydraulic Profile - Alternative 1D 7-41
Figure 7-14. Site Plan - Alternative 1D 7-43
Figure 7-15. P&ID - Alternative 1D 7-45
Figure 7-16. Alternative 3D - Enhanced Coagulation and Lime Softening with Post-Filter GAC Contactors 7-48

Figure 7-17. Hydraulic Profile - Alternative 3D 7-51
Figure 7-18. Site Plan - Alternative 3D 7-53
Figure 7-19. P&ID - Alternative 3D 7-55
Figure 8-1. Horizontal Paddle Wheel Flocculator 8-6
Figure 8-2. Chain and Flight Sludge Collector 8-7
Figure 8-3. Pressurized Solution Feed System 8-8
Figure 8-4. Filter Underdrain 8-9
Figure 8-5. Filter Building Lower Plan 8-13
Figure 8-6. Filter Building Upper Plan 8-15
Figure 8-7. Chemical Building Plan 8-19
Figure 8-8. Lime Storage and Feed System 8-21
Figure 8-9. Onsite Sodium Hypochlorite Generator 8-21
Figure 8-10. Pretreatment Building Plan 8-25
Figure 8-11. Solids Handling Building Plan 8-27
Figure 8-12. Garage/Maintenance Building 8-31
Figure 8-13. System Architecture 8-39

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List of Abbreviations

ADF .....Average Day Flow
AL..... Action Level
AOC..... Assimilable Organic Carbon
AOP .....Advanced Oxidation Process
ASTM..... American Society for Testing and Materials
AWWA .....American Water Works Association
BDOC..... Biodegradable Organic Carbon
BMP .....Best Management Practice
BVA ..... Buried Valley Aquifer
CCL .....Contaminant Candidate List
CCTV.....Closed Circuit Television
CFATS .....Chemical Facility Anti-Terrorism Standards
CFE .....Combined Filter Effluent
CIP..... Capital Improvement Projects
CMU ..... Concrete Masonry Unit
CT..... Concentration x Time
DAF ..... Dissolved Air Flotation
DBP .....Disinfection By-product
DEFA.....Department of Environmental and Financial Assistance
DOC.....Dissolved Organic Carbon
DWAF ..... Drinking Water Assistance Fund
EBCT ..... Empty Bed Contact Time
EDC.....Endocrine Disrupting Compounds

EPA.....Environmental Protection Agency
FBRR..... Filter Backwash Recycling Rule
FRP.....Fiberglass Reinforced Plastic
FPA.....Facilities Planning Area
FPM..... Feet Per Minute
GAC..... Granular Activated Carbon
GIS..... Geographic Information System
GMR.....Great Miami River
GPCD..... Gallons per Capita per Day
GPM.....Gallons Per Minute
HAA.....Haloacetic Acid
HI .....Hydraulic Institute
HP ..... Horsepower
HVAC.....Heating, Ventilation and Air Conditioning
IESWTR..... Interim Enhanced Surface Water Treatment Rule
IFE ..... Individual Filter Effluent
IOC .....Inorganic Chemicals
LT2ESWTR..... Long Term Enhanced Surface Water Treatment Rule
LCR .....Lead and Copper Rule
LPRO.....Low Pressure Reverse Osmosis
LRAA .....Locational Running Annual Average
MCL.....Maximum Contaminant Level
MCLG.....Maximum Contaminant Level Goal
MDF .....Maximum Day Flow
MF.....Microfiltration
MG ..... Million Gallons
MGD..... Million Gallons per Day
mg/L..... Milligrams per Liter
MIB 2-Methylisoborneol

MIEX.....Magnetic Ion Exchange
MOR.....Monthly Operating Reports
MRDL ..... Maximum Residual Disinfectant Level
MTBE ..... Methyl-t-Butyl-Ether
MVRPC ..... Miami Valley Regional Planning Commission
NDMA ..... N-nitrosodimethylamine
NEPA .....National Environmental Protection Act
NF ..... Nanofiltration
NOM ..... Natural Organic Matter
NPDWR ..... National Primary Drinking Water Regulation
NSDWS .....National Secondary Drinking Water Standards
NTU ..... Nephelometric Turbidity Units
ODNR ..... Ohio Department of Natural Resources
OHPO .....Ohio Historic Preservation Office
O&M .....Operation and Maintenance
OPWC..... Ohio Public Works Commission
OSHA.....Occupational Safety and Health Administration
OWDA .....Ohio Water Development Authority
PAC .....Powdered Activated Carbon
PER.....Preliminary Engineering Report
PPCP.....Pharmaceuticals and Personal Care Products
PVC..... Polyvinyl Chloride
PWS..... Public Water System
RAA ..... Running Annual Average
SCADA .....Supervisory Control and Data Acquisition
SDS..... Simulated Distribution System
SDWA ..... Safe Drinking Water Act
SOC .....Synthetic Organic Chemicals
SF .....Square Feet
SWD .....Side Water Depth

SWTP ..... **Surface Water Treatment Plant**

SWTR ..... **Surface Water Treatment Rule**

TAZ ..... **Traffic Analysis Zone**

TBD ..... **To Be Determined**

TCR ..... **Total Coliform Rule**

TOC ..... **Total Organic Carbon**

TON ..... **Threshold Odor Number**

TSS ..... **Ten States Standards**

TT ..... **Treatment Technique**

TTHMFP ..... **Total Trihalomethane Formation Potential**

TTHM ..... **Total Trihalomethane**

UCMR ..... **Unregulated Contaminant Monitoring Rule**

UF ..... **Ultrafiltration**

UFW ..... **Unaccounted For Water**

VOC ..... **Volatile Organic Chemicals**

VFD ..... **Variable Frequency Drive**

WQ ..... **Water Quality**

WSRLA ..... **Water Supply Revolving Loan Account**

WTP ..... **Water Treatment Plant**

Executive Summary

Background

To address the concerns of the existing water treatment plant, the City decided to investigate the opportunity to invest in a new Surface Water Treatment Plant (SWTP) at a new location on State Route 66. The purpose of this Preliminary Engineering Report (PER) is to:

- Determine treatment goals for the new plant
- Determine capacity requirements for the new plant
- Identify potential future regulatory requirements
- Assess the condition of facilities at the current plant that have potential for rehabilitation and integration with the new plant
- Identify process train alternatives for the new plant
- Perform an assessment of shortlisted process train alternatives
- Develop a cost summary for shortlisted process train alternatives
- Assist the City select a preferred alternative for new SWTP
- Develop a proposed implementation plan to define future tasks, schedule and funding opportunities needed to construct the new plant

The primary objective of the study is to determine water quality and quantity goals for the new treatment plant and select a preferred process alternative with the goal of continued regulatory compliance.

Water Quality

CDM evaluated the City's historic raw water quality (WQ) data from monthly operating reports and other sampling data with the objective of determining appropriate treatment technologies at the new plant to ensure high quality of finished water that meets all current regulations. Based on a review of the historic WQ data, CDM identified three major parameters of concern in Piqua's raw water: total organic carbon (TOC), atrazine, and hardness.

CDM evaluated the City's historic finished WQto establish a "benchmark" for the new treatment plant. Although finished water atrazine levels are below EPA's Maximum Contaminant Level (MCL) of 3 parts per billion (ppb), seasonal high episodes of atrazine are possible especially in the summer months. EPA may consider re-evaluating the atrazine MCL in the near future, so it is recommended that the City plan for this possibility in any future design.

CDM identified three major water quality concerns in the City's raw water: TOC, atrazine and hardness.

TOC levels must be closely monitored since naturally occurring organic materials are the precursors for Disinfection By-Product (DBP) formation in the distribution system. DBPs have detrimental health effects and, as a result, EPA has set the MCL for two significant DBPs: Total Trihalomethanes (TTHMs) and Haloacetic Acids (HAAs) at 80 ppb and 60 ppb respectively. CDM evaluated the City's current treatment plant performance in removing the TOCs and found that on an average the current plant removes approximately 50% of the TOCs. Only 25% removal is required by regulations. However, a close review of the City's historic TTHM and HAA data indicated that although the City is in compliance with the MCLs for these two compounds in accordance with the reporting requirements of the current regulations, meeting future regulations could be a challenge. CDM extrapolated the historic data for TTHMs and HAAs to see if the same data set would meet the reporting requirements of regulations which are scheduled to be effective October 2013 and found that it did not. Hence, TOC levels and treatment technologies to address the levels are a main issue for future treatment requirements.

It is unlikely regulatory compliance can be maintained by the current water plant without significant changes to treatment processes.

CDM set the finished WQ goals for the new plant based on the following:

- Review of historic composite raw WQ,
- Review of efficiency of existing treatment processes in removing contaminants,
- Review of USEPA and Ohio EPA drinking quality regulations, and
- Applying lessons from similar SWTPs in Ohio.

The WQ goals for the new plant were also discussed and agreed upon with the City in a workshop.

Capacity Requirements

Once (WQ) goals were established for the new plant, the next step was to define the quantity i.e, production goals for the new plant. CDM obtained water consumption data from the City billing records from 2006-2010 and analyzed the data by customer usage class, including residential, commercial, and industrial in order to establish a demand history.

In order to project future water demands, it is necessary to take into account both residential growth as well as industrial and commercial growth. Hence both population projections as well as landuse projections were determined. The first step was to obtain the Miami Valley Regional Planning Commission (MVRPC) Traffic Analysis Zone (TAZ) geographic information system (GIS) shapefiles that included a spatially mapped future population forecast (interpreted as 2030) and the 2000 census population data across the several counties that MVRPC supports. This MVRPC population projection is used by other entities for forecasts and was selected for use here in water utility planning. The MVRPC projection data were used to determine an annual growth rate for Piqua which was then used to project the population to the year 2031 which is approximately 20,039.

The next step was to obtain both current and future landuse data from MVRPC and Miami County to establish the future additional water service area for two major categories such as industrial and commercial. To quantify the change in water demands between current and future conditions, CDM combined the projected City of Piqua and MVRPC land use conversion for industrial and commercial customers, and the additional population for residential areas currently outside of the City as defined by MVRPC TAZ projections. Both factors were considered to develop the 20-year water demand.

Land use conversion was used as the basis of future commercial and industrial demands by applying a per acre water demand (obtained from existing customers) across the future development areas. The current demand for Piqua's water system was used as the starting point for future projections. The current demand was added to the future residential component (based on population) and the future non-residential component (based on industrial and commercial land use) to develop the future average day demand of 4.35 million gallons per day (MGD). The total future maximum day demand calculated by the above approach was approximately 9 MGD.

Total future maximum day demand is 9 MGD with the initial phase consisting of three 2.25 MGD treatment trains .

An analysis of historic maximum day demands over the past years shows that the maximum demand occurred in 2007 and was 4.73 MGD. Considering the potential for less aggressive development outside the City and a mid range in residential population growth, a future treatment plant of approximately 9 MGD is recommended to be planned for when siting the new plant. Because 9.0 MGD appears excessive for near-term needs, CDM recommends phasing the plant's construction such that any future expansion could be accommodated to achieve the 20-year 9.0 MGD capacity.

CDM presented two options to the City in a workshop. The first option is to construct two 3 MGD treatment trains and the second is to construct three 2.25 MGD treatment trains. Because of the operational flexibility and redundancy that the second option offers, the City selected three 2.25 MGD treatment trains for a total initial capacity of 6.75 MGD for the new plant.

Regulatory Requirements

Several regulations will impact treatment processes selected for the new plant but the most significant one is the Stage 2 Disinfectant and Disinfection By-Products (D/DBP) Rule. Historically, the City has been in compliance with Stage 1 D/DBP Rule requirements because compliance was based on averaging results from samples collected throughout the distribution system.

The primary water quality and treatment goals are driven by the new Stage 2 D/DBP Rule.

Piqua's first year of compliance monitoring under the new Stage 2 D/DBP Rule is October 1, 2013 and the City must be in full compliance by October 1, 2014.

The implementation of the new Stage 2 D/DBP Rule will require that compliance be met at each sampling location, and the current system-wide averaging will not be allowed. According to the Stage 2 D/DBP Rule, the timeline for Schedule 3 systems, such as Piqua, for the first year of compliance monitoring is October 1, 2013 and must be in compliance with the Stage 2 D/DBP rule MCLs at the end of a full year of monitoring.The City has already identified four compliance monitoring locations to Ohio EPA. CDM performed a data analysis of the TTHM and HAA data from the City. Based on the findings, it is unlikely that the City will be in compliance with the Stage 2 D/DBP Rule requirements without making significant changes to the treatment practices at the plant.

Condition Assessment of Existing Facilities

CDM evaluated several of the City's existing water treatment facilities in order to determine how they can be modified and/or integrated into the new treatment plant. The facilities that were evaluated are:

- River Pumping Station
- Gravel Quarry Pump Station
- Raw water intakes and screen/valve chambers at Swift Run Lake
- Sludge disposal force main and lagoon
- Filter Backwash and Sludge Disposal Facility

Each facility was assigned a condition score based on its physical condition, consequence of failure, redundancy, regulatory/code compliance, and health and safety compliance. The river pump station is generally in a poor condition and is located in the flood plain. Significant rehabilitation measures are required to continue using this pump station with the new treatment plant. The gravel quarry pump station is in fair condition, is located in the flood plain, but can possibly be used with the new plant with some modifications. Due to the poor condition of the screen chambers, CDM recommended that the City replace both the screen chambers. The offsite sludge disposal lagoon is generally in fair condition and could be used with the new plant if the City does not desire to invest in an onsite mechanical sludge deatering facility at the new plant. CDM recommended that the City add another lagoon for long term redundancy. The filter backwash and sludge disposal facility at the current plant is in good condition and has the potential to be used with the new plant.

Process Alternatives Development and Screening

A workshop was held with the City to evaluate process train alternatives for the new plant.

CDM presented ten process train alternatives for the new plant to the City at Workshop No. 1 (Process Evaluation Workshop) and the initial screening of the ten process train alternatives down to four was completed jointly by the CDM team and City staff at the workshop. The process train alternatives were developed for the City's new SWTP with the aim of meeting the following major WQ goals which were jointly established by the City and CDM:

- Turbidity
- Disinfection
- TOC/DBP
- Atrazine
- Taste and Odor
- Hardness
- Endocrine Disrupting Compounds (EDCs)/Pharmaceutical and Personal Care Products (PPCPs)

Of the ten process train alternatives presented, six were conventional treatment and four were membrane treatment alternatives. The preliminary screening was performed using the following five criteria:

- Ability to meet WQ goals - 30%

- Operation and Maintenance (O&M) -20 %
- Reliability and risk- 15%
- Implementation- 10%
- Relative cost- 25%

Weights as shown above were assigned to the above screening criteria at the workshop and the alternatives were ranked and the top four alternatives were selected for further evaluation by CDM.

Process Alternatives Assessment

CDM developed design criteria, preliminary hydraulic profile, site layouts, preliminary opinion of probable construction costs and 20-yr present worth costs for four shortlisted alternatives. The shortlisted alternatives are:

- Alternative 1B – Lime Softening with Ozone
- Alternative 1D – Lime Softening with Post-Filter Granular Activated Carbon (GAC) Contactors
- Alternative 3B – Enhanced Coagulation and Lime Softening with Ozone
- Alternative 3D – Enhanced Coagulation and Lime Softening with Post-Filter GAC Contactors

The design criteria for the unit processes for all four alternatives were developed in accordance with the requirements of Ten States Standards (TSS) and Ohio EPA's Approved Capacity document. Bench scale preoxidation studies and jar testing studies were also conducted and the results summarized. CDM assumed that the Great Miami River (GMR), Swift Run Lake (Reservoir) and the Gravel Quarry, which currently provide surface water supply for the City's existing WTP, will continue to provide raw water supply to the new plant also. To augment the surface water sources with groundwater, the City also decided to investigate the potential for groundwater in the area of the new plant as part of this study. As a result, CDM and its subconsultant were tasked with estimating the production potential from the recently identified buried valley aquifer (BVA) in the vicinity of the proposed plant. The study identified up to 2 MGD from the well sites under investigation. Because the implementation period for refining the potential safe yield and developing a groundwater source will extend significantly beyond the regulatory compliance deadlines, the City confirmed that the approach will be to construct a SWTP that can be effectively operated with a blend of surface and groundwater in the future.

CDM developed 20-yr present worth costs for the four shortlisted alternatives.

The results of process alternatives assessement were presented to the City at Workshop No. 2 (Process Decision Workshop). The four shortlisted alternatives were screened using the same preliminary screening criteria used at Workshop No.1. However, the 'Relative Cost' criterion was replaced with Capital and Present Worth O&M costs. The weights allocated to the subcriteria were changed based on discussions with the City at Workshop No.2. Each process train alternative was also assigned a score from 1 to 5 (5 being the best score) for each of the five screening criteria. Based on the score, the City selected Alternative 1D – "Lime Softening with Post-Filter GAC Contactors" for the new plant.

The City selected "Lime Softening with Post-Filter GAC Contatcors" for the new plant at the second workshop.

Selected Alternative

CDM prepared an Approved Capacity table for the selected alternative in accordance with Ohio EPA requirements. Several equipment options are available for each process for the selected alternative; however, equipment selection determinations will be made during detailed design phase based on discussions with the City. Conceptual design considerations for various categories of construction such as civil/site work, structural, architectural, heating, ventilation and air conditioning (HVAC), plumbing and fire protection, electrical, instrumentation and controls were also developed.

Implementation Plan

The overall construction of the new plant can be broken down into three separate elements such as water supply, offsite piping and water treatment plant construction. Construction of the WTP is expected to require a minimum 18 months for substantial completion using conventional design/bid/build procedures. If design is commenced in March 2012, substantial completion of the project would be expected in October 2014. With the first round of testing for the location running annual average (LRAA) data collection in late 2013, and the second round in early 2014, it may be possible for the individual samples needed for the LRAA to be maintained within the MCL. However with the summer 2014 samples it is unlikely that the City will be in compliance. If acceleration of the project schedule is desired there are several approaches that may improve the schedule delivery after the decision is made in early 2012 such as:

- Accelerated design schedule
- Pre-purchasing long lead items
- Offering a bonus for early completion with a cap on the maximum
- Procuring the facility construction through an alternative method such as design/build (D/B). Recent legislation passed in Ohio has expanded the opportunities for this approach to be considered by the State and by communities such as Piqua.

Several funding options are available to the City to help offset or reduce the overall costs of design and construction such as:

- Drinking Water Assistance Fund (DWAF), through Ohio EPA/Department of Environmental and Financial Assistance (DEFA)) under the Water Supply Revolving Loan Account (WSRLA)
- Ohio Water Development Authority (OWDA)
- Ohio Public Works Commission (OPWC)
- Conventional bonding

Section 1
Introduction and Project Background

The City of Piqua currently owns and operates a Surface Water Treatment Plant (SWTP) that was originally constructed in 1925. It is rated for 7 MGD. The original plant consisted of a low service pump station at the Great Miami River (GMR), screen chamber at Swift Run Lake, flocculation basin, clarifier, settling basin, recarbonation tank, filter building, pump room and a clearwell. The water treatment plant (WTP) has undergone the following improvements and additions in the years since the original construction in 1925:

- 1961 – Flocculation basin, clarifier, settling tanks, filters, clearwell, and high service pumps
- 1975 – Sludge disposal line between the treatment plant and the off-site lagoon
- 1976 – Fluoridation equipment
- 1980 – Miscellaneous concrete repair work on various plant structures and overflow spillways
- 1983 – Sludge and Filter Backwash Disposal facility
- 1990 – Chlorine and fluoride building
- 1993 – Clearwell baffles

The current plant provides conventional treatment through clarification, lime-soda softening, stabilization, filtration, fluoridation, and disinfection. These are described in detail below.

Water supply is obtained from three surface water sources: GMR, an abandonded gravel quarry (pit) located on the east side of the GMR, and the City's Hydraulic System that comprises a series of lakes of which Swift Run Lake (or the Reservoir) is the largest.

Raw water from the gravel pit and the GMR tie into a single line after the River Pump Station and are pumped to the treatment plant. Water from Swift Run Lake flows by gravity to the plant through one of two screen chambers. Treatment chemicals such as potassium permanganate, powdered activated carbon (PAC), lime, soda ash, ferric sulfate and caustic soda are typically added to the rapid mix channel at the plant to promote adequate mixing. The rapid mix channel is immediately followed by a flume where the water is split between two flocculation basins which are equipped with horizontal mechanical flocculators to promote floc formation. Following flocculation, flow is split between two clarifiers. Each clarifier is equipped

with mechanical sludge collection equipment to scrape settled sludge to a centrally located sump from where the sludge is withdrawn on a continuous basis. Sludge collected from the clarifiers is usually returned to the head of the plant to the rapid mix channel or wasted to the sludge and filter backwash disposal facility located across the road. From the backwash holding basins, the sludge is pumped to an off-site lagoon.

There are two rectangular settling basins. Both are equipped with chain and flight sludge collection mechanism that convey the settled solids to a hopper located at the inlet end of the basins. Sludge is periodically withdrawn from the sedimentation basins and sent to the sludge and filter backwash disposal facility and then transferred to the lagoon.

The current plant uses carbon dioxide for lowering the pH of the lime softened water and to stabilize the softened water by a process called recarbonation which is followed by two post-recarbonation settling basins. Following recarbonation and settling, filtration occurs in six rapid sand filters with a mix of sand and anthracite with a gravel support bed. There are a total of eight filters but two have been out of service for several years. Wash water for backwashing the filters is drawn from the newer clearwell by a wash water pump. Spent backwash water is conveyed by drains to the backwash sludge and filter backwash disposal facility. Decant waste water from the basin is recycled to the head of the plant with pumps located in the pump station associated with the sludge and filter backwash disposal facility.

Chlorine and fluoride are added to the filtered water and stored on site in two below-ground clearwells each with a capacity of approximately 550,000 gallons.

The existing plant has the following deficiencies:

- Treatment plant equipment has exceeded or is approaching the end of its useful life. Some equipment has been in continuous service since 1925.
- Some structures are also approaching end of their useful life. Some have been in continuous service since 1925 and have been repaired to extend structural integrity.
- Raw water and high service pump discharge metering is inaccurate.
- There is insufficient redundancy for processes and equipment which means the current plant may not be able to meet capacity requirements when one tank is taken out of service for maintenance.
- High service pumps do not have standby electric power.
- Chemical storage and handling practices at the plant are obsolete, labor intensive and unsafe.
- Existing plant is located in a flood-prone area and has been flooded in the past.
- The clarification and settling tanks do not meet Ohio Environmental Protection Agency's (EPA's) Approved Capacity requirements for detention time of at least 240 minutes.

The plant meets current regulations but meeting future regulations in a reliable manner could be a challenge.

To address the concerns of the existing plant, the City decided to investigate the opportunity to invest in a new SWTP at new location on State Route 66. The purpose of this Preliminary Engineering Report (PER) is to:

- Determine treatment goals for the new plant
- Determine capacity requirements for the new plant
- Identify potential future regulatory requirements
- Assess the condition of facilities at the current plant that have potential for rehabilitation and integration with the new plant
- Identify process train alternatives for the new plant
- Perform assessment of shortlisted process train alternatives
- Develop cost summary for shortlisted process train alternatives
- Assist City select a preferred alternative for new SWTP
- Develop a proposed implementation plan to define future tasks, schedule and funding opportunities needed to construct the new plant

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Section 2
Water Quality

CDM evaluated the City's historic raw water quality (WQ) data from monthly operating reports and other sampling data with the objective of determining appropriate treatment technologies at the new plant to ensure high quality of finished water that meets all regulations. Finished WQ goals for the new water treatment plant have been defined by addressing current WQ parameters of concern in the source water. These are described in this Section.

2.1 Review of Existing Data

The following reports and sampling data were provided to CDM and were analyzed to evaluate and summarize the current plant performance:

- 2005-2010 Monthly Operating Reports (MORs)
- City's annual WQ reports from 2006-2010
- Total Organic Carbon (TOC) Monitoring Reports and compliance data from 2005-2010

The City obtains its water supply from three surface water sources: GMR, a gravel pit located on the east side of the river and a Reservoir which comprises of a series of small lakes such as Swift Run Lake, North Pond, Echo Lake, Frantz Pond and connecting canals. A summary of the composite raw WQ data for Piqua is shown in **Table 2-1**. The plant composite data includes contributions from all three water sources and hence can vary widely. WQ data from individual water sources is not available except TOC data. WQ parameters of concern in Piqua's raw water sources that impact treatment needs include:

- TOC
- Atrazine
- Hardness

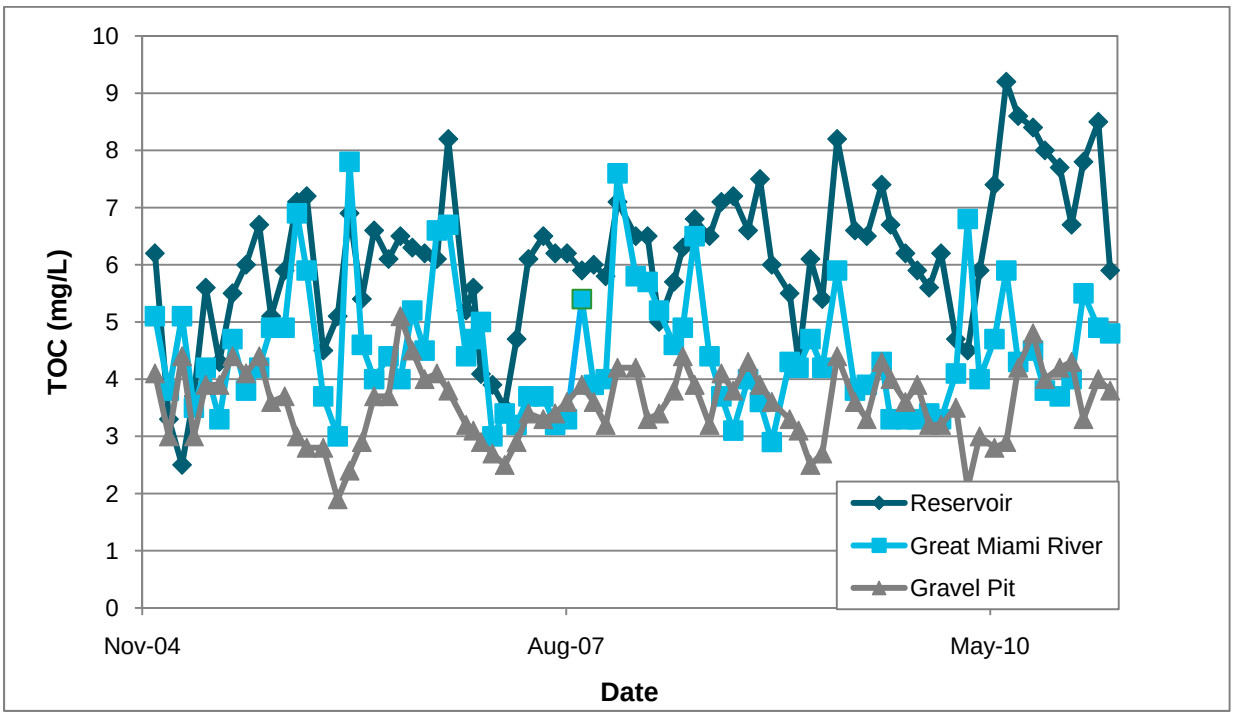
Table 2-1. Summary of Raw WQ Data (2005-2010)

Parameter	Minimum	Maximum	Average
Total Hardness (mg/L as CaCo3)	176	336	252
Total Alkalinity (mg/L as CaCo3)	146	288	208
pH	7.9	8.7	8.3
Turbidity (NTU)	4	56	19
Magnesium (mg/L)	30	114	82
Nitrates (mg/L)	0.32	8.06	1.7
Fluoride (mg/L)	0.22	0.53	0.35
TOC-Plant Composite *(mg/L)	1.9	7.3	4.4
TOC-Gravel Pit (mg/L)	1.9	5.1	3.6
TOC-Reservoir (Swift Run Lake) (mg/L)	2.5	9.2	6.1
TOC-Great Miami River (mg/L)	2.9	7.8	4.5
Atrazine-Gravel Pit (µg/L)	0.03	1.5	0.1
Atrazine-Reservoir (µg/L)	0.03	85	3.3
Atrazine-Great Miami River (µg/L)	0.03	33	1.2

2.1.1 TOC

TOC levels must be closely monitored since naturally occurring organic materials are the precursors for Disinfection By-Product (DBP) formation in the distribution system. A plot of the raw water TOC is shown in **Figure 2-1**. The reservoir has the highest TOC levels and the greatest variability based on the range of values.

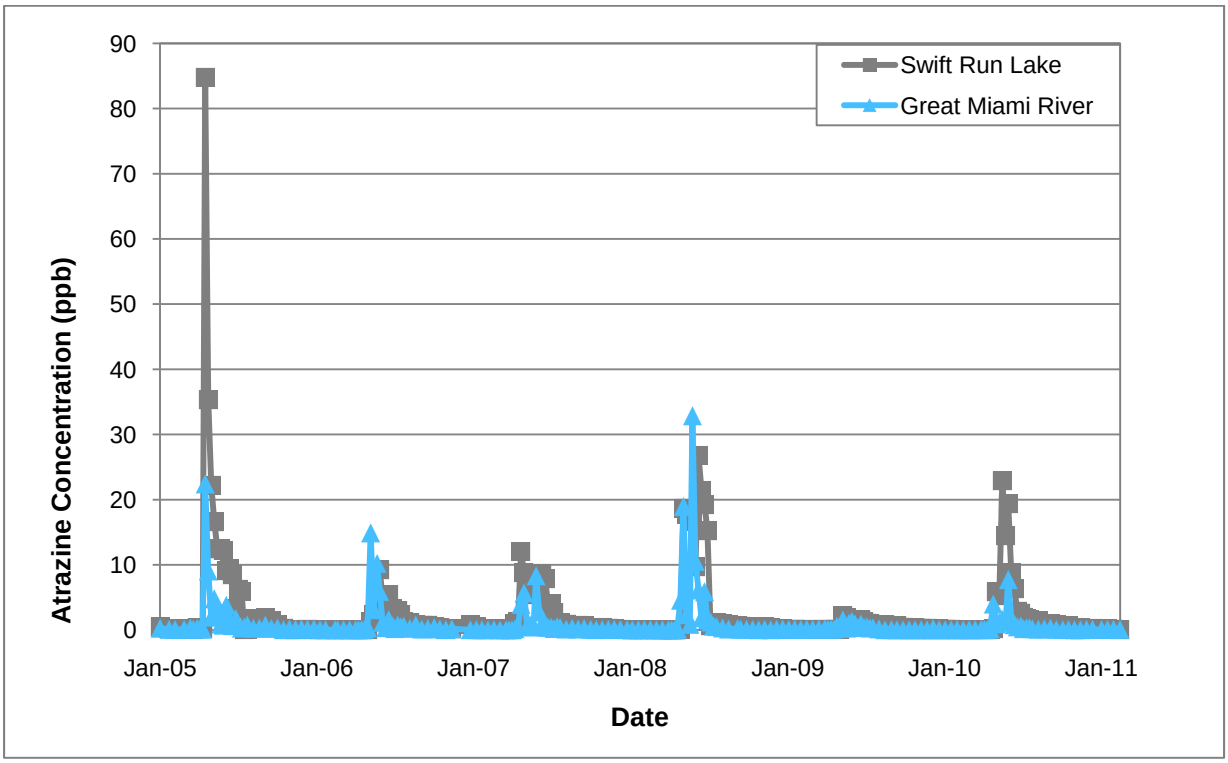
Figure 2-1. *Raw Water TOC*



2.1.2 Atrazine

Atrazine is a commonly used herbicide in agricultural applications. It is a regulated compound under the phase II rule as a synthetic organic chemical and standardized monitoring framework. For a system that serves more than 3,300 people such—as Piqua—that detects atrazine during routine monitoring, four quarterly samples are required at the entry points to the distribution system if detection is not consistently below the maximum contaminant level (MCL). The MCL for atrazine in finished water is 3 micrograms per liter (µg/L) based on a running annual average (RAA), and must be monitored if detected. In addition to the MCL, Ohio EPA also checks for individual test results that exceed four times the MCL (i.e., 12 µg/L). A plot of raw water atrazine from Swift Run Lake (reservoir) and Great Miami River based on data collected over the past five years is shown in **Figure 2-2**. The Gravel Pit had a very low concentration of atrazine (0.03-1.52 µg/L) and is not shown in the plot.

Figure 2-2. *Raw Water Atrazine*



Piqua has a voluntary atrazine monitoring program in place on all three water sources with bimonthly sampling. The Ohio EPA Inland Lakes Program, in 2008, initiated an extensive study of Swift Run Lake, the largest of the lakes in the three-lake system. Among the parameters they measured were atrazine concentrations in the lake at one to two-week intervals, from May to August, at the surface and at the bottom of the lake. It is recommended that the City continue to monitor atrazine concentrations and make efforts to reduce the levels of atrazine in both the raw and finished water supply. Treatment strategies selected for the new treatment plant should proactively address atrazine removal.

Of the three raw water sources, the reservoir has the highest atrazine levels. This can be controlled to some extent through proper water management practices such as restricting water pumping from the reservoir especially during summer months which are periods of likely high atrazine concentrations.

2.1.3 Hardness

The raw water is considered to be "hard" if the total hardness ranges between 176 and 336 milligrams per liter as calcium carbonate (mg/L $CaCO_3$). The existing softening process reduces the hardness to an average value of 108 mg/L $CaCO_3$. Carbonate hardness is removed using lime and then soda ash is added to remove non-carbonate hardness.

The City intends to continue the use of the three current water sources for the new treatment plant. Therefore, the raw water qualities discussed above will be an important consideration in the development of recommended treatment process design. Potential addition of groundwater to supplement the gravel pit supply in the future will be addressed after the analysis of groundwater resources is completed; however for initial planning purposes it is assumed that only the existing sources will be available for the new WTP.

2.2 Finished Water Quality

Table 2-2 shows a summary of the finished WQ data for Piqua based on MOR data from between 2005 and 2010.

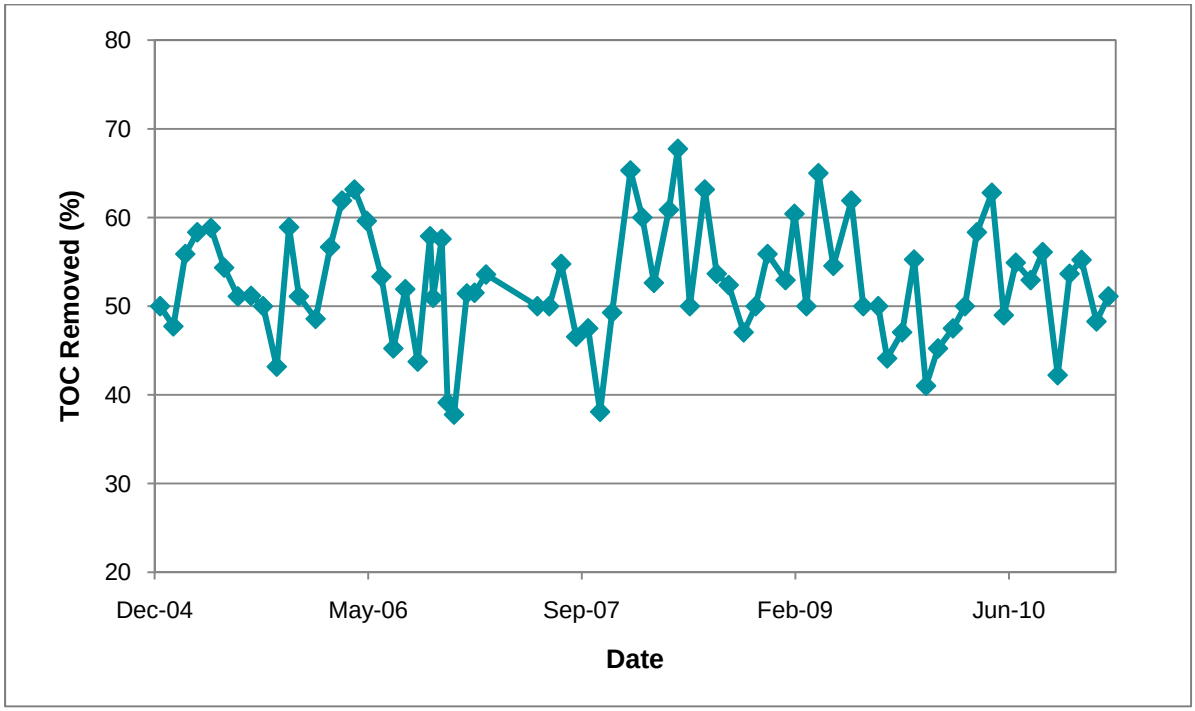
Table 2-2. Summary of Finished WQ Data (2005-2010)

Parameter	Minimum	Maximum	Average	MCL	MCLG
Total Hardness (mg/L as CaCo3)	85	138	108	-	-
Total Alkalinity (mg/L as CaCo3)	42	84	59	-	-
pH	9.7	10	9.8	-	-
Turbidity (NTU)	0.06	0.2	0.09	NA	TT
Chlorine (mg/L)	1.4	2.0	1.6	MRDL=4	MRDLG=4
Total Coliforms per 100 mL (#)	0	<1	< 1	TT	-
Nitrates (mg/L)	0.4	5.2	1.7	10	10
Fluoride (mg/L)	0.9	1.1	1.0	4	4
TOC (mg/L)	1.5	4.3	2.7	NA	NA
Atrazine (µg/L)	0.08	11.6	1.0	3	3
Simazine (µg/L)	0.4	1.3	0.8	4	4
Lead (µg/L)	<2.0	6.0	< 2	AL=15	0
Copper (mg/L)	<0.01	0.09	0.05	AL=1.3	1.3
Barium (mg/L)	NA	NA	0.005	2	2
Total Trihalomethanes (µg/L)	25	117	70	80	NA
Haloacetic Acids(µg/L)	12	40	25	60	NA

Although finished water atrazine levels are below the MCL of 3 parts per billion (ppb), seasonal high episodes of atrazine are possible especially in the summer months. Atrazine regulatory compliance is based on the RAA. EPA may consider re-evaluating the atrazine MCL is the near future, so it is recommended that the City plan for this possibility in any future design.

Figure 2-3 shows TOC removal through the plant based on raw composite TOC and finished water TOC. A review of historic finished water TOC data over the past five years shows that the average finished water TOC concentration is 2.7 mg/L. A 25% removal is required per the Stage 1 DBP Rule.

Figure 2-3. TOC Removal



A review of the City's historic DBP data such as Total Trihalomethane (TTHM) and Haloacetic Acids (HAA5) data indicate that the system-wide RAA values across their compliance monitoring locations are within the 80 µg/L and 60 µg/L MCLs set by USEPA for TTHMs and HAA5 respectively. These are shown in **Figure 2.4** and **Figure 2.5**.

Figure 2-4. TTHM RAA

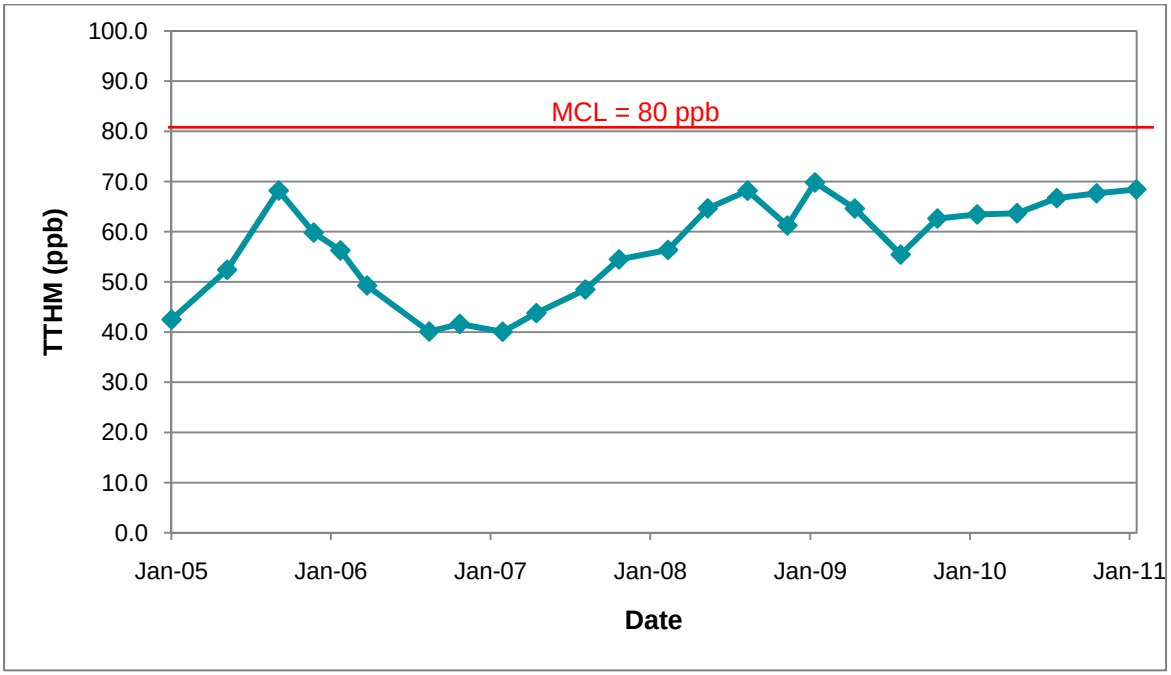


Figure 2-5. HAA RAA

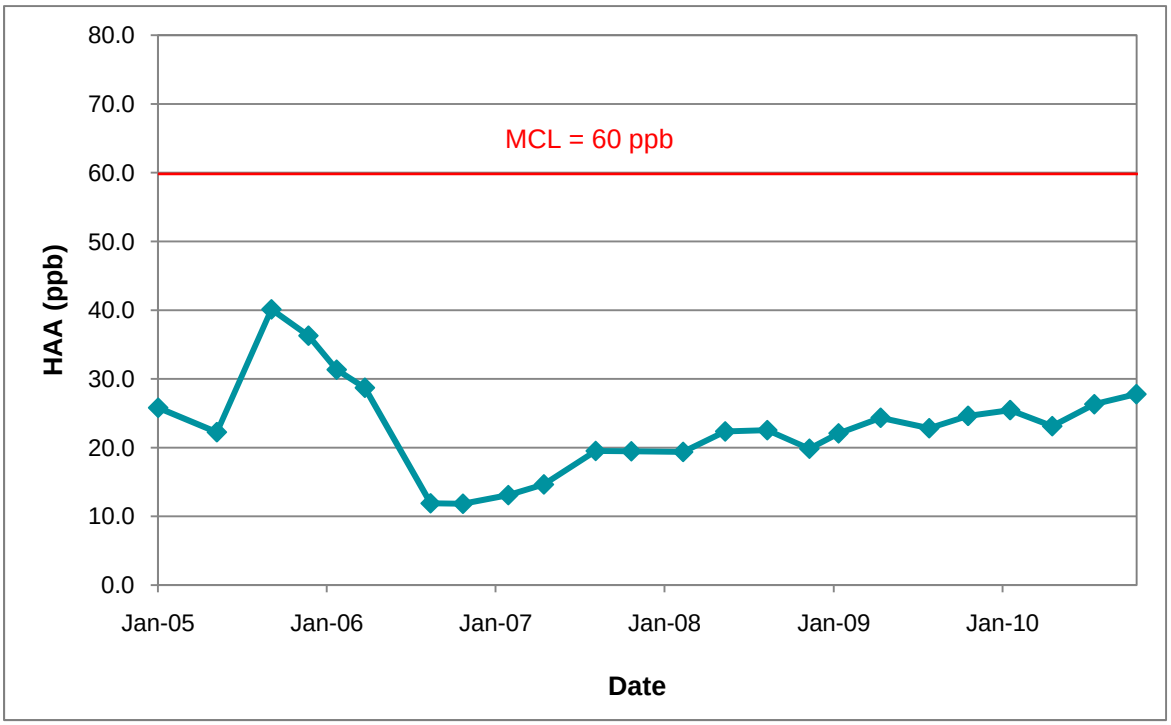


Figure 2-6 shows Locational Running Annual Average (LRAA) TTHM values at each of the City's compliance monitoring locations from 2005. From the figure, it can be seen that the Main Street location

has consistently high TTHM levels above the MCL of 80 µg/Land represents a future compliance challenge with the Stage 2 DBP Rule which will be enforced in October 2013.

The City staff reported periodic flushing at these locations to minimize TTHM concentrations. While periodic flushing may somewhat help in managing distribution WQ, it is not usually a long-term DBP compliance strategy. Studies have shown that flushing can temporarily improve water age and alleviate WQ factors that impact distribution WQ but will not permanently reduce water age.

The LRAA values for HAA at the compliance monitoring locations were well below the MCLs as shown in **Figure 2-7**.

