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Division of Water Treatment
City of Toledo

Collins Park Water Treatment Plant
Redundant Capacity Improvements – Basin 7 and Basin 8 Projects
General Plan

DRAFT REPORT

February 2015

In Association with:

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PMG Consulting, Inc.

Robert C. Stevenson
Project Director

John L. Kane, PE
Project Manager

Kurt A. Smith, PE
Senior Engineer

William A. Barhorst, PE
Principal Engineer

Collins Park
Water Treatment Plant

Redundant Capacity Improvements

Basin 7 and Basin 8 Projects
General Plan

Prepared for:

Toledo, Ohio

Prepared by:

ARCADIS U.S., Inc.
One SeaGate
Suite 700
Toledo
Ohio 43604
Tel 419 473 1121
Fax 419 473 2108

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Acronyms and Abbreviations

AOC	Assimilable Organic Carbon
AOP	Advanced Oxidation Process
CO_2	Carbon Dioxide
CSO	Combined Sewer Overflow
cf, cu.ft.	cubic feet
DAF	Dissolved Air Flotation
DT	Detention Time
EBCT	Empty Bed Contact Time
El, Elev	Elevation
ES	Effective Size
FRP	Fiber Reinforced Plastic (fiberglass)
ft	foot, feet
FWC	Filtered Water Conduit
GAC	Granular Activated Carbon
gal	gallons
gpd	gallon per day
gpg	grains per gallons
gpm	gallons per minute
GWP	Gross Water Production
HAB	Harmful Algal Bloom
HDPE	High Density Polyethylene
HPW	Horizontal Paddle Wheel
I&C	Instrumentation and Controls
kWh	kilowatt-hour
lbs	pounds
LSPS	Low Service Pump Station
L/W	Length to width ratio
mg/L	milligram per liter
MG	Million Gallons
MGD	Million Gallons per Day
min	Minimum, minutes
NPW	Non-potable Water
OEPA	Ohio Environmental Protection Agency
O&M	Operation and Maintenance

PAC	Powdered Activated Carbon
ppm	parts per million
psi	pounds per square inch
RWC	Raw Water Conduit
RSSCT	Rapid Small Scale Column Test
SCADA	Supervisory Control and Data Acquisition
SCC	Solids Contact Clarifiers
scfm	standard cubic feet per minute
sf	square feet
SOR	Surface Overflow Rate
SWC	Settled Water Conduit
SWD	Side Water Depth
UC	Uniformity Coefficient
µg/L	Microgram per liter
US	United States
VFD	Variable Frequency Drive
VT	Vertical Turbine
WLR	Weir Loading Rate
WSEL	Water Surface Elevation
WTP	Water Treatment Plant
yr	year

1. Introduction

1.1 Project Understanding

The City of Toledo retained ARCADIS-US, Inc. to provide engineering services related to redundant capacity improvements at the Collins Park Water Treatment Plant. The ARCADIS team for the work includes the following sub consultants: Black and Veatch, Northwest Consultants, PMG Consulting, Inc., TTL, and Vision Design Group.

Under this project, the City of Toledo Division of Water stated intent was to provide 40 million gallons per day (MGD) redundant capacity to the plant by the design and construction of Basins 7 and Basins 8 additions. The engineering services under this contract include the preparation of a general plan, conceptual design, final design, bidding, construction and post construction phase services.

Work began on the Project starting in July of 2014. In August of 2014, there was a significant algae event in the Western Basin of Lake Erie which led to the issuance of a “Do-Not-Drink” order to the customers of the Toledo Water System. As a result of this event and the increasing concern with the harmful algal blooms and algal toxin issue in the Western Basin of Lake Erie, ARCADIS was requested to develop a scope of engineering services for a more in-depth study covering the implementation of advanced treatment processes for ozone and Granular Activated Carbon (GAC) to serve as additional treatment barriers in addressing these treatment concerns on a long term basis. A contract modification was executed to include the supplemental work covering the evaluation of ozone and GAC in addressing Harmful Algal Blooms (HAB)., This effort also included scope work elements for the full scale conversion of Filter 24 at the Plant to a GAC filter and for assistance with Blue Ribbon Panel in review of the alternatives for the future treatment approach in addressing HAB.

This report serves to document facilities evaluation performed under the general planning effort covering these efforts.

Presented in Sections 1.2 and 1.3 below is an outline covering the scope of services for the work to be performed under the initial Redundant Capacity Improvements – Basins 7 and Basin 8 Projects and the expansion of the General Plan effort looking at ozone and GAC as additional treatment barriers for addressing HAB.

1.2 Project Scope - Redundant Capacity Improvements – Basin 7 and Basin 8 Projects General Plan

A general description of the work to be performed under the General Plan effort is as follows:

1. Miscellaneous meetings with the City of Toledo Division of Water staff including a kickoff meeting, regulatory review workshop, and technology review workshop
2. Review of facility drawings, reports, and operating data relating to the existing treatment facilities; site visits of the facilities; discussions with Division of Water staff to establish a thorough understanding of the facilities layout, operational aspects, maintenance issues and current needs of the Collins Park Water Treatment Plant.
3. Development of a list of necessary improvements to the treatment facilities covering the existing East and West Plants, Chemical Building, Chemical Storage Building, and residuals and backwash water handling facilities.
4. Meetings with Toledo Division of Water staff and Ohio EPA Division of Drinking and Groundwater staff to discuss scope of proposed improvements, included in the general plan, applicable policies, guidelines and rules, and establish Ohio EPA review and approval schedules.
5. Preparation of a general plan document for treatment plant improvements to include the following:
    a. Future increase in rated capacity
    b. Evaluation of needed improvements to the existing facilities
    c. Future incorporation of additional treatment processes
    d. Conceptual design of Basins 7 and Basin 8 Improvements
6. Submission of the draft general plan report to Division of Water Treatment (four bound copies and one electronic copy in pdf format), meeting with staff to present the general plan, and incorporating staff review comments in preparation of the final general plan document.
7. Submission of the general plan document to Ohio EPA on the City's behalf, meeting with Ohio EPA staff to present the general plan and discuss comments, and revising the general plan as necessary and as acceptable to the City to obtain Ohio EPA approval.
8. Submission of the final general plan document to Ohio EPA and to the City (City to receive four bound copies and one electronic copy in pdf format).

9. Submission of scope and fee for the engineering services related to the design and construction of Basin 7 and Basin 8 conceptual design approach as established by the general plan.

1.3 Project Scope - HAB Treatment Alternatives

Due to the increasing concern with the harmful algal blooms and algal toxin issue in the Western Basin of Lake Erie, ARCADIS was requested to develop a scope of services for a more in-depth study covering the implementation of advanced treatment processes for Ozone and Granular Activated Carbon (GAC) to serve as additional treatment barriers in addressing these treatment concerns on a long term basis. Presented below is work effort associated with the evaluation of these unit processes and the development of design concepts for incorporation into the Collins Park WTP

1.3.1 Ozone Evaluation

Ozone is a powerful oxidant and is capable of oxidizing many organic and inorganic compounds in water. Ozone is commonly applied at one of two locations within a treatment process. In pre-ozonation, ozone is applied to the raw water prior to coagulation (softening). Typically since raw water has a higher ozone demand, a greater ozone dosage must be applied to maintain the required ozone residual. In settled water ozonation, ozone is applied following sedimentation and prior to filtration. Settled water ozone demand is typically less due to the removals of particles through the coagulation (softening) process, and as such lower ozone dosages can typically be applied.

For the Collins Park WTP, it was considered that ozonation of the settled water followed by biologically active filtration would be the most preferred treatment scheme considering the treatment provisions currently in place. The work associated with the ozone evaluation will consist of a bench-scale testing program to determine ozone demand and to establish the required ozone dosage for addressing the treatment concerns, determination of facility sizing requirements and conceptual configuration of the facilities and preliminary design layouts for incorporation into the plant. Presented below are the tasks associated with each elements of the work.

1.3.1.1 Ozone Testing and Design Basis Evaluation

- Review water quality for the last three years

- Conduct Bench scale tests on the raw and settled water to determine ozone demand and decay kinetics, establish ozone dose and contact times to achieve targeted oxidation/disinfection levels and evaluate disinfection by-product formation levels.

- Review bromate formation potential since bromate is a byproduct of ozonation of waters with high concentration of bromide. Bromate control strategies may be necessary.

- Assimilable organic carbon (AOC) concentrations will be measured during the bench tests and reviewed to assess any modifications that may be required to maintain stable water quality in the distribution system. Consider means to foster biological active filtration in the existing filters and determine recommended changes.

- Using the ozone dosage determination, establish the size and number of ozone generators

- Establish the sizing requirements for the ozone contacting approach considering the hydraulic residence time, minimizing quench following required contact time, and hydraulic losses of the process

- Review current filter backwashing process and whether existing backwashing approach with chlorinated backwash could impact biological activity.

1.3.1.2 Ozone Facilities Conceptual Design

Based on the results of the Ozone Testing and Design Basis Evaluation task above, develop a conceptual arrangement of ozone facilities to be incorporated at the Collins Park WTP. Under this task, the following work will be performed:

- Develop conceptual facility layout(s) for the ozone facilities consisting of provisions for ozone contact time and ozone generation facilities including liquid oxygen storage and liquid oxygen vaporizers.

- Review hydraulic impacts and alternatives for integrating post settling ozonation into the existing treatment process; this will include investigating possible alternatives for re-purposing existing recarbonation facilities to serve in some manner as an ozone contactor. Should it be determined that intermediate pumping is necessary, ARCADIS will develop the conceptual layout for the pumping facilities.

- Prepare overall facility arrangement drawings showing plan and section views for the ozone contacting approach and ozone generation facilities for the alternative selected
- Conduct an assessment of the plant electrical system and establish the method for supplying power to the proposed ozone facilities
- Prepare conceptual site and piping plans showing the siting of facilities and interconnecting piping between facilities.
- Prepare opinion of probable construction costs for the alternative(s) evaluation and associated operating cost for the facilities.

1.3.2 GAC Evaluation

The work associated with the GAC Evaluation will consist of a bench-scale testing program to determine the appropriate GAC type and empty bed contact time for the Collins Park WTP to achieve adequate removal of algal toxins and other organic compounds, determination of facility sizing requirements and conceptual configuration of the GAC facilities and developing preliminary design layouts for incorporation into the plant. Presented below is the task description for the various each element of work to perform the GAC evaluation.

1.3.2.1 GAC Testing and Design Basis Evaluation

Activated carbons are derived from a variety of source materials (lignite and bituminous coals, wood, coconut shells, etc.); and source material strongly influences carbon properties such as pore size distribution, internal surface area, and surface charge characteristics. These properties are further influenced by the protocols employed in manufacturing activated carbons, which are not the same for all carbon vendors. As a result, the physical and chemical features of commercially-available GACs vary considerably; meaning their capacities for removing (adsorbing) NOM and algal toxins and other organic compounds also differ. It is therefore important to not simply select a GAC based on cost and uninformative “performance” criteria such as iodine number and/or molasses index. The best approach to selecting a GAC is to include RSSCTs in the decision-making process.

It should be emphasized that RSSCTs will also play a crucial role in establishing an appropriate EBCT (or range of EBCTs) for this application and estimating the GAC replacement frequency for any of the GAC configurations that are ultimately proposed.

As previously discussed, post-filter contactors require significant capital investment. However, this initial expense (as well as yearly operating costs) can be minimized by developing an efficient operating strategy that considers partial stream treatment (i.e. treating less than 100% of the flow with GAC) as well as staggered GAC replacement schedules that maximize the time between GAC change-outs. Our analysis will carefully consider both possibilities.

The following work would be performed under this task:

- Review currently available literature on GAC effectiveness for removing algal toxins
- Perform Rapid Small-Scale Carbon Tests (RSSCTs); it is anticipated that 5 to 7 RSCCTs will be performed to evaluate the effectiveness of several commercially available GACs for removing microcystins. ARCADIS will develop the experimental design, conduct all coordination with an outside laboratory to conduct the RSCCTs and complete the associated analyses.
- Based on the RSSCT results, ARCADIS will establish the design parameters for a GAC treatment system that would effectively protect against algal toxins. We will establish the necessary empty bed contact time and evaluate methods for extending the GAC bed life while providing an effective treatment barrier for the process. We will consider implementing GAC both in the form of filter absorbers (utilizing the existing filters) and post-filter GAC contactors (where the GAC would be installed downstream of the filters).
  - The filter absorber evaluation will determine if the existing filter box arrangement can be modified to provide the required contact time for effective treatment with GAC (this is a different approach as that considered for ozone treatment with biologically active filtration). We will determine: a) what modifications can be reasonably made to the existing filters to maximize the contact time, and b) if the additional contact time is enough to achieve adequate protection against algal toxins.

1.3.2.2 GAC Reactivation Evaluation

Based on the outcome of the GAC column testing , evaluate options for re-activation of the GAC. The following work would be performed under this effort.

- Review previous reactivation studies and establish most feasible technology to utilize for the re-activation furnaces

- Gather information on the operating experience of the GAC reactivation facilities at the Greater Cincinnati Water Works (GCWW) Richard Miller WTP
- Investigate the current regulatory and permitting requirements that would apply if re-activation facilities are constructed
- Develop the approach for on-site re-activation and prepare a preliminary opinion of probable cost for the re-activation facilities.
- Prepare a technical memorandum that summarizes the reactivation assessment, and recommends whether to pursue on-site or off-site reactivation.

1.3.2.3 GAC Facilities Conceptual Design

Based on the results of the GAC Testing and Design Basis Evaluation and the GAC re-activation evaluation tasks above, a conceptual arrangement of GAC facilities to be incorporated at the Collins Park WTP will be developed.

For the post filter contactor approach, the following will be performed:

- Develop conceptual facility layout considering the use of manufactured GAC vessels (if applicable based on facility sizing determination).
- Develop conceptual facility layout considering the use of cast-in-place concrete contactors;
- Develop plant hydraulics for incorporation of the GAC facilities; ARCADIS has assumed that intermediate plant pumping will be required to supply the post GAC contactor facility.
- Prepare budgetary opinion of probably construction costs for comparison of two approaches.
- Prepare overall facility arrangement drawings showing plan and sectional views based on the selected approach, including provisions for re-activation (if selected) and auxiliary systems covering transport and backwashing of the GAC.
- Conduct an assessment of the plant electrical system and establish the method for supplying power to the proposed GAC facilities
- Prepare conceptual site and piping plans showing the facility siting and interconnecting piping between facilities.

1.3.3 Design Memorandum

Work performed on HAB treatment alternatives will be documented in design memorandums covering the evaluation of the alternatives and documenting decisions regarding the selection of the approach for incorporating ozone and GAC facilities into the Collins Park WTP. These memorandums include conceptual drawings of the facilities and costing information covering the evaluation of facilities. Memorandums covering the ozone and GAC evaluations are included as appendices to the General Plan Report. In the performance of the work, there was some additional work tasks performed looking more closely at the performance of PAC for addition to the raw water and to flocculated/settled water at the treatment plant; information on this testing is presented in the within the main body of the General Plan Report.

2. Treatment Overview

The City of Toledo Division of Water Treatment treats water from the Western Basin of Lake Erie at their Collins Park Water Treatment Plant (WTP). Raw water is withdrawn from Lake Erie through an intake crib located approximately 3 miles offshore northeast of Reno Beach. Water is conveyed by gravity from the intake crib to the Low Service Pumping Station located along the coastline through a 108-inch diameter intake conduit. The intake crib and raw water conduit were originally designed for a water demand flow of 200 MGD.

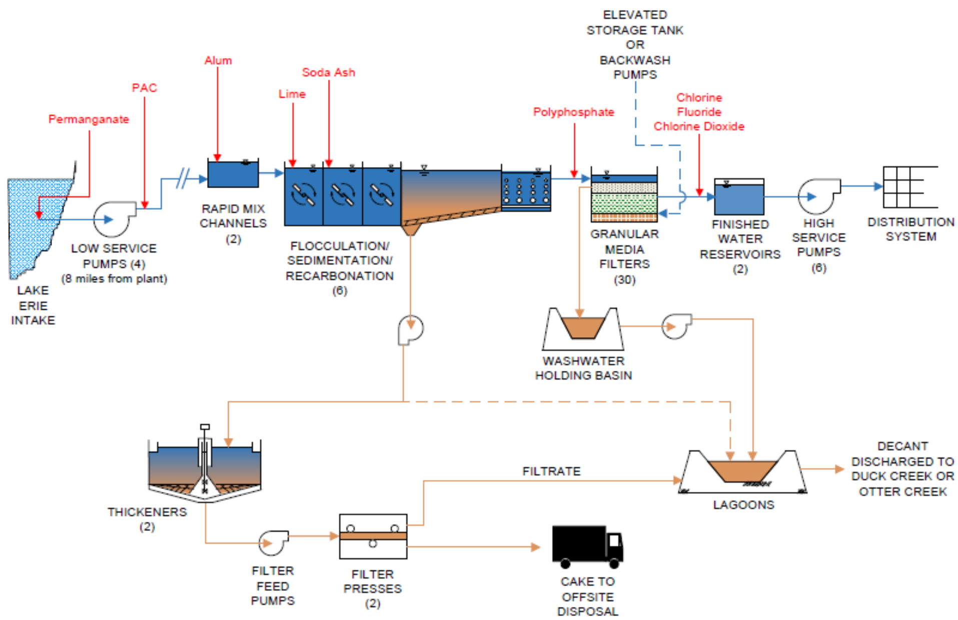
The Low Service Pump Station is equipped with traveling water screens and contains four horizontal split case centrifugal pumps for pumping raw water to the Collins Park WTP. Two raw water main, one 78-inch and one 60-inch, convey raw water approximately 8 miles from the Low Service Pump Station to the Collins Park Plant. Chemical storage and feed facilities are provided at the pump station to apply potassium permanganate out at the intake crib and powdered activated carbon at the pump station location.

The two raw water mains running from the Low Service Pump Station to the plant discharge into a flume within the Chemical Building. The unit treatment processes within the plant consist of rapid mixing, conventional flocculation/sedimentation /softening, recarbonation, and rapid gravity granular media filtration. Disinfection is achieved with free chlorine addition prior to the two 35 million gallon finished water storage reservoirs at the plant. Plant chemical storage and feed facilities are provided for the following: alum, lime, soda ash, polyphosphate, chlorine, chlorine dioxide, and sodium fluorosilicate for fluoride addition.

Residuals from the sedimentation/softening process normally discharge to thickener basins and are processed through plate and frame filter presses with final residual cake disposal hauled offsite. As an alternate means, sedimentation/softening residuals can be discharged to lagoons on a short term basis. Filter backwash water is discharged to lagoons where is it decanted and discharged to Duck Creek or Otter Creek.

A schematic diagram showing the overall treatment process for the treatment works is presented in Figure 2-1 below. The various chemical feed points at the different process facilities are depicted on the diagram.

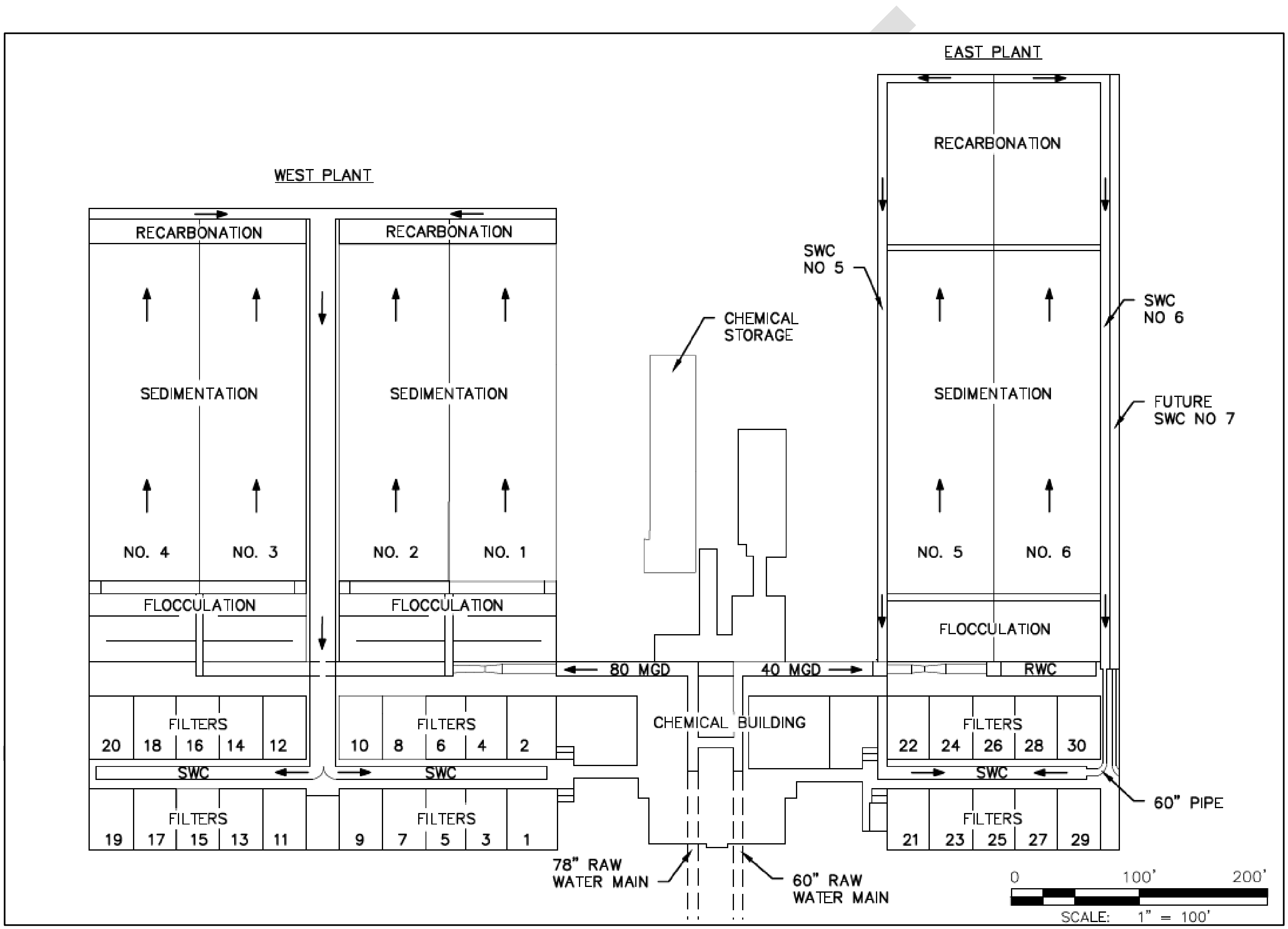
Figure 2-1: Overall Treatment Process Schematic



Detailed discussion regarding the existing process facilities at the Collins Park Plant is presented in the following section.

2.1 Existing Plant Facilities Description

A general arrangement schematic of the existing Collins Park Water Treatment Plant treatment process units is presented in Figure 2-2. As noted previously, the 78-inch and 60-inch diameter raw water mains discharge into a flume within the Chemical Building. The Chemical Building is centrally positioned between the process units for the two plants, referred to as the West Plant and the East Plant. The plants are also sometimes referred to as the 80 Plant (West Plant) and the 40 Plant (East Plant) based


EAST PLANT
WEST PLANT
RECARBONATION
RECARBONATION
RECARBONATION
SEDIMENTATION
SEDIMENTATION
SEDIMENTATION
NO. 4
NO. 3
NO. 2
NO. 1
NO. 5
NO. 6
FLOCCULATION
FLOCCULATION
FLOCCULATION
SWC NO 5
SWC NO 6
FUTURE SWC NO 7
CHEMICAL STORAGE
80 MGD
40 MGD
RWC
CHEMICAL BUILDING
FILTERS
20 18 16 14 12
FILTERS
10 8 6 4 2
FILTERS
22 24 26 28 30
SWC
SWC
SWC
FILTERS
19 17 15 13 11
FILTERS
9 7 5 3 1
FILTERS
21 23 25 27 29
60" PIPE
78" RAW WATER MAIN
60" RAW WATER MAIN
0 100' 200'
SCALE: 1" = 100'

on their nominal plant capacity rating at the time they were constructed. For discussion purposes in this report, the "West" and "East" plant terminology will be utilized throughout the remainder of this report. The West Plant facilities were designed and constructed in the late 1930's early 1940's, respectively along with the Chemical Building and the initial 35 million gallon clearwell for the plant. The East Plant was designed and constructed in the late 1950s. A second 35 million gallon clearwell was designed and constructed in the early 1970s.

2.1.1 West Plant Facilities Description

2.1.1.1 General Arrangement

Process facilities for the West Plant consists of four treatment trains for flocculation (referred to as reaction basins in the contract drawings for the facilities construction), sedimentation/softening, and recarbonation. Flow runs from south to north through the various unit processes. These treatment trains are designated as Basins 1 through 4, with Basin 1 located closest to the Chemical Building and the numbering increasing with the basins as they run from east to west. Located to the south of the flocculation basins are 20 rapid gravity media filters. There are two banks of ten filters located on north and south side of a main filter gallery running the length of the filter building in the east-west direction. There is a side gallery that runs in the east-west direction between the flocculation basins and the northern most bank of filters for the full width of the basins complex. A cross gallery running in the north-south direction is located between Basins 2 and 3 that runs the entire length of the basins and down through the Filter Building. Within the cross gallery is a common settled water conduit that conveys flow from the recarbonation basins to the rapid gravity filters. It also houses a plant drain conduit that receives drainage and overflow from the basins.

Located within the side gallery, flow from the Chemical Building Rapid Mix Channels is conveyed through a common channel/ pipeline to the two sets of basins, Basins 1 and 2 and Basins 3 and 4, where it discharges into a common splitter box which supplies the individual basins of the set. A 48-inch by 60-inch sluice gate at the influent to the flocculation basins is provided for isolation of each individual basin. For the purpose of balancing flow between sets of basins, flow to each set of basins is monitored and controlled through a venturi meter and a flow control valve on the pipeline section that serves that basin set. To balance the flow spilt to the two individual basins within the set, staff position the influent sluice gate based on operational experience gain through the years and past chemical testing performed to establish an equal flow balance between the basins.

2.1.1.2 Flocculation

The flocculation basins are arranged with a three pass serpentine flow pattern providing for three stages of flocculation. Concrete walls separate the different stages of flocculation and there is a high and low openings in the wall between for flow between the stages. . Each stage has a length of 78-feet 9-inches and a width of 17-feet 4-inches. The bottom elevation of the basins is 582.0 and the top elevation is 602.0. The current operating depth under maximum flow conditions is 599.5; this provides a basin side water depth (SWD) of 17.5 feet. An overflow conduit is provided along one side of the basin that runs the width of the three stages with an overflow weir elevation set at elevation 600.33. There is a drop box in the center of the conduit with a 24-inch diameter outlet pipeline that discharges to the plant drain conduit within the cross gallery (for Basins 2 and 3) and to a pipe drain system along the outside walls for Basin 1 and 4. The flocculation basin area is covered by a building superstructure providing access to the process equipment in the area.

The flocculation basins are currently equipped with horizontal paddlewheel style flocculator mechanisms with a common dry well drive arrangement for the basins within each set. The mechanism for each stage is equipped with six paddle assemblies and is operated by its own drive. Paddles are currently constructed of wood and the arrangement for the assemblies is the same as originally installed; repairs have been made to the mechanisms throughout the years to maintain operations.

Along the outlet wall of the third stage of flocculation is a series of 5 outlet weirs that discharges the flow from the flocculation basin into an outlet conduit that supplies the sedimentation basins. The weir openings are 7-feet 9-inches wide and are set at an elevation of 596.5. Additionally, a lower 4-foot high by 6-foot 3-inch width opening set 2-foot 6-inch above the bottom floor elevation was provided in the wall at the far end of the third stage flocculation pass.

Lime is applied at the influent end of the first stage of flocculation. Soda ash when needed for removal of additional non-carbonate hardness to meet targeted plant hardness levels is applied at the influent end of the second stage pass. Plant staff have experimented with different application points such as applying the two chemicals at the same location, but have indicated that this chemical addition approach has been the most effective. As such, the serpentine flow configuration for the flocculation basins is best suited for adding the chemicals in this fashion.

With a Water Surface Elevation (WSEL) of 599.5 and a basin flow rate of 20 MGD, the detention time for each stage of flocculation is 12.86 minutes, providing a total detention time for flocculation of 38.59 minutes. Ohio EPA has noted that the arrangement of the outlet weirs from the third stage of flocculation may result in short circuiting of the third stage pass. That said, the remaining two stages of flocculation provide 25.7 minutes of flocculation time at the 20 MGD basin flow rate. This is considered sufficient time for effective flocculation for a softening type application as is employed here.

The flocculation flow-through velocity at a basin flow rate of 20 MGD is 6.13 ft/min. With the pass arrangement as configured with the narrow width of the passes leads to higher flow-through velocities. This type of arrangement is that seen at the large softening plants within Ohio and the operating experience has shown that treatment is effective with higher velocities in these softening applications. To modifying the arrangement and reduce the flow-through velocity it would be necessary to modify the basins significantly to create a flow path across the basins in an opposite manner to that currently employed (flow distributed across the width of the basin trains). This arrangement was actually utilized when the East Plant was initially constructed. Performance of the East Plant basins was not as effective as the West Plant Basins, and as a result, the East Plant basins were modified to employ the serpentine style arrangement. Additionally, it would be more difficult to applying the soda ash following the first stage of flocculation since it would need to be distributed along the entire width of the basin train. It is our opinion that the general arrangement for the flocculation process provides effective treatment as currently configured and is the best configuration for chemical addition approach that the Water Division staff have indicated works most effectively for their process.

2.1.1.3 Sedimentation

As noted previously, flow from the flocculation basins discharges to an outlet conduit located ahead of the sedimentation basins. There is a 48-inch by 48-inch sluice gate in the dividing wall between the outlet conduits for Basins 1 and 2 and Basins 3 and 4 which by opening would create a common channel for the set of basins. Additionally, there is a 60-inch by 72-inch sluice gate located within the conduit for both Basins 2 and Basins 3 with a channel extension that connections to the settled water conduit.

Flow from the outlet conduit discharges to the sedimentation basins over a series of 5 louver gate openings equally spaced along the length of the conduit. The bottom elevation of the louver gate open is 596.0. Additionally, a lower 4-foot high by 6-foot 6-

inch opening positioned 3-feet above the channel bottom elevation of 582 was added in the channel wall discharging to the sedimentation basins. The louver gate blade which were installed during the original construction have been removed from the openings, leaving just the openings at this time. Solids accumulation occurs within the outlet conduit, however plant staff have indicated that there has been no appreciable impacts to downstream treatment processes due the solids within the conduit.

The sedimentation basins have a width of 83.5 feet and a length of 270.42 feet, providing a nominal surface area of 22,580 square feet. The basins have a top roof slab and an earthen cover providing protection from the elements. The bottom slab of the basin is at elevation 585.33 and the WSEL at a basin flow of 20 MGD is 599.5; providing a side water depth of 14.17 feet within the basin. The underside elevation of the basin top slab is 601.0 at points of maximum slab thickness.

The basins are equipped with 5 longitudinal chain and flight sludge collectors and a single chain and flight cross collector located at the influent end of the basin. The building superstructure covering the flocculation basin extends over the influent end of the sedimentation providing cover and access for the chain and flight longitudinal and cross collectors drives.

At the effluent end of the basins, a plaster baffle wall runs between the columns supporting the roof slab. The baffle top elevation is 596.5 and the total basin weir length is 62.1 feet. Flow is routed over the baffle wall and into the mixing chamber for recarbonation.

At a basin flow rate of 20 MGD, the basin detention time is 2.87 hours, the surface overflow rate (SOR) is 0.62 gpm/sf, the flow-through velocity is 1.57 fpm, and the weir loading rate is 322,000. A project meeting was held with Ohio EPA to discuss the project design parameters in relationship to the Redundant Capacity Improvements and the existing basins configuration. At this meeting information was presented on the various design parameters, and there was general agreement that the basin detention time, the surface overflow rate, and the flow-through velocity were acceptable process parameters for the softening application, but that the weir overflow rate would need to be brought into alignment with the 20,000 gpd/lf requirement.

2.1.1.4 Recarbonation and Settled Water Conduit

The recarbonation basins are situated at the effluent end of the sedimentation basins and essentially an extension of the basin structure with a similar top slab and soil cover. Each recarbonation basin has a width of 83.5 feet and an overall length of 16.58 feet, providing an overall basin area of 1,391 square feet. The basins have the same bottom elevation and flow depth as the sedimentation basins, El. 585.33 and 14.17 feet at a 20 MGD rate, respectively.

Flow from the sedimentation basin discharges over the effluent end baffle wall and is directed downward with a wood baffle positioned within the recarbonation basin and creating an under-baffle flow. The wood baffle is placed 3-feet 3-inch from the sedimentation baffle wall, creating a mixing zone compartment of 270.7 square feet and volume of 28,763 gallons. The remaining reaction zone compartment of the basin has a width of 13.33 feet, providing an area of 1110 square feet and a volume of 117,970 gallons. At a basin flow rate of 20 MGD, the associated mix zone compartment detention time is 2.1 minutes and the reaction zone compartment detention time is 8.5 minutes; this provides a total detention time of 10.6 minutes within the recarbonation basin.

Carbon Dioxide (CO_2) is applied within the mixing zone through a series of diffuser assemblies. The diffuser assemblies are positioned near the bottom of the under-flow baffle, mounted 12-inch above the basin floor. A 1-inch carbon dioxide header runs the width of the basin supplying a series of 3/4 –inch drop legs with two 3-inch diameter ceramic tube diffusers.

At the effluent end of each recarbonation basin there is a series of 5 effluent weirs that discharge into an upper outlet channel running the width of the basin. The weirs each have a length of 12-feet 5-inches long and are set at an elevation of 596.5. A common outlet gate chamber, one for Basins 1 and 2 and one for Basins 3 and 4 is provided at the end of the conduit located at the common wall separating the set of basins. Each basin is equipped with a 5-foot by 6-foot sluice gate for basin isolation purposes. The flow from the each set of basins drops down to a lower conduit running beneath the upper outlet channel for Basins 2 and 3, respectively and then rising up to meet the settled water conduit running the length of the cross gallery. The settled water conduit has a width of 17-feet and a height of 8-feet 6-inch. The bottom elevation of the channel is set at El. 593.33.

An additional six minutes of detention time prior to filtration (considering a total West Plant flow of 80 MGD) is provided through the travel time provided within the common settled water channel prior to entering the settled water channels that supply the rapid gravity filters. As such, this provides a total of 16.5 minutes of detention time prior to filtration at a West Plant flow rate of 80 MGD.

Polyphosphate is applied at the settled water conduit in the vicinity of the Flocculation Basins. Polyphosphate is fed on a continuous basis to help prevent deposition associated with the high pH water; the typical dosage is approximately 0.5 mg/L as P. A review of the alkalinity data performed as part of the 20-Year Master Plan and Needs Assessment Report showed no significant scale deposition on the filters and indicating the effective performance of the polyphosphate addition.

2.1.1.5 Filtration

As noted previously, there are 20 rapid gravity filters within the West Plant, a bank of ten filters located on each side of a main gallery running east and west, creating a north bank and a south bank of filters. Additionally, each north/south bank of filters is further separated into two banks of five filters by the cross gallery running between Flocculation/Sedimentation/Recarbonation Basins Nos. 2 and 3. The filters are number running from east to west with alternating numbers for the south and north bank (i.e. the first south eastern most filter is the No. 1 Filter, and the first north eastern most filter is the No 2 Filter).

The filters are dual cell filters with each cell measuring 14-feet wide by 49.75-feet long, providing a cell surface area of 696.5 square feet and correspondingly 1,393 square feet per filter. The cells are separated by a center gullet that is 3-feet wide. The top elevation of the walkways above the filters is 601.0 and the walkway slab is 9-inches thick; this provides a filter box depth of 10-feet 3-inches from the top elevation of the filter underdrain (590.0) to the underside of the walkways.

There are 9 concrete washwater troughs per filter cell. The concrete troughs have a width of 16-inches and slope in the direction of the center gullet; the overall depth of the troughs at the shallow end is 21-inches and at the deep end 24 ½-inches. The troughs have an adjustable weir plate with the weir crest set at an elevation of 596.5. Each filter cell is washed independent.

Each filter cell is equipped with four rotary surface wash sweeps for auxiliary scour during filter backwashing. The sweeps are supplied by a 6-inch surface wash line with a butterfly valve for isolation purposes.

The filters are equipped with a Wheeler style underdrain system with a 2-foot plenum beneath the underdrains. The current filter bed construction is presented in Table2-1 below.

Table 2-1: Existing Filter Bed Configuration

Bed Layer	Depth (inches)
Support Gravel	16
Torpedo Sand	5
Filter Sand	18
Anthracite	6

The filter sand and anthracite effective size are in the range of 0.40 to 0.50 mm and 0.85 to 1.0 respectively. The uniformity coefficient for the filter sand ranges from 1.29 to 1.44 and for the anthracite media ranges from 1.33 to 1.6. The existing media configuration provides a L/d_{10} ratio of 1,187 and is considered appropriate for effective filtration.

The top elevation of the wheeler bottoms is 590.0 and the top elevation of the media based on the current configuration is 593.75. Clearance from the top of the media to the underside of the washwater troughs varies from 7-inches at the deep end of the washwater troughs to 10 1/2 –inches at the shallow end of the washwater troughs. This would provide space for an expansion of 29 percent of the filter media depth before contacting the bottom of the washwater trough at the deep end of the trough.

The operating water depth within the filter is controlled within an elevation range of 598.50 to 599.5. This provides a water surface depth above the media in a range of 4.75 to 5.75 feet. This is considered a sufficient submergence to avoid issues with filter air binding and the filters should operate effectively to reasonable headloss levels.

Settled water is supplied to the filters through a concrete influent conduit running the length of the Main Gallery. The settled water conduit running from the Cross Gallery branches in two directions forming the influent conduits that serves the banks of filters on either side of the Cross Gallery. The influent conduit is 10-feet–4-inches wide by 6-feet deep. From the influent conduit, a 30-inch diameter influent line runs to the center gullet of the filter; the influent line is equipped with a 30-inch butterfly valve for isolation purposes.

On the effluent side of the filter box, each filter cell is isolated from the other. Each cell discharges independently through a special wall casting and Y-branch connection for connection to a 24-inch diameter backwash supply connection and a 14-inch diameter effluent line connection. The 14-inch effluent line is equipped with a 14-inch effluent isolation butterfly valve and a 14 rate controller (combined venturi flow meter and flow control butterfly). The two filter effluent lines from each filter cell combine through a 14-inch by 20-inch bull-nose tee and then discharge into an effluent conduit running beneath the filters along the outside wall of the Main Gallery lower level. Each 14-inch effluent line is equipped with a 3-inch drain line with an isolation valve.

A 24-inch diameter backwash supply header runs beneath the influent conduit in the Main Gallery. There is a 24-inch branch connection for backwash supply to each filter cell equipped with a butterfly valve for isolation purposes. Backwash water for the West plant filters is supplied through a one million gallon elevated tank on the plant site. The elevated tank is filled by tank fill pumps located in the High Service Pumping Station. Maximum backwash rates of approximately 23 gpm/ft^2 can be supplied through the current backwash system. Backwash rates are varied between summer and winter operations to appropriately control media expansion rates based on the water temperature. The summer rate is typically 11, 400 gpm (16.4 gpm/ft^2) and the winter rate is typically 9,600 gpm (13.8 gpm/ft^2).

The filter washwater drain is a 30-inch diameter line running from the center gullet and discharging into the Main Drain Conduit running beneath the floor of the Main Gallery lower level. The drain line is equipped with a 30-inch butterfly valve for isolation purposes.

Figures showing plan and sectional views of the filter piping general configuration as depicted on the original West Plant contract drawings are shown in Figures 2-3 through 2-5 below.

Figure 2-3: Typical Filter Piping Arrangement Plan

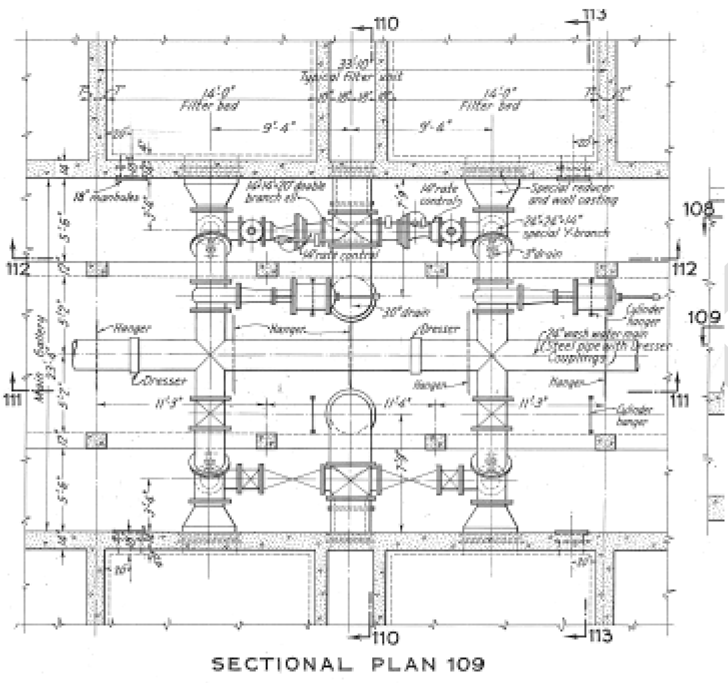


Figure 2-4: Filter Piping Sectional View 1

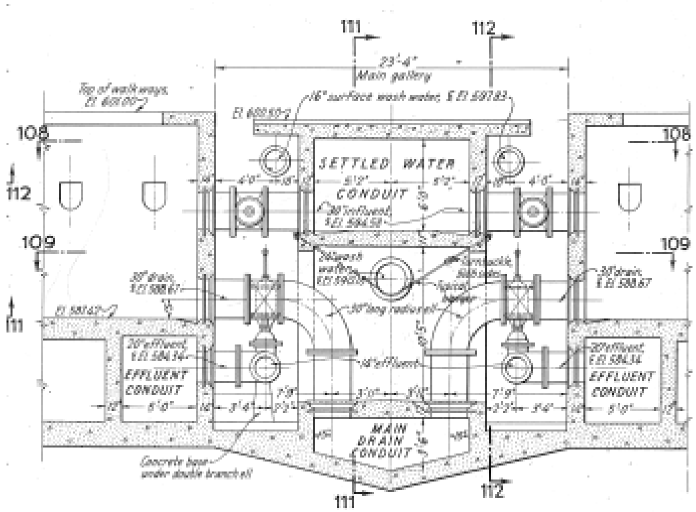
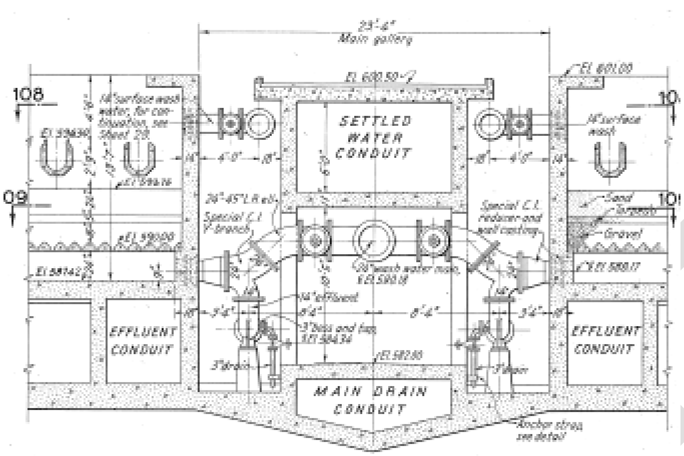


Figure 2-5: Filter Piping Sectional View 2



There is an individual a separate filtered water conduit for each bank of five filters running beneath the filters; filter groupings for each banks are as follows:

- Southeast Bank: Filters 1, 3, 5, 7 and 9
- Northeast Bank: Filters 2, 4, 6, 8, and 10
- Southwest Bank: Filters: 11, 13, 15, 17, and 19
- Northwest Bank: Filters 12, 14, 16, 18, and 20

The filtered water conduit has a typical cross section of 5-feet wide by 5-feet 7-inches high. The filtered water conduit for Southwest and Northwest Filter Banks runs the length of the filter bank, across the Cross Gallery and then turns and runs behind and parallel to the filtered water conduit for the Southeast and Southwest Banks, respectively. The two filtered water conduits for the north side banks of filters join at the far eastern end of the filter and flow into a 4-foot 6-inch square conduit and routes filtered water to the finished water reservoir. The two filtered water conduits for the south side banks of filters similarly join and flow into a second 4-foot 6-inch conduit which routes separately to the finished water reservoir.

With the West Plant operating at rate of 80 MGD, the corresponding filter loading rate with all 20 filters in service would be 2.0 gpm/ ft^2; and considering one filter off-line, the filter loading would increase to 2.1 gpm/ft^2. In the past, the West Plant has been operated at rates slightly exceeding 100 MGD. Under this flow condition, the filtration rates would be 2.5 gpm/ft^2 and 2.63 gpm/ft^2 with 20 and 19 filters in service, respectively. With the dual media filter configuration, it is considered that the plant will produce a high quality filtered water quality acceptable filter run times, which has been the case over the years. Plant staff has indicated that at times of the year, particularly during the late winter/early spring runoffs, the raw water supply from the lake will have a difficult to settle fine colloidal particle material and that the filters are able to capture the particle material but that at reduced filter loading rates. They have stated that their operating experience has shown that under these conditions, it would be difficult to operate at the higher loading rates and as such would have concerns in increasing the actual loading rate on the filters higher than is currently practice. Another factor in the consideration of increasing filter loading rates is the hydraulic constraints placed by the existing plant configuration. Based on the manner in which the filters have been constructed it would be difficult to reconfigure the filtered water effluent conduits to handle a greater flow with the headloss constraints of the system.

2.1.1.6 West Basins Summary

A summary of the design parameter for the flocculation, sedimentation/softening and recarbonation basins in the West Plant operating at a capacity of 80 MGD with all basins in operation; flow of 20 MGD per basin, a nominal basin WSEL of 599.5 is presented in Appendix A.

2.1.2 East Plant Facilities Description

2.1.2.1 General Arrangement

Process facilities for the East Plant currently consists of two treatment trains for flocculation, sedimentation/softening, and recarbonation. The general arrangement of the facilities is similar in nature to that of the West Plant with basin flow routed from the south to north direction. These treatment trains are designated as Basins 5 and 6, with Basin 5 the western most basin closest to the Chemical Building. Located to the south of the flocculation basins are 10 rapid gravity media filters; five filters on the north and five filters on the south of the Main Gallery running east to west. A side gallery is provided between the flocculation basins and the northern most bank of filters and there is a Cross Gallery along the eastern side of Basin No 6. The general flow arrangement differs from the West Plant in that the settled water is conveyed back to the filtration facilities through two settled water conduits, one running along the west wall of Basin 5 and one running along the east wall of Basin 6 within the Cross Gallery.

The design of the facilities included provision for the addition of two additional basins at a later date. As such, a second settle water conduit was included within the Cross Gallery. As in the West Plant, the Cross Gallery also contains a plant drain conduit that receives drainage and overflow from the basins.

Located within the Side Gallery, flow from the Chemical Building Rapid Mix Channels is conveyed through a common channel/pipeline to the splitter box for the two basins, discharging into a splitting box located within the Side Gallery. The pipeline section is equipped with a venturi meter and a 60-inch butterfly valve for flow control to the basins. Within the splitter box, there are two 60-inch diameter butterfly valves that provide for isolation of the flocculation basins and are used to balance the flow between the two basins. The pipeline segment and a raw water conduit for serving the future basin additions has been provided running the length of the side gallery and terminating at the eastern side of the Cross Gallery; a removable wall section was installed to accommodate the conduit extension and construction of future basins.

2.1.2.2 Flocculation

The flocculation basins were originally constructed with an influent channel spanning the east west length of the basin to distribute flow into basin, five paddlewheel type flocculator mechanism running north to south (parallel to the direction of flow) and five 36-inch by 48-inch sluice gates in the end wall of the flocculation basin discharging directly to the sedimentation basin for isolation purposes. As mentioned in the discussion concerning the West Plant, this arrangement did not perform as effectively as the serpentine arrangement in the West Plant. As such, the flocculation equipment was removed and the basins were reconfigured to provide a three stage (pass) serpentine type flow arrangement through the basins. The following modifications were made to the basins in creating this configuration:

- A new opening was cut in through the influent channel so that the flow discharged directly into the first stage of flocculation from the influent splitter box; existing channel port openings were isolated.
- Wood baffle walls were constructed to create the serpentine flow routing; opening provided at the end of baffle walls for transitions between stages.
- New paddlewheel style flocculator mechanisms for each stage with a series of five horizontal paddle assemblies running the length of the stage and with a single drive per stage.

- A series of five new upper openings were cut into the wall separating the flocculation basin and the sedimentation basin positioned along the length of the wall

- The sluice gate operators and stems were removed and the gates were left in the closed position.

The East Plant flocculation basins have an overall dimension of 83-feet 5-inch (83.41 feet) by 46-feet 11-inches (46.91 feet). Each stage has a length of 83-feet 5-inches and a width of 15 feet. This width assume approximately 1-foot wide walls for the permanent concrete baffle walls desired by the Water Department staff; currently the baffle walls are of wood construction as noted above. The top and bottom elevations of the basin are the same as the West Plant flocculation basins at 602.0 and 582.0, respectively. The current operating water surface elevation with a flow of 20 MGD provides a basin SWD of 17.5 feet. As in the West Plant, an overflow flume is provide along the outboard side of the basins that runs the width of the three stages of flocculation with an overflow weir crest of elevation set at 600.33. For Basin 5, the overflow flume discharges to overflow conduit and connects to a manhole to the west of the basin. For Basin 6, the flume discharges to a drop box outside of basin and connects to the drain conduit within the Cross Gallery. The basins are covered by a superstructure similar to the West Plant.

The lime and soda ash are applied in similar fashion that that described for the West Plant and the operational considerations are similar in nature as well.

With a WSEL of 599.5 and a basin flow rate of 20 MGD, the detention time for each stage of flocculation is approximately 11.8 minutes with the total basin detention time of 35.3 minutes. Considering the arrangement of the flocculation basin outlets, the detention time within the first two stages of flocculation is 23.6 minutes of flocculation time at the 20 MGD basin flow rate. Similar to the West Plant operations, this is considered sufficient time for effective flocculation for a softening type application as employed here.

The flocculation flow-through velocity at the basin flow rate of 20 MGD is 7.1 ft/min and is slightly higher than the flow-through velocity of 6.13 in the West Plant. It is our opinion that the general arrangement for the flocculation process provides effective treatment as currently configured and is the best configuration for the chemical addition approach that the Water Division staff has indicated works most effectively for their process.

2.1.2.3 Sedimentation

Unlike the West Plant, flow from the flocculation basins flows directly into the sedimentation basins. The sedimentation basins have a width of 83.42 feet and a length of 278.5 feet, providing a nominal surface area of 23,232 square feet. The basins have a top roof cover and earthen cover similar to the West Plant. The basin top slab and bottom elevations are the same as the West Plant, top slab of tank - 602.0, underside of top slab – 601.0 bottom slab 585.5.

The basins are equipped with two sets of five (5) longitudinal chain and flight sludge collectors and two chain and flight cross collectors, one located at the influent end of the sedimentation basin and the other located a distance of 190- feet from the influent end. Similar to the West Plant, the building superstructure covering the flocculation basin extends over the influent end of the sedimentation basin providing cover and access for influent end cross collector and the first set of longitudinal collectors. There is a separate superstructure above the sedimentation basin for access to the drives for the second sludge cross collector and set of longitudinal collectors in the basins.

At the effluent end of the basin, there is a concrete wall separating the sedimentation basin from the recarbonation basin. Along the wall is a series of five effluent weirs with a crest elevation of 596.50. The weirs are each 13-feet 5-inch long, providing a total weir length of 62.1 feet.

At a basin flow rate of 20 MGD, the basin detention time is 2.95 hours, the SOR is 0.6 gpm/ft^2 the flow-through velocity is 1.57 fpm, and the weir loading rate is 322,000. As previously noted the design parameters were considered acceptable for the softening process with the exception of the weir loading rate.

2.1.2.4 Recarbonation and Settled Water Conduits

As in the West Plant, the recarbonation basins are situated at the effluent end of the Sedimentation Basins and essentially an extension of the basin structure with a similar top slab and soil cover. Each recarbonation basin has a width of 83.29 feet and an overall length of 119-feet, providing an overall basin area of 9,910 square feet. The basins have a bottom elevation of 585.5 and with a water surface of 599.5 at a 20 MGD flow rate have a basin depth of 14- feet.

Flow from the sedimentation basin discharges over the effluent wall weirs and is directed downward with a wood baffle positioned within the recarbonation basin and creating an under-baffle flow. The wood baffle is placed 6-feet from the sedimentation outlet wall, creating a mixing zone compartment of 500 square feet and volume of 14,950 gallons. The remaining reaction zone compartment of the basin has a length of 113 feet, providing an area of 9,410 square feet and a volume of 985,600 gallons. At a basin flow rate of 20 MGD, the associated mix zone compartment detention time is 3.8 minutes and the reaction zone compartment detention time is 71 minutes; this provides a total detention time of 74.8 minutes within the recarbonation basin.