Figure 2-6. *TTHM LRAA at Compliance Monitoring Locations*

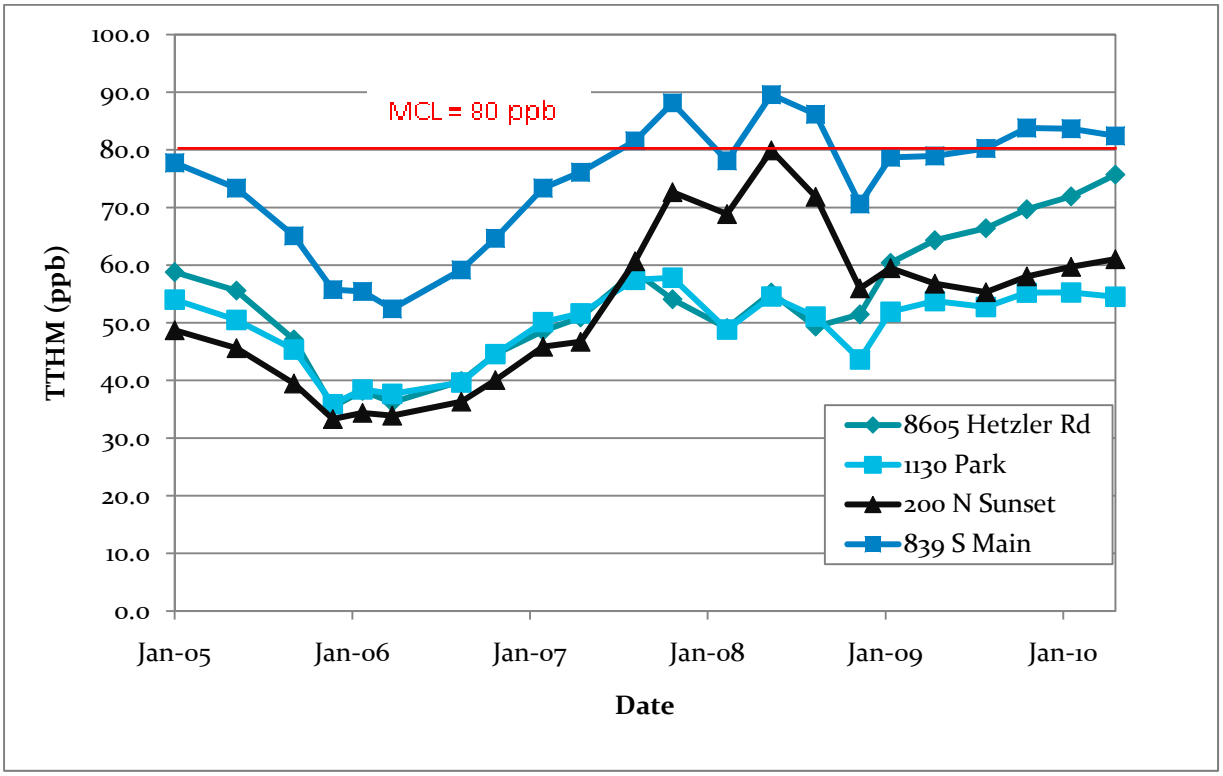
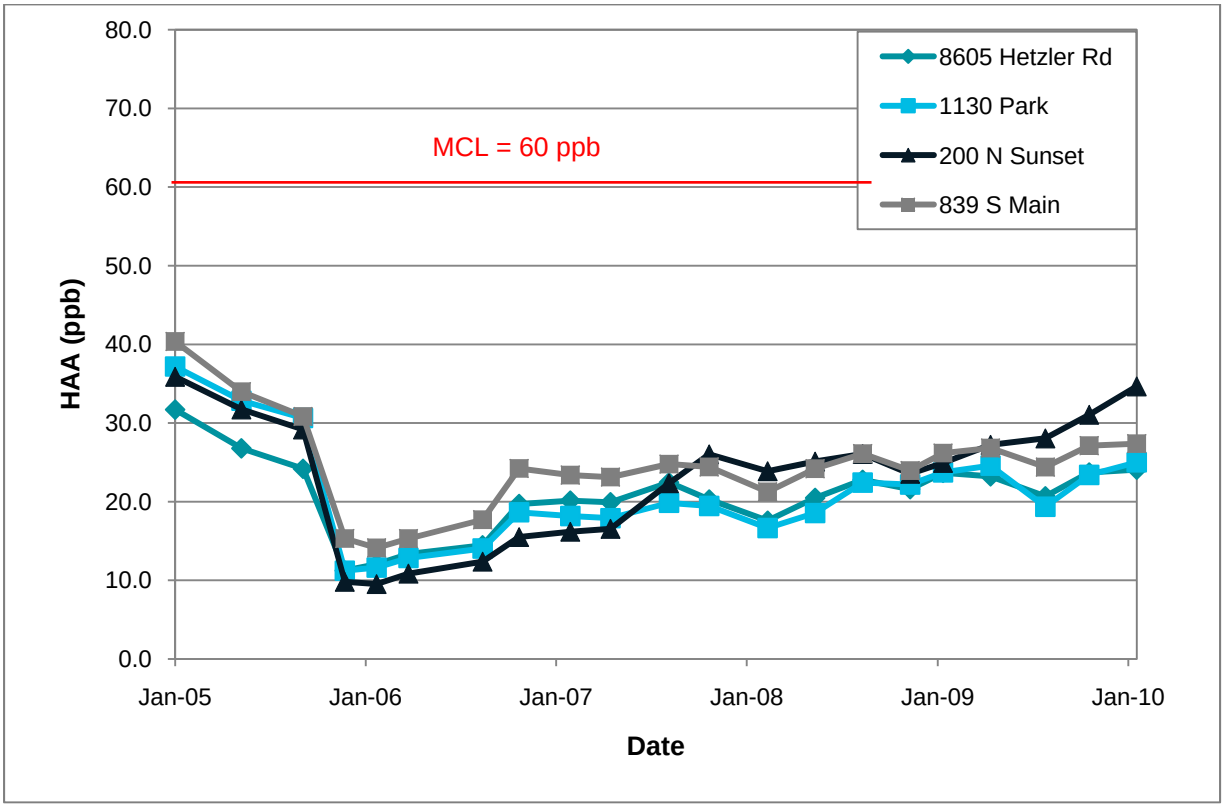


Figure 2-7. HAA LRAA at Compliance Monitoring Locations



2.2.1 Finished Water Quality Goals

It is expected that the new water treatment plant will provide a similar or better performance with respect to reduction of WQ parameters of concern. Finished WQ goals for the City of Piqua's new SWTP are summarized in **Table 2-3.**

The values have been determined by the following methods:

- Review of historic composite raw WQ,
- Review of efficiency of existing treatment processes in removing contaminants,
- Review of USEPA and Ohio EPA drinking-water quality regulations, and
- Applying lessons from similar surface water treatment plants in Ohio.

If the contaminant is not present or is present in low concentrations in the raw water, the requirement is to stay below the MCL since the probability of compliance without treatment is very high. A complete list of MCLs can be viewed at www.epa.gov/safewater/contaminants/index.html.

For those constituents that are presently close to the MCL, a safety factor has been applied to ensure comfortable compliance with the regulation.

Since Piqua's historical finished water average pH value is 9.8 and the City is in compliance with the requirements of Lead and Copper Rule (LCR), it is recommended to maintain the pH at 9.8 for the new treatment plant. However, prior research has shown that as finished water pH decreases, the TTHM

formation also decreases which may be beneficial to the City in controlling the formation of DBPs, especially in the remote portions of the distribution system. It should be noted that reducing the pH may result in an increase in distribution HAA concentrations, but the City's historic finished water HAA concentrations have been much lower than the MCLs. So compliance problems with this group of DBPs at reduced pH are not anticipated. It is important to note that a change in the pH may affect distribution WQ and lead to corrosion issues or jeopardize the City's compliance with the LCR. Any change in pH needs to be evaluated in detail. The City should conduct coupon or pipe loop testing in the near future to study the effect of pH on corrosion control in the distribution system before any operational changes are made to lower pH levels.

There are also additional WQ parameters that are not directly regulated, but are important to consider. For example, total hardness is not regulated; however, finished water with high hardness can cause scaling issues in the distribution system and provide aesthetic issues for customers. An additional aesthetic parameter is taste and odor or rusty water problems. This can lead to additional customer complaints if not handled properly; however, they have no adverse health effects. These parameters have to be addressed during the design of the new water treatment plant (WTP).

Based on a review of historic finished WQ data presented in this Section, the City has not exceeded any current state or federal drinking water regulations but compliance with the Stage 2 DBP Rule may be a challenge in the future. Although the City achieves substantial TOC removal through current treatment practices, distribution system TTHM concentrations are still high. The City should consider other process alternatives at the new treatment plant to address these parameters of concern.

Table 2-3. Finished WQ Goals

Parameter	Maximum Contaminants Level (MCL)[1] or Treatment Technique (TT)	Recommended Goal in Finished Water
Total Hardness (µg/L as CaCo3)	N/A	110[2]
Total Organic Carbon (µg/L)3	25% Removal	TBD[3]
Total Trihalomethanes (µg/L)	80	<64
Haloacetic Acids (µ/L)	60	<48
Total Nitrite + Nitrate (mg/L as N)	10	<5
Atrazine (ug/L)	3.0	<1.0
Iron (mg/L)	0.3	<0.05
Manganese (mg/L)	0.05	<0.02
Fluoride (mg/L)	2.0	0.7 to 1.2[4]
Chlorine (mg/L)	4.0	>0.2
Turbidity (NTU)	< 0.3 in 95% of Samples	< 0.1
pH4	6.5-8.5	9.5-10.0[5]
Stability	N/A	Non-corrosive, slightly depositing
Odor (TON)	3	0
Cryptosporidium	TT[6]	0

1. *The limit listed is either the MCL or the secondary MCL, whichever is more stringent.*
2. *The hardness in the finished water is not a regulated criterion; however, it is an aesthetic issue.*
3. *To be determined (TBD) based on bench scale testing. A minimum removal of 25% removal is required per the Stage 1 DBP Rule*
4. *The Department of Health and Human Services and USEPA are proposing to change optimal Fluoride level to 0.7 mg/L. Current recommended range is 0.7 to 1.2 mg/L based on temperature. This is expected to decrease even further and is currently under review.*
5. *This is a secondary MCL and is not generally enforced. The pH should be maintained at approximately 9.8, unless a corrosion control study is completed to determine whether any adverse impacts exist at a lower distribution system pH.*
6. *Cryptosporidium regulations are based on the number of cysts present in the source water.*

Section 3
Capacity Requirements

The purpose of this Section is to estimate the future service area, land use categories, and population to be served by the City's water system, and to determine the future design capacity of the new WTP based on a planning period of 20 years (2011-2031).

3.1 Data Sources

The following data sources and documents were used for this study:

- City of Piqua-water billing records from 2006-2010
- City of Piqua-water treatment plant production data from 2006-2010
- Geographic Information Systems (GIS) data from Miami Valley Regional Planning Commission (MVRPC)
- "Plan It Piqua– Redevelopment Analysis Report" dated April 2010
- "Water Treatment Plant Assessment and Master Plan" dated 2007
- "Plan It Piqua-Comprehensive Plan Update" dated 2007

3.2 Service Areas

For purposes for this project, the planning area includes the water service area currently served by the City of Piqua and includes additional future water service areas outside the current City boundary. A map of the planning area is shown in Figure 3.1. The service area boundaries were initially established at a meeting on April 25, 2011 with City of Piqua staff. The northern and southern boundaries were specifically addressed at that meeting and the east-west extent of the service area was generally established as being consistent with the Section 208 Facilities Planning Area (FPA) boundary. Upon further review of the area, CDM identified the east and west boundaries shown on Figure 1 that are less extensive than the FPA, but still represent a major service area extension.

Based on GIS data obtained from MVRPC, the City of Piqua incorporated area currently encompasses approximately 7,180 acres of which 3,903 acres is developed. The combination of City incorporated area and current water service area outside the City totals 7,737 acres. As the City continues to grow and develop, additional area is anticipated to be annexed and provided water service, or otherwise furnished water through contract agreements. Based on review of MVRPC data and planning documents, listed above, the expanded future water service area depicted in **Figure 3-1** is anticipated to encompass 13,240 acres for a 20-year planning horizon.

3.3 Demand History

The City measures the finished water flow leaving the plant through two water meters. The readings from these meters are totaled to calculate the total water production that is delivered to the water distribution system. **Figure 3-2** presents the monthly average day and maximum day demands obtained from plant production data for the past five years (2006-2010). **Table 3-1** shows the annual average day and maximum day demands for the same period with a calculated peaking factor for each year. Peaking factor is the ratio of maximum day production to the average day production.

Table 3-1. Water Production History

Year	Max Day (MGD)	Avg Day (MGD)	Max/Avg Day Ratio
2006	4.20	2.95	1.42
2007	4.73	3.10	1.52
2008	4.00	3.03	1.32
2009	4.25	2.74	1.55
2010	3.61	2.74	1.32
AVG	--	2.91	--
Max Peaking Factor			1.55

Average water production of 2.91 million gallons per day (MGD) has been very consistent, with production records in the past five years within 10% of the average. The maximum day production of 4.73 MGD occurred in May 2007.

CDM obtained water consumption data from the City billing records from 2006-2010 and analyzed the data by customer usage class, including residential, commercial, and industrial. Results are summarized in **Table 3-2** along with the per capita consumption as gallons per capita per day (GPCD). Population for 2010 was based on census data and population for other years was interpolated from 2000 and 2010 census data. Note that for industrial and commercial categories, this represents the average consumption per customer account instead of per capita.

***Figure 3-1**. Planning Area*

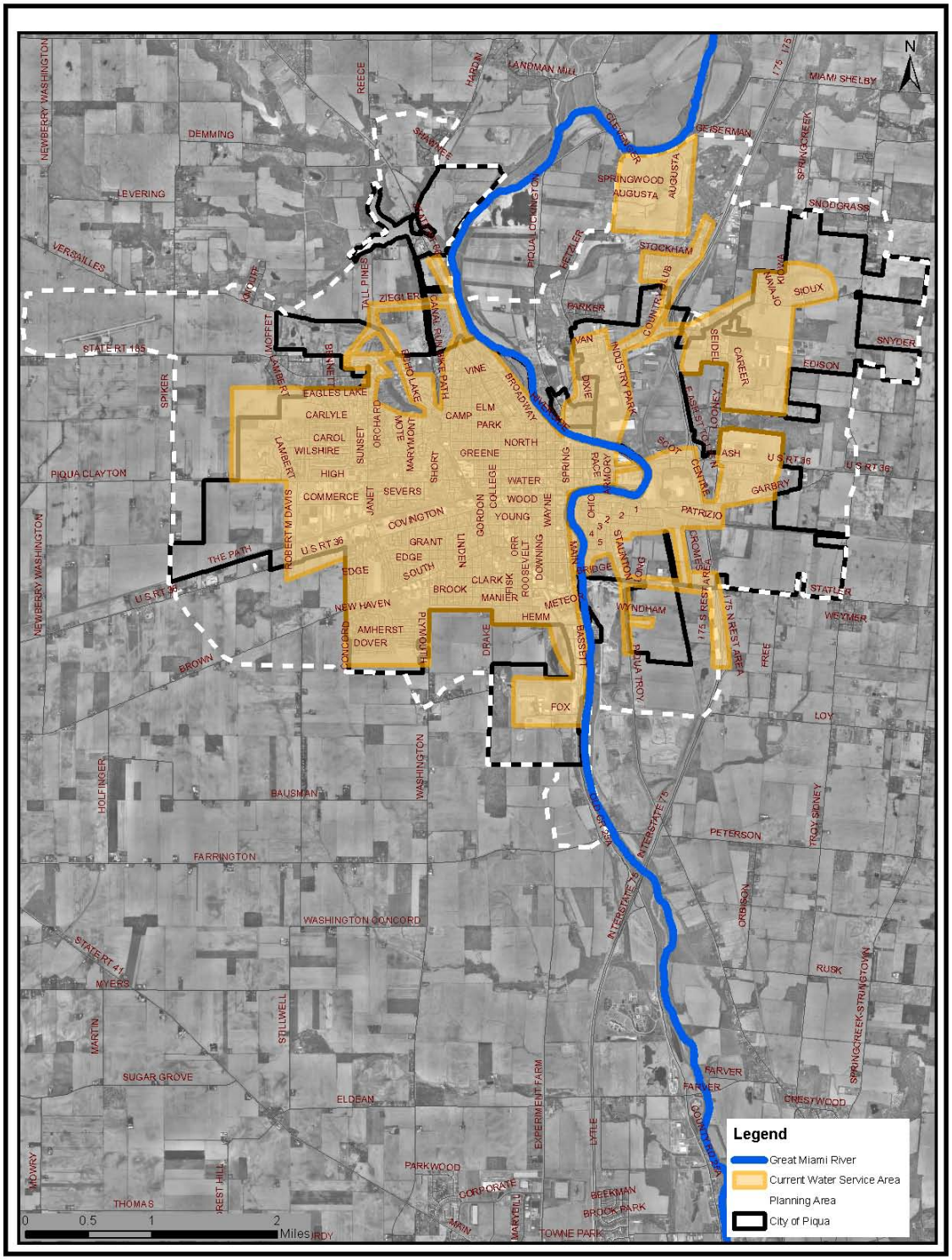


Figure 3-2. Water Production History

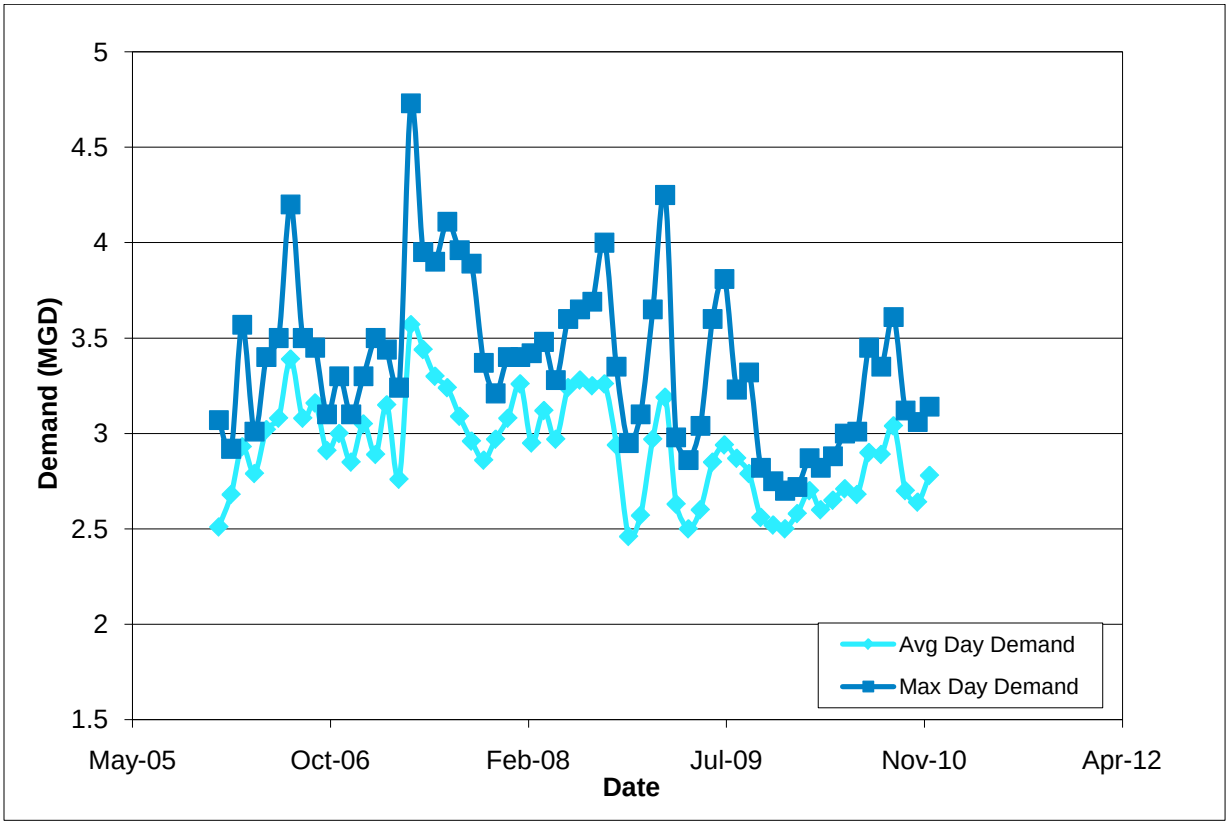


Table 3-2. Water Consumption History

Year	Residential		Commercial		Industrial	
	Gallons	GPCD	Gallons	GPD/Customer	Gallons	GPD/Customer
2006	437,129,000	58	156,082,000	722	41,575,000	2,712
2007	441,838,000	58	160,127,000	758	48,475,000	2,951
2008	417,479,000	55	139,917,000	667	56,398,000	2,809
2009	409,533,000	54	142,198,000	680	44,350,000	2,209
2010	406,495,000	54	132,298,000	610	43,209,000	2,277

3.4 Population Projections

The 2010 census showed that the City had a total population of 20,522, which decreased slightly from the City's 2000 census population of 20,738. This represents a population decrease of approximately 1 percent between 2000 and 2010. Previous census data indicates a similar population over the past several decades that indicate a stable population base within the City.

CDM obtained the MVRPC Traffic Analysis Zone (TAZ) GIS shapefiles that include a spatially mapped future population forecast (interpreted as 2030) and the 2000 census population data over large tracts of land not necessarily mapped within corporation limits across the several counties that MVRPC supports. This MVRPC population projection is used by other entities for forecasts and was selected for use here in water utility planning.

Figure 3-3 presents the population change forecast for the planning area over a 30-year period (2000-2030). In general, residential population growth is anticipated in the northeastern and western extents of the City. The total growth in TAZ population within the water service planning area is projected to increase by 806 people between 2000-2020 and by 1,204 people between 2000-2030 reflecting a moderate growth.

Using the TAZ population mapping requires some assumptions in using the data because the mapped areas do not align with the current or future water service area boundaries. Thus the distribution of population from individual TAZ areas cannot be accurately anticipated when spread over such a large area both inside and outside of the planning areas. CDM recommends assuming all of the population growth will be accommodated within the future water service area which is a conservative approach. In addition, the TAZ projections indicate some additional existing population which may be serviced by the City in the future.

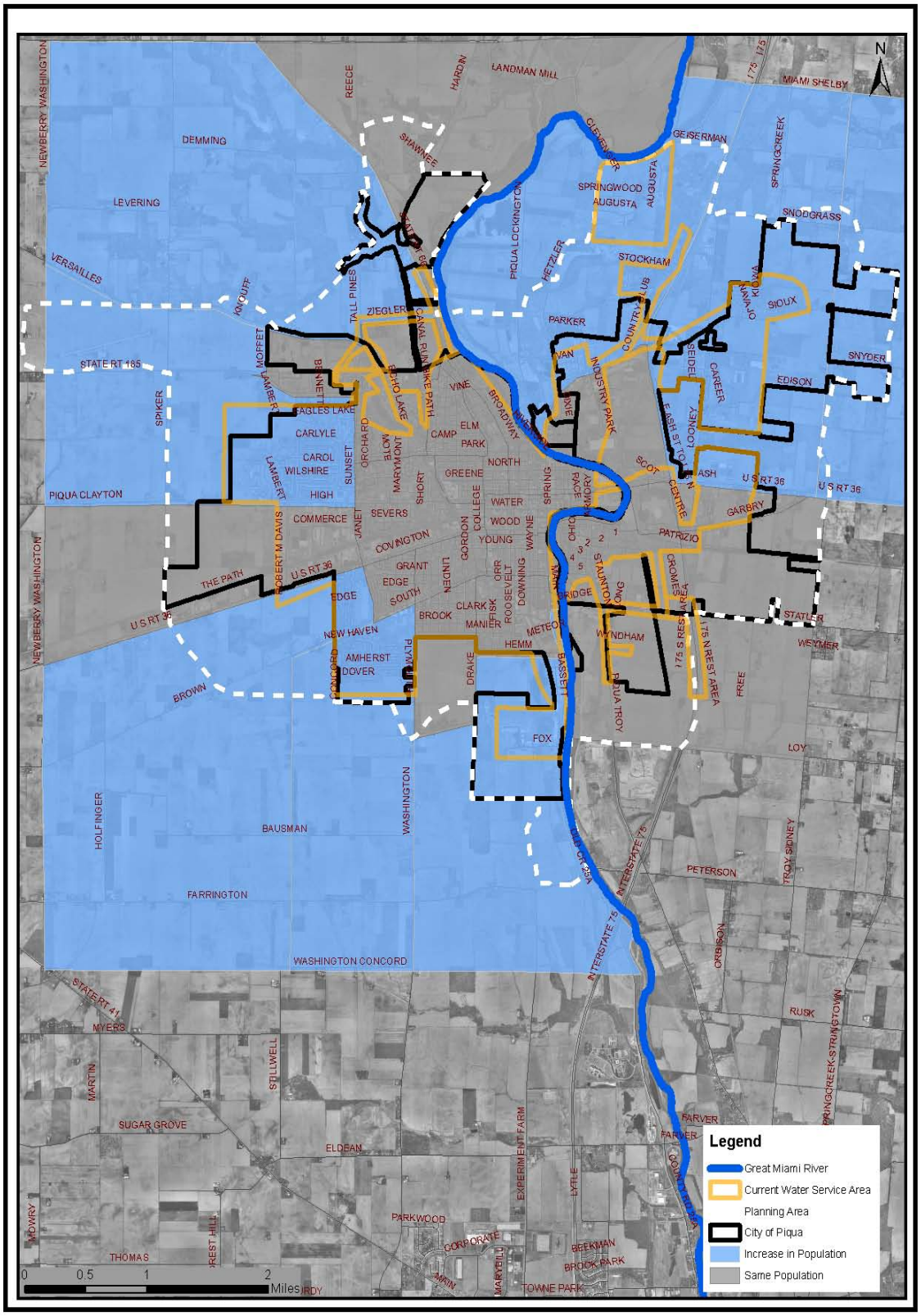
Table 3-3 provides an overview of the population trends for Piqua for the last 20 years and projected population data from MVRPC. The MVRPC projection data were used to determine an annual growth rate which was then used to project the population to intermediate years (5-year and 10-year intervals), as well as out to the year 2031 which is the 20-year planning period as shown in the Table.

Table 3-3. Population Trend and Projections

Year	Population
1990	20,612
2000	20,738
2010	20,522
2016	20,670
2021	20,793
2026	20,916
2030	21,019
2031	21,039

As a point of comparison, the projected City population for the year 2014 was shown as 20,153 in the "Plan it Piqua – Redevelopment Analysis Report" dated April 2010. It is understood that the purpose of the redevelopment report was to focus on the current City incorporated area and not evaluate the potential for expanding the water service area further outside the City limits. Therefore, this study uses the slightly more conservative (higher) population projection calculated from MVRPC data.

Figure 3-3. *Population Change Forecast*



3.5 Land Use Projections

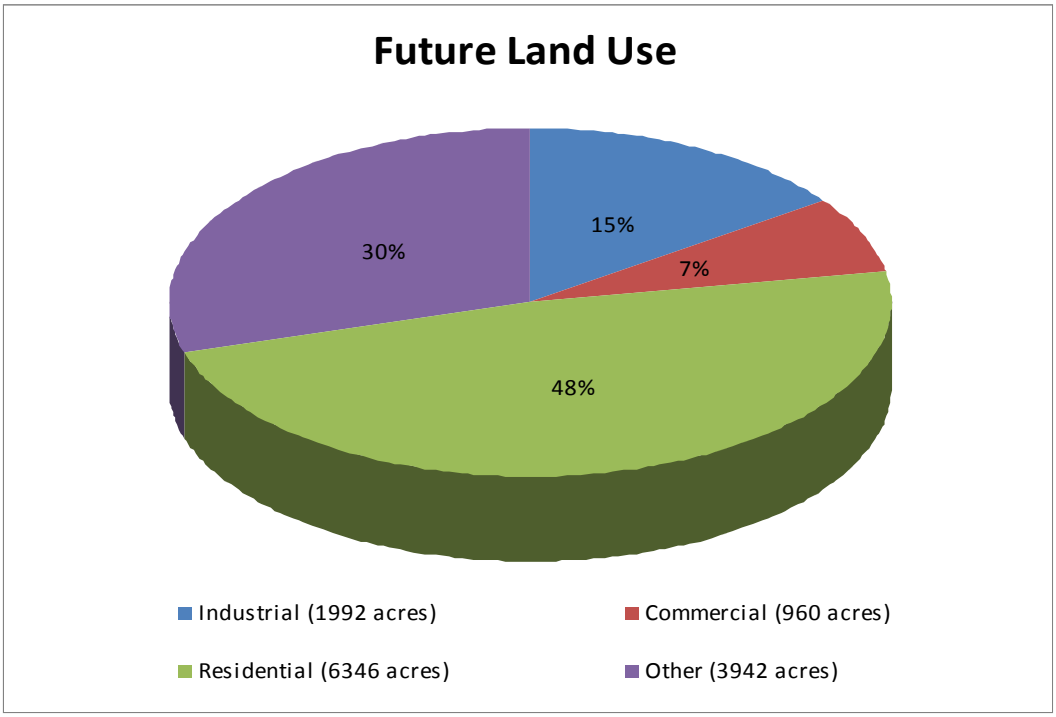
As the City and adjacent unincorporated areas continue to experience development, new non-residential land uses will bring increased water demands to service those new developments as either new construction or redeveloped parcels. CDM used the following data for developing land use projections for the 20-year planning period:

- MVRPC 's adopted land use shapefiles showing the current land use distribution for Miami County,
- The estimate of future land use based on observations of recent (2010) GIS data for Miami and Shelby Counties obtained from MVRPC, and
- Discussions with City personnel.

Figure 3-4 depicts the land use distribution for future service area for the year 2030. **Table 3-4** presents a comparison of the current and future land categories for the planning area totaling approximately 13,240 acres. There is a quarry located to the north of the intersection of Interstate 75 and Peterson Rd which is identified as industrial category in the future land use shape file from MVRPC. The quarry area would be an industrial land use but it is not expected to be served by City water; therefore, that area has been excluded from this study. The table also presents information about the planned redevelopment areas based on the report "Plan It Piqua– Redevelopment Analysis Report," dated April 2010, which totals 647.1 acres. It also depicts the net increase in each land use category for the future planning area.

Figure 3-5 presents the current City land use and **Figure 3-6** presents the future water service planning area with future land use overlaid.

Figure 3-4. *Future Land Use Distribution (Total Area = 13,240 acres)*



Note: Categories identified as "Other" include Institutional, Agricultural, Mixed, Municipal, and Open Space

Table 3-4 identifies an increase in the industrial, commercial, and residential land use categories. Open and agricultural lands in the existing water service area is forecast to decrease by 16.6%, indicating the land use shift with new developments having more substantial water demand within the 20-year planning horizon. Some of this area is outside the current water service area and would either be annexed into the City for water service, or provided with another means of water supply (perhaps through service area agreements with Miami County) to facilitate the development.

In addition to the growth from the expanded water service area, the City has opportunity to redevelop existing property within the City that could increase water demands. The existing water service area (within the City boundary) includes over 647 acres of mostly vacant or industrial and commercial properties without service that the City has identified as candidates for redevelopment. Supplying these properties with water service through the City's redevelopment activities would increase water demands without annexation or the capital expense of constructing new water mains to service them.

Table 3-4. Comparison of Current and Future Land Use Categories

Land Use Category	Current Water Service Area (acres)	Planning Area (acres)	Redevelopment Area (acres)	Future Additional Service Area (acres)
Industrial	538	1,992	409	1,863
Commercial	584	960	155	531
Residential	2,198	6,346	46	4,194
Others	1,872	3,942	37	2,107
TOTAL	**5,192**	**13,240**	**647**	**8,695**

3.6 Unaccounted for Water (UFW)

Water distribution systems typically lose finished water that is supplied from the WTP. The difference between the finished water produced and water billed to the City's customers is defined as UFW or non-revenue water. Lower UFW values are desirable to reduce operating costs of treating and distributing finished water without a customer sale. UFW comprises several categories, including the following:

- Fire flow testing and fire protection
- Hydrant flushing
- Water main breaks and pipe leakage
- Elevated water storage tank flushing
- Non-metered construction water usage
- Water meter inaccuracies
- Illegal connections

Figure 3.5. *Current Land Use for Planning Area*

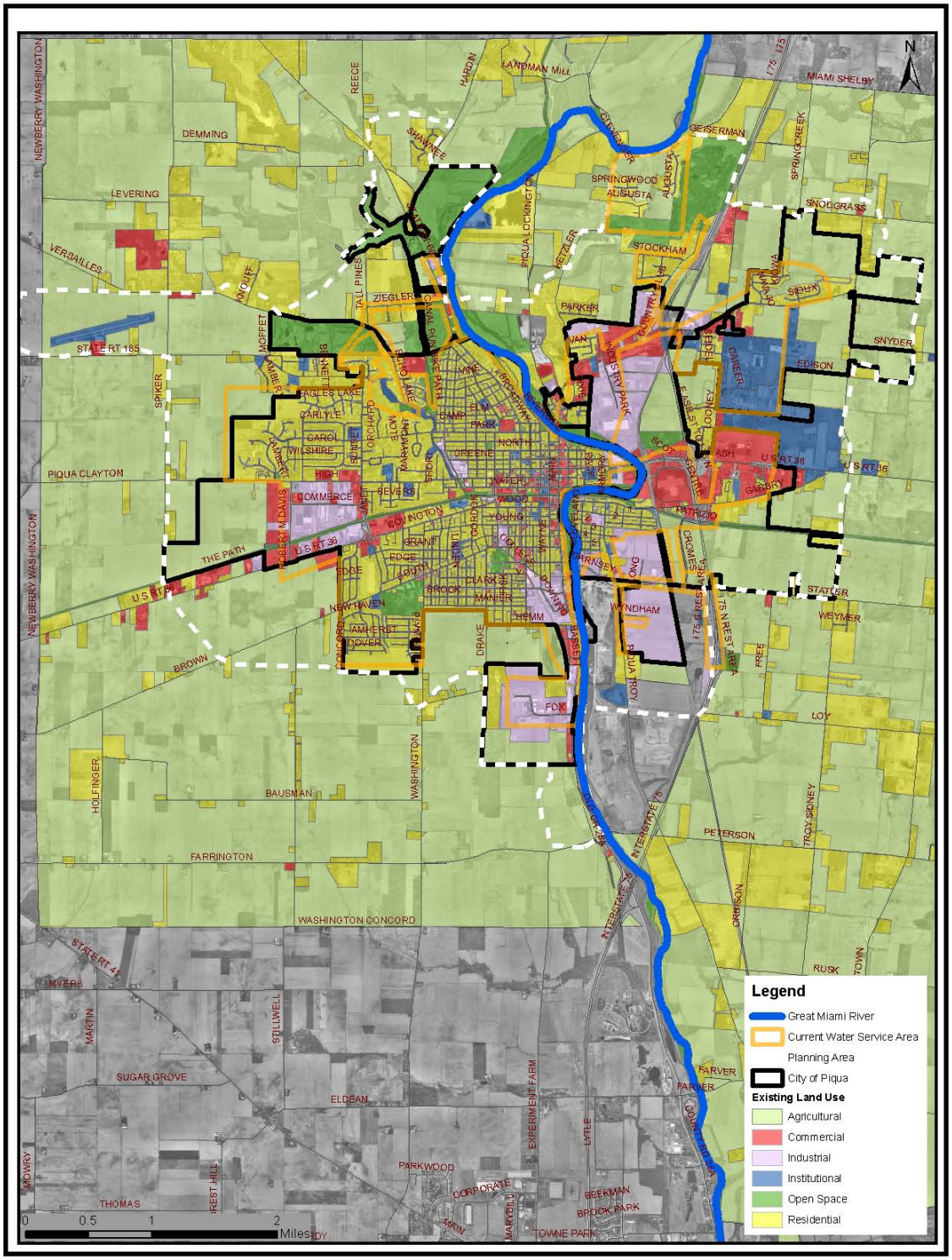
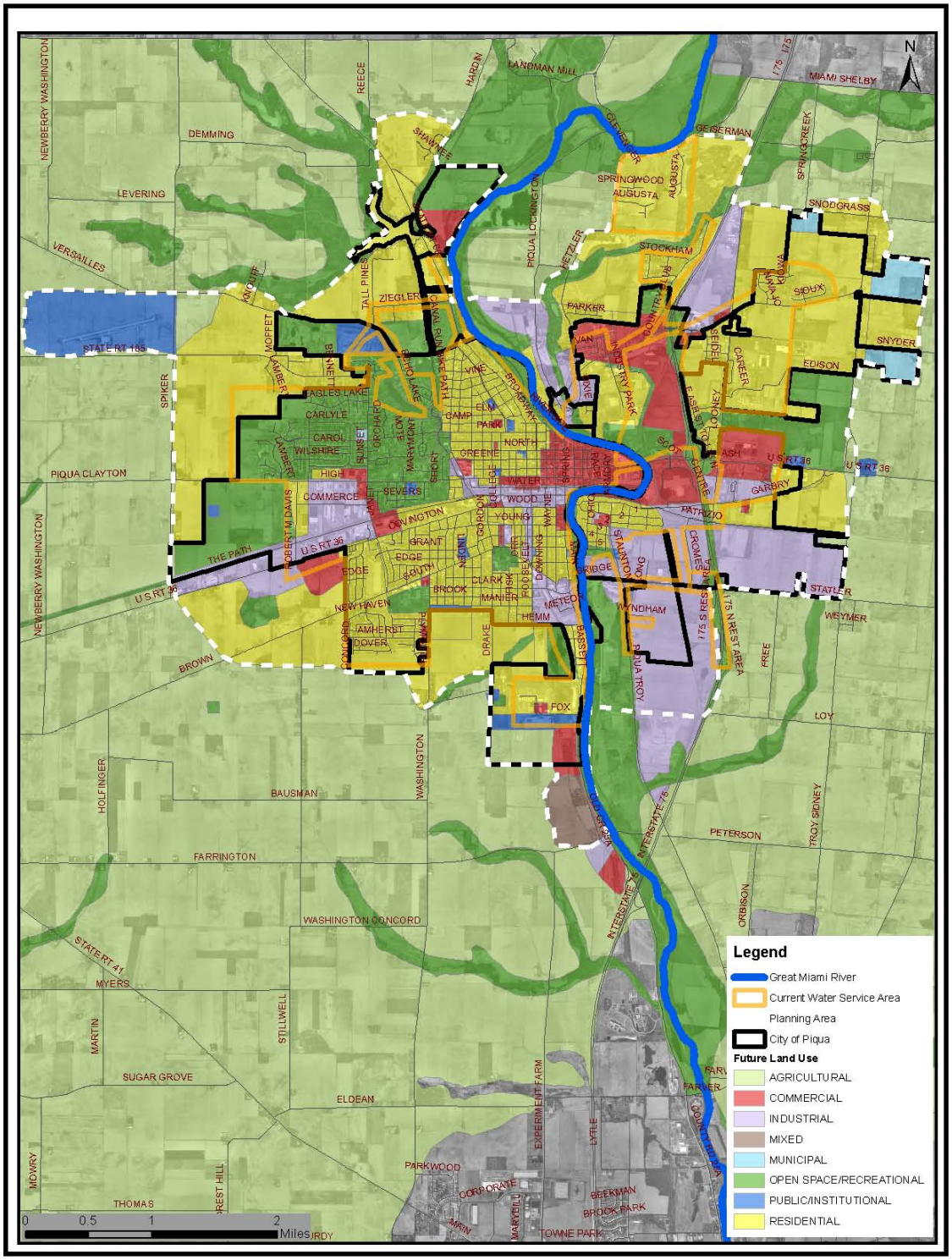


***Figure 3-6**. Future Land Use for Planning Area*



CDM compared the last 5 years of total plant production and total customer consumption to determine the UFW from the distribution system, which is presented in **Table 3-5.** Based on this data, the UFW is consistent with a range between 26% and 31% and an average UFW is 28%.

Table 3-5. Historic UFW (%)

Year	Total Consumption (Gallons)	Total Plant Production (Gallons)	% UFW
2006	634,786,000	861,696,000	26
2007	650,440,000	907,960,000	28
2008	613,794,000	887,424,000	31
2009	596,081,000	800,296,000	26
2010	582,002,000	800,040,000	27

Note: Total Plant Production data was reduced by 20% based on information provided by plant staff to correct for inaccurate meter readings at the plant

Many water utilities in southwest Ohio operate in the 15% - 25% UFW range, with those water utilities operating with older infrastructure generally exhibiting higher UFW percentages. In 1996, the American Water Works Association (AWWA) Leak Detection and Accountability Committee recommended a 10% UFW benchmark, but this recommendation has no regulatory force. The City's UFW performance is generally higher than its peer communities and industry benchmarks. Historically a 15% UFW has been an accepted range and may be an appropriate future target.

The City should undertake focused efforts to reduce this UFW percentage to lower its operating costs and the amount of finished water pumped from the plant. To do so, the City could replace or rebuild customer water meters more frequently, recalibrate or replace the plant's two finished water meters, and implement a rigorous leak detection and repair program for leaky pipes. The City could also re-evaluate its tank flushing program for DBP control by incorporating physical mixing instead of flushing water.

For planning purposes, UFW must be considered as part of the treatment plant capacity evaluation, because sufficient water must still be treated and distributed to meet customer demands plus the appropriate level of UFW. In addition, programs to reduce water loss take time to identify and implement. This analysis uses a conservative approach to incorporate an average UFW percentage of 15% to estimate future water demands. Efforts to reduce UFW would translate into lower overall demands on the treatment plant and require smaller production flows. Existing demands are assumed to remain at the current UFW rate of 28%.

3.7 Future Water Demands

Since the water demand from the City's current customers is expected to remain unchanged during the 20-year planning period, any increase in future water demand will be from population growth and new industrial/commercial customers. To quantify the change in water demands between current and future conditions, CDM combined projected City of Piqua and MVRPC land use conversion for industrial and commercial customers, and additional population for residential areas as defined by MVRPC TAZ projections. Both factors were considered to develop the predicted 20-year water demand to meet the expected development schedule and increased residential land use area and density.

Land use conversion was used as the basis of future commercial and industrial demands by applying a per acre water demand across the new development areas. Per capita water consumption is a more accurate method of estimating residential water consumption than residential land use areas, and has current data from existing customers to justify the values. The unit water demands shown in **Table 3-6** were developed from current land use and industrial and commercial demands taking into account an adjustment for the currently vacant redevelopment areas. These demands were then used for the projected land use modifications to calculate future system demands. For projecting future demand CDM rounded the industrial demand to 1,000 gallons per day/acre (gpd/Ac), and the commercial to 950 gpd/Ac.

Table 3-6. Unit Water Demands

Land Use Category	Current Area (acres)	Redevelopment Area (acres)	Adjusted Area (acres)	Total Water Consumption (gpd)	Current Unit Demand (gpd/Ac)
Industrial	538	409.1	128.9	128,223	995
Commercial	584	155	429	400,341	933

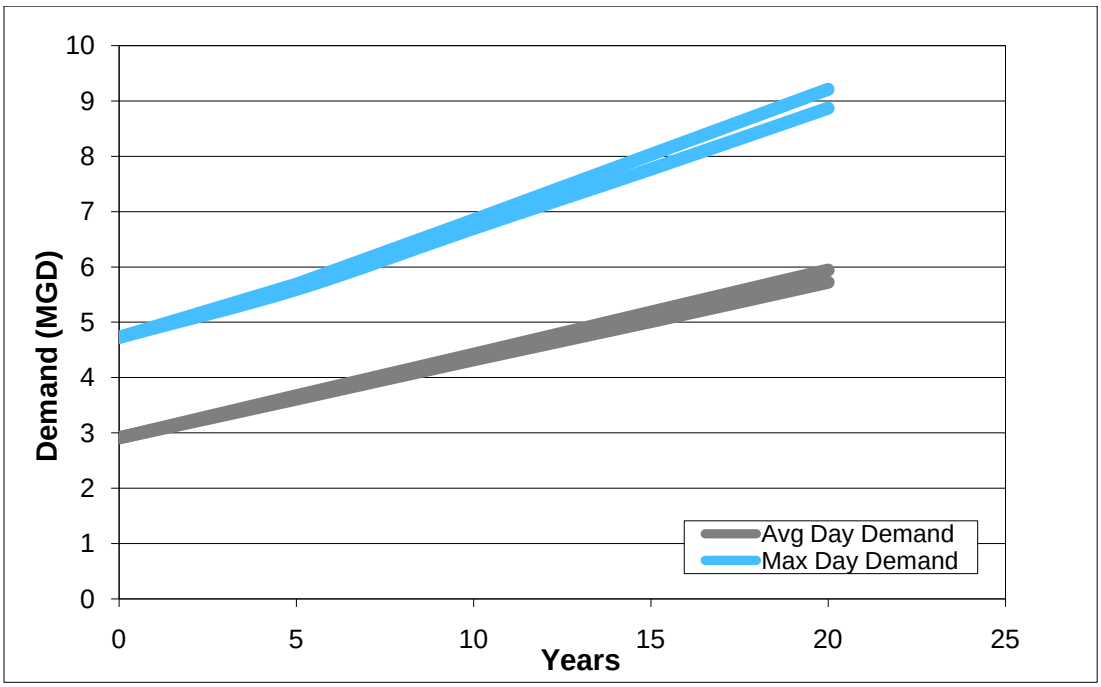
The current demand for Piqua's water system was used as the starting point for future projections. The future residential component (based on population) and the future non-residential component (based on industrial and commercial land use) was added to the current demand to develop the future average day demand. Peaking factors and 15% UFW were then applied to the future demands to develop peak demands for planning purposes.

Although there is a nominal decrease in per capita residential demand over the past 5 years from 58 gpcd to 54 gpcd, the projected future demand for average per capita residential consumption has been conservatively assumed to be 60 gpcd for planning purposes. The results of the above approach are presented in **Table 3-7**. This table depicts two different sets of assumptions for residential growth. The first column represents an aggressive growth projection that all population growth within the TAZ areas in and around the City (as previously discussed) would become water customers. The second column represents a less aggressive growth assumption that only a limited portion of the population within the TAZ areas would become water customers. The range of values (8.86-9.21 MGD) demonstrates that the impact of residential growth is minor compared to the much more significant impacts of the future industrial and commercial demands. The average and maximum future water demands for the above scenarios and the interim demands for 5-yr and 10-yr intervals are shown in **Figure 3-7.**

Table 3-7. Future Water Demands

Condition	Aggressive Residential Growth Projection (All of TAZ population)	Limited Residential Growth Projection (Portion of TAZ population)
Future Industrial Average Day Demand (MGD)	1.86	1.86
Future Commercial Average Day Demand (MGD)	0.50	0.50
Future Residential Average Day Demand (MGD)	0.27	0.08
Total Future Additional Average Day Demand (MGD)	2.64	2.44
Total Future Average Day Demand with UFW Factor (MGD)	3.03	2.81
Current Average Day Demand (MGD)	2.91	2.91
Total Future Average Day Demand (MGD)	5.94	5.72
Total Future Max Day Demand (MGD)	**9.21**	**8.86**

Figure 3-7. *Future Water Demands Range*



Note that the current year (shown as year "0" on the graph) uses a maximum day demand of 4.73 MGD, which is the maximum day demand over the past five years.

Using the more conservative aggressive growth approach, the projected Maximum Day Demand over a planning period of 20 yrs is approximately 9.2 MGD, which the new water treatment plant must be capable of producing in a 24-hr day. However, this does not consider the possibility that the industrial/commercial demand component will be significantly less than that projected by MVRPC. What seems equally likely is that the development opportunities including large industrial users will occur over a much longer period of time than the planning period. Constructing a new water treatment plant facility to meet these optimistic land use projections does not appear to be justifiable considering the risk of constructing an excessively large treatment facility that the current customers would be required to finance.

Also, analysis of historic maximum day demands over the past years shows that the maximum demand occurred in 2007 and was 4.73 MGD (from **Table 3-1**). Considering the potential for less aggressive development outside the City and a mid range in residential population growth, a future treatment plant of approximately 9 MGD is recommended to be planned for when siting the new plant. Because 9.0 MGD appears excessive for near-term needs, CDM recommends phasing the plant's construction such that any future expansion beyond the near term needs could be accommodated for future developments to achieve the 20-year 9.0 MGD capacity.

Several considerations are included in this recommended capacity in addition to the fact that the future demand is approximately 200% of the recent historical peak flow. These include the consideration of sizing of unit processes for phased future expansion. Phasing should be done modularly and that would require consideration of the life expectancy of the initial phase and ease of constructing additional modules. Future expansion with modular units is easier and less expensive to accomplish at a future date; however, sizing common facilities and piping for the full 9.0 MGD capacity may be more expensive to construct initially. Based on these factors, CDM considered two alternatives as shown in **Table 3-8.**

Table 3-8. New Water Treatment Plant Capacity Alternatives

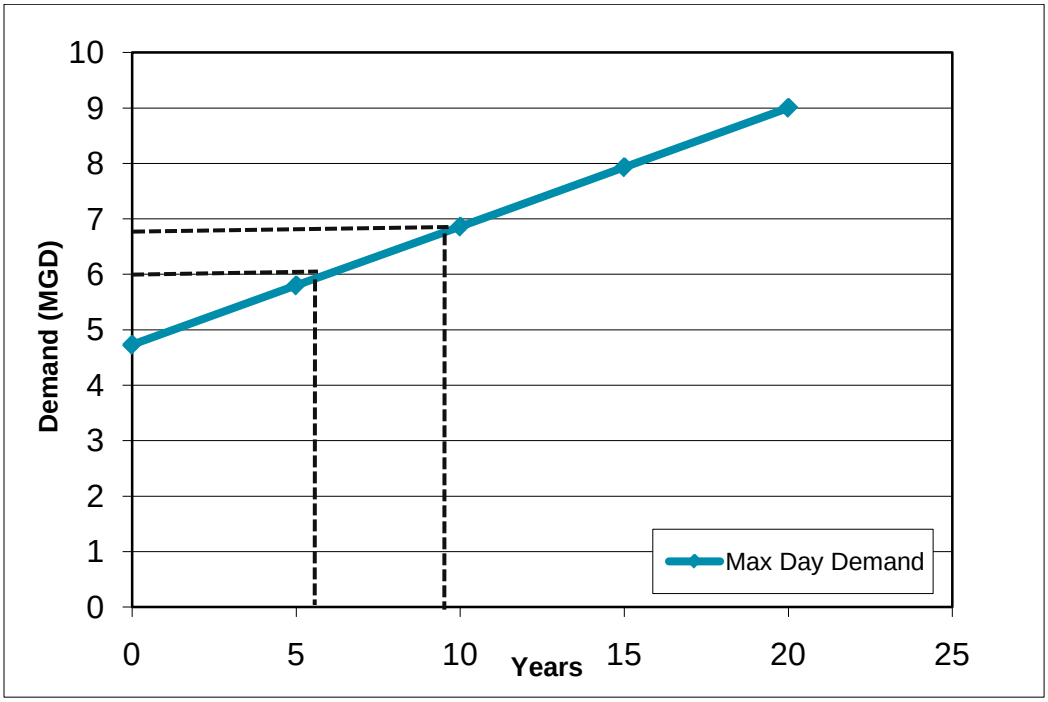
Scenario	Recommended Water Treatment Plant Capacity (MGD)	Number of Trains
Alternative 1	6.0	Two treatment trains @ 3.0 MGD each
Alternative 2	6.75	Three treatment trains @ 2.25 MGD each

Alternative 1 includes constructing a new facility with two 3.0 MGD modules to meet future capacity; this would provide a plant design capacity of 6.0 MGD which would provide for growth in demand of approximately 27% before additional treatment facilities would be required. The remaining third train could be developed at a later date when developments require it such that the full 9 MGD capacity would be met.

Alternative 2 provides three modules of 2.25 MGD each, which provides 6.75 MGD of initial capacity. Additionally, this configuration can provide 4.5 MGD of capacity with one treatment train out of service, which is near the maximum day demand, and offers more operational flexibility than Alternative 1. The higher overall capacity of 6.75 MGD represents an increase of more than 42% over existing recent maximum demands. The remaining fourth train could be developed at a later date when developments require it such that the full 9 MGD capacity would be met.

Using the above projections, if the new plant is sized for a future maximum day capacity of 6.0 MGD, it would be sufficient for approximately 6 years before any further expansions are necessary, whereas if the new plant is sized for a future maximum day capacity of 6.75 MGD, further expansions will not be needed for approximately 9.5 years as projected in **Figure 3-8.**

Figure 3-8. *Service Life from 6.0 MGD and 6.75 MGD Initial Capacities*



CDM has considered the two approaches to modular construction using the 2.25 MGD and 3.0 MGD modules with future expansion in each case requiring only a single additional module to achieve the 9.0 MGD target capacity. Note that if the City wants to provide future maximum day capacity of 9.21 MGD, the size of the individual modules will need to be adjusted only slightly and the following discussion applies with minimal changes.

Constructing two 3.0 MGD units initially to provide a 6 MGD plant capacity will afford reasonable initial construction costs; but will provide only a short period (between five and ten years) before an expansion of the plant is needed, assuming the demand projections are proven accurate. One additional negative consideration is that with two 3.0 MGD units, if one train is taken out of service for repair and/or maintenance, the remaining plant capacity can provide adequate redundancy for the average daily demands for only a short period into the future, and the average flows experienced in 2007 and 2008 have already exceeded that capacity.

Note that the initial construction of three 2.25 MGD trains for a 6.75 MGD total capacity will meet demands further into the future and also provides an easy expansion to 9 MGD. In addition, the 2.25 MGD modules will allow capacity of 4.5 MGD when taking one train out of service. This would provide average day capacity for approximately 10 years. The additional initial cost to construct a 6.75 MGD capacity modular plant would require a decision by the City to invest in the more expensive infrastructure based on assumed growth that has not been experienced during the recent past.

Based on meeting the City's stated desire to serve an expanded water service area; MVRPC estimates of growth in the Piqua area and past water usage data, CDM recommends that the City initially construct the 6.75 MGD option with space for a future fourth train to achieve 9 MGD and allow space for further future expansion of a fifth (or sixth) train which would allow for growth up to 11.25 (or 13.5) MGD for future planning well beyond 2030.

As discussed above, these water system demands do not consider any additional large water usage, commercial or industrial user, locating within the City service area. A high-demand industrial or commercial user could significantly increase the average and maximum day projections shown, which would require additional analysis. If the City expands service farther into currently un-served areas, water demands are estimated to further increase similar to the existing industrial and commercial demands.

Section 4
Regulatory Requirements

The purpose of this section is to discuss current, pending and future regulatory requirements for the City of Piqua's new water treatment plant. The regulations are authorized by the Safe Drinking Water Act (SDWA) and are issued by the EPA (EPA). Ohio EPA generally enforces the SDWA standards, but also has design and operational requirements that must be met under Ohio Administration Code (OAC) Section 3745.

4.1 Current Regulations

The City of Piqua currently is required to meet requirements of the following current regulations:

- Surface Water Treatment Rule (SWTR)
- Interim Enhanced Surface Water Treatment Rule (IESWTR)
- LCR
- Total Coliform Rule (TCR)
- Filter Backwash Recycling Rule (FBRR)
- Chemical Phased Rules (I, II, IIb, & V) for Inorganic Chemicals (IOCs), Volatile Organic Chemicals (VOCs), and Synthetic Organic Chemicals (SOCs)
- Stage 1 Disinfectant and Disinfection By-Products(D/DBP) Rule
- Radionuclide Rule
- Arsenic Rule
- Source Water Protection
- National Secondary Drinking Water Standards (NSDWS)
- Fluoride Rule

In addition, the City of Piqua will need to address two recent regulatory requirements:

- Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) and
- Stage 2 D/DBP Rule

These are described in detail in this Section. Ohio EPA requirements are noted where they are more stringent. Drinking water standards for Ohio Public Water Systems are included in **Appendix A.**

Anticipated Regulations and Contaminants of Interest that are under discussion are:

- Revised TCR
- Additional Chlorinated, Brominated and Iodinated DBPs
- Perchlorate
- Group Regulation of Carcinogenic VOCs
- EDCs and PPCPs
- Chemical Facility Anti-Terrorism Standards (CFATS)

4.1.1 SWTR

The SWTR was published by the EPA in 1989. It established primary drinking water regulations requiring the treatment of surface water supplies, or groundwater supplies under the direct influence of surface waters, that serve 10,000 or more people. It stipulates that treatment should consist of filtration and disinfection. The SWTR established Maximum Contaminant Level Goals (MCLGs) of zero for the following microbiological parameters: *Giardia lamblia*, Enteric Viruses, and Legionella. The SWTR established a treatment technique (disinfection and/or filtration) in lieu of MCLs for these contaminants. Disinfectant residuals of 0.2 mg/L within the distribution system must be detected in 95 percent of the samples each month for any two consecutive months to comply with Ohio EPA requirements. Currently Piqua monitors total chlorine in the distribution system with grab samples at several sampling sites to determine SWTR compliance.

The SWTR also established a combined filtered water turbidity standard of less than or equal to 0.5 nephelometric turbidity units (NTU), which has since been superseded.

4.1.2 IESWTR

In 1998, the Interim Enhanced SWTR established treatment techniques for removal of *Cryptosporidium* based on filtered water turbidity. The goal of the IESWTR was to optimize treatment reliability and to enhance physical removal efficiencies in order to minimize *Cryptosporidium* levels in finished water. The key provisions of the rule are:

- MCLG of zero for *Cryptosporidium*
- 2-log *Cryptosporidium* removal requirements for systems that filter
- Strengthened combined filter effluent turbidity performance standards
- Individual filter turbidity monitoring provisions
- Disinfection profiling and benchmarking provisions
- Requirements for covers on new finished water reservoirs
- Sanitary surveys, conducted by states, for all surface water systems regardless of size

The turbidity performance requirements are complex. The combined filter effluent (CFE) turbidity was lowered to 0.3 NTU, as measured in 95 percent of all samples collected monthly. The maximum turbidity must never exceed 1 NTU. Individual filters must also be continuously monitored for turbidity. Individual filter effluent (IFE) performance triggers were established which lead to additional reporting, investigation and regulatory compliance. A further description of the rule is given in **Table 4-1.**

Table 4-1. IESWTR Requirements

Turbidity Reporting Requirements	Monitoring/Recording Frequency	SWTR As of June 29, 1993	IESWTR ≥10,000 PEOPLE As of Jan 1, 2002
CFE 95% Value Report total number of CFE measurements and number and percentage of CFE measurements ≤ 95th % limit	At least every 4 hours *	≤ 0.5 NTU	≤ 0.3 NTU
CFE Maximum Value Report date and value of any CFE measurement that exceeded CFE maximum limit	At least every 4 hours *	5 NTU Contact State within 24 hours	1 NTU Contact State within 24 hours
IFE Monitoring Report IFE, monitoring conducted and any follow-up actions	Monitor continuously every 15 minutes	None	Monitor – exceedances require follow-up action

4.1.3 LCR

The LCR was established in 1991, requires waterworks owners to conduct sampling at consumers' taps. Under the LCR rule, EPA established action levels (AL) of 0.015 mg/L for lead and 1.3 mg/L for copper based on the 90th percentile level of tap water samples. First draw samples must be collected by all utilities at cold water taps in tier 1 sites (homes/buildings that are at high risk of lead/copper contamination as identified in the regulation). The number of sample sites required is based on system size. The applicable monitoring requirements are as follows:

- Samples must be collected between June and September, during the months of warmer temperatures, with reporting completed by the end of the calendar year.
- Sampling must be completed within a single year.
- Piqua is on reduced triennial monitoring for lead and copper. Based on population, 30 samples must be collected and reported to Ohio EPA by Piqua every three years.

Minor revisions to the regulation were published in 2000, 2004 and 2006. USEPA continues to examine this rule and it is currently under discussion for potential further revision. While these changes are relatively minor, congressional interest in lead and copper has been very high. It is possible that the legislature could become involved in the rule revision and mandate more frequent and extensive testing with a potential to affect Piqua.

4.1.4 TCR

EPA published the TCR in 1989. Total coliform includes both fecal coliform and Escherichia coli (E. coli). The MCLGs for total coliform, fecal coliform and E. coli are zero. Compliance with the MCL is based on the presence or absence of total coliform in a sample, rather than on an estimate of coliform density. The MCL for systems such as Piqua analyzing at least 20 samples per month is that no more than 5 percent of

the monthly samples may have a positive total coliform result. Samples are collected from representative sites in the distribution system, including high, medium and low water age areas.

All samples that are positive for total coliform must also be tested for either fecal coliform or E. coli. In addition, any sample that tests positive for total coliform requires repeat sampling at the same site and sampling within five service connections both upstream and downstream of the sample site. If either the original sample is positive for fecal coliform/E. coli or any of the repeat samples are positive for fecal coliform/E. coli, then this constitutes a violation of the MCL for total coliform.

4.1.5 FBRR

The FBRR was established in 2001 and is applicable to all surface water systems. The purpose of this rule is to improve public health protection by assessing and changing, where needed, recycle practices for improved contaminant control, particularly microbial contaminants. The rule requires systems that recycle to return specific recycle flows through all processes of the system's existing conventional or direct filtration system (i.e., head of the plant) or at an alternate location approved by the state. This rule requires that systems collect and retain information such as:

- List of all recycle flows and frequency with which they are returned,
- Average and maximum backwash flow rates through filters,
- Average and maximum duration of filter backwash process (in minutes),
- Type of treatment provided for recycle flows.

4.1.6 VOCs SOCsIOCs

Several phases of regulations have been developed by EPA to address SOCs and VOCs:

4.1.6.1 Phase I: VOCs

In 1987, the EPA established the first regulation for eight VOCs. MCLs and MCLGs were set for each individual VOC. Monitoring was initially required quarterly for one year. If the regulated VOCs were not detected then the sampling frequency could be reduced. Piqua conducts quarterly monitoring on VOCs as determined by Ohio EPA. The MCL is exceeded when the RAA (average of four consecutive quarters of monitoring) is greater than the MCL. DBPs such as TTHMs and HAAs which are discussed below in **Section 4.1.14** are classified as VOCs.

4.1.6.2 Phase I and V: Inorganics, SOCs and VOCs

The Phase II regulation was established in 1991 and the Phase V in 1992. These rules are typically considered in concert with each other. The Phase II rule regulates a wide variety of inorganics, pesticides, herbicides, asbestos, nitrate and nitrite. The Phase V rule expands on the list of regulated metals, pesticides, VOCs and added a new substance, cyanide. Under both the Phase II and Phase V rules, most monitoring is initially quarterly for one year. If substances are not detected, the monitoring frequency may be reduced and is established by the state. Exceptions to this monitoring include nitrate and nitrite (annual), and asbestos (once per nine years). Piqua currently monitors inorganics such as antimony, arsenic, aead, copper, barium, beryllium, cadmium, chromium, cyanide, fluoride, mercury, nickel, selenium, thallium and nitrate and SOCs such as atrazine, alachlor and simazine and is required to continue monitoring these compounds in the future.

4.1.7 Stage 1 D/DBP

The Stage 1 D/DBPR requires public water systems to comply with established MCLs for DBPs and maximum residual disinfection levels (MRDLs) for disinfectant residuals. It applies to all systems that add any disinfectant or oxidant to the treated water. Per requirements of this rule, MCL compliance is calculated using the RAA of all samples from all monitoring locations across the system. For systems using conventional filtration or lime softening, Ohio EPA requires a monthly operating report for TOC removal to be completed and filed.

4.1.8 Radionuclide Rule

In 2000, EPA revised the radionuclides regulation, which had been in effect since 1977. The revisions required new monitoring provisions to ensure that all customers of community water systems will receive water that meets the MCL for radionuclides in drinking water. EPA also issued a standard for uranium, as required by the 1986 amendments to the SDWA. Ohio EPA's detection limits on these radionuclides are more stringent than the Federal MCLs and are listed in **Table 4-2**.

Table 4-2. Ohio EPA Radionuclides Detection Limits

Radionuclide	Detection Limit
Cesium-134	10 pCi/L
Iodine-131	1 pCi/L
Gross alpha	3 pCi/L
Gross beta	4 pCi/L
Radium-226	1 pCi/L
Radium-228	1 pCi/L
Strontium-89	10 pCi/L
Strontium-90	2 pCi/L
Tritium 1,000	pCi/L
Uranium	1 µg/L
Other beta/photon emitters	1/10 of the MCL (dose eq. in pCi/L)

4.1.9 Arsenic Rule

The arsenic rule was promulgated in 2001 and became effective January 2006. This rule lowered the MCL to 0.010 mg/L with an MCLG of zero. Surface water systems must sample annually and groundwater systems triennially at each entry point to the distribution system. Compliance is based on the RAA at each sample point (LRAA). The rule also requires public notification in the Consumer Confidence Report if the water exceeds .005 mg/L arsenic.

4.1.10 Source Water Protection

Under the source water protection program, EPA identifies federal tools, including those available under the Clean Water Act, other EPA programs, and various agricultural programs, that can be used to help protect drinking water. Ohio EPA encourages communities such as Piqua to conduct source water assessments and produce studies or reports that provide basic information about the drinking water in each public water system. The assessment should consist of various steps such as delineation of source water assessment area, conducting an inventory of potential sources of contamination and determining the susceptibility of the water supply to contamination.

4.1.11 NSDWS

The NSDWS, or secondary standard, is a non-enforceable guideline regarding contaminants that may cause cosmetic effects such as taste and odor or color in drinking water. Ohio EPA recommends secondary standards to water systems but does not require compliance.

4.1.12 Fluoride Rule

The EPA promulgated the fluoride rule in 1986. The current regulation set an MCL of 4.0 mg/L, an MCLG of 4.0 mg/L and a secondary standard of 2.0 mg/L. Monitoring is at minimum annual, with the state allowed to set more frequency requirements. Daily monitoring is typical for treatment plants that feed fluoride such as Piqua.

This regulation is currently under review. It is expected that the MCL will be lowered to 2.0 mg/L, with a recommended fluoride level of 0.7 mg/L in drinking water.

4.1.13 LT2ESWTR

The LT2ESWTR addresses a number of issues related to improving control of microbial pathogens, specifically the protozoan *Cryptosporidium*, in drinking water and addressing risk trade-offs with DBPs. This rule requires systems to monitor their source water, calculate an average *Cryptosporidium* concentration, and use those results to determine if their source is vulnerable to contamination and may require additional treatment. The major provisions included in this rule are:

- Filtered systems such as Piqua must conduct 24 months of source water monitoring for *Cryptosporidium*. Filtered systems must also record source water E. coli and turbidity levels.
- Filtered water systems will be classified in one of four treatment categories (bins) as shown in **Table 4-3**, based on their source water monitoring results. Systems may also use previously collected data (i.e., Grandfathered data). Systems classified in higher bins must provide additional water treatment to further reduce *Cryptosporidium* levels by 90 to 99.7 percent (1.0 to 2.5-log), depending on the bin. Systems will select from different treatment and management options in a "microbial toolbox" to meet their additional treatment requirements. Systems classified in higher risk bins must provide 90 to 99.7 percent (1.0 to 2.5-log) additional reduction of *Cryptosporidium* levels in addition to filtration.

Table 4-3. Bin Classification For Filtered Systems

Crypto Conc. (oocysts/L)	Bin	Additional *Cryptosporidium* Treatment Required			
		Conventional Filtration	Direct Filtration	Slow Sand or Diatomaceous Earth Filtration	Alternative Filtration
<0.075	Bin 1	No additional treatment required	No additional treatment required	No additional treatment required	No additional treatment required
0.075 to < 1.0	Bin 2	1 log	1.5 log	1 log	(1)
1.0 to < 3.0	Bin 3	2 log	2.5 log	2 log	(2)
≥ 3.0	Bin 4	2.5 log	3 log	2.5 log	(3)

(1) As determined by the State such that the total removal/inactivation > 4.0-log.
(2) As determined by the State such that the total removal/inactivation > 5.0-log.
(3) As determined by the State such that the total removal/inactivation > 5.5-log.

Systems such as Piqua, serving populations of 10,000-49,999 are classified as **Schedule 3** systems. Piqua completed first round of source water monitoring (24 months) for *Cryptosporidium* in March 2010. By September 2012, the City of Piqua must report their initial bin classification to the EPA or state for approval. Based on source water monitoring results for *Cryptosporidium*, Piqua is classified under **Bin 1** (highest 12 month mean is 0.029 oocysts/L) with no further treatment required.

Piqua must also continue to comply with specific continuous combined filter effluent turbidity requirements, as required under previous regulations.

4.1.14 Stage 2 DBP Rule

The Stage 2 DBP Rule published in 2006 requires systems to conduct an evaluation of their distribution systems, known as an Initial Distribution System Evaluation (IDSE), to identify the locations with high DBP concentrations. These locations will then be used by the systems as the sampling sites for Stage 2 DBP rule compliance monitoring.

Compliance with the MCL for two groups of DBP (TTHM and HAA5) will be calculated for each monitoring location in the distribution system. This approach, referred to as the LRAA, differs from current requirements, which determine compliance by calculating the RAAof samples from all monitoring locations across the system.

The Stage 2 DBP rule also requires each system to determine if they have exceeded an operational evaluation level, which is identified using their compliance monitoring results. The operational evaluation level provides an early warning of possible future MCL violations, which allows the system to take proactive steps to remain in compliance. A system that exceeds an operational evaluation level is required to review their operational practices and submit a report to their state that identifies actions that may be taken to mitigate future high DBP levels, particularly those that may jeopardize their compliance with the DBP MCLs.

The Stage 2 DBP Rule builds incrementally on existing DBP rules and takes a risk-based targeted approach to require treatment changes by only those public water systems that are identified as having the greatest remaining risk. The first step is a multi-year process for systems to determine where higher levels of DBPs are likely to occur in their distribution system. These locations will become the system's new DBP monitoring sites. For Piqua, these locations are: 8605 Hetzler, 1130 Park, 200 Sunset and 839 Main Streets.

If the DBP levels at these locations exceed the MCLs of 80 µg/L for TTHM and 60 µg/L for HAA5, then the City must take corrective actions. It is imperative that corrective actions are taken immediately since planning, design, obtaining funding and permits for construction, and construction of new facilities can take significant time. Per the Stage 2 DBP Rule, the timeline for Schedule 3 systems such as Piqua for the first year of compliance monitoring is October 1, 2013 and must be in compliance with the Stage 2 DBP rule MCLs at the end of a full year of monitoring.

4.2 Pending Regulations

4.2.1 TCR Revisions

A proposed revision to the TCR was published by the EPA in 2010. The proposed rule establishes an MCLG and MCL for E. coli rather than total coliform. A treatment technique for total coliform is created and total coliform continues to serve as the indicator of potential contamination in the distribution system. Positive total coliform sample(s) will require an investigation into the probable cause of the coliform detection. Implementation of corrective actions and best management practices (BMPs) will be required. Monitoring practices will remain the same as under the original TCR (EPA, 1989) with the exception of eliminated fecal coliform testing and requiring E. coli testing when total coliform samples are positive.

4.2.2 Additional Chlorinated, Brominated and Iodinated DBPs

Recent epidemiological studies have raised interest in chlorinated, brominated and iodinated DBPs that are not currently regulated. It is possible that future regulations will target individual DBPs instead of groups of compounds. The health effects studies required for regulations are just beginning and the proposed regulations are many years away. By complying with the current Stage 1 and Stage 2 D/DBPR, the City will be taking appropriate action to control the potential new regulated DBPs as well.

4.2.3 Perchlorate

EPA has decided to regulate perchlorate under the SDWA. This decision reverses a 2008 preliminary determination. This action notifies interested parties of EPA's decision to regulate perchlorate, but does not in itself impose any requirements on public water systems (PWSs). However, this action initiates a process to develop and establish a national primary drinking water regulation (NPDWR). Once the NPDWR is finalized, certain PWSs will be required to take action to comply with the regulation in accordance with the schedule specified in the regulation.

4.2.4 Group Regulation of Carcinogenic VOCs

EPA announced in February 2011 that it plans to develop one national primary drinking water regulation covering up to 16 carcinogenic VOCs. EPA will propose a regulation to address carcinogenic contaminants as groups rather than individually in order to provide public health protections more quickly and also allow utilities to more effectively and efficiently plan for improvements.

Tetrachloroethylene (PCE) and trichloroethylene (TCE), which the EPA determined were candidates for regulatory revision under the second six year review of the existing NPDWRs will be included in the VOC drinking water standard. Besides PCE and TCE, the group may include up to six additional regulated VOCs; and up to eight unregulated VOCs from the EPA's Contaminant Candidate List (CCL) 3. In accordance with the requirements for setting new regulations, the Agency will evaluate the health effects of carcinogenic VOCs, the feasibility of treatment, the affordability of treatment for small systems, the costs and the benefits (as part of the Health Risk Reduction Cost Analysis), and implementation of a carcinogenic standard. The proposed rule is expected to be announced in 2013.

4.3 Future Regulations

4.3.1 Potential Individual Contaminant Regulations: CCL3 and Unregulated Contaminant Monitoring Rule (UCMR)3:

CCLs have been created every five years since 2003 to determine whether additional contaminants warrant regulation due to their prevalence in the environment and their health effects. The first two regulatory determination processes from the 2003 and 2008 CCLs did not result in any additional regulations. The final CCL3 was published in September 2009 and listed 116 contaminants for potential regulation and includes some EDCs and PPCPs. A complete list of CCL3 compounds is located at:

http://water.epa.gov/scitech/drinkingwater/dws/ccl/ccl3.cfm

It is expected that the next CCL process in 2013 will result in additional WQ regulations. Potential candidates for regulation include nitrosamines, especially N-nitrosodimethylamine (NDMA), and chlorate. Nitrosamines are a likely candidate due to the current health effects research showing a potential cause for concern and since NDMA was the most frequent analyte discovered in the first round of monitoring from the UCMR2.

If Piqua considers the implementation of chloramines for residual disinfection or chlorine dioxide as an oxidant at their new water plant, it is recommended to consider the impact that these potential regulations would have on their new treatment processes and distribution system operations. Additionally, systems with high naturally occurring or contamination from ammonia should carefully monitor the amount of nitrosamines formed.

The UCMR allows the EPA to collect data on the occurrence of different potential contaminants in drinking water. Participation in these regulations varies with EPA's data needs and the particular WQ or treatment processes of the utility. The UCMR3 list was released by EPA in February 2011 for initial review and can be found at:

http://water.epa.gov/lawsregs/rulesregs/sdwa/ucmr/ucmr3/methods.cfm

4.3.2 Six-Year Review

EPA is required under the SDWA to conduct an extensive review of all NPDWS every six years to determine whether modifying a current limit would provide for a meaningful opportunity for risk reduction. The first time this process was completed in 2003, EPA decided to revise the TCR.

The 2009 six-year review results were announced in March 2010. Of the 71 regulated contaminants assessed by EPA, four compounds were identified as candidates for more stringent regulations. The four regulated contaminants were:

- PCE
- TCE
- Actylamide
- Epichlorohydrin

PCE and TCE are industrial processing chemicals that have contaminated the nation's groundwater and surface water supplies, and which are impurities that are introduced into the finished water during the

treatment process. This determination will kick-start a process to determine what the new regulatory limits for these compounds will be based on health effects data.

As such, it is of utmost importance that the City of Piqua not only meets current, pending or future regulatory requirements, but be prepared for future regulations not yet promulgated. Other contaminants that are receiving the greatest attention and have significant potential to become parameters under new regulations are EDCs and PPCPs, iodoacetic acids, methyl-t-butyl ether (MTBE), etc. It is possible that EPA may issue new regulations in the future to better control these emerging contaminants to improve human health.

Section 5
Condition Assessment of Existing Facilities

This Section discusses the condition, need for rehabilitation or replacement and determination of whether and how the following existing facilities at the City's water supplies and treatment plant will be modified and integrated into the new WTP:

- River Pumping Station
- Gravel Quarry Pump Station
- Raw water intakes and screen/valve chambers at Swift Run Lake
- Sludge disposal force main and lagoon
- Filter Backwash and Sludge Disposal Facility

The condition assessment of the above facilities involved the following activities:

- Observing the existing physical conditions of the facilities
- Reviewing record drawings and operation and maintenance (O&M) manuals

The condition assessment of the facilities did not involve the following activities:

- Physical assessment of submerged facilities or equipment
- Analytical testing
- Visual observation of the interior of basins or piping to assess any corrosion or erosion

5.1 Evaluation Criteria

The existing facilities were assessed in terms of the following criteria:

Physical Condition of the Asset, C_A

- Each asset is evaluated on a scale of 1 to 5
- Excellent condition = 1
- Fair condition = 3
- Poor condition = 5
- Criticality of the Asset, CR

- Low = 1; minimal impact on treatment and control
- Medium = 3; alternatives available and/or have time to address problem
- High = 5; no or limited alternatives available and no or limited time to address problem

Consequence of Failure of the Asset, C_F

- Low = 1; minimal impact to process, staff or environment
- Medium = 3; specific training, awareness and/or equipment required to be in place to respond to the failure
- High = 5; may trigger an emergency response, response planning required

Redundancy of the Asset, R

- Poor = 1; component has no redundancy
- Moderate = 2; component redundancy is equal to 1 (e.g., one standby pump)
- Good = 4; component redundancy is greater than 1

Regulatory/Code compliance of the Asset, C_C

Each asset is assigned a score of either 30 or 0. If the asset does not comply with current regulations (e.g., OEPA, National Electric Code (NEC), Ohio Building Code (OBC), etc), then a value of 30 means it requires improvements /rehabilitation. For assets that comply with all codes and regulations, a C_C value of 0 is used.

Health and Safety Compliance of the Asset, C_H

Each asset is assigned a score of 30, 10, or 0. If the asset in its current condition and installation does not meet general Occupational Safety and Health Administration (OSHA) requirements and other safety features, it is assigned a C_H value of 30 or 10. A value of 30 represents an item of high concern and a value of 10 represents an item of moderate concern. If there are no health and safety issues associated with the asset in its current condition and installation, a C_H value of 0 is used.

Overall Condition Scoring

The ranking of each of the four criteria noted above is used to develop the overall Asset Condition Score. This score is calculated as follows:

$$\text{Asset Condition Score} = [(C_A \times C_R \times C_F) / R] + C_C + C_H$$

The Asset Condition Score is used as an indicator of items of concern that require immediate rehabilitation. This will also help the City identify assets that could potentially be retained for use with the new treatment plant. Problematic items are identified with an overall condition scoring of >30, require severe rehabilitation and therefore possess little or no value for integration with the new treatment plant. Items with an overall score of <30 could potentially be integrated with the new treatment plant with recommended minor improvements/rehabilitation.

The condition assessment of the five facilities is described in detail below.

5.2 River Pump Station

The River Pump Station was constructed as part of the original plant construction in 1925. A 36-inch raw water line connects the GMR to the pump station and a 20-inch discharge line conveys water from the pump station to the plant or to Swift Run Lake. The pump station has three horizontal split-case pumps. Plant staff indicated that Pump Nos. 1, 2, and 3 are rated for 10 MGD, 3 MGD and 6 MGD respectively.

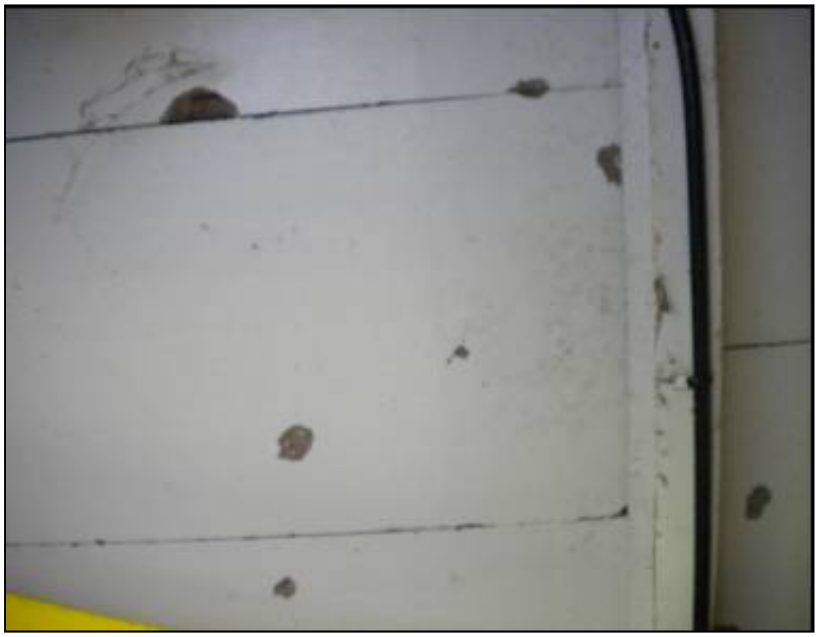
The pump station building is a concrete and masonry structure and is in generally fair to poor condition. The exterior masonry walls show signs of deterioration with visible spalling in some places. Brickwork above the door lintel has been tuck-pointed as shown in **Figure 5-1** and window openings are boarded up with painted plywood. Plant staff noted that the roof and the door have been replaced.

The interior ceiling concrete beams also appear to be spalling in several locations (**Figure 5-2**). There is a hoist beam in the first floor of the building to remove the raw water pumps that are housed in the lower level. This beam is not rated and hence possesses a major safety concern. It is currently not in use.

Figure 5-1. *River Pump Station Exterior Walls*

There are three raw water pumps in the lower level of the building that are manually controlled at the building. Although Pump Nos. 1, 2, and 3 are rated for 10, 3, and 6 MGD respectively, hydraulically they operate well below their rated capacity at 7.2, 2.4, and 4.7 MGD as indicated by the plant staff and would require extensive reconstruction or replacement to restore the original capacities. The 10 MGD pump is reported to have been replaced in the 1960s and the 6 MGD pump replaced in 1992. The pumps are critical in providing supply to the plant for treatment. Since there are three pumps, there is built-in redundancy at the pump station that allows for routine maintenance of the pumps, or allows one pump to be temporarily taken out of service.

Figure 5-2. *River Pump Station Interior Ceiling*



It is unclear how the River Pump Station would be integrated into the new plant scheme. However, based on a preliminary review of hydraulics and site elevations, it appears that the current pumps do not have sufficient head to pump water to the new site. This may be evaluated in detail further during preliminary design phase. In any event the existing pumps design would differ from the revised hydraulic conditions and the apparent inability of the pumps to meet their current design conditions would indicate a need to be replaced.

The condition of the piping in the pump station is also poor. As shown in **Figure 5-3**, the pump suction piping is rusted. The main steel discharge header pipe also leaks. Plant staff report that the suction pipe pit concrete chamber leaks at high ground water levels and the pump level concrete slab and walls leak at high ground water level during GMR floods.

The main electrical disconnect switch for the pump station is in poor condition. The breaker/feeder to Pump No. 1 is also in poor condition. The breaker/feeder for Pump Nos. 2 and 3 are newer and in better condition. Some of the wiring was replaced as part of the 1961 expansion. The pump station does not have standby electric power, remote controls or alarms.

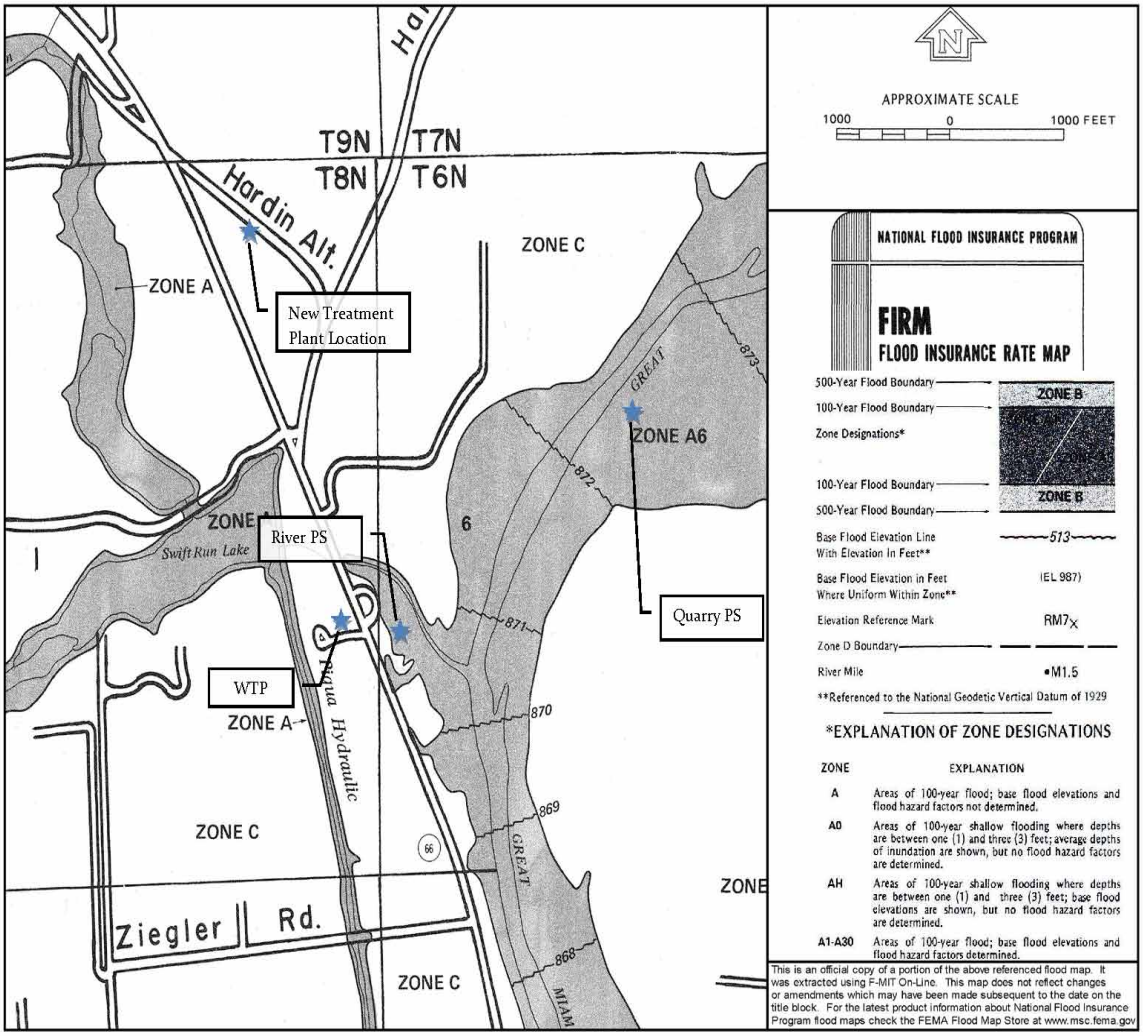
Figure 5-3. *River Pump Station Piping*

Other issues exist with this facility such as the location of the building in the flood plain. A flood map presenting the location of the existing WTP, the new treatment plant, River Pump Station and Quarry Pump Station is shown in **Figure 5-4.** The building does not comply with current OBC requirements or Ten States Standards (TSS) requirements which require all motors and electrical controls to be located above grade, although they are protected from flooding. TSS also requires all water supply facilities and treatment plant access roads to be protected to at least the 100-year (yr) flood elevation which is approximately 871 feet (ft) at this location. The entrance floor level of this building is at 874 ft according to the record drawings and this affords flood protection for the electrical gear and pumps which are located below grade. A major concern is the integrity of the structure long-term based on the evidence of leaks in response to high river level and ground water conditions. Because the pump station cannot be accessed nor remotely operated it is difficult and unsafe for staff to use this facility during flood periods.

Based on the above condition assessment, CDM recommends the following rehabilitation measures in order to use the river pump station to pump water to the new treatment plant:

- Replace all pumps with new pumps to meet future design requirements
- Replace all electrical equipment and controls with new equipment
- Install standby electric power
- Install remote controls and alarms
- Improve flood-protection for the building against 100-yr flood by locating and repairing sources of leaks
- Flood-protect electrical and controls equipment against 100-yr flood

Figure 5-4. *Flood Map*



- Replace hoist beam
- Replace/repair ceiling concrete beams to prevent loss of structural capacity

Rehabilitation of the raw water pump station will entail significant modifications to make the building comply with current regulations and codes and hence this facility possesses little value for integrating with the new treatment plant. This is further validated by the overall condition score for this facility as presented in **Appendix B1.** From Appendix B1, it is evident that all assets except the valves have an overall score >30 and hence possess little value for integrating with the new treatment plant. Serious consideration should be given to replacement of the river pump station instead of rehabilitation because of the need to maintain this facility in operation during the construction period.

5.3 Gravel Quarry Pump Station

The Gravel Quarry Pump Station was constructed in the 1970s to augment raw water supply. It is located northeast of the plant, east of the GMR adjacent to the lime sludge lagoon, and is supplied by both ground water recharge and the decanted supernatant from the lime sludge lagoon. There is a single variable frequency drive (VFD) vertical turbine pump that is rated for 2 MGD according to plant staff, with the pump and motor located in a concrete vault below grade as shown in **Figure 5.5**. Based on review of the pump O&M manual, it appears that this pump was installed in 1984 and the motor was replaced in 2005. Plant staff indicated that the pump performs below its rated capacity with an output of approximately 1.2 MGD. Electrical and controls including the VFD for the pump are housed in a concrete block masonry building adjacent to the pump station. This building appears to be in good condition.

As with the River Pump Station, there are several options available for using the Gravel Quarry Pump Station with the new treatment plant. Based on a preliminary review of hydraulics and site elevations, it appears that the current pump does not have sufficient head to deliver water to the new treatment plant site. This may be evaluated in detail further during preliminary design phase.

According to the existing pump station drawings, the pump station was originally designed to operate with a water level between 845.59 ft and 855.59 ft. The water level in the abandoned quarry is reportedly above the design operating level which causes the pump pit to flood according to plant staff. The pump vault and the electrical and controls building are both located in the flood plain as shown in Figure 5-4.

The operating floor of this building and the electrical equipment including the VFD are located at approximately elevation 868 ft which is below the 100-yr flood elevation of 873 ft. Ohio EPA requires electrical equipment to be located 3-ft above 100-yr flood elevation.

Figure 5-5. *Quarry Pump Station*

The discharge pipe penetration of the vault wall leaks due to the higher water level. The piping inside the vault is rusty and in poor condition. The concrete vault itself appears to be in good condition. The pump station has no remote control or remote monitoring capabilities.

The gravel pit is critical in augmenting raw water supply to the plant. It provides the best water in terms of WQ. Consequence of failure of the gravel pit pump is high because there is no redundancy provided for emergency purposes. Based on the above condition assessment, CDM recommends the following remediation measures in order to use the gravel quarry pump station with the new treatment plant and continue to retain the best WQ source:

- Replace pump vault with new bigger vault
- Replace existing pump with new pump to meet new design conditions
- Add one additional new pump to provide redundancy
- Install remote controls and alarms
- Flood-protect the existing electrical and controls building and the pump vault against 100-yr flood
- Evaluate electrical demands to confirm if there is sufficient capacity to power a second pump and associated control instruments (assuming it is desired to be able to operate two pumps at the same time).
- Install standby electric power

Overall condition score for this facility is presented in **Appendix B2.** From Appendix B2, it is evident that the valves, piping and drives have an overall score <30 and possibly can be re-used when this pump station is integrated with the new treatment plant. Compatibility of the pump, motor and drives with future design conditions would need to be confirmed during detailed design.

5.4 Raw Water Intakes and Screen/Valve Chambers at Swift Run Lake

CDM was unable to access the raw water intake structure due to high river level and hence its condition was not assessed. It is original to the plant and was built in 1925. From the record drawings it appears screening is not provided at the river intake.

There are two screen chambers located northwest of the treatment plant at Swift Run Lake. Both screen chambers are below grade. Screen Chamber No. 1 has a concrete block masonry building superstructure over the chamber and Screen Chamber No. 2 has no building.

The concrete block masonry building of Screen Chamber No. 1 appears to be in fair condition although the paint is in poor condition as shown in **Figure 5-6**. The exhaust hood on the exterior is damaged as shown in **Figure 5-7**. Some of the windows are broken and boarded up with plywood. The interior concrete ceiling is spalling in several locations. As shown in **Figure 5-8**, the gate and valve operators inside the building are rusted and are in very poor condition. Plant staff indicated that some of the operators do not function. Plant staff also indicated that the static 3/8 inch opening manually cleaned screens inside Screen Chamber No. 1 have holes in them.

Figure 5-6. *Screen Chamber No. 1*

Figure 5-7. *Screen Chamber No. 1 Exhaust Hood*

Figure 5-8. *Screen Chamber No. 1 Gate Operator*

The City installed galvanized steel hoods on the vent covers for Screen Chamber No. 2 which appear to be in good condition as can be seen in **Figure 5-9**. The condition of the screens in Screen Chamber No. 2 is unknown. Some sluice gates for Screen Chamber No. 2 are not operable. Some valves for Screen Chamber No. 2 are also inoperable because the valve operators are broken.

Figure 5-9. *Screen Chamber No. 2*

As part of the condition evaluation of the facilities, CDM did not inspect the screens since they are located below grade in vaults. The screen chambers are critical assets since they remove debris, which may damage downstream equipment, from the influent water prior to treatment.

Based on the above condition assessment, CDM recommends the following rehabilitation measures in order to use the screen chambers with the new treatment plant:

- Replace all valves and gates for both Screen Chamber Nos. 1 and 2
- Replace the screens inside Screen Chamber Nos. 1 and 2
- Demolish the building for Screen Chamber No. 1 and provide access to the screens from slab on grade similar to Screen Chamber No. 2.
- Consider replacement of the manually cleaned screens with mechanically cleaned intake screens especially since the screening structure will be remote from the treatment plant and less convenient for staff to access.

The overall condition score for this facility is presented in **Appendix B3** and it is evident that the sluice gates, piping and valves require replacement since they have an overall score of >30. While the screens have an overall score <30, CDM still recommends replacement of the screens due to their poor condition.

CDM also examined the condition of the raw water line junction chamber that is located ahead of and near the screen chambers. The valves, piping and the concrete associated with the junction chamber appear to be in good condition and appear to be suitable for incorporating into the new WTP facilities.

5.5 Sludge Disposal Force Main and Lagoon

CDM did not evaluate the sludge disposal force main because the piping is buried. The City has not reported any plugging or breakage since it has been in service. The sludge lagoon is located northeast of the River Pump Station on the east side of the GMR and is adjacent to the gravel quarry pumping station. The sludge influent pipe was originally built in 1975 it ties into the lagoon at the north end. There is a concrete decant structure at the north end which appears to be in good condition. The water decanted from the sludge lagoon drains to the gravel quarry raw water supply. The lagoon and the decant structure are shown in **Figure 5-10** and **Figure 5-11**. Overall, the lagoon appears to be in good condition and could be used with the new WTP.

Figure 5-10. *Sludge Disposal Lagoon*

Figure 5-11. *Sludge Disposal Lagoon Decant Structure*

The sludge disposal force main and lagoon are critical assets as they provide the only means of sludge disposal and storage for the existing treatment plant. However there is no redundancy provided since there is only one lagoon and one force main. TSS recommends that sludge lagoons include two cells at lime softening plants. There is an open pit adjacent to the existing lagoon where a new lagoon could be

constructed to provide additional storage. Alternatively, the City could evaluate if it desires new lagoons be located at the proposed plant site or whether to maintain the existing lagoon as a backup for mechanical dewatering alternatives.

Based on the above condition assessment, CDM recommends the following remediation measure in order to use the sludge disposal force main and lagoon with the new treatment plant:

- Construct additional lagoon in the open pit adjacent to the existing lagoon

The overall condition score for this facility is presented in **Appendix B4.** It is evident that the overall score for the force main is >30 but CDM does not recommend any rehabilitation measure for the force main at this time. However the City should re-assess the condition of the force main in the future if the discharge pressure on the sludge pumps increases indicating possibility of scale buildup in the force main.

5.6 Filter Backwash and Sludge Disposal Facility

The Filter Backwash and Sludge Disposal Facility includes a filter backwash reclaim basin and a waste sludge pump station. It was originally constructed in 1983 and is located to the east of the WTP. The operating floor of the pump station is above the 100-yr flood elevation and hence the building is not subjected to flooding. The holding basin appears to be in excellent condition as shown in **Figure 5-12** and shows no signs of structural deterioration. It should be noted that CDM did not visually observe concrete surfaces below the operating water level in the sludge holding basin. There is one sluice gate on the sludge drain line to the holding basin which also appears to be in good condition.

The filter backwash reclaim pump station has two pumps for pumping the supernatant from the basin into the raw water supply piping to the water plant influent. These pumps operate one at a time and alternate in lead/lag mode per cycle. The waste sludge pump station also has two pumps that pump the sludge from the basin to the remote sludge lagoon. These pumps are shown in **Figure 5-13.** Plant staff indicated that the sludge pumps experience extreme wear and are replaced every few years. The sludge pumps also alternate in a lead/lag mode per cycle. The piping and the valves in the filter backwash and waste sludge pump station appear to be in excellent condition.

The building superstructure and interior concrete are in excellent condition as shown in **Figure 5-14**. Electrical components also appear to be in good condition. A catastrophic failure of the filter backwash facility is unlikely based on visual observations made during this condition assessment. There is pumping redundancy in case of failure of either the supernatant or the sludge pumps. However, the basin itself does not have redundancy and has to be taken out of service for routine maintenance and cleaning. Based on field observations, this facility has the potential to be integrated with the new treatment plant without any immediate rehabilitation measures. The only drawback to this is that the backwash facility would be at a remote site which is difficult to deal with in terms of O&M. The City should add remote monitoring capability when this site is integrated with the new plant. The hydraulics of the existing supernatant pumps to convey water to the head of the new plant will be evaluated during the detailed design phase.