Carbon dioxide (CO_2) is applied within the mixing zone through a series of diffuser assemblies. The diffuser assemblies are positioned near the bottom of the under-flow baffle, mounted 12-inch above the basin floor. A 1-inch carbon dioxide header runs the width of the basin supplying a series of 3/4 –inch drop legs with two 3-inch diameter ceramic tube diffusers.

At the effluent end of each recarbonation basin there is a series of 5 effluent weirs that discharge into a settled water channel running the width of the basin. Each basin has four weirs that have a length of 12-feet 5-inches long; Basin 5 has a fifth weir that is 10-feet 6-inches long and Basin 6 has a fifth weir that is 5-foot 7-inches long. All of the weirs are set at an elevation of 596.5. These settled water conduits have a bottom elevation of 593.33 and have the same top elevation as the recarbonation basin. The conduit for each basins discharges to the outboard side of the basin and continue down along each side of the basins to supply the filters from each end of the complex. Along Basin No. 5, a concrete conduit runs the full length; at the raw water conduit running from the Chemical Building the settled water conduit dips down running beneath the raw water conduit and then back up before joining the settled water conduit running the length of the Filter Building Main Gallery. For Basin No. 6, the Concrete settled water conduit stops at the north side of the Side Gallery and a 60 inch diameter pipeline is routed within the Cross Gallery running the width of the north bank of filters, turning 90 degrees and connecting to the settled water conduit in the Main Gallery.

Polyphosphate is applied to the side basin settled water conduits in the vicinity of the flocculation basins in a similar fashion to the West Plant; as noted for the West Plant phosphate addition has been effective for stability control of the high pH water supplied to the filters.

2.1.2.5 Filtration

As note previously, the East Plant has 10 filters, five each side of the Main Gallery. The filters are numbered running from west to east, with the south odd numbers and the north even numbers beginning with filter 21 and 22 closest to the Chemical Building. The filtration facilities configuration is primarily the same manner as that of the West Plant. The filter effluent conduits exit the west side of the complex as opposed to the east side for the West Plant. The operational characteristics are similar to those presented for the West Plant based on the operation of the East Plant at a rate of 40 MGD.

2.1.2.6 East Plant Basins Summary

A summary of the design parameters for the flocculation, sedimentation/softening and recarbonation basins in the East Plant operating at a capacity of 40 MGD with all basins in service; flow of 20 MGD per basin and a basin WSEL of 599.5 is presented in Appendix A.

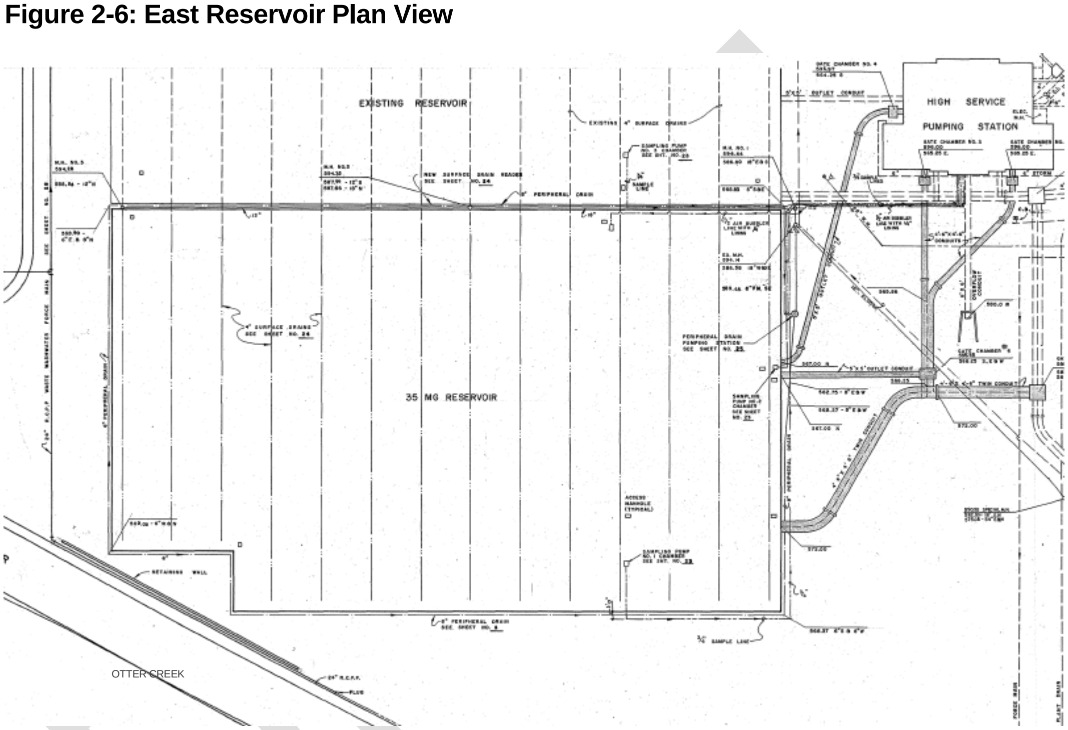
2.1.3 Finished Water Reservoirs

There are two Finished Water Reservoirs (clearwells) at the plant. Both reservoirs are situated south of the Chemical Building and High Service Pumping Station. The original 35 MG clearwell was constructed with the 1940 project. In 1976, a second 35 MG clearwell was constructed immediately east of the original clearwell, providing a total of 70 MG of finished water storage. The design of the East Reservoir and is nearly identical to the original with the exception of the modified southeast corner to avoid encroachment into Otter Creek. A plan view of the East (1976) Reservoir is shown in Figure 2-6.

Both reservoirs have a top slab elevation of 602.0, a bottom elevation of 572.0, a SWD of 19.42-ft, and an internal separated Weir Chamber. The Weir Chamber admits water to the reservoir and contains eight 6-ft long rectangular weirs that provide hydraulic control of the upstream water level. The weir crest is at elevation 590.0 and provides a maximum inlet flow of 166 MGD before the underside of the slab would be reached. A submerged 5' x 5' sluice gate in the weir chamber can outlet into the main body of the reservoir if needed, effectively bypassing the weirs.

The yard piping into the reservoirs (partially shown in Figure 2-6) allows for parallel or series operation of the reservoirs. In parallel mode, the West and East plants would deliver to the West and East Reservoirs respectively. In series operation, filtered water from both plants is sent to the East Reservoir. Transfer chambers within the reservoirs allow water to cross-over from the East to the West Reservoirs during series operation. The West Reservoir then supplies the High Service Pumping Station. Hydraulic testing has demonstrated that at flows above approximately 120 MGD it is advantageous to operate the reservoirs in parallel in order to limit the headloss from the filter plants.

Figure 2-6: East Reservoir Plan View



2.2 Plant Capacity Rating

Population projections and service area water demands were presented in the October 2011 Collins Park WTP 20-Year Master Plan and Needs Assessment Report. This information along with additional analysis looking at the plant operational data serves as the basis for establishing the plant capacity rating requirement for the 2035 planning period.

City of Toledo, like many other cities, is experiencing a shift in the concentration of population from the City to the outer limits of the metropolitan area. As stated in the 2011 Report, data obtained from the Toledo Metropolitan Area Council of

Governments (TMACOG) was utilized for the projection of future population and associated demand projections presented herein.

2.2.1 Population Projections

The Collins Park WTP supplies water directly to City customers in addition to multiple contract service areas that are located outside of the City. Water is delivered to these areas through either master meters (Maumee; Perrysburg; Sylvania; Fulton County; Wood County/Northwestern Water and Sewer District and Monroe County, MI/South County Water) or through booster pumping stations (Northwest Lucas County and Southwest Lucas County contract service areas).

Table 2-2 shows past populations of the City of Toledo and surrounding areas as well as the projected population of these areas for 2035. This data is provided by the U.S. Census, as well as from TMACOG.

Table 2-2: Population Projections

Service Area	2000 Census Population	2005 Service Population	2035 Service Population
Toledo	313,619	304,326	242,650
Wood Co./NWW&SD	16,012	16,498	19,812
Sylvania	18,670	21,200	21,200
Maumee	15,237	15,080	12,908
Monroe Co./South County Water	39,940	34,112	60,399
Perrysburg	29,197	24,621	37,744
Southwest Lucas County	45,367	39,498	61,730
Northwest Lucas County	36,452	26,453	35,943
Fulton Co.	42,084	0	15,789
Southeast Lenawee Co.	6,344	0	13,252
TOTALS	**562,922**	**481,787**	**521,428**

2.2.2 Current and Future Water Demands

A per capita water use was established for each of the services areas and used to project the average day water demand for the design year of 2035. The per capita water use factors and associated service area demands are presented in Table 2-3 As shown in the Table, the calculated 2035 average day service area demand was 88.3 MGD.

An analysis of the Collins Park WTP pumping records was performed to establish the annual maximum day and peak hour pumping from the plant. The original 2011 Master Plan and Needs Assessment Report looked at the period from 1980 until 2010; more recent years were included in this analysis. Table 2-4 presents average day, maximum day, peak hour pumping, and the ratios of maximum day to average day and maximum day to peak hour pumping for the various years.

Table 2-3: 2035 Service Area Demand Projections

Service Area	Per Capita Water Demand	Service Area Ave Day Demand (MGD)
Toledo	185	44.9
Wood County/NWW&SD	205	4.1
Sylvania	100	2.1
Maumee	155	2.0
Monroe Co./South County Water	100	6.0
Perrysburg	105	4.0
Southwest Lucas County	165	10.2
Northwest Lucas County	105	3.8
Fulton County	315	5.0
Southeast Lenawee	90	1.2
Total		**83.3**

Table 2-4: Table of Past Water Use

Year	Ave Day Flow (MGD)	Max Day Flow (MGD)	Peak Hour Flow (MGD)	Max Day to Ave Day Ratio	Ratio Max Day to Peak Hour (MGD)
1980	73	113	153	1.55	1.35
1981	71	117	136	1.65	1.16
1982	69	105	123	1.52	1.17
1983	68	114	140	1.68	1.23
1984	70	112	131	1.60	1.17
1985	72	119	146	1.65	1.23
1986	70	102	125	1.46	1.23
1987	71	117	142	1.65	1.21
1988	79	147	177	1.86	1.20
1989	73	115	133	1.58	1.16
1990	72	115	146	1.60	1.27
1991	75	129	148	1.72	1.15
1992	69	101	119	1.46	1.18
1993	72	118	158	1.64	1.34
1994	76	132	144	1.74	1.09
1995	75	120	157	1.60	1.31
1996	77	124		1.61	
1997	76	116	155	1.53	1.34
1998	81	132	160	1.63	1.21
1999	83	137	156	1.65	1.14
2000	77	109	138	1.42	1.27
2001	81	141	166	1.74	1.18
2002	82	139	170	1.70	1.22
2003	78	125	144	1.60	1.15
2004	79	117	137	1.48	1.17
2005	84	137	154	1.63	1.12
2006	78	118	144	1.51	1.22
2007	81	131	158	1.62	1.21
2008	77	111	153	1.44	1.38
2009	72	104	132	1.44	1.27
2010	73	117	147	1.60	1.26
2011	76	126		1.66	
2012	74	117		1.58	
2013	72	99.3	128	1.38	1.29
Average	**75**	**120**	**146**	**1.59**	**1.22**

For facility design purposes, the maximum day pumping for the 2035 design year was calculated using a maximum day to average day ratio of 1.6, considering the calculated average for the records analyzed was 1.59. As such, the projected maximum day demand for 2035 is 133.3 MGD. The actual maximum day rate could range higher or lower depending on weather conditions as can be seen by the range of maximum day to average day ratios of 1.42 to 1.86. The 133.3 MGD is considered a reasonable estimate for planning purposes in that utilities are seeing a somewhat declining trend over time due to the continued impacts of conservation measures (such as when more customers are replacing their clothes and dish washer with the newer water efficient models).

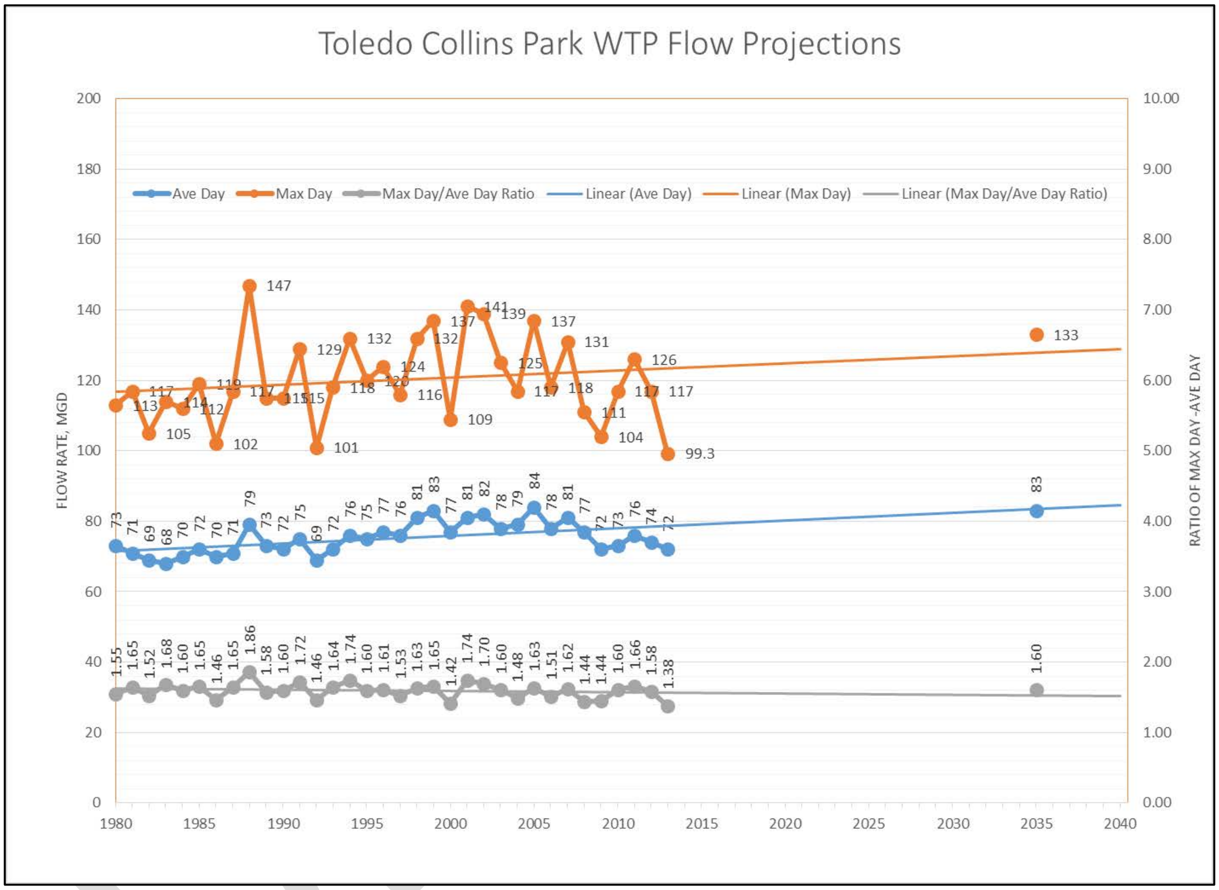
A peak hour rate of 162.6 MGD was estimated for the planning year of 2035 using a maximum day to peak hour factor of 1.22; the average peaking factor for planning records analyzed. Similarly to the maximum day demand considerations, the peak hour demand condition could range higher or lower as seen by the range of the peaking factors 1.09 to 1.38. We consider the use of the 1.22 factor a reasonable approach for the 2035 projection with the consideration being given to implementing increase storage in the northwest quadrant of the service area and the opportunities that exist to modify the operational practices to reduce the peak hour pumping requirements through better management of storage both at the plant and within the distribution system; particularly the storage associate with the supply to the suburban customers. A critical element of the suburban customers is that they should not draw water from Toledo at a rate above their peak hour demand rate. Peak hour demand should be supplied by the customers own pumps from ground storage reservoirs or from elevated tanks. In all cases, Toledo delivers water through pressure sustaining valves to a ground storage reservoir from which each suburban customer pumps into its own distribution system.

A graph of the average day and maximum day demands is presented in Figure 2-7.

The data shows that the expected growth demands over the Master Planning period is relatively flat and the overall system demands will remain fairly constant. Much of this trend is explained by the demographics of the population base of the metropolitan area.

As stated above, there will be a shift in population from the City of Toledo to the suburban communities. Some communities such as Maumee and Sylvania will remain constant or slightly decrease while most of the outlying communities increase significantly. This relative distribution of the increasing numbers in the outlying communities is directly related to the smaller numbers within the City itself. The amount of water needed for the City of Toledo will decrease and the anticipated demands for the outlying communities will increase. However, the overall amount will remain relatively constant as both areas are within the water service area. The overall volume of water needed does not significantly change over the study period.

Figure 2-7: WTP Flow Projections



2.2.3 Basis of Design of Existing Facilities

Table 2-5 is a summary of the Basin of Design for the existing facilities as required in the Ohio EPA Approved Capacity document. The basin dimensions shown in the table have been field verified during the preparation of this General Plan. The column numbering in the table is as shown in the Approved Capacity document.

Table 2-5: Existing Basis of Design

1 Component	2 Number of Units	3 Design Standards	4 Design Criteria	5 Required/ Recommended	6 Component Capacity (MGD)[5]	7 Flow Basis of Component Capacity/Ratio	8 Equivalent Maximum Day Capacity (MGD)
Rapid Mixing	2	Units	Minimum 2	Required			
		G Value	750	Recommended		1.0	
		Detention Time	Maximum 30 sec.	Recommended		1.0	
West Plant (Basins 1 through 4) – Rated at 80 MGD Total							
Flocculation	4	Units	Minimum 2	Required			
		Flow-thru Velocity	0.5-1.5 fpm	Recommended	20	1.0	
		Detention Time	Minimum 30 min.	Recommended	103	1.0	
Sedimentation	4	Units	Minimum 2	Required			
		Weir Overflow Rate	Max 20,000 gpd/ft.	Required	5	1.0	4
		Detention Time	Minimum 4 hours	Required	57 (80)[1]	1.0	57 (80)[1]
		Outlet Velocity	Maximum 0.5 fps	Required	N/A	1.0	
		Surface Load Rate	Maximum 0.75 gpm/sf	Recommended	98	1.0	
		Flow-thru Velocity	Maximum 0.5 fpm	Recommended	25	1.0	
		Length/Width Ratio	Minimum 3 : 1	Recommended	3.2		
Stabilization	4	Detention Time	Minimum 20 min.	Recommended	34	1.0	
		Mixing Time	Minimum 3 min.	Recommended	55	1.0	
		Diffuser Submergence	Minimum 7.5 ft.	Recommended			
Filtration	20	Filtration Rate	Maximum 3 gpm/sf[2]	Required	115	1.0	115
		Backwash Sources	Primary & Backup [3]	Required			
		Backwash Flow Capacity	Minimum 15 gpm/sf for 15	Required			

1 Component	2 Number of Units	3 Design Standards	4 Design Criteria	5 Required/ Recommended	6 Component Capacity (MGD)(5)	7 Flow Basis of Component Capacity/Ratio	8 Equivalent Maximum Day Capacity (MGD)
			min.				
East Plant (Basins 5 and 6) – Rated at 40 MGD Total							
Flocculation	2	Units	Minimum 2	Required			
		Flow-thru Velocity	0.5-1.5 fpm	Recommended	8.5	1.0	
		Detention Time	Minimum 30 min.	Recommended	47	1.0	
Sedimentation	2	Units	Minimum 2	Required			
		Weir Overflow Rate	Max 20,000 gpd/ft.	Required	2.5	1.0	2.5
		Detention Time	Minimum 4 hours	Required	30 (40)(1)	1.0	30 (40)(1)
		Outlet Velocity	Maximum 0.5 fps	Required	N/A	1.0	
		Surface Load Rate	Maximum 0.75 gpm/sf	Recommended	51	1.0	
		Flow-thru Velocity	Maximum 0.5 fpm	Recommended	13	1.0	
		Length/Width Ratio	Minimum 3 : 1	Recommended	3.4		
Stabilization	2	Detention Time	Minimum 20 min.	Recommended	141	1.0	
		Mixing Time	Minimum 3 min.	Recommended	50	1.0	
		Diffuser Submergence	Minimum 7.5 ft.	Recommended			
Filtration	10	Filtration Rate	Maximum 3 gpm/sf (2)	Required	55	1.0	55
		Backwash Sources	Primary & Backup 3)	Required			
		Backwash Flow Capacity	Minimum 15 gpm/sf for 15 min.	Required			
Clearwells	2	Units	Minimum of 2	Required			
		Giardia lamblia	0.5-log	Required	210	1.22	172

1 Component	2 Number of Units	3 Design Standards	4 Design Criteria	5 Required/ Recommended	6 Component Capacity (MGD)[5]	7 Flow Basis of Component Capacity/Ratio	8 Equivalent Maximum Day Capacity (MGD)
		inactivation					
		Viruses inactivation	2.0-log	Required	2,555	1.22	2,094
High Service Pumps	6	Flow Capacity	Peak Hour Demand [3]	Required	180	1.22	148

(1) At initial process review meeting held with Ohio EPA, EPA staff expressed acceptance of basin detention time less than 4 hours with addition of effluent weirs.

(2) Current rating of 3 gpm/sf. Can potentially be increased to 4 gpm/sf but subject to meeting criteria established in the Approved Capacity Document (page 25 of 38).

(3) Backup system could consist of two (2) pumps or a tank and a pump.

(4) Evaluation must consider meeting the demand with the largest pump out of service.

(5) Component capacities calculated using field measured basin dimensions and as-built drawings.

2.3 Redundant Capacity Alternatives Evaluation Overview

The City of Toledo Water Division staff has expressed the need for additional capacity to provide greater flexibility for operation and maintenance activities at the Collins Park WTP facilities. As such various alternatives were investigated to determine their viability, sizing and general configuration requirements, and associated costs. An initial workshop session was held to discuss potential alternatives for consideration in providing additional redundant capacity. In this initial session, the possible approach of eliminating the current practice for softening (to a level of approximately 80 mg/L of total hardness) was discussed and it was determined that since the softening practice had been established through the political process and a vote of the customers in the past that this softening practice should continue. Various options considering the softening approach were discussed, and it was agreed that the following three alternatives should be investigate for the possible approach to providing redundant capacity at the plant:

- Conventional approach with basins similar sized to that of the existing treatment facilities
- Solids Contact Clarification approach
- High Rate Softening

Discussion concerning the evaluation of these alternatives is presented in Section 3 below.

3. Redundant Capacity Alternatives Assessment

Redundant capacity improvements can be constructed as conventional rectangular basins using common wall construction as currently existing in the West and East Plants or by constructing stand-alone process tanks that are interconnected by buried piping. In the stand-alone configuration, softening would be performed in solids contact basins and not combined with sedimentation as is currently done. The conventional approach is described as Alternative 1 – Conventional Sedimentation/Softening and the stand-alone tank approach will be described as Alternative 2 – Solids Contact Units.

3.1 Alternative 1 - Conventional Settling/Softening

3.1.1 Basin Hydraulics

Hydraulic considerations of the design impact the flow distribution to the basins, operating levels of the basins, head loss between the basins and filters, and the head allowance for the potential ozone process.

3.1.1.1 Influent Flow Distribution

The current influent piping configuration to the basins does not provide a means to separately meter the flow into each basin. Currently the 60-inch piping, having a common venturi meter, brings flow to a pair of basins (1–2, 3–4, or 5–6). Flow is then controlled into the basins by positioning either the sluice gates or butterfly valves in the basin splitter box. Consequently the flow split between these basins cannot be accurately determined.

A future arrangement for Basins 7 and 8 inlet piping is shown in Figure 3-1. The 60-inch influent pipe would be split into parallel 42-inch inlet pipes and have a magnetic flow meter and motorized control valve for each basin. The configuration would provide metered and controlled flow into each basin. Influent piping in the existing West and East Plants are recommended to be retrofitted with the parallel 42-inch piping so that all influent conditions are identical.

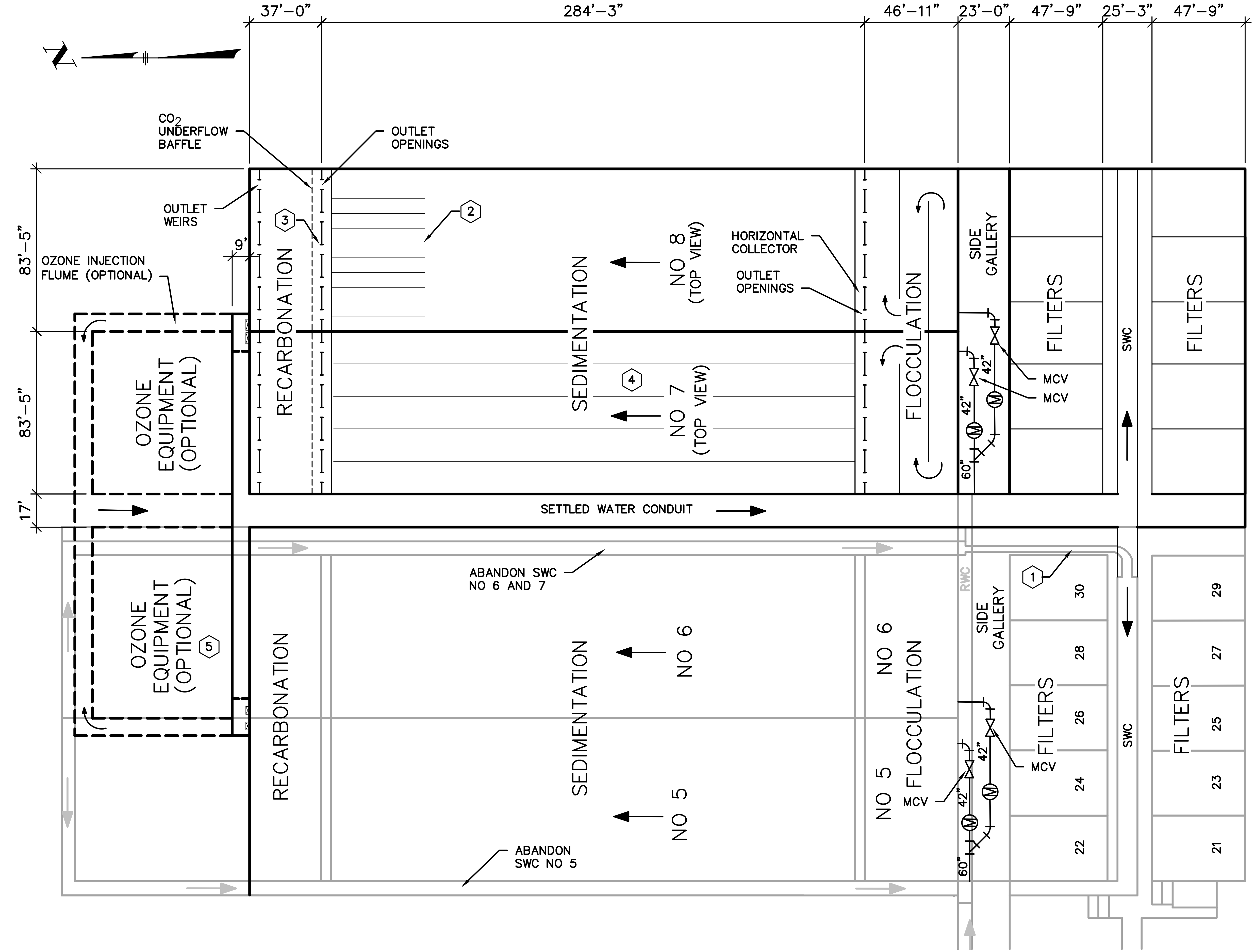
3.1.1.2 Settled Water Conduits

The East Plant was constructed with a different configuration of the Settled Water Conduits (SWCs) than at the original West Plant. The West Plant (Basins 1 through 4) utilizes a larger capacity common SWC that runs between the basins as shown in Figure 2-2. The common SWC splits into two SWCs running east and west at the filter galleries. This arrangement provides both filter galleries with consistent water quality due to the blending that occurs in the common SWC.

Currently, the East Plant utilizes two separate SWCs, one for Basin No. 5 that runs along the west side of that basin, and one for Basin No. 6 that runs along the east side of that basin as shown in Figure 2-2. These SWCs convey flow to the common SWC in the filter gallery from the east and west ends of the SWC. Also of note is a 90-ft length of 60-inch pipe that connects the Basin No. 6 SWC to the filter SWC as shown in Figure 2-2. Due to the separate SWCs for Basins 5 and 6, plant staff has observed some water quality differences at the filters. However, this existing arrangement provides operational flexibility by allowing Basins 5 and 6 and the associated settled water conduit to be removed from service independently.

If the current arrangement of SWCs at Basins 5 and 6 is repeated for Basins 7 and 8, the result would be four SWCs and potentially four separate water qualities entering the filters. Thus, it is recommended that a new common SWC be constructed at the East Plant to convey all flow from Basins 5, 6, 7, and 8 to the filters as shown in Figure 3-1. This common SWC would reduce head loss and provide for more consistent water quality received by the filters. A drawback of this approach is that a common SWC will remove all four East Plant Basins 5 through 8 from service if maintenance is required in the SWC.

Another alternative to constructing a new common SWC is to convey all flow from Basins 5 and 6 through the existing Basin No. 6 SWC (and abandon the existing Basin No. 5 SWC). Likewise, all flow from Basins 7 and 8 would be conveyed in the existing Basin No. 7 SWC. Thus, the East Plant settled water flow would be in two separate SWCs instead of four. These conduits would convey twice the originally intended design flow. This arrangement would provide the ability to isolate and shut down either Basins 5 & 6 or Basins 7 & 8 and keep the remaining two basins in service. Hydraulic calculations indicate that this scenario would have approximately 0.13 feet higher headloss than a common SWC. Water quality differences would likely be observed between the two SWCs.



NOTES:

1. EXISTING 60" SWC PIPE TO BE REMOVED
2. 10 WEIR TROUGHS, 50' LONG, 2" ORIFICES
3. CO_2 DIFFUSERS USING EXISTING ARRANGEMENT
4. 5 LONGITUDINAL SLUDGE COLLECTORS
5. EXISTING RECARBONATION AREA TO BE DEMOLISHED OR REUSED FOR OZONE PROCESS EQUIPMENT.

LEGEND:

RWC – RAW WATER CONDUIT
SWC – SETTLED WATER CONDUIT
MCV – MOTORIZED CONTROL VALVE
Ⓜ – MAGNETIC FLOW METER

ARCADIS Infrastructure · Water · Environment · Buildings

CITY OF TOLEDO
COLLINS PARK WTP
BASIN 7 & BASIN 8 PROJECTS

FUTURE EAST PLAN LAYOUT

FEB 2015

FIGURE 3–1

3.1.1.3 Basin Operating Water Levels

Currently the operating level in the filters controls the upstream basin operating levels. The filters currently operate in a band from 598.50 to a maximum water surface elevation (WSEL) of 599.50. Thus, the 599.50 filter operating level dictates the current maximum WSEL of 599.50 in the Recarbonation, Sedimentation, and the Flocculation Basins at the maximum design flow of 20 MGD per basin. Upstream of the Flocculation Basins, the frictional headloss through the conduits, valves, meters, and gates controls the maximum water level at the inlet flumes in the Chemical Building.

Recent discussions with Ohio EPA resulted in an agreement to install sedimentation basin outlet troughs having adequate weir length to comply with 10 States Standards. A preliminary design for submerged orifice finger troughs and outlet channel, as further discussed below, will require 0.25 feet of headloss. Thus, it is recommended that the operating level in the Sedimentation and Flocculation Basins is raised to a maximum WSEL of 600.00 and that the maximum operating WSEL for the filters is lowered to 599.25 in order to provide the combined 0.75 ft headloss for ozone and the troughs.

During development of the General Plan, the possible incorporation of an ozone process and the effect on plant hydraulics was studied. If it is implemented, ozone would be located immediately downstream of recarbonation. Preliminary design calculations indicate that approximately 0.5 feet of headloss is required for the inclusion of an ozone injection flume.

3.1.2 Flocculation

Basins 7 and 8 would each have a Flocculation Basin of similar design to the existing basins. The basins would utilize a three-pass (stage), serpentine, design with concrete dividing walls as shown in Figure 3-1. Each basin would have a width of 83.42 feet and an overall length of 46.92 feet matching the dimensions in Basins 5 and 6. A design water level of 600.00, as discussed above, would provide a basin depth of 18 feet and a volume of 527,000 gallons. The overflow weir should be raised 0.5 feet to 600.83 to account for the raised basin operating level.

The design parameters for Flocculation Basins 7 and 8 are presented in Table 3-1 and in Appendix A.

Table 3-1: Flocculation Basin 7 and 8 Design Parameters

Parameter	Value
Flow per Basin (MGD)	20
Number of Stages	3
Stage Length (ft)	83.42
Stage Width (ft)	14.97
Max WSEL	600
Bottom (elev)	582
SWD (ft)	18
Volume (gal)	504,527
Detention Time (min)	36.33
Flow thru Velocity (ft/min)	6.90
Basin overflow weir (elev)	600.83

Along the third pass, the flow would exit into the Sedimentation Basin through five openings. As a result, the third stage would provide varying amounts of detention time. Assuming the third stage receives half of the normal stage detention, the total detention time could be assumed to be 30.3 minutes as a worse case.

Vertical turbine (VT) flocculators and horizontal paddlewheel (HPW) flocculators were considered for flocculation equipment. As noted previously, HPW flocculators are currently being used in all basins and have been the typical flocculator style used for other large softening plants such as Columbus and Dayton. Within each pass of the basin, several horizontal paddlewheels are mounted on a common shaft and driven by a single motor and variable frequency drive (VFD). While HPW flocculators are less costly than other types, the common shaft design has a disadvantage of effectively removing a basin from service if one of the drive motors fails.

New HPW flocculators would utilize fiberglass (FRP) blades to discourage biological growth. Tapered flocculation will be achieved by incrementally reducing the G-value with each stage as shown in Table 3-2.

Table 3-2: Flocculation Basin G-Values

Stage	G-Value (sec-1)
1	50 to 60
2	35 to 45
3	20 to 30

Vertical turbine (VT) flocculators have a separate motor and VFD for each unit mounted above the water level on support platforms. By varying the power to each unit, the G-value can be tapered along each stage. A further advantage is that the loss of one VT flocculator should not compromise the entire basin performance. Preliminary vendor inquiries indicate five VT flocculators are required per stage, 15 per basin.

The relative equipment cost for HPW flocculators including a 25 percent installation factor per basin is $340,000 whereas the preliminary equipment cost for VT flocculators per basin is $400,000 not including the additional support platforms needed for the VT flocculators. Based on existing performance and lower cost of the HPW flocculators, they are recommended for Basins 7 & 8.

3.1.3 Sedimentation /Softening

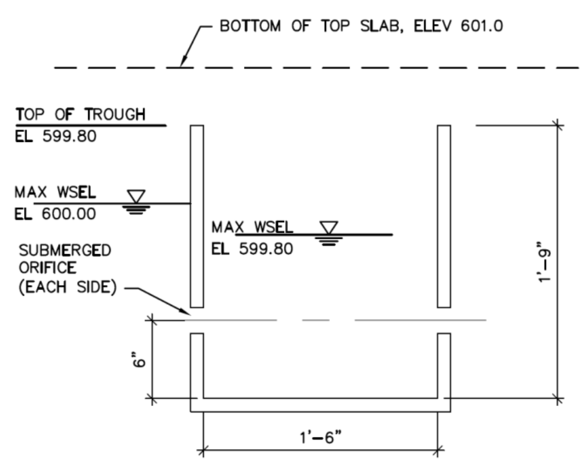
Sedimentation Basins 7 and 8 would have a width of 83.42 feet and a length of 284.25 feet, providing a nominal surface area of 23,712 square feet, consistent with Basins 5 and 6. Consistent with the existing sedimentation basin construction, the basins would have a top roof slab and an earthen cover. From the previous hydraulics discussion, the maximum WSEL would be raised to 600.00, providing a side water depth of 14.67 feet. At the 20 MGD design flow, the basin detention time is 3.12 hours and the surface overflow rate (SOR) is 0.59 gpm/sf. The basin flow-through velocity is 1.52 fpm.

Sedimentation basin effluent troughs will be provided as agreed to during a preliminary meeting with Ohio EPA to discuss the required process design parameter for the redundant capacity improvements. Troughs will be configured meeting the 10 States Standards recommended maximum weir loading rate of 20,000 gpd/lf and will utilized a submerged orifice type approach to minimize the additional headloss introduced. In each basin there would be ten 50-ft long troughs, having submerged orifices on each side, and providing an effective outlet length of 100 feet per trough as shown in Figure 3-1. The troughs would be designed for a 0.2 ft head loss through the orifices. The troughs would have a depth of 1.75 feet as shown in Figure 3-2. The troughs would outlet directly into the Recarbonation Basin.

In order to provide access to the outlet troughs for maintenance, the top slab of the Sedimentation Basin would not extend over the troughs. A superstructure of sufficient height to allow for access and maintenance to the troughs would be constructed over the troughs. Preferably the superstructure would allow natural light into the basin to facilitate maintenance but artificial lighting would likely be needed also.

Similar to the sludge collection equipment in Basins 1 through 4, Basins 7 and 8 would be equipped with 5 longitudinal chain and flight sludge collectors and utilize a single chain and flight cross collector located at the influent end of the basin. The building superstructure covering the flocculation basin would extend over the influent end of the sedimentation providing cover and access for the chain and flight longitudinal and cross collectors drives.

Figure 3-2: Proposed Submerged Orifice Outlet Trough



The design parameters for the Basin 7 and 8 Sedimentation Basins are in Table 3-3 and included in Appendix A.

Table 3-3: Sedimentation Basin 7 and 8 Design Parameters

Parameter	Value
Flow per Basin (MGD)	20
Length (ft)	284.25
Width (ft)	83.42
Max WSEL	600
Bottom (elev)	585.33
SWD (ft)	14.67
Volume (gal)	2,601,971
Det. Time (hrs)	3.12
Surface Area (sf)	23,712
SOR (gpm/sf)	0.59
Flow thru Velocity (ft/min)	1.52
Weir Length (ft, each)	100
Number of Weirs	10
Total Basin Weir Length (ft)	1000
Weir Loading Rate (gpd/lf)	20,000
L/W Ratio	3.41

3.1.4 Recarbonation

Recarbonation Basins 7 and 8 receive flow directly from the Sedimentation Basin outlet troughs as shown in Figure 3-1. Recarbonation Basins 7 and 8 would be a total of 37 feet in length and provide a total of 24 minutes of detention at the design flow of 20 MGD and design WSEL of 600.00.

An under-baffle located 6 feet downstream of the basin inlet would define the length of the mixing zone and provide 4 minutes detention in this zone. The carbon dioxide diffusers would be located within this mixing zone. The reaction zone would be 31 feet in length and provide 20 minutes detention time.

Flow would exit the Recarbonation Basin through an outlet channel having multiple submerged weir openings. If an ozone process is incorporated, the channel would direct flow to the ozone channel or tank. If ozone is not incorporated, the flow would be conveyed to the common SWC.

The design parameters for the Basin 7 and 8 Sedimentation Basins are presented in Table 3-4 and in Appendix A.

Table 3-4: Recarbonation Basin 7 and 8 Design Parameters

Parameter	Value
Flow per Basin (MGD)	20
Total Basin Length (ft)	37
Basin Width (ft)	83.42
Maximum WSEL	600
Bottom (elev)	585.50
SWD (ft)	14.5
Mixing Zone Volume (gal)	54,286
Mixing Zone Det. Time (min)	3.91
Reaction Zone Volume (gal)	280,480
Reaction Det. Time (min)	20.19
Total Basin Det. Time (min)	24.10

3.1.5 Conventional Basins 7 & 8 Probable Cost

An opinion of probable construction cost for Basins 7 and 8 was prepared as shown in Table 3-5. The Filter Building costs are not included in this estimate and are presented below separately. The Basin 7 and 8 probable cost opinion includes construction of the following components as shown on Figure 3-1:

- Side Gallery including all piping and superstructure building;
- Flocculation Basins, flocculation equipment, and superstructure over basins;
- Sedimentation Basins with new sludge collection equipment, outlet troughs, and superstructure over outlet troughs;
- Recarbonation Basins including carbon dioxide diffuser and piping;
- Settled Water Conduit in its entirety connecting with the existing East Filter Building.

The opinion of probable cost includes a 30% contingency on construction costs and other factors as listed in Table 3-5. Inflation during construction is estimated at 3% of total construction costs. Total construction cost and total project costs are rounded to the nearest $100,000.

Table 3-5: Conventional Basins 7 & 8 Probable Cost

Component	Probable Cost
Earthwork	$1,206,000
Substructures	$6,671,000
Superstructures	$1,331,000
HVAC / Plumbing	$497,000
Equipment	
Horizontal Paddlewheel Flocculators	$680,000
Sedimentation Basin Sludge Collectors	$1,160,000
Submerged Orifice Weir Troughs	$280,000
Carbon Dioxide Piping and Diffusers	$313,000
Piping, Valves, and Gates	$690,000
Electrical and I&C (10% excluding earthwork)	$706,000
General conditions and mobilization (10%)	$897,000
Subtotal	**$15,389,000**
Contingency (30%)	$4,617,000
Subtotal with Contingency	**$20,006,000**
Contractor insurance/bonding (5%)	$1,000,000
Contractor overhead/profit/general (15%)	$3,001,000
Total Construction Costs	**$24,000,000**
Construction costs inflation to construction midpoint (3%)	$720,000
Subtotal	**$24,720,000**
Engineering/Legal/Administrative (20%)	$4,944,000
Total Project Probable Cost	**$29,700,000**

3.1.6 Filtration

The Filtration Plant currently has 30 dual media filters that are proportionately divided between the two treatment trains. The West plant has 20 filters and the East plant has 10 filters. Based on the design of each plant at their time, water from the West plant can only be treated by Filters 1 through 20 in the West plant. Similarly, water from the East plant can only be treated by Filters 21 through 30 in the East plant. There is physically no means to extend the settled water conduits or the filtered water conduits between the two existing treatment plants. Therefore, additional filters are needed in the East plant to provide sufficient filtration capacity in the East plant for the planned production rates up to 80 MGD.

The proposed filters will have a similar configuration to the existing filters. Settled water from treatment will be directed from the proposed conduit structure extensions to the proposed filters.

Each existing filter is planned to be comprised of two equally sized cells – Cell A and Cell B. Each cell will measure approximately 49.75 feet long by 14 feet wide producing a surface area in each cell of 696.5 square feet (1,393 square feet per filter). The filters will be designed to permit operation at filtration rates up to 3 gpm/ft^2; this approach will match the design capabilities established for the current filters.

Each proposed filter cell will have a centrally located trough along its length sufficiently sized for maximum backwash flow rates. This trough will introduce backwash water into the underdrain system and to collect filtered water from the media during operations. The proposed underdrain system will be a low profile media-retaining lateral type system used to distribute filtered water into the bottom trough and to introduce washwater into the filter box from the backwash systems. The underdrain systems under consideration are the AWI Phoenix stainless steel underdrain system and the Robert Filter Group stainless steel “trilateral” underdrain lateral system. These systems are constructed of stainless steel materials and are capable of supplying air scour operations during backwash. Washwater for the proposed filters will be provided by the existing washwater pumps located in a section of the East Filtered Water Conduit. Maximum backwash rates up to approximately 23 gpm/ft^2 (16,000 gpm washwater pump) can be produced with the existing washwater facilities in the East plant.

An air scour system will be used for the proposed filters rather than a surface wash system. It is well established that air scour systems provide better media cleaning than surface wash systems. The air scour system will be operated from the existing control room during backwash as needed. The design air flow rate being considered is four (4) scfm per square foot (2,786 scfm total). A new 2,800 scfm blower (125 hp) will be installed in the filter gallery either within a noise-containing enclosure or within a new blower room built with the plant expansion and filters. A new 14-inch air supply header will be installed through the length of the proposed filter gallery to provide low pressure air to each proposed filter cell for air scour operations. Differential air pressure for the air scour system is planned for about 7 psig. Each filter cell will be equipped with a 10-inch distribution header that ties into the underdrain system for air scour operations. The distribution headers are proposed to be constructed in a trench below the underdrain system. A series of 1-¼-inch J pipes from the 10-inch headers will be located just below the underdrains as needed for the air supply. Electric-actuated air supply valves on the distribution headers will be controlled by the SCADA system to start and stop air scour operations as intended. The blower will cycle on and off based on the control signal to start and stop air scour operations. Air flow rates are planned to be captured and recorded by the SCADA system during air scour operations.

Fiberglass washwater troughs are proposed in each filter cell for the new filters. A total of nine troughs equally spaced along the length of the filter will collect washwater and direct it to the center gullet (drain). Washwater from each cell will flow by gravity through fiberglass washwater troughs to the center gullet. Washwater from the gullet will flow through the existing drain piping to the washwater lagoon (Lagoon D) just to the south of the Collins Park plant. Washwater currently is routed to Lagoon D for storage and subsequent recycle to the head of the existing treatment plants. The washwater troughs under consideration are 18-inches wide and 24-inches deep, running the width (14 feet) of each filter cell. The weir crest of each washwater trough will be established at the same elevation as the existing filters (approximately 596.5 feet). The washwater trough system in each cell will be capable of carrying the maximum backwash rate to the center gullet without flooding the filter.

Each proposed filter will be equipped with valves and controls as shown below. Filters will normally be operated and backwashed using computerized control systems by the Senior Control Room Operator. SCADA controls provide operational performance related to flow rates, run times, effluent turbidity, head loss, washwater flow rate, air scour flow rate, and total backwash flow. Each pair of filter cells will combine the effluent flows into a common 20-inch filtered water effluent header. The filtered water header will direct filter effluent flows into the existing Filtered Water Conduit that directs water to the existing clearwells.

- a common 30-inch influent valve,
- a common 30-inch drain valve,

- two 14-inch effluent valves and rate of flow controllers,
- one common 24-inch filtered water effluent header,
- two 24-inch washwater valves and rate of flow controllers,
- four 8-inch distribution header air scour valves, and
- two 3-inch drain valves.

The new filter floor elevation will be set to match the existing East Plant filters. Construction of each new filter bed will be dual media and the materials being considered conform to layers as shown below. The intent is to provide deeper bed filtration capabilities than the existing filters to improve overall performance and to maximize run times as well as gross water production (GWP). Torpedo sand will be placed between the new stainless steel laterals up to 1-inch above the laterals in each cell for a total depth of about 8-inches.

Table 3-6: Proposed Filter Media Specifications

Media Type	Depth	Effective Size (mm)	Uniformity Coefficient
Torpedo Sand	8-inches	0.8 to 1.0	≤1.65
Filter Sand	18-inches	0.45 to 0.55	≤1.4
Anthracite	12-inches	0.85 to 0.95	≤1.45

The designed L/D_{10} ratios and D_{90}/D_{10} ratios are 1,250 and 3.4, respectively. An 18-inch sand depth was chosen to match closely to the existing filters. The planned media design is expected to result in a mixed interface depth of about 2-inches to 6-inches and allow for filter run times in excess of 100 hours. GWP is expected to average about 10,000 or greater depending on filtration rates used. Maximum backwash rates (based on the media specifications above) are expected to be about 15 gpm/ft^2 and will provide 30% bed expansion in the summer months (warmest water temperatures). Winter backwash rate will be reduced to approximately 8 gpm/ft^2 to maintain 30% bed expansion and to minimize media loss.

Typical water levels in the settled water conduits are expected to provide about 5.9 feet more or less (approximate elevation 599.02 feet) of water above the filter media elevation (approximate elevation 593.09 feet). The media elevation provides sufficient space for 50% bed expansion between the top of the media and the bottom of the washwater troughs.

Similar computer controls systems and monitoring will be used for the new filters to match closely to the existing control systems. This plan provides continuity between the existing filters and the new filters for operations, maintenance, and filter evaluations. Record keeping and computer captured filtration information will match closely to the existing systems under SCADA monitoring and control. Similar effluent turbidimeters will be provided for each filter matching the existing equipment as closely as possible. Hach 1720E turbidimeters are being considered for each proposed filter for effluent monitoring and SCADA interfacing. Intended operating information that will be collected from each of the filters is shown below.

- Filter rate of flow each cell
- Loss of head in feet each cell
- Air scour flow rate each cell
- Backwash flow rate each cell
- Total backwash volume per wash cycle
- Filter effluent turbidity (combined for the filter) recorded every 15 minutes of operation after the initial four hours (ripening)
- Filter run time accumulated during the cycle

3.1.7 Filter Building Probable Cost

Probable costs for the proposed Filter Building are outlined in Table 3-7. The probable costs shown below are related to the Filter Building including earthwork, concrete substructure, superstructure, SWC adjacent to building, piping, valves, and equipment. The cost opinion includes a 30% contingency and other factors as listed in the table. Total construction cost and total project costs are rounded to the nearest $100,000.

Table 3-7: Filter Building Probable Costs

Component	Probable Cost
Earthwork	$417,000
Substructures	$2,880,000
Superstructures	$884,000
HVAC / Plumbing	$365,000
Equipment	
Trilateral stainless steel lateral underdrain system	$2,867,000
Fiberglass washwater trough system	$707,000
Air scour system, blower, piping, valves	$1,037,000
Piping, valves, actuators, rate of flow controllers	$7,115,000
Monitoring equipment (turbidimeters)	$65,000
Electrical and I&C (10%, excl. earthwork)	$1,517,000
General conditions and mobilization (10%)	$1,710,000
Subtotal	**$19,722,000**
Contingency (30%)	$5,917,000
Subtotal with Contingency	**$25,639,000**
Contractor insurance/bonding (5%)	$1,282,000
Contractor overhead/profit/general (15%)	$3,846,000
Total Construction Cost	**$30,800,000**
Construction costs inflation to construction midpoint (3%)	$924,000
Subtotal	**$31,724,000**
Engineering/Legal/Administrative (20%)	$6,345,000
Total Project Probable Cost	**$38,100,000**

3.1.8 Chemical Feed Provisions

Chemical feed provisions for Alternative 1, Conventional Basins 7 and 8 are discussed in Section 5.6.10.1, Future Chemical Conveyance.

3.1.9 Alternative 1 – Conventional Sedimentation / Softening Probable Costs

The opinion of probable cost for Alternative 1 - Conventional Settling/Softening is presented in Table 3-8. The opinion of probable cost is the total of the cost of Basins 7 and 8, Side Gallery, and SWC (from Table 3-5) and the cost of the Filter Building (from Table 3-7).

Table 3-8: Alternative 1 – Conventional Basin 7 and 8 Probable Cost

Component	Probable Cost
Basins 7 and 8, Side Gallery, and SWC	$29,700,000
Filter Building	$38,100,000
Total Alternative 1 Project Cost	**$67,800,000**

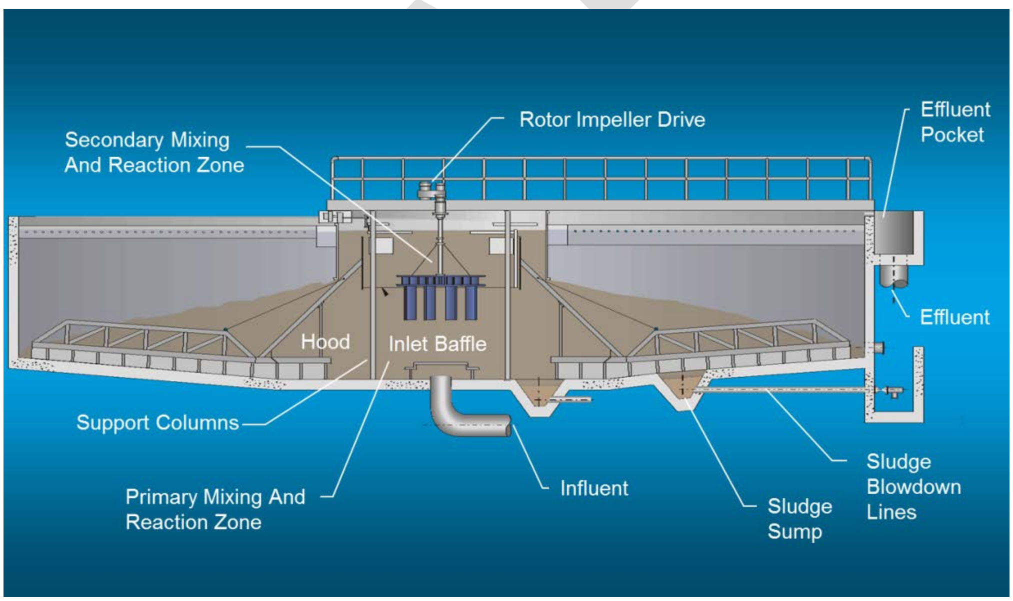
3.2 Alternative 2 – High-Rate Clarification / Softening

Newer treatment facilities designed to provide precipitative softening typically use upflow solids contact clarifiers (SCCs) because of their higher allowable hydraulic loading rates, reduced size and construction costs, and improved softening efficiencies as compared to conventional rectangular flocculation/sedimentation basins. SCCs are designed to ensure that the influent water comes into contact with previously precipitated softening solids in a central mixing/reaction zone. The intimate contact of the incoming flow with previously precipitated solids accelerates the softening reactions, thereby maximizing process efficiency and often improving solids settling characteristics. SCCs commonly used in precipitative softening applications include (1) conventional internal-recirculation units, (2) helical flow sludge-blanket units, and (3) high-rate external-recirculation units. Helical flow units are typically limited to maximum flows of approximately 7 - 8 MGD per basin, and are usually installed above-grade using steel tanks (thus requiring additional source water pumping head). Therefore, helical flow units were not considered for this evaluation. Conventional and high-rate SCCs are discussed below.