The overall condition score for this facility is presented in **Appendix B5**. It is evident that the overall score for all assets in this facility are >30 and hence can be used with the new treatment plant.

Figure 5-12. *Filter Backwash Holding Basin*

Figure 5-13. *Filter Backwash Sludge Pumps*

Figure 5-14. *Filter Backwash and Sludge Disposal Facility*

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Section 6
Process Alternatives Development and Screening

This Section presents a summary of WTP unit processes and process train alternatives considered for the City's new SWTP and the screening procedure used to short list the alternatives for further preliminary design development. CDM presented ten process train alternatives to the City at Workshop No. 1 (Process Evaluation Workshop). The initial screening of the ten process train alternatives down to four was completed jointly by the CDM team and City staff at the workshop, and the results are presented in this Section.

6.1 Description of Process Alternatives

Ten process train alternatives were developed for the City's new SWTP with the aim of meeting the following major WQ goals:

- Turbidity
- Disinfection
- TOC/ DBPs
- Atrazine
- Taste and Odor
- Hardness
- EDCsPPCPs

These WQ goals were jointly established by the City and CDM. All of the developed alternatives met or exceeded the above WQ goals except EDCs and PPCPs. The capability of the process train alternative to meet emerging contaminants of concern such as EDCs and PPCPs varied significantly across alternatives, depending on the unit processes employed. An array of unit processes was presented to the City and used for development of the process train alternatives as presented below.

6.1.1 Unit Process Alternatives

6.1.1.1 Pretreatment Alternatives

PAC

Activated carbon is known to be an effective adsorbent to remove naturally occurring organic matter (NOM), color, taste and odor compounds such as 2-methylisoborneol (MIB), geosmin or other seasonal WQ issues.

Activated carbon can be used both in the form of GAC or as PAC. PAC is usually fed as a slurry to the raw water to maximize contact time prior to sedimentation. The primary difference between PAC and GAC is that the particle size of PAC is typically smaller than GAC. PAC is also effective in the control of a variety of synthetic organic compounds such as herbicides and pesticides like atrazine which is a major seasonal contaminant in the City's water source. However, PAC may not be necessary if another form of enhanced organic contaminant removal such as GAC or Magnetic Ion Exchange (MIEX) are part of the final design for the new SWTP.

Magnetic Ion Exchange

The MIEX® process is a patented ion exchange process for enhanced organics removal. The MIEX DOC resin is specifically used for removal of dissolved organic carbon (DOC) from drinking water. Negatively charged anions are removed by exchanging with a chloride ion on the resin surface. In the regeneration process, a resin bead loaded with DOC undergoes a reversed ion exchange reaction, where chloride ions are substituted for DOC in a concentrated brine solution (sodium chloride or sodium bicarbonate). Previous research reported that MIEX could remove 75% of DOC after 30 minutes of contact time for a surface water with the concentration of 6.0 mg/L. However, there are insufficient reports about atrazine removal capacity for MIEX; however, based on recent research results, it appears that MIEX by itself is not effective to remove atrazine. Since high atrazine concentrations are a cause of concern to the City, this process has to be used in conjunction with another process such as GAC or PAC in order to address atrazine levels in raw water. As a result, this unit process was not considered in the development of process train alternatives.

Pre-Oxidation

An oxidant may be added to the raw water to improve organics removal in the downstream coagulation/flocculation process. It is effective for the reduction of SOCs such as atrazine. Pre-oxidants may also control tastes and odors and biological growth in the plant and the distribution system. Commonly used oxidants include chlorine, chlorine dioxide, potassium permanganate and ozone.

A comparison of the pretreatment alternatives is shown in **Table 6-1**.

Table 6-1. Pretreatment Alternatives Comparison

Process Name	Advantages	Disadvantages
PAC	▪ Dosage can be varied ▪ Ease of introduction in rapid mix chamber ▪ High efficiency for a variety of contaminants	▪ Labor intensive feed system ▪ PAC dust is explosive in nature and requires additional care for handling ▪ PAC storage and feed system must be in a separate area with explosion-proof electrical and control systems ▪ Increased solids handling
PRE-OXIDATION	▪ Dosage can be varied ▪ Ease of introduction in rapid mix chamber ▪ May reduce downstream coagulant dosage	▪ Chemical storage and feed system must be in a separate area with corrosion-proof electrical and control systems ▪ Use of chlorine compounds or ozone results in chlorinated and brominated by-products that are harmful to health

6.1.1.2 Clarification Alternatives

Conventional Sedimentation

Conventional coagulation, flocculation and clarification are proven surface water technologies for the removal of colloidal and suspended particles from water. Usually, this is achieved by the addition of aluminum or an iron-based coagulants such as aluminum sulfate, ferric chloride or ferric sulfate in a rapid mix chamber to reduce the dissolved organics content in raw water. Flocculation is the process of slowly mixing the water to provide opportunities for particles to collide. After particles are destabilized and/or precipitated by coagulation in the rapid mix tank, they are agglomerated into larger particles or "floc" in the flocculation tank which are typically rectangular basins. Flocculation is used to improve the removal of particles and DOC through sedimentation and filtration.

Clarification/sedimentation is a solid-liquid separation process. Conventional sedimentation tanks are usually circular or rectangular. Sedimentation tanks have longer detention time to allow ample time for floc particles to settle to the bottom, where they are removed. This process reduces the solids loading to filters. The recommended minimum detention time by TSS for conventional coagulation, flocculation and sedimentation are:

- Rapid Mix – 30 seconds
- Flocculation – 30 minutes
- Sedimentation – 4 hours

Plate and Tube Settling

Inclined plate technology utilizes a shorter hydraulic detention time and a smaller footprint than a conventional sedimentation process. Plate and tube settlers can be operated at a much higher loading rate than conventional clarifiers. A flash mixer and flocculator are usually provided ahead of the plate and tube settlers to mix in coagulant and/or polymer that promote floc growth and enhance the clarification process. A typical plate settler arrangement consists of three major components positioned along a rectangular settling tank with an inclined backside at a slope of 45-60 degrees. The sections are:

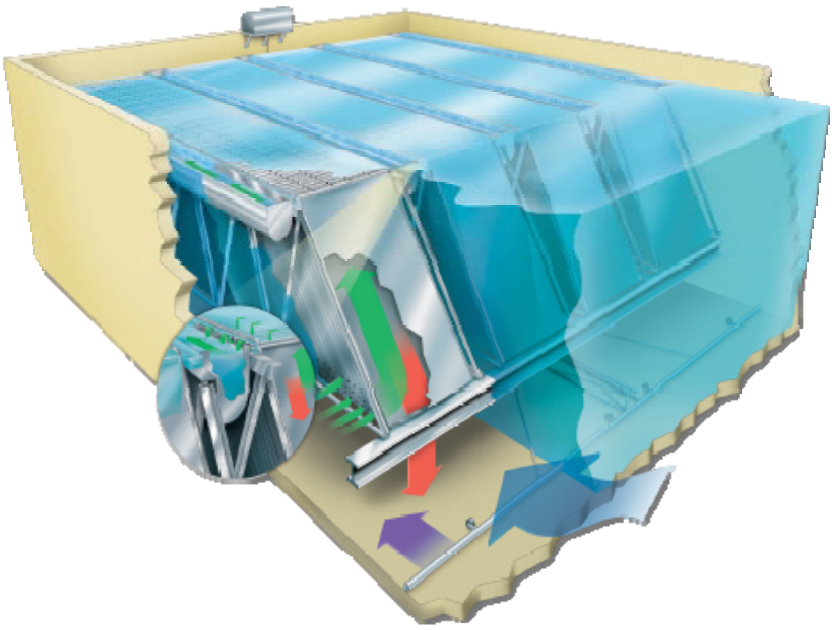
- A central slurry feed launder with a feed connection
- Inclined plate pack compartments
- Overflow launders with supernatant discharge connection

The influent feed enters through the feed inlet and is directed down and sideways towards the plate compartments. As the slurry reaches the bottom end of the inclined plates stack it turns upwards and flows between the parallel plates in a laminar flow pattern. Here the free settling solid particles separate to a sludge that slides down the plates while the gradually clarifying liquid moves up in a counter-current pattern. The sludge is withdrawn from the sludge hopper at the bottom. A typical arrangement is shown in **Figure 6-1**.

As with the addition of any new unit process, it is recommended that plate settlers be sized by laboratory settling tests to determine the particle settling velocity, sludge volume and polymer dosage required. Optimization of the design loading rate could have a significant impact on the number of units needed and the associated construction costs.

Like plate settlers, tube settlers also shorten detention times for sedimentation. By using several shallow parallel tubes, surface area can be greatly increased within a very small footprint. A bundle of tubes provides many "false floors" onto which floc particles could settle, agglomerate, and slough down to the bottom end of the tube to be discharged and settle to the bottom of the settling basin. Recently the use of thicker, more ultraviolet (UV) resistant polyvinyl chloride (PVC) has greatly improved material life expectancy over previous materials and models.

Figure 6-1. *Plate Settler*



Although inclined plate settler systems are similar in principle to the tube settlers, the inclined plates provide higher flows per square foot over the same surface area. Unlike tube settlers, plate settlers use orifices along the sides of each plate to equalize and distribute flows across each plate and are not subject to surges and uneven basin flows. While tube settlers tend to be PVC, fiberglass reinforced plastic (FRP) or other light weight material, the inclined plate settlers may be constructed of stainless steel, aluminum, PVC or FRP and provide greater structural integrity.

Dissolved Air Flotation (DAF)

In the DAF process, suspended raw water particles are flocculated and separated out of the water by floating them to the surface (flotation), rather than settling them to the bottom of the basin (settling). Solids that are formed during the coagulation process are removed by attaching microbubbles to the floc, floating solids to the surface, and skimming by mechanical or hydraulic means. Flotation is achieved through formation of tiny air bubbles which are introduced through diffusers at the bottom of the contactor to float the floc. The clarified water is removed by laterals from the bottom of the basin.

DAF is particularly effective in removing low-density solids such as turbidity, color, algae, *Giardia*/*Cryptosporidium*, and precipitated organics and metals. These contaminants do not settle well and have a tendency to float. The additional complexity of the DAF process limits it to raw waters with

higher turbidity, color, and algae than present in the City's source water. As a result, this unit process was not considered in the development of process train alternatives.

Ballasted Clarification Process

The ACTIFLO process is a high rate clarification process that utilizes microsand as a seed for floc formation. The microsand provides surface area that enhances flocculation and acts as a ballast, or weight for the floc particles. The resulting sand-ballasted floc has very effective settling characteristics which allow for the ACTIFLO process to operate at high overflow rates and short detention times. In general, the footprint for an ACTIFLO system is five to twenty times smaller than conventional clarification systems. Because it is a high-rate process, Ohio EPA will likely require piloting to get ACTIFLO approved for full-scale implementation. As a result, this unit process was not considered in the development of process train alternatives.

A comparison of the clarification alternatives is shown in **Table 6-2.**

Table 6-2. Clarification Alternatives Comparison

Process Name	Advantages	Disadvantages
CONVENTIONAL	▪ Ease of operation ▪ Known and proven process ▪ Piloting not required	▪ Large footprint
PLATE AND TUBE SETTLING ▪ Lamella packaged settlers ▪ Plate settlers in rectangular basins	▪ Shorter hydraulic detention time ▪ Smaller footprint ▪ Partially preassembled from factory	▪ Difficult to clean lime and scale deposits from plates ▪ Require significant structural supports ▪ Ohio EPA requires piloting

6.1.1.3 Softening Alternatives

Lime-Soda Softening

Lime-soda softening is a chemical precipitation process that uses calcium hydroxide and sodium carbonate for the removal of carbonate and non-carbonate hardness, respectively, resulting in hardness causing minerals to form insoluble precipitates. The precipitates are usually removed in downstream processes. Lime soda softening requires coagulation, flocculation, sedimentation and recarbonation, typically with carbon dioxide.

Membrane Softening

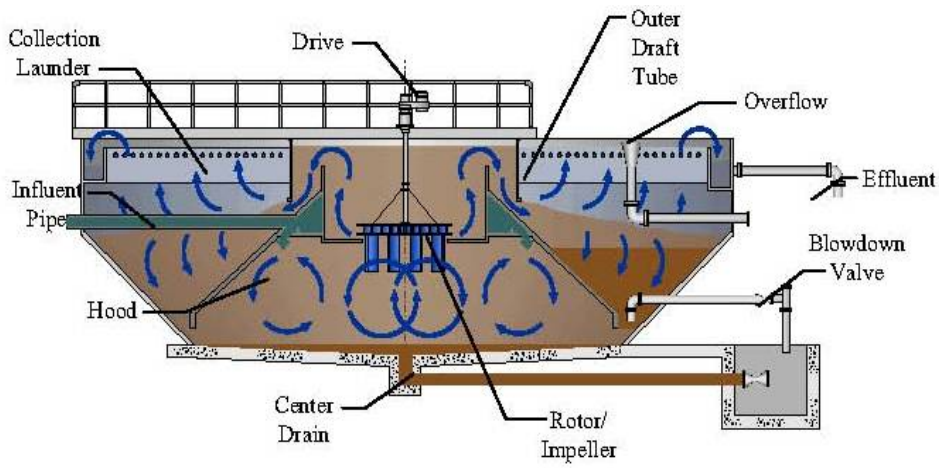
Membrane softening typically requires conventional sedimentation or membrane filtration process such as microfiltration (MF) or ultrafiltration (UF) as pretreatment. Nanofiltration (NF) and low pressure reverse osmosis (LPRO) are most common high pressure membrane softening processes. Both technologies use a semi-permeable membrane and cross-flow filtration to produce a purified "permeate" and a waste concentrate. Membrane softening effectively removes hardness, organics, color and bacteria. Both NF and LPRO are effective in the removal of TOC and SOCs, such as atrazine, and may provide other treatment benefits.

Solids Contact Clarifier

In a solids contact clarifier, coagulation, flocculation, and sedimentation take place in one basin. The unit employs internal solids recirculation to accelerate chemical reactions and dense particle growth. Key to operation is the "rotor impeller," which controls mixing and slurry recirculation at independent rates to

handle rapid changes in water characteristics. The solids contact clarifier maintains active slurry volume throughout the treatment zones, with settling taking place only in the sludge concentrators. It also allows for waste sludge removal with or without the use of bottom scrapers. Raw water typically enters the primary mixing and reaction zone, where it is mixed with previously formed slurry and treatment chemicals such as ferric sulfate, lime, soda ash etc. The rotor-impeller combination prevents solids from settling on the basin floor and promotes dense particle growth. The flow is directed to the secondary mixing and reaction zone, where continued slurry contact allows the treatment reactions to approach equilibrium. Flow is discharged downward through the return flow zone and onto the surface of the slurry pool. Treated water is displaced upward and collected in launders, while slurry is redirected to the primary mixing and reaction zone. Excess sludge is collected in the sludge concentrator, from where it is withdrawn. A typical solids contact clarifier is shown in **Figure 6-2.**

Figure 6-2. *Solids Contact Clarifier*



A comparison of the softening alternatives is shown in **Table 6-3.**

Table 6-3. Softening Alternatives Comparison

Process Name	Advantages	Disadvantages
LIME /SODA SOFTENING	▪ Ease of operation ▪ Known and proven process ▪ Piloting not required	▪ Large footprint ▪ Efficient lime slaking is key to operation ▪ Requires lime storage and feed facility which are messy ▪ Requires handling and disposal of lime sludge ▪ Requires efficient process control
MEMBRANE SOFTENING	▪ NF and LPRO are effective in reducing atrazine concentrations ▪ THM formation potential reduced ▪ Smaller footprint	▪ High waste volume-typically 10-20% of incoming flow ▪ Requires disposal of concentrate stream ▪ Ohio EPA requires piloting
SOLIDS CONTACT CLARIFIER	▪ Minimum chemical requirement ▪ Piloting not required	▪ Requires handling and disposal of lime sludge ▪ Requires sludge wasting frequency optimization

6.1.1.4 Filtration Alternatives

Conventional Filtration

Rapid sand filtration refers to filtration rates on the order of 2 to 10 gallons per minute per square foot (gpm/sf). There are three basic media configurations: monomedia, dual media, and mixed or multi-media. The most common media types are sand, anthracite, GAC, and garnet. Monomedia filters are generally sand. Dual media filters are most commonly sand and anthracite, and mixed media filters are typically sand, anthracite, and garnet.

Membrane Filtration

MF and UF are low-pressure membrane filtration processes primarily used for particle removal. MF and UF are effective in removing *Giardia* and *Cryptosporidium* to below detection, as well as meeting the turbidity requirements of surface water treatment regulations. However, only UF is able to achieve any significant removal of viruses. Selection of a membrane is usually determined during pilot-testing. Ohio EPA will require pilot-testing of membrane filtration systems. Two types of membrane systems commonly used in water treatment are pressure and submerged.

GAC Filter Adsorbers

GAC is used in water treatment to adsorb a variety of organic and inorganic compounds, in particular NOM, which are the precursors to DBP formation. GAC is also effective for removal of taste and odor compounds, volatile organic compounds (VOCs), SOCs such as herbicides and pesticides including atrazine, EDCs, P&PCP and other emerging contaminants.

Fixed bed downflow contactors offer the most common contactor configuration for drinking water treatment. Flow through contactors can be driven either by gravity or pressure; however, when used as the primary filter system Ohio EPA requires gravity flow.

A comparison of the filtration alternatives is shown in **Table 6-4.**

Table 6-4. Filtration Alternatives Comparison

Process Name	Advantages	Disadvantages
CONVENTIONAL FILTRATION	▪ Ease of operation ▪ Known and proven process ▪ Piloting not required	▪ Does not remove viruses
MEMBRANE FILTRATION	▪ UF can remove viruses ▪ Better pathogen removal than conventional sand filters	▪ Requires periodic chemical cleaning ▪ Ohio EPA requires piloting
GAC FILTER ADSORBERS	▪ Cost effective since it eliminates the need for post-filter full-scale GAC contactors ▪ Requires replacement of spent GAC media	▪ May lead to media loss during backwash since GAC has a lower density than anthracite ▪ Increases backwash frequency because of bacterial growth in GAC beds

6.1.1.5 Advanced Treatment Alternatives

Post-Filter GAC Contactors

These are mainly used to adsorb NOM; however, GAC also has been proven effective for removal of taste and odor compounds, VOCs and SOCs. Fixed bed vertical GAC pressure contactors with downward flow pattern are most common for removal of TOCs. The efficiency of the GAC adsorption process is dependent on GAC particle size, contact time, bed depth and loading rate. A typical GAC pressure vessel is shown in **Figure 6-3**.

Figure 6-3. *GAC Adsorber*

Advanced Oxidation Process (AOP) with UV/Peroxide

The UV/peroxide process combines UV light with hydrogen peroxide to inactivate *Giardia* and *Cryptosporidium* parasites and bacteria in the water by preventing replication of the organism. This process can be designed for complete destruction of taste and odor compounds, including geosmin. The required UV dose for advanced oxidation is typically 10 to 30 times higher than for disinfection applications, which significantly increases the number of UV reactors, UV lamps and power requirements. The required hydrogen peroxide dose ranges from 8 to 12 mg/L. Chlorine must be added downstream of the UV/peroxide process to quench the peroxide residual at a 2:1 chlorine/peroxide dose ratio (or 16 to 24 mg/L) assuming that the peroxide acts as a catalyst and is not consumed through the oxidation process.

AOP with Ozone

Because of the strong oxidizing power of ozone, it is effective for removing many emerging contaminants (including many EDC and PPCP), reducing taste and odor causing organics, and converting high molecular weight TOC into low molecular weight biodegradable organic carbon (BDOC), which can be more easily adsorbed by GAC or consumed by microorganisms on the GAC media surface. The process which uses a combination of ozone and GAC to create a biological filtration system is referred to as enhanced biological GAC filtration

A comparison of the advanced treatment alternatives is shown in **Table 6-5.**

Table 6-5. Advanced Treatment Alternatives Comparison

Process Name	Advantages	Disadvantages
POST-FILTER GAC CONTACTORS	▪ Effective in removing wide variety of organic contaminants ▪ Effective for taste and odor control ▪ GAC can be regenerated and 're-used' ▪ Sludge-free process	▪ High capital and O&M cost ▪ Bacterial growth will occur in the media resulting in elevated heterotrophic plate counts in the effluent that must be managed.
ADVANCED CHEMICAL OXIDATION	▪ Very strong oxidation ▪ Effective for taste and odor control ▪ Does not produce halogenated DBPs	▪ Produces biodegradable organic byproducts that may contribute to biofouling in the distribution system

6.1.1.6 Disinfection Alternatives

Non-Chlorine Alternatives

UV Disinfection

UV disinfection is a relatively low cost and simple technology that provides a significant additional barrier against microbial contaminants in the water. UV disinfection is a process where UV light is used to inactivate *Giardia* and *Cryptosporidium* parasites and bacteria in the water by preventing replication of the organism. UV disinfection works best on filtered water, after the removal of suspended solids. While UV has no impact on removal of organic or inorganic compounds at typical disinfection doses, it is generally less expensive to construct and operate than ozone for inactivation of *Cryptosporidium,* a chlorine-resistant pathogen.

Ozonation

The use of ozone in water treatment provides multiple WQ benefits such as oxidation, primary disinfection and control of DBP. While ozone does not contribute to TTHM and HAA5 formation, it can form bromate (a regulated DBP) when high concentrations of bromide and ozone are present at neutral or higher pH. An ozone generation system is shown in **Figure 6-4**.

***Figure 6-4**. Ozonation System*

Chlorine Alternatives

Chlorine is most commonly used for reduction of pathogens. Efficiency of the chlorine disinfection process depends on disinfectant concentration, contact time, temperature and pH. Since Ohio EPA requires some form of chlorine residual in the finished water, non-chlorine disinfectants should always be used in conjunction with a chlorine disinfectant at the WTP.

Gaseous Chlorine

Chlorine gas is released from a liquid chlorine cylinder by a pressure reducing and flow control valve operating at a pressure less than atmospheric. Chlorine gas is dangerous and is lethal at concentrations as low as 0.1 percent air by volume. Due to safety concerns, industry standard has moved away from the use of gaseous chlorine and hence CDM does not recommend the use of gaseous chlorine for disinfection at the City's new treatment plant.

Sodium Hypochlorite

Sodium Hypochlorite is available as a solution in concentrations of 5 to 15 percent chlorine, but is more expensive than chlorine gas. Sodium Hypochlorite can also be generated onsite by electrolysis of sodium chloride solution in specialized equipment.

Chlorine Dioxide

Chlorine dioxide is commonly used as a pre-oxidant for improved coagulation and filterability, without formation of chlorinated disinfection by-products. It can also be used for primary disinfection. Chlorine dioxide is less effective than ozone at breaking down high molecular weight organics or oxidation of emerging contaminants and taste and odor compounds. Chlorine dioxide is typically generated onsite using a mixture of sodium chlorite and one to two other chemicals. Chlorine dioxide can cause excessive chlorite or chlorate residuals if over dosed, which generally limits the maximum dose of chlorine dioxide to around 1.0 mg/L. Since chlorine dioxide residual is relatively unstable compared to chlorine, CT compliance is usually more difficult to achieve and as a result, this disinfectant was not considered for the new plant.

Chloramines

Chloramines are formed when water containing ammonia is chlorinated or when ammonia is added to water containing chlorine. It is usually generated onsite. It is the least cost option for DBP control. However, Ohio EPA requires additional distribution system monitoring for plants using chloramines for primary disinfection. The approval process through Ohio EPA is also intensive. As a result, this was not considered in the in the development of process train alternatives.

A comparison of the disinfection alternatives is shown in **Table 6-6.**

6.1.1.7 Summary

A summary of the WQ goals and unit process alternatives available to accomplish these goals are shown in **Table 6-7.**

Table 6-6. Disinfection Alternatives Comparison

Process Name	Advantages	Disadvantages
SODIUM HYPOCHLORITE	▪ Chlorine based disinfectants have a measurable residual ▪ Safer to handle than gaseous chlorine ▪ Effective in inactivating viruses and cysts	▪ Corrosive substance ▪ Does not inactivate *Giardia* and *Cryptosporidium*
NON-CHLORINE ALTERNATIVES	▪ May reduce DBPs ▪ Effective in inactivating viruses and cysts	▪ UV and ozone do not have a residual. Ohio EPA requires a minimum disinfectant residual (provided by free chlorine or chloramines) of 0.2 mg/L in the distribution system. ▪ UV has higher capital costs, is energy intensive and more difficult to operate for primary disinfection

Table 6-7. WQ Goals and Unit Process Alternatives

WQ Goals	Unit Process Alternatives
Turbidity	▪ MF/UF ▪ Conventional Filtration
Disinfection	▪ UV ▪ Ozone ▪ Chlorine ▪ MF/UF ▪ Filtration
TOC/DBP Control	▪ Chloramines ▪ Ozone ▪ GAC ▪ Enhanced Coagulation
Atrazine	▪ PAC ▪ GAC ▪ NF/RO
Taste and Odors	▪ GAC ▪ UV/Peroxide ▪ Ozone ▪ NF/LPRO
Hardness	▪ NF/LPRO ▪ Lime Softening ▪ Solids Contact Clarifier

6.1.2 Conventional Process Train Alternatives

CDM developed the following six conventional process train alternatives with the aim of meeting the WQ goals defined in **Section 2.0** and **Section 6.1** above:

- Alternative 1A – Lime Softening
- Alternative 1B – Lime Softening with Ozone
- Alternative 1D – Lime Softening with Post-Filter GAC Contactors
- Alternative 3A – Enhanced Coagulation and Lime Softening
- Alternative 3B – Enhanced Coagulation and Lime Softening With Ozone
- Alternative 3D – Enhanced Coagulation and Lime Softening with Post-Filter GAC Contactors

Conventional process train alternatives include processes such as clarification, lime-soda softening in conventional basins, stabilization, intermediate ozonation, conventional sand filtration or dual-media (GAC/sand)biological filtration, post-filter GAC contactors for advanced treatment and disinfection. Process flow diagrams for these alternatives are shown in **Figure 6-5** to **Figure 6-10**.

***Figure 6-5**. Alternative 1A – Lime Softening*

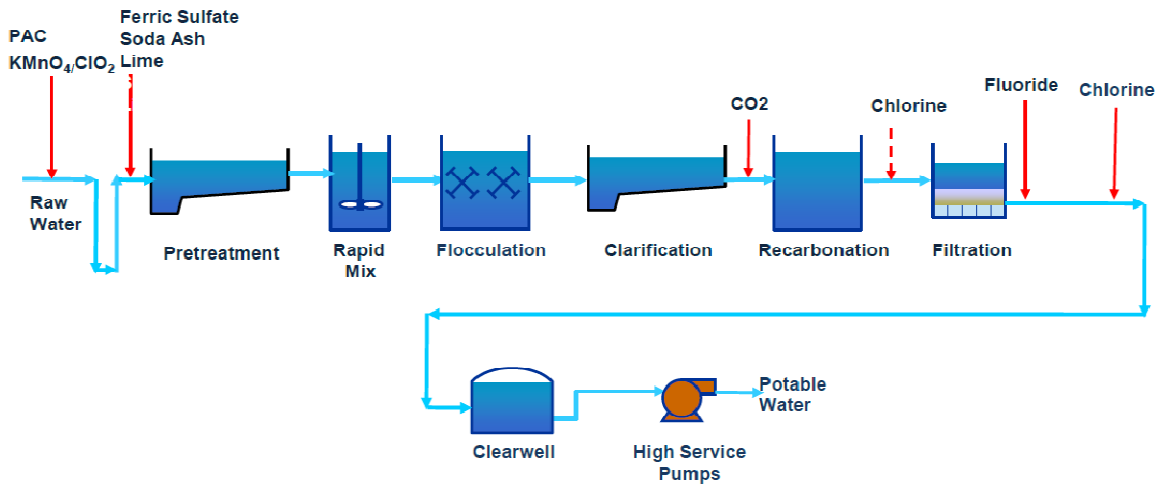


***Figure 6-6**. Alternative 1B – Lime Softening with Ozone*

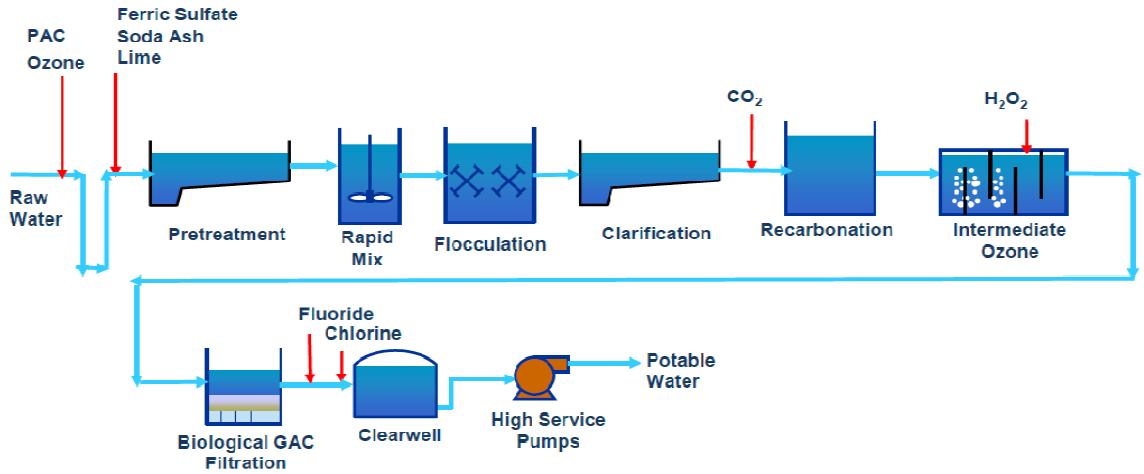


Figure 6-7. Alternative 1D – Lime Softening with Post-Filter GAC Contactors

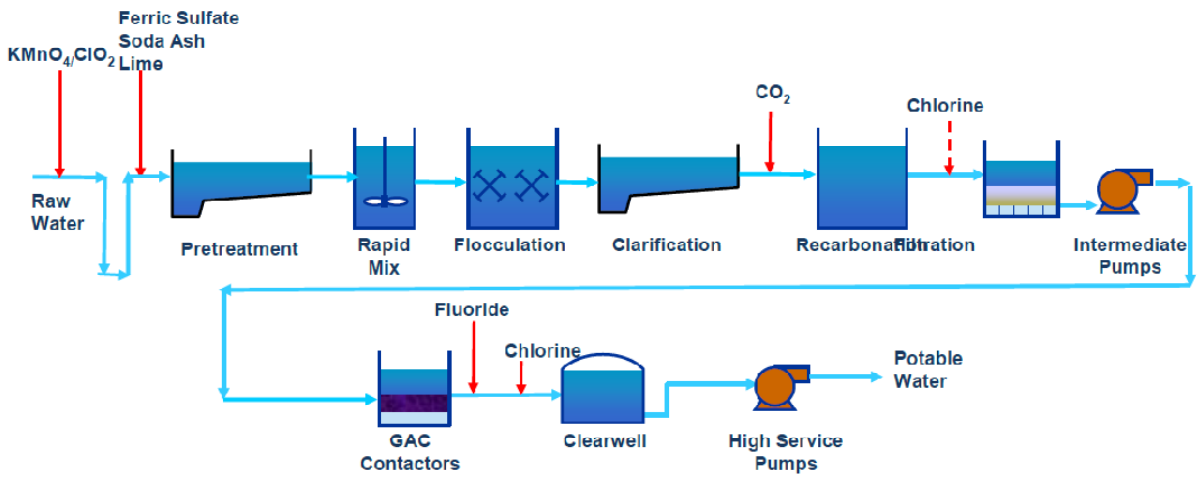


Figure 6-8. Alternative 3A – Enhanced Coagulation and Lime Softening

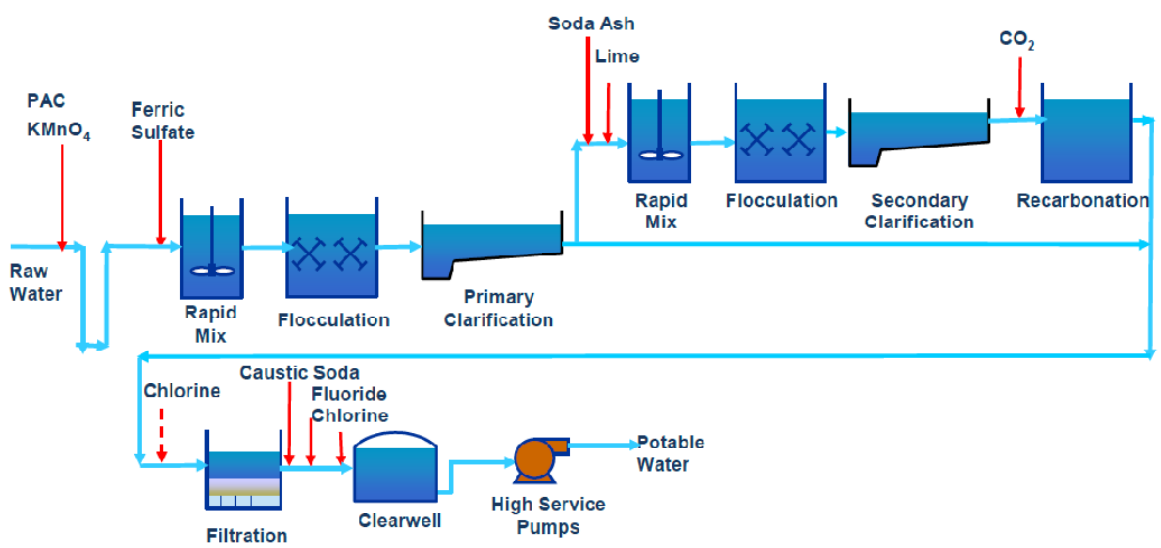


***Figure 6-9**. Alternative 3B – Enhanced Coagulation and Lime Softening with Ozone*

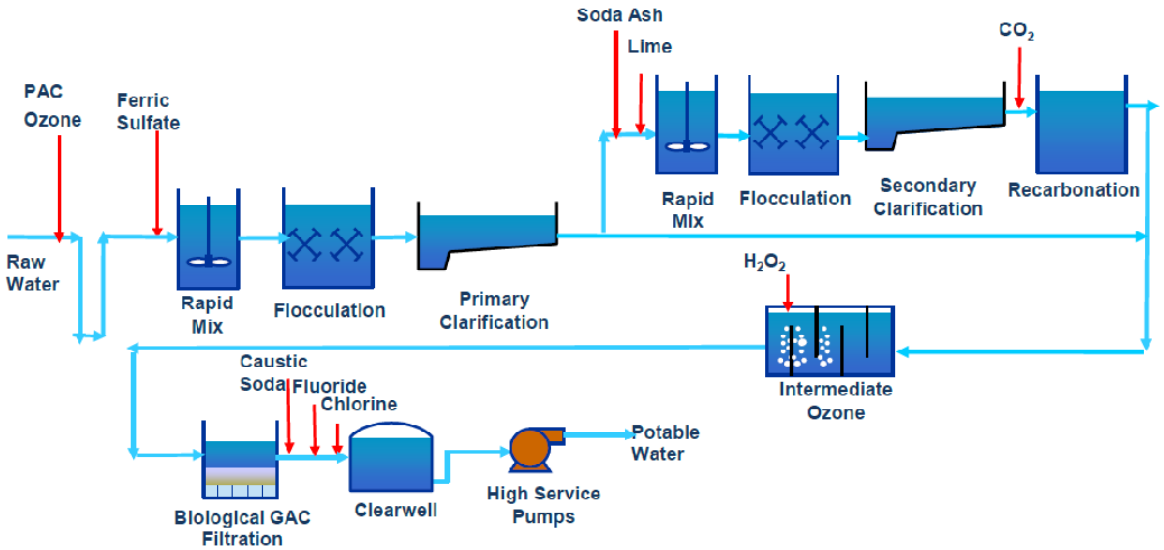
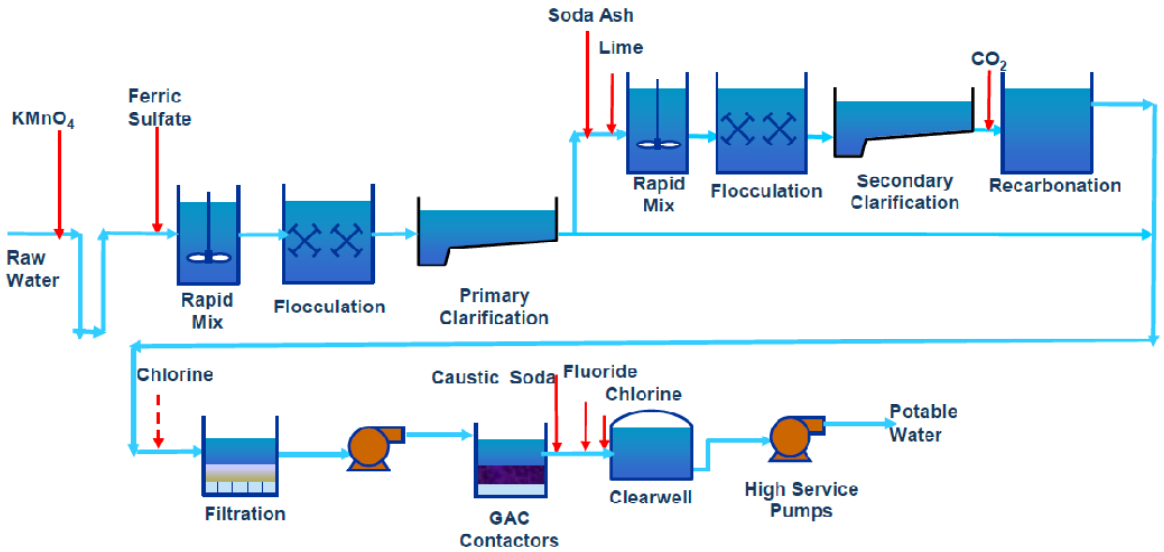


***Figure 6-10**. Alternative 3D – Enhanced Coagulation and Lime Softening with Post-Filter GAC Contactors*



6.1.3 Membrane Process Train Alternatives

CDM developed the following four membrane process train alternatives with the aim of meeting the WQ goals defined in **Section 2.0** and **Section 6.1** above:

- Alternative 1C – Lime Softening with MF/UF Filtration
- Alternative 2A – Enhanced Coagulation and Membrane Softening
- Alternative 2B – Enhanced Coagulation and Membrane Softening with MF/UF
- Alternative 3C – Enhanced Coagulation and Lime Softening with MF/UF Filtration

Membrane process train alternatives include processes such as clarification, lime-soda or membrane softening, stabilization, sand filtration or MF/UF and disinfection. Process flow diagrams for these alternatives are shown in **Figure 6-11** to **Figure 6-14.**

***Figure 6-11**. Alternative 1C – Lime Softening with MF/UF Filtration*

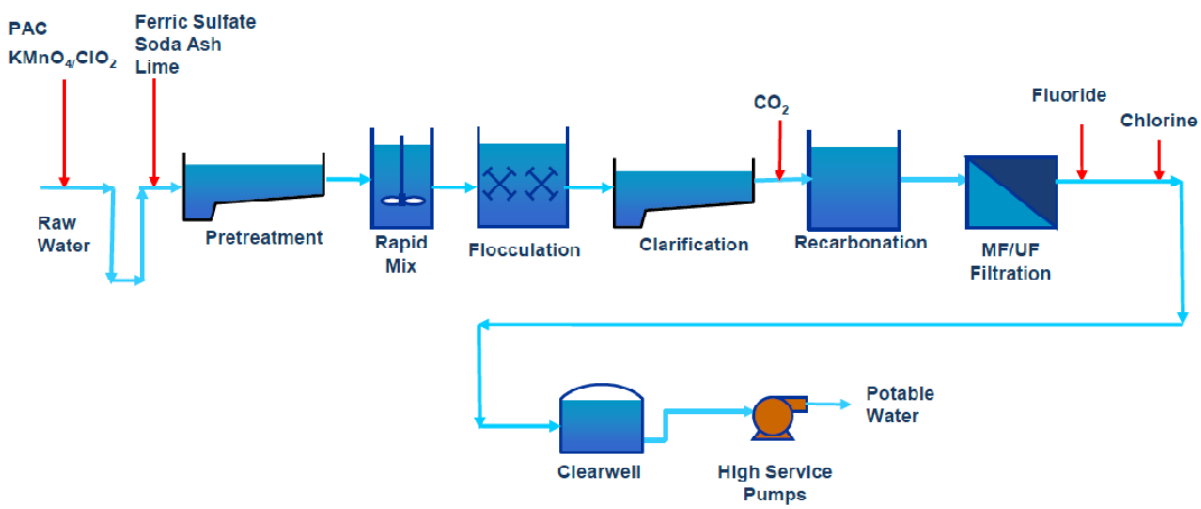


***Figure 6-12**. Alternative 2A – Enhanced Coagulation and Membrane Softening*

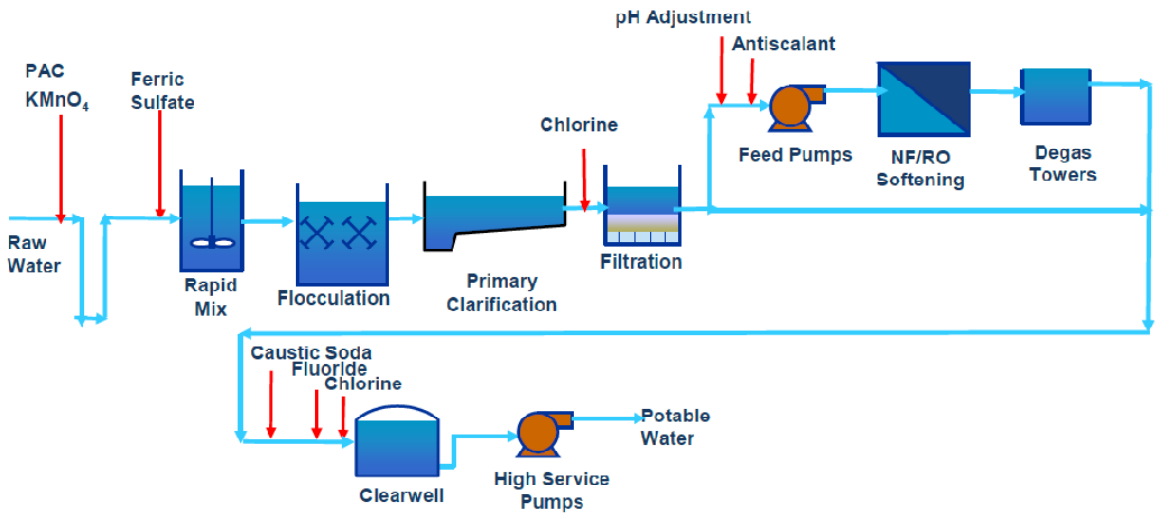


***Figure 6-13**. Alternative 2B – Enhanced Coagulation and Membrane Softening with MF/UF*

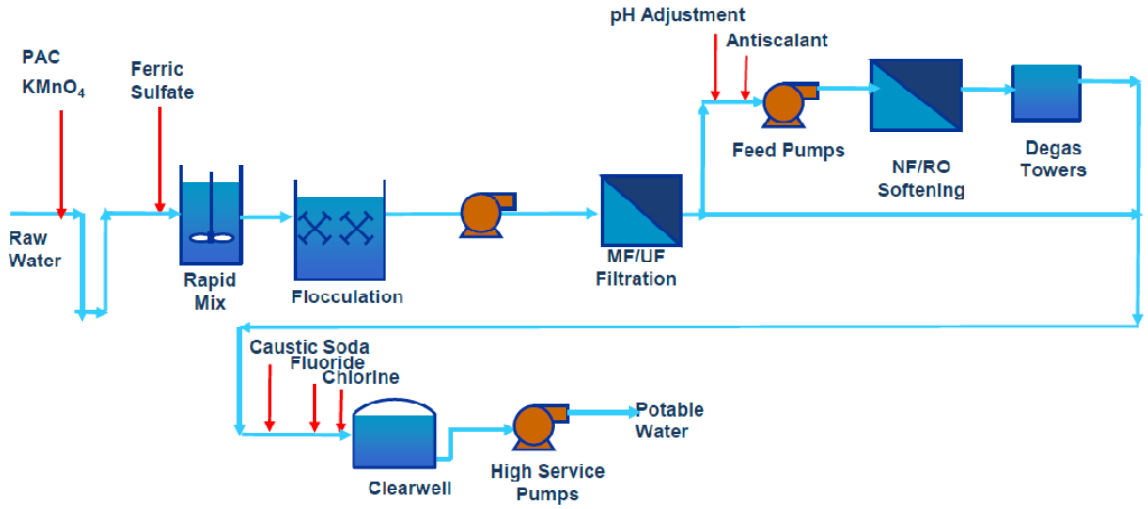
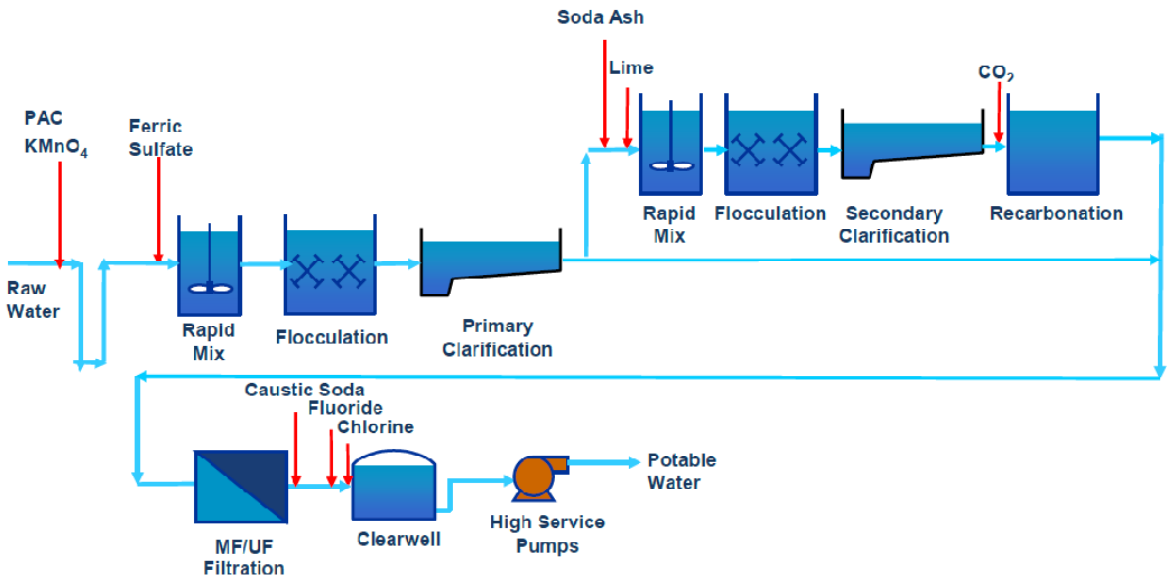


Figure 6-14. Alternative 3C – Enhanced Coagulation and Lime Softening with MF/UF Filtration



6.2 Alternatives Screening and Selection

The ten process train alternatives were screened at the Process Evaluation Workshop using the five screening criteria described below.

6.2.1 Ability to meet WQ Goals

The process train alternatives were evaluated by their ability to meet the following WQ goals as defined in Section 2.0 and Section 6.1.

- Turbidity < 0.1 NTU
- Disinfection – Chlorine residual of > 0.2 mg/L in the distribution system
- TOC/DBP – 80% of MCLs for DBPs
- Atrazine - <1.0 ppb
- Threshold Odor Number - 0
- Hardness – 110 mg/L as $CaCO_3$
- EDCs/PPCPs – Below detection

Each of the ten process train alternatives was assigned a score from 1 to 3 (3 being the best score) for each of the above WQ goal. The results of the scoring are presented in **Table 6-8**.

Table 6-8. Treatment Performance Comparison for Process Train Alternatives

WQ Goal	Alt 1A	Alt 1B	Alt 1C	Alt 1D	Alt 2A	Alt 2B	Alt 3A	Alt 3B	Alt 3C	Alt 3D
Turbidity	2	2	3	2	2	3	2	2	3	2
Disinfection	1	3	2	1	1	2	1	3	2	1
TOC/DBPs	2	3	2	3	2	2	3	3+	3	3+
Atrazine	1	2	2	3	2	2	1	2	2	3
T&O	1	3	1	3	2	2	1	3	1	3
Hardness	3	3	3	3	3	3	3	3	3	3
EDCs/PPCPs	0	2	0	2	1	1	0	2	0	2

3-Excellent, 2-Good, 1-Fair, 0-No treatment

6.2.2 Operation and Maintenance (O&M)

The following O&M sub-criteria were considered in the evaluation of process train alternatives at the Workshop:

- Ease of Operation
- Maintenance Requirements
- Automation and Control
- Residuals Handling and Disposal
- Staff and Training Requirements

6.2.3 Reliability and Risk

The following Reliability and Risk sub-criteria were considered in the evaluation of process train alternatives at the Workshop:

- Ability to Meet Future Regulations
- Process and Equipment Redundancy
- Proven Process History
- Staff and Public Safety Risks
- Operational Risks

6.2.4 Implementation

The following Implementation sub-criteria were considered in the evaluation of process train alternatives at the Workshop:

- Site Utilization
- Ability to Expand Plant in the Future

- Impacts on Surrounding Neighborhoods
- Construction Complexity
- Schedule Impacts

6.2.5 Relative Cost

This was based upon a comparison of unit processes across the alternatives and not actual construction cost estimates.

Weights were allocated to each of the screening criteria at the Workshop. Although sub-criteria were used as discussion points during the Workshop, weights were not allocated to each sub-criterion. Each process train alternative was also assigned a score from 1 to 5 (5 being the best score) for each of the five screening criteria. The results of the scoring and initial screening process are presented in Table 6-9.

Table 6-9. Process Train Alternatives Scoring Matrix

Treatment Alternatives	Screening Criteria (1=lowest/worst, 5=highest/best)					Weighted Score (1-5)
	Meet WQ Goals 30%	O&M Issues 20%	Reliability/Risk 15%	Implementation 10%	Relative Cost* 25%	
Alternative 1A	3	2	2	5	4	3.10
Alternative 1B	**4**	**2**	**3**	**4**	**3**	**3.20**
Alternative 1C	3	2	2	2	1	2.05
Alternative 1D	**4**	**3**	**3**	**4**	**3**	**3.40**
Alternative 2A	3	2	3	2	4	2.95
Alternative 2B	3	2	3	2	4	2.95
Alternative 3A	3	3	2	5	3	3.05
Alternative 3B	**4**	**2**	**4**	**3**	**3**	**3.25**
Alternative 3C	4	2	3	2	3	3.00
Alternative 3D	**4**	**3**	**4**	**5**	**3**	**3.65**

**Low-5, Med-3, High-1*

Based on the results of the initial screening process and the results presented in Table 6-9, the following four highest ranked process train alternatives were selected by the City for further evaluation and design development by CDM:

- Alternative 1B – Lime Softening with Ozone
- Alternative 1D – Lime Softening with Post-Filter GAC Contactors
- Alternative 3B – Enhanced Coagulation and Lime Softening with Ozone
- Alternative 3D – Enhanced Coagulation and Lime Softening with Post-Filter GAC Contactors

The next stage of design development for these four shortlisted alternatives included development of process design criteria and conceptual layouts of treatment processes, general site plans, preliminary hydraulic gradeline analyses and detailed construction and O&M cost estimates. These are presented in **Section 7.**

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Section 7
Process Alternatives Assessment

This Section describes the four process train alternatives that were short-listed at the Process Evaluation Workshop and presents the design criteria, bench scale testing results, and cost summary for all four alternatives. Two capacity alternatives were presented to the City at the Process Evaluation Workshop. Alternative 1 was to construct two 3 MGD treatment trains and Alternative 2 was to construct three 2.25 MGD treatment trains. The City selected Alternative 2 (three 2.25 MGD trains for a total initial plant capacity of 6.75 MGD) because of better operational flexibility and redundancy it offers over Alternative 1.

As a result, the unit processes and treatment facilities for these alternatives have been sized and developed for a 6.75 MGD facility that is expandable to 9 MGD in the future. The new facility will be located on SR 66 on two parcels that have been recently purchased by the City. A map of the parcels is shown in **Figure 7-1**.

The design criteria for each of these alternatives' unit processes have been developed in accordance with the requirements of TSS and Ohio EPA's Approved Capacity document, and configured to meet the City's finished WQ goals.

For purposes of this PER, it is assumed that the GMR, Swift Run Lake (Reservoir) and the Gravel Quarry, which currently provide surface water supply for the City's existing WTP, will continue to provide raw water to the new plant. To augment the surface water sources with groundwater, the City also decided to investigate the potential for groundwater in the area of the new plant as part of this study. As a result, CDM and its subconsultant Terran Corporation were tasked with estimating the production potential from the recently identified buried valley aquifer (BVA) in the vicinity of the proposed plant. This is an ongoing study and the quantity of future potential capacity of ground water to augment the surface water supply cannot yet be defined.

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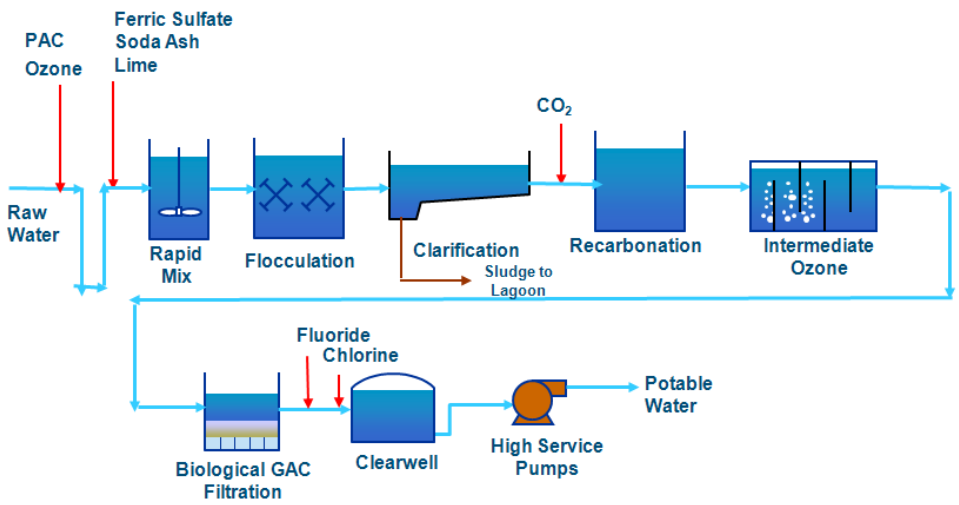
Figure 7-1
New Water Treatment Plant Location
City of Piqua, OH

7.1 Lime Softening With Ozone (Alternative 1B)

The process flow diagram for Alternative 1B is shown in **Figure 7-2.**

Figure 7-2. *Alternative 1B – Lime Softening with Ozone*



7.1.1 Alternative Description

This alternative includes the following unit processes and treatment facilities:

- Raw Water Pump Station
- Pretreatment Facility (Optional)
- Chemical Facility
- Single-Stage Conventional Flash Mixing, Flocculation/Clarification Processes
- Recarbonation
- Intermediate Ozonation
- Biological GAC Filtration
- Finished Water Storage
- High Service Pump Station
- Solids Handling Facility (Optional)

Raw water from the Reservoir will flow by gravity via a control structure to a new pump station that will be located near the existing River Pump Station as shown in **Figure 7-3**. The new pump station will also draw water from the GMR. Water from both of these sources will pass through separate screens to remove debris, and flow into a single wet well at the new pump station where it will then be pumped via a single 24-inch raw water main to the new plant location. Water from the Gravel Quarry source will be pumped from the quarry through two 12-inch lines (one existing 12-inch and a new 12-inch line) that will combine to a 24-inch line near the River Pump Station and run north along SR 66 to the new plant. A preliminary review of hydraulics indicates that the existing quarry pump may not have sufficient pumping head to deliver water to the new site which is located at a higher elevation and is approximately 3,000 ft from the existing plant. Thus, new pumps will be required at the Gravel Quarry Pump Station, consisting of two 2 MGD pumps (one duty pump and one standby pump). A cross-tie will be provided between the two raw water supply lines to the new plant in order to provide the City with operational flexibility and redundancy. Since the quarry water represents the best WQ source, CDM recommends constructing a separate line from the quarry to the new plant. The new raw water pump station will include three 3.75 MGD pumps, with one operating as a standby pump to supply 7.5 MGD to the plant. Although future maximum day production is only 6.75 MGD, raw water flow was assumed to be 7.5 MGD in order to account for basin losses, chemical feed etc. The pumps will be operated with variable speed drives to provide capability to satisfy a wide range of flow conditions.

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Figure 7-3
Proposed Raw Water Pump Station Location
City of Piqua, OH

Raw water from the various sources will ultimately be pumped to the pre-treatment facility or the rapid mix basins located at the new plant site. The pre-treatment facility is optional and can be built at a later stage if the City does not wish to construct it initially. Bypass piping will also be provided around the pre-treatment units to route the raw water directly to the rapid mix basins and conventional treatment. Two plate settlers are considered for pre-treatment to remove approximately 50-60% of the total raw water turbidity. **Table 7-1** presents the design criteria for the plate settlers.

Table 7-1. Pretreatment Plate Settlers Design Criteria

Design Criteria	Value
No. of Units	2 (1 Duty, 1 Standby)
Capacity, each	4.8 MGD
Suspended Solids Removal %	50-60%

Following pretreatment, water will flow through single-stage conventional treatment processes including rapid mix, flocculation, and sedimentation. The rapid mix basins will be located inside the chemical building for easier access for the chemical feed lines. PAC, ferric sulfate, soda ash, and lime will be added to the rapid mix unit from their storage and feed systems housed in the chemical building. The rapid mixers are "flash mixer" type mixers for near-instantaneous dissolution of chemicals. **Table 7-2** presents the design criteria for the rapid mix basins.

Table 7-2. Rapid Mix Basin Design Criteria

Design Criteria	Value
No. of Units	2 (1 Duty, 1 Standby)
Capacity, each	7.5 MGD
Dimensions, each	5 ft L x 5 ft W x 10 ft SWD
No. of Mixers per Unit	1
Mixer Type	Vertical Shaft
Motor, each	15 HP

The water from the rapid mix basins flows through a channel to three parallel treatment trains comprising of three flocculation basins and three sedimentation basins. The flocculation basins will be equipped with horizontal paddle wheel flocculators. The purpose of flocculation is to form floc to aggregate particles and promote better settling. **Table 7-3** presents the flocculation basin design criteria.

Table 7-3. Flocculation Basin Design Criteria

Design Criteria	Value
No. of Parallel Basins	3
Capacity, each	2.5 MGD
Dimensions, each	32 ft L x 16 ft W x 16 ft SWD
No. of Flocculators per Basin	2
Flocculator Type	Horizontal Paddle Wheel
Detention Time per Basin	35 min
Motor, each	5 HP

Flocculated water then flows through an effluent channel, a cross-over channel, and enters an influent channel ahead of the three sedimentation basins. Gates are provided for flow split between the basins. The sedimentation process removes the flocculated solids by gravity, which settles as lime sludge in the bottom of the basin. The sedimentation basins are equipped with chain and flight collector mechanisms similar to those at the City's existing plant, which rakes settled lime sludge into a sludge hopper channel located at the inlet side of the basins. The accumulated lime sludge is then removed with sludge pumps, which pump the sludge to the existing Sludge and Filter Backwash Disposal facility that is located near the existing treatment plant. **Table 7-4** provides the design criteria for the sedimentation basins.

Table 7-4. Sedimentation Basin Design Criteria

Design Criteria	Value
No. of Parallel Basins	3
Capacity, each	2.5 MGD
Dimensions, each	110 ft L x 35 ft W x 16 ft SWD
Surface Area, each basin	3,830 sq. ft
Detention Time per Basin	4.4 hours
Flow Through Velocity	0.42 fpm
No. of Sludge Collectors	3
Type	Chain and Flight
Motor, each	2 HP

In single-stage treatment, clarification and softening occur in the same sedimentation basin. Clarified lime softened water typically has a high pH and is unstable. As a result, the pH needs to be lowered with an acid or an acid-forming compound to stabilize the water. This pH adjustment takes place in recarbonation basins where carbon dioxide is injected in mixing chambers ahead of the basins and immediately followed by the recarbonation settling basin to precipitate out calcium and magnesium and effectively lower the pH.

Table 7-5 provides the design criteria for the recarbonation basins.

Table 7-5. Recarbonation Basin Design Criteria

Design Criteria	Value
No. of Parallel Basins	2
Capacity, each	7.2 MGD
Dimensions, each	44 ft L x 13 ft W x 12 ft SWD
Detention Time per Basin	20 min
No. of Mixing Chambers	2
Capacity, each	7.2 MGD
Detention Time per Mixing Chamber	3 min
Dimensions, each	13 ft L x 13 ft W x 12 ft SWD
Type of Mixer	Vertical Shaft
Motor, each	1.5 HP

Ozonation facilities are included in this alternative for purposes of taste and odor control and DBP control. The ozonation process by itself does not eliminate organic compounds, but does oxidize organics into biodegradable organics which are ultimately removed by downstream biologically active adsorption process, such as GAC. The ozonation facilities for this alternative include liquid oxygen (LOX) storage tanks and vaporizers for converting the LOX into oxygen gas. Oxygen is used as the feed gas to the ozone generators. The LOX storage tanks and vaporizers are located outside in storage tanks on a concrete pad in close proximity to the ozone building. Two medium frequency ozone generators are provided inside the building that can operate within a range of 6% to 12% ozone in oxygen concentration. **Table 7-6** presents the design criteria for the ozone generation system.

Table 7-6. Ozone Generation System Design Criteria

Design Criteria	Value
Plant Capacity	7.2 MGD
No. of Generators	2 (1 Duty, 1 Standby)
Generator Capacity, each	26-263 ppd
Design Ozone Dose	4 mg/L
Ozone Concentration	12% wt
Outlet Ozone Gas Pressure	16 psig
Cooling Water (plant finished water) Flow Rate	32 gpm

The ozone dissolution and contacting system consists of submerged fine bubble diffusers located inside two baffled contact basins. Two ozone destruct units are also provided inside the ozone building. The building itself is located on top of the two contact basins for a consolidated footprint and more direct introduction of the ozone. **Table 7-7** presents the design criteria for these facilities.

Table 7-7. Ozone Contactor Design Criteria

Design Criteria	Value
No. of Contactors	2
Capacity, each	7.2 MGD
Dimensions, each	20 ft L x 15 ft W x 23 ft SWD
Detention Time per Basin	10 min
Dissolution System Type	Submerged Fine Bubble
No. of Off-Gas Destruct Units	2

Following ozonation, the water flows to four gravity-fed dual-media GAC/sand filters with an underdrain system. These filters will primarily remove suspended solids following conventional treatment. However, these filters will also act as "biological" filters for removal of assimilable organic carbon (AOC) that is produced as a byproduct of the ozonation process. If AOC is not removed, it can result in microbial re-growth in the distribution system and impact WQ. The GAC filters also provide for taste and odor control and TOC reduction including atrazine and other compounds. A combination of ozone and biological GAC filtration provides a robust treatment of the City's main concerns of DBPs, atrazine, and taste and odor. The GAC bed life for TOC and taste and odor control could range anywhere from a few months to a few years. Operating the GAC biologically will increase bed life by taking advantage of biological treatment. The GAC filters would be backwashed using backwash water and auxiliary air to clean the GAC media. Table 7-8 lists the design criteria for the biological GAC filters for this alternative.

Table 7-8. Biological GAC Filtration Design Criteria

Design Criteria	Value
No. of Filters	4
Type	Gravity Filters
Firm Filtration Capacity	7.2 MGD
Filtration Rate, each	4 gpm/sf
Filter Box Dimensions, each	22 ft L x 19 ft W x 20 ft H
Filter Area, each	418 sq. ft
Media Type	GAC/sand
Media Depth	60 in GAC; 12 in Sand
No. of Backwash Pumps	2 (1 Duty, 1 Standby)
Backwash Rate	10-15 gpm/sf
Bed Expansion	50%
Air Flow Rate	5 scfm/sf

After filtration, chlorine, fluoride and a corrosion inhibitor are added to the filtered water line. Filtered water flows by gravity to two onsite clearwells that are provided for finished water storage and chlorine contact time. CDM recommends two clearwells for redundancy purposes. The clearwells will be rectangular in shape and cast-in-place with common wall construction. Baffles will provide enhanced CT for disinfection. Due to the site topography and based on preliminary plant hydraulics, these clearwells will be

located mostly below grade with access hatches and vents to the surface. Table 7-9 presents the design criteria for these clearwells.

Table 7-9. Clearwell Design Criteria

Design Criteria	Value
No. of Clearwells	2
Type	Cast-in-place Concrete
Volume, each	1.5 MG
Total Volume as % of Plant Capacity	22.2%
Dimensions, each	100 ft L x 100 ft W x 20 ft SWD
Baffles	Concrete Walls/Supports

Four high service pumps are provided in the filter/administration/pump building of the new facility to pump finished water from the plant clearwells to the City's distribution system. One of the four pumps will be standby. The high service pumps have been sized to discharge into the Central Low Service District. As with the existing plant configuration, the existing Ziegler and Hetzler booster pumping stations will continue to transfer the finished water to the East and West High Service Districts. The finished water mains from the new plant will tie into the existing 20-inch and 24-inch transmission mains along SR 66. Table 7-10 presents the design criteria for the high service pump station.

Table 7-10. High Service Pump Station Design Criteria

Design Criteria	Value
No. of Pumps	4 (3 Duty, 1 Standby)
Capacity, each	3 MGD
Total Pumping Capacity	12 MGD
Firm Pumping Capacity	9 MGD
Pump Type	Horizontal Split Case

In addition to the above processes, chemical storage and feed facilities will be provided in a stand-alone building for ferric sulfate, lime, sodium hypochlorite, potassium permanganate, fluoride, sodium hydroxide, soda ash and hexametaphosphate. Since PAC is explosive in nature, the automatic bulk bag feeder for PAC will be located outside the chemical building in a separate self contained pre-fabricated assembly to isolate this from other chemicals and equipment. Bulk chemical storage tanks have been sized to provide 30 days of storage at average flow conditions in accordance with TSS requirements. Chemical feed pumps have been sized for maximum flow conditions, including one standby feed pump for each chemical. Table 7-11 presents the chemical doses for this alternative, which have been projected using current treatment plant's historic chemical usage data (except ozone and hydrogen peroxide).

Table 7-11. Alternative 1B-Chemical Doses

Chemical Name	Average Dose (mg/L)
Ferric Sulfate	8.5
Soda Ash	0.5
Lime	186
Carbon Dioxide	21
Sodium Hypochlorite	3.5
Hydrofluosilicic Acid	0.75
Sodium Hexametaphosphate	1.2
Ozone	3
Powdered Activated Carbon	1.4
Hydrogen Peroxide	0.5

The City has several options for handling spent filter backwash water and sludge from the sedimentation basins at the new plant. A condition assessment of selective existing facilities performed by CDM as part of this study (Section 5.0) recommends that the City continue to use the existing Sludge and Filter Backwash Disposal facility located near the existing treatment plant. The baseline scenario for all alternatives described in this section assumes that the City will continue to use the remote Sludge and Filter Backwash Disposal facility to process and ultimately dispose the sludge to the existing off-site lagoon. New piping will be provided to convey the sedimentation basin lime sludge from the new plant to the existing facility.

Another residuals management option is the construction of a new on-site solids handling facility to house mechanical dewatering equipment to dewater the sludge from the new sedimentation basins. This new facility would improve the dewatering efficiency and avoid pumping the sludge and supernatant each way. Table 7-12 presents the design criteria for the optional solids handling facility. Note that the belt filter press sizing is based on single shift operation, 5 days a week; redundancy would be provided by using a single unit for longer operating periods in event of a unit out of service.

Table 7-12. Solids Handling Facility Design Criteria

Design Criteria	Value
No. of Basins	2
Volume, each	147,000 gal
Volume, days	3
Dimensions, each	31 ft L x 31 ft W x 20 ft SWD
No. of Sludge Pumps	3 (2 Duty, 1 Standby)
Pump Type	Progressive Cavity
Dewatering Equipment Type	2-m Belt Filter Press
No. of Units	2 (2 Duty, 0 Standby)
Capacity, each	2,000 Lbs/hr dry sludge
Feed Sludge Solids	3%

The baseline scenario also assumes that two new backwash holding basins and recirculation pumps will be provided at the new plant sized to equalize spent filter backwash water. The recirculation pumps will recycle the water back to the head of the plant at a rate that is less than 10% of average plant capacity in accordance with the Filter Backwash Recycling Rule (FBRR) requirements. **Table 7-13** presents the design criteria for the backwash holding tanks.

Table 7-13. Backwash Holding Tank Design Criteria

Design Criteria	Value
No. of Basins	2
Volume, each	170,000 gal*
Dimensions, each	44 ft L x 32 ft W x 16 ft SWD
No: of Recycle Pumps	3 (2 Duty, 1 Standby)
Pump Type	Centrifugal End Suction

**Assumes 2 filter backwashes per day*

All off-site improvements for this alternative are shown in **Figure 7-4**. The preliminary hydraulic profile and proposed site layout with major yard piping for this alternative are shown in **Figure 7-5** and **Figure 7-6** respectively. The proposed site layout of the facility was developed with consideration of geotechnical conditions, site topography, and future expansions. An overall process and instrumentation diagram for this alternative is shown in **Figure 7-7**. This PER assumes that all treatment processes for this alternative will be monitored and controlled by plant supervisory control and data acquisition (SCADA). A detailed control scheme and specific monitoring and control points will be assessed further during detailed design.

A new primary electric service will be required for the new WTP. The City's Power System will install new utility poles and extend overhead wires to deliver high voltage electricity to the facility. The power requirements are estimated to require a 2,000 kilovolt-amperes (kVA) service and would be metered on the primary side. An on-site pad-mount transformer would enable 480-volt (V), 3-phase service for major electrical equipment such as pump motors. Additional transformers will provide for 120-V, 1-phase low voltage electric service for building needs such as receptacles, lighting, and other nominal loads.

Standby power would consist of a 1,500 kilowatt (kW) diesel engine-driven generator to provide standby power to operate necessary equipment to meet average day demand conditions.

Additional on-site power generation options are available to the City, and may assist the City in acquiring grant or loan money to fund the design and construction of the new WTP. It is not expected that all power could be provided by an alternative source given the relatively large electric usage; however, there may be sufficient interest from public perception or by minimizing future electric costs, such that these may be financially and socially attractive.

- Solar: Without grants or other external funding, solar (photovoltaic electric) has a typical installed cost of approximately $10,000-15,000 per kW of panel rating. A 100 kW system would carry a capital cost of $1M to $1.5M. Based on actual solar exposures, solar power equates to a payback on the order of greater than 25 years.

- Wind: The best opportunities for wind power are with smaller building integrated/mounted wind turbines. Average wind velocities in the Piqua area are generally not viable for strong enough wind forces, such as in a low-lying valley, to justify large installations. The equipment ratings are generally in the 5 to 25 kW range, and paybacks are typically on the order of 10 years, where viable.
- Geothermal: These systems consist of either an open or closed loop system that transfers heat either to/from the earth subsurface and the equipment location. Generally these systems are configured to operate HVAC heating/cooling systems to partially or completely eliminate supplemental electricity needs. Geothermal systems can have an acceptable payback just below 10 years if the installation conditions are optimal.

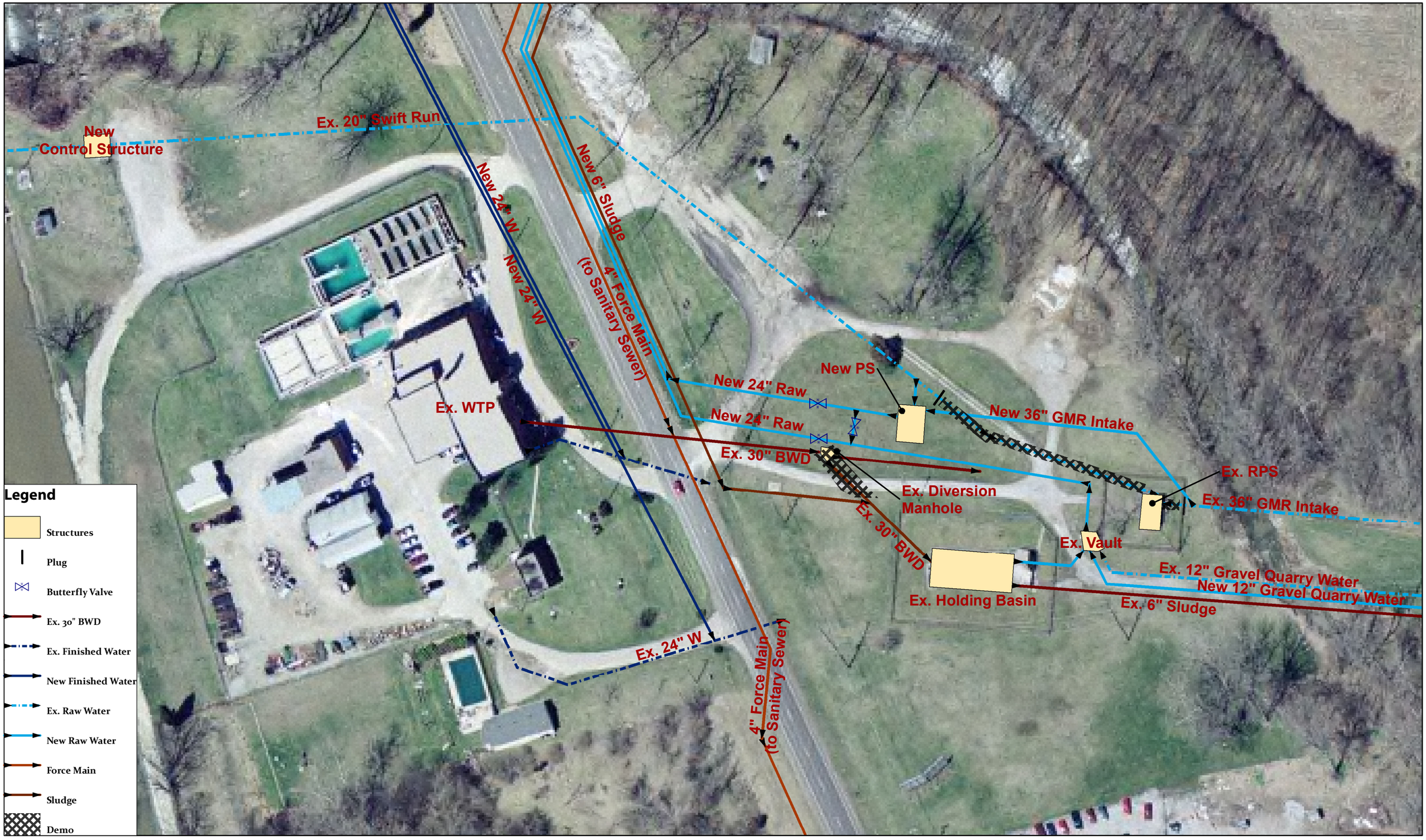

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Figure 7-4
Offsite Improvements
City of Piqua, OH

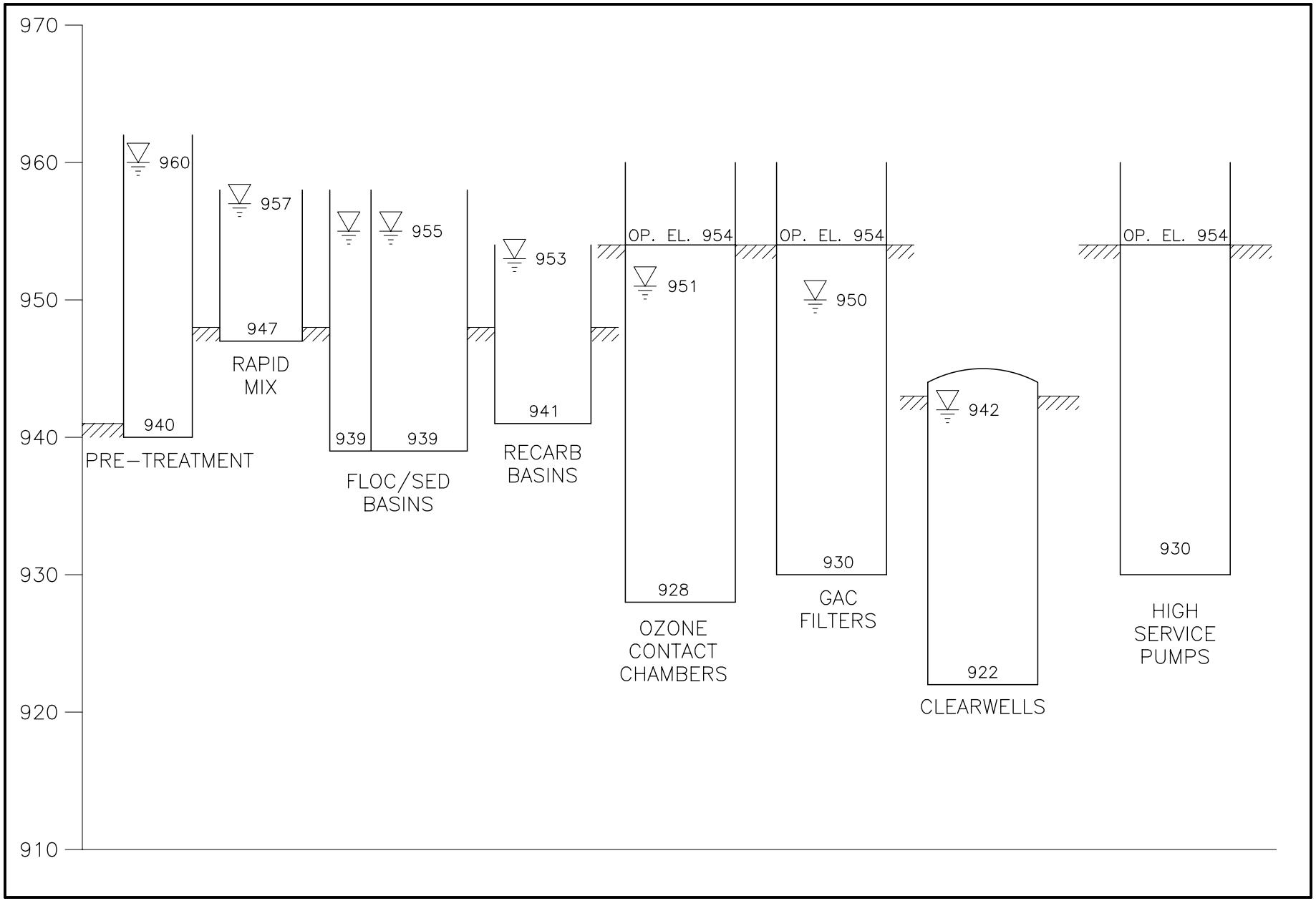

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Figure 7-5
Alt. 1B - Lime Softening with Ozone
Hydraulic Profile

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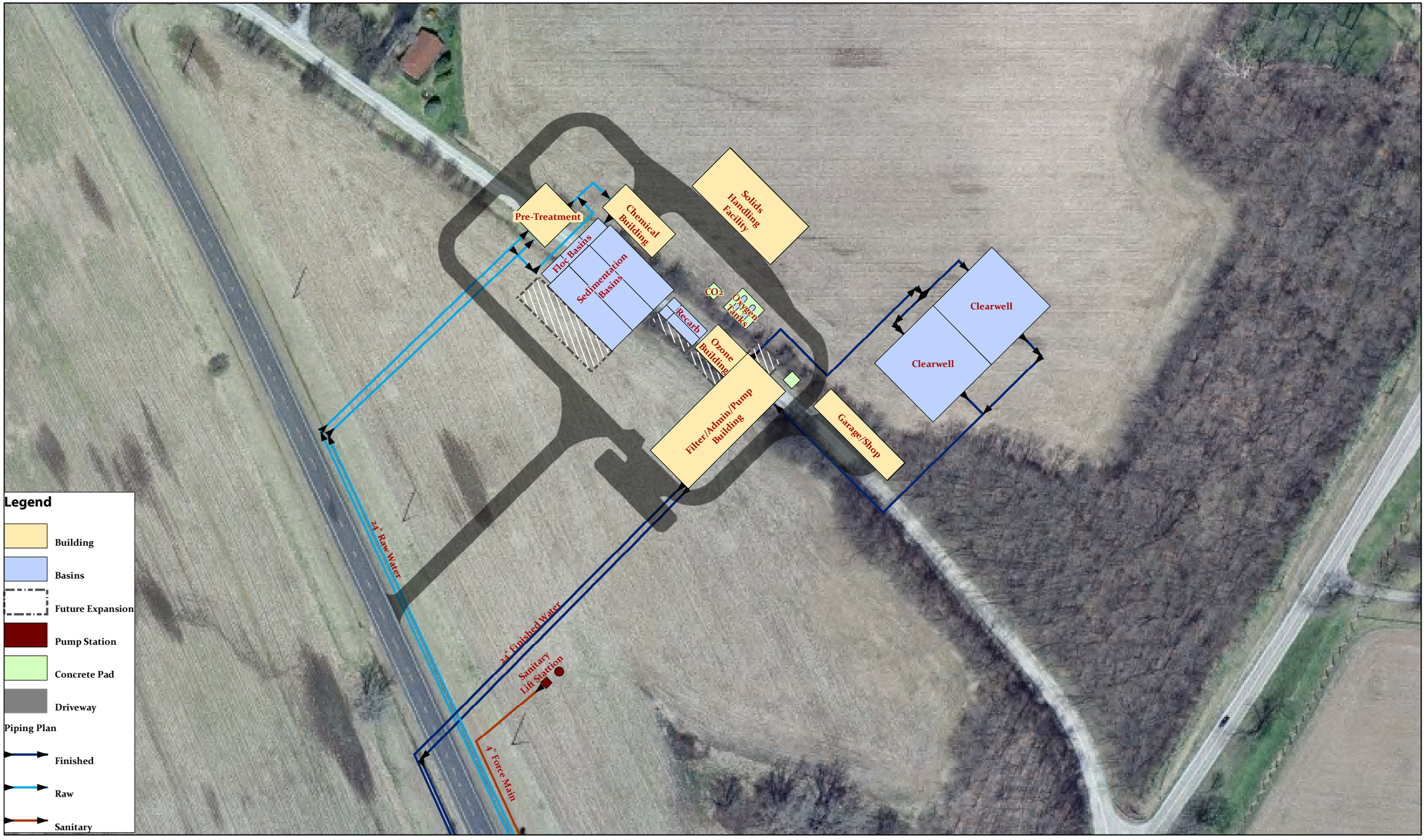

Figure 7-6:
Alt. 1B - Lime Softening with Ozone
City of Piqua, OH

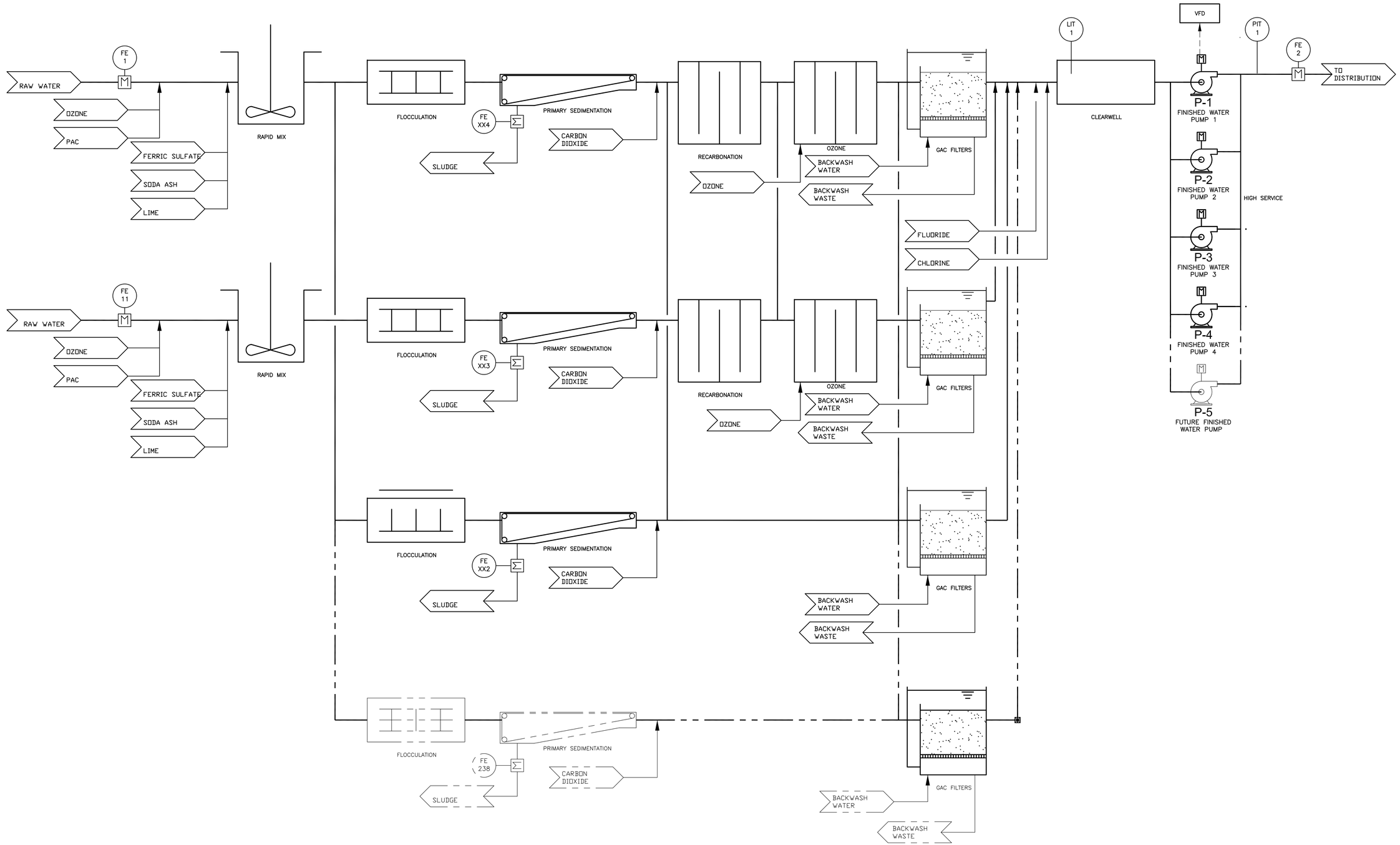

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Figure No. 7-7
Alt 1B - Process and Instrumentation Diagram
City of Piqua, OH

7.1.2 Bench Scale Testing Results

Any planning or preliminary level assessment is incomplete without bench scale studies which simulate treatment processes and provide results in a short span of time. CDM recommended performing bench-scale preoxidation tests and coagulation/jar tests to the City for this alternative. Bench scale test data for this study is included in **Appendix C** and the results are summarized in this section.

Preoxidation demand/decay testing studies were conducted on Swift Run Lake water (worst in terms of WQ) at ambient pH followed by simulated distribution system (SDS) experiments to measure 3-day total trihalomethane formation potential (TTHMFP) in order to establish the pre-oxidant dose for ozone at 0.1 mg/L residual after 5 min contact time. The decay results are shown in **Table 7-14**. Ozone doses of 3 mg/L, 4 mg/L and5 mg/L showed strong potential to oxidize raw water TOC. At these doses, reaction times ranged from 2 to 7 minutes to exhaust the ozone. Based on the results a 3-4 mg/L ozone dose is recommended for oxidation purposes.

The SDS experiments were conducted using an ozone dose of 5 mg/L, chlorine dose of 4.5 mg/L and a pH of 9.8. The measured TTHM concentration was approximately 133 µg/l. The baseline scenario with no pre-oxidant resulted in a TTHM concentration of 92.9 µg/l. This shows that the addition of ozone as a pre-oxidant alone resulted in an increase in the TTHM concentration. However the benefits of biofiltration were not tested on the bench-scale.

Table 7-14. Ozone Demand/Decay Data

Time (min)	Ozone Dose (mg/L)		
0	5	4	3
1	1.27	0.82	0.37
2	0.79	0.34	0.09
3	0.51	0.09	
5	0.17		
7	0.02		

In addition, ferric sulfate coagulation of blended plant composite raw water samples were also performed at lime softening pH of 10.8 to determine the optimal dose using jar tests for turbidity/TOC reduction with pH adjustment. This test was conducted to simulate lime/soda ash softening in the same basin such as the single-stage treatment described above for this alternative. The results are shown in Table 7-15. Based on the results the optimal coagulant dose is 40 mg/L. The SDS experiments were conducted using a Ferric Sulfate dose of 50 mg/L, chlorine dose of 3.1 mg/L and a final pH of 9.8. The measured TTHM concentration was approximately 91 µg/l. Stage1 DBP Rule requires a 25% TOC removal and the single stage treatment process described for this alternative was able to achieve a 46% removal through bench scale studies. This proves that single stage treatment is an effective and feasible alternative for the City's new plant.

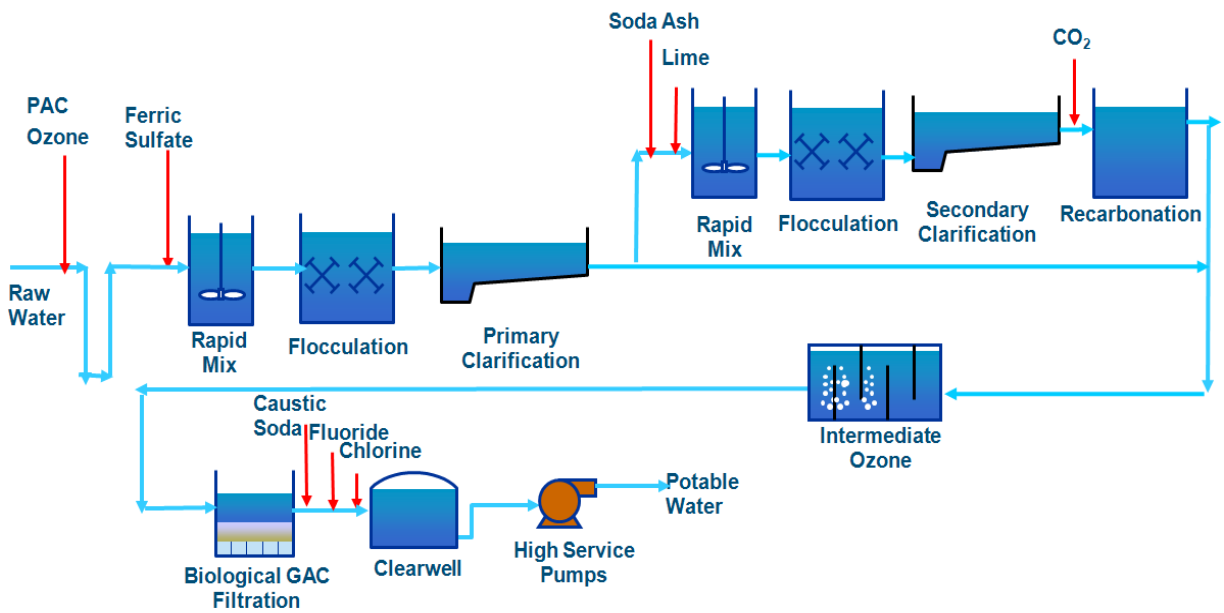
Table 7-15. Jar Test Results for Single-Stage Treatment

Ferric Sulfate Dose (mg/L)	pH	TOC (mg/L)	% Removal
40	10.77	2.92	46
50	10.81	2.79	48

7.2 Enhanced Coagulation and Lime Softening With Ozone (Alternative 3B)

The process flow diagram for Alternative 3B is shown in **Figure 7-8**.

Figure 7-8. *Alternative 3B – Enhanced Coagulation and Lime Softening with Ozone*



7.2.1 Alternative Description

This alternative includes the following unit processes and treatment facilities:

- Raw Water Pump Station
- Chemical Facility
- Two-Stage Conventional Flash Mixing, Flocculation/Clarification Processes
- Recarbonation
- Intermediate Ozonation
- Biological GAC Filtration
- Finished Water Storage
- High Service Pump Station
- Solids Handling Facility (Optional)

This alternative is similar to Alternative 1B described previously. The main difference between Alternative 1B and 3B is single versus two-stage treatment for the conventional flash mixing, flocculation and clarification processes. Additionally, Alternative 1B includes a pretreatment facility whereas Alternative 3B

does not because the additional detention time obtained through the second stage basins would be sufficient to reduce turbidity and TOC and hence a separate pretreatment facility is not necessary for this alternative. The single-stage basins would essentially serve as the "pre-treatment" basins.

The raw water supply and pumping scenarios for this alternative are identical to Alternative 1B. Raw water is directly sent to the two rapid mix chambers for first stage treatment. The rapid mix chambers will be located inside the chemical building and will be used for addition of various treatment chemicals such as PAC and ferric sulfate for first stage treatment.

Following rapid mix, the water flows through a channel to three parallel treatment trains comprising of three first stage flocculation basins and three first stage sedimentation basins. The flocculation basins will be equipped with horizontal paddle wheel flocculators similar to Alternative 1B. Flocculated water from the flocculation basins flows to the sedimentation basins which are equipped with chain and flight sludge collectors. **Table 7-16** provides the design criteria for first stage rapid mixing, flocculation and sedimentation.

Table 7-16. First Stage Conventional Treatment Design Criteria

Design Criteria	Value
Rapid Mix	
No. of Units	2 (1 Duty, 1 Standby for first-stage)
Capacity, each	7.5 MGD
Dimensions, each	5 ft L x 5 ft W x 10 ft SWD
No. of Mixers per Unit	1
Mixer Type	Vertical Shaft
Motor, each	15 HP
Flocculation Basins	
No. of Parallel Basins	3
Capacity, each	2.5 MGD
Dimensions, each	32 ft L x 16 ft W x 16 ft SWD
No. of Flocculators per Basin	2
Flocculator Type	Horizontal Paddle Wheel
Detention Time per Basin	35 min
Motor, each	5 HP
First-Stage Sedimentation Basins	
No. of Parallel Basins	3
Capacity, each	2.5 MGD
Dimensions, each	80 ft L x 27 ft W x 16 ft SWD
Surface Area, each basin	2150 sq. ft
Detention Time per Basin	2.5 hours
Flow Through Velocity	0.53 fpm
No. of Sludge Collectors	3

Type	Chain and Flight
Motor, each	2 HP

Following clarification in the first stage basins, flow is routed through an intermediate channel to the rapid mix chambers for second stage flocculation and sedimentation basins. Here softening chemicals including lime and soda ash (if necessary) are mixed with the water and routed to the influent channel of the second stage basins. These basins act as softening basins to precipitate out carbonate hardness with the lime and non-carbonate harness with the soda ash, such that the desired total hardness is attained. **Table 7-17** presents the design criteria for second stage rapid mixing, flocculation and softening. Note that for two-stage treatment, detention time is cumulative. Based on the criteria presented in **Table 7-16** and **Table 7-17**, the total detention time for two-stage treatment is five hours.