3.2.1 Conventional Solids Contact Clarifiers

Conventional internal-recirculation SCCs are typically constructed in concrete basins, and resemble conventional circular clarifiers. Internal equipment includes a center mixing/reaction zone, with a large impeller/turbine to maintain the previously-precipitated softening solids in suspension; baffles; rotating sludge collectors; and settled water collection weirs. The influent flow enters the center mixing/reaction zone, where lime (and soda ash if necessary) is added, and comes into contact with previously formed precipitates. The turbine continuously draws precipitated solids upward from the floor of the clarifier to maintain a substantial solids inventory within the mixing/reaction zone (suspended solids concentrations of 5 to 10 percent by volume within the reaction zone are typical). Turbine pumping capability is usually 5 to 10 times the unit's design maximum flow treatment capacity, and the turbine is provided with a variable-speed drive for adjusting the solids recirculation rate. The softened water/solids slurry flows either upwards over a weir and radially out into the clarifier, or downward under the reaction zone hood, where solids are separated from the softened water. The clarified/softened water then flows upwards and exits the clarifier over the effluent collection weirs. The depth and density of the residuals in the basin are controlled by periodic discharge of solids to waste and by the degree of internal recirculation provided by the mixing turbine. An example of a conventional solids contact clarifier is shown on Figure 3-3.

Figure 3-3: Solids Contact Clarifier (Infilco Degremont Accelator Type IS©)



Sidewall water depths for conventional SCCs usually vary from 16 to 22 ft and depend on basin size and the equipment manufacturer's specific requirements. The hydraulic contact time in the center mixing/reaction zone is typically 15 to 30 min, measured by the volume of water within and directly under the baffle wall. Surface loading rate in the sedimentation zone is generally measured 2 ft below the water surface, and is based on the surface area in the settling zone between the reaction zone and the basin wall. Surface loading rates for precipitative softening applications are usually in the range of 1.0 to 1.75 gpm/ft2; where coagulation for turbidity removal is also required, design surface loading rates are generally specified at or near the lower end of this range.

Conventional solids contact clarification equipment can be installed in either round or square basins. Square basins have been used where site area is limited or where the basins must be enclosed, but circular basins are generally preferred due to concerns regarding ability to effectively remove settled solids from the corners of square basins. (While provision of corner sweeps at the ends of circular sludge scraper mechanisms can enhance ability to remove settled solids from basin corners, corner sweeps can be problematic with regard to maintenance requirements, and may not provide reliable removal of solids from corners of basins larger than 90 to 100 feet square.)

Based on widespread use over more than 50 years of application, SCCs are generally regarded as the "industry standard" for precipitative softening, and Ohio EPA has confirmed their acceptability for softening applications within Ohio. While SCC equipment is available from several manufacturers, the most commonly specified/constructed SCCs in use today are as follows:

- Infilco Degremont Accelator Type IS©
  - More than 1,000 units installed
  - 45 units installed in Ohio at 37 municipal water treatment facilities
- WesTech
  - *Contact Clarifier*™
    - More than 450 units installed (majority in industrial applications)
  - *Contraflo*®
    - Former Siemens / General Filter design
    - More than 1,000 units installed (majority in municipal applications)
- Ovivo USA *HRC Reactor-Clarifier*™
  - Former Eimco Water Technologies design
- Tonka Equipment Company *RotaClear*™

While SCCs can provide significant advantages over conventional rectangular softening basins with regard to process stability/efficiency and to reduced footprint area requirements, they may also require more routine operator attention for monitoring and control of solids concentrations within the basins, particularly during initial startup and operation following periods of inactivity. However, several utilities report that once operators become familiar with SCC operation, routine O&M is no greater than, and actually may be less than for conventional rectangular flocculation/sedimentation basins used for precipitative softening. A comparison of operational parameters related to use of SCCs, based on input from several utilities which operate both conventional basins and SCCs in parallel, is presented in Table 3-7.

It has been reported in literature that SCCs may not respond well to start/stop operation (for example, if basins are routinely removed from service during nighttime hours in response to reduced system demands). However, operators with extensive experience in operating SCC units report that if the mixing turbines remain in service to maintain solids in suspension within the reaction zone during periods when the units are not producing settled/softened water, negative impacts associated with start-stop operation can be minimized or eliminated, and time after startup required to produce high-quality settled water is minimized.

Current Ohio EPA design requirements for conventional solids contact clarifiers, as specified in section VII.D.1 (“Conventional Surface WTPs”) of the March 2010 “Approved Capacity” document, are summarized in Table 3-9.

Table 3-9: Comparison of O&M Parameters for SCCs vs. Conventional Softening Basins*

Parameter	SCC vs. Conventional Basins
Chemical (lime) Utilization	No Definitive Differences
Maintenance Requirements	SCC Comparable or Somewhat Less
Settled Turbidity	SCC Comparable or Slightly Better
Routine Operator Attention Required	Less Required for SCC
Process Stability / Consistency	SCC Superior
Reaction to Changes in Raw Water Quality	SCC Superior
Impact of Temporary Lime Feed Interruption	SCC Clearly Superior
*From conversations with utilities utilizing both conventional rectangular floc/sed basins and SCCs for precipitative softening.	

Table 3-8: Ohio EPA Design Criteria for Solids Contact Clarifiers*

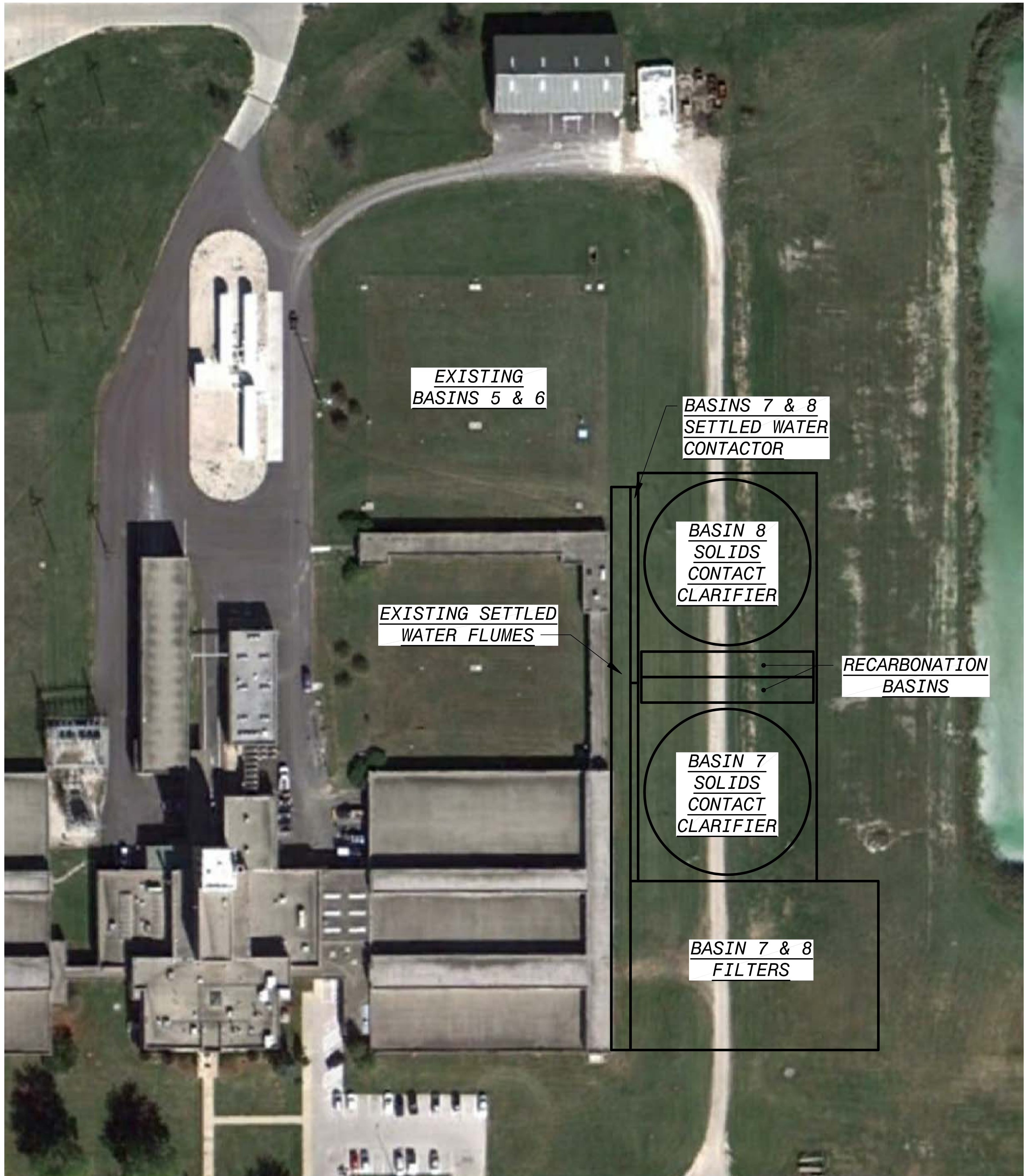
Parameter	Value
Surface Loading Rate, gpm/sq ft (maximum)	1.75
Reaction Zone Detention Time, minutes (minimum)	30
Detention Time Within Clarification Zone, hours (minimum)	2 – 4
Launder Loading Rate, gpm/ft (maximum)	20
*For systems practicing precipitative softening in conventional SCCs	

The existing flocculation/sedimentation basin trains at the Collins Park plant incorporate recarbonation zones at the end of each basin train for adjustment of settled water pH through addition of carbon dioxide, which is introduced as a gas through diffusers installed on the basin floor. Settled water enters the recarbonation zone by flowing over a divider wall which spans the entire width of each basin. Use of solids contact clarifiers would require construction of separate baffled recarbonation basins, which would be located adjacent to the SCCs. (The current "Approved Capacity" document recommends that a hydraulic retention time of at least 20 minutes be provided within recarbonation basins.) The need for separate recarbonation basins downstream of the SCCs would result in additional hydraulic head losses attributable to friction losses within interconnecting piping and entrance/exit losses.

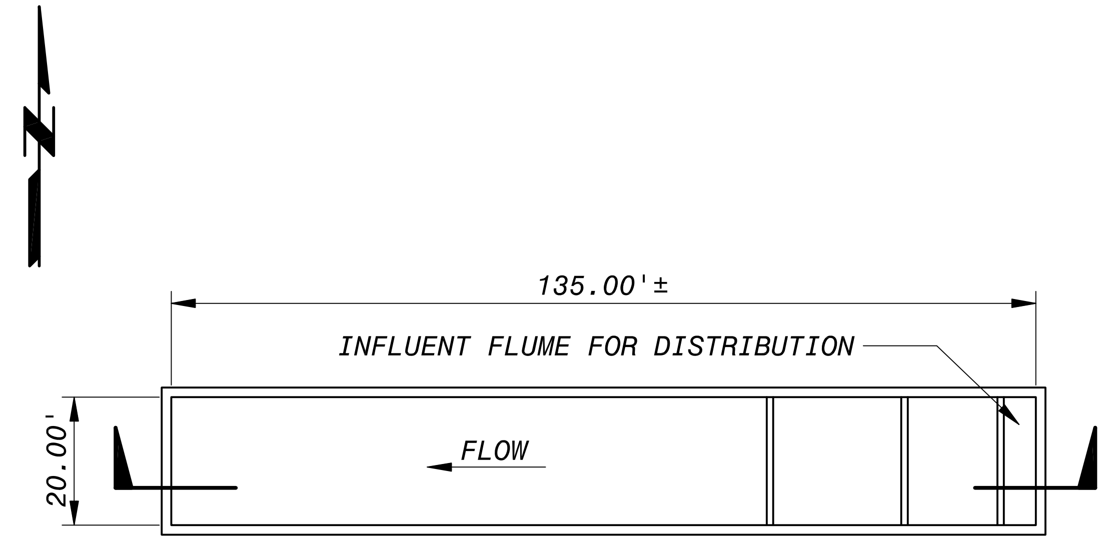
Preliminary clarifier size information was requested from two SCC manufacturers to facilitate development of site layout information and probable construction costs. Proposed basin sizes and design parameters for each of the SCC configurations for which manufacturer proposals were received are summarized in Table 3-9. A preliminary site layout for Basins 7 & 8 using conventional solids contact equipment is shown on Figure 3-4.

Table 3-9: Conventional Solids Contact Clarifier Sizing

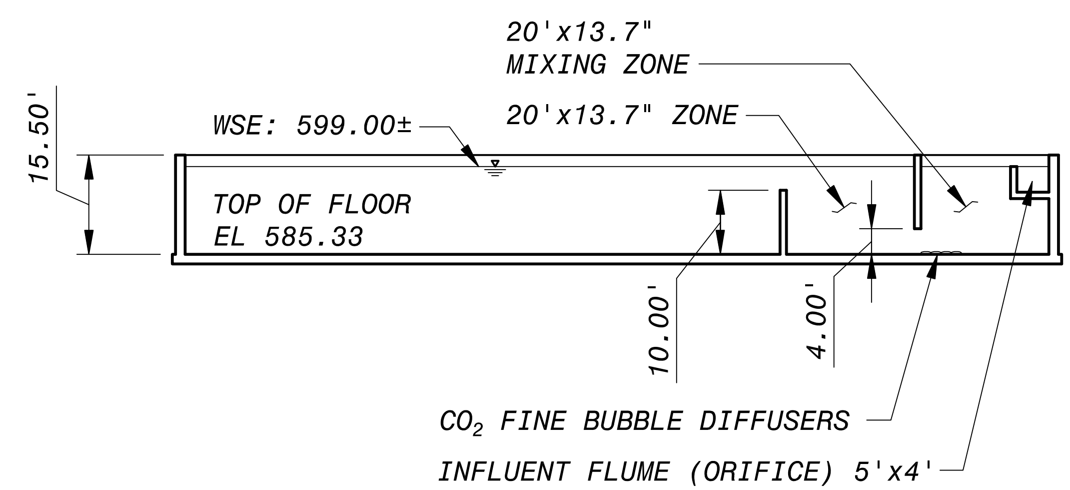
Parameter	Requested Value	WesTech *Solids Contact Clarifier™*	IDI *Accelator IS©*
Diameter, feet	-	130	130
Upper Reaction Zone Diameter, feet	-	38	38.5
Sidewall Water Depth, feet	-	19.25	19.86
Total Basin Depth at Center, feet	-	25.38	24
Impellor Drive Hp	-	25	25
Scraper Drive Hp	-	**	1
Capacity per Basin, MGD	20	20	20
Surface Loading Rate, gpm/sq ft	≤ 1.25	1.20	1.15
Reaction Zone DT, minutes	≥ 30	30	30
Clarification Zone DT, hours	≥ 2	2.0	2.1
**Not specified by manufacturer			



SOLIDS CONTACT CLARIFIER / RECARBONATION BASIN PLAN
1" = 100'-0"



RECARBONATION BASIN NO. 7 - PLAN
RECARBONATION BASIN NO. 8 - PLAN (SYM)
1"=30'-0"



RECARBONATION BASIN NO. 7 - SECTION
RECARBONATION BASIN NO. 8 - SECTION (SYM)
1"=30'-0"

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SOLIDS CONTACT CLARIFIER RECARBONATION BASIN PLAN AND SECTION

CITY OF TOLEDO

DATE: 02/05/15

B&V PROJECT
186505

FIGURE
3-4

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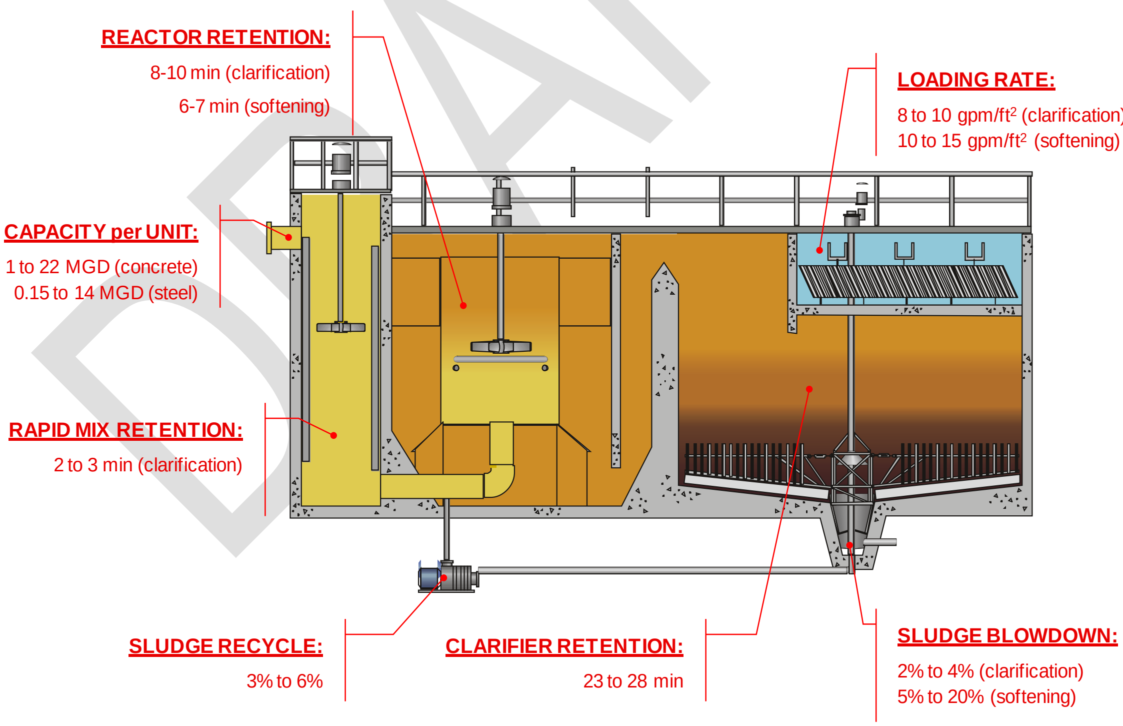
3.2.2 High-Rate Solids Contact Clarifiers

Proprietary alternatives to conventional solids contact clarifiers are available which incorporate a combination of internal and external solids recirculation, high solids concentrations in the reaction zone, and tube-assisted sedimentation to significantly reduce the overall basin footprint and to yield high concentrations of settled solids. Treatment occurs in conjoined reactor and clarifier/thickener vessels or a single multi-compartment circular basin. The high concentrations of reaction zone solids maintained within these units allow the use of surface loading rates of 6 to 12 gpm/ft2 in the settling zone, and total basin detention times of only 25 to 45 minutes (vs. typical minimum 2.5 hours for conventional solids contact clarifiers.) The high levels of solids recirculation maintained within these clarifiers can result in the production of residuals blowdown with solids concentrations of 5% to 20% by weight, which in some cases can reduce or even eliminate the need for further gravity thickening prior to mechanical dewatering. An example of a high-rate solids contact clarifier is shown on Figure 3-5.

Figure 3-5: High-Rate Solids Contact Clarifier (Infilco Degremont DensaDeg®)



There are currently two manufacturers of high-rate SCC equipment:

- Infilco Degremont *DensaDeg*® process
  - 250+ installations since 1984
  - Six U.S. operating municipal softening installations
- WesTech *Contrafast*® process
  - ~20 units installed at 10 locations since 2004
  - One U.S. operating municipal softening installation (2 additional pending)

Both systems feature similar design parameters (reaction and settling zone detention times, solids recirculation requirements, and settling zone hydraulic loading rates), as shown in Table 3-10, and similar operating requirements. While the Contrafast® is a relatively new entry in the high-rate SCC market, there are a significant number of DensaDeg® installations within the US for precipitative softening in both municipal and industrial treatment applications.

Table 3-10: Typical Design Parameters for High-Rate SCCs

Parameter	WesTech Contrafast®	IDI DensaDeg®
Reaction Zone Detention Time, minutes	10 – 12	8 – 10
Clarification Zone Detention Time, minutes	15 – 30	25 - 30
Clarification Zone Tube Loading, gpm/sq ft	≤ 8	8 – 12
Solids Concentration in Discharge, % by weight	6 – 20	5 – 20
Internal Recirculation Rate, x basin influent flow	10	10
External Recirculation Rate, x basin influent flow	0.1	0.05 - 0.1
Largest Single-Basin Capacity, MGD	14*	22*
*Per manufacturer claims; however, no known operating units practicing precipitative softening exist in this capacity range.		

While the treatment technology used by these processes has been well proven through extensive pilot-scale testing and full-scale application (primarily for production of industrial process water), the municipal water treatment industry has been relatively slow to adopt this technology, primarily because of the proprietary nature of the equipment and, in some cases, difficulties in obtaining regulatory agency approval.

There are currently only four operating DensaDeg® installations in Ohio, and no ContraFast® installations. None of the four DensaDeg® installations are municipal plants producing drinking water, and with the exception of the City of Toledo's large CSO treatment facility, are all relatively small installations with design treatment capacities of approximately 0.5 MGD. It is therefore likely that Ohio EPA would require successful pilot-scale demonstration testing (potentially over an extended period) prior to approval of this technology for construction of Basins 7 & 8. However, several relatively large potable water treatment facilities which utilize these high-rate processes have been constructed, including 17 MGD and 50 MGD installations in Colorado.

3.2.3 Comparison of Solids Contact Clarifier Options

Both conventional and high-rate solids contact clarifiers would provide the following advantages as compared to conventional rectangular softening basins:

- Elimination of maintenance-intensive conventional paddle flocculation and chain & flight-type residual solids collection systems.
- More efficient softening reactions attributable to the high degree of internal solids recirculation provided, which promotes more efficient softening reactions and provides potential reductions in lime and soda ash dosages and residual solids production.
- Higher allowable hydraulic loading rates, which yield reduced basin footprint areas (as shown in Table 3-11), lower construction costs, and, where basins must be covered to prevent icing/freeze damage, lower basin covering costs.

Potential disadvantages of both conventional and high-rate solids contact clarifiers as compared to conventional rectangular softening basins include:

- Initial operator unfamiliarity with a "different technology".
- Difficulties in conveying lime and soda ash to the reaction zones of SCCs using the existing feed systems.
- Additional hydraulic head losses attributable to required SCC influent flow splitting, v-notch effluent launder weirs, and the need for separate recarbonation basins.

Table 3-11: Comparative Footprint Requirements for Softening Basin Alternatives

Basin Alternative	Projected Footprint Area Required, sq ft[1]	Area Reduction vs. Conventional Basins, %
Conventional Rectangular Clarifier[2]	28,500	-
Conventional Solids Contact Clarifier		
Circular Basin Footprint Area Only	13,700	52
Equivalent Square Footprint Area[3]	17,500	39
High-Rate Solids Contact Clarifier	5,500	81

[1]Basis:

Area shown is per 20 MGD basin train

Area for recarbonation basin is not included

Area requirements for yard/interconnecting piping/flumes not included

[2]Based on approximate area requirements for existing basins 5 & 6

[3]Total square footprint area required for 130 ft diameter basin

In addition to the disadvantages outlined above, potential adverse consequences associated with selection of high-rate solids contact clarification for construction of Basins 7 and 8 include the following:

- There are still relatively few operating installations which use this technology for precipitative softening of surface water supplies to produce drinking water.

- Largest single-basin treatment capacity for an operating precipitative softening installation is currently 12.5 MGD. While one manufacturer has indicated ability to provide units capable of treating up to 20 MGD if a relatively high sedimentation zone hydraulic loading rate can be used, ability to operate reliably at this high loading rate would need to be demonstrated through pilot-scale testing. Providing the required 40 MGD of treatment capacity using only two high-rate SCC basin trains therefore cannot be guaranteed based on current experience. Four treatment trains would likely be required, which would increase both operating and maintenance requirements as compared to use of two conventional SCCs.

- While experience / data to either confirm or refute this concern is not readily available, the high mixing energy levels and continuous solids recycle pumping required for efficient operation of high-rate SCCs may also increase the potential for shearing of algal cells present in the source water, thereby increasing the potential for release of algal toxins into the process stream.

- Lack of presedimentation facilities for removal of turbidity and color prior to softening, in conjunction with the inherent short detention times associated with use of high-rate SCCs would require operators to closely monitor operations, particularly during periods when source water quality changes are frequent and/or rapid.

- Optimization of treatment operations has been reported to be a challenging process, requiring significant operator attention for an extended period.

It should also be noted that catastrophic failure of reaction zone impeller shafts has occurred recently at one large facility which uses this technology. Requirements for redesign and replacement of these components to permit return of the clarifiers to service have not yet been identified.

Based on the high degree of operational uncertainty and risk associated with these potential adverse consequences, and the likely need for pilot-scale demonstration over an extended period to obtain Ohio EPA approval, use of high-rate SCCs is not considered to represent a technically viable nor desirable option for construction of Basins 7 & 8 at this time.

A preliminary assessment of hydraulic characteristics for new Basins 7 & 8 incorporating conventional SCC equipment was conducted to determine if this technology can be implemented without the need for significant hydraulic improvements/modifications. This assessment was based on information developed using the existing Basin 6 hydraulic model (as discussed in Section 4.2 of the appended Ozone Facilities Conceptual Design Report). Comparison of hydraulic requirements for construction of Basins 7 & 8 using conventional rectangular basins with contiguous recarbonation basins and solids contact basins with separate recarbonation basins indicates that projected hydraulic losses for basin trains utilizing SCCs exceed those for the conventional basin alternative by approximately 0.3 ft (3.5 inches). This suggests that if maintenance of current water surface elevations at the filters is a requirement, the water surface elevation within the raw water channel would need to be increased by approximately 0.3 feet to compensate for the additional losses incurred across the SCC basins and the new recarbonation basins. As this modification would still result in 0.75 feet of freeboard within the raw water channel, it appears that use of conventional SCC basins would be a viable option. However, it is emphasized that this evaluation was conducted using an uncalibrated hydraulic model for Basin 6; therefore, additional detailed hydraulic analyses should be conducted to verify these preliminary results prior to any decision to incorporate SCC equipment in the design of Basins 7 & 8.

The distance that lime and soda ash must be conveyed to be fed at the reaction zones of the solids contact clarifiers is not significantly greater than that between the chemical feed area and the location where the flocculation basins for conventional Basins 7 & 8 would be constructed. Costs in excess of those for lime and soda ash feed capability for the conventional basin alternative would therefore not be incurred for the solids contact clarifier alternative.

3.2.4 Basin 7 and 8 Solids Contact Clarifiers Probable Cost

Opinions of probable cost were developed for (1) conventional rectangular flocculation/sedimentation basins with contiguous recarbonation basins located at the effluent end of the sedimentation zones (similar to the existing basin trains), and (2) circular solids contact clarifiers with separate recarbonation basins. Probable construction costs are presented for all major structures and components. Construction costs were developed based on information provided by equipment suppliers and on past construction costs for similar projects. To account for items not included in the major cost component tabulations, and for engineering and administrative/legal fees, a 30 percent contingency and a 20 percent allowance for engineering and legal/administrative fees have been added to the individual component tabulations to arrive at the total probable project cost.

Probable project costs for construction of new basins 7 & 8 utilizing solids contact clarification equipment are summarized in Table 3-12. Probable costs are based on construction of two 20 MGD capacity, 130-foot diameter clarifiers. The basins would be equipped with aluminum geodesic dome-type covers for freeze protection, with access to the basin center reaction zone area provided through use of dormer-type doors and walkway covers. Two 20 MGD capacity recarbonation basins (one per SCC basin train) would be provided. Recarbonation basins would be of concrete construction and sized to provide inlet dispersion/mixing zone and total basin hydraulic times of 3 minutes and 20 minutes, respectively, in order to comply with current Ohio EPA “Approved Capacity” criteria and the 2012 “Recommended Standards for Water Works”. Carbon dioxide would be fed in gaseous form through fine-bubble diffusers installed at the floor of the inlet dispersion/mixing zones.

Table 3-12: Basin 7 & 8 Solids Contact Clarifiers Option Probable Cost

Component	Probable Cost
Earthwork	$697,000
Substructures	$3,571,000
Equipment	
Solids Contact Clarifiers	$2,567,000
Aluminum Geodesic Covers	$856,000
Recarbonation Basin Diffusers	$263,000
Piping, Valves, Gates, and Meters	$975,000
Site Civil (3%)	$268,000
Electrical and I&C (10%, excl. earthwork)	$850,000
General conditions and mobilization (10%)	$1,005,000
Subtotal	**$11,052,000**
Contingency (30%)	$3,316,000
Subtotal with Contingency	**$14,368,000**
Contractor insurance/bonding (5%)	$718,000
Contractor overhead/profit/general (15%)	$2,155,000
Total Construction Cost	**$17,200,000**
Construction costs inflation to construction midpoint (3%)	$516,000
Subtotal	**$17,716,000**
Engineering/Legal/Administrative (20%)	$3,543,000
Total Project Probable Cost	**$21,300,000**

3.2.5 Filtration

Design of the Filtration Plant for Alternative 2 is as described above for Alternative 1.

3.2.6 Chemical Feed Provisions

Chemical feed provisions for Alternative 2, Solids Contact Clarifiers are discussed in Section 5.6.10.1, Future Chemical Conveyance.

3.2.7 Alternative 2 – Solids Contact Clarifiers Project Costs

The opinion of probable cost for Alternative 2 – Solids Contact Clarifiers is presented in Table 3-13.

The opinion of probable cost is the total of the probable costs of Solids Contact Clarifier Option from Table 3-12 and the probably cost of the Filter Building from Table 3-7.

Table 3-13: Alternative 2 – Solids Contact Clarifiers Project Cost

Component	Probable Cost
Solids Contact Clarifiers	$21,300,000
Filter Building	$38,100,000
Total Alternative 2 Project Cost	**$59,400,000**

3.3 Comparison of Redundant Capacity Alternatives

3.3.1 Annual Operation and Maintenance Costs

Projected annual operation and maintenance (O&M) costs were developed for the conventional rectangular basin and SCC basin options discussed above, based on the following parameters:

- Average daily treated water production = ~50 percent of rated design capacity
  - Average production rate = 10 MGD per basin
- Electrical energy cost = $0.056 per kWh
- Operations/maintenance labor cost (average including benefits) = $39.50 per hour
- Chemical doses (based on current annual average feed rates):
  - Lime = 100 mg/L
  - Soda ash = 8 mg/L (~31 mg/L average dosage when fed)
- Chemical unit costs:
  - Lime = $146 per ton
  - Soda ash = $309 per ton

Annual O&M costs are summarized in Table 3-13. Maintenance materials include parts and supplies required for routine maintenance of the treatment facilities.

Electrical costs were developed based on connected motor horsepower for mixers and residual solids collection equipment and/or projected energy requirements to achieve targeted velocity gradient (“G”) values during flocculation. Electrical energy costs also include an allowance for ventilation of the SCC basin enclosures, as operators would need to routinely enter the enclosed areas to collect samples for determination of solids concentrations within the clarifier reaction zones. Labor costs are based on projected requirements for routine maintenance of process equipment (gearbox oil changes, periodic maintenance of drive mechanisms and weirs, etc.), for routine monitoring of treatment conditions such as solids concentrations within the SCC reaction zones, and for determination of and maintenance of appropriate chemical feed rates.

Table 3-13: Projected Annual Operation & Maintenance Costs for Basin Alternatives

Component	Annual O&M Cost, $/year	
	Conventional Rectangular Clarifiers	Solids Contact Clarifiers
Maintenance Materials	27,500	11,500
Electrical Energy	12,000	24,000
Labor	50,000	39,500
Chemicals	520,000	494,000
Total Annual O&M	$609,500	$569,000

Chemical costs reflect projected annual costs for maintaining required lime and soda ash dosages to achieve targeted finished water hardness concentrations, and are based on historical feed rate data. While potential reductions in historical chemical dose requirements for softening attributable to use of solids contact clarifiers are difficult to predict and highly site-specific, a 5 to 10 percent reduction attributable to improved mixing conditions and ability to provide for more complete softening reactions may be achieved, based on experience of utilities which operate both conventional and solids contact clarifiers in parallel. The chemical O&M costs presented in Table 3-13 assume a conservative 5 percent reduction in required lime and soda ash dosages for the SCC alternative as compared to the conventional rectangular basin alternative.

3.3.2 Present Worth Costs

Total costs associated with constructing and operating each of the new basin alternatives discussed above can be compared by adding the probable costs for the treatment facilities to the present worth of the annual operation and maintenance costs incurred over an extended period of time for each alternative. In general, the alternative with the lowest present worth cost is the most desirable with respect to total cost. For the comparison presented below, a planning period of 20 years and an effective interest rate (interest minus inflation) of 3 percent was assumed. Present worth costs are summarized in Table 3-14.

Table 3-14: Present Worth Costs for Treatment Basin Alternatives

Redundant Capacity Alternative	Total Project Cost	Projected Annual O&M Cost	Present Worth Cost of Annual O&M	Total Present Worth Cost
Alternative 1 – Conventional Basins	$67,800,000	$609,500	$9,068,000	$76,868,000
Alternative 2 – Solids Contact Basins	$59,400,000	$569,000	$8,465,000	$67,865,000
*20 years @ effective interest rate of 3 percent				

3.3.3 Conclusions / Recommendations

Solids contact clarifiers have been recognized for many years as the “industry standard” where precipitative softening treatment is required to achieve finished water hardness goals. Use of SCC equipment for the Redundant Capacity Improvements would provide several advantages such as improved chemical efficiencies, reduced footprint area requirements, lower construction cost, and annual operation and maintenance costs. However, it is also recognized that implementation of a technology which differs from that used for the existing West and East Plant 120 MGD precipitative softening treatment capacity may not represent a desirable option when staff familiarity with current plant operating and maintenance requirements is considered.

Further, results of preliminary hydraulics analyses suggest that projected hydraulic head loss across treatment trains equipped with SCC equipment would exceed that for conventional rectangular basins by approximately 0.3 feet at full rated design flow rates. Therefore, implementation of the SCC alterative would require either a reduction in current filter operating levels or an increase in raw water flume water surface elevation. As discussed above, a more detailed hydraulics evaluation using a calibrated model for the existing flocculation/sedimentation basin trains is recommended to confirm assumptions and results from the preliminary hydraulics evaluation for the Basin 7 and 8 improvements using SCC equipment.

3.4 Redundant Capacity Improvements Basis of Design

Following is the Basin of Design Table for the Redundant Capacity Improvements as required in the Ohio EPA Approved Capacity document. The Basis of Design Table is based on the Alternative 1 - Conventional Sedimentation / Softening for Basins 7 and 8.

Section 5, Upgrades to Existing Facilities discusses modifications to Basins 1 through 6. Those modifications will result in Basins 1 through 4 having an identical Basis of Design. Furthermore, Basins 5 and 6 will have an identical Basis of Design to new Basins 7 and 8. For convenience in this General Plan, the Basis of Design for the Upgraded Basins 1 through 4 are also presented in the following Table 3-14.

Table 3-15: Basis of Design for Upgraded Basins 1 – 6 and Proposed Basins 7 & 8

1 Component	2 Number of Units	3 Design Standards	4 Design Criteria	5 Required/ Recommended	6 Component Capacity (MGD)(5)	7 Flow Basis of Component Capacity/Ratio	8 Equivalent Maximum Day Capacity (MGD)
Rapid Mixing	2	Units	Minimum 2	Required			
		G Value	750	Recommended		1.0	
		Detention Time	Maximum 30 sec.	Recommended		1.0	
Upgraded Basins 1 through 4 – Rated at 80 MGD Total							
Flocculation	4	Units	Minimum 2	Required			
		Flow-thru Velocity	0.5-1.5 fpm	Recommended	20	1.0	
		Detention Time	Minimum 30 min.	Recommended	106	1.0	
Sedimentation	4	Units	Minimum 2	Required			
		Weir Overflow Rate	Max 20,000 gpd/ft.	Required	80	1.0	80
		Detention Time	Minimum 4 hours	Required	56(80)(1)	1.0	56(80)(1)
		Outlet Velocity	Maximum 0.5 fps	Required	N/A	1.0	
		Surface Load Rate	Maximum 0.75 gpm/sf	Recommended	92	1.0	
		Flow-thru Velocity	Maximum 0.5 fpm	Recommended	26	1.0	
		Length/Width Ratio	Minimum 3 : 1	Recommended	3.0		
Stabilization	4	Detention Time	Minimum 20 min.	Recommended	71	1.0	
		Mixing Time	Minimum 3 min.	Recommended	106	1.0	

1 Component	2 Number of Units	3 Design Standards	4 Design Criteria	5 Required/ Recommended	6 Component Capacity (MGD)(5)	7 Flow Basis of Component Capacity/Ratio	8 Equivalent Maximum Day Capacity (MGD)
		Diffuser Submergence	Minimum 7.5 ft.	Recommended			
Filtration	20	Filtration Rate	Maximum 3 gpm/sf (2)	Required	115	1.0	115
		Backwash Sources	Primary & Backup (3)	Required			
		Backwash Flow Capacity	Minimum 15 gpm/sf for 15 min.	Required			
Upgraded Basins 5 & 6 and Proposed Basins 7 & 8 – Rated at 80 MGD Total							
Flocculation	4	Units	Minimum 2	Required			
		Flow-thru Velocity	0.5-1.5 fpm	Recommended	17	1.0	
		Detention Time	Minimum 30 min.	Recommended	97	1.0	
Sedimentation	4	Units	Minimum 2	Required			
		Weir Overflow Rate	Max 20,000 gpd/ft.	Required	80	1.0	80
		Detention Time	Minimum 4 hours	Required	62 (80)(1)		62 (80)(1)
		Outlet Velocity	Maximum 0.5 fps	Required	N/A	1.0	
		Surface Load Rate	Maximum 0.75 gpm/sf	Recommended	102	1.0	
		Flow-thru Velocity	Maximum 0.5 fpm	Recommended	26	1.0	
		Length/Width Ratio	Minimum 3 : 1	Recommended	3.4		
Stabilization	4	Detention Time	Minimum 20 min.	Recommended	80	1.0	

1 Component	2 Number of Units	3 Design Standards	4 Design Criteria	5 Required/ Recommended	6 Component Capacity (MGD)(5)	7 Flow Basis of Component Capacity/Ratio	8 Equivalent Maximum Day Capacity (MGD)
		Mixing Time	Minimum 3 min.	Recommended	104	1.0	
		Diffuser Submergence	Minimum 7.5 ft.	Recommended			
Filtration	20	Filtration Rate	Maximum 3 gpm/sf (2)	Required	115	1.0	115
		Backwash Sources	Primary & Backup (3)	Required			
		Backwash Flow Capacity	Minimum 15 gpm/sf for 15 min.	Required			
Clearwells	2	Units	Minimum of 2	Required			
		Giardia lamblia inactivation	0.5-log	Required	210	1.22	172
		Viruses inactivation	2.0-log	Required	2,555	1.22	2094
High Service Pumps	6	Flow Capacity	Peak Hour Demand (4)	Required	180	1.22	148

(1) At initial process review meeting held with Ohio EPA, EPA staff expressed acceptance of basin detention time less than 4 hours with addition of effluent weirs.

(2) Current rating of 3 gpm/sf. Can potentially be increased to 4 gpm/sf but subject to meeting criteria established in the Approved Capacity Document (page 25 of 38)

(3) Backup system could consist of two (2) pumps or a tank and a pump

(4) Evaluation must consider meeting the demand with the largest pump out of service

(5) Component capacities calculated using field measured basin dimensions and as-built drawings.

3.5 Recommended Approach for Redundant Capacity Project

- *This section will be completed following the work of the Blue Ribbon Panel and discussions with City of Toledo Water Division staff concerning the recommended actions of the Blue Ribbon Panel*

4. Alternatives for Additional Treatment Barriers for HAB Events

4.1 Introduction

4.1.1 Alternatives Screening

There were several alternatives initially considered for additional treatment barriers for HAB events which included:

- Increased feed of potassium permanganate at the Intake Crib/Low Service Pumping Station (LSPS)
- Powdered activated carbon (PAC) feed improvements at the LSPS
- PAC post sedimentation at the East & West Plants
- DAF (dissolved air flotation)
- ozone post-sedimentation
- post filtration GAC contactors
- UV Advanced Oxidation Process (AOP)

Due to the urgency of being able to implement treatment technologies by the 2015 algal season, HAB Short Term Measures projects were identified from these potential alternatives that can be implemented in this short time period. These projects include:

- LSPS Potassium Permanganate System Improvements
- LSPS PAC Feed Improvements
- East & West Plant PAC Feed Improvements

It was determined the other alternatives of DAF (dissolved air floatation), ozone raw water or post-sedimentation, post-filtration GAC contactors and UV AOP were too large and complex to design and construct in this short term period. These technologies are considered to be longer term treatment barriers options for HAB events.

After initial screening of these longer term treatment barriers, DAF was determined no longer feasible and was removed from further consideration and evaluation. Although DAF is an excellent process to remove unlysed algal cells there were many site specific disadvantages that included available space limitations at the LSPS (that is surrounded by Cedar Point National Wildlife Refuge), LSPS site waste stream disposal and treatment (City of Oregon wastewater system ~6 miles away), and inability to treat lysed dissolved microcystin in the water (if located at the Collins Park WTP Facility with existing primary LSPS PAC feed upstream and unavailable).

UV AOP using hydrogen peroxide was also determined no longer feasible and removed from further consideration and evaluation as a long term treatment barrier option. For UV AOP with hydrogen peroxide to be effective against harmful algal bloom toxins, the amount of UV and power requirements must be many times greater than required for UV levels used in normal water treatment. This increased sizing along with the probable need of having to re-pump the treated flow did not make this alternative attractive for further evaluation.

Therefore, three alternatives were evaluated that include:

- Alternative 1 – Existing PAC and Potassium Permanganate System Enhancements,
- Alternative 2 – Ozone Raw Water or Post Sedimentation, and,
- Alternative 3 – Post filtration Granular Activated Carbon (GAC).

4.1.2 Microcystin Data August 2015

During the summer of 2015 the City performed extensive microcystin testing at the intake, surge well, raw water at the WTP, post sedimentation, filtered, entering the clearwell and tap. To provide some background data on the microcystin concentration results, the following three figures have been developed:

- Figure 4-1 Microcystin Lysed 0 – 50 ppb, August 8 – 23, 2015,
- Figure 4-2 Microcystin Lysed 0 – 10 ppb, August 8 – 23, 2015,
- Figure 4-3 Microcystin Extracellular, August 15 – 23, 2015,

The data in these figures should be used as a trending tool over time, as the samples were not directly sequenced with the water as it flows through the plant processes. Figures 4-1 and 4-2 shows the total microcystin in the samples by lysing all algal cells and releasing the microcystin so it can be measured.

When lake water and weather conditions become favorable for HAB growth the algae can reproduce very quickly. This can be seen in Figure 4-1 where the raw water at the intake microcystin levels spiked in mid-August to a peak of approximately 50 µg/L.

Figure 4-1: Microcystin Lysed – August 2014

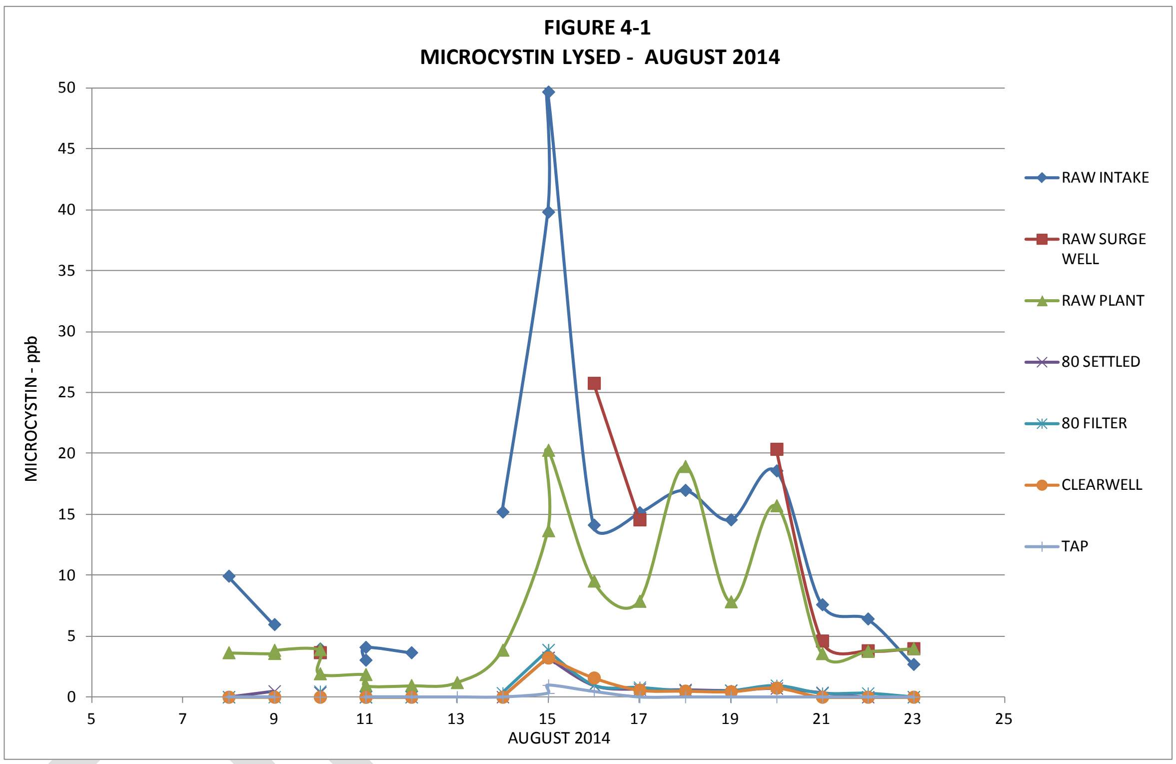


Figure 4-2: Microcystin Lysed – August 2014

FIGURE 4-2
MICROCYSTIN LYSED - AUGUST 2014

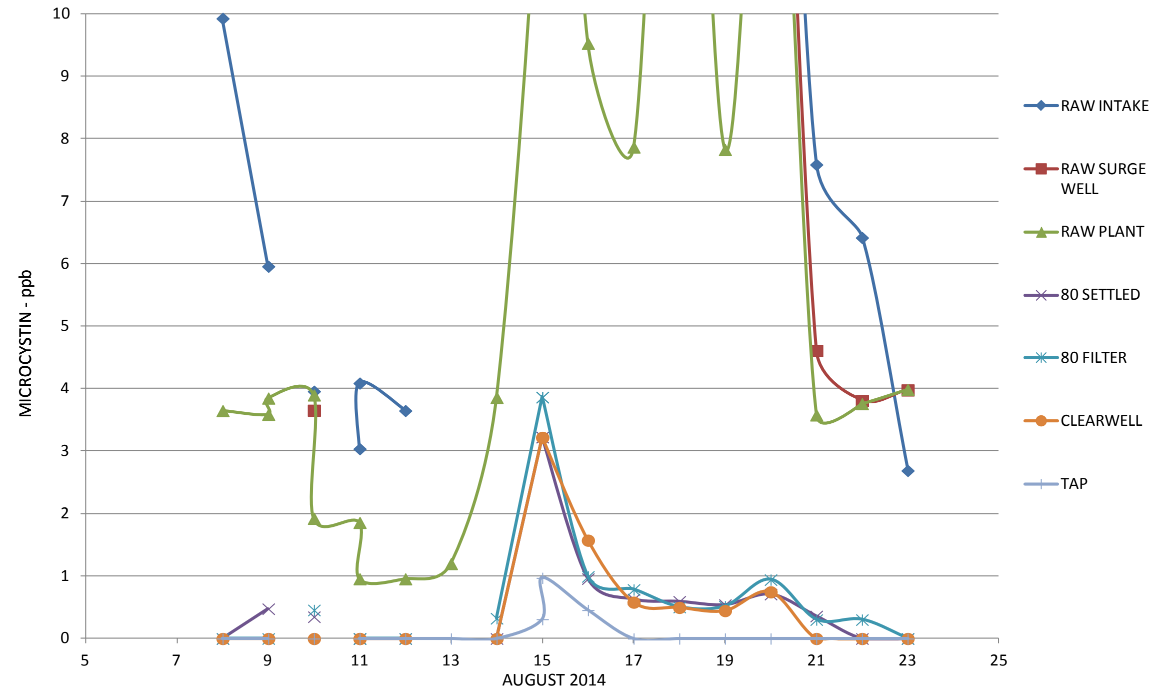
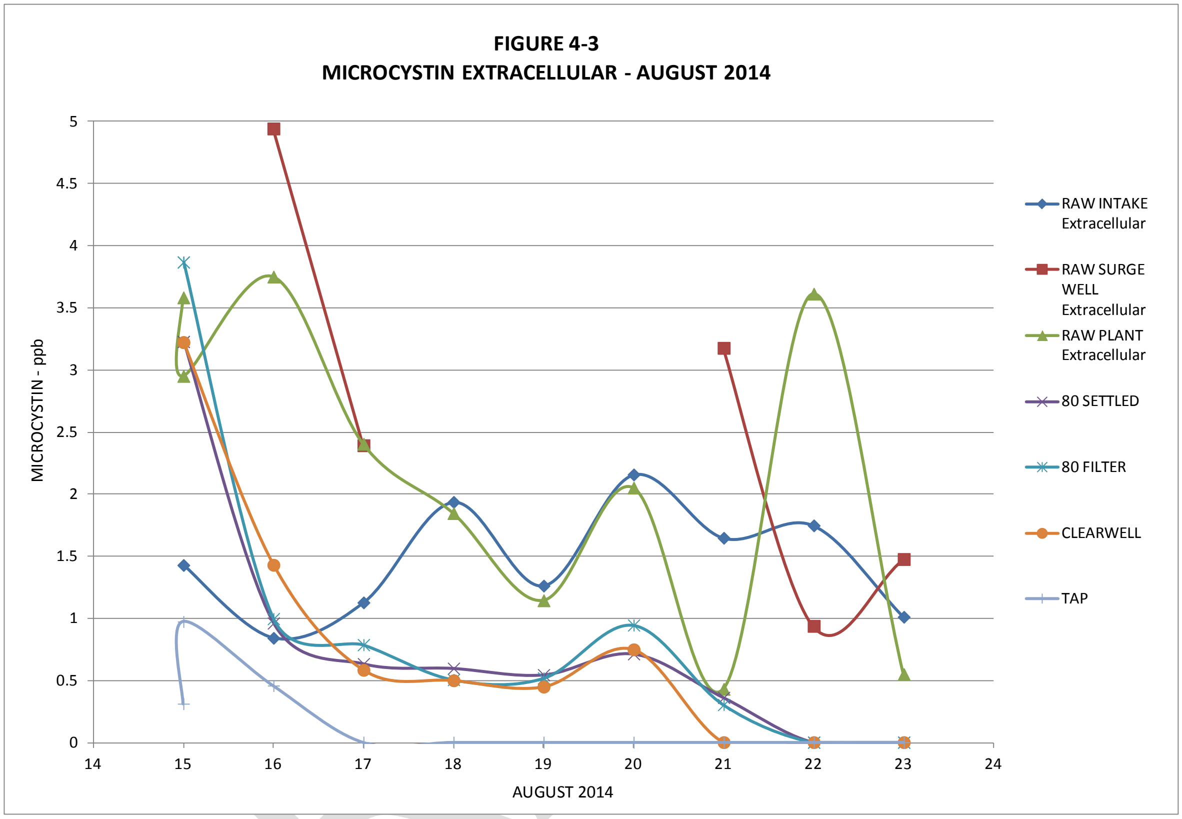


Figure 4-3 only shows existing extracellular microcystin in the water and is filtered first to remove all unlysed cells from the sample before testing for microcystin levels. Of special note, on August 16 and 17 when comparing levels between the intake and the surge well the data is indicative that some lysing is occurring after feeding potassium permanganate for quagga and zebra mussel control at the intake

Figure 4-3: Microcystin Extracellular – August 2014



4.2 Alternative 1 - Existing Powdered Activated Carbon and Potassium Permanganate System Enhancements

Alternative 1 includes increasing the feed of potassium permanganate at the Intake Crib/Low Service Pumping Station (LSPS), powdered activated carbon (PAC) feed improvements at the LSPS and PAC pre or post sedimentation at the East & West Plants.