Table 7-17. Second Stage Conventional Treatment Design Criteria

Design Criteria	Value
Rapid Mix	
No. of Units	2 (1 Duty, 1 Standby for second-stage)
Capacity, each	7.35 MGD
Dimensions, each	5 ft L x 5 ft W x 10 ft SWD
No. of Mixers per Unit	1
Mixer Type	Vertical Shaft
Motor, each	15 HP
Flocculation Basins	
No. of Parallel Basins	3
Capacity, each	2.45 MGD
Dimensions, each	32 ft L x 16 ft W x 16 ft SWD
No. of Flocculators per Basin	2
Flocculator Type	Horizontal Paddle Wheel
Detention Time per Basin	35 min
Motor, each	5 HP
Second-Stage Softening Basins	
No. of Parallel Basins	3
Capacity, each	2.45 MGD
Dimensions, each	80 ft L x 27 ft W x 16 ft SWD
Surface Area, each basin	2,150 sq. ft
Detention Time per Basin	2.5 hours
Flow Through Velocity	0.53 fpm
No. of Sludge Collectors	3
Type	Chain and Flight
Motor, each	2 HP

The design criteria for all downstream processes such as recarbonation, ozonation, biological GAC filtration, finished water storage and high service pumping as well as the chemical doses for this alternative are identical to Alternative 1B described above. The design criteria for the onsite backwash holding tanks and the optional solids handling facility are also similar to Alternative 1B described above, with the exception of multiple sludge draw-off locations, one from each stage's sedimentation basins.

Offsite improvements for this alternative are also identical to Alternative 1B. The preliminary hydraulic profile and proposed site layout with major yard piping for this alternative are shown in **Figure 7-9** and **Figure 7-10** respectively. The proposed site layout of the facility was developed with consideration of geotechnical conditions, site topography, and future expansions. An overall process and instrumentation diagram for this alternative is shown in **Figure 7-11**. Similar to Alternative 1B, it is assumed that all processes for Alternative 3B will also be monitored and controlled by plant SCADA. A detailed control scheme and specific monitoring and control points will be assessed further during detailed design.

Electrical requirements for power needs and standby power are similar to those in Alternative 1B. The additional tanks and drive units have nominally more power requirements, but not significant enough to increase the primary electric or standby power needs.

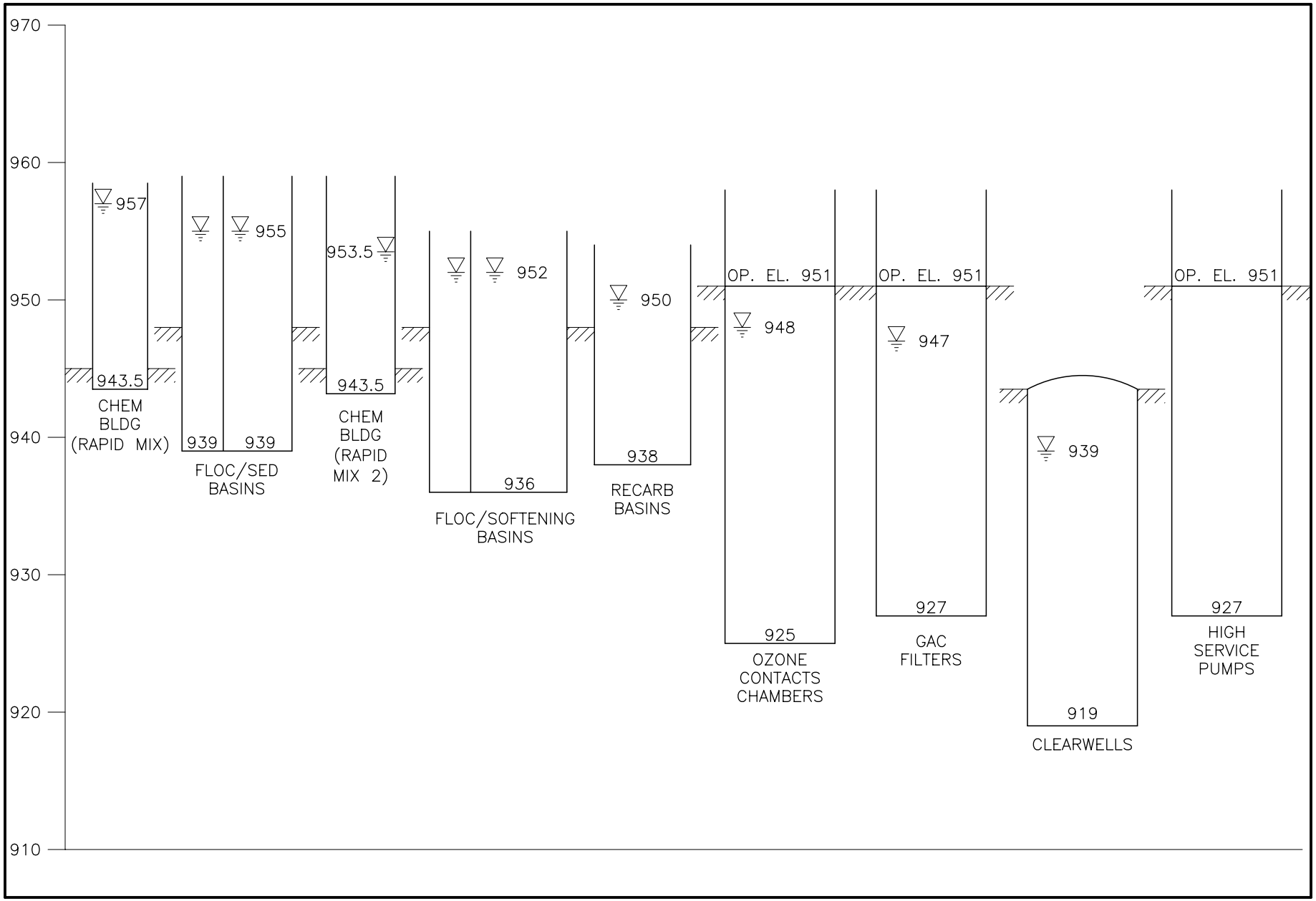


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Figure 7-9
Alt. 3B - Enhanced Coagulation and Lime Softening with Ozone
Hydraulic Profile

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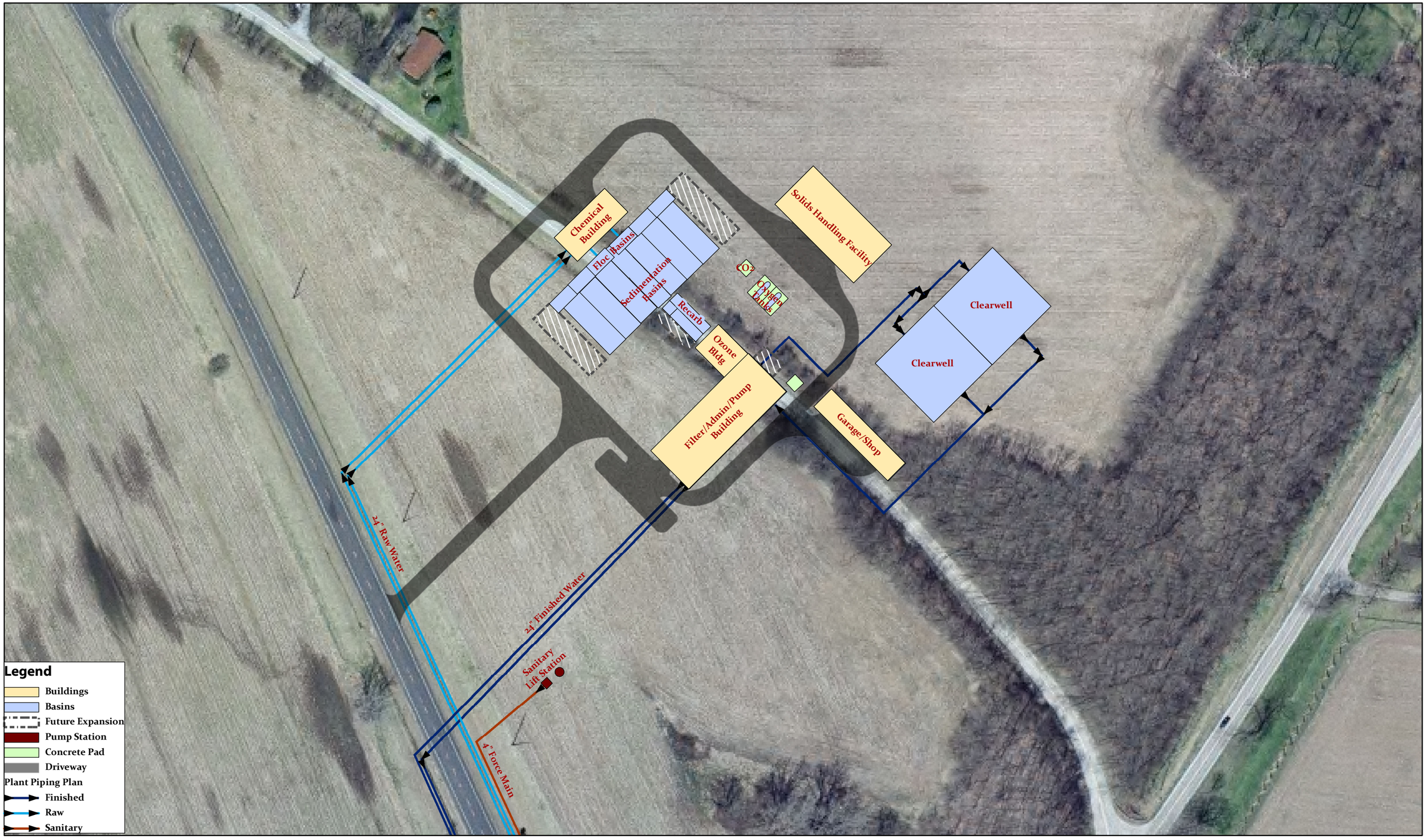

Figure 7-10
Alt. 3B - Enhanced Coagulation and Lime Softening with Ozone
City of Piqua, OH

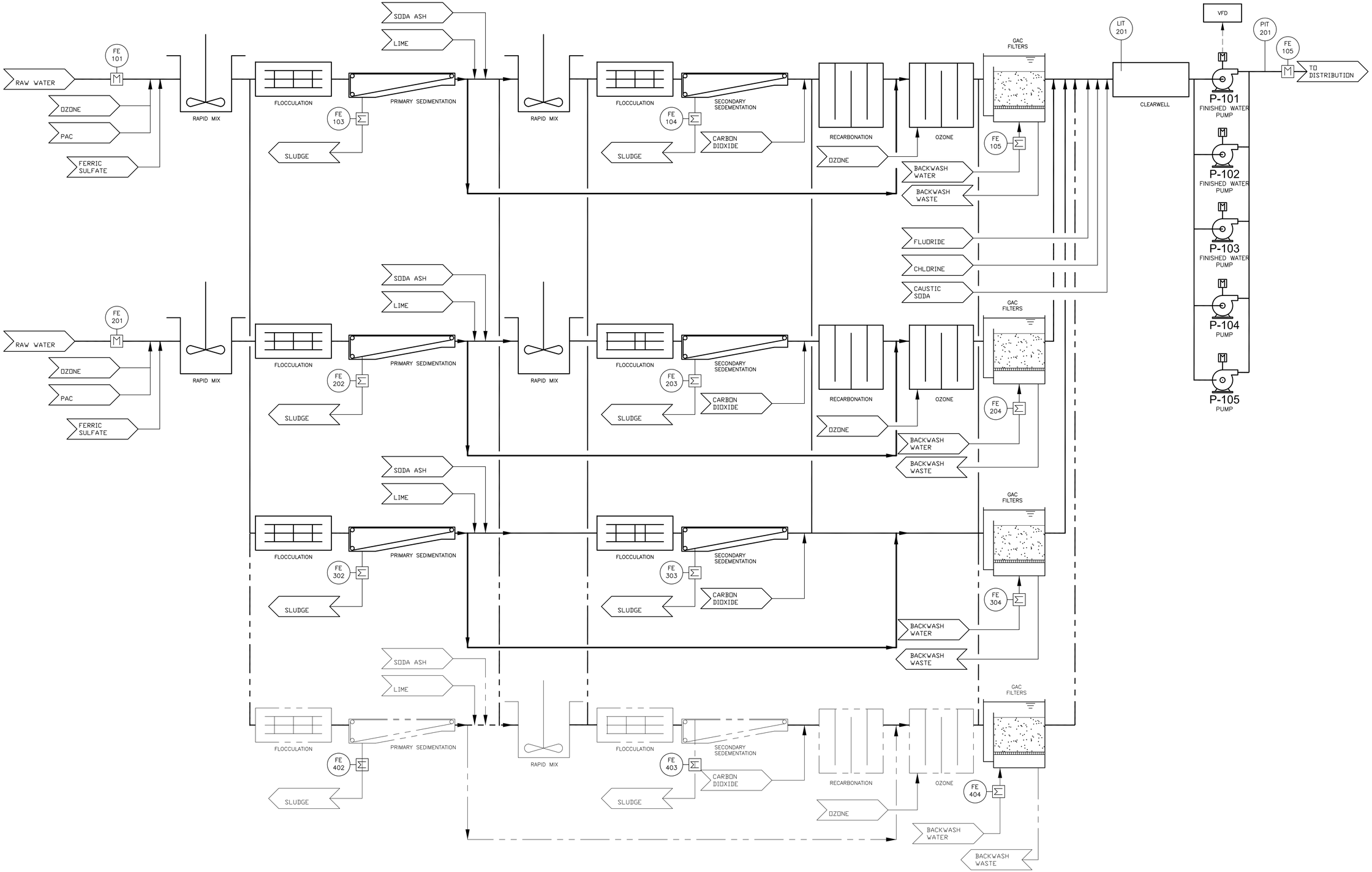

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Figure No. 7-11
Alt 3B - Process and Instrumentation Diagram
City of Piqua, OH

7.2.2 Bench Scale Testing Results

Preoxidation test results for ozone presented in **Table 7-14** are valid for this alternative also. In addition, ferric sulfate coagulation of blended plant composite raw water samples were also performed at ambient pH to determine the optimal dose using jar tests for turbidity/TOC reduction. Lime was not added to these tests. The results are shown in **Table 7-18**. Ferric sulfate doses ranged from 30 mg/L to 60 mg/L. Based on the results, the optimal coagulant dose is 40 mg/L. Since the TOC removal percentages for these tests are similar to those observed for single stage treatment simulation as shown in **Table 7-15,** the two-stage treatment alternative brings no additional benefit to the City.

Table 7-18. Jar Test Results for Two-Stage Treatment

Ferric Sulfate Dose (mg/L)	pH	TOC (mg/L)	% Removal
30	6.71	3.48	35
40	6.54	3.05	43
50	6.40	2.88	46
60	6.28	2.84	47

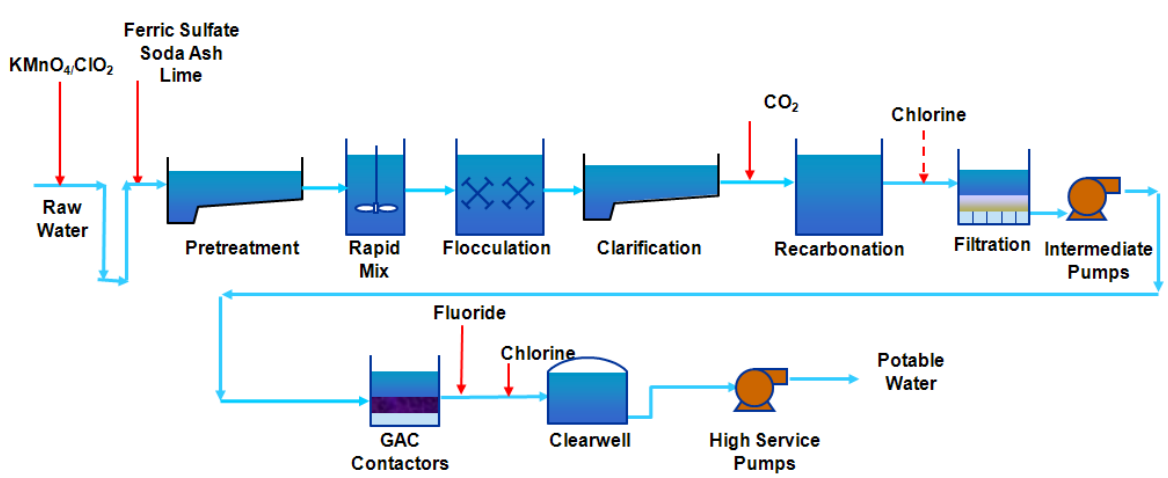
The SDS experiments were conducted using a ferric sulfate dose of 50 mg/L, chlorine dose of 3.1 mg/L and a final pH of 9.8. The measured TTHM concentration was approximately 104 µg/l. The higher TTHM concentrations for these tests compared those for single-stage simulation (91 µg/l) further validates the fact that two-stage treatment is not required at the new plant.

The main design implication from the bench scale test results presented in **Section 7.1.2** and **Section 7.2.2** is that integrated coagulation/lime softening processes with two-stage sequential mixing as opposed to two-stage treatment will achieve optimum removal of raw water TOC and TTHM precursors.

7.3 Lime Softening With Post-Filter GAC Contactors (Alternative 1D)

The process flow diagram for Alternative 1D is shown in **Figure 7-12**.

Figure 7-12. *Alternative 1D – Lime Softening with Post-Filter GAC Contactors*



7.3.1 Alternative Description

This alternative includes the following unit processes and treatment facilities:

- Raw Water Pump Station
- Pretreatment Facility (Optional)
- Chemical Facility
- Single-Stage Conventional Flash Mixing, Flocculation/Clarification Processes
- Recarbonation
- Conventional Filtration
- Post-Filter GAC Contactors for Adsorption
- Finished Water Storage
- High Service Pump Station
- Solids Handling Facility (Optional)

This alternative is also a single-stage treatment process where clarification and softening occur in the same sedimentation basins. The raw water supply and pumping scenarios for this alternative are identical to Alternative 1B. Raw water from various sources will be pumped to the rapid mix basins located inside the chemical building at the new plant site. An optional pretreatment facility is considered for this option for turbidity removal similar to Alternative 1B. Two packaged plate settlers are considered for pre-treatment to remove approximately 50-60% of the total raw water turbidity. The design criteria for the plate settlers are similar to those presented in Alternative 1B.

The design criteria for rapid mixing, flocculation, sedimentation, recarbonation, finished water storage, high service pump station and chemical dosages are similar to those presented under **Section 7.1** for Alternative 1B. The main difference with Alternative 1D is the use of conventional filtration instead of biological GAC and the elimination of ozonation. The conventional gravity filters in this alternative follow the recarbonation process and include sand and anthracite layers on top of a gravel support layer and underdrains. Rapid sand filters are usually backwashed at rate of 20 gpm/sf or a rate necessary to provide for a 50% expansion of the filter bed. The recommended auxiliary air scour rate is similar to help clean the mixed media. Conventional sand filters are mainly employed for turbidity/suspended solids reduction and achieve minimal TOC reduction. **Table 7-19** presents the design criteria for rapid sand filters.

Table 7-19. Conventional Filtration Design Criteria

Design Criteria	Value
No. of Filters	4
Type	Gravity Filters
Firm Capacity	7.2 MGD
Filtration Rate, each	4 gpm/sf
Filter Box Dimensions, each	22 ft L x 19 ft W x 20 ft H
Filter Area, each	418 sq. ft
Media Type	Sand, Anthracite and Gravel
Media Depth	24 in Anthracite; 12 in Sand; 12 in Gravel
No. of Backwash Pumps	2 (1 Duty, 1 Standby)
Backwash Rate	20 gpm/sf
Air Flow Rate	5 scfm/sf

Following filtration, water is pumped to six GAC pressure vessels for adsorption of organic contaminants and taste and odor control. **Table 7-20** presents the design criteria for the GAC feed pumps. Although GAC contactors could be located either pre-filtration or post-filtration, the high TOC content of the source water may prematurely exhaust the GAC media faster in pre-filtration. Since this replacement process is relatively expensive, CDM recommends that GAC contactors be considered in only post-filter configuration. **Table 7-21** presents the design criteria for post-filter GAC contactors. A 15-min empty bed contact time (EBCT) was used for conceptual design purposes. It is important to note that seasonal use of GAC contactors and/or a lower EBCT will result in significant cost savings in both capital and operating costs. This will be evaluated further during detailed design.

Table 7-20. GAC Feed Pumps Design Criteria

Design Criteria	Value
No. of Pumps	4 (3 Duty, 1 Standby)
Capacity, each	2.5 MGD
Pump Type	Vertical Turbine

Table 7-21. Post-Filter GAC Contactors Design Criteria

Design Criteria	Value
No. of Contactors	6 (Parallel)
Type	Pressure Adsorbers
Treatment Capacity, each	1.4 MGD
Empty Bed Contact Time (EBCT)	15 mins*
Contactor Dimensions, each	14 ft Dia x 27 ft H
Weight of carbon, each vessel	60,0000 lbs
Media Type	GAC
Media Depth	11.5 - 15.5 ft (cone bottom)
No. of Feed Pumps	4 (3 Duty, 1 Standby)
Pump Capacity, each	2.5 MGD

**To be verified by bench scale testing*

The baseline scenario assumptions for handling spent backwash water and solids handling are similar to those described for Alternative 1B. The design criteria for the optional solids handling facility are also similar to Alternative 1B. However, the design criteria for the backwash holding basins are slightly different for this alternative because of the additional backwash flow from higher filter backwash rates and a GAC contactor, that are periodically backwashed. **Table 7-22** presents the design criteria for the backwash holding tanks.

Table 7-22. Backwash Holding Tank Design Criteria

Design Criteria	Value
No. of Basins	2
Volume, each	232,500 gal*
Dimensions, each	61 ft L x 32 ft W x 16 ft SWD
No: of Recycle Pumps	3 (2 Duty, 1 Standby)
Pump Type	Centrifugal End Suction

**Provides capacity for 2 filter backwashes and 1 GAC contactor backwash in one day*

Offsite improvements for this alternative are similar to Alternative 1B. The preliminary hydraulic profile and proposed site layout with major yard piping for this alternative are shown in **Figure 7-13** and **Figure 7-14** respectively. The proposed site layout of the facility was developed with consideration of geotechnical conditions, site topography, and future expansions. An overall process and instrumentation diagram for this alternative is shown in **Figure 7-15**. This PER assumes that all treatment processes for this alternative will be monitored and controlled by plant SCADA. A detailed control scheme and specific monitoring and control points will be assessed further during detailed design.

Electrical requirements for power needs and standby power are similar to those in Alternative 1B. The only additional minor power requirements are for the GAC feed pumps.

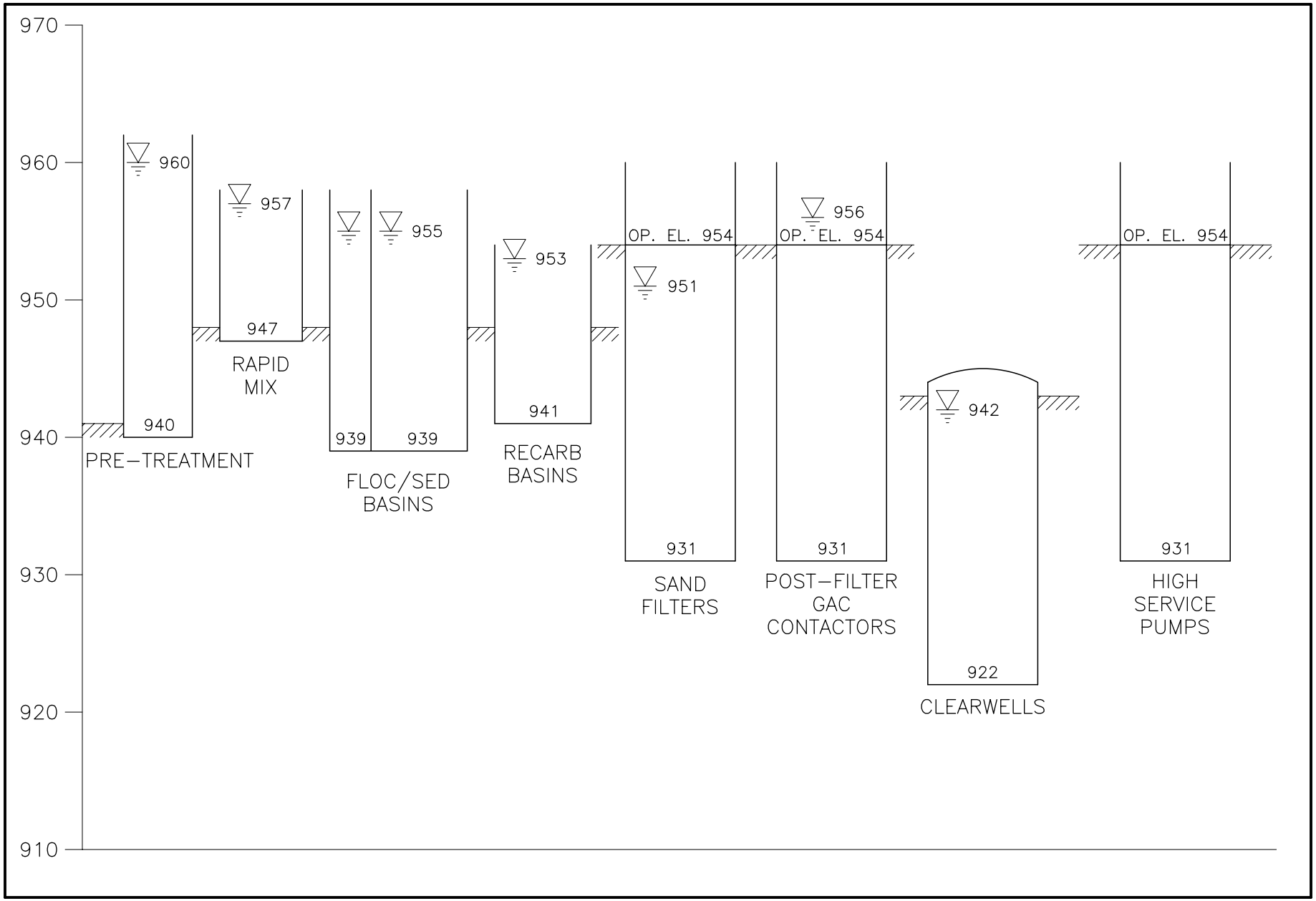


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Figure 7-13
Alt. 1D - Lime Softening with Post-Filter GAC Contactors
Hydraulic Profile

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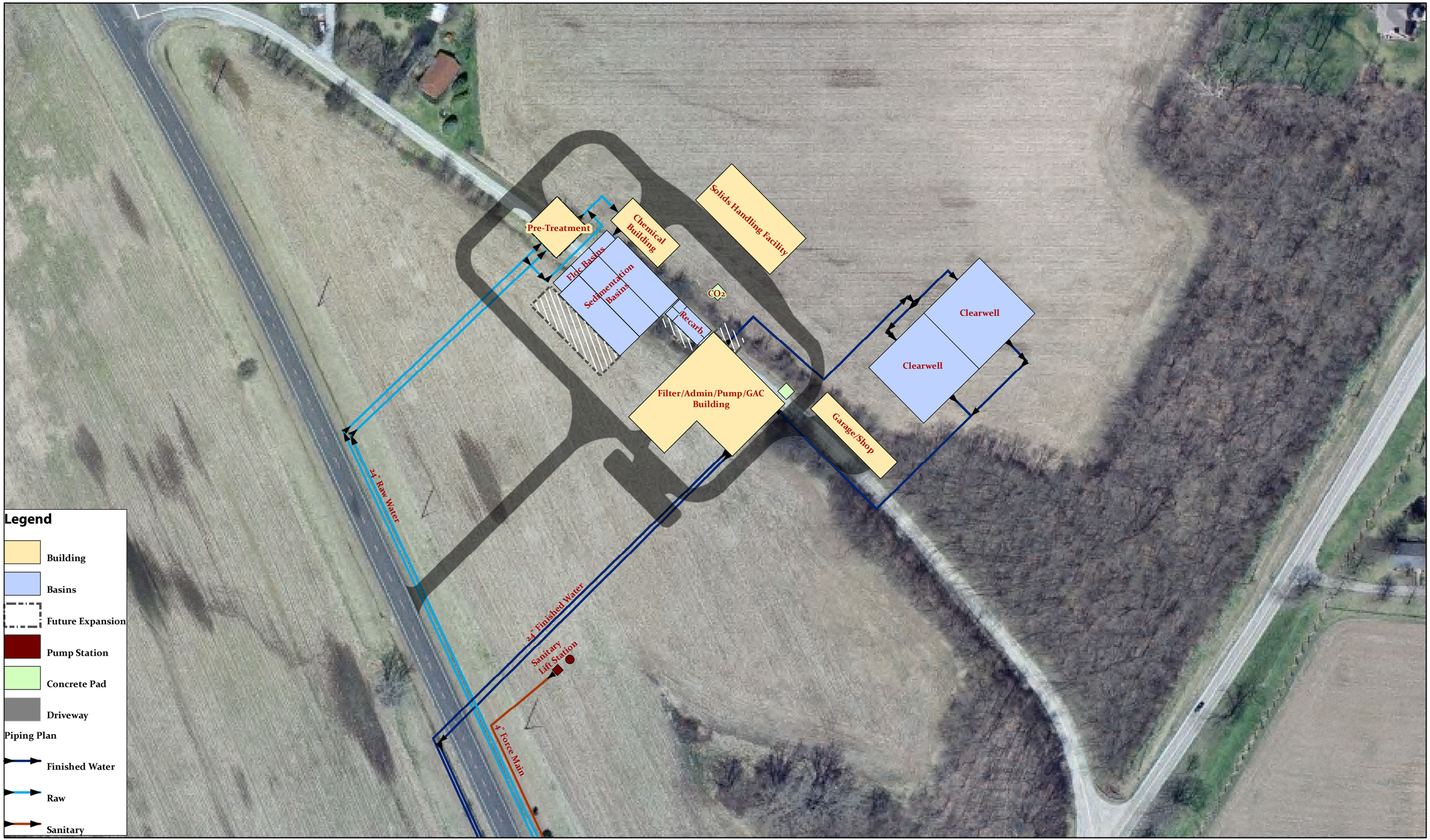

0 80 160 320 Feet

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Figure 7-14
Alt. 1D - Lime Softening with Post-Filter GAC Contactors
City of Piqua, OH

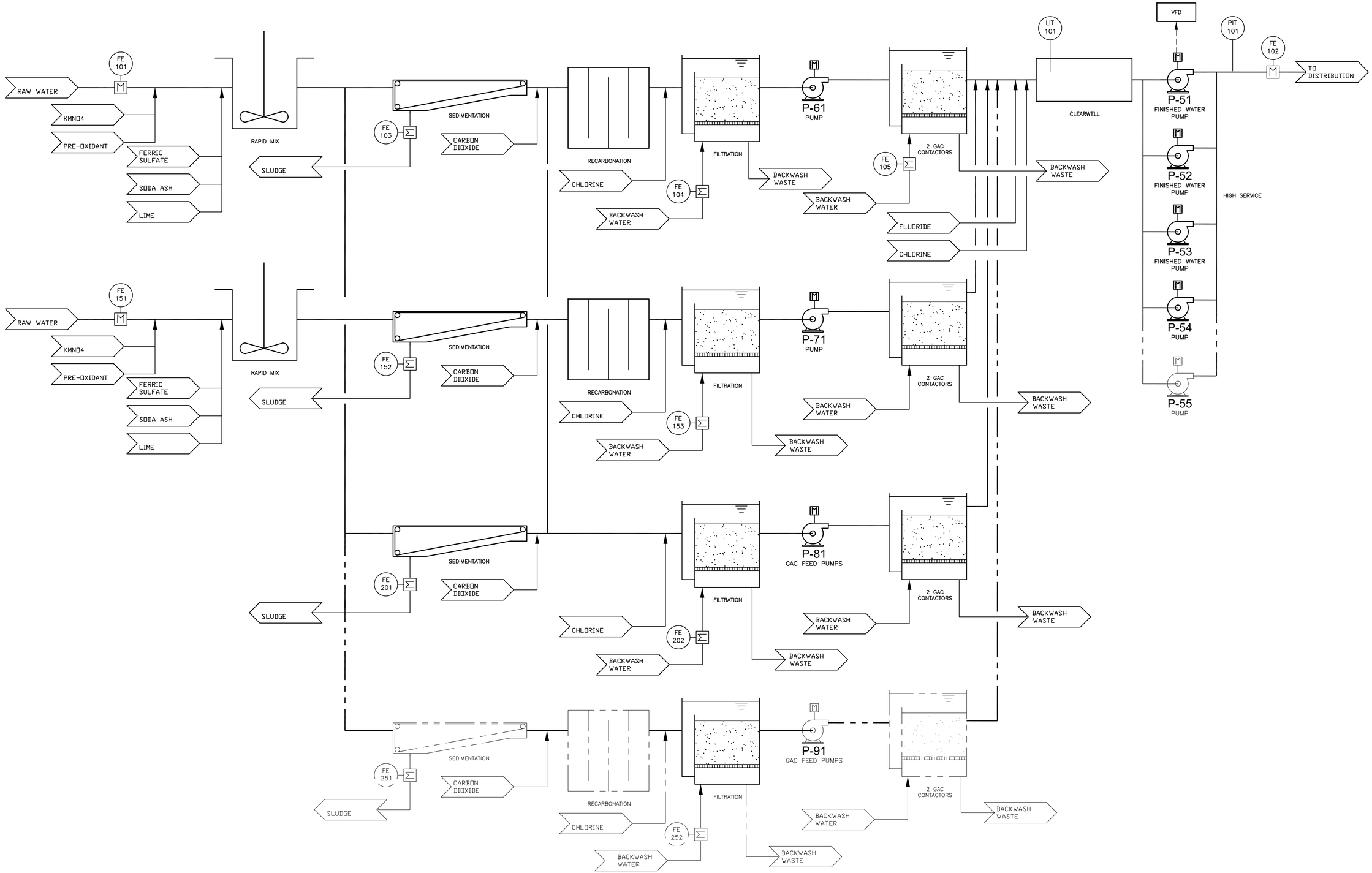

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Figure No. 7-15
Alt 1D - Process and Instrumentation Diagram
City of Piqua, OH

7.3.2 Bench Scale Testing Results

Preoxidation demand/decay testing studies were conducted on Swift Run Lake water (worst in terms of WQ) at ambient pH followed by simulated distribution system (SDS) experiments to measure 3-day TTHMFP in order to establish the pre-oxidant dose for potassium permanganate and chlorine dioxide at 0.1 mg/L residual after 5 min contact time. The decay results are shown in **Table 7-23** and **Table 7-24**.

Potassium permanganate doses of 0.35 mg/L, 0.5 mg/L and 0.75 mg/L showed moderate potential to oxidize raw water TOC as evident from the slow decay. At these doses, the reaction times ranged from 10 to 20 minutes to exhaust the potassium permanganate. Chlorine dioxide doses of 1 mg/L, 1.25 mg/L and 1.5 mg/L showed strong potential to oxidize raw water TOC. At these doses, the reaction times ranged from 5 to 10 minutes to exhaust the chlorine dioxide. The optimal dose for permanganate is approximately 0.35 mg/L since this dose maintained a 0.1 mg/L minimum residual after 5 min of contact time.

The SDS experiments for permanganate were conducted at a dose of 0.4 mg/L, chlorine dose of 4.8 mg/L and a final pH of 9.8. The measured TTHM concentration was approximately 90.5 µg/l. The SDS experiments for chlorine dioxide were conducted at a dose of 1.2 mg/L, chlorine dose of 4.5 mg/L and a final pH of 9.8. The measured TTHM concentration was approximately 97.5 µg/l. However chlorite concentration was not measured. Research has shown that 40-70% of the chlorine dioxide dose gets converted to chlorite which is a regulated contaminant. As a result, the optimal chlorine dioxide dosage cannot be determined at this stage.

It is important to note that only one of these preoxidants will be used if this alternative is selected by the City.

Table 7-23. Potassium Permanganate Demand/Decay Data

Time (min)	Potassium Permanganate Dose (mg/L)		
0	0.75	0.50	0.35
5	0.42	0.23	0.13
10	0.29	0.13	0.03
20	0.13	0.03	

Table 7-24. Chlorine Dioxide Demand/Decay Data

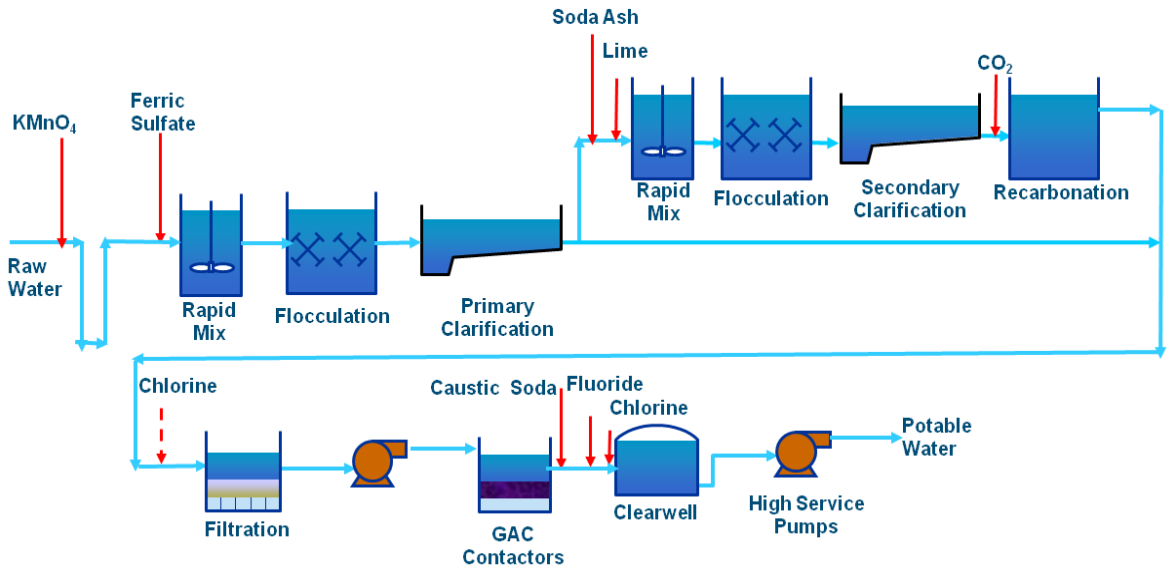
Time (min)	Chlorine Dioxide Dose (mg/L)		
0	1.5	1.25	1.00
1	0.65	0.47	0.31
3	0.41	0.27	0.14
5	0.29	0.17	0.07
10	0.14	0.02	

Jar test results for single-stage treatment presented in **Table 7-15** are conservative since they were performed without the addition of a preoxidant and likely valid for this alternative also.

7.4 Enhanced Coagulation and Lime Softening With Post-Filter GAC Contactors (Alternative 3D)

The process flow diagram for Alternative 1D is shown in **Figure 7-16**.

Figure 7-16. *Alternative 3D – Enhanced Coagulation and Lime Softening with Post-Filter GAC Contactors*



7.4.1 Alternative Description

This alternative includes the following unit processes and treatment facilities:

- Raw Water Pump Station
- Pretreatment Facility
- Chemical Facility
- Two-Stage Conventional Flash Mixing, Flocculation/Clarification Processes
- Recarbonation
- Conventional Filtration
- Post-Filter GAC Contactors for Adsorption
- Finished Water Storage
- High Service Pump Station
- Solids Handling Facility (Optional)

This alternative is similar to Alternative 1D described previously. The main difference between Alternative 1D and 3D is single versus two-stage treatment for conventional flash mixing, flocculation and clarification processes. Besides, Alternative 1D includes a pretreatment facility whereas Alternative 3D does not. It is assumed that the additional detention time obtained through the second stage basins is sufficient to reduce turbidity and TOC and hence a separate pretreatment facility is not recommended for this alternative. The first-stage basins would essentially serve as the "pre-treatment" basins.

The raw water supply and pumping scenarios for this alternative are similar to Alternative 1B. Raw water is directly sent to the two rapid mix chambers for first stage treatment. The rapid mix chambers will be located inside the chemical building and will be used for addition of various treatment chemicals such as PAC and ferric sulfate for first stage treatment.

Following rapid mix, the water flows through a channel to three parallel treatment trains comprising three first stage flocculation basins and three first stage sedimentation basins. The flocculation basins will be equipped with horizontal paddle wheel flocculators similar to Alternative 1B. Flocculated water from the flocculation basins flows to the sedimentation basins which are equipped with chain and flight sludge collectors. Design criteria for first-stage and second-stage conventional treatment presented in **Table 7-16** to **Table 7-17** are valid for this alternative also.

The design criteria for all downstream processes such as recarbonation, finished water storage, high service pumping, optional solids handling facility as well as the chemical doses for this alternative are similar to Alternative 1B described above. The design criteria for conventional filtration and post-filter GAC contactors are similar to that presented for Alternative 1D. The design criteria for the backwash holding basin are similar to Alternative 1D and is shown in **Table 7-22**.

Offsite improvements for this alternative are similar to Alternative 1B. The preliminary hydraulic profile and proposed site layout with major yard piping for this alternative are shown in **Figure 7-17** and **Figure 7-18** respectively. The proposed site layout of the facility was developed with consideration of geotechnical conditions, site topography, and future expansions. An overall process and instrumentation diagram for this alternative is shown in **Figure 7-19**. It is assumed that all processes for Alternative 3D will also be monitored and controlled by plant SCADA. A detailed control scheme and specific monitoring and control points will be assessed further during detailed design.

Electrical requirements for power needs and standby power are similar to those in Alternative 1B. The additional tanks, drive units and GAC feed pumps have nominally more power requirements, but not significant enough to increase the primary electric or standby power needs.

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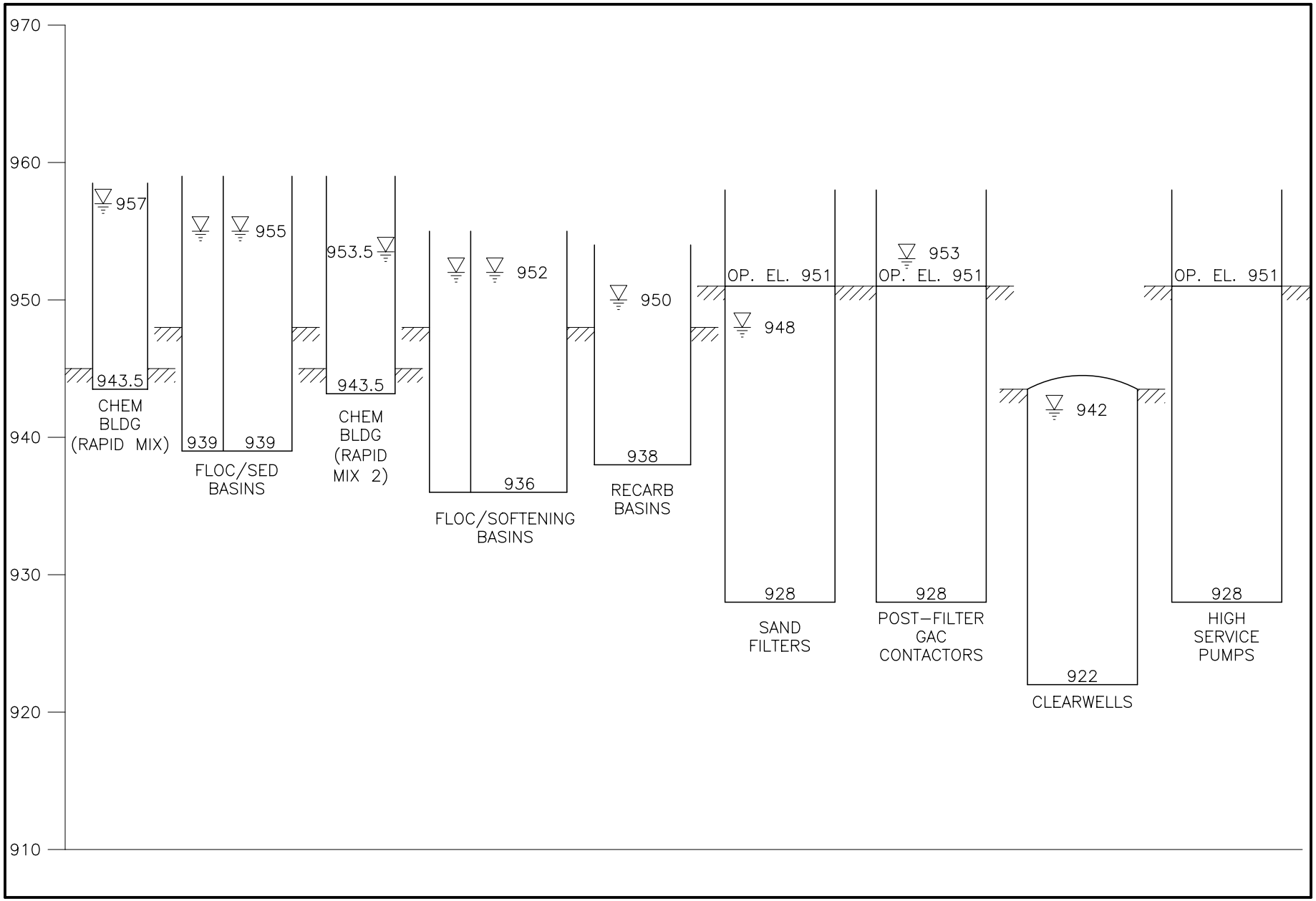

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Figure 7-17
Alt. 3D - Enhanced Coagulation and Lime Softening with Post-Filter GAC Contactors
Hydraulic Profile

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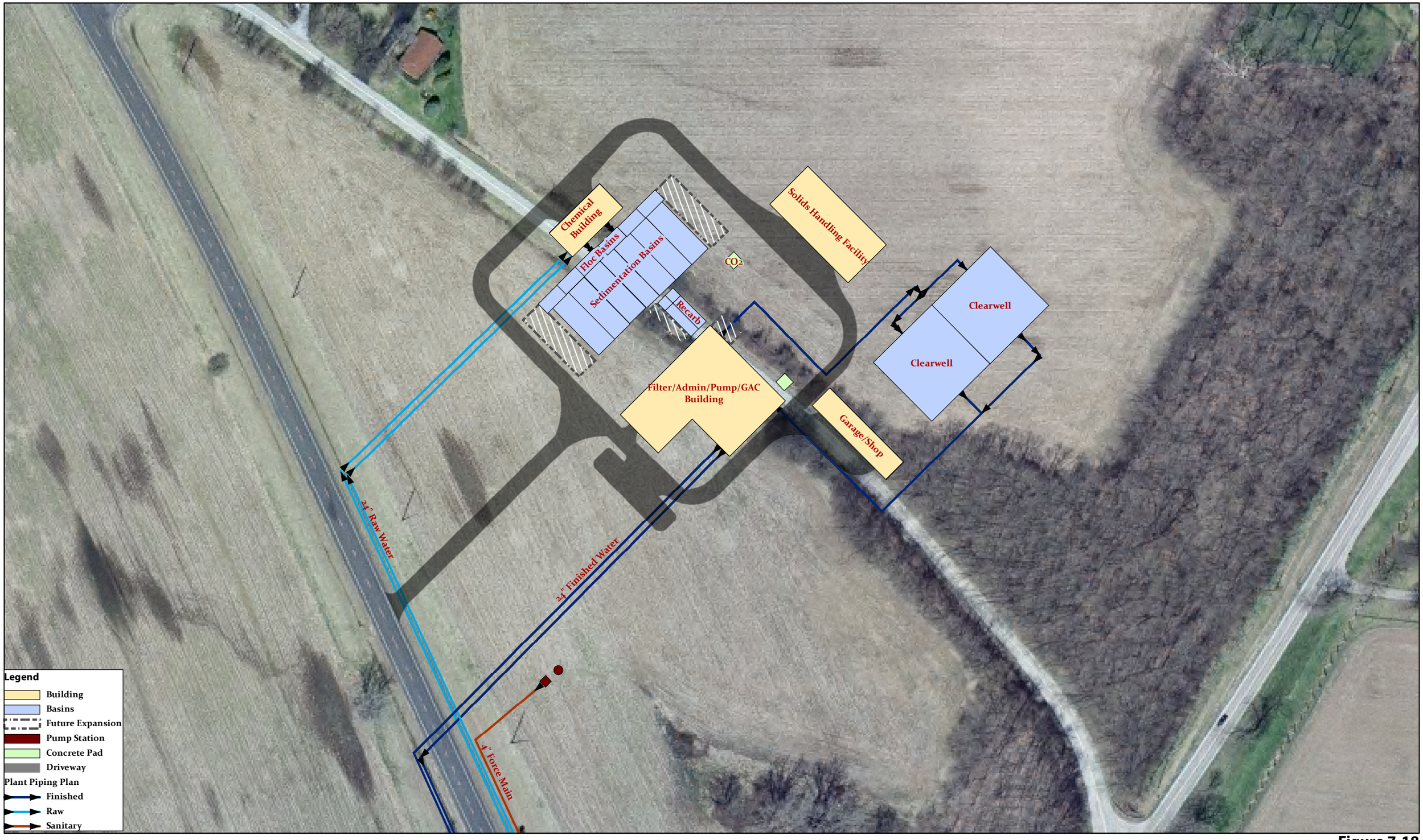

0 80 160 320 Feet

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Figure 7-18
Alt. 3D - Enhanced Coagulation and Lime Softening with Post-Filter GAC Contactors
New Water Treatment PlantCity of Piqua, OH

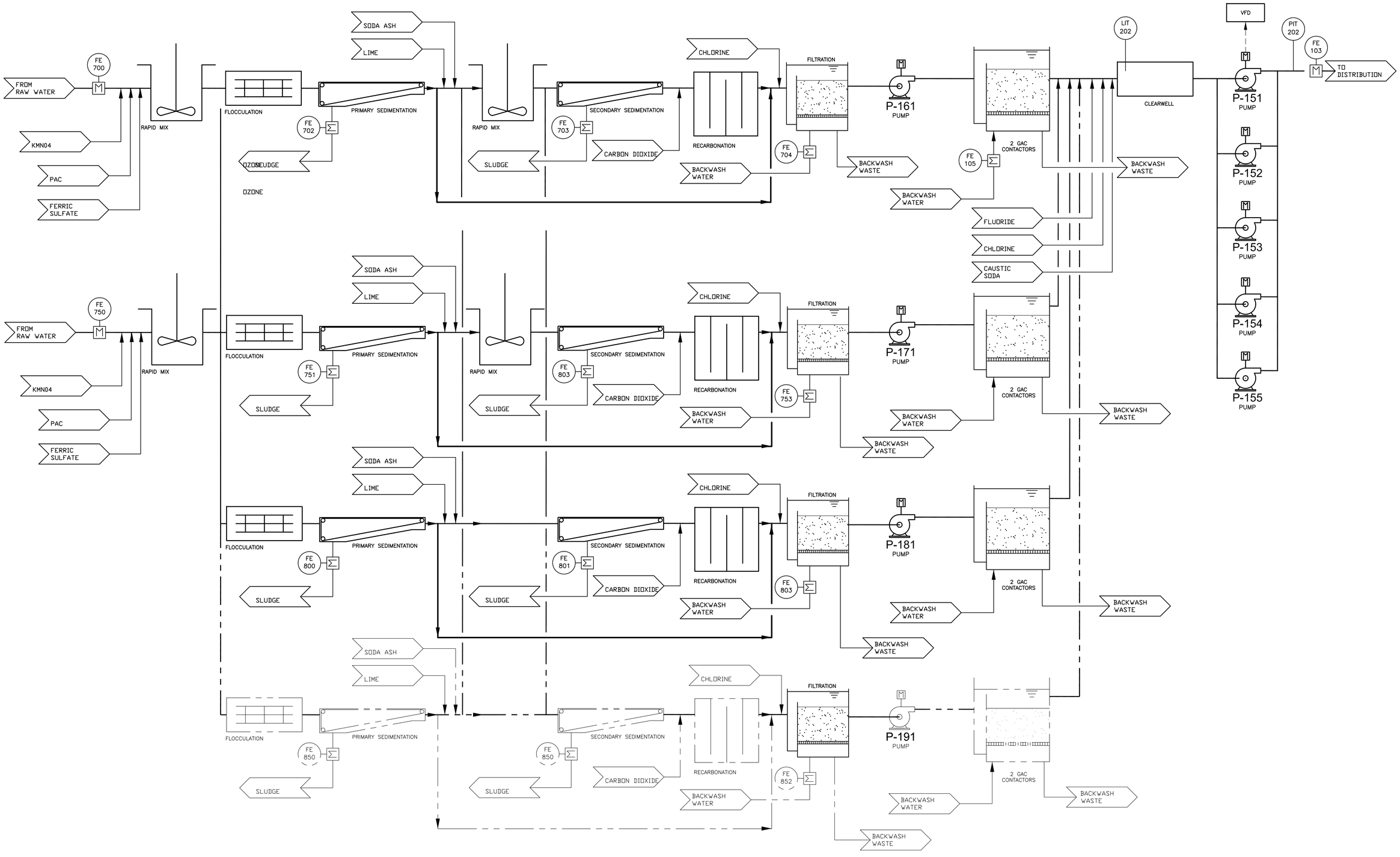

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Figure No. 7-19
Alt 3D - Process and Instrumentation Diagram
City of Piqua, OH

7.4.2 Bench Scale Testing Results

Preoxidation tests with potassium permanganate and jar tests for two-stage treatment presented for the other alternatives are valid for this alternative also.

7.5 Cost Summary

The cost evaluation of the four short-listed process train alternatives is presented below. The cost evaluation includes construction costs, annual operating costs, and present worth costs for each alternative.

7.5.1 Construction Costs for Alternatives

Construction costs for each alternative were developed based on the conceptual design of facilities, as presented in this Section. Equipment costs were obtained from manufacturers and from similar recent construction projects. An allowance was used for electrical, instrumentation and controls, HVAC, plumbing, fire protection and drinking water lab equipment. An overall contingency of 10% was included. The preliminary opinion of probable construction cost for each alternative for conceptual phase for each alternative are presented in **Table 7-25** to **Table 7-28**.

Table 7-25. Alternative 1B Preliminary Opinion of Probable Construction Cost

Description	Cost
WTP	
Site Work	$2,720,000
Onsite Filter Backwash Facility	$330,000
Yard Piping	$720,000
Chemical Building	$3,600,000
Flocculation Basins	$900,000
Sedimentation Basins	$1,890,000
Recarbonation Basins	$520,000
Ozone Building	$3,390,000
Filter/Administration/Pump Building	$4,590,000
Clearwells	$2,340,000
Subtotal	**$21,000,000**
Offsite Work	
Raw Water PS	$1,010,000
Gravel Quarry Pump Station Improvements	$280,000
24" Raw Water Piping	$1,170,000
12" Gravel Quarry Raw Water Piping	$100,000
24" Finished Water Piping	$1,250,000
6" Sludge Piping	$190,000
Subtotal	**$4,000,000**
Construction Contingencies	$2,500,000
Engineering Design	$2,150,000
Engineering Construction Administration	$2,100,000
Project Total	**$31,750,000**
Optional Facilities	
Pretreatment Building	$2,280,000
Garage/Maintenance Building	$1,020,000
Solids Handling Building[1]	$2,530,000
Subtotal	**$5,830,000**
Contingencies	$583,000
Total Optional Facilities	**$6,413,000**

1. Note that the cost of an additional sludge lagoon has not been included. Instead, cost of a mechanical dewatering facility has been presented based on the City's request.

Table 7-26. Alternative 3B Preliminary Opinion of Probable Construction Cost

Description	Cost
WTP	
Site Work	$2,720,000
Onsite Filter Backwash Facility	$330,000
Yard Piping	$720,000
Chemical Building	$3,620,000
Flocculation Basins	$1,440,000
Sedimentation Basins	$3,860,000
Recarbonation Basins	$520,000
Ozone Building	$3,390,000
Filter/Administration/Pump Building	$4,590,000
Clearwells	$2,340,000
Subtotal	**$23,530,000**
Offsite Work	
Raw Water PS	$1,010,000
Gravel Quarry Pump Station Improvements	$280,000
24" Raw Water Piping	$1,170,000
12" Gravel Quarry Raw Water Piping	$100,000
24" Finished Water Piping	$1,250,000
6" Sludge Piping	$190,000
Subtotal	**$4,000,000**
Construction Contingencies	$2,760,000
Engineering Design	$2,150,000
Engineering Construction Administration	$2,100,000
Project Total	**$34,540,000**
Optional Facilities	
Garage/Maintenance Building	$1,020,000
Solids Handling Building[1]	$2,530,000
Subtotal	**$3,550,000**
Contingencies	$355,000
Total Optional Facilities	**$3,905,000**

1. Note that the cost of an additional sludge lagoon has not been included. Instead, cost of a mechanical dewatering facility has been presented based on the City's request.

Table 7-27. Alternative 1D Preliminary Opinion of Probable Construction Cost

Description	Cost
WTP	
Site Work	$2,720,000
Onsite Filter Backwash Facility	$330,000
Yard Piping	$720,000
Chemical Building	$3,600,000
Flocculation Basins	$900,000
Sedimentation Basins	$1,890,000
Recarbonation Basins	$520,000
Filter/Administration/Pump/GAC Building	$7,870,000
Clearwells	$2,340,000
Subtotal	**$20,890,000**
Offsite Work	
Raw Water PS	$1,010,000
Gravel Quarry Pump Station Improvements	$280,000
24" Raw Water Piping	$1,170,000
12" Gravel Quarry Raw Water Piping	$100,000
24" Finished Water Piping	$1,250,000
6" Sludge Piping	$190,000
Subtotal	**$4,000,000**
Construction Contingencies	$2,490,000
Engineering Design	$2,150,000
Engineering Construction Administration	$2,100,000
Project Total	**$31,630,000**
Optional Facilities	
Pretreatment Building	$2,280,000
Garage/Maintenance Building	$1,020,000
Solids Handling Building[1]	$2,530,000
Subtotal	**$5,830,000**
Contingencies	$580,000
Total Optional Facilities	**$6,410,000**

1. Note that the cost of an additional sludge lagoon has not been included. Instead, cost of a mechanical dewatering facility has been presented based on the City's request.

Table 7-28. Alternative 3D Preliminary Opinion of Probable Construction Cost

Description	Cost
WTP	
Site Work	$2,720,000
Onsite Filter Backwash Facility	$330,000
Yard Piping	$720,000
Chemical Building	$3,620,000
Flocculation Basins	$1,440,000
Sedimentation Basins	$3,860,000
Recarbonation Basins	$520,000
Filter/Administration/Pump Building	$7,870,000
Clearwells	$2,340,000
Subtotal	**$23,420,000**
Offsite Work	
Raw Water PS	$1,010,000
Gravel Quarry Pump Station Improvements	$280,000
24" Raw Water Piping	$1,170,000
12" Gravel Quarry Raw Water Piping	$100,000
24" Finished Water Piping	$1,250,000
6" Sludge Piping	$190,000
Subtotal	**$4,000,000**
Construction Contingencies	$2,750,000
Engineering Design	$2,150,000
Engineering Construction Administration	$2,100,000
Project Total	**$34,420,000**
Optional Facilities	
Garage/Maintenance Building	$1,020,000
Solids Handling Building[1]	$2,530,000
Subtotal	**$3,550,000**
Contingencies	$355,000
Total Optional Facilities	**$3,905,000**

1. Note that the cost of an additional sludge lagoon has not been included. Instead, cost of a mechanical dewatering facility has been presented based on the City's request.

7.5.2 Annual O&M Costs for Alternatives

Annual O&M costs for each alternative were developed to include the following:

- Power
- Sludge Disposal Costs
- GAC Replacement Costs
- Parts and Materials Replacement Costs

The following assumptions were made for development of the annual O&M costs:

- Chemical costs were developed using average chemical doses and chemical costs currently used by the City.
- It is assumed that no additional staff will be required by the City at the new plant.
- Power costs were developed based on the horsepower requirements for equipment which was obtained from equipment cut-sheets and using the City's current cost for power which is $0.08 per kW-hour.
- 20-yr present worth O&M costs were developed based on the Office of Management and Budget Circular A-94, Appendix C discount rate of 2.1%.
- It is assumed that one GAC media replacement will be required per year for Alternative 1D and Alternative 3D (will be verified through bench scale testing) at full design and an average of 2/3 of the units on average for the 20 year period. This is a worst-case scenario. However, there is opportunity for potential savings in O&M costs with seasonal use of GAC under Alternatives 1D and 3D. This will be evaluated during detailed design phase.
- For Alternates 1B and 3B, the replacement rate is assumed to be half that of post filter contactors or one replacement every two years.
- A GAC replacement cost of $1.5/lb was used based on manufacturer's recommendation.
- Annual replacement costs for miscellaneous equipment parts and materials were assumed to be 0.5% of the initial construction cost estimate.

Table 7-29 presents the total annual O&M costs and the 20-year present worth costs for each alternative.

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Table 7-29. Annual O&M Cost Summary

Description	O&M Costs			
	Alternative 1B	Alternative 3B	Alternative 1D	Alternative 3D
Power Cost	$550,000	$575,000	$700,000	$728,000
Chemical Cost	$272,000	$272,000	$264,000	$264,000
Lime Sludge Disposal Cost	$100,000	$100,000	$100,000	$100,000
GAC Replacement Cost	$180,000	$180,000	$360,000	$360,000
Replacement Parts and Materials Cost	$165,000	$165,000	$166,000	$165,000
Total Annual O&M Costs	$1,267,000	$1,293,000	$1,590,000	$1,618,000
20-year Present Worth Cost	$21,000,000	$21,000,000	$26,000,000	$26,00,000

Table 7-30 presents a summary of the present worth costs for each alternative.

Table 7-30. Present Worth Cost Summary

Alternative	Total Construction Cost	Present Worth Operating Cost	Total Present Worth Project Cost
1B	$27,500,000	$21,000,000	$48,500,000
3B	$30,290,000	$21,000,000	$51,290,000
1D	$27,380,000	$26,000,000	$53,380,000
3D	$30,170,000	$26,000,000	$56,170,000

The present worth cost evaluation resulted in the following ranking of alternatives:

- 1. Alternative 1B
- 2. Alternative 3B
- 3. Alternative 1D
- 4. Alternative 3D

7.6 Final Screening of Alternatives and Selection

The four shortlisted alternatives were presented to the City at Workshop No. 2 (Process Decision Workshop) and evaluated using the same screening criteria described in **Section 6.** However, the 'Relative Cost' criterion was replaced with Capital and Present Worth O&M costs. The weights allocated to the individual criteria were changed based on discussions with the City at Workshop No.2. Each process train alternative was also assigned a score from 1 to 5 (5 being the best score) for each of the five screening criteria. The results of the scoring and final selection process are presented in **Table 7-31.**

Section 8
Selected Alternative

This Section describes Alternative 1D- "Lime Softening with Post-Filter GAC Contactors" which is the selected alternative for the new treatment plant.

8.1 Design Criteria for New WTP

The design criteria for the selected alternative is presented in **Section 7** and summarized below in **Table 8-1** which has been prepared using Ohio EPA's Approved Capacity Document.

Table 8-1. Design Criteria for New WTP

1 Component	2 Number of Units	3 Design Standards	4 Design Criteria	5 Required/ Recommended	6 Component Capacity (MGD)	7 Flow Basis of Component Capacity/Ratio	8 Equivalent Maximum Day Capacity (MGD)
Rapid Mixers	2		detention time <30 seconds	required	7.5	maximum-day 1.0	7.5
		G value of +/- 1,000		recommended	7.5	maximum-day 1.0	
Flocculation Basins	3		detention time >30 minutes	required	7.5	maximum-day 1.0	7.5
		GT value of 25 – 100		recommended	7.5	maximum-day 1.0	
Sedimentation Basins	3		detention time >4 hours	required	7.5	maximum-day 1.0	7.5
		flow-through velocity < 0.5 fpm					
		peak-hour treatment with all in-service		recommended	7.5	maximum-day 1.0	

1 Component	2 Number of Units	3 Design Standards	4 Design Criteria	5 Required/ Recommended	6 Component Capacity (MGD)	7 Flow Basis of Component Capacity/Ratio	8 Equivalent Maximum Day Capacity (MGD)
Solids Contact Clarifiers[1]	3		Upflow rate <1 gpm/SF for clarification and <1.75 gpm/SF for softening	required	7.5	maximum-day 1.0	7.5
			Weir overflow rate < 10 gpm/SF for clarification and <20 gpm/SF for softening	required	7.5	maximum-day 1.0	
			detention time >30 minutes for flocculation and 2-4 hrs for clarification	recommended	7.5	maximum-day 1.0	
Recarbonation	2		detention time>=20 minutes	recommended	7.2		7.2
			mixing chamber detention time >=3 minutes	recommended	7.2	maximum-day 1.0	
			diffuser submergence >=7.5 ft	recommended	7.2		
Filters	4	peak-hour treatment without all in service	maximum day without largest	required	7.2	maximum-day 1.0	7.2
				recommended	7.7	peak-hour treatment 1.07	
			Filtration rate each 4 gpm/SF	recommended	7.2	maximum-day 1.0	
GAC Contactors	6		Empty Bed Contact Time - 15 min based on bench scale testing	recommended	7.2	maximum-day 1.0	7.2
			Hydraulic Loading Rate 5.4 gpm/SF		7.2		
Clearwells	2		CT for 0.5 log Giardia	required	7.2	peak-hour treatment 1.07	6.8
		storage for peak-hour demand		recommended	9	peak-hour treatment 1.07	
Finished Water Pump Station	3		Peak-hour production without largest	required	9	peak-hour production 1.33	6.8
		fire-flow demand with all in-service		recommended	Un-determined	fire-flow demand	

1. Note that this is an alternative that will be considered during detailed design phase; final design will comply with appropriate Approved Capacity design criteria.

Table 8-2 presents the basis of design for the City's water supply sources in accordance with Ohio EPA's Approved Capacity Document.

Table 8-2. Design Criteria for Water Supply Sources

1 Component	2 Number of Units	3 Design Standards	4 Design Criteria	5 Required/ Recommended	6 Component Capacity (MGD)	7 Flow Basis of Component Capacity/Ratio	8 Equivalent Maximum Day Capacity (MGD)
Gravel Quarry Source	1		Safe Yield 2.0 MGD[1]	required	2.0	Average Day	
Gravel Quarry PS	2 Pumps		Average-day without largest	required	2	Average Day 1.55	3.1
		Maximum-day with all in-service		recommended	4	Maximum Day 1.0	
Great Miami River (GMR) and Swift Run Source	2		Safe Yield 8.6 MGD[2]	required	8.6	Average Day 1.55	13.3
Great Miami River and Swift Run PS	3 Pumps		Average-day without largest	required	7.5	Average Day 1.55	11.6
		Maximum-day with all in-service		recommended	11.25	Maximum Day 1.0	

1. Source: Based on gravel pit withdrawals rates recorded by the City in the last five years
2. Source: 7-day, 50-yr low flow (7Q50) extrapolated from "Low-Flow Characteristics of Streams in Ohio through Water Year 1997", USGS

8.2 Source Water Supply

The raw water supply source for a treatment plant can either be surface water, groundwater, a combination of surface water and groundwater or groundwater under the direct influence of surface water. The type of water source has a major influence on the treatment required. In general, surface water requires a higher degree of treatment to remove turbidity, organics and taste and odors when compared to groundwater and is prone to seasonal variations. Groundwater is typically harder than surface water; but has consistent WQ and, hence, is easier to treat. Surface water has historically been subjected to more stringent regulations than groundwater and this trend will continue in the future. The existing supply has been in use for a number of years and no significant increase in demand is expected above the existing approved capacity.

8.2.1 Surface Water

For the selected alternative, Piqua's existing raw water supply, obtained from three surface water sources, will continue to be a significant contributor to the new WTP. The three sources include the GMR, a gravel pit located on the east side of the river adjacent to the lime sludge lagoon and the City's Hydraulic System that is comprised of a series of small lakes-Swift Run Lake, North Pond, Echo Lake and Frantz Pond and several

connecting canals. Note that a series of improvements are proposed for the source water pumping stations and associated infrastructure as described in **Section 5.** The cost associated with these improvements is included in the preliminary opinion of probable construction cost for this alternative as presented in **Table 7-27** in **Section 7.**

The GMR has sufficient capacity to provide supply to Piqua for the foreseeable future. Swift Run Lake (or Reservoir) is the largest contributor to the City's Hydraulic System. The total volume of the Hydraulic System is estimated to be approximately 195 MG. The new combined raw water pump station for both these sources will include three pumps rated at 3.75 MGD each, with one pump operating as a standby pump and room for one additional future pump. Thus the total firm capacity from these water sources is 7.5 MGD (11.25 MGD in the future).

The gravel pit water source was constructed to augment raw water supply. It is located on the east side of the GMR and adjacent to the lime sludge lagoon. As described in **Section 7.0**, water from this source will be pumped from the quarry to the new plant via a dedicated 24-inch line. Two 2 MGD pumps will be provided at this source, with one pump operating as a standby pump. Thus the total firm capacity from the gravel pit water source is 2 MGD.

The total firm capacity from all surface water sources is approximately 9.5 MGD; expandable to over 13 mgd. This is based on the hydraulic capacity of the pumping facilities and is not reflective of a rigorous assessment of assured sustainable capacity.

8.2.2 Groundwater

To augment the surface water sources, the City decided to investigate the potential for groundwater in the area of the new plant. As a result, CDM and its subconsultant, Terran Corporation, were tasked with estimating the production potential from the recently identified BVA in the vicinity of the proposed plant on two properties (Fry property and Smith property) and preparing an initial characterization of the groundwater quality. The hydrogeologic investigation consisted of drilling two exploratory soil borings, construction of the monitoring wells (piezometers), water level tracking, groundwater sampling/analysis and preliminary computer modeling. Results of the investigation are summarized below and included in a separate report by Terran Corporation titled "Preliminary Technical Evaluation of the Fry and Smith Properties for a Groundwater Augmentation of a Surface Water Supply, City of Piqua, OH" dated September 20, 2011.

- Sand and gravel deposits are encountered at various depths throughout the Piqua BVA and appear to be hydraulically interconnected.
- Depth to water in the Fry property piezometer was approximately 4-10 ft below ground surface. The water level in the Smith property piezometer was approximately 15 ft above ground surface.
- The groundwater from both the properties were sampled for Ohio EPA new well parameters. The Fry property had no significant detectable organic or inorganic constituents of concern. The Smith property did have detectable parameters such as dissolved arsenic, iron and total dissolved solids which indicate that potential treatment may be needed for raw water from this area.
- Computer modeling indicates that each property has the potential of supporting a single well production of 1500 gpm, which is 2.16 MGD when run continuously over a 24-hr period. Although the simulations were steady state and assumed 24-hr/day operation, it can safely be assumed that the production rate is approximately 1 MGD when each well is operated for 12 hrs per day.

According to the report, the total potential capacity that could be available from future groundwater source is approximately 2 MGD. However this is an ongoing study and the exact quantity of future potential sustainable capacity of ground water to augment the surface water supply cannot yet be defined.

8.3 Rapid-Mix, Flocculation and Sedimentation

The City's new WTP will be provided with conventional treatment processes comprising rapid mix, flocculation and sedimentation. These processes will assist in the removal of turbidity and TOC. Residual turbidity in settled water shortens the run time of filters. Particles in the water also shield microorganisms from disinfectants. For these reasons, there are regulatory requirements for achieving low turbidity levels after filtration process. The design considerations for the City's conventional treatment facilities are presented below.

8.3.1 Rapid Mixing Basins

Following pretreatment with a pre-oxidant, the water enters the rapid mixing chambers located inside the chemical building. Based on conceptual design, two rapid mix systems in concrete basins in series will be provided, each equipped with variable-speed vertical shaft mixers. The main design implication from the bench scale testing results, presented in **Section 7,** is that integrated coagulation/lime softening processes with two-stage sequential mixing as opposed to two-stage treatment will achieve optimum removal of TTHM precursors. As a result, two side by side rapid mix compartments with mixers for ferric sulfate addition and two side by side rapid mix compartments with mixers for lime addition and all sized to handle a total raw water flow of 7.5 MGD is recommended. Of these 4 units, 2 will be in service and 2 will serve as backup thereby providing complete redundancy in accordance with the requirements of TSS.

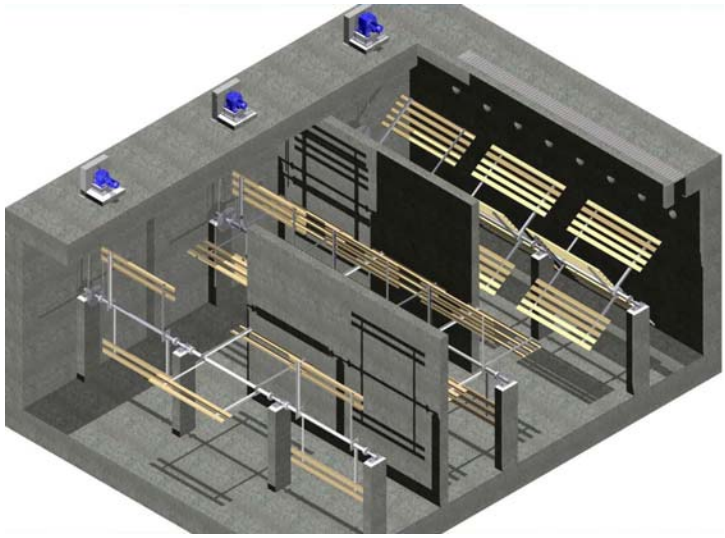
Pretreated water enters the mixing chambers through a pipe at the bottom and flows upward into the rotating mixing impeller. Ferric sulfate and lime are injected into the rapid mix basins. Slide gates will be provided at the inlet and outlet of each individual rapid mix basin for isolation purposes. Each mixer will be mounted above the rapid mix basin on a pedestal to facilitate maintenance. The rapid mix design criteria for the selected alternative are presented in **Table 7-2** in **Section 7**.

8.3.2 Flocculation Basins

The design for the selected alternative includes three flocculation basins operating in parallel. Following rapid mixing, flow is conveyed by a coagulated water channel to the common influent channel ahead of the flocculation basins. Slide gates will be provided that allow flow to be diverted from the influent channel to each of the three flocculation basins. The flocculation basins are each equipped with two horizontal paddle wheel flocculator with variable speed motors, to achieve tapered energy gradient. The flocculator orientation can be perpendicular or parallel to the flow. A flocculator includes components such as a drive motor, gear reducer, shaft connections, shaft and bearing assembly.

A wide variety of flocculation mixing mechanisms have been used in water treatment such as vertical shaft mechanical mixers, horizontal shaft mechanical mixers and, hydraulic mixing systems. Paddle wheel flocculators are the most common. There are three types of paddle wheel flocculators: walking beam, horizontal and vertical. Horizontal paddle wheel flocculators are recommended because of the City's familiarity with the operation and because of the low maintenance they offer to attain homogenous mixing. A typical arrangement for a multi-stage horizontal paddle wheel flocculator is shown in **Figure 8-1**. The selection of the type of flocculator will be made during detailed design phase after discussions with the City.

Figure 8-1. *Horizontal Paddle Wheel Flocculator*



The proposed layout also allows for expansion of the plant to a future flow of 9 MGD which may require a fourth treatment train consisting of one flocculation basin and one sedimentation basin. The fourth train will have common wall construction with one of the three initial treatment trains. The flocculation basin design criteria for the selected alternative are presented in **Table 7-3** in **Section 7**. There is an effluent channel to collect flocculated water from the three flocculation basins and also a common cross-over channel prior to the influent channel of the sedimentation basins. Gates are provided in the cross-over channel to allow flexibility of routing flow from any flocculation basin to any sedimentation basin.

8.3.3 Sedimentation Basins

The purpose of the sedimentation facilities is to remove floc particles formed by coagulation and flocculation. The sedimentation basins will be designed to effectively remove the flocculated particles from the raw water, and the basins will include rectangular chain and flight sludge collector mechanisms to prevent the buildup of sludge. The basins will also be provided with drains to empty the basins.

The new treatment plant will be designed with three sedimentation basins operating in parallel. Flocculated water will flow from the common cross-over channel into the influent channel ahead of the sedimentation basins. A ported baffle wall will equally distribute flocculated water across the width of the sedimentation basin to reduce the potential of short-circuiting. Each sedimentation basin will be designed to treat 2.5 MGD of flocculated water. Each sedimentation basin will have a dimension of 110-ft L x 35-ft W x 16-ft SWD and will be equipped with chain and flight collector mechanism. The settled water will flow to common settled water channels at the end of each sedimentation basin. From the common settled water channels, water will be directed to a common cross-over channel and then into the inlet channel of the two downstream recarbonation basins. Sludge collected from the inlet end of the basins will be withdrawn by sludge pumps located in a tunnel below the influent channel and pumped to the existing sludge holding basin across from the existing treatment plant. The underflow from the sludge holding basin will be pumped to the off-site lagoon for final disposal. The general design criteria for the proposed sedimentation basins are listed in **Table 7-4** in **Section 7**.

As described in **Section 6**, several types of clarifiers are available including conventional, plate and tube settlers, etc. For purposes of this PER, rectangular sedimentation basins with conventional chain and flight equipment has been assumed. The system is easy to install and is comprises light-weight components such as fiberglass flights which also makes maintenance tasks simple and convenient. To operate, flights mounted on two parallel strands of metallic or non-metallic chain scrape the settled solids along the tank floor to sludge hoppers. A typical chain and flight system us shown in **Figure 8-2.**

Figure 8-2. *Chain and Flight Sludge Collector*

Several manufacturers offer non-metallic chain and flight sludge collection equipment such as Envirex, Polychem, and Walker Process.

Another option is to install circular solids contact clarifiers. In a solids contact clarifier coagulation, flocculation, and sedimentation take place in one basin. The unit employs the principle of internal slurry recirculation to accelerate chemical reactions and dense particle growth. Mixing and reaction, return flow, and sludge removal functions are performed in a single-basin unit. The solids contact clarifier maintains active slurry volume throughout the treatment zones, with settling taking place only in the sludge concentrators. It also allows for waste sludge removal with or without the use of bottom scrapers; however, sludge collection is anticipated for Piqua's facility. While solids contact clarifiers offer an attractive clarification option with a smaller footprint and minimal chemical flocculation requirement as compared to conventional rectangular basins, they require optimization of sludge wasting frequency to perform efficiently. A conventional flash/floc/settling process offers better operational flexibility and control for chemical additions. Several manufacturers offer solids contact clarifiers such as Infilco, Envirex/Dorr-Oliver, and Siemens.

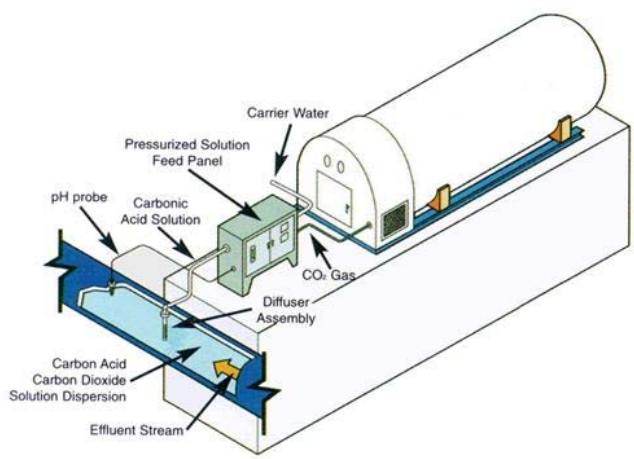
Final determination of the efficacy of solids contact clarification process compared to conventional flash /floc/sedimentation technology will be assessed during preliminary design. If solids contact clarification is determined to be appropriate for the new Piqua water treatment plant, equipment selection will be made during the detailed design phase. The design criteria for solids contact clarification presented earlier in this section are taken from the Ohio EPA Approved Capacity Document and represent the standard for design comparison.

8.4 Recarbonation Basins

Two new cast-in-place recarbonation basins will be provided downstream of the sedimentation basins for the selected alternative. Flow will be routed through channels from the sedimentation basins to the recarbonation basins. Each recarbonation basin is sized to handle the total plant flow of 7.2 MGD (assuming 4% loss through the sedimentation basins). This will allow one basin to be taken out of service for cleaning or inspection when needed. This will provide redundancy and operational flexibility to the plant staff. Both basins will be in service during normal operation. Each basin will have its own mixing chamber and reaction chamber. The mixing and reaction chambers are configured such that the plant staff will have the flexibility to use both mixers even when one recarbonation basin is taken down for maintenance. The recarbonation basin design criteria for the selected alternative are presented in **Table 7-5** in **Section 7**.

Several options are available for feeding liquid CO_2 to the mixing chamber of the recarbonation basins such as direct CO_2 gas injection system and pressurized solution feed system. Pressurized solution feed systems are rapidly gaining popularity in the water industry for pH control and offer several advantages. The system includes a gaseous CO_2 storage tank, pressurized solution feed panel, diffuser assembly and pH probes. A typical arrangement is shown in **Figure 8-3**. The CO_2 gas is forced into solution under high pressure and remains in solution until injected into the water to be treated.

Figure 8-3. *Pressurized Solution Feed System*



Since the chemical reaction times are short, the pH is controlled and the water is stabilized within a three minute period. The conceptual design for the recarbonation system is based on a 20-min detention time in accordance with TSS requirements. However there may be opportunity for reducing the detention time, decreasing the basin footprint and thereby reducing construction costs. This will be discussed with Ohio EPA in detail and addressed during the design phase.

8.5 Filtration

The City's new treatment plant will incorporate conventional dual-media filtration as part of the treatment process. The filtration process will follow conventional treatment and recarbonation processes. Four filters are provided, each rated for 2.4 MGD. A design filtration rate of 4 gpm/sf was

assumed for planning purposes based on Ohio EPA's Approved Capacity document. Each filter consists of gravel, sand and anthracite for a total media depth of 4 ft.

The proposed filters will be constructed with a combination air/water underdrain system and have a filtration area of approximately 420 sf per filter. Based on the design maximum backwash rate of 20 gpm/sf, a total backwash flow rate of 8,400 gpm will be required. Two backwash pumps (1 duty, 1 standby) will be provided and housed in the same room as the finished water pumps in the filter building. The proposed air scour system will include two (one standby) 2,100-standard cubic feet per second (scfm) capacity blowers and associated control panel for air scour supply to all four filters. A single blower will deliver the necessary air flow during a backwash cycle. The blowers were sized based on TSS requirements of 5 scfm/sf of filter. The blowers will be located in a separate room in the first floor of the filter building to minimize noise impacts throughout the facility. The filtration design criteria for the selected alternative are presented in **Table 7-19** in **Section 7**.

Several underdrain options are available for these conventional filters such as false bottom with pre-cast blocks, clay tile, wheeler bottoms, pipe laterals, trilateral, folded plate, plastic blocks or low profile underdrains. A detailed underdrain evaluation and selection will be determined during detailed phase in conjunction with the City. For conceptual design and cost estimation purposes, a dual parallel lateral underdrain with air/water configuration as shown in **Figure 8-4** has been used. A variety of underdrains are offered by manufacturers such as Leopold, Tonka, Roberts Filter, Siemens, and AWI.

Figure 8-4. *Filter Underdrain*

Rate of flow controllers will be installed on the outlet piping from each filter to allow automatic filter operation at a relatively constant filtration rate and water surface elevation. In addition to filtration rate, turbidity will also be monitored for each filter to record filter performance. As a filter becomes dirty, a backwash cycle is typically initiated based on one of three criteria: high head loss, high turbidity, or cumulative filter run time. Most commonly, filters are backwashed due to high head loss. Upon completion of the backwash cycle, the filter will be returned to service. Waste backwash water collected from the filters will flow by gravity to two onsite backwash holding basins.

8.6 Granular Activated Carbon Contactors

GAC pressure contactors will be used primarily for control of organics after filtration. The GAC contactors will provide for additional taste and odor control, TOC removal, and atrazine removal.

Six GAC pressure contactors are provided for a total treatment capacity of 7.2 MGD. The design of GAC contactors has been based upon a minimum EBCT of 15 minutes for preliminary cost estimating purposes as

verified through bench-scale Rapid Small Scale Column Testing (RSSCT) presented in **Appendix C.** There is potential for a lower EBCT, fewer pressure vessels and considerable cost savings by optimizing the operations based on the results of bench scale testing. It is recommended that a detailed optimization assessment be performed during preliminary design to finalize the sizing and configuration of the pressure GAC vessels. Several manufacturers offer GAC pressure vessels such as Calgon, Norit, Westech, and Roberts Filter. The design criteria for GAC contactors are presented in **Table 7-21** in **Section 7.**

Filtered water will be routed to a wet well located in the basement of the Filter/GAC Building. Four vertical turbine pumps will be located above the wet well to draw suction from the filtered water wet well and pump the water to the GAC contactors. Each GAC contactor is sized to hold 60,000 pounds of media and is 14 ft in diameter. Each contactor has a stainless steel inlet distributor with laterals to evenly distribute water during normal operation and evenly collect water during backwash. The contactors will be located in the filter building and will span two floors because of their height.

A conceptual layout of the combined filter/administration/pump/GAC building is shown in **Figure 8-5** and **Figure 8-6**.

8.7 Finished Water Storage

Following the GAC adsorption process, finished water will flow by gravity from the GAC contactors to two below ground concrete clearwells for disinfection contact time and to buffer peak hour pumping requirements. From the clearwells, the water is pumped into the distribution system for consumer use. The clearwells will be located on the eastern side of the treatment plant to take advantage of the nominally lower elevation and proximity to the customer base.

The clearwells are sized based on the design criteria of providing a finished water storage volume that would match future average day flow (ADF). Using a future maximum day flow of 9 MGD and a peaking factor of 1.55, the future ADF is approximately 6 MGD. Currently the total elevated storage available is 3 MG; therefore, the new clearwells are sized to provide a volume of 1.5 MG each for a total storage of 3 MG. For planning purposes, rectangular clearwells with common wall construction has been assumed. The clearwell design criteria for the selected alternative are presented in **Table 7-9** in **Section 7**.

Other design considerations related to the clearwell design include hydraulics, inlet/outlet configuration, drain and overflow connections, structural design, maintenance access, and other miscellaneous issues which will be evaluated during detailed design phase. Concrete baffle walls will be provided inside the clearwells to comply with SWTR for disinfection CT.

Note that an assessment of alternate structural systems should be considered during the final design phase to determine whether cast-in-place concrete structures such as described above or pre-cast, pre-stressed concrete tanks as defined by AWWA D-110 or even D-115 standards may provide a cost savings. The basis of this design has been developed using the cast-in-place approach which has been supported by several instances of local contractors offering savings for such design over the D-110/115 type tanks.

8.8 High Service Pump Station

The high service pump station will be located in the basement of the filter building in the pump room. The new pump station will be supplied with treated water from the clearwells. The high service pumps will

discharge into the Central Low District. Four pumps will be provided each rated for 3 MGD. Of these, one pump will be operated as a standby pump.

Pump sizing and selection was developed based on providing a firm pumping capacity equivalent to or greater than the Max Day condition. The firm pumping capacity is the total pumped flow at a given head condition with the largest available pump off-line. Total pumping capacity is the total pumped flow at a given head condition with all available pumps operating. Based on these, the firm pumping capacity of the high service pump station is 9 MGD and the total pumping capacity is 12 MGD.

The high service pumps will be designed for delivery of the ultimate plant production capacity to the distribution system. To provide a facility which can accommodate projected future expansions, space will be provided for additional pumps, suction and discharge header piping will be sized for ultimate flows, and pump branch connections will permit future installation of larger diameter piping and fittings for future additional pumps.

The high service pump station will consist of horizontal split-case centrifugal pumps. The high service pump layout will be based on Hydraulic Institute (HI) standards for suction header and branch connection spacing and will provide sufficient space for future pump installation. The electrical room will be located near the high service pumps for providing power to the pumps.

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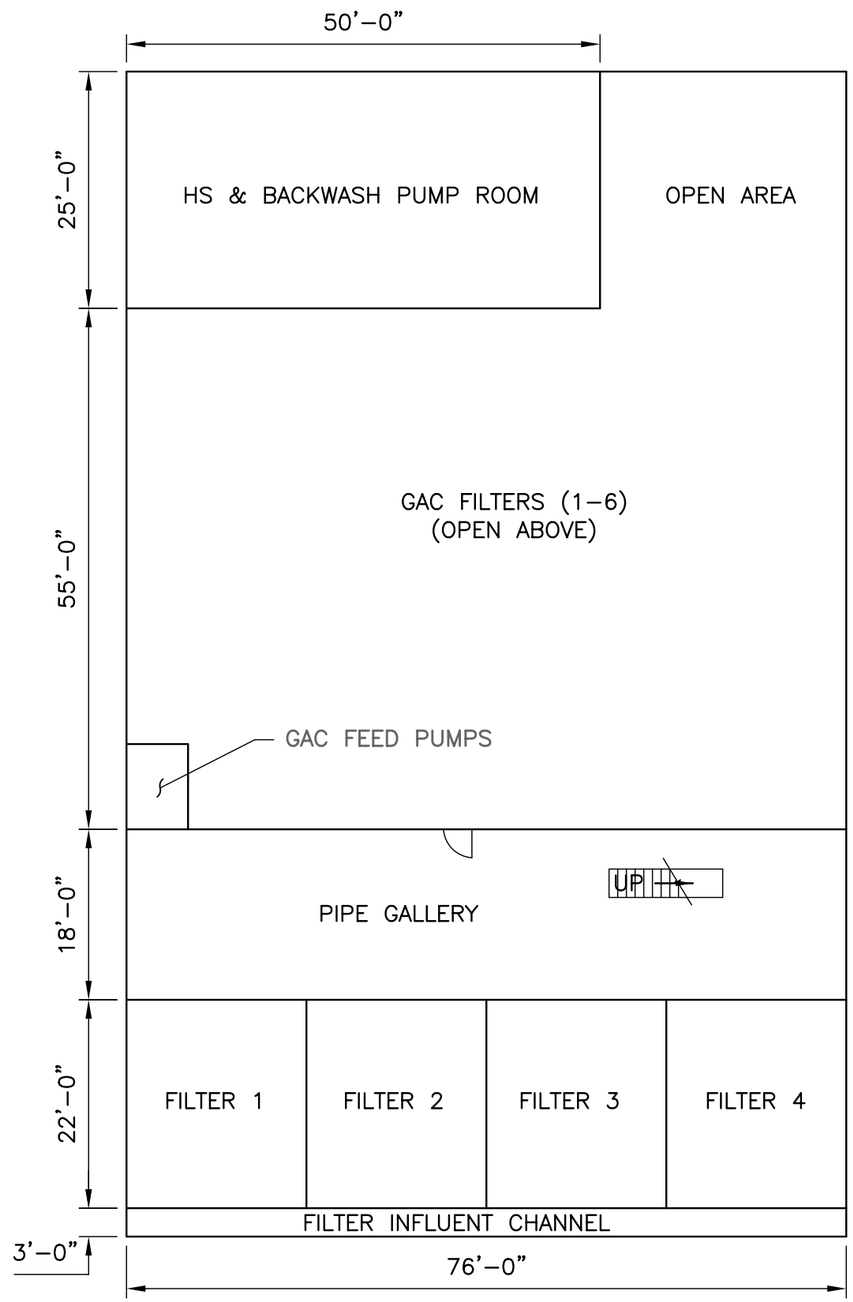
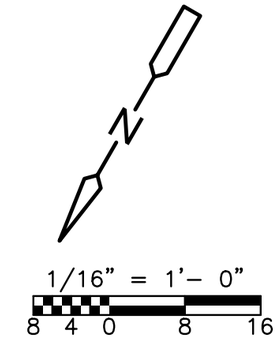


PLAN

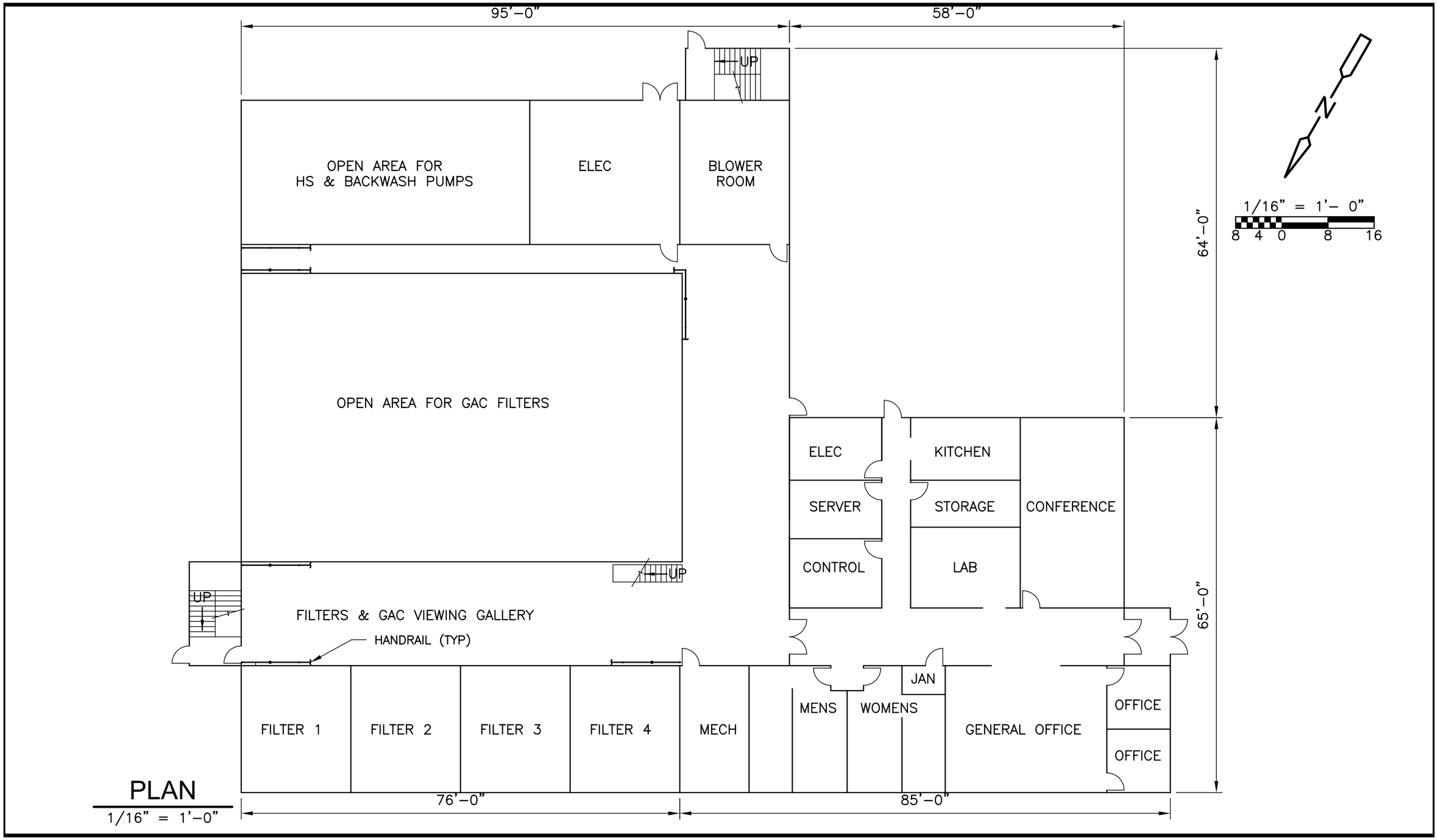
1/16" = 1'–0"



CDM

consulting • engineering • construction • operations

Figure 8-5
Filter Building Lower Plan
City of Piqua, OH.