4.2.1 Low Service Pump Station - Potassium Permanganate Improvements

Collins Park operates a potassium permanganate feed system at the Low Service Pumping Station for quagga and zebra mussel control. Permanganate dosing was designed for a 1 mg/L feed rate at a 150 MGD production flow. Although this feed rate is sufficient for mussel control, it does not significantly assist in the reduction of algal toxins. The plant staff requested that the permanganate feed system be upgraded to allow for higher feed rates up to 6.5 mg/L at 160 MGD production flow. Feed rates are limited to 6.5 mg/L due to concerns of elevating dissolved manganese levels and the ability for the treatment processes to be able to sufficiently remove downstream. Considerations are to lyse the algae cells with permanganate solution during source water pumping, apply needed dosages of activated carbon to remove algal toxins in the raw water, and continue treatment for algal toxins at the Collins Park Treatment Plant. The improvements needed to provide the necessary permanganate feed rates are outlined below.

The existing permanganate feed system consists of two 650 cubic foot storage silos housing dry potassium permanganate. Each silo has two 0.8 cubic foot per hour volumetric feeders attached to the drop chutes and feed material to a common 200 gallon dissolving tank. Raw water from low service is supplied to the feed system and booster pumps are used to dilution water and for motive water to carry permanganate solution to the intake crib through two 3-inch HDPE feed lines some 16,000 feet from the chemical feed building. Solution strength is controlled to less than 2 percent to prevent dissolving and feed problems. Typically, one feeder and dissolving tank are used for plant production. Eductors draw permanganate solution from either dissolving tank and convey the solution through the 3-inch feed lines to a diffusing ring at the intake structure in Lake Erie.

The improvements involved in this project include removal of one volumetric feeder from each silo and installing a new 2.8 cubic foot per hour volumetric feeder in its place. This provides a large feeder and a small feeder on each silo for redundancy and feed system reliability. The larger feeder will be used for HAB treatment applications. The small feeders will be used as current operation for mussel control in non-algae season operations. The existing booster supply pumps will remain and supply water to the dissolving tanks and provide additional carrier water to the intake application point. Metered dilution water flow at about 15 gpm will be provided to each dissolving tank as needed. The existing tank mixers will be used as current operations. The eductor systems will be removed and replaced with new 15 gpm (140 psi) progressive cavity feed pumps drawing permanganate solution from each dissolving tank and pumping solution through the respective 3-inch feed lines to the intake diffusion ring. A VFD drive and control system for each pump and dissolving tank will maintain the water level in each tank based on expected operations. The dilution of permanganate in operations should be 3.0 percent or less to minimize dissolving and feed issue in the warmer summer water temperatures. A spare permanganate feed pump will be purchased and stored for future use and reliability of the HAB treatment system.

Using a maximum feed rate capacity of 6.5 mg/L of potassium permanganate and conservative summertime monthly average flow of 105 MGD equates to approximately 19 days of storage capacity (using the existing 109,200 pounds of silo storage). The average summer feed rate may likely be much less than maximum feed rate capacity and likely provide a supply in excess of 30 days of storage.

Figure 4-4 shows the proposed potassium permanganate feed changes layout plan and Figure 4-5 shows the proposed process flow diagram changes.

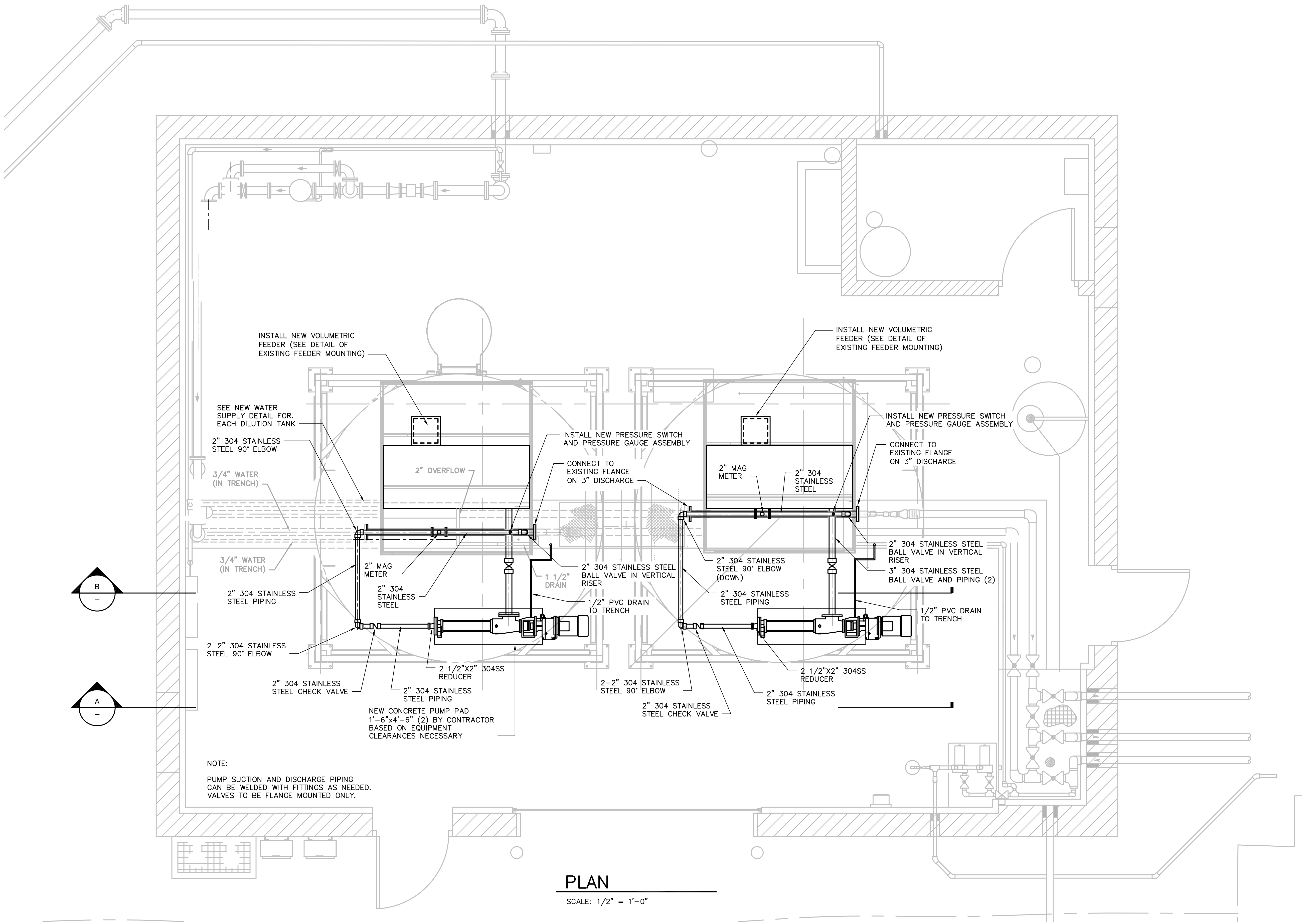
4.2.2 Low Service Pump Station – Powder Activated Carbon (PAC) Improvements

The existing Low Service Pumping Station PAC feed system is presently limited to a feed rate of approximately 15 mg/L and is also in need of additional PAC storage.

Planned feed improvements include increasing the peak feed rate to 40 mg/L to allow for more complete absorption of microcystin during the 4 – 6 hours contact time of travel from the Low Service Pump Station to the Collins Park WTP. This would require replacing all the existing three (3) carbon feeders, eductors, and jet pumps to handle this increased feed rate. In lieu of replacing the (3) carbon feeders, 3 new additional hose pump feeders will be provided and capable of feeding 40 mg/L at an ultimate plant design flow rate of 160 MGD (One hose pump for each 60” & 78” raw water main with one spare redundant hose pump).

Planned storage improvements include the addition of an approximately 11,667 cu.ft. (175,000 lbs) dry PAC storage silo to supply carbon to the two existing in-ground bulk carbon slurry tanks. A pneumatic conveyance system will be provided to deliver the stored carbon from the silo to the existing slurry tanks. Including the two existing underground PAC slurry storage tanks of 60,000 lbs of PAC, total storage is increased to 235,000 lbs. Using an average HAB season feed rate of 15 mg/L of PAC and conservative summertime monthly average flow of 105 MGD equates to approximately 18 days of storage capacity. Using normal summertime feed rates of 6 mg/L provides 45 days of storage capacity.

Figure 4-6 shows the site plan of these improvements. Figures 4-7 and 4-8 show the general layout and a schematic of the new PAC feed improvements.



NOTE:

PUMP SUCTION AND DISCHARGE PIPING CAN BE WELDED WITH FITTINGS AS NEEDED. VALVES TO BE FLANGE MOUNTED ONLY.

PLAN

SCALE: 1/2" = 1'-0"

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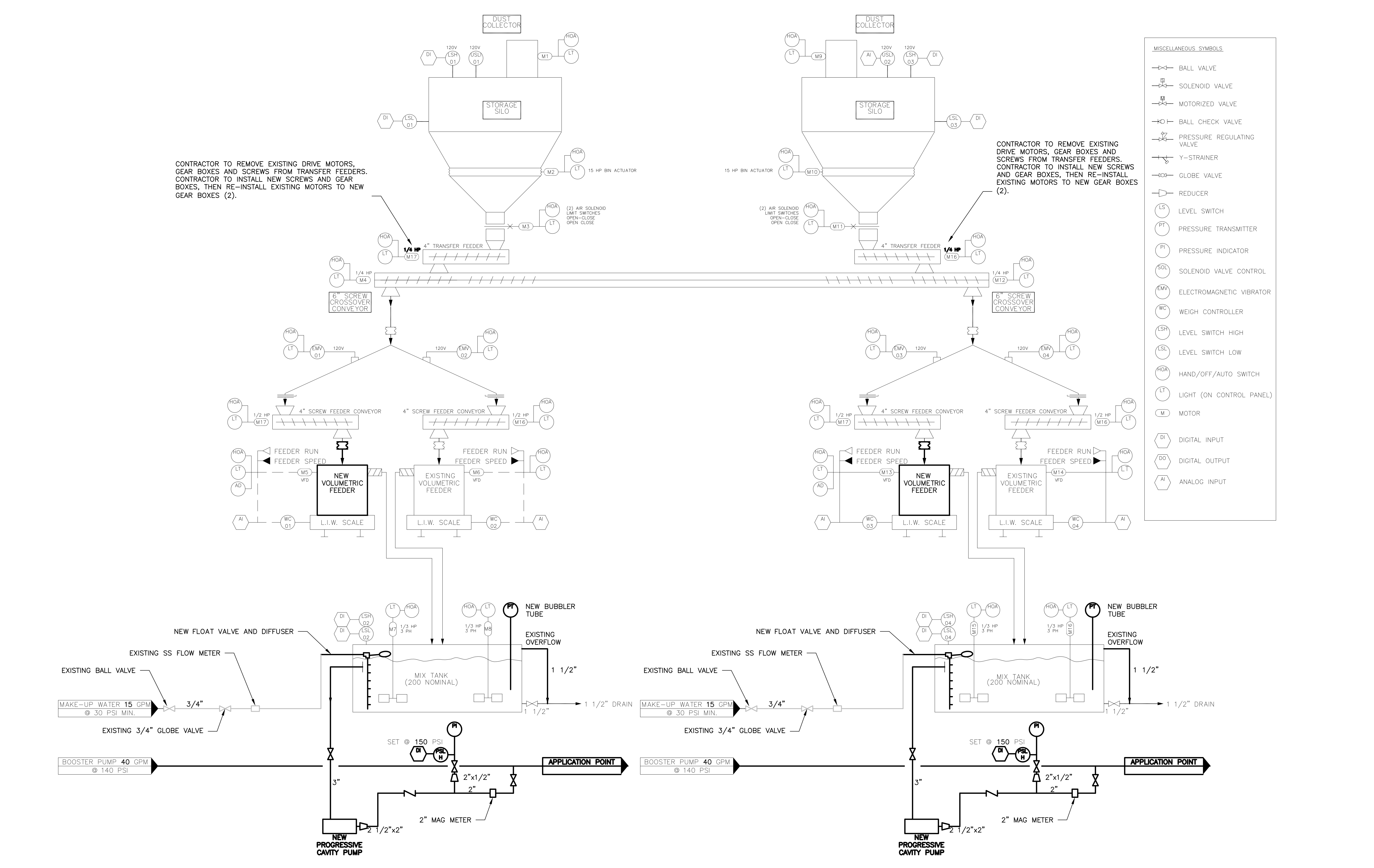
COLLINS PARK WTP
HAB CHEMICAL FEED IMP.

LOW SERVICE PUMPING STATION
POTASSIUM PERMANGANATE FEED
IMPROVEMENTS

FEB 2015

FIGURE 4-4

User: CKIRSCH Spec: AUS-NCSMOD File: G:\PROJECT\TO000747.M002\DRAWING\PAC\FIGURE 4-4.DWG Scale: 1:1 SavedDate: 2/3/2015 Time: 10:26
Plot Date: Kirsch, Clair; 2/4/2015; 10:20 ; Layout: D-06



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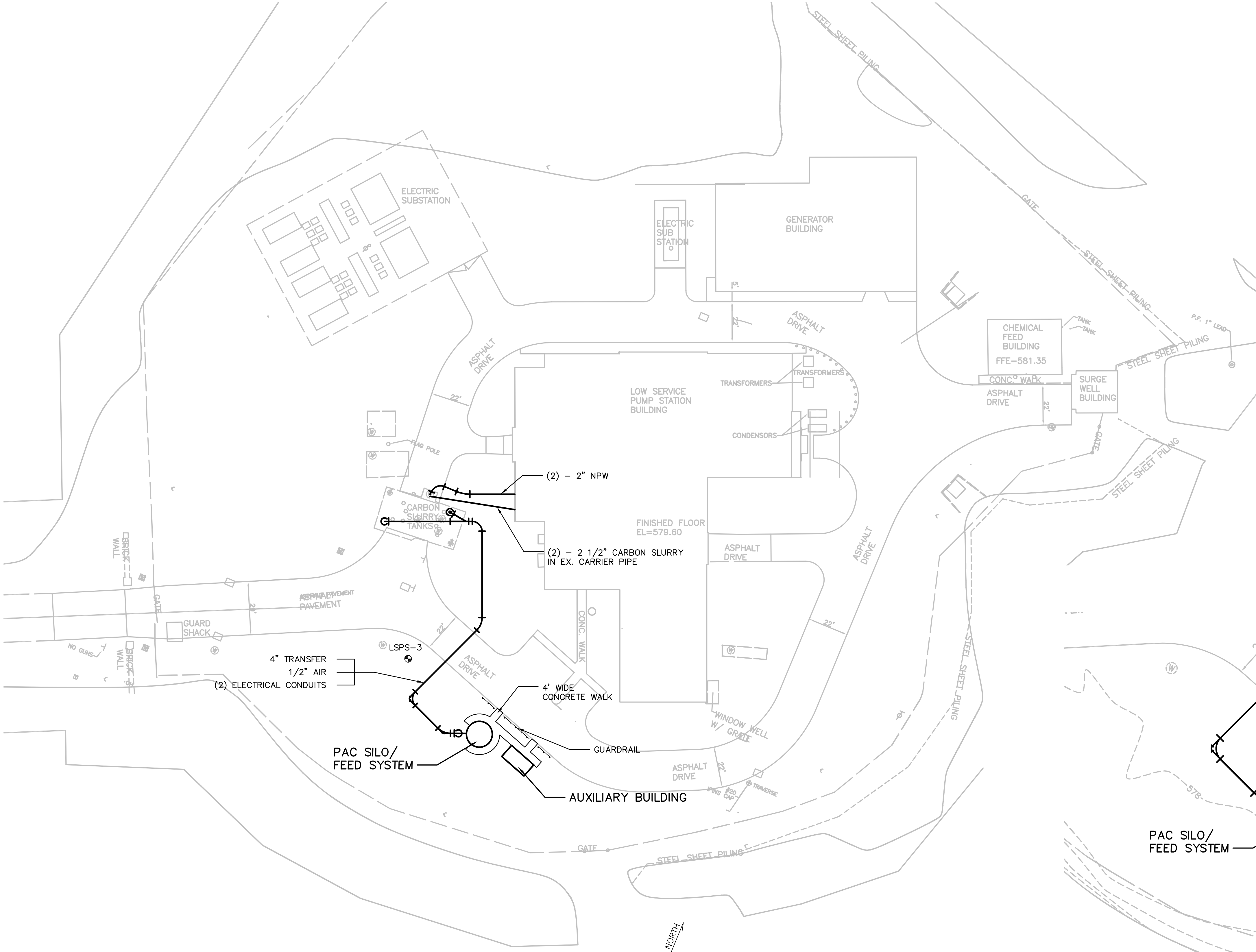
HAB CHEMICAL FEED IMP.

LOW SERVICE PUMPING STATION
POTASSIUM PERMANGANATE PROCESS
DIAGRAM

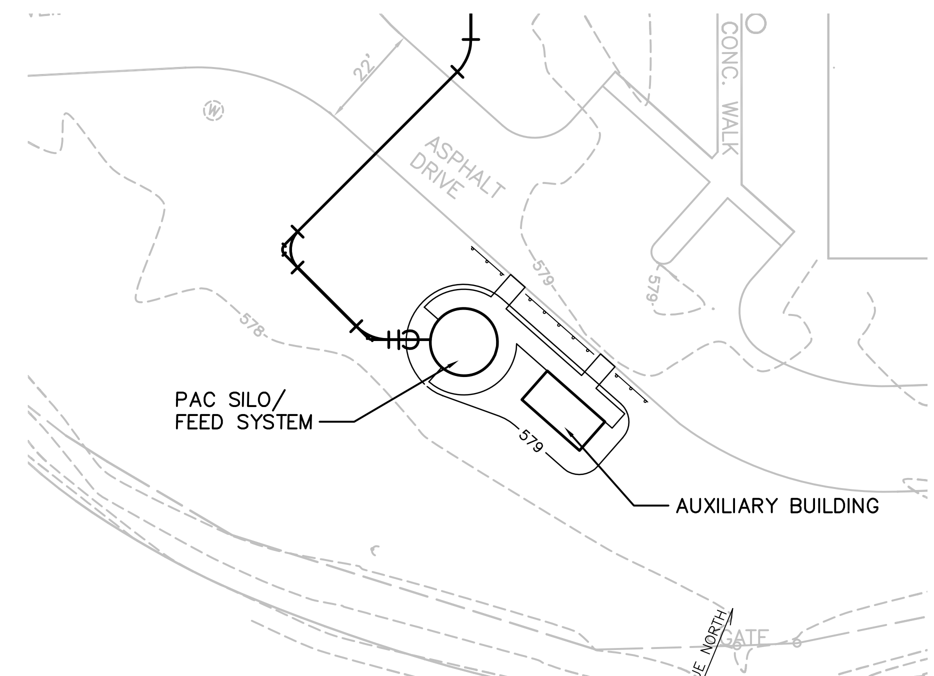
FEB 2015

FIGURE 4-5

User: CKIRSCH Spec: AUS-NCSMOD File: G:\PROJECT\TO000747.M002\DRAWING\PAC\FIGURE 4-5.DWG Scale: 1:1 SavedDate: 2/4/2015 Time: 10:33
Plot Date: Kirsch, Clair; 2/4/2015; 11:17 ; Layout: D-04



1 PIPING PLAN
03 1" = 30'



2 GRADING PLAN
03 1" = 20'

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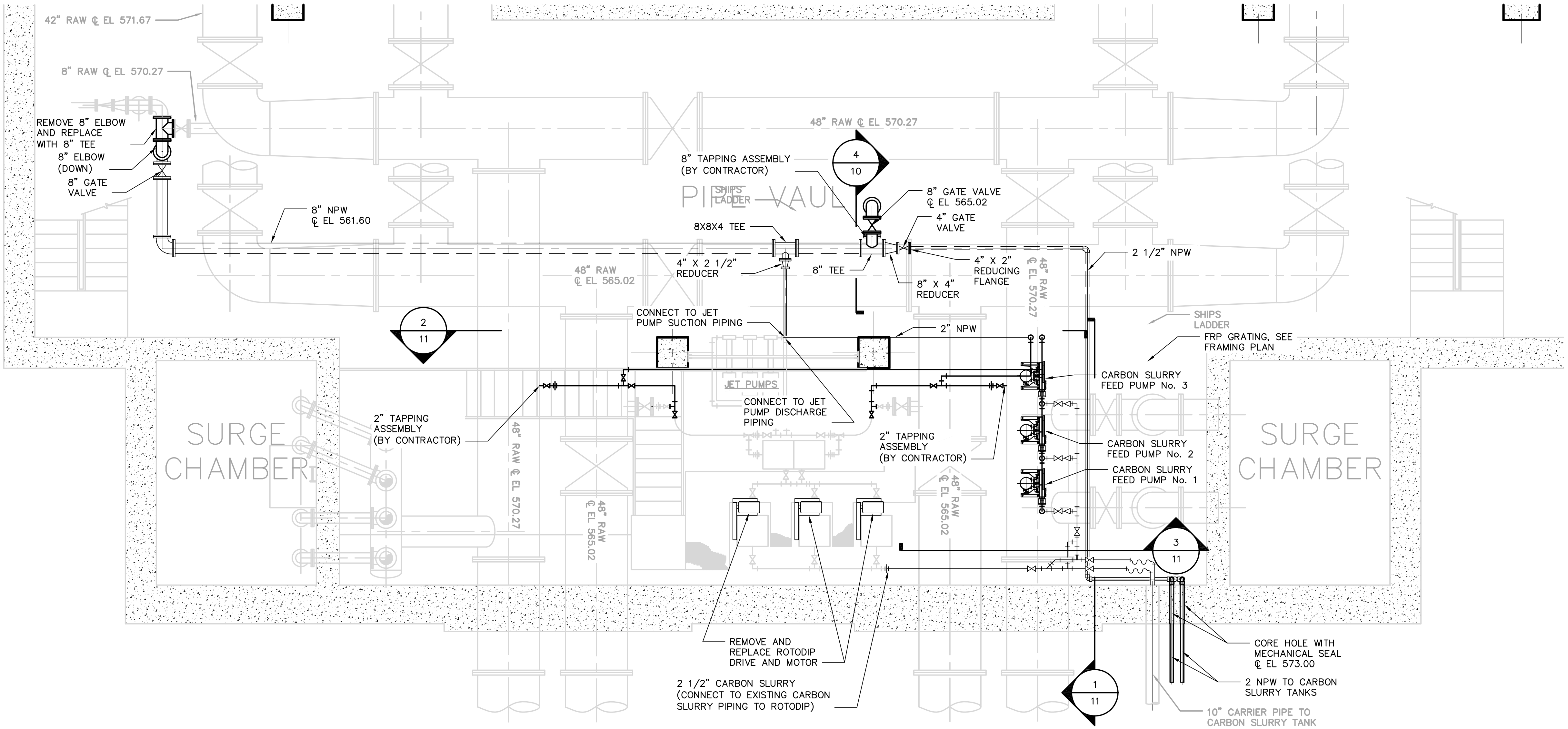
HAB CHEMICAL FEED IMP.

LOW SERVICE PUMPING STATION
PAC SYSTEM SITE PLAN

FEB 2015

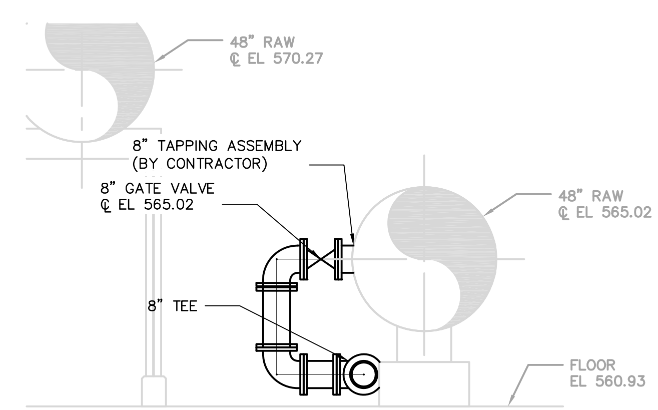
FIGURE 4–6



SECTIONAL PLAN

SCALE: 1/4" = 1'-0"

0 2' 4' 8'



4 / 10 SECTION

SCALE: 3/8" = 1'-0"

0 2' 4' 6'

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COLLINS PARK WTP
HAB CHEMICAL FEED IMP.

LOW SERVICE PUMPING STATION
PAC PROCESS PIPING PLAN

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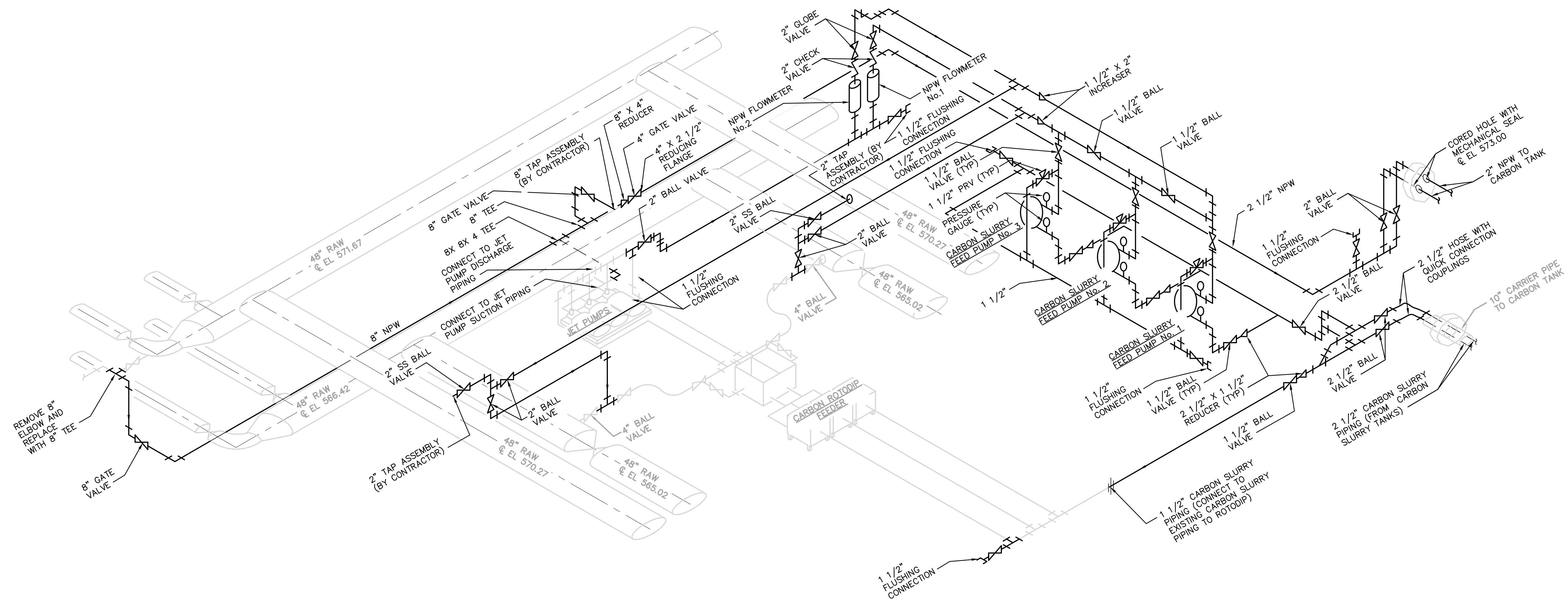
FIGURE 4-7

User: CKIRSCH Spec: AUS-NCSMOD File: G:\PROJECT\TO000747.M002\DRAWING\PAC\FIGURE 4-7.DWG Scale: 1:1 SavedDate: 2/3/2015 Time: 10:26
Plot Date: Kirsch, Clair; 2/4/2015; 10:34 ; Layout: D-01



CARBON FEED SCHEMATIC

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LOW SERVICE PUMPING STATION
PAC FEED SCHEMATIC

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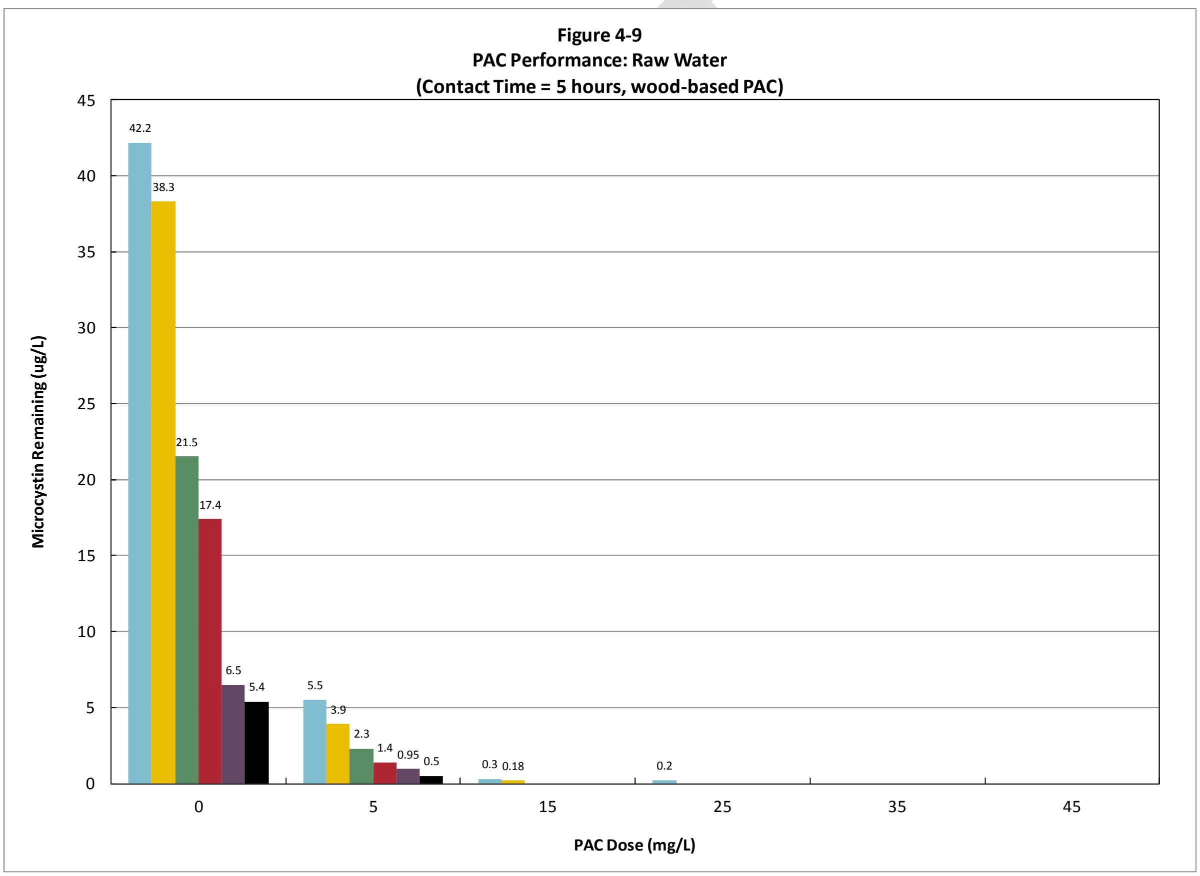
FIGURE 4–8

4.2.2.1 Low Service Pump Station – PAC Jar Testing Results

PAC jar testing on removal efficacy of microcystin was performed to simulate the approximately 5 hour contact time that occurs between the Low Service Pump Station and Collins Park WTP. The PAC the City currently uses from MeadWestvaco NUCHAR SA was used. Figure 4-9 shows microcystin removal rates based PAC feed rates 0, 5, 15, 25, 35 and 45 mg/L.

Figure 4-9: PAC Performance: Raw Water



The data shows that PAC is successful in removing large quantities of microcystin in the water. Of special note, the water that was tested was collected in December and is of higher quality than would be experienced in the summer months where higher TOC and algae will be present. So efficacy will be reduced some and would expect the removal rates to shift some to the right on Figure 4-9.

4.2.3 Collins Park WTP – Powder Activated Carbon (PAC) Improvements

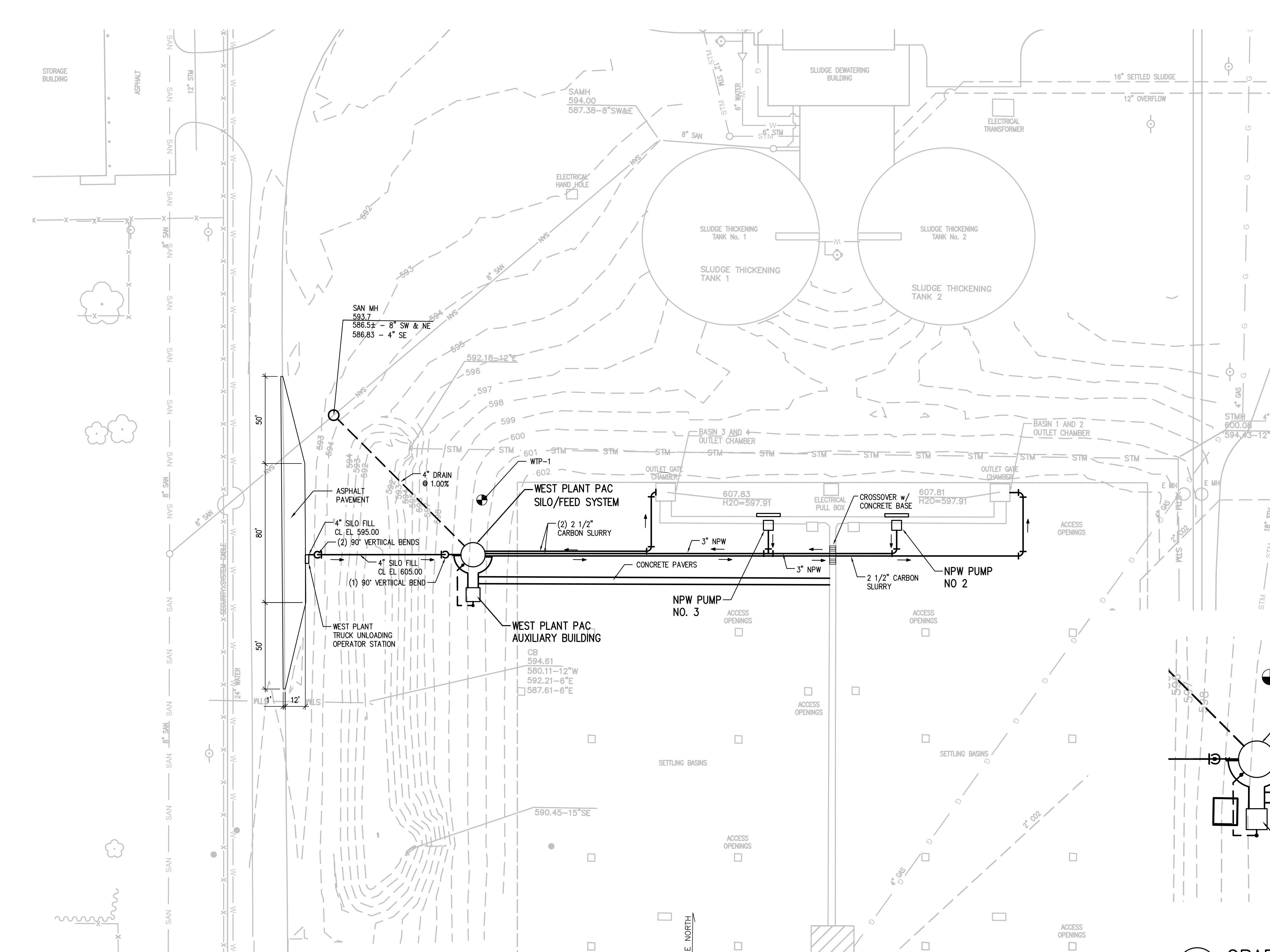
The City presently does not have capabilities to feed PAC at the Collins Park WTP facilities. This project will add the capability to feed PAC at the 3rd pass of the flocculation basin at a feed rate of up to 6 mg/L. This feed rate is the upper limit that can be handled without causing operational concerns with the filters. Originally it was planned to feed the PAC at the end of the recarbonation basins in the settled water conduits, however it was determined that the 7 to 15 minutes of detention time before contacting the filters was not adequate.

Planned improvements include the addition of two package PAC storage silo and feed systems, one for the West Plant and one for the East Plant, to provide the ability to feed PAC to the flocculation basins.

Each PAC silo and feed system will include an approximately 3,000 cu.ft. (45,000 lb) dry PAC storage silo, two (2) weight hopper bins on a scale, two (2) dry feeders, and a splitter box with two (2) eductors to convey the carbon to the 3rd pass of the flocculator basins. A splitter box will be provided to split the flow between flocculator basins 1 & 2 and basins 3 & 4. Process water will be supplied by four (4) new NPW well type pumps located in the recarbonation basins.

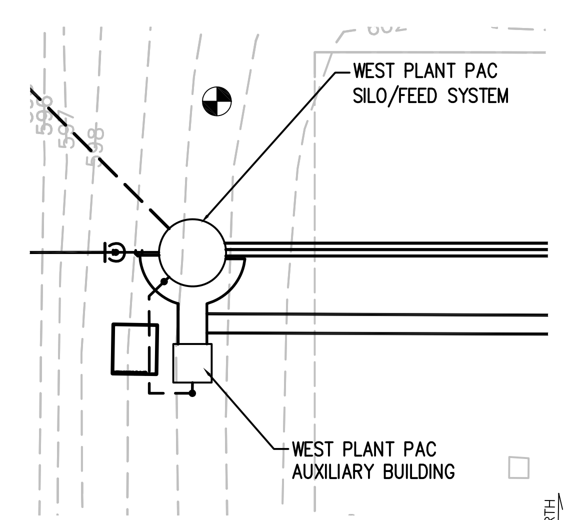
Using an average HAB seasonal feed rate of 3 mg/L of PAC and conservative summertime monthly average flow of 105 MGD equates to approximately 34 days of combined storage capacity.

Figures 4-10 and 4-11 show the West and East Plant site plans of these improvements. Figures 4-12 through 4-14 show the silo and feed systems layout and schematic of the new PAC feed improvements.



1 PIPING PLAN 05

SCALE: 1" = 30'



2 GRADING PLAN 05

SCALE: 1" = 20'

1. CARBON SLURRY AND NPW PIPING SHALL SLOPE CONTINUOUSLY DOWNWARD FROM SILO TO SETTLING BASINS.
2. PROVIDE CLEANOUTS AT ALL BENDS IN CARBON SLURRY PIPING, AND EVERY 100 LF OF STRAIGHT PIPING.
3. ON SETTLING BASINS, MAXIMUM ALLOWABLE VEHICLE WEIGHT (LOADED) IS 15,000 POUNDS. EXCAVATION MAY BE DONE USING A TRENCHER OR SKID STEER IF WITHIN WEIGHT LIMIT. BACK HOES AND EXCAVATORS ARE NOT PERMITTED ON TOP OF BASINS. NO MORE THAN TWO VEHICLES, INCLUDING EXCAVATION EQUIPMENT, ARE ALLOWED ON THE BASINS AT ONE TIME.
4. DEPTH OF SOIL COVER ON TOP OF SETTLING BASINS IS APPROXIMATELY 24 INCHES.
5. THERE IS CLAY FIELD TILE ON TOP OF SETTLING BASINS. WHEN ENCOUNTERED, IF CLOGGED, AS DETERMINED BY OWNER, IT SHALL BE REMOVED, IF NOT CLOGGED IT SHALL BE REPAIRED AT NO ADDITIONAL COST TO OWNER.

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HAB CHEMICAL FEED IMP.

WEST PLANT PAC SYSTEM SITE PLAN

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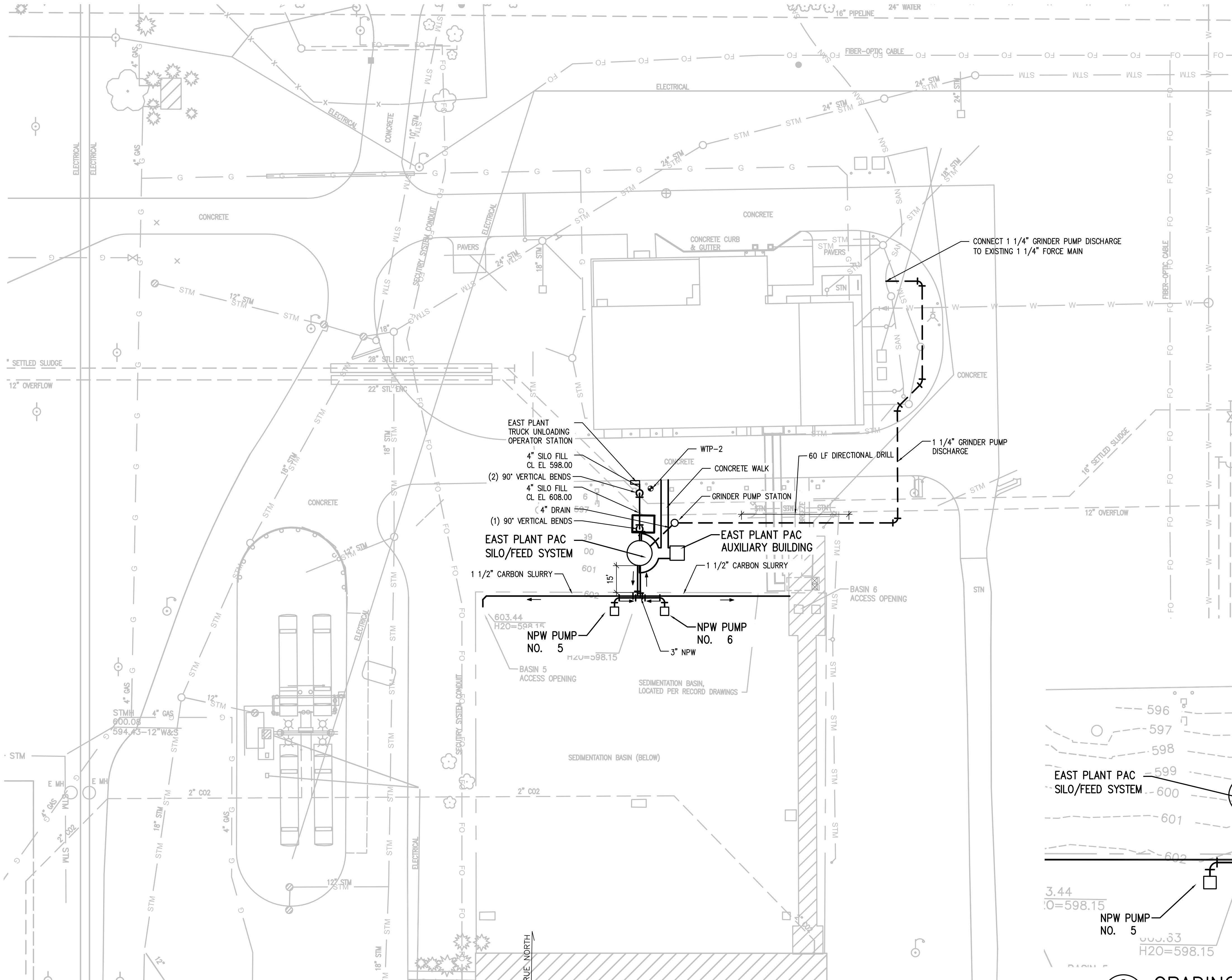
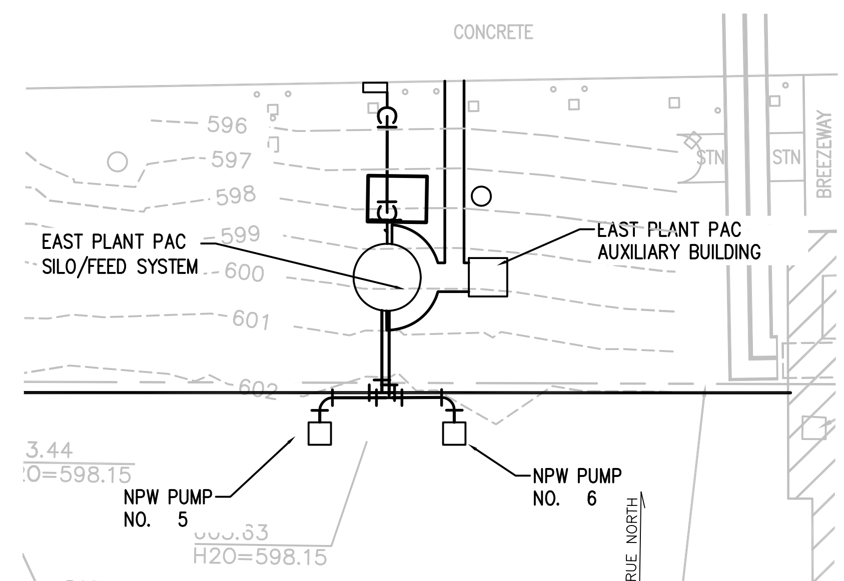
FIGURE 4–10

User: CKIRSCH Spec: AUS-NCSMOD File: G:\PROJECT\TO000747.M002\DRAWING\PAC\FIGURE 4-10.DWG Scale: 1:1 SavedDate: 2/3/2015 Time: 11:27
Plot Date: Kirsch, Clair; 2/4/2015; 10:36 ; Layout: C-05



1 PIPING PLAN
06
SCALE: 1" = 30'



2 GRADING PLAN
06
SCALE: 1" = 20'

1. CARBON SLURRY AND NPW PIPING SHALL SLOPE CONTINUOUSLY DOWNWARD FROM SILO TO SETTLING BASINS.
2. PROVIDE CLEANOUTS AT ALL BENDS IN CARBON SLURRY PIPING, AND EVERY 100 LF OF STRAIGHT PIPING.
3. ON SETTLING BASINS, MAXIMUM ALLOWABLE VEHICLE WEIGHT (LOADED) IS 15,000 POUNDS. EXCAVATION MAY BE DONE USING A TRENCHER OR SKID STEER IF WITHIN WEIGHT LIMIT. BACK HOES AND EXCAVATORS ARE NOT PERMITTED ON TOP OF BASINS. NO MORE THAN TWO VEHICLES, INCLUDING EXCAVATION EQUIPMENT, ARE ALLOWED ON THE BASINS AT ONE TIME.
4. DEPTH OF SOIL COVER ON TOP OF SETTLING BASINS IS APPROXIMATELY 24 INCHES.
5. THERE IS CLAY FIELD TILE ON TOP OF SETTLING BASINS. WHEN ENCOUNTERED, IF CLOGGED, AS DETERMINED BY OWNER, IT SHALL BE REMOVED, IF NOT CLOGGED IT SHALL BE REPAIRED AT NO ADDITIONAL COST TO OWNER.

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COLLINS PARK WTP
HAB CHEMICAL FEED IMP.

EAST PLANT PAC SYSTEM SITE PLAN

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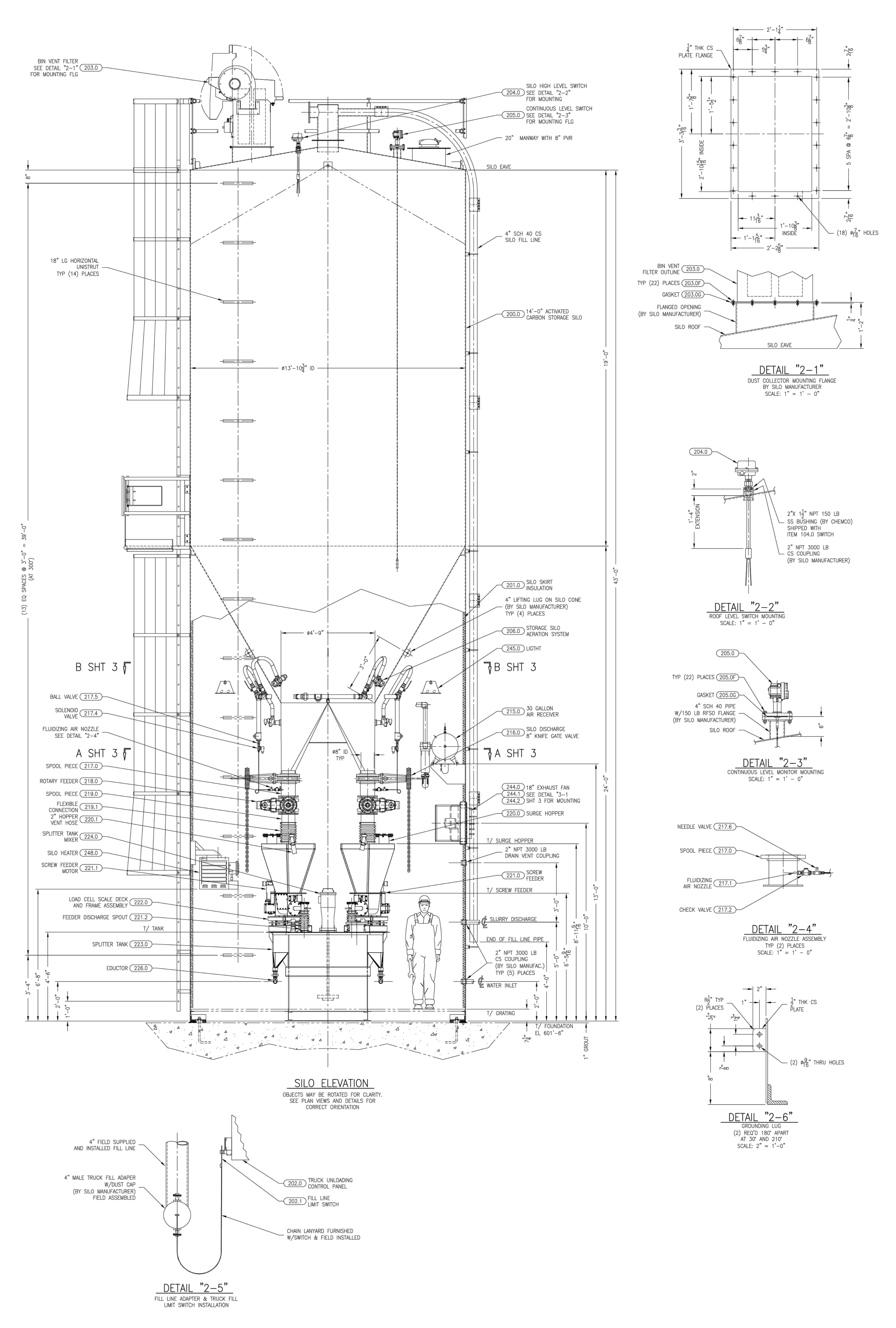
FIGURE 4-11



FEB 2015

FIGURE 4-12

WEST PLANT PAC SILO SECTION (TYPICAL EAST PLANT)

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COLLINS PARK WTP
HAB CHEMICAL FEED IMP.

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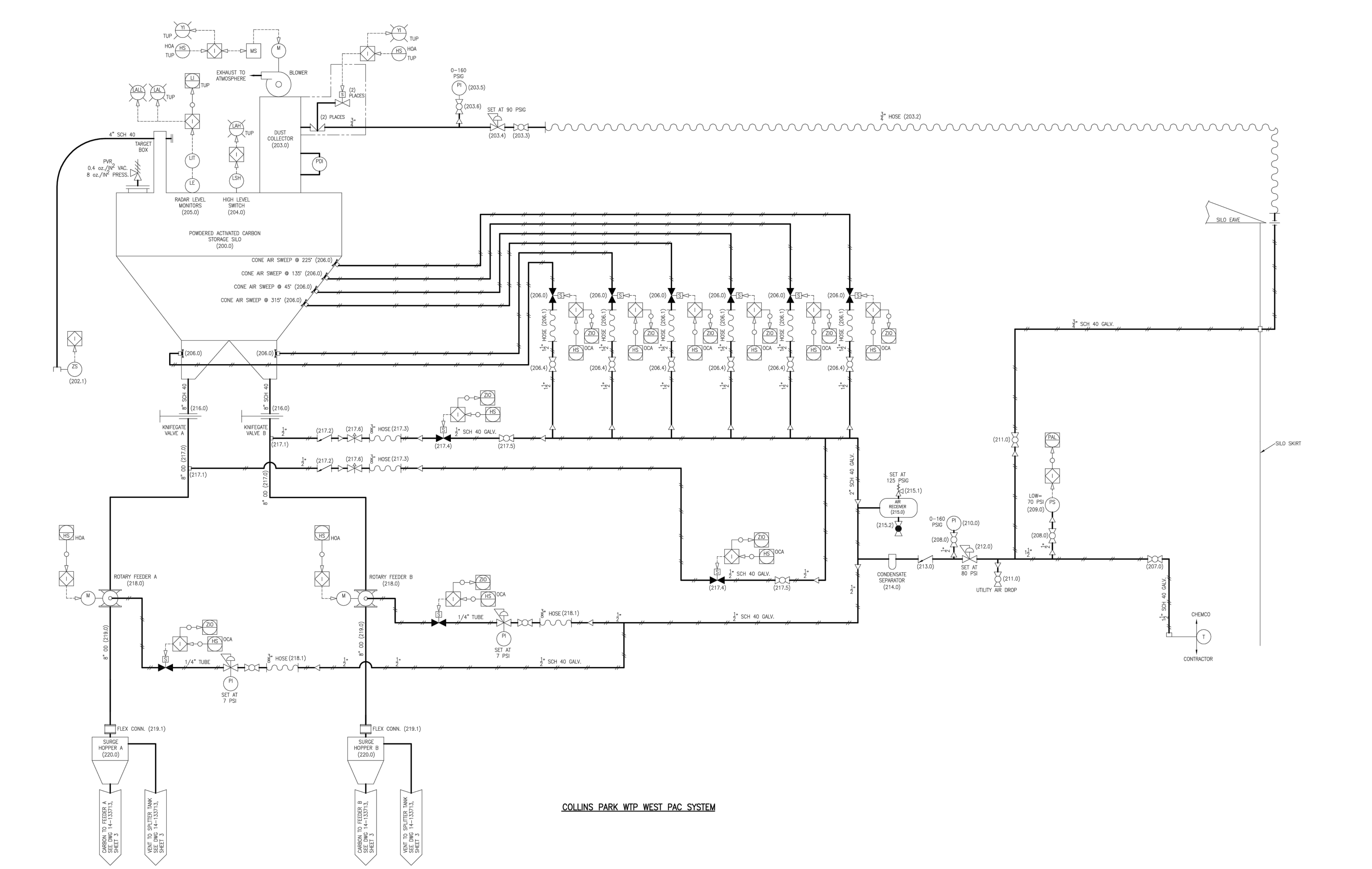
User: CKIRSCH Spec: AUS-NCSMOD File: G:\PROJECT\TO000747.M002\DRAWING\PAC\FIGURE 4-12.DWG Scale: 1:1 SavedDate: 2/3/2015 Time: 14:24
Plot Date: Kirsch, Clair; 2/4/2015; 10:40 ; Layout: D-12



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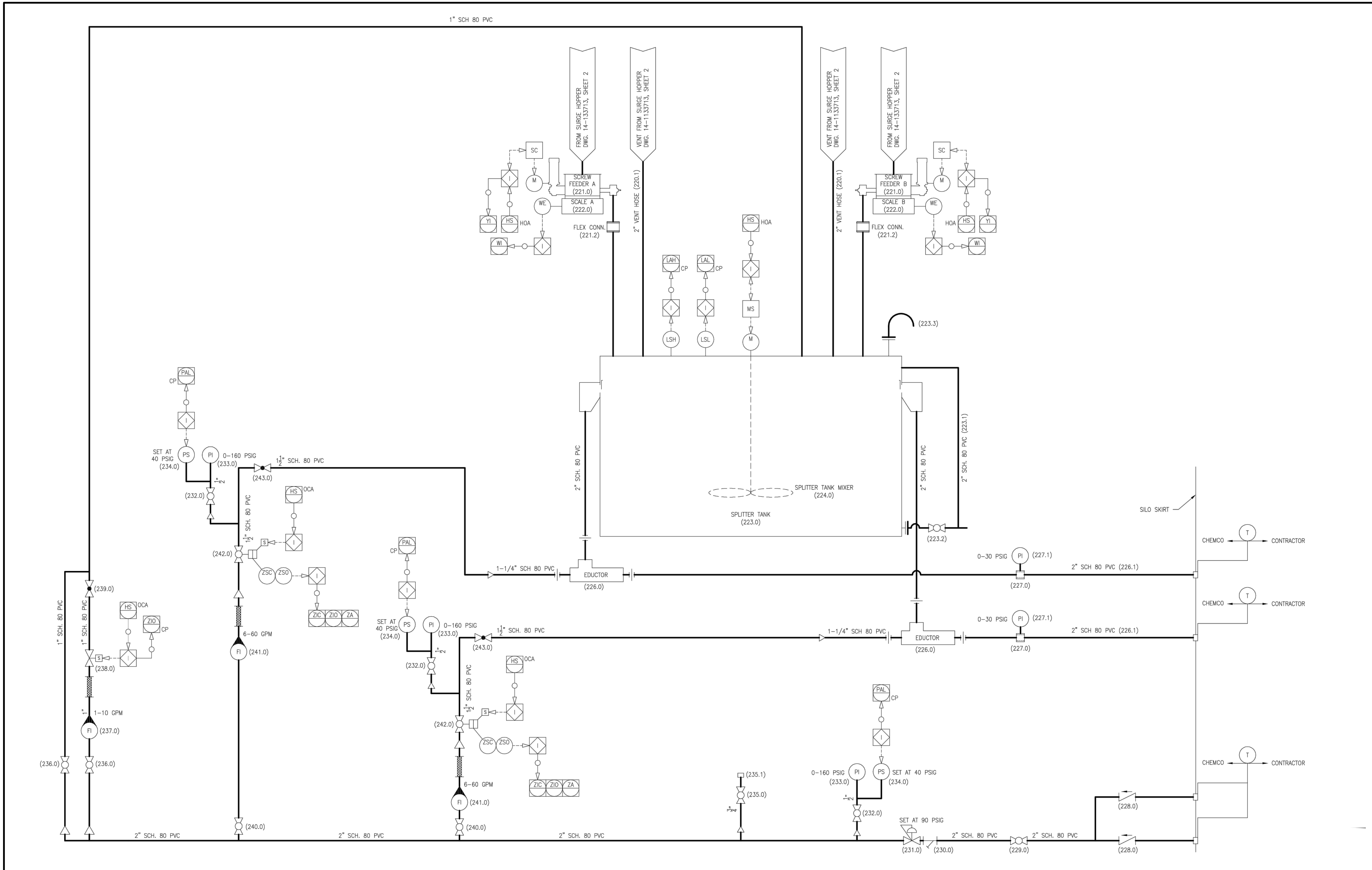
CITY OF TOLEDO
COLLINS PARK WTP
HAB CHEMICAL FEED IMP.

WEST PLANT PAC FEED PROCESS DIAGRAM

FEB 2015

FIGURE 4–13



CITY OF TOLEDO
COLLINS PARK WTP
HAB CHEMICAL FEED IMP.

WEST PLANT PAC FEED PROCESS DIAGRAM (TYPICAL EAST PLANT)

FEB 2015

FIGURE 4-14

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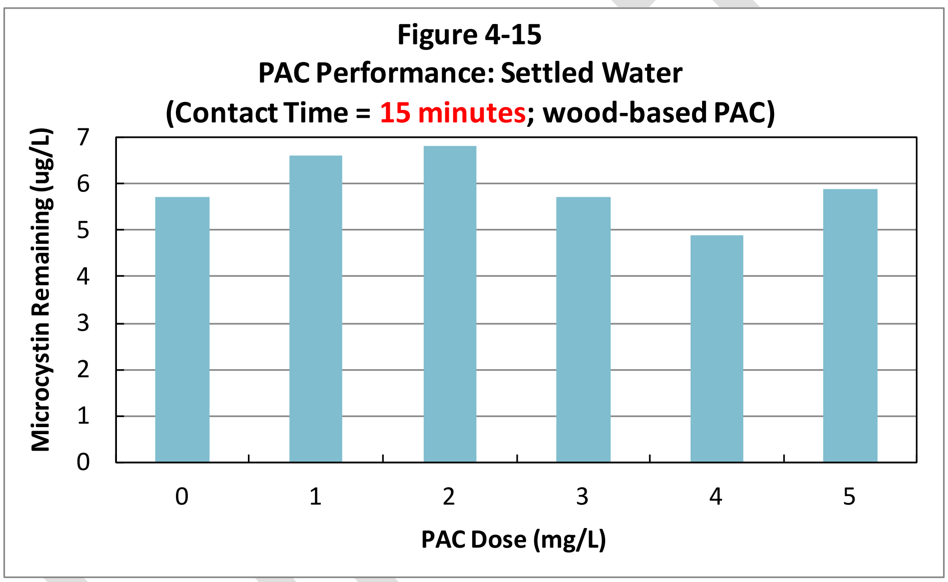
User: CKIRSCH Spec: AUS-NCSMOD File: G:\PROJECT\TO000747.M002\DRAWING\PAC\FIGURE 4-14.DWG Scale: 1:1 SavedDate: 2/3/2015 Time: 13:30
Plot Date: Kirsch, Clair; 2/4/2015; 10:41 ; Layout: D-06

4.2.3.1 Collins Park WTP - PAC Jar Testing Results

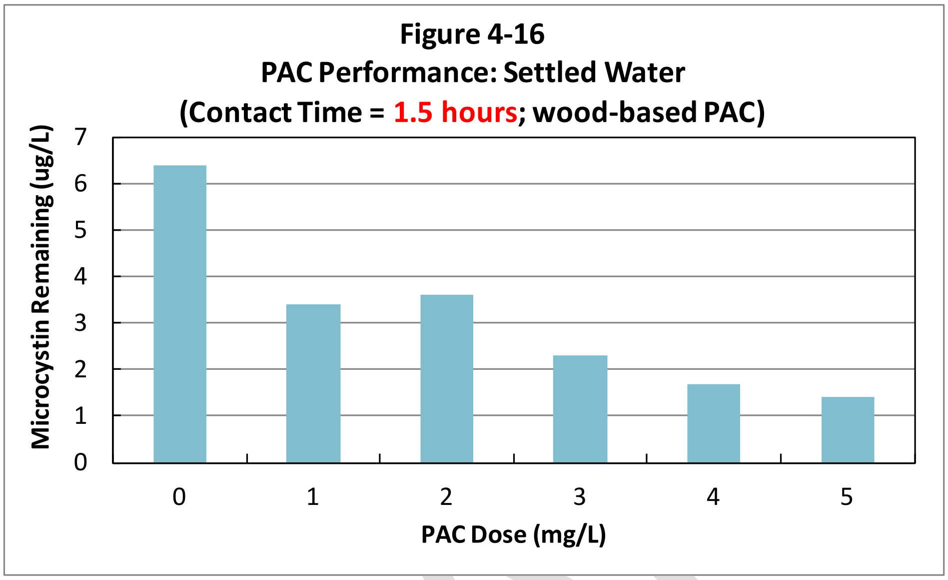
Initially, PAC jar testing on removal efficacy of microcystin was performed to simulate the approximately 7 – 15 contact time that occurs in the settled water conduit between the recarbonation basins and filters. Figure 4-15 shows microcystin removal rates based PAC feed rates 0, 1, 2, 3, 4 and 5 mg/L using the wood based PAC the City uses MeadWestvaco NUCHAR SA.

Figure 4-15: PAC Performance, Settled Water, 15 minutes



Subsequently, PAC jar testing on removal efficacy of microcystin was performed to simulate a 1.5 hour contact time to observe the difference in performance. Figure 4-16 shows microcystin removal rates based PAC feed rates 0, 1, 2, 3, 4 and 5 mg/L using the wood based PAC the City uses MeadWestvaco NUCHAR SA.

Figure 4-16: PAC Performance, Settled Water, 1.5 hours



This data indicates that when extending the PAC contact time microcystin removal rates significantly increase. It should be noted that lignite based PAC was also tested but did not perform as well as the wood based. With these favorable results, testing proceeded to simulate adding 6 mg/L of PAC at the beginning of the 3rd pass of the flocculation tanks. Figure 4-17 shows microcystin removal rates based on a PAC feed rates of 6 mg/L and different levels of alum and lime feed rates using the wood based PAC the City uses MeadWestvaco NUCHAR SA.

Figure 4-17 shows a significant reduction in microcystin with potentially a lesser reduction when overfeeding alum with 4.0 grains/gallon. While performing this testing, concern was expressed about how much of the PAC would be removed by the floc in suspension in the 3rd pass. Therefore, 1 L of the jar test was filtered to identify how much PAC remained after the 3rd pass flocculation and 1.5 hour contact time. Figure 4-18 shows the filtering results that indicates a significant amount of the PAC remains in suspension and available for further microcystin treatment in the sedimentation basins.

Figure 4-17: PAC Performance, 3rd Pass Flocculation, Microcystin Test 1 µg/L

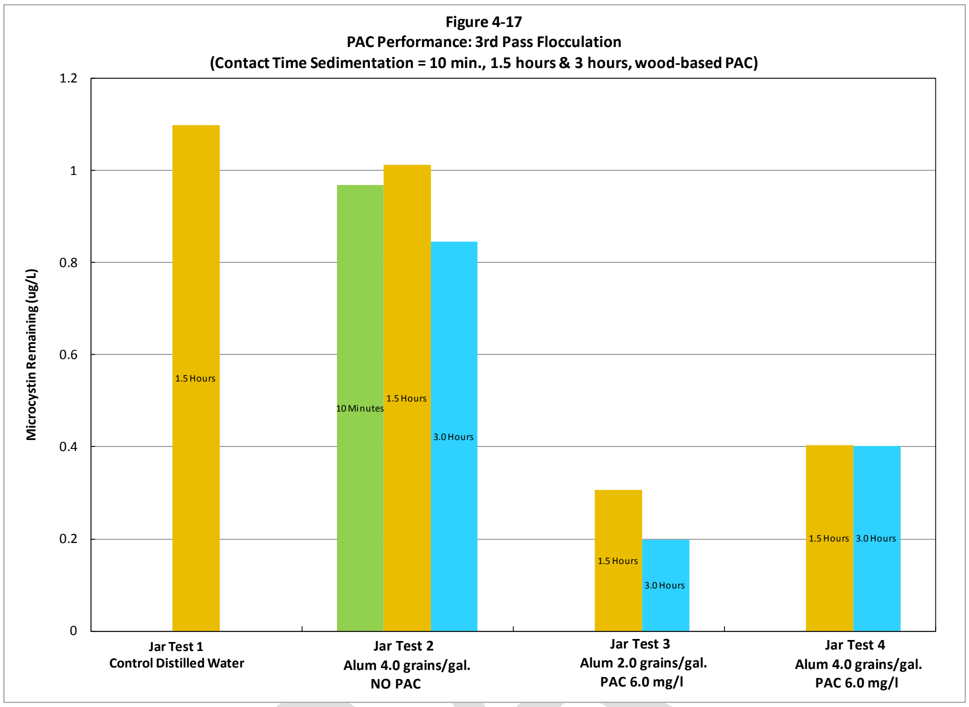
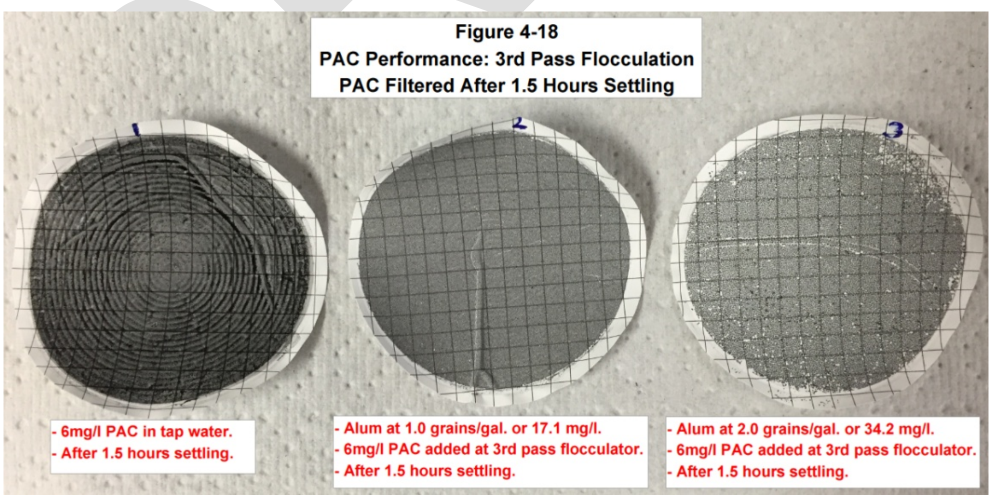
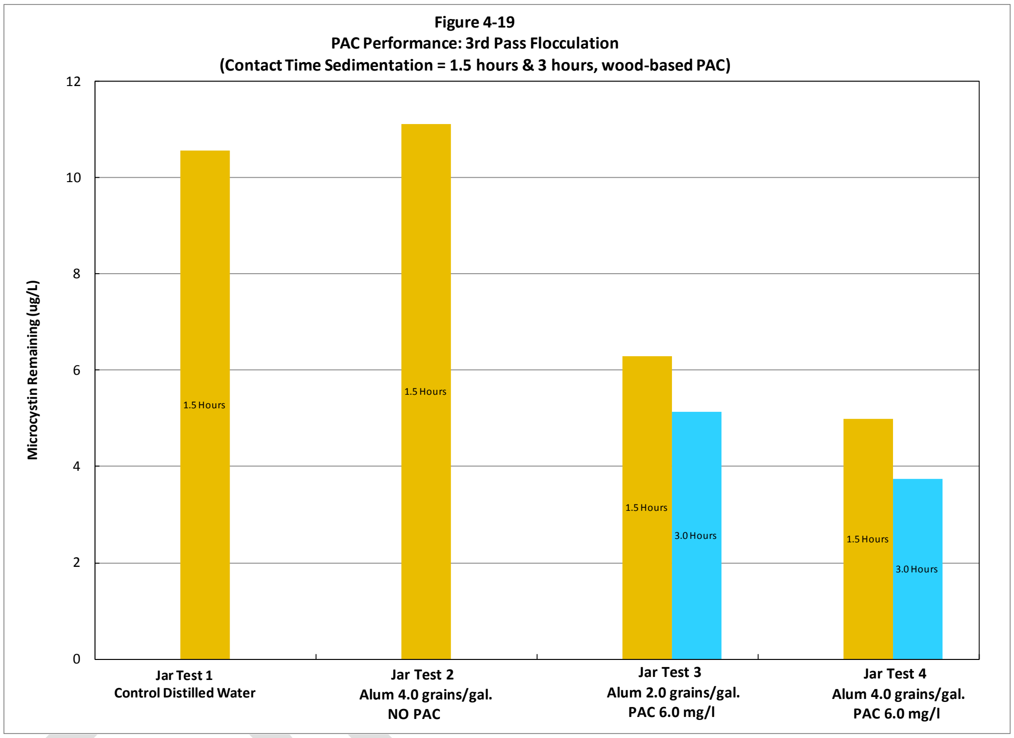


Figure 4-18: PAC Filtered After 1.5 Hours Settling



In order to see how microcystin removal rates would be at an elevated rate of 10 µg/L, additional jar testing was performed and is shown in Figure 4-19. At this elevated concentration, microcystin removal rates of 55 – 65% are observed after 3 hours of sedimentation time.

Figure 4-19: PAC Performance, 3rd Pass Flocculation, Microcystin Test 10 µg/L



Interestingly, the conventional alum lime treatment process does not appear to remove dissolved microcystin from the water.

4.2.3.2 Collins Park WTP – PAC Filter Demonstration Study

The City is currently underway with performing a filter demonstration study and testing to observe the performance of filter when PAC is introduced and present in the settled water. The City's checklist for temporary PAC feed demonstration protocol document and OEPA approval letter dated December 3, 2014 is provided under Appendix B.

Once all 60 days of winter data collection is complete, a report will be submitted to OEPA for approval. A second 2 week demonstration study will be performed once the PAC systems for the East & West Plants are completed and placed into operation during the HAB season. A final report shall be submitted to OEPA within 30 days of the completion of this second study.

4.2.4 Alternative 1 - Advantages/Disadvantages

Some advantages and disadvantages to this alternative are presented in Table 4-1.

Table 4-1: HAB Treatment Advantages and Disadvantages

Advantages
Available for operation by next algal season in July 2015.
Increasing LSPS storage by 3 times and feed capacity from 15 mg/L to 40+ mg/L.
LSPS PAC system highly effective with ~5 hour contact time in raw water mains.
WTP PAC system will provide a secondary treatment barrier.
Disadvantages
PAC less effective at WTP with less contact time and partial removal by floc particles.
Potential to lyse more algal cells than PAC system may treat.
May not be able to achieve 100% removal rate of microcystin.
High PAC feed rates and chemical operating costs.

Generally, the PAC feed systems at the WTP cannot achieve 100% removal of microcystin and are viewed as a short term treatment measure and will ultimately be replaced with a long term option of either ozone or GAC post filtration contactors.

4.2.5 Probable Project Costs

Construction bids have been received and contracts executed with contractors to complete construction of alternative 1 by July 2015. Table 4-2 provides a summary of these costs including 15% construction contingencies, engineering and technical services.

Table 4-2: Alternative 1, Existing PAC and Potassium Permanganate System Enhancements

Project Description	Probable Cost
Construction Projects	
PAC Feed Equipment for HAB Chemical Feed Improvements	$1,559,184
HAB Chemical Feed Improvements	$1,814,300
Engineering & Technical Services	
Planning	$19,500
Design Engineering	$540,000
Construction Engineering and Resident Project Representation	$480,000
Financial & Contingencies	
Loan Application Fee	$68,472
15% Contingencies	$659,022.60
Total	**$5,140,478.60**

Since these improvements are already in the process of being implemented, a life cycle cost effective analysis was not performed.

4.3 Alternative 2 - Ozone

Due to the short time frame that was available to perform the necessary bench testing and establish design parameters for the addition of ozone in providing an additional barrier for the treatment of algal toxins, it was decide to present the information on the ozone evaluation as a separate design document in the interest of time. This report is presented in Appendix C.

4.4 Alternative 3 - Post filtration Granular Activated Carbon (GAC)

In similar nature to the work for the ozone evaluation, the evaluation associated with the bench testing evaluation and conceptual design for the use of Granular Activated Carbon (GAC) in providing an additional treatment barrier for algal toxins is presented in Appendix D.

5. Upgrades to Existing Facilities

The section discusses improvements and modifications to the existing West and East Plant to:

- improve the hydraulic flow splitting and metering between individual basins,
- upgrade the flocculation process and replace aging equipment,
- add the Sedimentation Basin outlet troughs,
- replace the aging sludge collection equipment,
- enlarge the recarbonation facilities in the West Plant, and,
- standardize the process parameters across both the West and East Plant as much as practical for consistency with proposed Basins 7 and 8.

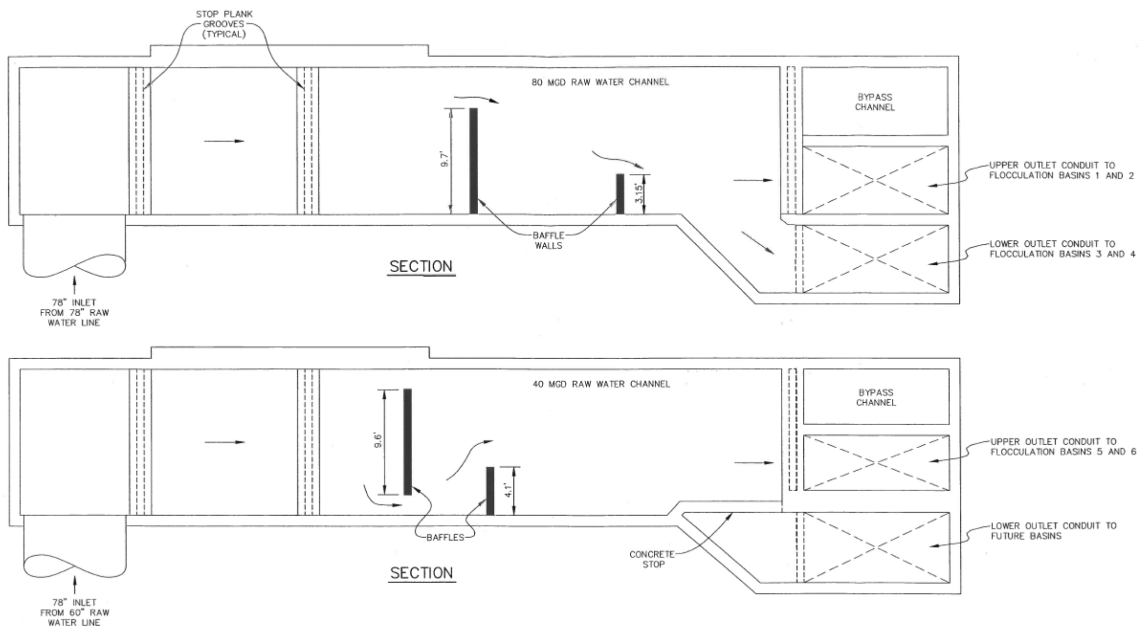
5.1 Rapid Mixing

Pretreated water from the low service pumping station is conveyed to the WTP, which is essentially divided into two separate treatment plants capable of independent operation. Flow enters the plant through two interconnected raw water rapid mix channels that split flow between the East Plant (40 MGD) and West Plant (80 MGD) as shown in Figure 5-1.

Figure 5-1: Rapid Mix Channel

Liquid alum is applied to each raw water channel about 20 feet downstream of the inlet. Each channel is equipped with alternating baffles to impart mixing energy to increase dispersion of alum into the water. As alum is mixed in the raw water, the coagulation process begins. Alum-treated water reaches the end of each raw water channel and enters either the upper or lower outlet conduit, where it is then directed to the appropriate flocculation basins for further treatment as shown in Figure 5-2.

Figure 5-2: Rapid Mix Channel, Section View



The following observations and deficiencies have been identified for the rapid mix:

- Lower hydraulic mixing occurs in the 40 MGD channel (since channel is sized for another 40 MGD addition in flow). In addition, mixing intensity varies with flow.
- Additional baffling or alternate mixing is needed to improve coagulation in the rapid mix channels.

The capacity associated with this area of the treatment process requires that a minimum of two units be provided. The Collins Park Water Treatment Plant does satisfy this minimum requirement as it has two (2) units. The Ohio Capacity Rule recommends that a mixing intensity G-Value of 750 be attained and that the detention time not exceed 30 seconds. Given the current arrangement of mixing in the raw water channels, mixing intensity G-Values have been projected at levels lower than 750. The detention time between the existing baffles in each of the raw water channels is approximately 10 seconds.

The plant generally achieves good mixing and floc formation with the current channels. However, it is recognized that additional mixing intensity and improved chemical application may be beneficial to this process.

The City is current underway with implementing design and constructing improvements to improve rapid mix conditions. These improvements will consist of:

- Adding an alum overhead weir trough with v-notches to disperse alum feed more evenly across the channel.
- Adding additional baffling to increase G-Values.
- Study routing all plant flow through one rapid mix channel at lower flow conditions to increase G-Values.

The recommended additional baffle wall improvements include:

- West Plant (80 MGD) – Add an intermediate "under" baffle between the two "over" baffles.
- East Plant (40 MGD) – Replace existing "under" baffle with one that extends to 1.1 feet from floor of channel to bottom of baffle and placed 4 feet apart.
- G-Values at 120 MGD:
  - West Plant (80 MGD) – 753 sec^{-1}.
  - East Plant (40 MGD) 764 sec^{-1}.
  - At lower flows G-Values cannot be attained.
- Hydraulic Impact – At 120 MGD water level at head end of the raw water channel is estimated to be 1.7 feet below concrete floor.

OR

- Divert All Flow Through West Plant (80 MGD) Rapid Mix Channel.
- Relocate first baffle to 8 feet from the second baffle.
- G-Values at 120 MGD: 1,008 sec^{-1}.
- G-Values at 80 MGD: 740 sec^{-1}.
- Hydraulic Impact – At 120 MGD water level at head end of the raw water channel is estimated to be 1.52 feet below concrete floor. This approach will need to be approached with caution because of limited freeboard.

5.2 Flocculation

5.2.1 West Plant

Improvements to Flocculation Basins 1 through 4 would include modifications to the inlet piping and gates, replacement of flocculation equipment, and associated demolition.

Modifications to the inlet piping for Basins 1 and 2 includes demolition of the inlet splitter box and gates and piping improvements as described in the Plant Hydraulics section above. The current piping consisting of a common 60-inch venturi meter followed by a common 60-inch butterfly valve and sluice gates at the basins does not allow for metered and controlled flow into each basin. As discussed above, it is recommended that the 60-inch influent pipe is divided into parallel 42-inch inlet pipes having a magnetic flow meter and motorized control valve for each basin. This configuration will provide the ability to meter and control the flow to each basin separately.

For Basins 3 and 4, the future 60-inch raw water pipe could be connected to the 9-ft wide raw water conduit (RWC) by construction a concrete box extending from the RWC. This 60-inch piping would split into two separate 42-inch pipes with meters and valves for each basin. The RWC beyond the connection box could be “walled-off” internally.

As recommended for Basins 7 and 8, horizontal paddlewheel (HPW) flocculator equipment would replace the existing HPW equipment. The new flocculation equipment would have RFP blades and be VFD driven.

It was noted in the 2011 20-Year Master Plan and Needs Assessment Report (ARCADIS) that significant solids accumulated in the effluent channel of the Flocculation Basins. This effluent channel follows the third stage of flocculation and has no impact on the treatment process. Since the plant operational staff has not observed any negative treatment impacts from the solids accumulation, no corrective action is recommended.

The process parameters for West Plant Flocculation Basins 1 through 4 are unchanged.

5.2.2 East Plant

Improvements to Flocculation Basins 5 and 6 include modifications to the inlet piping and valves, addition of interior walls between stages, replacement of flocculation equipment, and associated demolition.

Modifications to the inlet piping will include the removal the existing 60-inch venturi, common 60-inch butterfly valve, piping and individual 60-inch butterfly valves to each basin. As described above, new parallel 42-inch pipes with meters and valves would split the flow to Basins 5 and 6. The new 42-inch piping would continue through the existing inlet flume and enter directly into the first stage of flocculation.

Interior walls would be constructed to form stages within the basins. To facilitate basin draining the openings between stages would be full depth and as wide as the stage channel width. HPW flocculator equipment is recommended to replace the existing HPW equipment as discussed above. The design parameters for the modified Flocculation Basins 5 and 6 are in Table 5-1 and included in Appendix A.

Table 5-1: Modified Flocculation Basin 5 and 6 Parameters

Parameter	Value
Flow per Basin (MGD)	20
Number of Stages	3
Stage Length (ft)	83.42
Stage Width (ft)	14.97
Maximum WSEL	600
Bottom (elev)	582
SWD (ft)	18
Volume (gal)	504,527
Det. Time (min)	36.33
Flow thru Velocity (ft/min)	6.90
Basin overflow weir (elev)	600.83

5.3 Sedimentation /Softening

5.3.1 West Plant

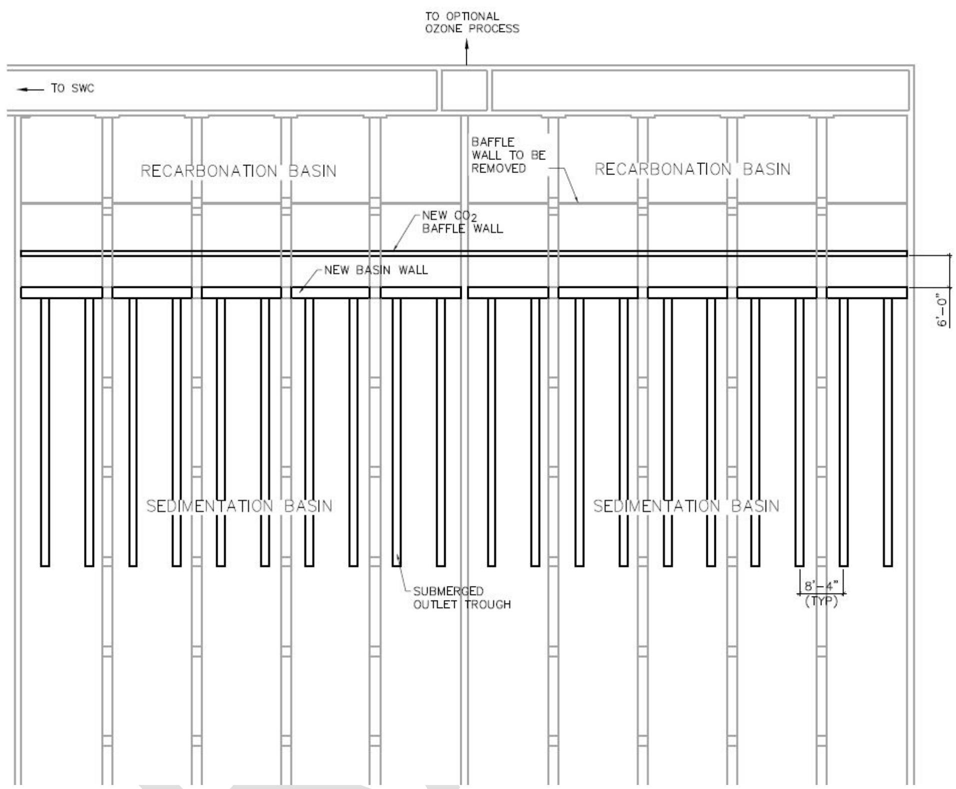
Improvements to the Sedimentation Basins 1 through 4 include the replacement of the sludge collection equipment, addition of outlet troughs and a superstructure over the outlet troughs, interior wall construction, and associated demolition.

The existing sludge collection equipment is recommended for replacement. The basins are equipped with 5 longitudinal chain and flight sludge collectors and a one chain and flight cross collector located at the influent end of the basin.

Submerged orifice outlet troughs would be provided as discussed for Basins 7 and 8. In each basin there would be ten submerged-orifice troughs providing weir length to meet the 10-States Standards requirements for a maximum of 20,000 gpd/lf weir loading. The troughs would be supported by the FRP beams running between the existing columns in the basin.

A wall will be constructed across the existing basin at the outlet end of the Sedimentation Basin to support of the outlet troughs flowing into recarbonation. It is recommended to construct this basin wall one column-line (17’-0”) upstream of the existing plaster baffle wall in the basin as shown in Figure 5-3. The plaster baffle wall will be removed. With this modification, the Sedimentation Basins meet the appropriate softening basin requirements on surface overflow rate and length-to-width ratio.

Figure 5-3: Sedimentation and Recarbonation Basin 1 – 4 Modifications



As discussed for Basins 7 and 8, a superstructure of sufficient height would be constructed over the troughs to allow for access and maintenance to the troughs. Removal of the existing top slab over these troughs is required.

Based on a maximum WSEL 600.00, the modified Sedimentation Basin 1 through 4 parameters are shown in Table 5-2 and Appendix A.

Table 5-2: Modified Sedimentation Basin 1 – 4 Parameters

Parameter	Value
Flow per Basin (MGD)	20
Length (ft)	253.92
Width (ft)	83.5
Maximum WSEL	600.0
Bottom (elev)	585.33
SWD (ft)	14.67
Volume (gal)	2,326,564
Det. Time (hrs)	2.79
Surface Area (sf)	21,202
SOR (gpm/sf)	0.66
Flow thru Velocity (ft/min)	1.52
Weir Length	100
Number of Weirs	10
Total Basin Weir Length (ft)	1000
Weir Loading Rate (gpd/lf)	20,000
L/W Ratio	3.04

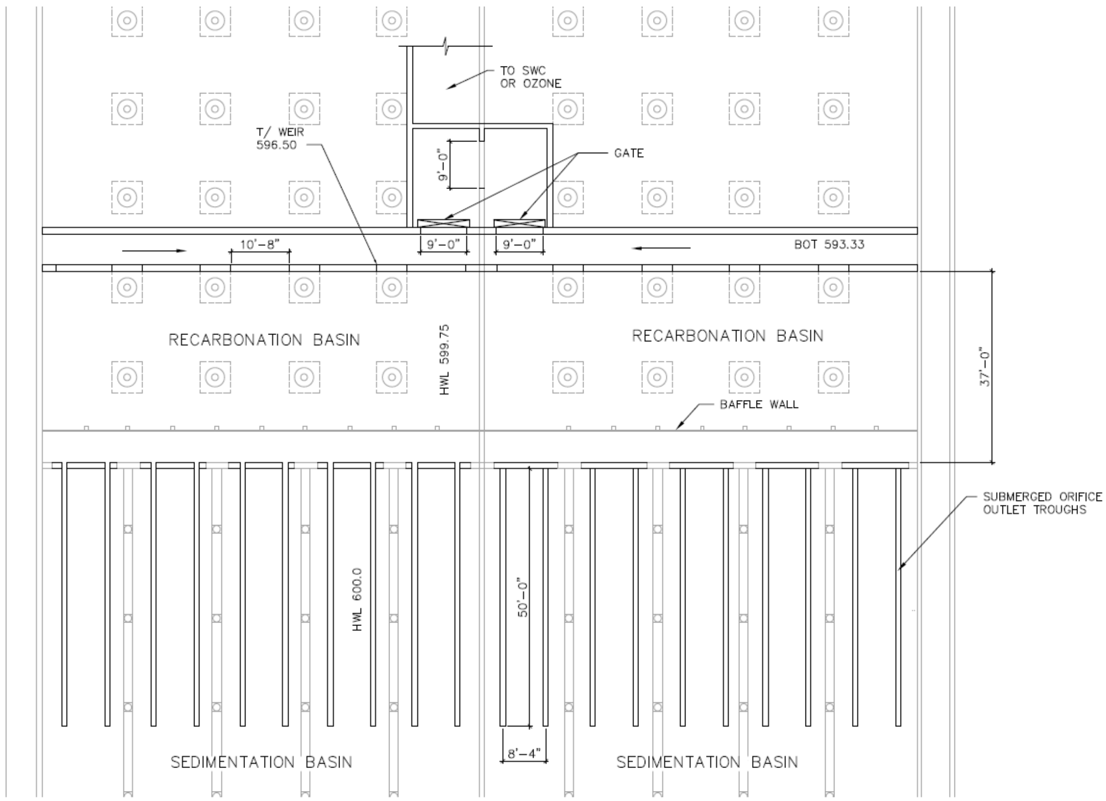
5.3.2 East Plant

Improvements to the Sedimentation Basins for the East Plant Basin 5 and 6 include the replacement of the sludge collection equipment, addition of outlet troughs and a superstructure over the troughs, and associated demolition.

The existing sludge collection equipment is recommended for replacement. The basins are equipped with five longitudinal chain and flight sludge collectors, a chain and flight cross collectors located at the influent end, and a second cross collector midway along the basin.

Submerged orifice outlet troughs would be provided as discussed for Basins 1 through 4 above. The planned 8’-4” spacing of the trough outlets allows two of them to fit within each of the five existing 12’-5” existing submerged weir openings in the wall. The troughs would be supported by FRP beams running between the existing columns. The modification concept plan is shown in Figure 5-4.

Figure 5-4: Sedimentation and Recarbonation Basin 5 and 6 Modifications



Construction of a superstructure and removal of the existing top slab over the troughs area is required for access due to the low headroom over the troughs.

Based on a maximum WSEL 600.00, the modified Sedimentation Basin 5 and 6 parameters are shown in Table 5-3.

Table 5-3: Modified Sedimentation Basin 5 and 6 Parameters

Parameter	Value
Flow per Basin (MGD)	20
Length (ft)	284.25
Width (ft)	83.42
Maximum WSEL	600
Bottom (elev)	585.33
SWD (ft)	14.67
Volume (gal)	2,601,971
Det. Time (hrs)	3.12
Surface Area (sf)	23,712
SOR (gpm/sf)	0.59
Flow thru Velocity (ft/min)	1.52
Weir Length	100
Number of Weirs	10
Total Basin Weir Length (ft)	1000
Weir Loading Rate (gpd/lf)	20,000
L/W Ratio	3.41

5.4 Recarbonation Facilities

5.4.1 West Plant

The Recarbonation Basins in the West Plant are currently separated from the Sedimentation Basins by a plaster baffle wall at the last column-line before end of the basin. As discussed above, a wall would be constructed across the basin at the second-to-last column row in the existing basin. This would provide a Recarbonation Basin having a 33 feet length and making the basins roughly equivalent to the modified Recarbonation Basins 5 and 6 discussed below. Figure 5-1 shows the conceptual plan for location of the outlet troughs, wall, and future Recarbonation Basin area.

Of further consideration is that the keeping the Recarbonation basins within the existing basin footprint will preserve the available land area north of the basins for the construction of the optional ozone process. Of significant concern is that the existing sludge thickening tanks are located approximately 80 feet to the north.

The carbon dioxide diffusers and diffuser piping are recommended for replacement. Following the removal of the existing plaster baffle wall and wooden under-baffle, a new carbon dioxide under-baffle wall will be required at the outlet of the mixing zone.

Based on a maximum WSEL 600.00, the modified Recarbonation Basins 1 through 4 parameters are presented in Table 5-4 and Appendix A.

Table 5-4: Modified Recarbonation Basin 1 – 4 Parameters

Parameter	Value
Flow per Basin (MGD)	20
Total Basin Length (ft)	33
Basin Width (ft)	83.5
Maximum WSEL	600
Bottom (elev)	585.33
SWD (ft)	14.67
Mixing Zone Volume (gal)	54,976
Mixing Zone Det. Time (min)	3.96
Reaction Zone Volume (gal)	248,123
Reaction Det. Time (min)	17.86
Total Basin Det. Time (min)	21.82

5.4.2 East Plant

In order to provide space to the north of the East Plant basins for the optional ozone process, the recarbonation basin length could be reduced. The existing Recarbonation Basins have a length of 119 feet and provide 71 minutes of detention time. This recarbonation detention time can be significantly reduced without negatively impacting water stability. Figure 5-2 shows conceptual modifications to Basins 5 and 6 to shorten the existing Recarbonation Basins to provide approximately 24 minutes of total detention.

From each Recarbonation Basin, an outlet channel would direct the flow to an outlet gate located near the center wall between the basins. From this point, the flow can be conveyed to the SWC or north into the optional ozone process channels as shown on Figure 3-1.

The carbon dioxide diffusers and diffuser piping is recommended for replacement.

Based on a maximum WSEL 600.00, the modified Recarbonation Basin 5 and 6 parameters are presented in Table 5-5 and Appendix A.

Table 5-5: Modified Recarbonation Basin 5 and 6 Parameters

Parameter	Value
Flow per Basin (MGD)	20
Total Basin Length (ft)	37
Basin Width (ft)	83.42
Maximum WSEL	600
Bottom (elev)	585.50
SWD (ft)	14.5
Mixing Zone Volume (gal)	54,286
Mixing Zone Det. Time (min)	3.91
Reaction Zone Volume (gal)	280,480
Reaction Det. Time (min)	20.19
Total Basin Det. Time (min)	24.10

5.4.3 Probable Cost of Basins 1 through 4 Modifications and Upgrades

An opinion of probable construction cost for modifications and upgrades to Basins 1 and 4 is shown in Table 5-6. Modifications and upgrades to the existing West Plant Filter Building is not included in this estimate and are presented below separately. The probable cost opinion for modifications to Basins 1 through 4 includes the following components:

- Side Gallery: Modifications to inlet piping;
- Flocculation Basins: Replace HPW flocculator equipment;
- Sedimentation Basins: Replace sludge collection equipment, construct end of basin outlet wall and add outlet troughs, add superstructure over outlet troughs;
- Recarbonation Basins: Increase length of basin, remove existing plaster baffle wall, construct new carbon dioxide under-baffle wall, and replace carbon dioxide diffusers and piping.

The opinion of probable cost includes a 30% contingency on construction costs and other factors as listed in Table 3-5. Inflation during construction is estimated at 3% of total construction costs. Total construction cost and total project costs are rounded to the nearest $100,000.

Table 5-6: Basin 1 through 4 Upgrades Probable Cost

Component	Probable Cost
Substructures	$1,441,000
Superstructures	$1,524,000
HVAC / Plumbing	$434,000
Equipment	
Horizontal Paddlewheel Flocculators	$1,360,000
Sedimentation Basin Sludge Collectors	$2,320,000
Submerged Orifice Weir Troughs	$560,000
Carbon Dioxide Piping and Diffusers	$625,000
Piping, Valves, and Gates	$3,060,000
Electrical and I&C (10%)	$1,132,000
General conditions and mobilization (10%)	$1,246,0000
Subtotal	**$13,702,000**
Contingency (30%)	$4,111,000
Subtotal with Contingency	**$17,813,000**
Contractor insurance/bonding (5%)	$891,000
Contractor overhead/profit/general (15%)	$2,672,000
Total Construction Costs	**$21,400,000**
Construction costs inflation to construction midpoint (3%)	$642,000
Subtotal	**$22,042,000**
Engineering/Legal/Administrative (20%)	$4,408,000
Total Project Probable Cost	**$26,500,000**

5.4.4 Probable Cost of Basins 5 and 6 Modifications and Upgrades

An opinion of probable construction cost for modifications and upgrades to Basins 5 and 6 was prepared as shown in Table 5-7. Modifications and upgrades to the existing East Plant Filter Building is not included in this estimate and are presented below separately. The Basin 5 and 6 modifications probable cost opinion includes the following components:

- Side Gallery: Modifications to inlet piping;
- Flocculation Basins: Construction of internal basin walls, replace HPW flocculator equipment;
- Sedimentation Basins: Replace sludge collection equipment, modify basin outlet wall and add outlet troughs, add superstructure over outlet troughs;
- Recarbonation Basins: Shorten existing basin by constructing new wall and channels, replace carbon dioxide diffuser and piping; and,
- Construct channel to convey to Settled Water Conduit.

The opinion of probable cost includes a 30% contingency on construction costs and other factors as listed in Table 3-5. Inflation during construction is estimated at 3% of total construction costs. Total construction cost and total project costs are rounded to the nearest $100,000.

Table 5-7: Basin 5 and 6 Upgrades Probable Cost

Component	Probable Cost
Substructures	$1,238,000
Superstructures	$725,000
HVAC / Plumbing	$203,000
Equipment	
Horizontal Paddlewheel Flocculators	$680,000
Sedimentation Basin Sludge Collectors	$1,160,000
Submerged Orifice Weir Troughs	$280,000
Carbon Dioxide Piping and Diffusers	$313,000
Piping, Valves, and Gates	$690,000
Electrical and I&C (10%)	$529,000
General conditions and mobilization (10%)	$582,000

Component	Probable Cost
Subtotal	**$6,399,000**
Contingency (30%)	$1,920,000
Subtotal with Contingency	**$8,319,000**
Contractor insurance/bonding (5%)	$416,000
Contractor overhead/profit/general (15%)	$1,248,000
Total Construction Costs	**$10,000,000**
Construction costs inflation to construction midpoint (3%)	$300,000
Subtotal	**$10,300,000**
Engineering/Legal/Administrative (20%)	$2,060,000
Total Project Probable Cost	**$12,400,000**

5.5 Filtration

5.5.1 West Plant

The filters in the West plant were completely refurbished in the 1990s, including media and surface wash piping replacement. A filter evaluation program has been put into place following the 2011 master planning effort, and the evaluation has shown that the filter beds are generally in good conditions. Alkalinity testing across the filters has shown that there is essentially no alkalinity drop across the filters causing calcium carbonate deposition on the media, and as such no growth of the sand media layers. The filter evaluations performed to date shows that there has been a loss of media within the filter beds, generally in the range of 2-3 inches. This may be the result of the sloped washwater troughs and the available clearance beneath the trough for expansion of media during backwash. At the deep end of the troughs media may be expanding to a level where the rise velocity is such that media is more easily carried out of the filter. The filters are performing and achieving good filtered water turbidity meeting regulatory standards.

As noted, an air scour system is proposed for the new filters as the auxiliary wash means for filter backwashing. With the current media configuration for the existing filters, it is considered that the present surface wash system is adequate for auxiliary sour, although the evaluation has shown that the coverage range of the surface wash agitators may not be optimal, particularly since there has been media loss and the agitators are not positioned at the appropriate height above the media based on the agitator manufacturers specifications. Consideration should be given to upgrading the filter beds and underdrain systems to implement auxiliary air scour similar that proposed for the new filter addition. This is not a critical improvement and there is no immediate driver for this improvement; staff should seriously consider the air scour approach at the point in time when it is time to perform a complete media replacement.

In 1987, there was a Filter and Control Improvements project performed where many of the filter valves and actuators were replaced. The 2011 Master Plan and Needs Assessment report recommended that the valves and actuators be replaced since they are nearing the end of their useful life. This is certainly true of the valve actuators, but may not be the case for the valves themselves. It is recommended that a filter be isolated and valves inspected to confirm this is or is not the case.

As presented in the 2011 Master Plan Report, there are several miscellaneous improvements necessary within the West Plant filter gallery that address various filter piping and pumping modifications and filter gallery improvements as outlined below:

- Replacement of the backwash and surface wash supply piping and valves in the filter gallery due to the poor condition and significant leaks and corrosion.
- Replacement of two backwash pump cone valves
- Replacement of the rate of flow controllers on the backwash and surface wash headers
- Refurbishment of the elevated storage tank fill pump
- Replacement of the plant water piping
- Painting of the process piping within the filter gallery
- Repair of spalled concrete areas and cracks within the gallery.

5.5.2 East Plant

The existing filters in the East Plant are as originally constructed. As with the West plant, consideration should be given to the installation of new underdrains and utilizing air scour auxiliary wash when the decision is made to rebuild the filter beds in their entirety. Surface wash piping for the filters is in a deteriorated condition and should be replaced if the filters are not to undergo a major renovation to implement air scour.

Similar to the West Plant, there were several miscellaneous improvements outlined in the 2011 Master Plan Report covering necessary work in the East Filter gallery as follows:

- Replacement of the backwash and surface wash supply piping and valves in the filter gallery due to the poor condition and significant leaks and corrosion.
- Refurbishment of two backwash pumps and pump cone check valves
- Replacement of the rate of flow controllers on the backwash and surface wash headers
- Replacement of the plant water piping
- Painting of the process piping within the filter gallery
- Repair of spalled concrete areas and cracks within the gallery.

5.5.3 West and East Filter Building Upgrade Probable Costs

Probable costs for the West and East Filter Buildings are shown in Table 5-8.

Table 5-8: West and East Filter Building Upgrade Probable Costs

Component	Probable Cost
Replace and refurbish washwater piping, valves, actuators (East and West Plants)	$15,663,000
Refurbish backwash pumps (West Plant)	$263,000
Replace backwash pump cone check valves (East Plant)	$422,000
Replace washwater cone check valves (East Plant)	$422,000
Replace washwater rate-of-flow controllers	$168,000
Replace surface wash rate-of-flow controller	$168,000
Replace filter valves and actuators (East and West mgd plant)	$10,980,000
Pipe gallery rehabilitation and repainting	$295,000
Refurbish elevated storage tank fill pump near backwash pumps	$130,000
Process piping painting and identification	$529,000
Replace plant water piping	$1,180,000
Raw water flow splitting valve replacement (East plant)	$161,000
Piping galleries - concrete repairs for spalling / cracking (East and West Plants)	$445,000
Subtotal	**$30,826,000**
Contingency (30%)	$9,248,000
Subtotal with Contingency	**$40,074,000**
Contractor insurance/bonding (5%)	$2,004,000
Contractor overhead/profit/general (15%)	$6,011,000
Total Construction Cost	**$48,100,000**
Construction costs inflation to construction midpoint (3%)	$1,443,000
Subtotal	**$49,543,000**
Engineering/Legal/Administrative (20%)	$9,909,000
Total Project Probable Cost	**$59,500,000**

5.6 Probable Cost for Upgrades to Existing Facilities

The opinion of probable cost for upgrades to the existing facilities is presented in Table 5-9. The opinion of probable cost is the total of the cost of upgrades to Basins 1 through 4 (Table 5-6), upgrades to Basins 5 and 6 (Table 5-7) and Upgrades to the existing West and East Filter Buildings (Table 5-8).

Table 5-9: Upgrades to Existing Facilities Probable Cost

Component	Probable Cost
Basins 1 through 4 Upgrades	$26,500,000
Basins 5 and 8 Upgrades	$12,400,000
West and East Filter Building Upgrades	$59,500,000
Total Upgrades to Facilities Project Cost	**$98,400,000**

5.7 Chemical Storage and Feed Facilities

There are a number of chemical feed systems at the Collins Park WTP. In determining if the plant can meet future hydraulic demands, the capacity of the chemical feed systems also needs to be verified. The plant uses the following major chemicals:

- Alum
- Lime
- Soda Ash
- Carbon Dioxide
- Polyphosphate
- Chlorine
- Chlorine Dioxide
- Fluoride
- Powdered Activated Carbon (PAC)
- Potassium Permanganate.

5.7.1 Alum

The chemical system for alum consists of storage tanks, transfer pumps, day tanks and alum Rotadip feeders.

The total bulk storage tank volume of 36,924 gallons provides approximately 34 days of storage under average flow and dosage conditions. This is sufficient to meet proper operational demands and the recommended Ten States Standard of 30-day storage under average usage conditions. Under the Ten State Standard provisions, the 30-day storage capacity is determined by the average feed rate at the average flow rate during the design year. In the 20 year Master Plan, the design year was 20 years out and determined to be 83.3 MGD. This is slightly higher than the historical average day of 79 MGD, but relatively flat over this time period. For alum, the Ten States Standard 30-day requirement would be 1,114 gals per day or 33,422 gals/month. The plant has 36,924 gallons storage capacity which meets the standard.

The Rotodip feeders are capable of feeding 43,200 gallons per day. The actual projected feed requirements under historical, average day feed rates and maximum flow is 2,139 gal/day. As such, the alum rotodip feeders have sufficient capacity to meet future demands.