PLAN
1/16" = 1'–0"

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Figure 8-6
Filter Building Upper Plan
City of Piqua, OH.

8.9 Chemical Systems

The new treatment plant will have the capability to feed the following chemicals to the treatment processes:

- PAC
- Ferric Sulfate
- Lime
- Potassium Permanganate
- Soda Ash
- Hydrofluosilicic Acid
- Sodium Hypochlorite
- Carbon Dioxide
- Sodium Hexametaphosphate
- Caustic Soda (Sodium Hydroxide)

Although both potassium permanganate and chlorine dioxide are both viable pre-oxidants with favorable bench scale testing results, it is recommended that the City use potassium permanganate as a pre-oxidant at the new plant because of their familiarity in feeding this chemical at the existing plant. Besides, it is also beneficial to avoid the formation of chlorite and chlorate which are regulated contaminants that are formed as a result of using chlorine dioxide. All the above chemicals will be housed in a separate chemical building except PAC. Since PAC is explosive in nature, the automatic bulk bag feeder for PAC will be located outside the chemical building in a separate self contained pre-fabricated assembly to isolate this from other chemicals and equipment. Bags, bulk bag unloaders and feeders for soda ash and sodium hexametaphosphate will also be located inside the chemical building.

Bulk chemical storage facilities are sized to store at least a 30-day supply of all chemicals, as required by TSS. Sizing of bulk storage is based on the ultimate future average plant flow of 6.4 MGD and average chemical dosage rate of the treatment plant (see **Table 7-11**). Each storage tank will be provided with a level sensor for monitoring and to facilitate filling operation. Chemical feed equipment will be designed to provide proper dosage under all operating conditions.

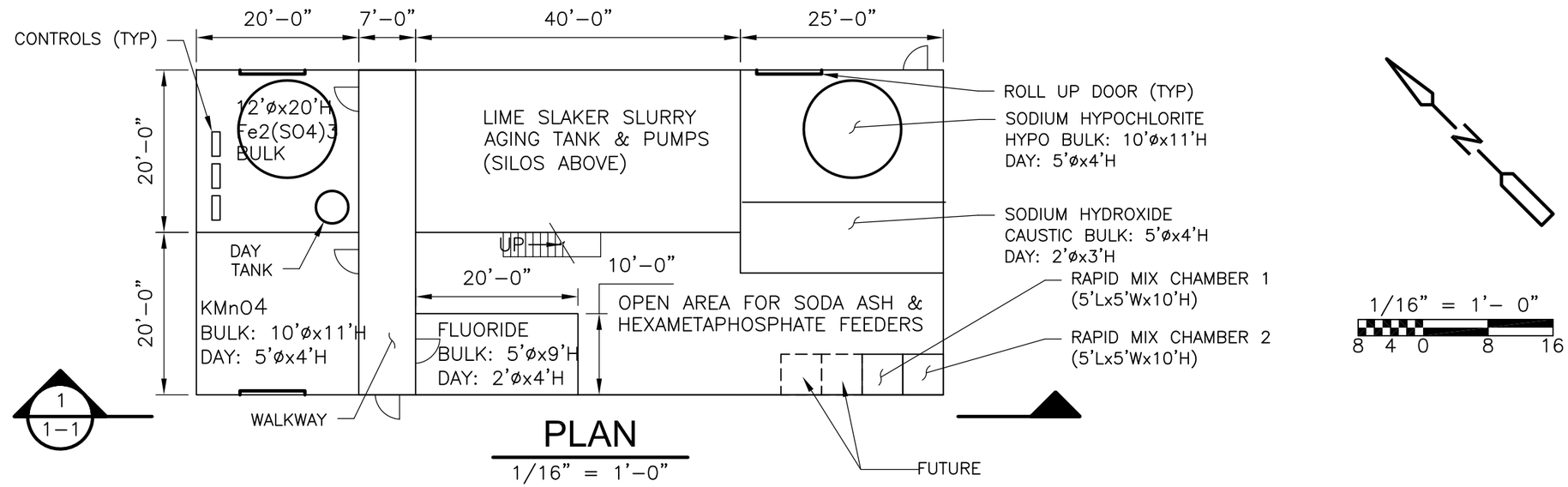
Bulk storage required for the new plant is summarized in **Table 8-3**.

Table 8-3. Chemical Bulk Storage Summary

Chemical Name	Number Required	Design Criteria	Number of Days of Storage Provided
PAC	56	50-lb bags	50
Ferric Sulfate	2	7,500 gal 8.5' Dia x 19.5' H	30
Lime	2	Bulk storage bins 4215 cu ft each	30
Potassium Permanganate	1	5,500 gal 10' Dia x 11' H	30
Soda Ash	20	50-lb bags	50
Hydrofluosilicic Acid	1	1,200 gal 5' Dia x 9' H	50
Sodium Hypochlorite	1	5,500 gal 10' Dia x 11' H	30
Caustic Soda	1	450 gal 5' Dia x 4' H	30
Sodium Hexametaphosphate	30	50-lb bags	30

The chemical bulk storage tanks, day tanks and pumps will be located inside the chemical building. Day tanks are sized to provide no more than 30 hr supply in accordance with the requirements of TSS. Space is provided in the rooms of the chemical building for the installation of future pumps to provide chemicals to the treatment trains installed as part of future plant expansions. A conceptual layout of the chemical building is shown in **Figure 8-7**.

From **Table 8-2**, it is evident that the largest bulk storage tanks/silos are for lime. The batch lime slaking system such as that considered for the City's new plant mixes quick lime (CaO) and water, hydrates the CaO and separates the impurities and grit from the resultant calcium hydroxide slurry which is fed to the application point. A schematic of a typical lime system is shown in **Figure 8-8.**The lime system for consideration for the City's new water plant includes two storage silos, two slakers, two slurry aging tanks, two slurry delivery loops, pumps, dosing assemblies and a common fine grit classifier. Except the silos, all the other equipment will be housed inside the chemical building. The silos will be located on top of the chemical building.



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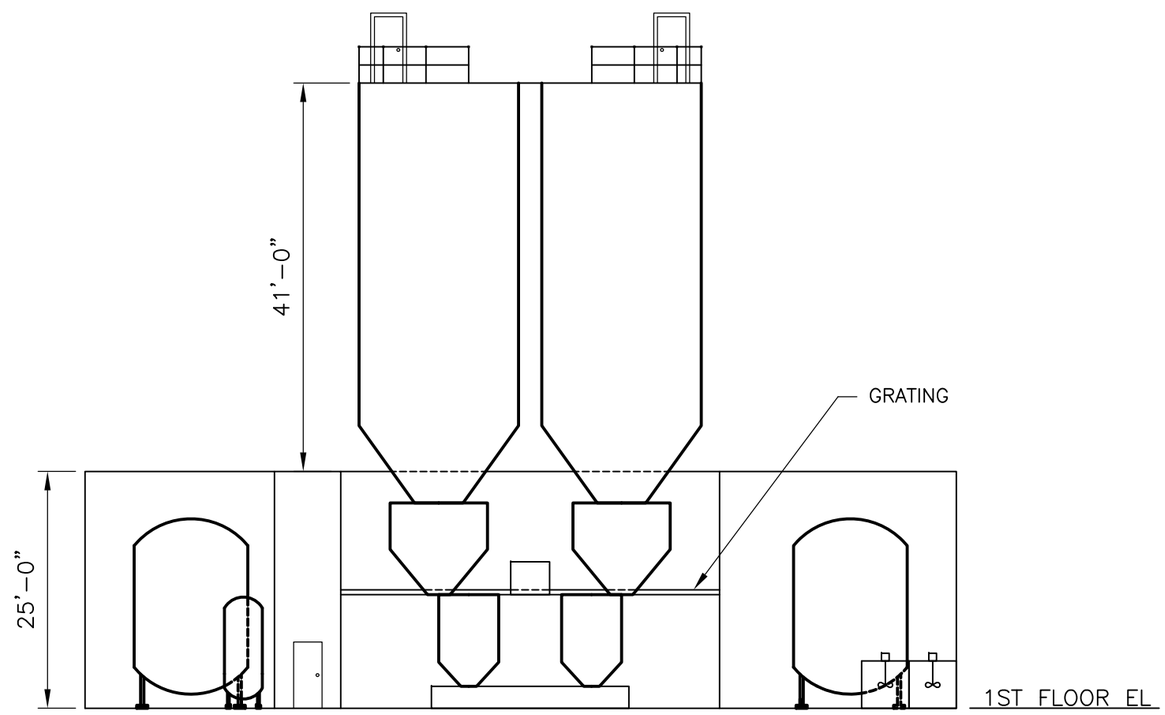

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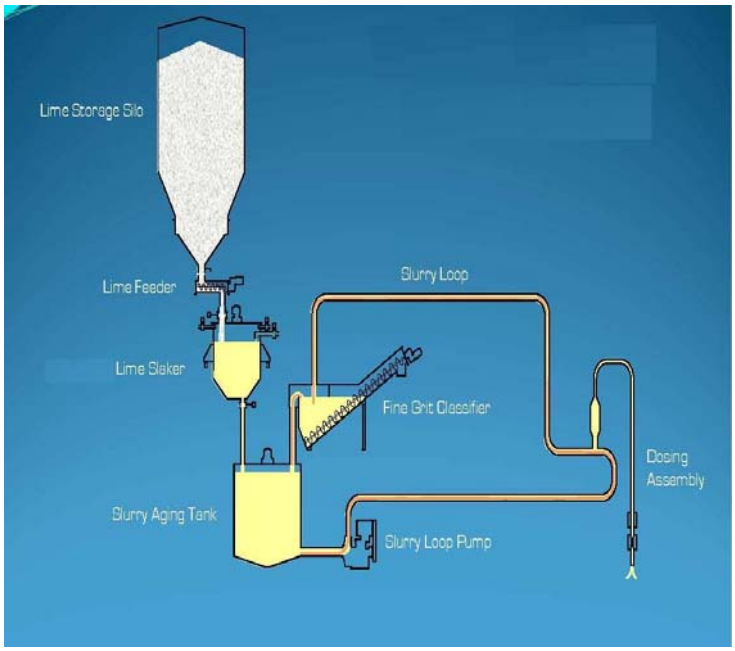
Figure 8-7
Chemical Building Plan
City of Piqua, OH.

Figure 8-8. *Lime Storage and Feed System*



8.9.1 Sodium Hypochlorite System

High concentrations of sodium hypochlorite of 12% or greater have been known to degrade over time and lose concentration within 2 weeks. Hence an alternative solution is to install on-site sodium hypochlorite generators to produce either dilute concentrations (0.8%), or concentrated product (10% - 15%) of the sodium hypochlorite solution. The sodium hypochlorite generation process requires precursors of salt and water, and an input of energy. A typical system is shown in **Figure 8-9**.

If the City does not wish to consider on-site generation for the new plant, the conversion could be facilitated in the future at which point the sodium hypochlorite room could be converted to provide space for the generation system and the salt storage tanks or the chemical building expanded to provide additional room.

Figure 8-9. *Onsite Sodium Hypochlorite Generator*

8.10 Washwater Recovery

The City's new WTP will produce residual flows during the treatment process. The liquid residual stream will include wash-water from backwashing the conventional filters and the post-filter GAC contactors. These residual flows must either be treated and disposed or recycled. The fate of the residual streams is discussed in this subsection. The purpose of the wash-water recovery system is to settle waste backwash water, recycle the settled water for reuse, and dispose of the settled solids. Two backwash holding basins will be provided with similar size and configuration and be operated in a fill and draw mode. The proposed size of the combined basins will be able to hold two sand filter backwash volumes and one GAC filter backwash volume. The wash-water recovery system also consists of a recycle pump station. The recycle pumps will be designed to prevent return flows in excess of 10% of raw water flow to the influent that might impact treatment process, but sufficient in size to keep up with the backwash flows. The design criteria for the wash-water recovery system for the selected alternative are shown in **Table 7-21** under **Section 7.**

8.10.1 Sand Filter Backwash Water

Under normal operation, the sand-filter wash-water recovery begins operation with initiation of a filter backwash using a backwash pump. The backwash pumps draw suction from the clearwell and used plant finished water to backwash the filters. Once a filter has been air scoured, backwash water is pumped through the filter to clean the media and flush trapped solids from the filter media. The filter backwash waste valve is opened during the backwash to allow the 'spent' backwash water containing these solids to flow through the backwash drain line to the two holding tanks. Once the backwash process is complete, the waste backwash water is allowed to settle out in the holding basins. Each backwash cycle lasts approximately 20 minutes.

8.10.2 GAC Backwash Water

The procedure for GAC contactor backwash is similar to that of a sand filter described as above. Under normal operation, the GAC wash-water recovery begins operation with initiation of a contactor backwash. Once a contactor has been taken offline, backwash water is pumped from the clearwells through the contactor to clean the media and flush trapped solids from the contactor media. The backwash waste valve is opened during the backwash to allow the 'spent' backwash water containing these solids to flow through the backwash drain line to the two holding tanks. Once the backwash process is complete, the waste backwash water is allowed to settle out in the holding basins. Plant facilities will be designed assuming there will be 1 contactor backwash a day, even though typically the run time on a contactor will be longer. Each backwash cycle lasts approximately 20 mins.

The 'clarified' water or the supernatant from the holding tanks will be pumped by recycle pumps back to the raw water rapid mix basins at the head of the plant at a rate that is less than 10% of the plant influent flow rate in accordance with the requirements of the FBRR . The underflow or the sludge from the bottom of the tanks will be pumped to the filter backwash facility located at the existing plant.

8.11 Optional Facilities

In addition to the facilities that are part of the process train for the selected alternative, several optional facilities were considered based on request by City staff. These include the pretreatment facility, residual solids handling facility and the garage/maintenance facility. The costs for these facilities were presented in **Section 7.** These facilities could be added by the City at a later stage than during the initial construction phase if the City wishes to do so or could be designed and bid as "Additive Alternates". These facilities are described in detail in this sub-section.

8.11.1 Pretreatment Facility

The conceptual design for the pretreatment facility includes two packaged Lamella plate settler units complete with electrical and controls and polymer feed system in a standalone building. This building will be located in close proximity to the chemical building and the flocculation/sedimentation basins. Bypass piping will be provided around the pre-treatment units to route the raw water directly to the rapid mix basins and conventional treatment. The Lamella plate settlers could be used seasonally during periods of high turbidity and could also serve to remove solids from the backwash recycle flows prior to being returned to the head of the plant. The design criteria for this facility are shown in **Table 7-1.** The building plan for this facility is shown in **Figure 8-10**.

8.11.2 Residual Solids Handling Facility

The solids handling facility includes two sludge storage tanks with mixers, two belt filter presses, a single sludge cake belt conveyor, three sludge feed pumps and a polymer dilution and feed system as shown in **Figure 8-11**. The proposed facility is a single-story building with a sub-level for the belt filter press feed pumps. If the City wishes to construct the solids handling facility during the initial construction of the plant, it will be co-located with the wash-water recovery facility which is currently included in the baseline scenario.

Solids that will be removed during the water treatment processes at the new Piqua WTP include turbidity sludge, coagulant sludge, lime sludge and PAC sludge. **Table 8-4** summarizes the estimated sludge production for the new plant. These solids are usually accumulated in the sedimentation basins during treatment and are removed from the basins. The sludge from each sedimentation basin will be pumped to the sludge holding basins located adjacent to the solids handing building.

The solids content of the combined raw sludge entering holding basins is assumed to be 3%. The sludge from the holding basins will be pumped to the two belt filter presses (BFPs) by two progressive cavity sludge feed pumps furnished with VFDs. A third pump is provided for redundancy and the pump discharge lines are interconnected so that each pump can pump sludge to either of the two BFPs. The dewatered sludge cake will be discharged into open top semi trailers via a sludge conveyance and distribution system and hauled off the site.

Table 8-4. Estimated Sludge Production

Design Criteria	Value
Max Day Flow	6.75 MGD
Turbidity Sludge	2140 lbs/day
Ferric Sulfate sludge	1380 lbs/day
PAC Sludge	80 lbs/day
Lime Sludge	21,000 lbs/day
Total Approx. Sludge Production	25,000 lbs/day

The design criteria for the solids handling facility are presented in **Table 7-12** under **Section 7** of this PER. When the plant is expanded to 9 MGD, a expanded hours of operation of the two filter presses will be required to process the maximum sludge loads. Alternatively, the City may consider pumping sludge to the off-site lagoon at the future peak loads conditions. This could allow the presses to catch up on dewatering when solids production returns to normal.

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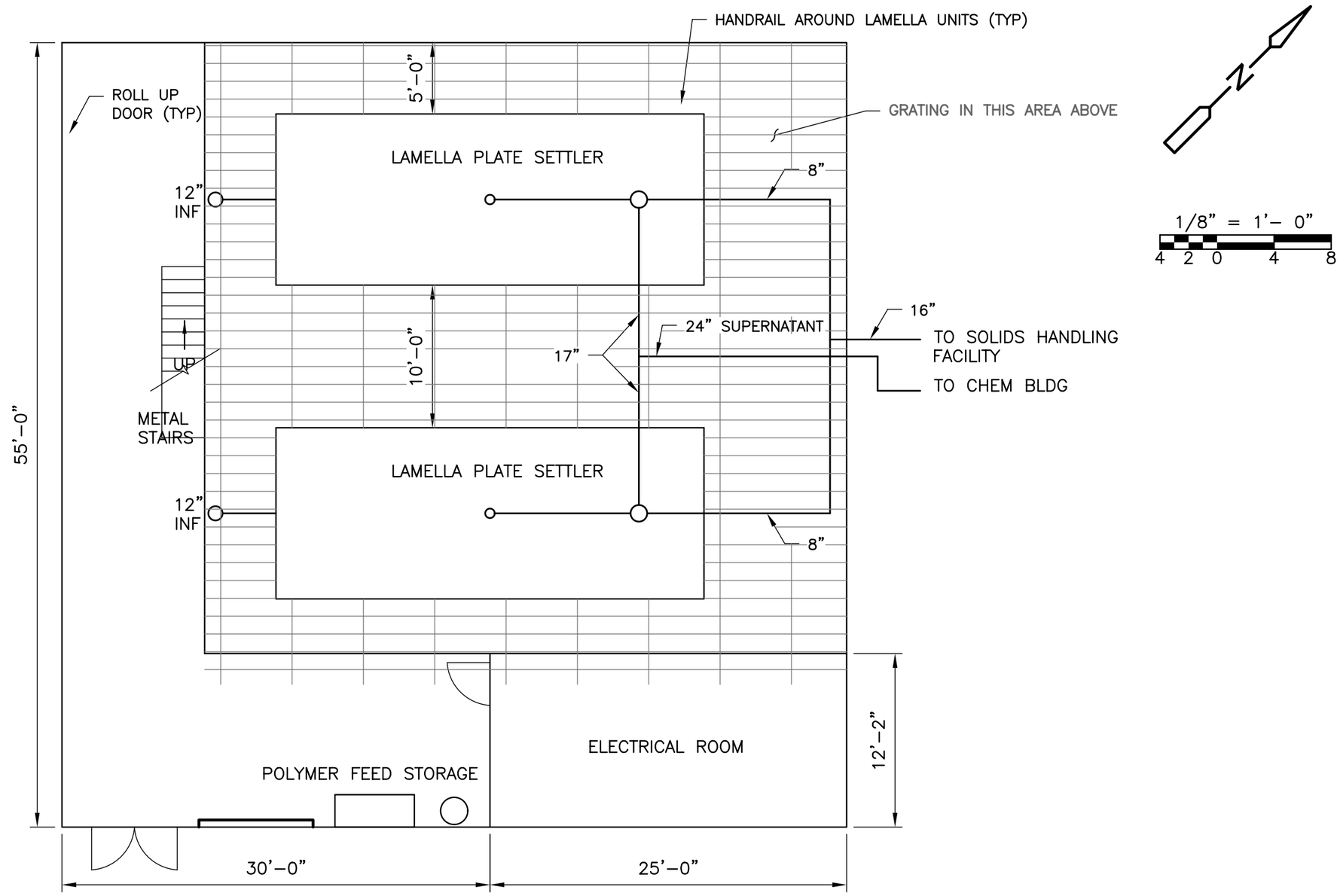

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Figure 8-10
Pretreatment Building (Optional)
City of Piqua, OH.

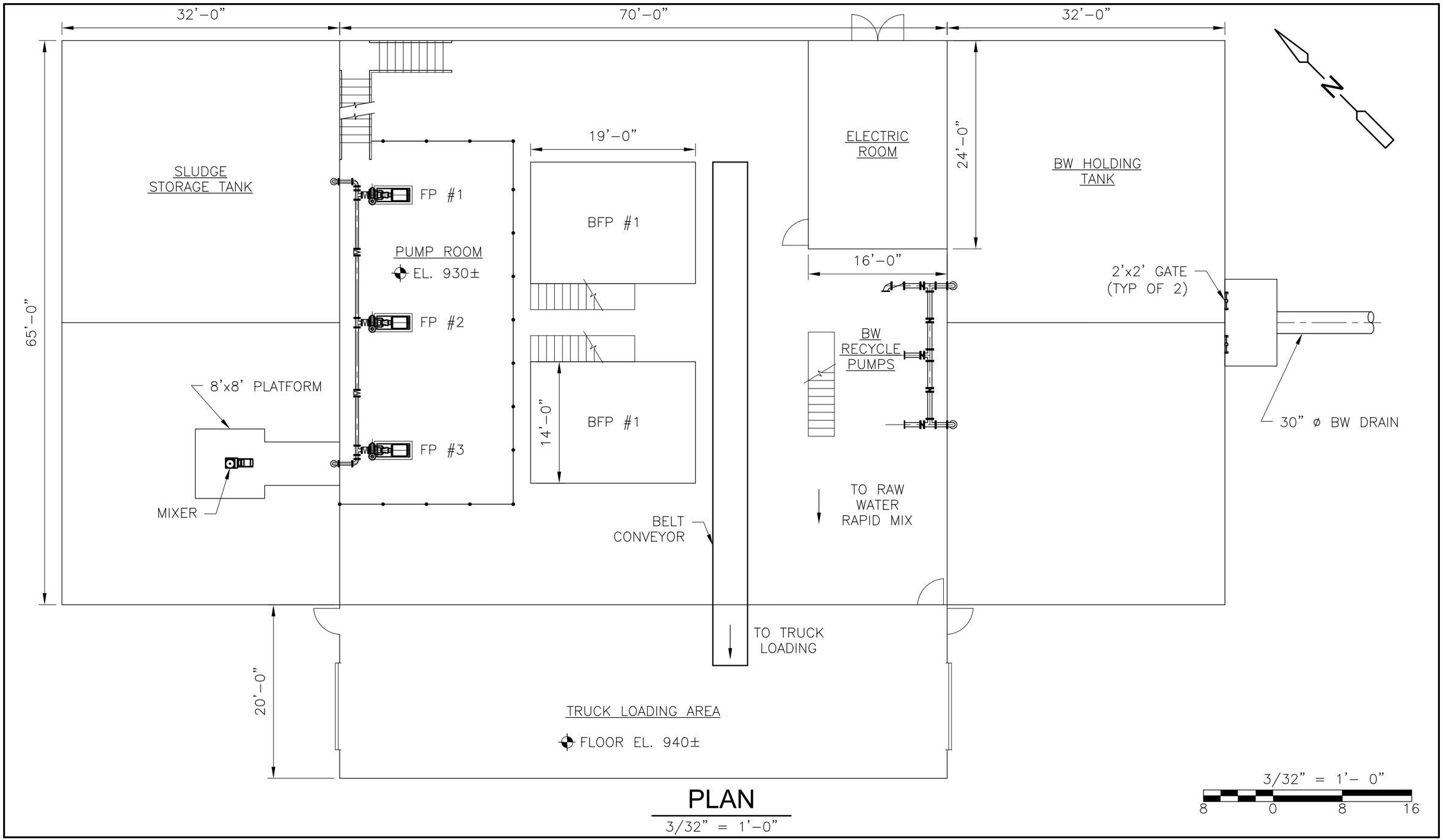

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Figure 8-11
Solids Handling Building (Optional)
City of Piqua, OH.

8.11.3 Garage/Maintenance Facility

Conceptual design for the garage/maintenance facility includes space for parking three vehicles, office, maintenance shop, and store room for parts in an enclosed building as shown in **Figure 8-12**.

8.12 Civil/Site Work

The project site is in the vicinity of SR 66 and Hardin Road outside the City of Piqua, OH and has an area of approximately 40 acres across the two parcels. The proposed raw water pump station will be located near the City's existing raw water pump station and washwater recycling facility located across SR 66 from the existing WTP. Site work associated with the new WTP will include:

- Access Roads - The main access to the site will be from SR 66 with an anticipated deceleration turn lane and acceleration lane northbound and turn lane southbound for passenger vehicles and delivery trucks. To facilitate truck traffic, the entrance road to the new WTP will have a queuing capability of at least one tractor trailer outside a security fence. Onsite turning radii will be suitable for tractor trailers with a minimum of 25 ft wide access drives. Access roads and parking areas will provide access to each building area as depicted on the site plans included in **Section 7**. Additional existing access from Hardin Road via Hardin Alternate can be maintained for service garage access and as a secondary access point; however, this road is deteriorated and should be improved if expected to convey significant traffic. The existing western portion of Hardin Alternate should to be changed to a cul-de-sac to maintain access to the two residential properties west of the new WTP, and avoid through traffic.

- Fencing - Security fencing will enclose the majority of the site near the WTP operating facilities and ancillary structures. Chain link fencing topped with barbed wire for security can be vinyl coated and or furnished with colored vinyl slats for a more aesthetic appearance. Security gates for the truck entrance will include a pedestrian gate and a motorized remote controlled slide gate to access SR 66. Access gates to Hardin Road via Hardin Alternate are expected to be manually operated, but should be used only for secondary access.

- Landscaping- Aesthetic considerations such as landscaping to soften the visibility of the WTP and security fencing can be incorporated into the site planning. Low maintenance vegetative cover can be selected for the site; however, concerns for aesthetics will need to be tempered with concerns over maintenance of the process areas (i.e., avoiding significant tree cover with potential for falling leaves collecting in the clarifiers. These issues will be further addressed during final design.

- Security systems - A card control system for the main gate access is anticipated along with a call box at the gate for visitor's convenience. A CCTV camera will provide plant personnel with the ability to verify identity prior to authorizing gate opening. Building security is anticipated to be based on card readers.

- Sanitary sewage pump station and force main - Building sanitary wastes are proposed to be handled by a submersible pump station located near SR 66. The pump station is expected to include two pumps sized for the anticipated flows from the WTP including sanitary flows and laboratory discharges. Note that tank drainage on site is anticipated to be recycled. The sanitary sewage pump station is anticipated to be sized for the wet well mounted pumps to have a third pump added and possibly replacement pumps sized to accept flows from a proposed subdivision located on the west side of SR 66. A 4-inch PVC force main will be extended from the WTP site to an existing sanitary sewer located in SR 66 approximately 6,000 feet east of the existing WTP.

Onsite storm water detention pond is expected to provide for maintaining the same peak flow from the site as pre-existing conditions. Storm water quality may be designed as part of the detention basin as either perforated underdrain pipes, wetland discharge or similar BMP. This will be determined during detailed design phase.

8.13 Environmental Permitting

The approvals required for the new WTP are expected to include:

- Plan approval from Ohio EPA for the WTP and off-site water facility improvements
- Permit to Install from Ohio EPA for the sanitary sewage pump station and force main
- Storm water construction general permit Notice of Intent (contractor to develop storm water plan for construction activities)
- Nationwide permit 12 for 401 crossing of the Swift Run Lake outfall stream and the GMR for installation of the parallel raw water transmission main from the quarry supply

If funding is sought from Ohio EPA, additional environmental reviews needed to comply with NEPA (National Environmental Protection Act) would be triggered including Fish and Wildlife Service, Ohio Historic Preservation Office (OHPO), and others. Note that if the Johnston Farm property is affected by any of the off-site construction, OHPO may be involved along with Ohio Department of Natural Resources (ODNR).

Detailed assessment of final environmental permitting requirements will be included as part of the final design.

8.14 Geotechnical Investigation

A Geotechnical Investigation study was conducted by Bowser Morner. The findings of this study were presented to the City in a report titled "Preliminary Soil Study for Proposed WTP, North Hardin Road, Piqua, Ohio", dated May 2, 2011. Ten exploratory borings were performed by Bowser Morner at locations staked by CDM at the new plant site.

The proposed site appears suitable for use as a new WTP based on previous geotechnical borings and investigation by Bowser Morner. Soils are predominately brown lean clay with sand layer. Soils are capable of being reused on-site with some dewatering to meet their optimum moisture contents.

Groundwater was encountered at a depth of 1 ft below the existing grade in the vicinity of Boring 1 and at depths of 3 ft to 16 ft below the existing grade in the remaining borings, except in Boring 5 where groundwater was not encountered. The implication is that groundwater will be encountered during the removal of unreliable soil and in the foundation and pit excavations. So it should be lowered to at least three feet below the bottom of the excavations in the sand or gravel layer using sumps and pumps during construction.

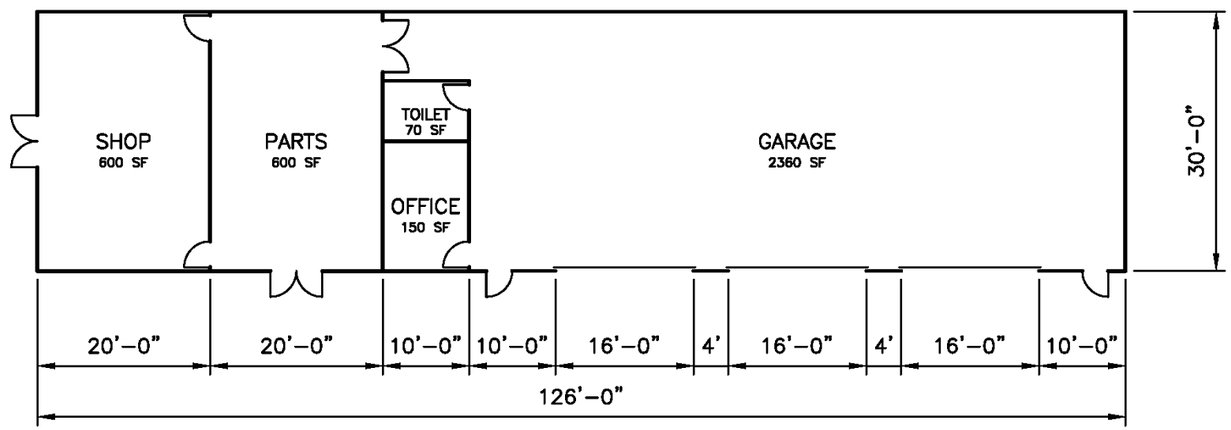

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Figure 8-12
Maintenace Garage
City of Piqua, OH.

8.15 Structural Design

The structural systems for the treatment facility will be designed using the approved codes and utilizing appropriate materials for the particular environment to which the structure is subjected. For this preliminary design, the building facilities are anticipated to be constructed from steel framed building systems supporting steel roofing on cast-in-place concrete foundations. Use of concrete masonry units (CMU) is expected to be incorporated in the final design depending on the architectural treatments selected by the City. The roof systems are anticipated to be composed of conventional roofing membrane on insulation panels.

The hydraulic (liquid containing or conveying) structures would be constructed from cast-in-place concrete. Not only would these structures be designed for the pressure due to the liquids inside the basin or tank, but also for any soil pressure resulting from any portion of the structure being below grade. Additional design measures would be taken to address any buoyancy issues as a result of a structure being founded below grade.

The structural design for the various buildings and structures will be designed in accordance with the structural design criteria set forth in the applicable sections of the following design standards (when and where they apply):

- Local building or design codes
- International Building Code
- ASCE 7-05, Minimum Design Loads for Buildings and Other Structures
- American Concrete Institute (ACI)
- ACI-318 - Building Code Requirements for Reinforced Concrete
- ACI-350 - Environmental Engineering of Concrete Structures
- American Institute of Steel Construction (AISC) with regards to the steel framing systems and systems supporting the various pieces of elevated equipment.
- American Society for Testing and Materials (ASTM) with regards to identifying the appropriate materials to be used for the design and construction.

8.16 Architectural Design

Final building exterior design, finish and color selections will be coordinated with City input and City facility standards to ensure a desirable look during the detailed design phase. National Codes will include, but not be limited to the following:

- OBC: 2007 (with 2009 updates)
- Factory Mutual for Wind Requirements
- ASTM
- National Fire Protection Association [NFPA]:
- NEC: 2011

- International Energy Conservation Code [IECC]:

All systems designed and installed with meet the International Fire Code and Chapter 9 FIRE PROTECTIVE SYSTEMS of the International Building Code for the applicable Occupancy Groups. Applicable systems used will be, but not limited to, the following:

- Automatic Sprinkler Systems
- Portable Fire Extinguishers
- Fire Alarm and Detection System[s]
- Emergency Alarm System[s]
- Smoke Control System[s]
- Smoke and Heat Vents
- Fire Command Center

Building code determination will be part of the detailed design phase.

8.17 HVAC, Plumbing and Fire Protection

The new WTP will be provided with central heating and air conditioning in the office and administrative areas of the Filter/Administration building. Electrical and control rooms will typically be provided with air conditioning.

Individual process areas will typically be provided with ventilation to remove humidity and/or heat from operating equipment. Process areas will also be fitted with unit heaters to maintain adequate (50-55 degrees F) temperatures during cold weather. Specific HVAC design has not been identified as part of this analysis, but will be included in the design phase.

Similarly, plumbing and fire protection determinations will be addressed during final design. One element of plumbing that is noted here is the requirement that all eyewash deluge shower facilities with the WTP must be provided with tempered water. Laboratory and service building sanitary wastes may require special considerations including lined piping for the laboratory wastes and oil and grease traps for the garage/service building floor drains.

8.18 Electrical Design

A new primary electric service will be required for the new WTP. As the local electric utility, the City will install new utility poles and extend overhead conductor to deliver high voltage (assumed to be 15kV class) electricity to the facility.

A preliminary electrical load was developed for each area based on preliminary equipment lists as summarized in **Table 8-5**.

Table 8-5. Estimated Electrical Load

Description	Load (kVA)
Gravel Quarry Raw Water Pumps	93
Combined Raw Water Pumps	233
Rapid Mixers	14
Flocculators	14
Clarifier Equipment	6
Carbon Dioxide Mixers	3
Backwash Supply Pumps	9
Air Scour Blowers	61
GAC Feed Pumps	196
Finished Water Pumps	280
Sludge Pumps	9
Washwater Recirculation Pumps	9
Chemical System	47
Sanitary Pumps	5
Miscellaneous	47
Lighting Loads	19
HVAC Loads	375
Undefined Loads	580
Total kVA	2000

The power requirements are estimated to require a 2,000 kVA service and would be metered at the primary voltage. Switchgear or other power distribution will be used the distribute power throughout the plant. On-site pad-mount power transformer(s) would provide 480-volt, 3-phase service for major electrical equipment such as pumps, blowers, and etc. via motor control centers, variable frequency drives, and motor starters. Additional lighting transformers and panelboards will provided for 120-V, 1-phase low voltage electric service for building needs such as receptacles, lighting, and other nominal loads.

Standby power requirements were based on operating select pieces of equipment associated with average day demand conditions as shown in the table above. Based on these loads, it is estimated that a 1,500 kW diesel engine-generator would be required to satisfy the standby power motor starting and operation requirement. The voltage of the standby engine-generator will be determined as part of detailed design of the project.

The electrical system will be designed to meet or exceed all applicable codes and standards including the following:

- Ohio Energy Code
- IECC, 2009 Edition
- American Society of Heating, Refrigerating, and Air Conditioning Engineers (ASHRAE) 90.1, 2007 Edition

- NFPA 70, 2011 Edition: National Electrical Code
- NFPA 72, 2010 Edition: National Fire Alarm Code
- NFPA 110, 2010 edition: Emergency and Standby Power Systems
- IEEE 141, 1993 Edition: Recommended Practice for Electric Power Distribution for Industrial Plants
- IEEE 142, 2007 Edition: Recommended Practice for Grounding of Industrial and Commercial Power Systems
- IEEE 242, 2001 Edition: Recommended Practice for Protection and Coordination of Industrial and Commercial Power Systems
- IEEE 446, 1995 Edition: Recommended Practice for Emergency and Standby Power Systems for Industrial and Commercial Applications
- IEEE 519, 1992 Edition: Recommended Practices and Requirements for Harmonic Control in Electrical Power Systems

Materials of construction for major electrical equipment are summarized below:

- All wiring will be copper. Wiring will be installed in a conduit system.
- Exposed conduits will be rigid aluminum except where designated to be Schedule 80 PVC in chemical areas.
- All exposed electrical fittings will be aluminum.
- Underground conduits will be PVC EB duct. Conduits outside of buildings will be installed in reinforced concrete underground duct banks.
- Spare conduits will be installed in all duct banks.
- A grounding grid system will be installed per NEC requirements to reduce the hazards of an abnormal electrical condition. During detailed design, the need for a resistance grounded electrical system will be evaluated.
- All junction boxes, pull boxes, control panels and electrical enclosures will be NEMA 4X, Type 316 stainless steel.
- All metallic supports will be 316 stainless steel strut with 316 stainless steel hardware.
- Plated copper bus will be used in motor control centers.
- Equipment will have appropriate arc flash warning labels.
- Electrical equipment will be UL listed where available.
- All transformers will have copper windings.

8.19 Instrumentation and Control Design

Instruments will be utilized to monitor process parameters such as flows, levels, pressures, and other parameters. Operator workstations will be provided in the control room and in the superintendant's office to enable operation of the WTP. Other locations may be provided with local indications or operator workstations as necessary. System architecture for the new plant is shown in **Figure 8-13**.

The new operations control room is anticipated to monitor and control the distribution system operations as well. Existing radio signals from elevated tank sites will need to be reconfigured to point towards the new plant site and antenna to receive these signals. The plant SCADA system will be able to view booster pump station operations at Hetzler and Ziegler, and storage tanks. A detailed control system will be developed as part of final design activities and include automation routines and desired control philosophy.

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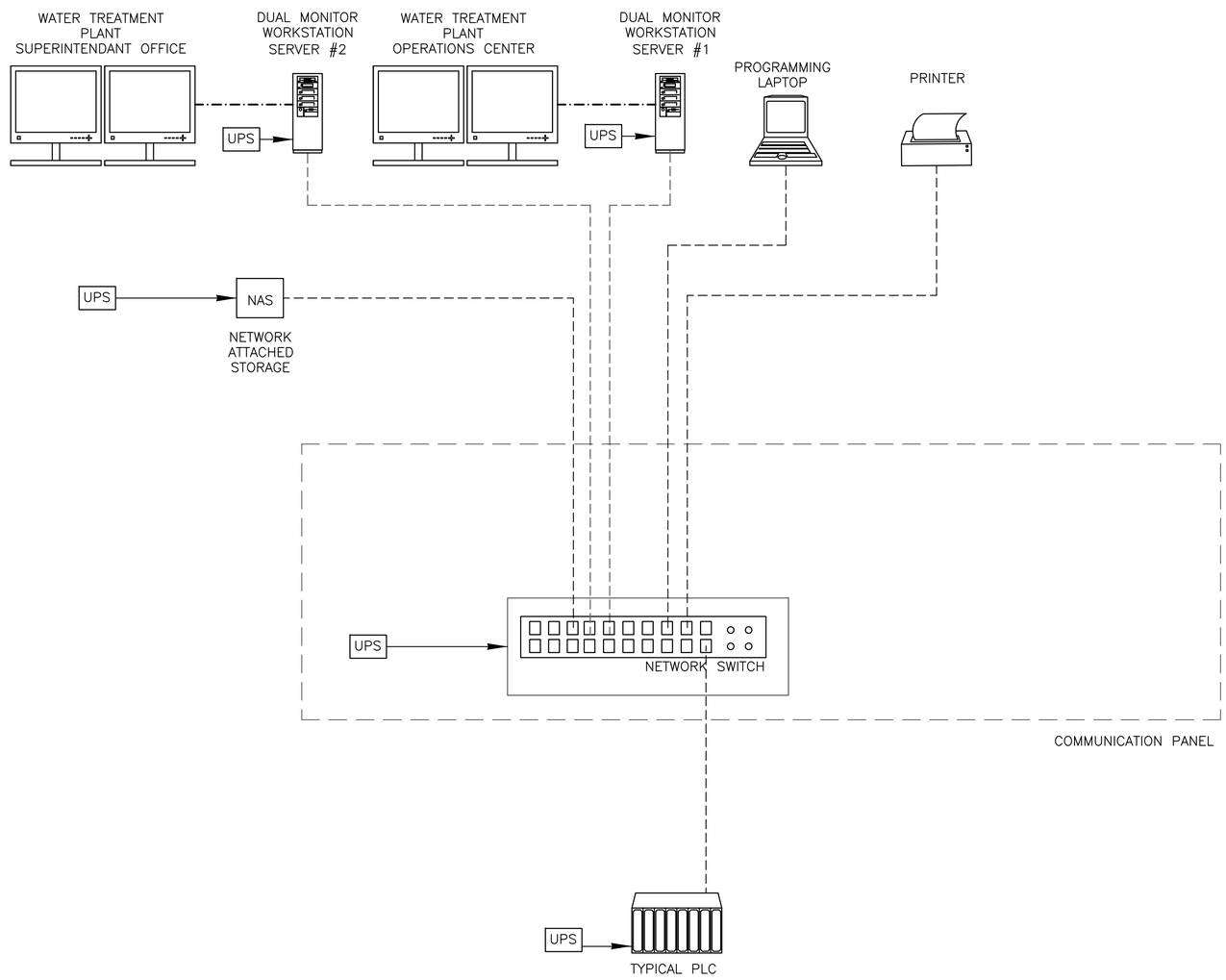

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Figure No. 8-13
System Architecture
City of Piqua, OH

Section 9
Implementation Plan

This Section describes the implementation plan and defines future tasks, schedule, and funding opportunities needed to construct the new WTP and accompanying infrastructure.

9.1 CIP Schedule

The overall project lends itself to implementation of a series of coordinated individual projects. For convenience, these projects are broken into three separate components as follow:

- **Water supply** - This includes the new raw water intakes on the GMR and Swift Run Lake, new hydraulic control structure and screens, and raw water pump station for the Swift Run Lake and GMR surface water supply.
- **Off-site piping** - This involves the interconnecting raw water piping from the raw water pump station to the new WTP site; the finished water from the new WTP to the vicinity of the existing WTP to connect to existing water transmission mains; sludge discharge line from the WTP site to the existing sludge and filter backwash facility; and new wastewater force main from the new WTP to the existing sanitary sewer south of the existing WTP.
- **WTP** - This includes the new WTP rapid mixing systems, flocculation basins, sedimentation/softening basins, recarbonation basins, conventional filters, post filter GAC contactors, chemical feed/storage facilities, clearwells, high service pumps, backwash reclamation facility, new administration building.

In reviewing these three separate components, the longest lead time for design and construction is the WTP portion. Under normal circumstances, the pipelines can be designed and bid within nine months, and construction completed within a year. The pump station and screening facilities schedule is slightly longer, but the WTP is the critical path. The following schedule specifically addresses the WTP portion, but could be applied to all three components.

Schedule for implementing the new WTP construction assumes current schedule for presenting the WTP master plan to the City Commission in January 2012 and then:

- conducting design between March 2012 and December 2012
- obtaining plan approval by Ohio EPA in February 2013 (assumes early agreement by Ohio EPA with design parameters)
- bidding for construction by March 2013

Construction of the WTP is expected to require a minimum of 18 months for substantial completion using conventional design/bid/build procedures. This places completion of the project at October 2014, which is a year after the October 1, 2013 deadline for Phase 2 DBP compliance. What is uncertain with this schedule is the ability of the existing WTP to meet the Phase 2 DBP limitations. With the first round of testing for the LRAA data collection in late 2013, and the second round in early 2014, it may be possible for those individual samples to be within the MCLs during the colder season. However, the summer 2014 samples will be more challenging for the City to meet MCLs based on the existing plant's performance, and may lead to a potential violation.

This may be acceptable to Ohio EPA since the construction project will be well underway and the City can be expected to be in full compliance with the rule within the next reporting cycle. It is unknown if this would be acceptable to the City and its customers.

If acceleration of the project schedule is desired, there are several approaches that may improve the schedule delivery after the decision is made in early 2012 such as:

- An accelerated design schedule can be implemented; however, under conventional means this can only save a few months.
- Bids can be solicited upon submittal of the plans to Ohio EPA; however, this approach risks change orders for any small items that Ohio EPA may require to be changed.
- Other approaches that may generate greater schedule compression include pre-purchasing of long lead time items such as electric gear, pumps, etc.
- Establishing an aggressive schedule can also be facilitated by offering a bonus to the contractor for early completion with a cap on the maximum bonus.

Each of these approaches has opportunity for accelerating the schedule for substantial completion and could save an estimated 3 months, possibly resulting in the project being on line by June 2014, which may be in time to positively impact the warm weather DBP sampling results.

- The final approach is procuring the facility construction through an alternative delivery method such as design/build (D/B). Recent legislation passed in Ohio has expanded the opportunities for this approach to be considered by the State and by communities such as Piqua. Detailed discussion of this approach is beyond the scope of this report; however, using an alternative method may enable the City to dramatically decrease the design and construction time such that the facility can be substantially complete to improve the opportunity to achieve compliance with the Phase 2 DBP deadlines.

9.2 Funding Options

Several funding options are available to the City to help offset or reduce the overall costs of design and construction. Shown below is a summary of funding available from public sources, including descriptions, and estimated loan terms and current interest rates (Interest rates through OWDA change at least quarterly):

- **Drinking Water Assistance Fund** (through Ohio EPA/Department of Environmental and Financial Assistance (DEFA)) under the Water Supply Revolving Loan Account (WSRLA) - 3.25% for standard rates up to 20 years.
- **Ohio Water Development Authority (OWDA)** – 4.05% for up to 30 years based on conventional funding and further reduction of 0.5% for projects slated to address health related issues (such as DBPs).

- **Ohio Public Works Commission (OPWC)** – interest rates and mix of grant and loan will vary. Project awards are competitive with other projects, and an award would require significant effort to secure, with competition from other projects needing financial support in a multi-county District. A request for participation on a single project element may be advantageous, i.e. implementation of a part of the off-site piping work could be submitted for consideration.

- **Conventional bonding** - variable rates depending on market conditions and community bond rating and possibly requiring bond insurance. Current interest rates for AAA rated, 30 year maturity municipal (general obligation) bonds found on internet listings are approximately 4% and revenue bonds may be higher. More detailed information on current bond market funding as it relates to Piqua should be obtained from the City's financial advisor.

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