There are some recommended improvements as follows:

- The alum application method in the raw water channel is not optimal for effective pretreatment. It is unlikely that this application method allows for complete mixing throughout the cross section of flow. A more effective chemical application and mixing method is necessary to increase the distribution of alum. The 20 year Master Plan recommended a sparger unit with a “T” section across the channel to provide better mixing.

- The Rotadip feed system works; however in the long term peristaltic pumps with the appropriate turndown ratio should be installed to provide more flexibility needed at all pumping ranges.

- Install two 600 gallon day tanks to replace the existing tanks.

- Install more storage tanks to increase the capacity to better meet the maximum feed rates at higher pumpages. The recent addition of two 6,462 gallon tanks has help significantly, yet more storage should be considered.

- No spill containment is provided for the alum bulk storage tanks. This is a safety issue in particular considering that the boiler system is housed in the same room. The alum transfer pumps are located on ground level between the bulk storage tanks and would be inaccessible in the event of an alum leak. Spill containment and relocation of the transfer pumps is required to address this safety issue.

Table 5-10: Alum Usage and Feed Rates

Alum Usage	Average flow rates	Maximum Flow rates
Average Feed rates	1,056 gal/day	2,139 gal/day
Maximum Feed rates	9,348 gal/day	18,934 gal/day

Basis:
Equipment Feed Capabilities- 1800 gph= 43,200 gal/day
Storage volume- 6 tanks @ 4,000 gals and 2 @ 6,462=36,924 gals
Historical, average flow rates- 79 MGD
Maximum flow rates- 160 MGD
Average feed rates- 1.04 gpg or 17.8 ppm
Maximum feed rates- 9.21 gpg or 157.5 ppm
Specific weight = 11.1 lbs/gal

5.7.2 Lime

The lime feed system is composed of storage bins and lime slakers. Under the historical, average day feed rates and maximum flow the required amount is 123,899 lbs/day. The slaker capacity is sufficient to meet this requirement with two feeders running. Under unusually high demands, three slakers may need to be used. Each slaker is capable of feeding 96,000 lbs. /day.

The 30-day storage capacity under the Ten States Standard requirement is 967 tons at an average feed rate of 5.43 gpg or 92.8 ppm at 83.3 MGD. The plant has 900 tons of storage and could increase this by using another bin. They meet this standard.

Overall, the slakers have the necessary capacity to meet the present and future demands, however, spare parts are a stock item that should be in place.

Other considerations regarding the alum system are:

- For the 80 MGD plant, lime addition occurs on the surface of the first pass, which is not ideal to provide complete mixing. Relocation and/or reconfiguration of the lime application should be considered to improve chemical dispersion.

- Bulk storage has several inoperable valves and a number of tanks that are not used. The plant currently cannot rotate nor utilize all storage bins due to the damaged valves.

- The vacuum system works; however, it is old and replacement is in order in the future. Consideration should be given to replacing the entire vacuum system including conveyance piping, new rotary valves, replacement of the fifth floor conveyors and replacement of the second floor conveyors and chutes. The chemical building dust collection system and heating should also be upgraded.

- Plant staff must routinely manually remove lime reject solids to an outside dumpster using a wheelbarrow, which is a labor intensive operation. This system should be evaluated for better operation.

Table 5-11: Lime Usage and Feed Rates

Lime Usage	Average flow rates	Maximum Flow rates
Average Feed rates	61,175 lbs/day	123,899 lbs/day
Maximum Feed rates	143,170 lbs/day	289,965 lbs/day

<u>Basis:</u>
Equipment Feed Capabilities- 4,000 lbs/hr/ slaker or 96,000 lbs/day each or 384,000 lbs/ day
Storage volume- 9 available, but use 5 regularly @ 100 tons each
Historical, average flow rates- 79 MGD
Maximum flow rates- 160 MGD
Average feed rates- 5.43 gpg or 92.8 ppm
Maximum feed rates- 12.71.gpg or 217.3 ppm

5.7.3 Soda Ash

The soda ash feed system is composed of storage bins and dry, gravimetric feeders.

The requirement under the historical, average day feed rates and maximum flow is 12,316 lbs/day. Two machines feeding the East and West sides of the plant can handle this demand. The highest soda ash feed over the ten year data period was approximately 6.11 grains per gallon (GPG) or 105 parts per million (PPM). At the peak hour flow rates, the current soda ash feeders would still be able to meet this demand. This feed rate, along with the other highest feed rates, occurred in the spring of the year during the lake turnover. This does not correspond to the highest pumpage times of the summer.

The 30-day storage capacity under the Ten States Standard requirement is 96 tons at an average feed rate of 0.54 gpg or 9.23 ppm at 83.3 MGD. The plant has 300 tons of storage and they meet this standard.

The soda ash feed equipment capacity is sufficient to meet projected future demands.

Other considerations regarding the soda ash system are:

- Soda ash application occurs on the surface of the second pass for both plants and does not provide complete mixing. Feed application dispersal is suggested.

- The vacuum conveyance system is the same as that used in the lime conveyance and is in need of upgrades.

Table 5-12: Soda Ash Usage and Feed Rates

Soda Ash Usage	Average flow rates	Maximum Flow rates
Average Feed rates	6,081 lbs/day	12,316 lbs/day
Maximum Feed rates	68,851 lbs/day	139,449 lbs/day

Basis:
Equipment Feed Capabilities- 33.3 lbs/ min= 47,952 lbs/day X 4= 191,808 lbs/day
Storage volume- 3 @ 100 tons= 600,000 lbs
Historical, average flow rates- 79 MGD
Maximum flow rates- 160 MGD
Historical, average feed rates- 0.54 gpg or 9.23 ppm
Maximum feed rates- 6.11 gpg or 104.5 ppm

5.7.4 Carbon Dioxide

The carbon dioxide feed system was recently renovated in 2009. At that time, the storage capacity of the feed system was investigated and designed accordingly. The storage was increased to 546,000 lbs storage. The 30-day storage capacity under the Ten States Standard requirement is 293 tons at an average feed rate of 1.66 gpg or 28.4 ppm at 83.3 MGD. The plant has 273 tons of storage and they essentially meet this standard as the future feed rate is an estimated quantity. Also, an additional storage tank can be added if necessary.

The vapor heaters and rotometer were not part of this project and may still need to be examined. In the Basin 7&8 plant addition, the rotometers will be capable of doing at least 400 lbs/hr which would result in two basins able to produce 19,200 lbs/hr or an overall capacity of 51,600 lbs/ day. This would meet the operational requirements, however, the upgrade of the remaining 6 basins to 400 lbs/hr should be examined.

Table 5-13: Carbon Dioxide Usage and Feed Rates

Carbon Dioxide Usage	Average flow rates	Maximum Flow rates
Average Feed rates	18,711 lbs/day	37,897 lbs/day
Maximum Feed rates	61,735 lbs/day	125,033 lbs/day

<u>Basis:</u>
Equipment Feed Capabilities- Rotometers are 225 lbs/hr X 6 existing= 32,400 lbs/ day
Note: Rotometers will be improved to 400 lbs/hr X 8= 76,800 lbs/day
Storage volume- 3 @ 91 tons= 546,000 lbs
Historical, average flow rates- 79 MGD
Maximum flow rates- 160 MGD
Average feed rates- 1.66 gpg or 28.4 ppm
Maximum feed rates- 5.48 gpg or 93.7 ppm

5.7.5 Polyphosphate

The polyphosphate system is fed at a relatively constant feed rate and is flow paced to meet the demand of the system. The existing metering pumps are sized to be able to meet this demand. The required average day feed rates and maximum flow is 101 gals/day. The existing pumps are capable of feeding 9 times that amount. The storage of bags is maintained on the second floor and the make-up of the day tank is maintained on an on-going basis which is not a limiting factor on the feed system.

The 30-day storage capacity under the Ten States Standard requirement is 16,465 lbs at an average feed rate of 0.79 ppm at 83.3 MGD. The plant normally has 12,500 lbs of 50 lbs bags in a dry, broken plate glass form in pallets on hand, but has sufficient storage space to accommodate the additional bags. They meet this standard.

Table 5-14: Polyphosphate Usage and Feed Rates

Polyphosphate Usage	Average flow rates	Maximum Flow rates
Average Feed rates	50 gal/day	101 gal/day
Maximum Feed rates	120 gal/day	242 gal/day

Basis:
Equipment Feed Capabilities- 20 gph + 10gph + 10 gph= 40 gph= 960 gal/ day
Storage volume- 2@ 770 gals or 1,540 gals
Historical, average flow rates- 79 MGD
Maximum flow rates- 160 MGD
Average feed rates- 0.79 ppm
Maximum feed rates- 1.89 ppm
Specific Weight = 10.4 lbs/gal

5.7.6 Chlorine and Chlorine Dioxide

A new chlorine building at the Collins Park WTP is under construction and has the capacity for 18 1-ton cylinders on-line per half of the building. The withdrawal rate per tank can be pushed to 500 lbs/day. At these higher withdrawal rates, the pounds per day per side on-line is about 9,000 lbs/day or 18,000 lbs/ day total. The requirement for the average day feed rates and maximum flow is 2,962 lbs/day. Even at maximum pumpage and feed rate, the system is able to produce enough chlorine to meet the demand.

The 30-day storage capacity under the Ten States Standard requirement is 46,268 lbs or 23 tons at an average feed rate of 2.22 ppm at 83.3 MGD. The plant will have 68 tons cylinders on hand of chlorine and they meet this standard.

Before any improvements, each chlorinator was rated at 2,000 lbs/day and with 4 units, the total rated capacity was only 8,000 lbs/day.

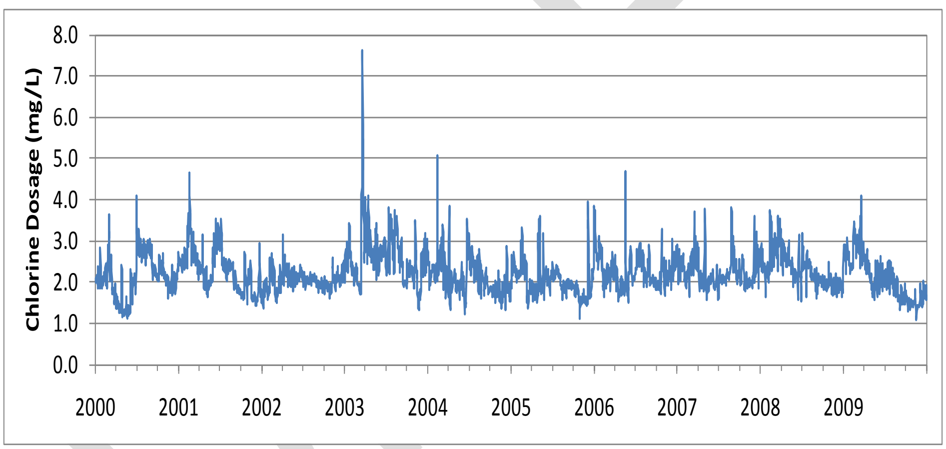
The capacity is based upon the peak hour flows at average feed rates. The average feed rates from a ten year data review was 2.22 ppm. At the 160 MGD capacity of the new treatment facility, this equates to a requirement of 2,962 lbs/day. Chlorine is a critical element of the chemical treatment process and the ability to meet peak conditions is important. Under maximum feed rates at peak flows, the chlorine demand would exceed the old systems ability to produce chlorine. The new system is able to exceed the 10,141 lbs/day requirement.

Table 5-15: Chlorine Usage and Feed Rates

Chlorine Usage	Average flow rates	Maximum Flow rates
Average Feed rates	1,463 lbs/day	2,962 lbs/day
Maximum Feed rates	5,007 lbs/day	10,141 lbs/day

Basis:
Equipment Feed Capabilities- 8 @ 2000 lbs / day chlorinators or 16,000 lbs/ day on-line
Storage volume- 68 @ 2000 lbs or 136,000 lbs of chlorine
Historical, average flow rates- 79 MGD
Maximum flow rates- 160 MGD
Average feed rates- 2.22 ppm
Maximum feed rates- 7.60 ppm

Figure 5-5: Chlorine Dosages from 2000 – 2010



When the new chlorine building was designed, the feed for the chlorine dioxide was also reviewed and improved. The new system has the capability of producing 1500 lbs/day which far exceeds the average day feed rates and maximum flow requirement.

Table 5-16: Chlorine Dioxide Usage and Feed Rates

Chlorine Dioxide Usage	Average flow rates	Maximum Flow rates
Average Feed rates	145 lbs/day	293 lbs/day
Maximum Feed rates	263 lbs/day	533 lbs/day

Basis:
Equipment Feed Capabilities- 2 @ 750 lbs/day or 1500 lbs/day
Storage volume- Made on site; 2 @ 6000 gals sodium chlorite tanks
Historical, average flow rates- 79 MGD
Maximum flow rates- 160 MGD
Average feed rates- 0.22 ppm
Maximum feed rates- 0.4 ppm

5.7.7 Fluoride

The fluoride feed, at peak flow and average feed rates, is approximately 1,781 pounds per day. The feeders are capable of producing 3,744 pounds per day. Therefore, the existing feed equipment is adequate to meet the demand. In addition, a recent reduction in the recommended fluoride concentration guideline to 0.7 mg/L will lower the amount of fluoride that needs to be fed.

The 30-day storage capacity under the Ten States Standard requirement is 16,881 lbs at an average feed rate of 0.81 ppm at 83.3 MGD. The plant has 40,000 lbs on hand of Sodium fluorosilicate (Na_2SiF_6) and they meet this standard.

Table 5-17: Fluoride Usage and Feed Rates

Fluoride Usage	Average flow rates	Maximum Flow rates
Average Feed rates	879 lbs/ day	1,781 lbs/ day
Maximum Feed rates	4,276 lbs/ day	8,659 lbs/ day

Basis:
Equipment Feed Capabilities- 2 @ 3,744 lbs/ day or 7,488 lbs/ day
Storage volume- Bag dumped as needed
Historical, average flow rates- 79 MGD
Maximum flow rates- 160 MGD
Average feed rates- 0.81 ppm
Maximum feed rates- 3.94 ppm
Conversion= Sodium fluorosilicate is 60.7% fluoride by weight

5.7.8 Powdered Activated Carbon (PAC)

The carbon system at Low Service Pumping Station consists of two in-ground storage tanks that takes a truck load of PAC and quenches it to 1 lb/gal with an overall approximate weight of 30,000 lbs per tank of carbon. The maximum feed rate of the current system is 13 to 15 ppm depending upon pumpage, however, renovations have been designed and will be under construction shortly to increase the carbon feed to 40 ppm including increased storage of 175,000 lbs.

The 30-day storage capacity under the Ten States Standard requirement is 44,580 lbs at an average feed rate of 2.14 ppm at 83.3 MGD. The plant has 235,000 lbs on hand of carbon and they meet this standard.

The maximum feed rate for carbon has not been accomplished in full scale application as the system was limited. In testing, we believe a better feed rate for algae treatment will be 15 to 20 ppm. Our data shows that at 15 ppm, the carbon is able to treat up to 43.3 ppb of the algal byproduct. In the rare case that additional carbon needs to be feed, additional capacity is available. Under the average day feed rates and maximum flow, the system needs to be able to feed 2,856 lbs/day. Because of its critical use, the new system will produce 40 ppm at 160 MGD or able to feed 53,376 lbs/day. Even at this unanticipated feed rate, the system could last for 3.8 days and be able to restock their supplies before they ran out. At 20 ppm, this would be 7.6 days.

Table 5-18: Powdered Activated Carbon (PAC) Usage and Feed Rates

Carbon (PAC) Usage	Average flow rates	Maximum Flow rates
Average Feed rates	1,410 lbs/ day	2,856 lbs/ day
Maximum Feed rates	13,177 lbs/ day	26,688 lbs/ day

<u>Basis:</u>
Equipment Feed Capabilities- 15 ppm to be improved to 40 ppm
Storage volume- 60,000 lbs + 175,000 lbs = 235,000 lbs
Historical, average flow rates- 79 MGD
Maximum flow rates- 160 MGD
Average feed rates- 2.14 ppm
Maximum feed rates- 20 ppm
Carbon density = 23 lbs/ft^3

There will also be a temporary PAC feed and storage at the Collins Park WTP. PAC will be feed in the third pass of the flocculation chambers to provide additional treatment for microcystin. The system will consist of two additional dry storage silos of

3,000 ft3 of carbon, two dry feeders and a splitter box with two eductors. One will be located on the West side and the other will be on the East side. The flow will be separated between basins 1&2 and 3&4 by a splitter box while the flow on the East side will go to basins 5&6. This feed system will be able to provide up to 6 mg/L for 120 MGD flow. As this is a temporary measure, the system was sized to match the current rated capacity of the plant not the expanded capacity of 160 MGD. When all of the advanced treatment is in place, these storage tanks will be moved to Low Service to increase the storage of the PAC feed there.

5.7.9 Potassium Permanganate

The potassium permanganate system was installed to supply a continuous feed of 1 ppm for zebra mussel control. The occurrence of the algal bloom in the Western Basin of Lake Erie has required a re-design of this system to feed a higher dosage of potassium permanganate to help control algal byproducts. Potassium permanganate use to lyse the algae cells to allow for the removal of the cellular components is being proposed. Improvements to this system have been designed and will soon be constructed to increase the minimum feed rate to at least 4.0 ppm for algal treatment.

The 30-day storage capacity under the Ten States Standard requirement is 20,841 lbs at an average feed rate of 1.0 ppm at 83.3 MGD. The plant has over 50,000 lbs in silos on hand and they meet this standard.

Table 5-19: Potassium Permanganate Usage and Feed Rates

Pot. Permanganate Usage	Average flow rates	Maximum Flow rates
Average Feed rates	659 lbs/day	1,335 lbs/day
Maximum Feed rates	2,635 lbs/day	5,336 lbs/day

<u>Basis:</u>
Equipment Feed Capabilities- 600 lbs/day screw feeder
Storage volume- 25 ton silo to store 50,000 lbs
Average flow rates- 79 MGD
Maximum flow rates- 160 MGD
Average feed rates- 1.0 ppm
Maximum feed rates- 4.0 ppm

5.7.10 Chemical Feed Summary

In summary, the chemical feed systems of the Collins Park Water Plant have been upgraded to meet the demands of a 160 MDG treatment facility. The carbon, potassium permanganate, carbon dioxide, alum, chlorine and chlorine dioxide system have been or are in the process of being improved. With the exception of further enhancements to the carbon dioxide rotometers and vaporizers, the chemical feeds are adequate to meet the future demands.

5.7.10.1 Chemical Feed Conveyance

Conveyance of chemical feed to a conventional construction of Basin 7&8 (Alternative 1) would mirror the West plant. The chemical feeders are located centrally in the chemical feed room next to the raw water channels. The storage of lime and soda ash are conveyed to this room via the pneumatic equipment located on the 5th floor. As such, relocating these feeds or de-centralizing these processes becomes extremely difficult. Similarly, alum is fed into the raw water at the head of the channels which serve as the rapid mix for the process. Basins 7&8 will use the East channel as its source of raw water. Changing the location of these feeds would also be impractical.

If the Solids Contact Clarifiers (Alternative 2) are chosen rather than the conventional basin configuration, lime feed to these units would be conveyed via the same method as the conventional basin configuration. The units would need to be contiguous to the existing plant and lime feeds would be routed through the basement from the splitter boxes located in the basement directly below the lime slakers. Fire hoses are presently used to convey the slurry to the reaction basins in all parts of the plant. In the past, standard plastic (rigid) pipes were used, however, plugging caused excessive maintenance. When a plugged occurred, the pipe would need to be dismantled, clean and reassembled. With the fire hoses, maintenance individuals can 'walk the hose' to relieve the plugged area without taking them apart which significantly reduces maintenance time. The distance for these feed lines are consistent with the routing on the West side of the plant and should not present difficulty.

5.7.10.2 Future Chemical Feeds

The final comments as far as chemical feeds are concerned are in respect to the newer technologies. Currently, ozone and GAC are being considered as supplemental treatment processes. Both technologies will use different chemicals. The ozone application will need to install liquid oxygen tanks to use in the making ozone for the process. The tanks and feed system will be new additions and identified in that section. Similarly, the granular activated carbon that will be used in a GAC system will be specified in that section of the manual. GAC will be regenerated either on-site or returned to the manufacturer for regeneration.

5.8 Residuals Handling Facilities

5.8.1 Introduction

Residuals streams are generated from the softening, coagulation, filter backwash processes, and various maintenance activities. Residuals are pumped from the WTP into the dewatering facility thickeners and then pumped into one of two plate and frame presses. The presses force water out of the solids resulting in a cake containing about 65 percent solids material. Cake is dumped into semi-trucks and hauled from the plant for disposal. Water removed from the solids is captured, pumped to a filtrate lagoon, and discharged to Otter Creek under a current NPDES permit. Alternately, spent lime can be pumped to Lagoons A, B, C or E. Water decanted from these lagoons flows either to Duck Creek or to Otter Creek under a current NPDES permit.

Residuals consist of naturally occurring colloidal matter, suspended solids, other particulates from the raw water, and chemical residues formed during softening and coagulation. Residuals production was estimated based on hardness and turbidity removal data and average chemical dosages. Figure 5-6 shows the average monthly sludge production from January 2007 through November 2009 from the 20 Year Master Plan. As indicated in the figure, the overall average and 90th percentile sludge production is approximately 180,000 and 265,000 lb/day, respectively. The sludge is composed of lime, turbidity, alum and PAC sludge with the largest contribution from the lime solids as shown in Figure 5-7.

Figure 5-6: Average Monthly Sludge Production

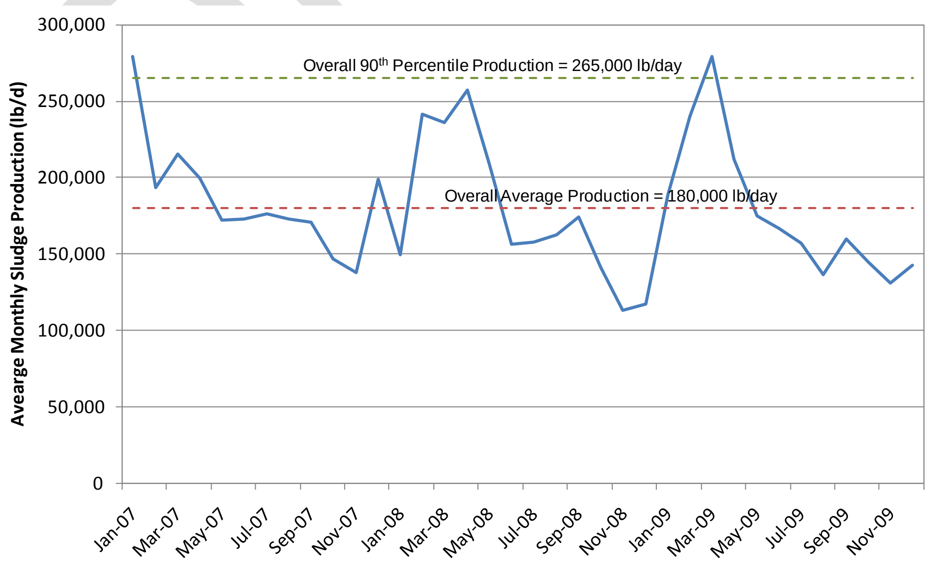
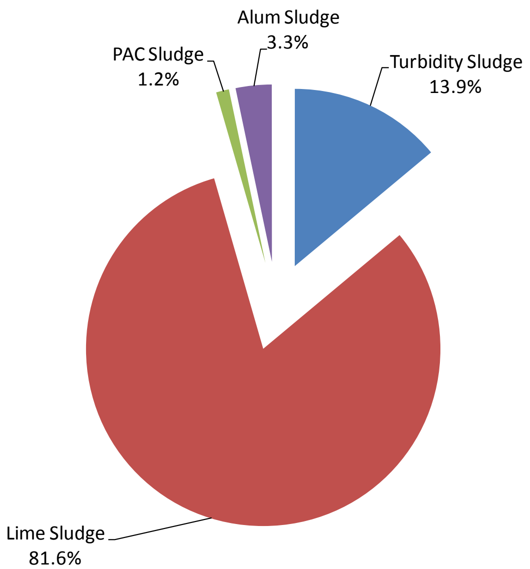


Figure 5-7: Breakdown of Residual Solids Components



5.8.2 Residuals Handing and Sludge Dewatering Facility

The sludge dewatering facility was constructed in 1997 to provide an alternate means to the lagoons of dewatering and disposing of WTP residuals. A majority of the equipment in the facility is original. Residuals are pumped from the WTP into the dewatering facility thickeners (Figure 5-8). Thickened sludge is pumped into a press pre-fill tank by three thickened sludge pumps. These pumps are 10 horsepower, horizontal vortex centrifugal pumps, each rated for 750 gpm at 16 feet of total dynamic head.

Figure 5-8: Dewatering Facility Thickener

The residuals are then pumped into the plate and frame presses by a combination of the quick fill and high pressure pumps. There are two horizontal vortex centrifugal type quick fill pumps manufactured by WEMCO, and three 75 horsepower, 350 gpm at 225 psi, piston type high pressure pumps manufactured by ABEL (see Figure 5-9). There are two plate and frame presses, each capable of producing 437 cubic feet of cake per day (see Figure 5-10). Each press contains 125 plates and can operate with a maximum closing pressure of 4,500 psi. The presses are located above a tractor trailer bay so that upon completion of the pressing run, the cake is dropped into a semi-trailer for removal from the site.

Figure 5-9: High Pressure Pump

Figure 5-10: Plate and Frame Press

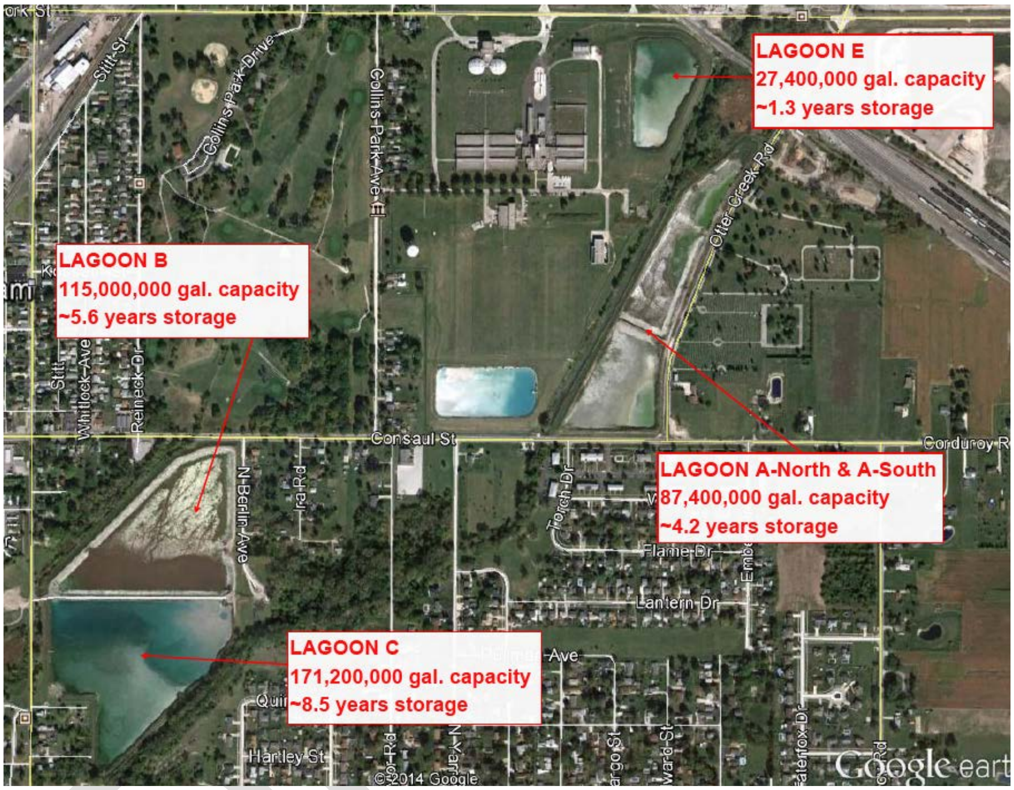
It should be noted that the Sludge Dewatering Facility does not have standby power. As a result, the facility is unable to operate during a power outage.

5.8.3 Spent Lime Lagoon Storage

Existing Lagoons A, B, C or E are used for spent lime storage. Figure 5-11 shows the capacity in gallons and approximate years of storage if the lagoons were empty. Presently, Lagoons A-North, B, C and E are full and not available for additional storage. Lagoon A-South has just been cleaned and is available for storage. Lagoon A-North is scheduled to be cleaned next with the contract completed and underway.

Generally, it has been past practice to operate the Sludge Dewatering Facility in lieu of cleaning and using the lagoons. However, during certain seasonal variations in raw water quality and high alum dosages the Sludge Dewatering Facility is unable to operate due to plate and frame press filter fabric fouling issues. So it is important to have some reserve lagoon storage available to operate during these conditions as well as to allow for general maintenance of the Sludge Dewatering Facility.

Figure 5-11: Spent Lime Storage Lagoon Capacities



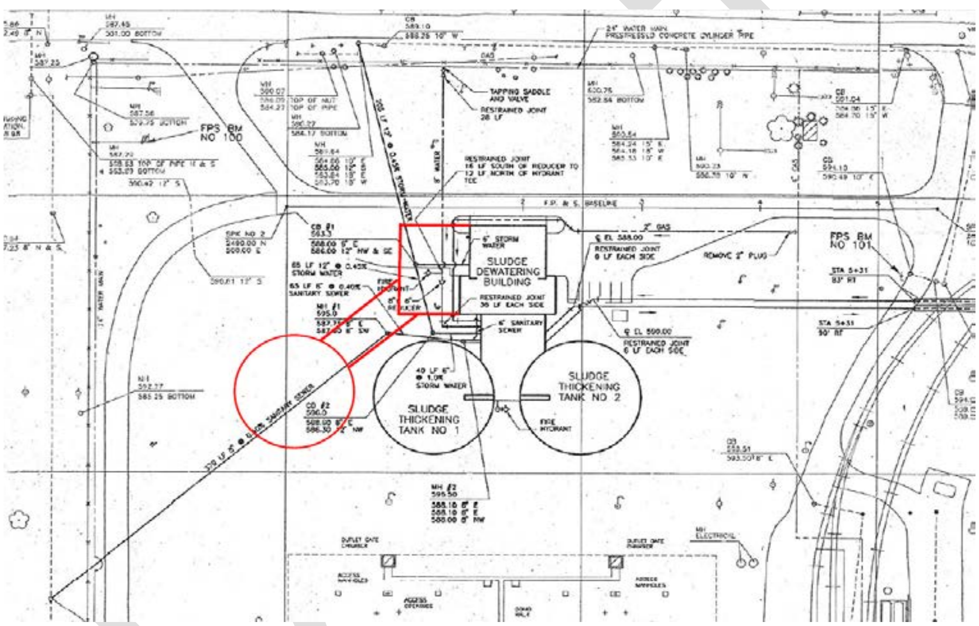
5.8.4 Recommended Residuals & Sludge Dewatering Facility Improvements

Based on field investigations, a complete rehabilitation of the residuals handling facilities is recommended and generally includes the following improvements:

- <u>Rehabilitate Thickeners and Filter Presses</u> - Re-paint the thickener mechanisms, re-build thickener drives, and re-waterproof thickener tanks. Rehabilitate filter presses including recoating plates and replacing clothes, and providing a spare set of plates to aid in maintenance activities. Replace thickened sludge, quick fill, and high pressure pumps. Replace valves including press ball valves and pneumatic actuators.

- Add a New Third Sludge Press and Sludge Thickener - The third press and appurtenances would need to be housed in a new building adjacent to existing facility because insufficient space is available in the existing facility for an additional press. A new thickener is recommended to provide additional solids storage prior to dewatering. This will help address overloading of the existing thickeners during peak solids production periods and when there is limited dewatering/offloading capacity. Figure 5-12 shows a potential conceptual layout plan. If desired, the third press and thickener could also be located on the east/right side of the existing facilities.

Figure 5-12: Potential Layout – 3rd New Sludge Press and Sludge Thickener



- Complete Miscellaneous SDF Improvements - This includes press control modifications, lighting and HVAC improvements and replacement of the floor door with a larger unit.

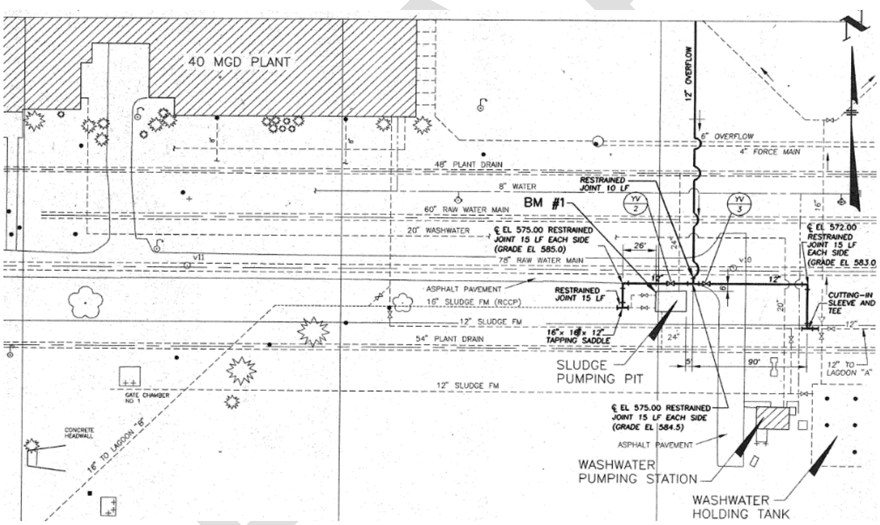
- Lagoon Cleaning Program - In lieu of or in combination of Sludge Dewatering Facility Improvements, implement a regular lagoon cleaning program. This could entail cleaning Lagoon B next and then following with Lagoon C.

- Replace the 40 and 80 mgd plant sludge pumps with the same type of pumps.

- Replace Waste Washwater Pumping Station - It has been observed that the pump check valves and discharge header at the wall penetration are leaking. As a result, it is recommended that the washwater pumps and check valves be replaced. In addition, sump pump discharge modifications are recommended and include replacing the existing sump pump and piping and a new connection to the existing sanitary manhole.

- Basin Drain Pumping Station - Recommended improvements to the Basin Drain Pumping Station include miscellaneous modifications/improvements to the sludge pumping pit, plumbing, piping, HVAC, electrical and sanitary sewer systems. These improvements will allow for draining the flocculation and sedimentation basins and no longer require the use of the Waste Washwater Handling Facility. Figure 5-13 shows the location of this pumping station.

Figure 5-13: Existing Basin Drain Pumping Station



5.8.5 Probable Project Costs

Table 5-20 provides a summary of probable project costs for the potential improvements identified for residuals handling facilities and are as follows:

Table 5-20: Probable Cost, Residual Handling Facilities

Project Description	Probable Cost[1]
Residuals Handling and Dewatering Improvements	
Filter Press and Thickening Systems Rehabilitation	$8,600,000
Thickener System Expansion – Dewatering Building and Press	$13,200,000
Thickener System Expansion – Additional Sludge Thickening Tank	$2,990,000
Replace Sludge Pumps (40 and 80 mgd plants)	$270,000
Miscellaneous Sludge Dewatering Facility Improvements (includes process control, lighting and HVAC improvements)	$340,000
Waste Washwater Pumping Station Improvements	
Washwater Pump Replacement	$500,000
Sump Pump Piping Discharge Modifications (includes replacement of sump pump and piping and new connection to sanitary sewer)	$70,000
Lagoon Cleaning Improvements[2]	
Lagoon B Cleaning[2]	$11,850,000
Lagoon C Cleaning[2]	$17,750,000
Basin Drain Pumping Station Improvements	
Basin Drain Pumping Station Improvements (includes miscellaneous modifications/improvements to sludge pumping pit, plumbing, piping, HVAC, electrical and sanitary sewer systems)	$2,600,000
Total	**$58,170,000**
[1] Includes 30% for Contingencies and 20% Engineering and Technical Services.	
[2] In lieu of or in combination of Sludge Dewatering Facility Improvements.	

Further detailed evaluation may be needed to determine the most feasible options between improvements to the Sludge Dewatering Facility and Lagoon Cleaning Improvements. A combination of portions of these improvements may be desirable and more feasible.

5.9 Future Additional Treatment Considerations and Space Allocation

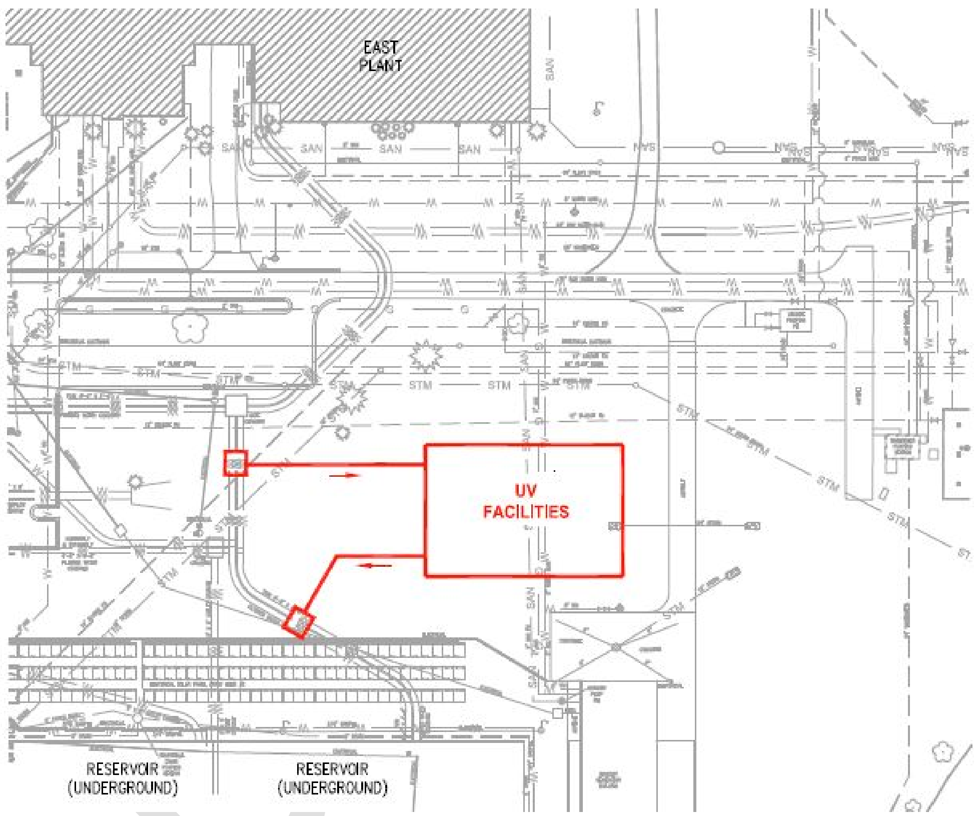
5.9.1 UV Addition for Cryptosporidium Inactivation

When considering Cryptosporidium, it is our understanding that the Toledo Collins Park WTP has typically not found Cryptosporidium in the source water and that they current are classified in Bin1 based on sampling performed to date. As such, no additional treatment is required to achieve a further level of removal or inactivation of cryptosporidium. In the future, should additional log inactivation credit be required or there is a desire to provide multi-barrier treatment for Cryptosporidium dictated by more recent sampling results and re-classification to a higher bin level, UV could be implemented. With current reactor design, additional log credit ranging from 2 to 4 additional logs can be readily achieved. The UV process would be installed downstream of the gravity filters (or following GAC Post filter Contactors if they are installed) where the highest UV transmittance level would be provided.

There are two general styles of UV reactors used in water treatment today; one using medium pressure (MP) lamps and one using low pressure high output lamps. Each has certain advantages and disadvantages, which must be weighed for each installation. To establish the space allocation required for installation of a UV facility, a general layout arrangement was developed looking at a total of five UV reactor units each rated for a flow of approximately 40 MGD; four in-service reactors and one standby unit. A building footprint of approximately 100 feet by 150 would be required for the facility. Flow in the filtered water conduits running from the West and East Filters to the Finished Water Reservoirs would be intercepted and routed to the new UV facilities; the supply and discharge headers serving the reactors were preliminary sized for 96-inch lines. Each reactor line would be equipped with a flowmeter, a flow control valve and isolation valves to allow the reactor units to be taken off line for periodic cleaning and maintenance. Flow would be returned to the filtered water conduit running to the reservoir. It will most likely be necessary to provide low lift pumping or reduce the operating level within the reservoirs to provide sufficient hydraulic head for the additional hydraulic losses required for the operation of the UV system.

A potential location for the facility is shown in Figure 5.14. With the arrangement shown, flow would be intercepted and returned to the same conduit running between the filters and the reservoirs; with this approach flow through the reservoirs would have to be handled in a series manner. To provide additional flexibility, piping could potentially be routed within the reservoirs to allow either series or parallel flow through the reservoirs. Further detailed investigation of the hydraulic conditions and operating constraints would be required for the incorporation of UV treatment at the time when a decision is made that an additional treatment barrier is required. For the present, it is recommended that Water Division staff reserve the space in the vicinity of the facility shown on Figure 5.14 for its future construction should circumstances dictate that an additional treatment barrier is necessary.

Figure 5-14: UV Facility Addition Conceptual Layout



6. Design Project Definition

This section will be completed following the work of the Blue Ribbon Panel and discussions with City of Toledo Water Division staff concerning the recommended actions of the Blue Ribbon Panel.

6.1.1 Facility Design Criteria

6.1.2 Construction Sequencing Approach

6.1.3 Opinion of Probable Construction Costs

ARCADIS

Appendix A

Basin Dimensions and Design Parameters

Toledo Collins Park WTP
Redundant Capacity Improvements
Basins 1 - 4 As Built Dimensions and Design Parameters

Total Flow (MGD)	80
Basin Flow (MGD)	20
Maximum WSEL	599.5
Flocculation Basin	
Number of Basins	4
Basin Length (ft)	62.58
Basin Width (ft)	78.75
Stage Length (ft)	78.75
Stage Width (ft)	17.33
Number of Stages	3
Maximum WSEL	599.5
Top (elev)	602
Bottom (elev)	582
SWD (ft)	17.5
Volume (gal)	535,932
Detention Time (min)	38.59
Flow through Velocity (ft/min)	6.13
Basin Overflow Weir (elev)	600.33
Sedimentation Basin	
Number of Basins	4
Basin Length (ft)	270.42
Basin Width (ft)	83.5
Maximum WSEL	599.5
Top (elev)	602
Bottom (elev)	585.33
SWD (ft)	14.17
Volume (gal)	2,393,298
Detention Time (hrs)	2.87
Surface Area (sf)	22,580
SOR (gpm/sf)	0.62
Flow through Velocity (ft/min)	1.57
Weir Length (ft)	12.42
Number of Weirs	5.00
Total Basin Weir Length (ft)	62.10
Weir Loading Rate (gpd/lf)	322,061
L/W Ratio	3.24
Recarbonation Basin	
Number of Basins	4
Total Basin Length (ft)	16.58
Mixing Zone Length (ft)	3.25
Mixing Zone Width (ft)	83.5
Maximum WSEL	599.5
Top (elev)	602
Bottom (elev)	585.33
SWD (ft)	14.17
Volume (gal)	28,763
Detention Time (min)	2.07
Reaction Length (ft)	13.33
Reaction Width (ft)	83.5
Maximum WSEL	599.5
Top (elev)	602
Bottom (elev)	585.33
SWD (ft)	14.17
Volume (gal)	117,974
Detention Time (min)	8.49
Total Basin Detention Time (min	10.57

Toledo Collins Park WTP
Redundant Capacity Improvements
Basins 1 - 4 Modified Dimensions and Design Parameters

Total Flow (MGD)	80
Basin Flow (MGD)	20
Maximum WSEL	600
Flocculation Basin	
Number of Basins	2
Basin Length (ft)	62.58
Basin Width (ft)	78.75
Stage Length (ft)	78.75
Stage Width (ft)	17.33
Number of Stages	3
Maximum WSEL	600
Top (elev)	602
Bottom (elev)	582
SWD (ft)	18
Volume (gal)	551,245
Detention Time (min)	39.69
Flow through Velocity (ft/min)	5.96
Basin Overflow Weir (elev)	600.83
Sedimentation Basin	
Number of Basins	2
Basin Length (ft)	253.92
Basin Width (ft)	83.5
Maximum WSEL	600
Top (elev)	602
Bottom (elev)	585.33
SWD (ft)	14.67
Volume (gal)	2,326,564
Detention Time (hrs)	2.79
Surface Area (sf)	21,202
SOR (gpm/sf)	0.66
Flow through Velocity (ft/min)	1.518
Weir Length (ft)	100.00
Number of Weirs	10.00
Total Basin Weir Length (ft)	1000.00
Weir Loading Rate (gpd/lf)	20,000
L/W Ratio	3.04
Recarbonation Basin	
Number of Basins	2
Total Basin Length (ft)	33.08
Mixing Zone Length (ft)	6
Mixing Zone Width (ft)	83.5
Maximum WSEL	600
Top (elev)	602
Bottom (elev)	585.33
SWD (ft)	14.67
Volume (gal)	54,976
Detention Time (min)	3.96
Reaction Length (ft)	27.08
Reaction Width (ft)	83.5
Maximum WSEL	600
Top (elev)	602
Bottom (elev)	585.33
SWD (ft)	14.67
Volume (gal)	248,123
Detention Time (min)	17.86
Total Basin Detention Time (min	21.82

Toledo Collins Park WTP
Redundant Capacity Improvements
Basins 5 - 6 As Built Dimensions and Design Parameters

Total Flow (MGD)	40
Basin Flow (MGD)	20
Maximum WSEL	599.5
Flocculation Basin	
Number of Basins	2
Basin Length (ft)	46.92
Basin Width (ft)	83.42
Stage Length (ft)	83.42
Stage Width (ft)	14.97
Number of Stages	3
Maximum WSEL	599.5
Top (elev)	602
Bottom (elev)	582
SWD (ft)	17.5
Volume (gal)	490,512
Detention Time (min)	35.32
Flow through Velocity (ft/min)	7.10
Basin Overflow Weir (elev)	600.33
Sedimentation Basin	
Number of Basins	2
Basin Length (ft)	284.25
Basin Width (ft)	83.42
Maximum WSEL	599.5
Top (elev)	602
Bottom (elev)	585.33
SWD (ft)	14.17
Volume (gal)	2,513,287
Detention Time (hrs)	3.02
Surface Area (sf)	23,712
SOR (gpm/sf)	0.59
Flow through Velocity (ft/min)	1.57
Weir Length (ft)	12.42
Number of Weirs	5.00
Total Basin Weir Length (ft)	62.10
Weir Loading Rate (gpd/lf)	322,061
L/W Ratio	3.41
Recarbonation Basin	
Number of Basins	2
Total Basin Length (ft)	118.42
Mixing Zone Length (ft)	6
Mixing Zone Width (ft)	83.42
Maximum WSEL	599.5
Top (elev)	602
Bottom (elev)	585.5
SWD (ft)	14.0
Volume (gal)	52,414
Detention Time (min)	3.77
Reaction Length (ft)	112.42
Reaction Width (ft)	83.42
Maximum WSEL	599.5
Top (elev)	602
Bottom (elev)	585.5
SWD (ft)	14
Volume (gal)	982,072
Detention Time (min)	70.71

Toledo Collins Park WTP
Redundant Capacity Improvements
Basins 5 - 6 Modified Dimensions and Design Parameters

Total Flow (MGD)	40
Basin Flow (MGD)	20
Maximum WSEL	600
Flocculation Basin	
Number of Basins	2
Basin Length (ft)	46.92
Basin Width (ft)	83.42
Stage Length (ft)	83.42
Stage Width (ft)	14.97
Number of Stages	3
Maximum WSEL	600
Top (elev)	602
Bottom (elev)	582
SWD (ft)	18
Volume (gal)	504,527
Detention Time (min)	36.33
Flow through Velocity (ft/min)	6.90
Basin Overflow Weir (elev)	600.83
Sedimentation Basin	
Number of Basins	2
Basin Length (ft)	284.25
Basin Width (ft)	83.42
Maximum WSEL	600
Top (elev)	602
Bottom (elev)	585.33
SWD (ft)	14.67
Volume (gal)	2,601,971
Detention Time (hrs)	3.12
Surface Area (sf)	23,712
SOR (gpm/sf)	0.59
Flow through Velocity (ft/min)	1.520
Weir Length (ft)	100.00
Number of Weirs	10.00
Total Basin Weir Length (ft)	1000.00
Weir Loading Rate (gpd/lf)	20,000
L/W Ratio	3.41
Recarbonation Basin	
Number of Basins	2
Total Basin Length (ft)	37
Mixing Zone Length (ft)	6
Mixing Zone Width (ft)	83.42
Maximum WSEL	600
Top (elev)	602
Bottom (elev)	585.5
SWD (ft)	14.5
Mixing Zone Volume (gal)	54,286
Mixing Detention Time (min)	3.91
Reaction Length (ft)	31
Reaction Width (ft)	83.42
Maximum WSEL	600
Top (elev)	602
Bottom (elev)	585.5
SWD (ft)	14.5
Reaction Volume (gal)	280,480
Reaction Detention Time (min)	20.19
Total Basin Detention Time (min	24.10

Toledo Collins Park WTP
Redundant Capacity Improvements
Basins 7 - 8 Design Dimensions and Parameters

Total Flow (MGD)	40
Basin Flow (MGD)	20
Maximum WSEL	600
Flocculation Basin	
Number of Basins	
Basin Length (ft)	46.92
Basin Width (ft)	83.42
Stage Length (ft)	83.42
Stage Width (ft)	14.97
Number of Stages	
Maximum WSEL	600
Top (elev)	602
Bottom (elev)	582
SWD (ft)	18
Volume (gal)	504,527
Detention Time (min)	36.33
Flow through Velocity (ft/min)	6.90
Basin Overflow Weir (elev)	600.83
Sedimentation Basin	
Number of Basins	
Basin Length (ft)	284.25
Basin Width (ft)	83.42
Maximum WSEL	600
Top (elev)	602
Bottom (elev)	585.33
SWD (ft)	14.67
Volume (gal)	2,601,971
Detention Time (hrs)	3.12
Surface Area (sf)	23,712
SOR (gpm/sf)	0.59
Flow through Velocity (ft/min)	1.520
Weir Length (ft)	100.00
Number of Weirs	10.00
Total Basin Weir Length (ft)	1000.00
Weir Loading Rate (gpd/lf)	20,000
L/W Ratio	3.41
Recarbonation Basin	
Number of Basins	
Total Basin Length (ft)	37
Mixing Zone Length (ft)	6
Mixing Zone Width (ft)	83.42
Maximum WSEL	600
Top (elev)	602
Bottom (elev)	585.5
SWD (ft)	14.5
Mixing Zone Volume (gal)	54,286
Mixing Detention Time (min)	3.91
Reaction Length (ft)	31
Reaction Width (ft)	83.42
Maximum WSEL	600
Top (elev)	602
Bottom (elev)	585.5
SWD (ft)	14.5
Reaction Volume (gal)	280,480
Reaction Detention Time (min)	20.19
Total Basin Detention Time (min	24.10

Appendix B

Temporary PAC Feed Demonstration Protocol

Toledo Ohio
Department of Public Utilities
Division of Water Treatment
Collins Park Water Treatment Plant

Checklist for temporary Powdered Activated Carbon feed demonstration

- Purpose for chemical
  State reason for trying the new chemical and state what its purpose is.

  Powdered activated carbon (PAC) has been demonstrated as effective in reducing Microcystin toxin levels. We currently apply PAC at our Low Service Pumping Station and have a project under design to apply PAC to the settled water at the plant. This is an accepted practice and is anticipated to provide an additional barrier against Microcystin toxin. The goal of the pilot is to determine maximum PAC dose that will allow normal filter operation. We will utilize the same PAC in the pilot as is utilized at our LSPS application point. A single filter (Filter #10) will be used for the purposes if the pilot.

- Provide schematic of chemical feed system showing bulk tanks, transfer pumps, day tanks, feeders, and application points. If dilution water will be added to chemical, show how it will be protected from backflow.

  See attached.

- Provide sketch of temporary feed system location in relation to other chemicals.

  See attached.

- Chemical specifications
  1. NSF Standard 60 Certification?
     Yes
  2. Solution strength?
     N/A
  3. Specific gravity?
     N/A

- Chemical supply availability
  How will you ensure an adequate amount of the new chemical is available during the demonstration so that the process is not interrupted and the quality of water is maintained?

  We have estimated and pre-purchased the quantity required for the duration of the pilot.

- Dosages
  Please answer the following questions regarding the chemical application:
  - What is the normal lime dose, in mg/L?

  114 mg/L

  - What is the anticipated chemical dose, in mg/L?

  0.5-5 mg/L

  - What will be the point of application for the chemical?

  Carbon will be educted as slurry into the gullet of Filter #10 and will be allowed to accumulate on the surface of the filter.

- Feed System Info
  - What will be the means of measuring the amount of applied chemical?

  Gravimetric feeder

  - Indicate number and size of bulk tanks and day tanks.

  A single day hopper with approximate capacity of 135 lbs – carbon is to be reloaded daily.

  -If this is an essential chemical, you will need to have two pumps available, each capable of meeting the target dose. If only one pump is to be utilized, the second pump must be capable of being readily activated if pump one fails.

  N/A

  - What will be the feed pump make, model, and capacity?

  N/A – educted using house water supply

  - What type of spill containment will be provided?

  N/A

  - What type of alarming will be provided for pump failure?

  N/A

- Goals of the study
  -Establish appropriate goals for the study and determine what parameters will determine whether or not this is a successful demonstration.

Water quality goals (i.e. measured at raw, settled, finished, etc. where appropriate)
Operational goals
Sludge management goals where appropriate.

The goal of this pilot project is to determine the relative amount of PAC that may be fed ahead of a filter without causing unacceptable filter loss of performance. Filtered water turbidity is to be maintained below 0.3 NTU in 95% of the samples. Carbon effectiveness for microcystin removal will not be tested in the filter since there is none currently present; a separate testing project will use bench scale testing to determine carbon dosage effectiveness at removing microcystin through the use of spiked samples.

- Data Collection
  What parameters will be monitored and at what frequency to evaluate whether or not the goals are met?

  Effluent turbidity from filter 10 will be continuously monitored and recorded. Also, headloss, flow and filter run times will be continuously monitored and recorded for filter 10. Filter 8, which is adjacent to Filter 10, will also be continuously monitored for effluent turbidity, headloss, flow and filter run times; Filter 8 will serve as a control filter.

  Settled water upstream of Filter 10 will be monitored at least every 4 hours for turbidity and pH.

  The criteria used to determine backwashing frequency for Filter 10 will be the same for the pilot period as during normal operations. Filter 10 will be backwashed when headloss reaches 6.5 feet or when the filter has 100 hours of run time, whichever occurs first.

  What will the finished water goals be for various parameters during the course of the demonstration study?

  Keep a log of maintenance issues and equipment performance. A maximum filtered water turbidity of 0.3 NTU will be allowed in 95% of the samples.

- Retain ability to feed original chemical
  You will be required to maintain a supply of your normal lime feed during the course of the demonstration study in the event the chemical fails to meet Ohio Administrative Code requirements. Please indicate how rapidly you will be able to switch feeds from the chemical to your normal chemical applied (if applicable).

  The demonstration feed of carbon will be able to be terminated at any time if it is found to be unacceptably detrimental. Backwashing the demonstration filter will remove the PAC from the filter and return to original conditions.

- Demonstration Timeline
  How long do you anticipate conducting the study? (Timelines of 30 to 60 days have been acceptable in the past with potential for extensions.)

  The demonstration is expected to last 60 days. Pending OEPA approval, proposed pilot timeline is from Dec. 1, 2014 to Jan. 31, 2015.

- Potential impacts from study
  How will the effects on sludge characteristics and filterability be evaluated during the study?

  The limited quantity of carbon fed during the demonstration will have negligible effects on sludge quality. Filter performance will be monitored via normal turbidity monitoring along with daily visual observation of the filter performance.

- Demonstration Study Results

  At the end of the study, submit a report which summarizes the data, provides an evaluation and analysis of the data to determine how effective the chemical was in meeting the established goals, and a conclusion. Additionally, include analysis on the effect of the chemical on corrosivity, disinfection byproduct formation, surface water treatment rule requirements (turbidity and CT), and other chemical feeds utilized by the plant. Effects on sludge will likely need to be addressed, which may include permit issues and/or land application issues.

  Reporting of demonstration study results will be submitted within two weeks after the conclusion, in which the data collected, results of the data analysis, and the conclusions and recommendations are presented & clearly summarized. Turbidity data analysis will include minimum, maximum, and 95th percentile per filter run and per the entire duration of the study as a whole. Data to be included in the report will include dosages, run times & flow rates. The production efficiency for each filter run will be calculated and presented in the report for both Filter 10 and Filter 8.

- Permanent Installation

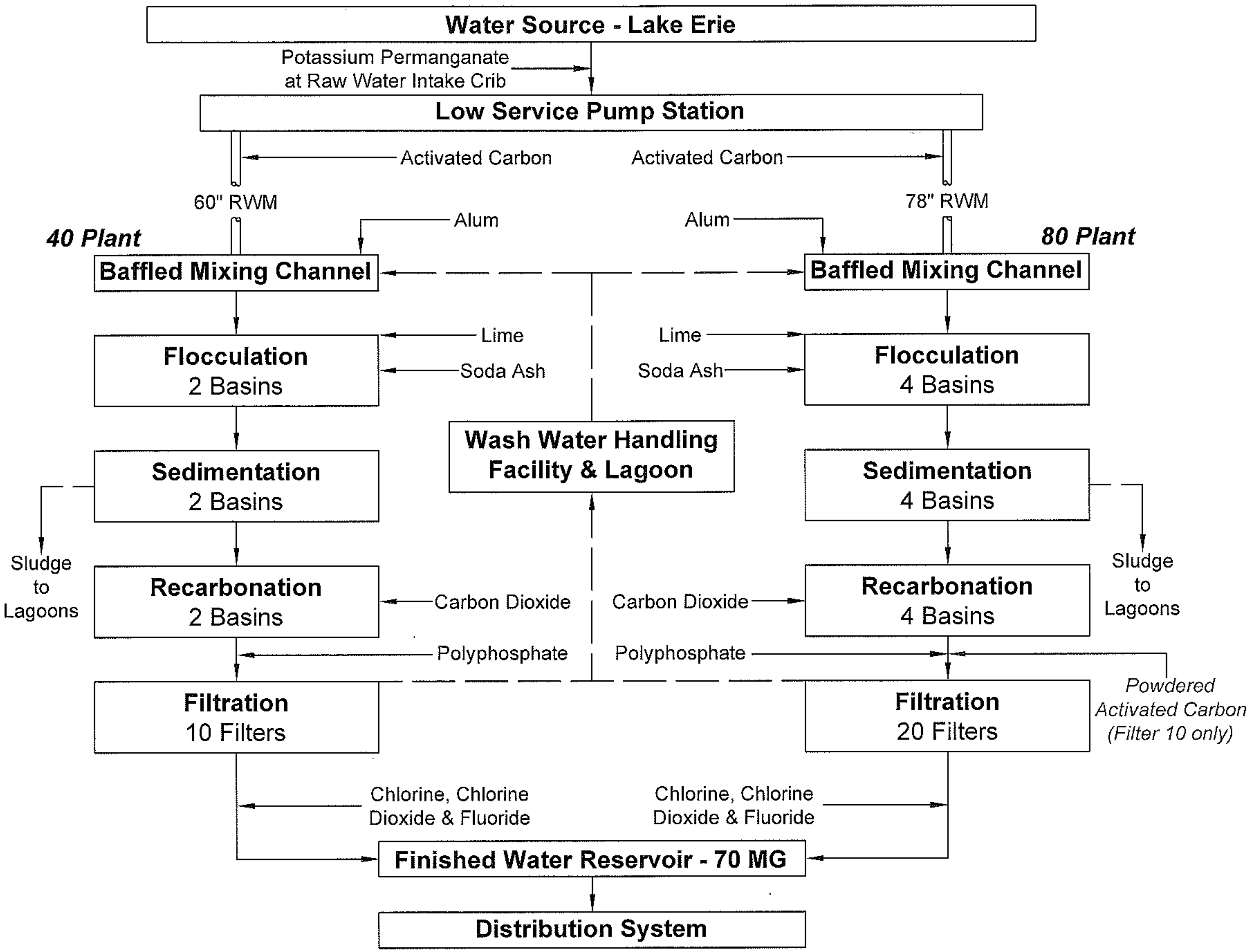
  At the end of the study, you will need to switch back to your normal chemical feed if equipment changes will be required. These equipment changes will likely need detail plan approval by Ohio EPA prior to construction.

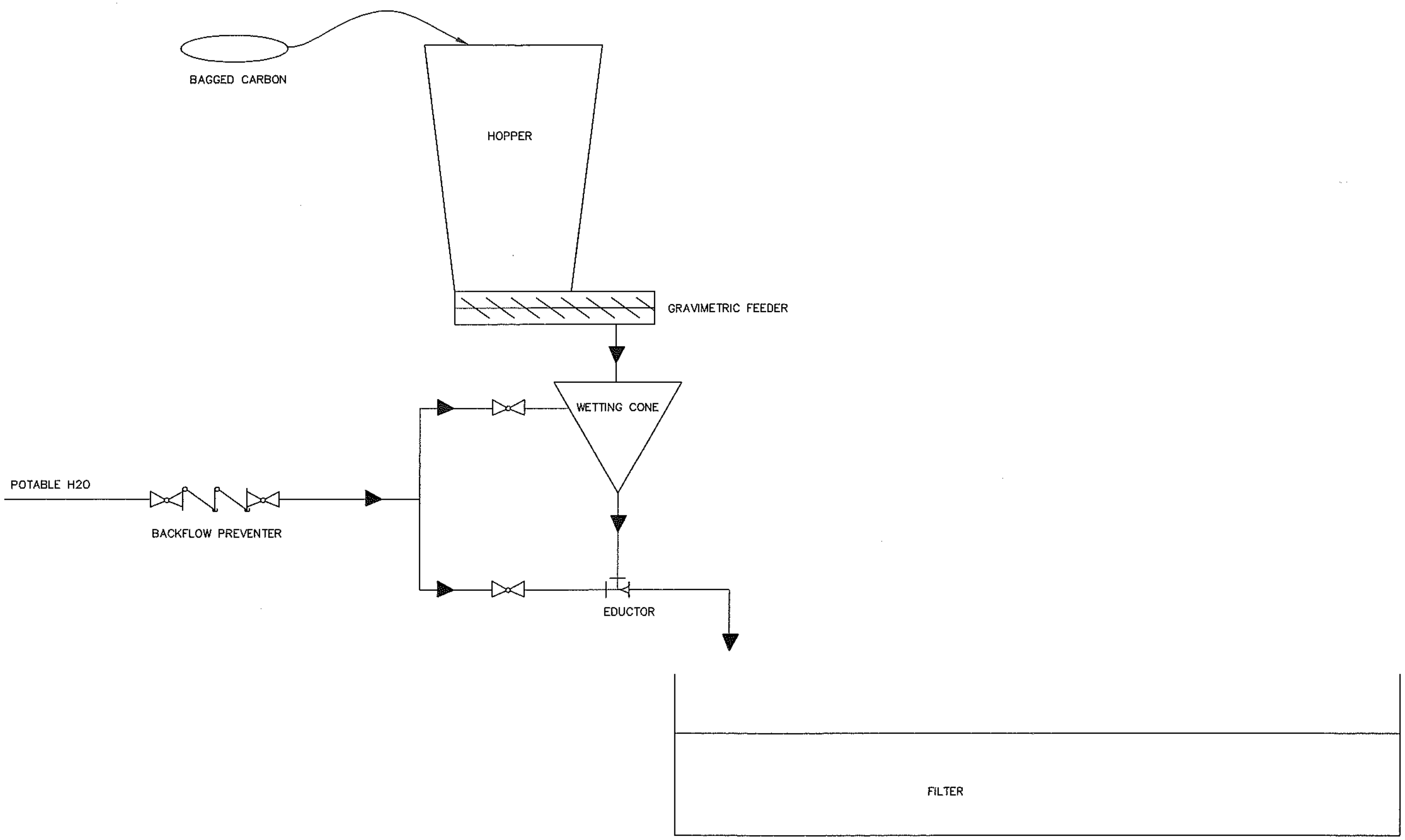
  At the conclusion of the demonstration, temporary feed equipment will be removed and operations will return to original conditions. Plan approval for permanent PAC feed equipment at Collins Park WTP will be concurrent with this demonstration study.

City of Toledo
Collins Park Water Treatment Plant

Design Flow = 120 MGD

PAC Pilot Project





PAC FEED DEMONSTRATION SCHEMATIC
NOT TO SCALE

DATE	STATUS	INITIALS
11-13-14		AMG

TOLEDO, OHIO
DEPARTMENT OF PUBLIC UTILITIES
DIVISION OF WATER TREATMENT

PAC FEED DEMONSTRATION

COLLINS PARK WATER TREATMENT | NOT TO SCALE | 1/1

DRAWING FILE: PAC FEED.dwg

Ohio EPA

John R. Kasich, Governor
Mary Taylor, Lt. Governor
Craig W. Butler, Director

December 3, 2014

Re: City of Toledo
Plan
Correspondence
Drinking Water Program
Lucas County
PWSID: 4801411

Mr. Andy McClure, P.E.
City of Toledo
3040 York Street
Toledo, Ohio 43605

Dear Mr. McClure:

We have reviewed a revised protocol for conducting a powdered activated carbon (PAC) demonstration study at the City of Toledo Water Treatment Plant. The revised protocol was received on November 26, 2014 with additional information being provided on December 1, 2014 and December 2, 2014. The revised protocol is acceptable and the City may begin the demonstration study. The following is a summary of the accepted protocol:

Study Criteria

1. The demonstration study turbidity goal will be to achieve no greater than 0.3 NTU in 95 percent of the samples.
2. The demo filter (Filter No. 10) and the control filter (Filter No. 8) will be backwashed when the headloss reaches 6.5 feet or when the filter has 100 hours of run time, whichever occurs first.

Data Collection

3. Turbidity, headloss, flow, and filter run times will be continuously recorded both on Filter No. 10 (for demonstration) and Filter No. 8 (for control) so a comparative analysis can be done at the end of the study.
4. Turbidity and pH of the settled water upstream of Filter No. 8 and 10 will be monitored at least every four hours during the study. This will provide a frame of reference for the water quality conditions that the filters treated at the time when PAC was applied. In addition, alum and lime dosages that were applied during the study will be included in the report.

Report

5. The 60 day study will begin on December 3, 2014. A report for the 60 day study will be submitted by February 17, 2015 in which the data collected, results of the data analysis, and the conclusions and recommendations are presented and clearly summarized. Turbidity data analysis will include minimum, maximum, and 95th percentile per filter run and per the entire duration of the study as whole. In addition to turbidity and pH results of the filter influent and filter effluent, the report will include alum, lime, and PAC dosages, times for all filter runs, flow rates, etc. The production efficiency for each filter run will be calculated and presented in the report for both Filter Number 10 (the demo filter) and Filter Number 8 (the control filter).

$$PE = \frac{(GUFRV - UBWV)}{GUFRV} \times 100,$$

Where:
PE= Production efficiency
GUFRV = gross unit filter run volume
UBWV= unit backwash volume

Acceptance/Approval Process

6. The City is proposing to have a PAC feed system in place to feed PAC to the filter influent prior to the next HAB season. To evaluate the effects of PAC application on filter performance, the City will conduct a 60 day period study beginning on December 3, 2014. Approximately two weeks into the 60 day study, on December 15, 2014, the City will submit detail plans for the permanent PAC and Potassium Permanganate Feed Systems. Due to the time constraints from the construction and loan timelines, the City will only have results from a small portion of the 60 day study at the time plans need to be approved. The City must provide some preliminary data before the detail plans can be approved.

   The detail plans for the PAC feed system will be approved based on the following special conditions:

   a) Successful results are obtained from the complete 60 day study during the winter of 2014-2015. A report of the results on the 60 day study shall be provided to me by February 17, 2015.

   b) Successful results are obtained from a minimum two week study during a HAB event. A report of the results shall be submitted to me within 30 days of completion of the HAB follow up study.

If both sets of results are found to be acceptable, the condition of the plan approval will be met.

The revised protocol for the PAC study is acceptable. If you have any questions regarding this review, please contact me at (614) 644-2752.

Sincerely,

Judy Stottsberry

Judy Stottsberry, P.E.
Engineering Section
Division of Drinking and Ground Waters

JS/fbs

cc: Warren Henry, City of Toledo
Beth Messer, Assistant Chief, DDAGW
Shannon Nabors, NWDO, DDAGW
Ellen Gerber, NWDO, DDAGW
Justin Bowerman, NWDO, DDAGW
Susan Schell, CO, DDAGW

Appendix C

Ozone Facilities Conceptual Design Report (Black and Veatch)

DRAFT

COLLINS PARK WATER TREATMENT PLANT REDUNDANT CAPACITY IMPROVEMENT PROJECT

Ozone Facilities Conceptual Design Report

B&V PROJECT NO. 186505
B&V FILE NO. 42.0000

This document is issued for preliminary review purposes under the authority of Dianne Sumego, State of Ohio Professional Engineer No. 58331.

PREPARED FOR

City of Toledo, OH

6 FEBRUARY 2015

BLACK & VEATCH
Building a world of difference.

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Executive Summary

The Collins Park Water Treatment Plant (WTP) experiences seasonal algal blooms in it's water source—Lake Erie. Algal blooms have the potential to produce algal toxins; such as, microcystin, that are harmful to public health and produce unique treatment challenges for WTPs. Black & Veatch was tasked with evaluating ozone treatment as one of the advanced treatment processes for the elimination of the microcystin from the City of Toledo's public water supply. The Ozone Facilities Conceptual Design Report presents information on how ozone could be implemented at the WTP as an oxidant for algal toxins, documents the issues associated with incorporating ozone into the raw water supply or post-sedimentation, and provides a conceptual level design for post-sedimentation ozone along with hydraulic impacts, layout requirements, and capital and operating cost estimates. Finally, a construction sequence for major items is presented.

OZONE TESTING AND PROCESS RECOMMENDATIONS

A review of source water quality revealed microcystin concentrations of up to 50 μg/L in Lake Erie water in comparison to the World Health Organization (WHO) established guideline of 1 μg/L for microcystin-LR. The goal of the bench-scale tests was to determine the amount of ozone required to produce non-detectable levels of microcystin-LR from raw water containing 50 μg/L and settled water containing 20 μg/L. Along with the amount of ozone required to achieve this oxidation, ozone demand and decay data was obtained to determine sizing criteria for ozone contact chambers. In addition, ozone's impact on disinfection and bromate formation was estimated.

During bench-scale ozone testing, microcystin-LR was oxidized to less than detectable limits (greater than 99.5 percent removal) when a dissolved ozone residual (0.3 to 0.5 mg/L) was established. This oxidation levels were met without exceeding the established bromate maximum contaminant level (MCL) of 10 ug/L. In some tests where higher ozone residuals were established, settled water bromate exceeded the MCL. In full-scale application, bromate formation would be controlled by limiting the ozone dose to just that required to achieve microcystin oxidation. Those conditions should also provide disinfection credits if the City desired to utilize ozone as a primary disinfectant instead of chlorine.

The bench-scale test results show that a contact time of five (5) minutes was sufficient for an initial dissolved ozone residual of 0.5 mg/L to decay to non-detectable levels. However, for the purposes of this conceptual evaluation, a hydraulic retention time (HRT) of ten (10) minutes was assumed for the ozone contact facilities to allow for operational flexibility during cold water conditions (where ozone decays more slowly) and for the side benefit of disinfection if the City chose to claim partial disinfection credits from ozone.

An evaluation of the conceptual design ozone dosages required for the full-scale facility is completed in this study by adjusting the bench-scale data to account for historical higher TOC that would be present during spring and summer. Taking into account higher TOC levels and changes in raw water pH when algal blooms occur, the recommended transferred ozone dosages for the full-scale facilities are 3.6 mg/L in raw water and 1.7 mg/L in settled water.

OZONE FACILITIES CONCEPTUAL DESIGN

Raw and settled water ozone contact location alternatives are evaluated in this report. Options to provide contact time for raw water ozonation include utilizing the existing raw water pipelines as ozone contactors, installing new pipelines, or a construction a new concrete basin. However, existing pipeline materials are not compatible with ozone and installing new pipelines would not be economically advantageous compared to a conventional contact basin. When considering a new contact basin for ozone, high turbidity levels would result in maintenance challenges. Finally, it is projected that high TOC concentrations would result in additional ozone production and result in higher capital and operating costs than settled water ozonation. Therefore, settled water ozonation was determined to be the most advantageous approach for the Collins Park WTP.

A more detailed conceptual-level evaluation was conducted for settled water ozonation. This included constructing new contact basins to provide 10 minutes of contact time (Settled Water Ozone Contact Basins alternative) and utilizing a portion of the existing settled water conduit flumes as part of the ozonation process – reducing the amount of new contactor construction to approximately 4 minutes (Settled Water Flume Ozonation alternative). Hydraulic evaluation of these alternatives revealed the availability of sufficient hydraulic head capacity for the implementation of the Settled Water Flume Ozonation alternative using gravity-driven flow. However, there is a potential requirement of a low-head-high-flow lift station for high-flow conditions for the Settled Water Ozone Contact Basins alternative. Conceptual design alternatives were developed and illustrated in Section 4.

Ozone generation and liquid oxygen storage and feed facilities, and other related ozone system ancillary facilities were also evaluated, including conceptual sizing and layout of equipment and buildings. Requirements for these facilities are presented in Section 5 and are essentially the same for the two identified settled water ozone contact alternatives.

CONCEPTUAL-LEVEL COST OPINION AND CONSTRUCTABILITY

A conceptual-level evaluation of Capital, Operating & Maintenance, and Present Worth Costs, resulted in an Engineer's opinion of these costs, as quantified in Section 6. These cost opinions are summarized below. Both alternatives are viable, although the Flume alternative has the lower PW.

	SETTLED WATER FLUME CONTACTOR	EXTERNAL SETTLED WATER OZONE CONTACTOR
Capital Cost	$29,000,000	$40,000,000
Annual Operating Cost	$320,000	$330,000
Annual Maintenance at 2 % Equipment Cost	$82,000	$82,000
20 Year Present Worth Cost	**$35,000,000**	**$46,200,000**

Finally, the constructability and potential implementation sequence of the proposed ozone facilities was evaluated at a high level and presented in Section 7. Per this analysis, it appears that construction of the ozone facilities is possible while maintaining the minimum required WTP capacity if the appropriate sequencing is specified in the construction documents. It appears that the best approach would be to substantially complete the new proposed Basins 7 and 8 prior to completing other improvements that require significant plant shutdowns to ensure limited effects on plant capacity. An estimated construction schedule is also provided in Section 7 for consideration.

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1.0 General Requirements

1.1 OVERVIEW

The Collins Park Water Treatment Plant (WTP) takes water from Lake Erie, which is subject to seasonal algal blooms that can produce algal toxins, specifically microcystin. Black & Veatch was retained to evaluate the use of ozone treatment as an option for advanced treatment for oxidation of microcystin at the WTP. This report presents a conceptual design of the facilities required to provide ozone treatment of microcystin.

The conceptual design of ozone treatment facilities includes a water quality review and ozone bench-scale of the raw water and settled water to evaluate ozone effectiveness and byproducts. Also included is an evaluation of potential locations in the WTP process trains for ozone contact facilities, as analysis of the existing WTP hydraulics, the conceptual design of liquid oxygen (LOX) facilities, the conceptual design of ozone generation facilities, a review of two recommended ozone contact locations, a life-cycle cost analysis, a preliminary construction schedule, and general phasing considerations.

1.2 BACKGROUND

The City of Toledo (the City) Collins Park WTP currently has six (6) Sedimentation and Recarbonation Basin trains, each with a rated capacity of 20 MGD of capacity, and associated conventional, high-rate, granular media Filters (5 filters per 20 MGD basin train) for a total rate capacity of 120 MGD. An overview of the existing facility is provided in Figure 1-1. Redundant capacity improvements are planned in the near future for this WTP, which include addition of two (2) Sedimentation and Recarbonation Basin trains and ten (10) Filters to provide a redundant capacity of 40 MGD.

Due to the increasing concerns with the harmful algal blooms and algal toxin in the western basin of Lake Erie, the Redundant Capacity Improvements project will include advanced treatment processes for oxidation and removal of these toxins, specifically microcystin. Prior to developing designs for the above improvements, the City desires to complete an in-depth study covering the implementation of either ozone or granular activated carbon (GAC) to provide additional treatment barriers in addressing microcystin removal on a long term basis. This report focuses on the evaluation of conceptual ozone treatment facilities.

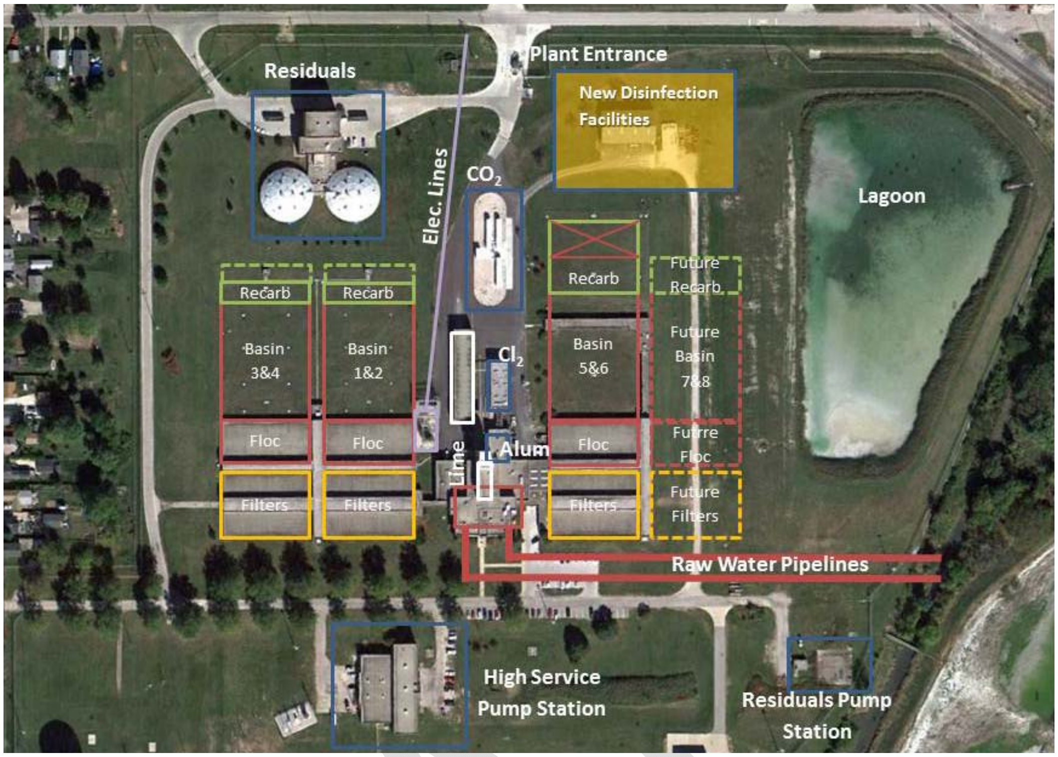

Figure 1-1 Existing Facility Plan

1.3 PROJECT DESCRIPTION

Evaluation of ozone treatment for microcystin oxidation includes the following tasks:

1.3.1 Task 1 – Ozone Testing and Design Basis Evaluation

- Water quality review of the last three years data.
- Conduct bench-scale tests on the raw water and settled water to determine ozone demand and decay kinetics, establish ozone dose and contact times to achieve targeted oxidation/disinfection levels and evaluate disinfection by-product formation levels.
- Review bromate formation potential and bromate control strategies if necessary.
- Using the established ozone dosage, calculate the size and number of ozone generators and liquid oxygen (LOX) facilities.
- Establish the hydraulic residence time for the ozone contactors, the need to quench the ozone residual after the required contact time, and hydraulic losses through the process.
- Review current filter backwashing process and whether existing backwashing approach with chlorinated backwash could impact biological activity.

1.3.2 Ozone Facilities Conceptual Design

- Based on the results of the Ozone Testing and Design Basis Evaluation task above, a conceptual design of ozone facilities to be incorporated at the Collins Park WTP and a life-cycle cost analysis.
- Develop conceptual facility layout(s) for the ozone facilities consisting of provisions for ozone contact time and ozone generation facilities including liquid oxygen storage and liquid oxygen vaporizers.
- Review hydraulic impacts and alternatives for integrating post settling ozonation into the existing treatment process. This will include investigating possible alternatives for re-purposing existing recarbonation facilities to serve in some manner as an ozone contactor.
- Prepare overall facility arrangement drawings showing plan and section views for the ozone contacting approach and ozone generation facilities for the alternative selected.
- Conduct an assessment of the plant electrical system and establish the method for supplying power to the proposed ozone facilities.
- Prepare conceptual site and piping plans showing the siting of facilities and interconnecting piping between facilities.
- Prepare opinion of probable construction costs for the alternative(s) evaluation and associated operating cost for the facilities.

1.4 ABBREVIATIONS AND ACRONYMS

The following abbreviations and acronyms are used in this Conceptual Design Report:

AOC	Assimilable organic carbon
BAF	Biologically active filtration
BW	Backwash
ºC	Degrees Celsius
CCL	Contaminant Candidate Lists
CE	Categorical Exclusion
cfs	Cubic feet per second
CT	Contact time
DIP	Ductile Iron Pipe
EPA	United States Environmental Protection Agency
fps	Feet per second
FRP	Fiberglass reinforced plastic
ft	Feet
gal	Gallons
gfd	Gallons per square feet per day
GAC	Granular Activated Carbon
GOX	Gaseous oxygen
gpd	Gallons per day
gpm	Gallons per minute
gpm/sf	Gallons per minute per square foot
HGL	Hydraulic grade line
HPC	Heterotrophic plate counts
HVAC	Heating, ventilating, and air conditioning
IBC	International Building Code
I&C	Instrumentation and control
I/O	Input / Output
kW	Kilowatt
kWh	Killowatt-hour
lb(s)	pound(s)
LOX	Liquid oxygen
MCL	Maximum contaminant limit
MG	Million gallons
mgd	Million gallons per day
mg/L	Milligrams per liter
MIB	2-methylisoborneol
N	Nitrogen
NAD	North American Datum
NGVD 29	National Geodetic Vertical Datum of 1929
NTU	Nephelometric turbidity unit
O_3	Ozone
OEPA	Ohio Environmental Protection Agency

O&M	Operations and maintenance
ORP	Oxidation-reduction potential
OSHA	Occupational Safety and Health Administration
PAC	Powdered Activated Carbon
pcf	Pounds per cubic foot
PLC	Programmable logic controller
ppb	Part per billion (same as ug/L)
ppd	Pounds per day
pph	Pounds per hour
ppm	Parts per million
PRV	Pressure reducing valve
psf	Pounds per square foot
psi	Pounds per square inch
psig	Pounds per square inch (gage)
PSU	Power supply unit
PVC	Polyvinyl Chloride
PVDF	Polyvinylidene Fluoride
rpm	Revolutions per minute
RTU	Remote terminal unit
SCADA	Supervisory Control and Data Acquisition
scfh	Standard cubic feet per hour
scfm	Standard cubic feet per minute
sec	Second
SS	Stainless Steel
TBD	To be determined
TDH	Total dynamic head
TDS	Total dissolved solids
TOC	Total organic carbon
µg/L	Micrograms per liter
UPS	Uninterruptible power supply
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
WHO	World Health Organization
WSE	Water Surface Elevation
WTP	Water Treatment Plant

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2.0 Water Quality and Treatment Investigations

2.1 RAW WATER SOURCE – LAKE ERIE

2.1.1 General Description

Raw water is drawn from a shallow intake in Lake Erie and treated with permanganate for the control of zebra mussels. The current approach for control of algal toxins and taste- and odor-causing compounds is the addition of powdered activated carbon (PAC) applied at the raw water pumping station. The water is then conveyed through two pipelines to the Collins Park WTP where the water undergoes softening, recarbonation, and filtration by multimedia filters. Downstream from filtration, chlorine is applied to achieve primary disinfection and held in reservoirs prior to conveyance to the distribution system.

Source water quality challenges include:

- Presence of algal toxins; namely, microcystin, during late summer and early fall months (June through September)
- Turbidity events
- Taste- and odor-causing compounds
- Presence of zebra mussels that potentially restrict flow into the intake or through the raw water pipelines

2.1.2 Trophic Status and Algal Blooms

Microcystin is an algal toxin that is produced by cyanobacteria. Microcystin is not currently regulated by the United States Environmental Protection Agency (USEPA); however, the USEPA and the OEPA are currently evaluating the future enforcement of limits. Nevertheless, Microcystin is included in the USEPA's Contaminant Candidate Lists (CCL) 3. The World Health Organization (WHO) has established a guideline of 1 ug/L for microcystin-LR.

The concentration of the algal toxin experienced in the severe bloom that occurred in the summer of 2014 is illustrated on Figure 2-1. The maximum observed value was 50 ug/L. The majority of the contaminant is present within the algal cells which if left intact, the toxin and the algal cell can be removed through sedimentation and filtration processes. The concentration of microcystin present outside the algal cell (extracellular) is illustrated in Figure 2-2. The maximum extracellular microcystin value observed was 5 ug/L.

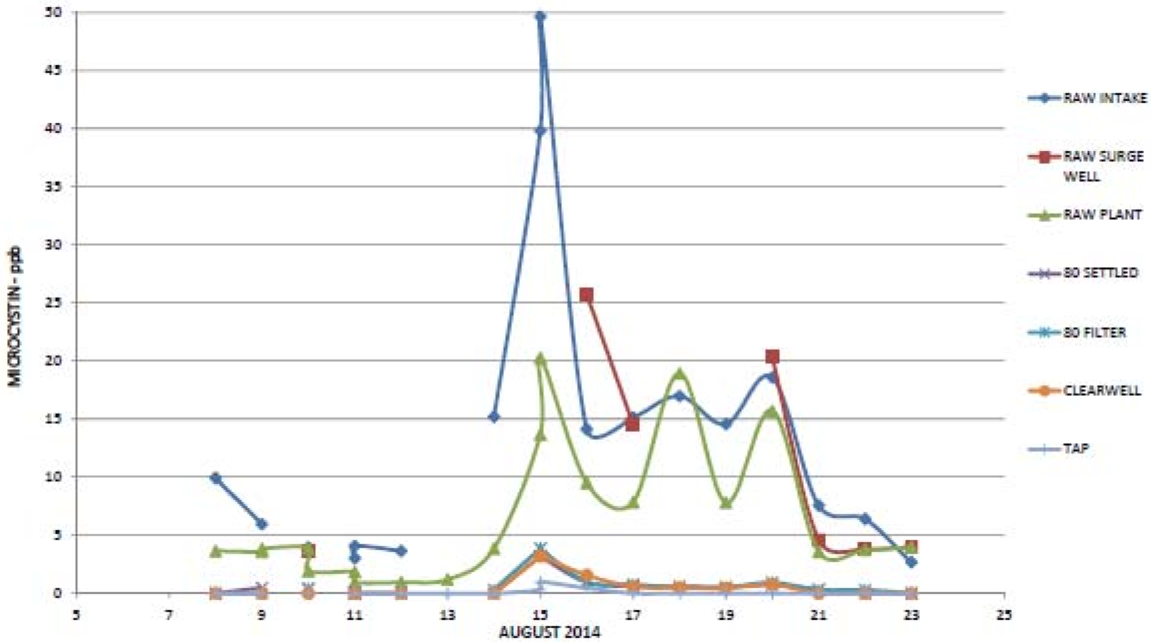


Figure 2-1 Microcystin Levels (August 2014)

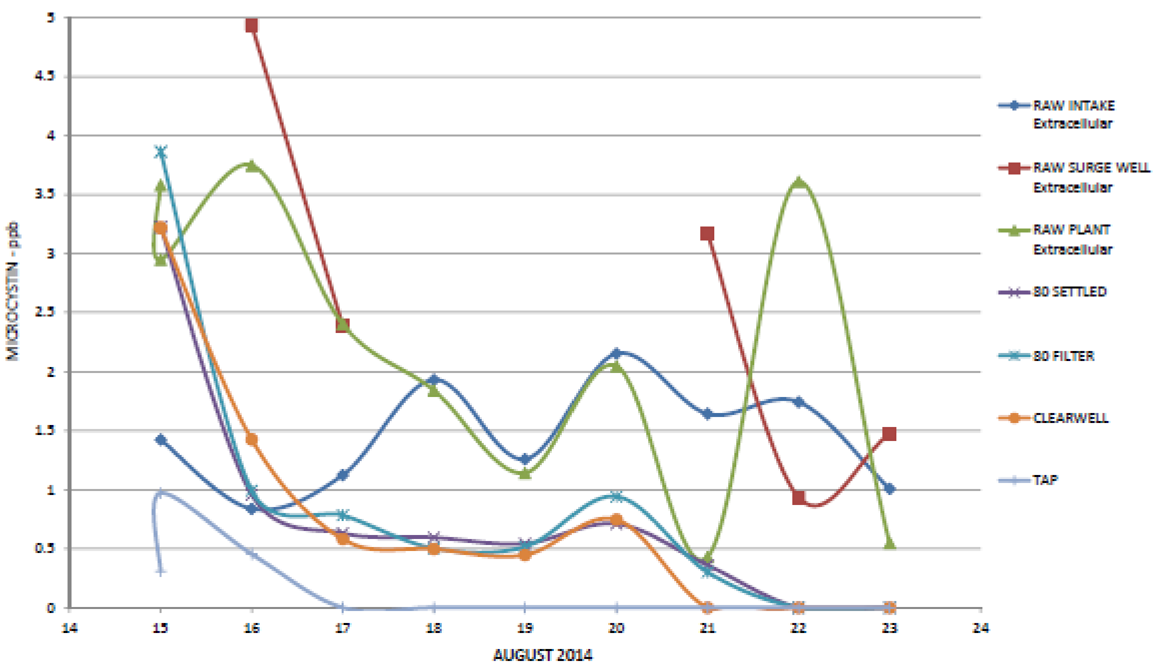


Figure 2-2 Extracellular Microcystin Levels (August 2014)

2.1.3 Turbidity

Raw water and settled water turbidity data was evaluated because the higher quantity of solids present generally require higher ozone dosages and the potential for solids to settle out in ozone contact basins, which would require additional maintenance. Cleaning ozone contact basins requires special care, as ozone contact basins are considered confined spaces. Therefore, ozone contact basins require the evacuation of ozone off-gas to provide a safe working environment, which generally takes several days or even multiple basin fill and drain sequences.

Raw water and settled water turbidity is illustrated in Figure 2-3. Raw water turbidity can exceed 100 NTU several times throughout the course of the year (the average is 40 times per year). If raw water ozonation were to be implemented, this condition would require that staff enter the basin periodically to remove the deposited solids or risk the solids going anaerobic or sceptic and releasing iron, manganese, or taste- and odor-causing compounds back into the water. The high dissolved oxygen content present in the water following ozonation along with the high pH of the softening process would oxidize the metals again, but the retreatment process is undesirable.

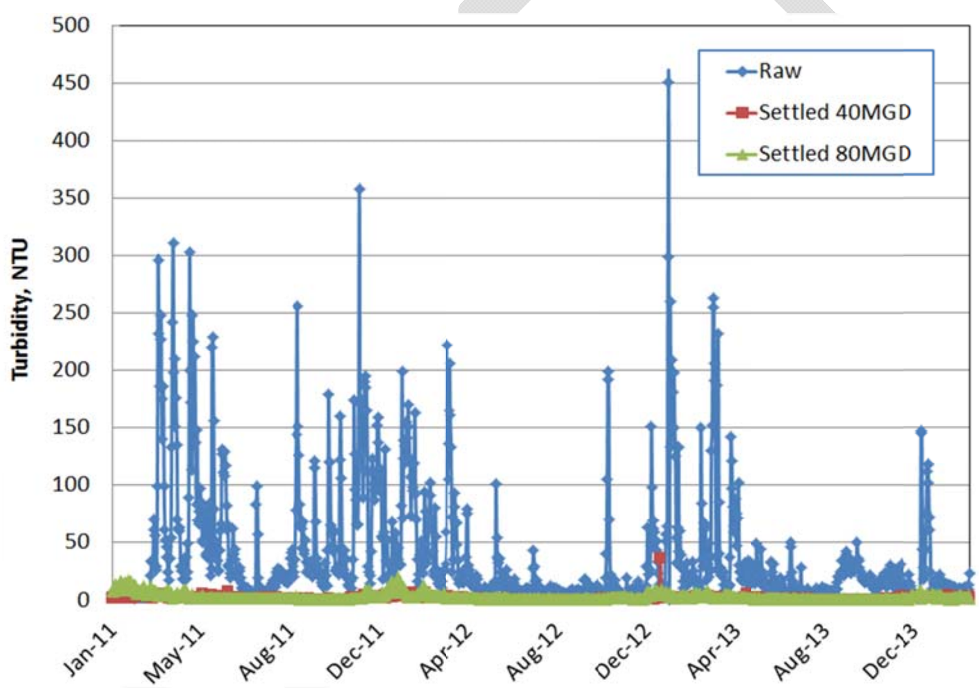


Figure 2-3 Average Daily Turbidity Values (2011 - 2014)

For the settled water, the turbidity is low—generally less than 2 NTU, with an average is 1.8 NTU. This results in a low potential for solids deposition and in reducing the need to clean settled water from contactors.

2.1.4 Taste- and Odor-Causing Compounds

The raw water periodically contains aesthetically unpleasing tastes and odors, which are removed by powdered activated carbon (PAC). A dosage of 4 mg/L is normally applied year round to control taste- and odor-causing compounds. The PAC is applied at the raw water pumping station and contact time is provided in the raw water pipeline. The PAC is removed during sedimentation and filtration. The annual operating cost for applying PAC applied using a 4 mg/L dosage is between $1.5 million and $2.0 million.

At numerous WTPs in the United States, ozone is used for the control of taste- and odor-causing compounds; namely, geosmin and 2- methylisoborneol (MIB). If ozone is implemented for microcystin oxidation at the Collins Park WTP, then ozone use could also provide the additional benefit of reducing the taste- and odor- causing compounds—potentially eliminating the need for PAC.

2.1.5 Zebra Mussels

Zebra mussels are present in Lake Erie and the use of permanganate is necessary to control zebra mussels in order to retain the hydraulic capacity of the intake and raw water pipelines. The ozonation process will need to allow continued use of permanganate for mussel control. Ozone applied to the raw water will potentially oxidize the manganese dioxide precipitant formed after treatment with permanganate back to permanganate, which results in purple colored water. The re-formation of permanganate is not problematic in the process but the oxidation of manganese dioxide exerts an ozone demand. Fortunately, organic reactions occur prior to the oxidation of manganese dioxide with ozone, which requires a high dissolved ozone residual and high oxidation-reduction potential (ORP).

Manganese, in the form of manganese dioxide, is removed during sedimentation and filtration, therefore, the use of permanganate in the raw water pipeline is not expected to impact the design of the ozone system if ozone is applied to the settled water.

2.1.6 Total Organic Carbon

Total organic carbon (TOC) data is summarized in Figure 2-4 and Figure 2-5. Generally, TOC is much higher in the raw water than in the settled water. The raw water TOC has exceeded 10 mg/L whereas the settled water TOC is always less than 4.0 mg/L. As TOC exerts an ozone demand, it must be accounted for in estimating ozone dosages for full-scale operations. The use of historical TOC values can be used to estimate the design ozone dosage. The maximum TOC value is generally not used to size the ozone generating equipment, instead the 90th, 95th or 99th percentile value is used, as illustrated in Figure 2-5. Bench-scale testing was performed on raw and settled water containing 2.0 and 1.4 mg/L TOC, as described later in Section 3. The TOC present in the raw water and settled water samples tested represent the 1st and 5th percentile TOC values, respectively.

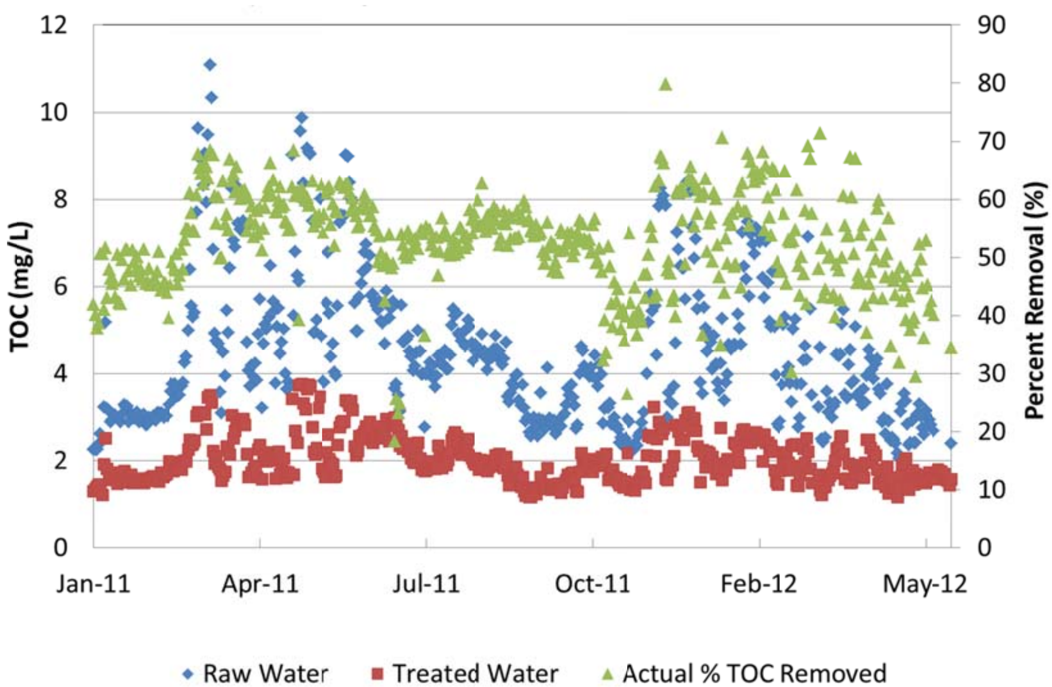


Figure 2-4 Raw and Settled TOC Data (January 2011 through June 2012)

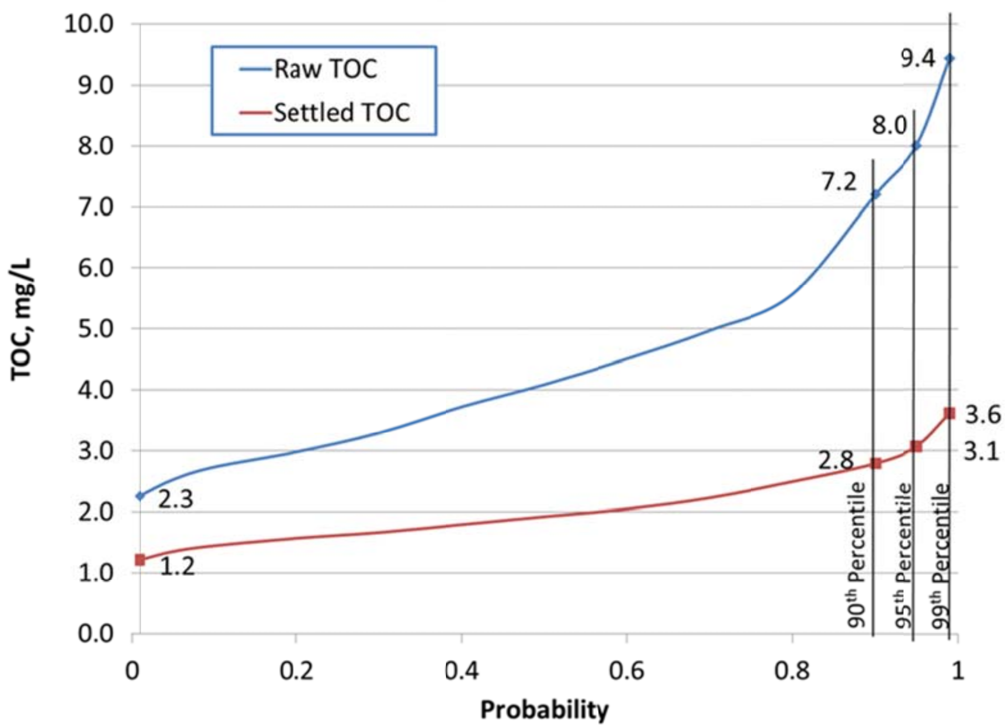


Figure 2-5 Raw and Settled Water TOC Probability Plot (January 2011 through June 2012)

2.1.7 Bromide

Bromide (Br^-) is a compound that reacts with ozone to form a regulated ozonation disinfection byproduct, bromate (BrO_3^-). Bromate has a primary Maximum Contaminant Limit (MCL) of 10 ug/L and must be measured monthly during ozone use. Compliance is through a running annual average (12 months), which must be less than the MCL.

Limited raw water bromide data was collected, but the concentration of the compound in the Great Lakes is not anticipated to vary significantly given the volume of water in the system. The bromide concentration ranged between 21 to 42 ug/L during testing, which is quite low since national average was approximately 70 ug/L (based on Information Collection Rule data analysis). Additional bromate formation information is provided in Section 3.3.

2.2 FINISHED WATER QUALITY GOALS

Finished water quality goals are as follows:

1) Provide potable water in compliance with all Ohio Environmental Protection Agency (OEPA) limits, both primary and secondary standards.
2) Provide aesthetically pleasing water free of mineral deposits, iron and manganese, and taste- and odor-causing compounds.
3) Provide water with undetectable levels of algal toxins.

3.0 Ozone Testing and Process Development

3.1 OZONE DEMAND AND DECAY TESTING

Bench-scale ozone demand and decay testing was performed using samples obtained from the Collins Park WTP on December 4, and 18, 2014. The City of Toledo shipped raw and settled water to the Black & Veatch research facility in Kansas City to undergo bench-scale ozone testing. Water quality data is listed in Table 3-1.

Table 3-1 Bench-scale Raw and Settled Water Quality Data

DESIGN VALUES	DECEMBER 4		DECEMBER 18	
	Raw	Settled	Raw	Settled
Turbidity, NTU	23	0.5	13	0.6
pH	8.0	9.6	8.0	9.6
Pheonol Alk., mg/L as $CaCO_3$	0	16	0	16
Total Alk., mg/L as $CaCO_3$	90	60	93	51
Total Hardness, mg/L as $CaCO_3$	114	80	119	80
Non-carb. Hard., mg/L as $CaCO_3$	23	23	26	30
Total Organic Carbon, mg/L	1.9	1.3	2.0	1.4

Ozone demand can be determined through addition of ozone and measurement of a dissolved ozone residual. Dissolved ozone residual was measured 30 seconds after the addition of the ozonated solution to provide adequate mixing and satisfy initial demand reactions. Therefore, all stated ozone demand values are stated as the demand after 30 seconds of reaction time.

Ozone demand is a function of water quality, primarily the presence of reduced compounds such as iron (II); manganese (II) or sulfide and organic compounds. Given the samples collected did not contain soluble iron, manganese, or sulfide, initial ozone reactions were with organic carbon. Initial ozone residual is plotted against applied ozone dosage in Figure 3-1. The x-intercept is the ozone demand for the two waters tested. Ozone demand was 0.35 and 0.10 mg/L for the raw and settled water and TOC was 2.0 and 1.4 mg/L for the raw and settled water, respectively. This represents an ozone to TOC demand ratio of approximately 0.07 to 0.18 mg ozone per mg TOC, which is below the range typically observed, 0.25 to 0.50 mg ozone per mg TOC. Ozone demand was very low in the raw and settled water, likely the result of performing the test on water that has some of the lowest recorded TOC values relative to the historical TOC data from 2011 and 2012.

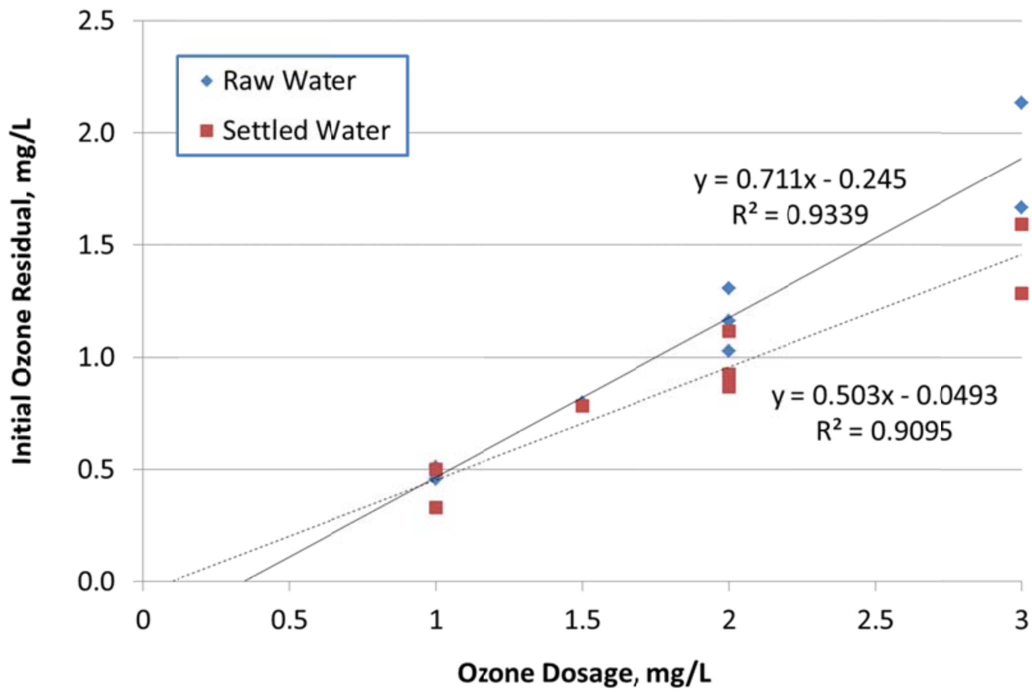


Figure 3-1 Ozone Demand Data (December 2014)

Ozone residual data is illustrated in Figure 3-2. Transferred ozone dosages of 1.0, 2.0, and 3.0 mg/L were applied to the raw water and settled water. The raw water dissolved ozone residual values are the solid lines and the settled water residual values are the dashed lines. The initial residual after 30 seconds of contact time was similar for both the raw water and settled water, likely because the organic content in the two samples was similar. The decay rate was much faster in the settled water because the pH of the settled water was much higher than the raw water, 9.7 vs. 8.2 respectively. Testing was performed at 23°C, therefore, results illustrate decay rates during summer conditions when algal toxin concentrations are highest. Ozone residual is more persistent in cooler water temperature. Results for the tests performed on December 4, 2014, are presented because ozone decayed more slowly in those samples. Therefore the more conservative data for the two rounds of testing are illustrated from a perspective of hydraulic retention time (HRT) required to allow full ozone decay without the need for quenching.

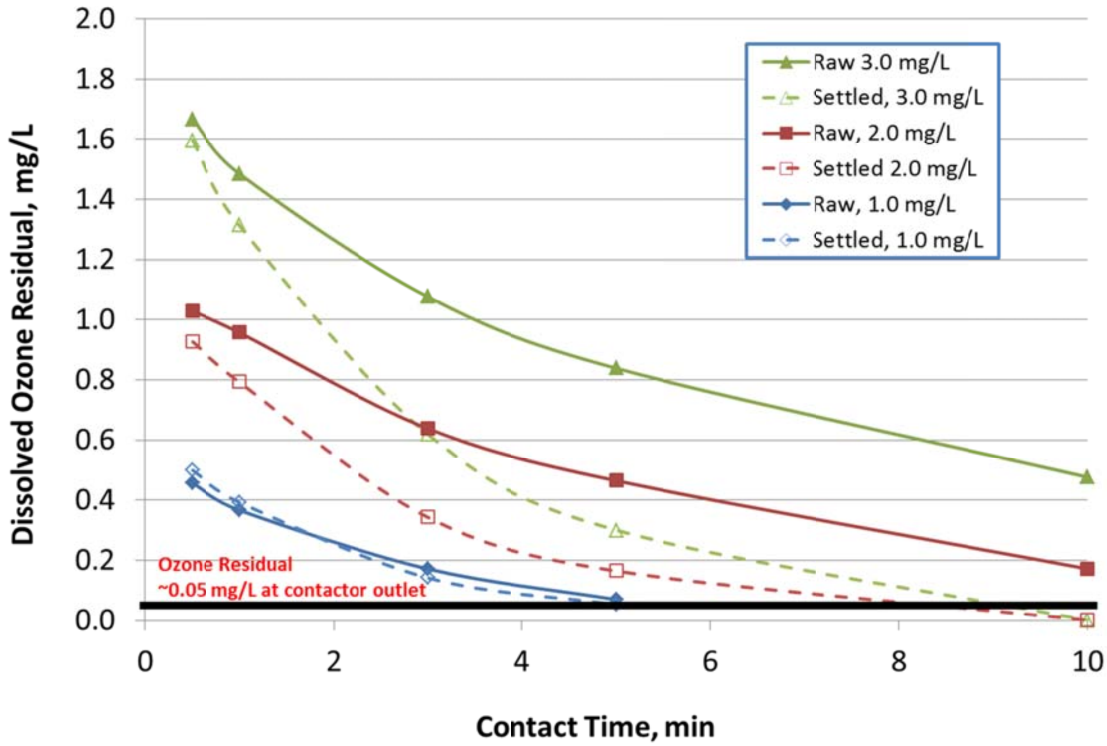


Figure 3-2 Dissolved Ozone Residual Data (December 4, 2014)

3.2 MICROCYSTIN OXIDATION TESTING

The raw water and settled water was spiked with microcystin and ozonated to develop an oxidation curve. The first round of testing (December 4, 2014) provided data for ozone residual decay rates and bromate formation but all of the microcystin concentrations measured were below detectable levels including the unozonated samples. It is likely that the microcystin source did not go into solution. Therefore, a second round of testing was performed that utilized a different method to solubilize the microcystin.

The second round of testing, (December 18, 2014) summarized in Table 3-1 resulted in measurable concentrations similar to the desired spike. Spiked concentrations of 50 and 20 ug/L in the raw water and settled water were selected. The measured values were 36 and 20 ug/L in the raw and settled waters, respectively. Testing was performed using transferred ozone dosages of 1, 2, and 3 mg/L. All ozone dosages resulted in oxidation of the microcystin to the non-detectable level; that is, <0.10 ug/L. The conclusion is that a dissolved ozone residual of at least 0.3 mg/L provides complete microcystin oxidation. Ozone dosages less than 1.0 mg/L were not evaluated.

Oxidation of micro-contaminants can often be predicted by the CT achieved. The resultant CT values for the test conditions and the log *Giardia* inactivation credit that would be achieved is provided as a bench-mark comparison. Providing inactivation credit is not a unit process goal although disinfection would be occurring.

Table 3-2 Microcystin Oxidation Results (December 18, 2014)

LOCATION	RAW			SETTLED		
pH	8.2			9.7		
Ozone Dosage, mg/L	1	2	3	1	2	3
Raw Microcystin, ug/L	36			20		
Effluent Microcystin, ug/L	ND[1]	ND	ND	ND	ND	ND
Oxidation Efficiency, percent	≥99.7%	≥99.7%	≥99.7%	≥99.5%	≥99.5%	≥99.5%
Initial Residual, mg/L	0.5	1.2	2.1	0.3	0.9	1.3
CT (10 min), mg/L-min	0.8	3.7	10.4	0.3	0.8	1.3
***Giardia* Inactivation, logs**	4.8	21	60	1.4	4.5	7.2

[1] Analytical detection limit = 0.10 ug/L

3.3 BROMATE TESTING

Bromate is formed through oxidation of naturally occurring bromide and ozone. The maximum contaminant limit (MCL) for bromate is 10 ug/L. If ozone is used in the treatment process, bromate must be measured monthly. Compliance is through a running annual average (12 months), which must be less than he MCL. Factors that impact bromate formation include:

- pH – in general, higher pH results in more bromate formation.
- Ozone exposure (CT) – in general, higher ozone residuals (and resulting CT values) result in more bromate formation.
- TOC – in general, higher TOC results in less bromate formation as ozone reacts with the organic compounds (resulting in a lower ozone residual and CT value).

Means to control bromate formation include:

- Reducing pH
- Reducing ozone exposure
- Ammonia addition (ammonia reacts with one of the intermediaries in bromate formation resulting in lower bromate formation)
- Combinations of chlorine and ammonia addition

The bromide concentration ranged between 21 to 42 ug/L during testing, which is quite low since national average was approximately 70 ug/L (based on Information Collection Rule data analysis). Bromate values for the two rounds of testing are illustrated in Figure 3-3 and Figure 3-4. Bromate levels were higher in the settled water because the pH was higher. The bromate formation was not consistent between the two rounds of testing. The ozone decay rate was much faster in the second round of testing resulting in lower CT values. When the data is plotted against ozone exposure (CT), the formation of bromate was consistent between the two rounds of testing.

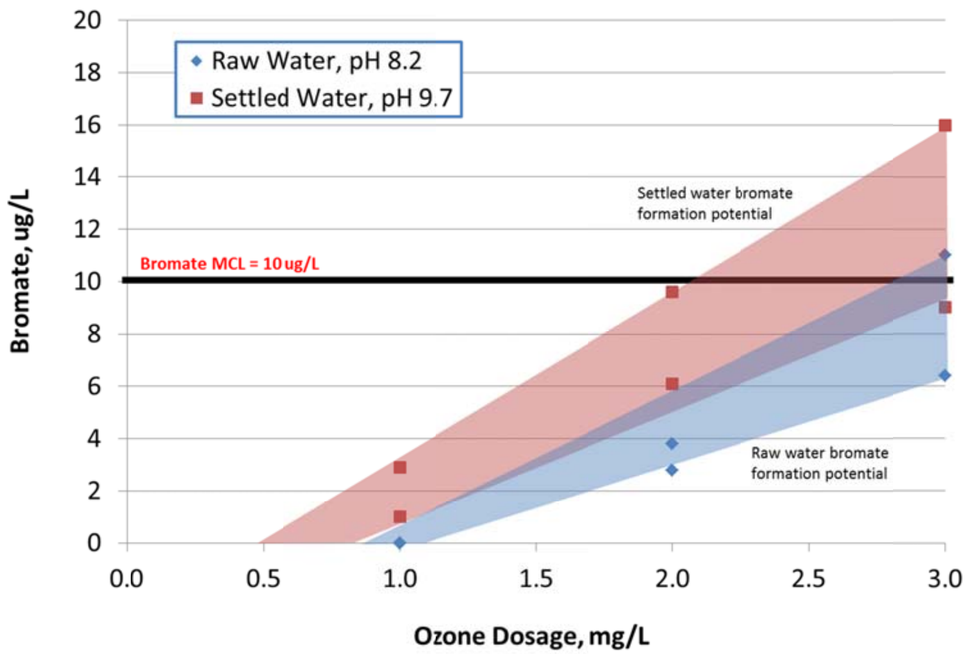


Figure 3-3 Bromate Formation and Ozone Dosage (December 2014)

Bromate levels appear to be less than the MCL if a CT value of less than 2.0 mg/L-min is maintained in the settled water and less than 10 mg/L-min in the raw water. Note that 0.5 log inactivation of *Giardia* using ozone is 0.12 and 0.48 mg/L-min at 20° and 1° C, respectively. Therefore a CT of 2.0 mg/L-min represents a very high level of disinfection. Initial residual of 1 mg/L in settled water and 1.5 mg/L in raw water are the upper limits for initial residual to maintain bromate levels below the MCL.

Microcystin oxidation was accomplished with initial dissolved ozone residuals of 0.3 and 0.5 mg/L in settled and raw water, respectively. Therefore, bromate should remain below the MCL and microcystin treatment goals can be achieved by employing a bromate mitigation strategy of minimizing ozone exposure. This will require ozone residual monitoring and control.

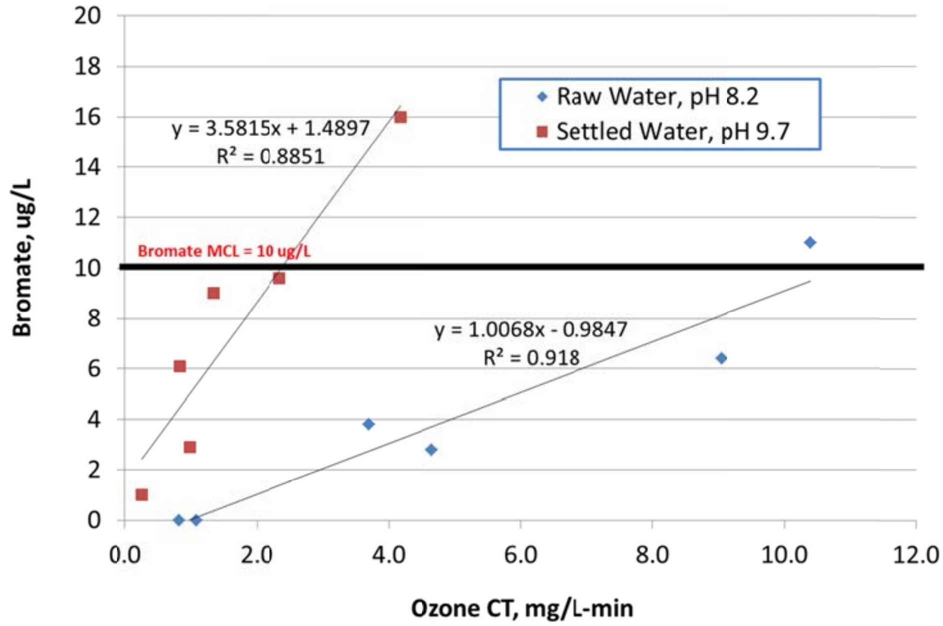


Figure 3-4 Bromate Formation and Ozone Exposure (December 2014)

3.4 ASSIMILABLE ORGANIC CARBON TESTING

Assimilable organic carbon (AOC) is a highly biodegradable fraction of TOC. It is formed through treatment with ozone as ozone is a powerful oxidant that reacts with complex TOC molecules and in some cases breaks the TOC down to move biologically available sources of carbon. High levels of AOC can results in increased biogrowth in the distribution system leading to increases in heterotrophic plate counts (HPCs) and faster decay of the chlorine residual.

AOC formation is controlled through ozonation of waters lower in TOC, controlling the ozone dosage to prevent overdosing, and through biological processes such as biofiltration.

AOC samples were not taken during the bench-scale testing. Additional testing will be needed to confirm decay rates and design dosages and AOC should be sampled during that time. A chlorine residual is not carried through the filters at the CPWTP, therefore, some biological activity is likely occurring across the filters. It is expected that the filters will provide some AOC removal upon implementation of ozonation and an additional AOC removal step will not be necessary.

3.5 TASTE AND ODOR CONSIDERATIONS

Taste- and odor-causing compound oxidation was not evaluated during the bench-scale testing. Generally, ozone applied at dosages to provide primary disinfection, achieve high levels of taste- and odor-causing compound (geosmin and MIB) oxidation. To quantify the reduced dosage of PAC for taste and odor control, additional bench-scale testing may be performed prior to detailed design.

3.6 CHLORINATED FILTER WASHWATER CONSIDERATIONS

Removal of AOC through biological filtration requires the presence of bacteria on the filter media. Backwashing with chlorine tends to result in less biomass. During summer, when the water temperature is above 20 °C and bacteria are able to reproduce quickly, the effect of chlorinated backwash is minor or perhaps even beneficial as it controls biomass that could otherwise restrict flow paths through the media and increase headloss in the top few inches of the filter bed. During the winter however, when water temperature is cooler and biological activity is much slower, the use of chlorine in the backwash water can decrease the performance of the biological filter in regard to AOC removal. Allowing higher levels of AOC in the distribution system during winter does not necessarily result in faster chlorine decay or higher HPC values because biological activity in the distribution system is slowed as well.

Many utilities successfully practice biofiltration with chlorinated backwash water. Implementing biofiltration does not automatically mean that a non-chlorinated backwash supply is required, but to optimize the process, particularly during cool water periods, a non-chlorinated backwash supply should be provided. Means to do so include connected the backwash pump supply header upstream from chlorine addition or use of a chlorine quench chemical.

3.7 PROCESS DEVELOPMENT RECOMMENDATIONS

3.7.1 Design Ozone Contact Time

Ozone contactors are sized based on the initial dissolved ozone residual needed to meet process goals and the resultant decay rate. The contactor either must have sufficient contact time to allow the dissolved ozone residual to naturally decay to less than 0.05 mg/L or include a quench chemical to react with the remaining ozone residual present at the outlet of the contactor. Operations staff would select a target dissolved ozone residual for microcystin (and taste and odor oxidation) and the dosage adjusted to maintain the target dissolved ozone residual.

As determined previously in Section 3.2, process treatment goals were met using an initial dissolved ozone residual of 0.3 to 0.5 mg/L or lower (raw and settled). At 23 °C, the ozone contactor will be sized to allow an initial dissolved ozone residual of 0.50 mg/L to decay to less than 0.05 mg/L. The size of the ozone contactor therefore needs to be a minimum of 5 minutes as shown in Figure 3-2.

The bench-scale testing did not include determination of the decay rate at cooler water temperatures. The decay rate for previous projects[1] was analyzed to develop a ratio for the ozone decay rate at warm and cool temperature. The range of results indicated that decay rate could be two (2) to five (5) times slower at the coldest water temperature. The most complete analysis included data (n=12,251 points) that was summarized for a period of ten years. The range in decay rate therefore represents variations in water quality year to year as well as temperature. The results are illustrated in Figure 3-5. For the temperature range between 5 and 22 °C, the decay rate (k) averaged approximately -0.025 to -0.125, respectively. Therefore, a reasonable approximation would be to assume the decay rate would be five (5) times slower at cooler water temperatures.

[1] Sites investigated included: Phoenix, Arizona; Lincoln, Nebraska; Modesto, California; and Vancouver, British Columbia.

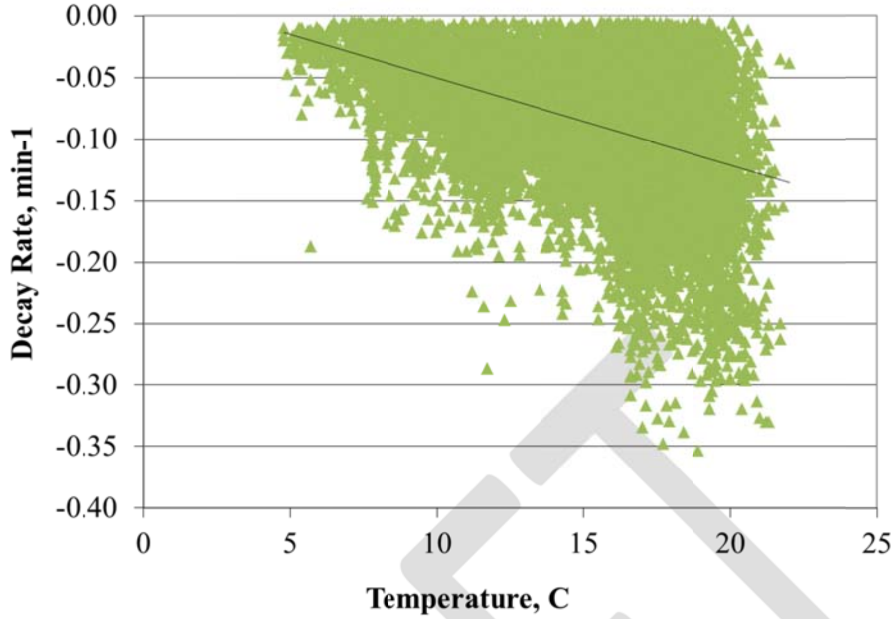
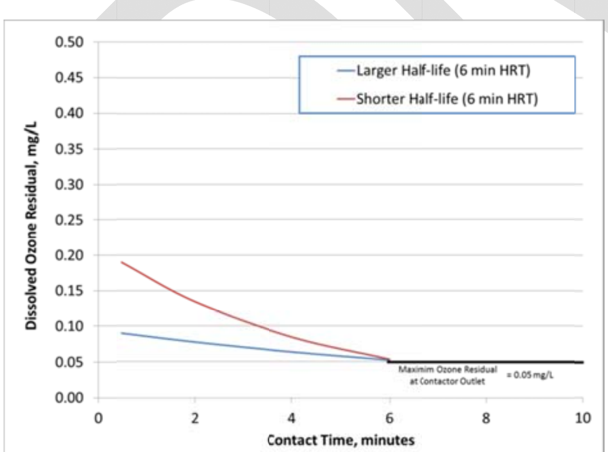
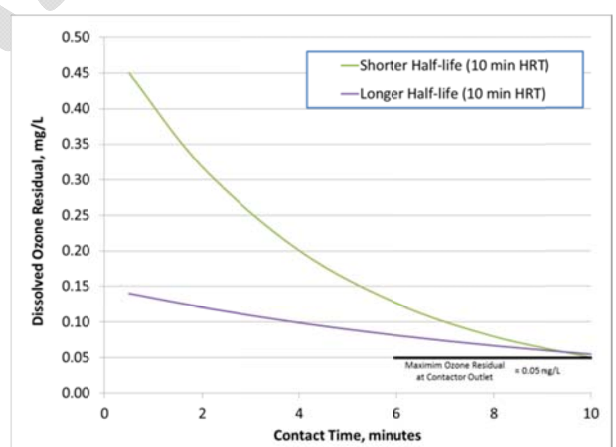


Figure 3-5 Dissolved Ozone Decay Rate (1996 to 2005)

The approach was to evaluate the two decay rates observed in the bench-scale testing and use a factor of five (5) to calculate the decay rate at the cooler water condition. The resultant dissolved ozone residuals are presented on Figure 3-6. The graphs illustrate the maximum allowable initial dissolved ozone residual for ozone contactors with a hydraulic retention time of 6 and 10 minutes that would result in a final dissolved ozone residual of 0.05 mg/L. To prevent unintended exposure to ozone, maintaining a dissolved ozone residual of less than 0.05 mg/L is desirable. Values in excess of 0.05 mg/L can result in corrosion on non-compatible materials and off-gassing to levels that exceed OSHA personal safety limits.



6-min Contact Time



10 min Contact Time

Figure 3-6 Dissolved Ozone Residual (1 °C)

The figures illustrate that adding contact time allows higher initial dissolved ozone residuals. Adding a quench chemical is a second approach that would allow higher initial dissolved ozone residuals. The excess residual present at the outlet of the contactor is reduced using the quench chemical. E.g.: sodium bisulfite, hydrogen peroxide, or calcium thiosulfate.

Commonly, providing *Giardia* inactivation credit is an ozonation process goal. Regardless if inactivation credit is obtained, use of ozone is resulting in some level of *Giardia* and virus inactivation. Future flexibility including means of providing disinfection credit should be considered in the ozone contactor basin design.

The disinfection performance of 6 and 10 minute hydraulic retention time (HRT) ozone contactors is listed in Table 3-3. At the coldest water temperature, a contactor sized for 6 minute HRT may not provide 0.5 *Giardia* inactivation credit. To achieve primary disinfection, a contactor with a hydraulic retention time of 6 minutes that includes a quench chemical or a contactor with a hydraulic retention time of 10 minutes is required.

Table 3-3 Settled Water Quality Data (December 4, 2014)

	6 MIN HRT CONTACTOR	6 MIN HRT CONTACTOR & QUENCH CHEMICAL	10 MIN HRT CONTACTOR
Observed Half-life (22 °C), min	1.4	1.4	1.4
Cold Water Half-life (1 °C)[1], min	7.0	7.0	7.0
Maximum initial dissolved ozone residual[2], mg/L	0.09	0.18	0.14
CT Calculated (Simple 3-point Method), mg/L-min	0.39[3]	0.75[3]	0.75[4]
Giardia Inactivation Credit[5], logs	**0.26**	0.50	0.50

[1]The cold water half-life was calculated using the observed half-life and multiplying by a factor of five (5)

[2]Maximum initial dissolved ozone residual that can be achieved and have an outlet residual of $\leq$ 0.05 mg/L

[3]CT calculated based on simple CxT method using three dissolved ozone monitoring locations placed at 2, 4, and 6 minutes in the 6 minute HRT contactor.

[4]CT calculated based on simple CxT method using three dissolved ozone monitoring locations placed at 2, 6, and 10 minutes in the 10 minute HRT contactor.

[5]Log inactivation credit calculated based on three monitoring stations, baffling factor of 0.65, and 1 °C

The analysis demonstrated that a contact time of 10 minutes or use of a quench chemical allows increased operational flexibility and the use of higher initial dissolved ozone residual values. To ensure sufficient space for settled water ozone, a hydraulic retention time of 10 minutes should be used during conceptual design and prior to detailed design, the decay rate at the coldest water temperature should be determined.

The ozone contactor recommended hydraulic retention time and need for a quench chemical should be determined during detailed design. Additional bench-scale testing information will be available and process goals finalized with the City staff at that time.

3.7.2 Design Ozone Dosage

The design ozone dosage was projected based on the bench-scale testing results, the water quality at the time of testing, and historical water quality data. The approach was consistent for both raw and settled water. An ozone dosage was calculated for each day based on the TOC concentration using the following equation:

$$Ozone\ Dosage\ = D + (TOC_n - TOC_t) \times 0.5$$

Where:

Ozone Dosage = Transferred Ozone Dosage, mg/L

D = Design dosage, mg/L (1.0 mg/L for raw and settled water)

TOC_n = TOC for day *n*, mg/L

TOC_t = TOC for the bench-scale test, mg/L (2.0 mg/L raw and 1.4 mg/L settled)

0.5 = Ozone to TOC ratio, mg ozone per mg TOC (assumed)[2]

For example, based on the 90th percentile TOC value in the settled water of 2.8 mg/L, the resultant ozone dosage would be as follows:

$$1.0\ mg/L + (2.8\ mg/L - \ 1.4\ mg/L) \times 0.5\ = Ozone\ Dosage\ = 1.7\ mg/L$$

The ozone dosages were therefore compiled and a probability plot developed as illustrated in Figure 3-7. The minimum allowable dosage for either raw or settled water condition was 1.0 mg/L.

The ozone dosage selected for the basis of design was the 90th percentile value, a design flow rate of 160 mgd and an assumed ozone transfer efficiency of 95 percent. The resultant generator size, turndown required, etc. is summarized in Table 3-4.

[2] The approached used an ozone to TOC ratio of 0.5 mg/mg. Typically, this ratio can vary between 0.3 and 0.5 depending on the nature of the TOC present. Because bench-scale testing was performed on water with TOC in the lower 5th percentile of the historical values, a more conservative ratio was used, 0.5 mg/mg.

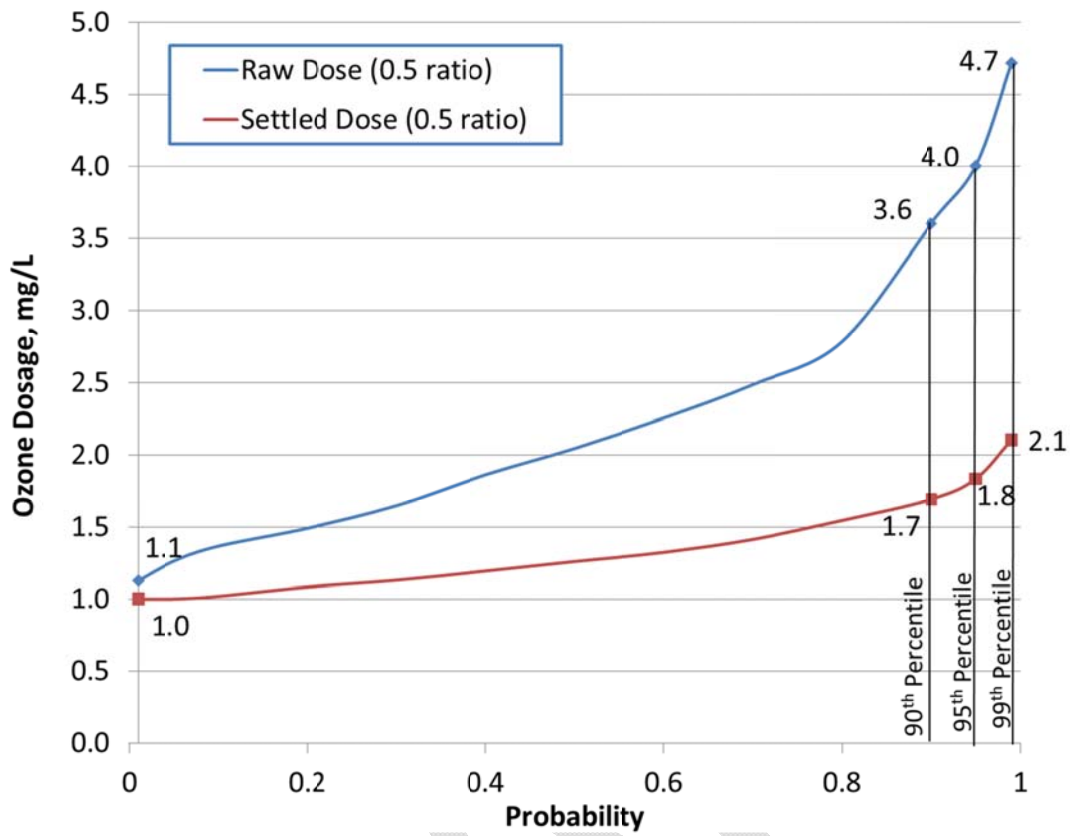


Figure 3-7 Ozone Dosage Probability Plot (January 2011 through June 2012)

Table 3-4 Ozone System Conceptual Design Information

DESIGN VALUES	RAW	SETTLED
Design Transferred Dosage, mg/L	3.6	1.7
Design Applied Dosage, mg/L	3.8	1.8
Design Water Flow Rate, mgd	160	160
Firm Ozone Generating Capacity, ppd	5,000	2,400
Average Ozone Production, ppd	1,280	800
Minimum Ozone Production, ppd	580	530
Generator Capacity, ppd	2,500	1,200
Number Generators	3	3
Turndown for One Ozone Generator	4:1	3:1
Contactor HRT, min	10	10

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4.0 Ozone Alternatives

4.1 OVERVIEW

Typical WTP ozone utilization is either a raw water ozonation application or a post-sedimentation application. For the Collins Park WTP, both application points can be considered. However, post-sedimentation application would require either that additional hydraulic head be made available in between the Recarbonation Basins and the Filters to allow flow through new ozone contactors by gravity or that settled water pumping be implemented to raise the hydraulic grade line of the WTP prior to the ozone contactors. In this section of the report, the Collins Park WTP hydraulics are evaluated to determine the requirements associated with fitting new settled water ozone contact facilities in the existing treatment train and with this information, ozone contactor location alternatives are evaluated.

4.2 PLANT HYDRAULICS

The WTP currently has six (6) sedimentation treatment trains, four (4) on the west part of the site (i.e. "West Plant") and two (2) on the east part of the site (i.e., "East Plant"). The two additional trains to be added under this project will be constructed on the East Plant. Initial East Plant hydraulic profile calculations were performed by ARCADIS and provided to Black & Veatch in December 2014. These calculations were reviewed versus the WTP drawings, modified where applicable, and checked against the Collins Park Water Treatment Plant Hydraulic Testing performed on November 9th, 1982, for validation. Based on these calculations, an updated hydraulic profile model was developed for the East and the West plants from the raw water channel within the Chemical Building to their respective filters. While this model has not been calibrated versus recent hydraulic testing at the WTP, use of this model should result in enough accuracy for conceptual design purposes since verification was performed versus testing performed in 1982.

Listed below are the main observations from the initial hydraulic analysis at 40 mgd per basin pair, which will continue to be the design peak demand for the WTP:

i. Under current operating conditions, with a filter operating level of 599.50 (as reported in the original ARCADIS model), water levels at the raw water influent channel and at the sedimentation basins are anticipated to be approximately as follows:

   West Plant:
   - Raw Water Channel: 600.75
   - Sedimentation Basins: 599.75

   East Plant:
   - Raw Water Channel: 601.25
   - Sedimentation Basins: 599.75

   The bottom elevation of the decks in the raw water channels and the sedimentation basins are 602.70 and 601.33, respectively. Thus, based on the above predicted water elevations and assuming a minimum of 6 inches of freeboard, it appears there is a little over a foot of available vertical space in the facilities upstream of the sedimentation basins that could be used for elevating the operating water level to provide additional hydraulic head for potential settled water ozone contact facilities (i.e., between the recarbonation basins and the filters). If a freeboard higher than 6 inches is desired, then the available room for raising the operating water level upstream of the basins would need to be reduced accordingly.

ii. Preliminary analysis of the WTP hydraulic models for the Inlet to the Filters and from the Filters to the Clearwells revealed that with a clearwell operating level of 590.0 (as reported in the original ARCADIS model), there would seem to only be a few feet (between 2 and 3 feet) of head available for filter operations between filter backwashes at both the East and the West plants. Thus, it appears that lowering the filter water operating level without a corresponding reduction in clearwell operating level and capacity would not be possible, as such a reduction could affect available filter head. Based on discussions with ARCADIS, it appears a reduction of clearwell operating level by about 0.5 feet is possible to maintain the same available headloss across the filters.

iii. It appears that by adjusting up the operating water level of the facilities upstream of and including the recarbonation basins and reducing the operating level of the facilities downstream of and including the filters, an additional hydraulic head of up to between 1 and 1.5 feet could be made available for building settled water ozone contactors without the need for settled water pumping.

4.3 PRELIMINARY ALTERNATIVE EVALUATION

For the purposes of this study, four (4) ozone contactor location alternatives were considered—Raw Water Pipeline Ozonation, Raw Water Ozone Contact Basins, Settled Water Ozone Contact Basins, and Settled Water Flume Ozonation —as shown schematically in the attached Figures 4-1 through 4-3. A summary of pros and cons of the four alternatives can be found at the end of this section in Table 4-1.

4.3.1 Raw Water Ozonation Alternatives

As seen in Figure 4-1, raw water ozonation treats water prior to entering the plant. In both the Raw Water Pipeline Ozonation and the Raw Water Ozone Contact Basin alternative, ozone generation facilities will be required to be constructed. However, the mechanism for ozone injection differs in the two alternatives. The Raw Water Pipeline Ozonation alternative would use the existing raw water pipelines for ozone contact and include insertion of an ozone injection nozzle into each of the existing pipelines. The Raw Water Contact Basins alternative includes the construction of a separate contact basin facility at the WTP site.

The main negative aspect of raw water ozonation is that it typically results in relatively high ozone demands due to increased TOC ahead of settling. Higher ozone demands translate into relatively higher operating costs and in higher potential for AOC formation as an ozonation byproduct. Higher AOC levels in the ozonated water, will in turn, require a higher level of treatment in the filters through biologically active filtration (BAF).

In the case of the Collins Park WTP, it appears that pursuing Raw Water Pipeline Ozonation would be challenging, as the 60-inch and the 78-inch existing raw water pipes would require significant reconditioning. Per the available as-built WTP drawings, the above pipelines appear to be pre-stressed embedded concrete cylinder pipe and steel pipe lined with a bituminous liner, respectively. In the case of the existing 60-inch raw water line, the pre-stressed embedded concrete cylinder pipe is aged and is constructed with a relatively thin concrete lining. The high ozone concentration resulting from the inline ozone addition may penetrate the thin concrete lining through cracks, attack the steel wire, and weaken the pipe. Therefore, the 60-inch raw water piping would require replacement. For the 78-inch raw water line, the existing bituminous lined steel pipe would require either that the bituminous lining be replaced—as it is not compatible with ozone—and the pipe be relined with a material resistant to corrosion due to contact with ozone or that the

78-inch raw water line be replaced altogether. It is estimated that replacing both the 60-inch and 78-inch raw water pipelines for a distance long enough to provide the 10-minute ozone contact time used for the purposes of this study would be on the order of $10 million, excluding ancillary facilities for off-gas collection and treatment. Further, inline injection of ozone into the raw water pipelines would likely generate some off-gassing of ozone for a certain distance downstream from the injection point, which will collect at high points in the pipeline. Safe evacuation of collected gas would need to be accomplished using air relief valves or a small concrete basin that acts as an off-gas collection chamber. Ozone destruct units at gas collection points near the injection points would be needed.

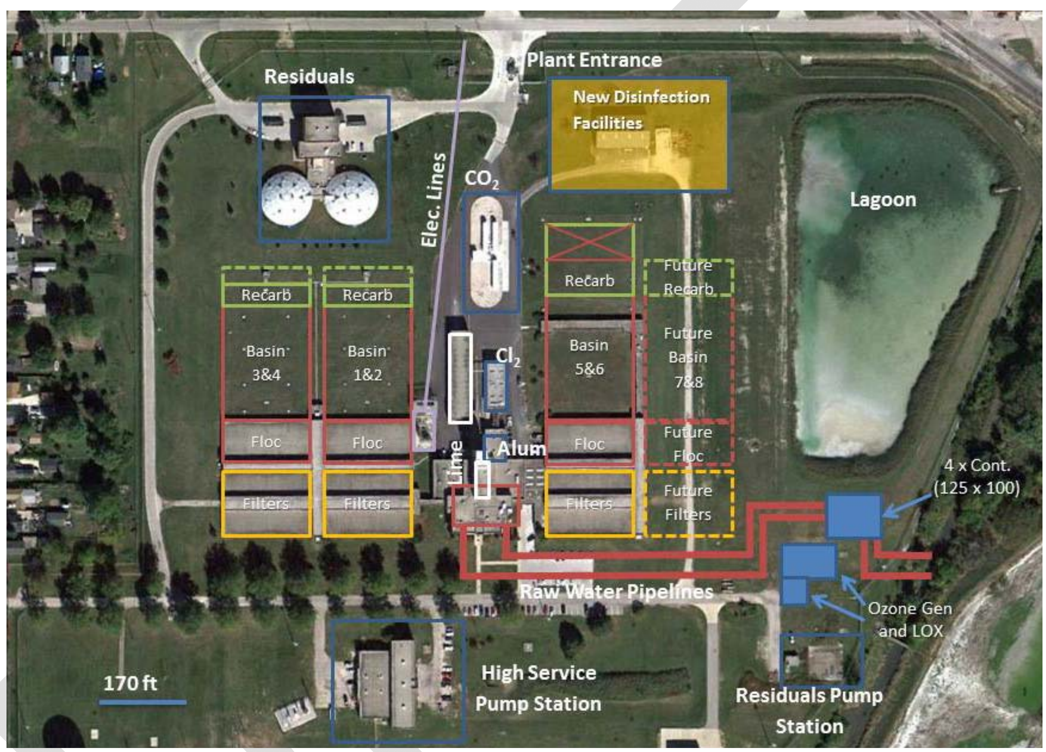


Figure 4-1 Raw Water Ozone Contact Basins Alternative

* The Raw Water Pipeline Ozonation Alternative is not shown. This option would not require the four ozone contactors shown in the above figure and would likely include the ozone generation facilities off-site by the Raw Water Intake.

The Raw Water Contact Basins alternative has advantages of not imposing hydraulic constraints on the existing facility (although a relatively small additional hydraulic head would be imposed on the raw water intake pumps) and of being able to be constructed almost entirely without plant operation interruptions. However, in addition to the negative effects of increased TOC ahead of settling, periodic high turbidity spikes, which have been reported to exist in the raw water, could cause maintenance issues with raw water contact basins due to premature settling in these basins. Also, at the raw water pump station, potassium permanganate ($KMnO_4$) is added to control zebra mussels and PAC is added for microcystin adsorption and control of taste- and odor causing compounds. While it is likely that continuous PAC feed would be discontinued when ozone is installed, $KMnO_4$ addition would continue in order to control mussel infestation in the raw water intake. The byproduct of KMnO4 addition, manganese dioxide (MnO_2) is oxidized by ozone, reforming $KMnO_4$. These compounds would complicate raw water ozone addition.

Per the above discussion, while the use of raw water ozonation would have the advantage of avoiding the existing WTP hydraulic constraints, initial review suggests that post-sedimentation would be a more suitable, long-term solution for the Collins Park WTP; especially, as bench-scale testing discussed above confirms the feasibility of post-sedimentation ozonation. Therefore, it is Black & Veatch's recommendation that the two raw water options noted herein be eliminated from further consideration.

4.3.2 Post-sedimentation Ozonation Alternatives

Figures 4-2 and 4-3 illustrate an aerial view of the two, alternative post-sedimentation ozonation considered. These alternatives are: the Settled Water Ozone Contact Basins and the Settled Water Flume Ozonation.

Both alternatives have the advantage of relatively lower ozone demands when compared to raw water ozonation. Lower ozone demand will translate into lower ozone generation costs. However, post-sedimentation ozonation requires that either modifications be made to the existing WTP hydraulics to facilitate the inclusion of contact basins or that settled water pumping be implemented.

Based on previous experience, new Settled Water Ozone Contact Basins constructed as separate facilities external to the existing Recarbonation Basins generally require an available hydraulic head of 1.5 to 3 feet, depending on the configuration of influent and effluent piping and baffling. Giving the relatively tight hydraulic constraints of the existing plant; that is, less than 1.5 feet of available head, as described in Section 4, the Settled Water Ozone Contact Basin Alternative would require a pumping station to convey flows from the Recarbonation Basins to the new ozone contact basins. The pumping station adds capital and operating cost to this alternative.

The Settled Water Flume Ozonation Alternative, as illustrated in Figure 4-3 has lesser hydraulic head requirement compared to the Settled Water Ozonation Contact Basins alternative because there is a more direct and simple path for water to flow through the ozone injection and contact facilities. Also, the existing settled water flumes located along the sedimentation basins would be used for partial additional ozone contact time. Based on initial hydraulic assessments noted above and with the inclusion of the additional ozone injection channels noted in the illustration, it appears the Settled Water Flume Ozonation Alternative will require approximately 0.25 to 0.5 feet of hydraulic head depending on final layout and dimensions. This required hydraulic head appears to be compatible with the current hydraulic capabilities of the WTP without the need for settled water pumping, as described above.

Per Section 3.7, Process Development Recommendations, an ozone contact time of 10 minutes was selected for this study to ensure sufficient space for settled water ozone. For the Settled Water Ozonation Contact Basins alternative, the contact basins would be sized to provide this contact time. For the Settled Water Flume Ozonation Alternative, since the existing Settled Water Flumes can provide a contact time of about 6 minutes at both the West and East plants, the new ozone injection chambers will be sized for a contact time of 4 minutes for the purposes of the conceptual design. The ozone contact flume section will capture ozone off-gas and be exposed to the highest dissolved ozone concentrations. The existing settled water flumes have been in service for a relatively long time and the concrete cover over the steel rebars was not designed with ozonation in mind. Therefore, a design that allows for most of the ozone off-gas to be captured in the new flumes and initial ozone residual to decay in the new flumes to a level such that the existing flumes are exposed to relatively low ozone concentrations would be ideal. If Settled Water Flume Ozonation Alternative is selected as the preferred alternative to control toxins, then means to monitor and control the effect of remaining ozone residuals in the existing settled water flumes would need to be considered (periodical inspection of the existing flumes, coatings for the gas space in the flume, dissolved ozone sampling station location, etc.).

TABLE 4-1: ALTERNATIVES ADVANTAGES AND DISADVANTAGES	ALTERNATIVES			
	Raw Water Pipeline Ozonation	Raw Water Ozone Contact Basins	Settled Water Ozone Contact Basins	Settled Water Flume Ozonation
Ease of Layout	⊙	⊙	⊕	⊕
Hydraulic Limitations	⊙	⊙	⊕	⊙
Ozone Demand	⊗	⊗	⊙	⊙
Bromate Formation	⊙	⊙	⊕	⊕
Assimilable Organic Compound Formation	⊕	⊕	⊕	⊕
Construction Interference with Existing Facility	⊙	⊙	⊕	⊕
Operational Interference with Existing Facility	⊕	⊗	⊙	⊙
Estimated Construction Cost	⊗	⊗	⊕	⊙
Estimated Operational Costs	⊗	⊗	⊕	⊙

Key:

⊙ **Positive / Plus**

⊕ **Neutral**

⊗ **Negative / Con**

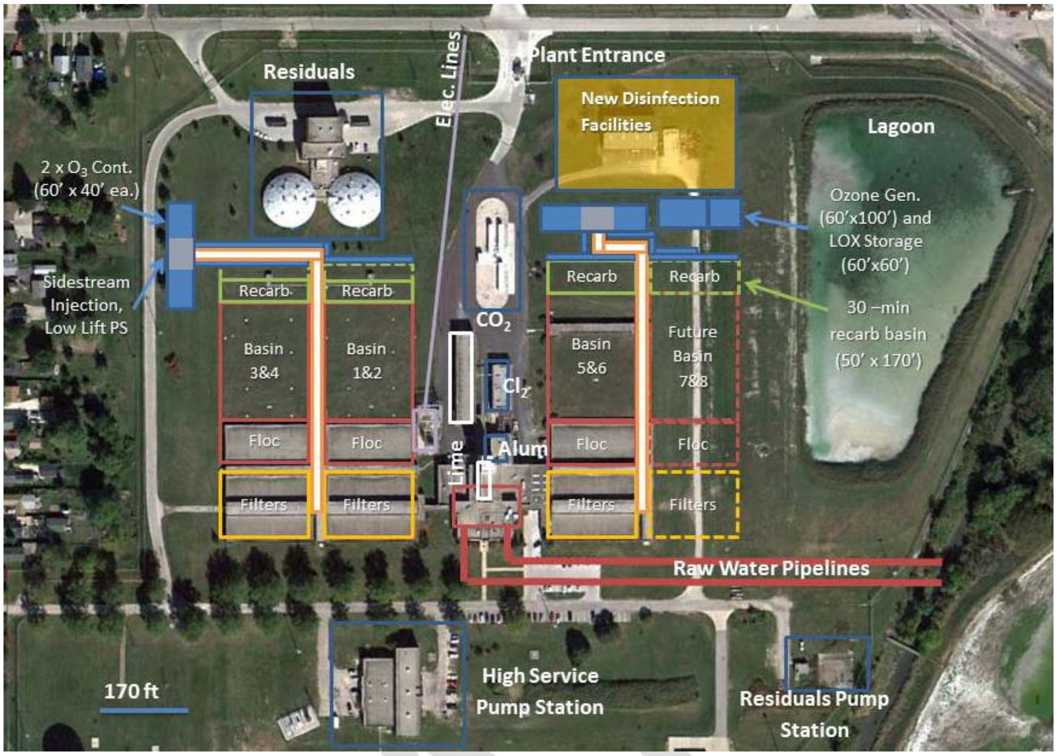


Figure 4-2 Settled Water Ozone Contact Basins Alternative

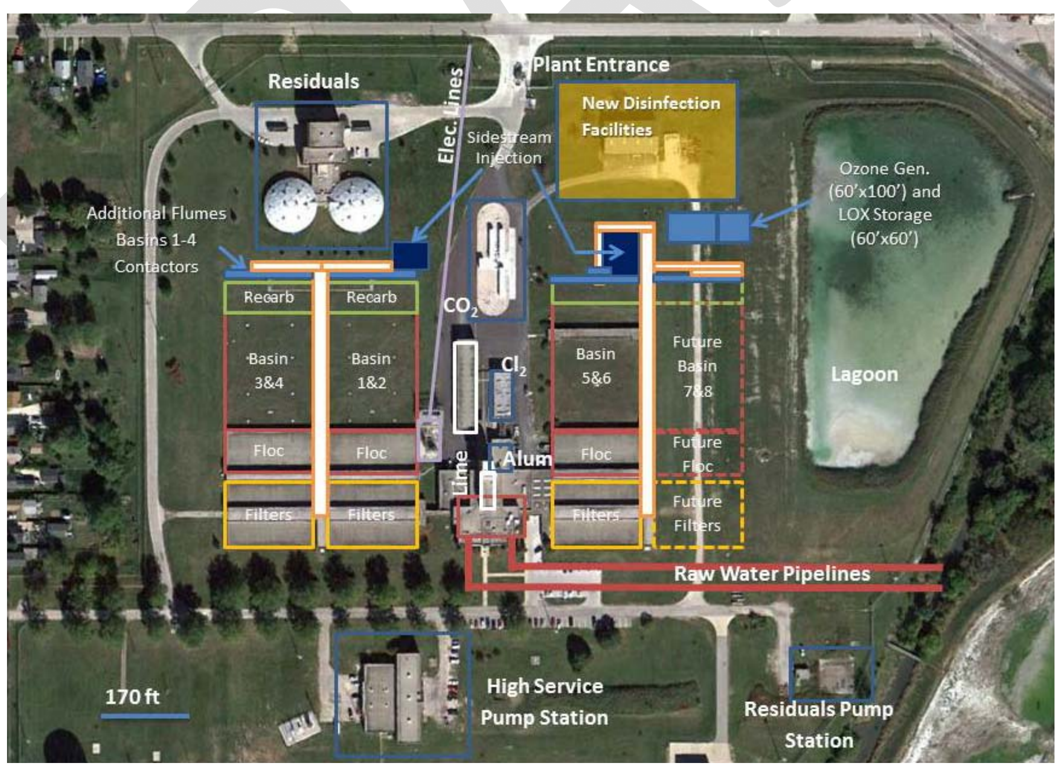


Figure 4-3 Settled Water Flume Ozonation Alternative

4.4 DETAILED ALTERNATIVE EVALUATION

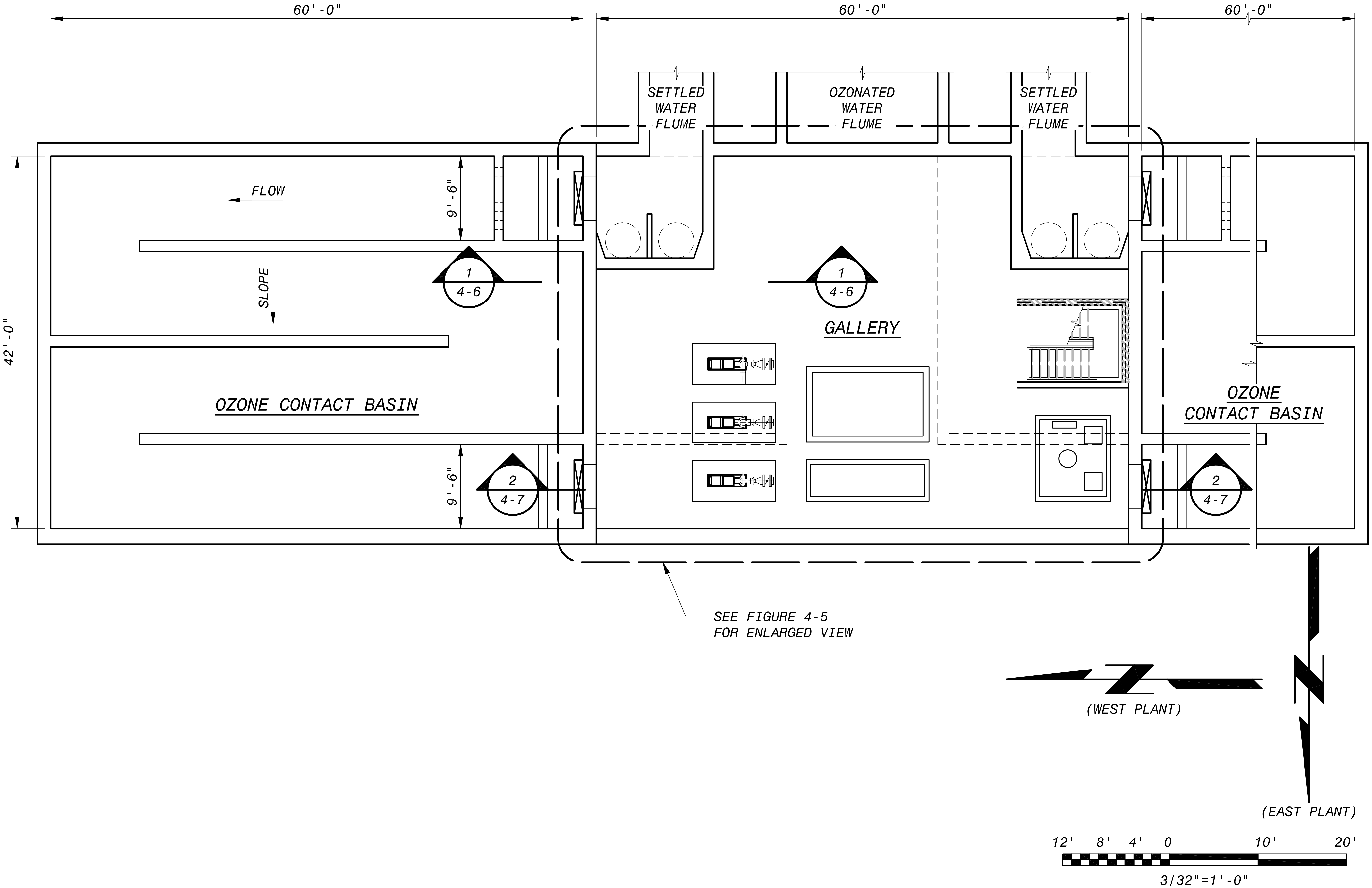
4.4.1 Settled Water Ozone Contact Basins

Settled Water Ozone Contact Basins would be constructed externally to the Sedimentation and Recarbonation Basin structures. Settled water and ozonated water would be conveyed into and out of these structures through flumes, as shown in Figure 4-2. Separate ozone contact facilities are proposed for the West and the East plants to minimize conveyance distance from and to the existing basin structures. The ozone contact facilities on each side of the WTP would consist of two (2) 40-mgd ozone contact basins with common equipment and pipe gallery for housing ozone injection and destruct equipment, piping, ozone sampling stations, and associated instrumentation, and the likely needed low-lift pump station. Smaller 20-mgd ozone contact trains that match the capacity of each basin train are not recommended, as such arrangement would be less economical and unnecessary for plant operation. If the ozone feed is shut-down, settled water would be routed through the ozone contact basins. If these basins need to be shut-down and drained for service, provisions would be made to allow bypassing of these basins to direct settled water to the filters in the same manner as the plant is presently operated. The ozone injection system will be provided with the turndown capability to allow treatment of flowrates equal to a single basin train or less so that, when a 20-mgd basin train is taken out of service, the contiguous train can still be operated with its corresponding ozone contact basin. Therefore, the use of a single 40-mgd ozone contactor that serve two sedimentation and recarbonation basins should not affect plant operational flexibility.

As part of the Redundant Capacity Improvements Project, the Recarbonation Basins will be modified so that the footprint of all trains—including the proposed Basin 7 and 8 trains—are similar and result in a recarbonation contact time of 20 minutes. As shown in Figure 4-2, after the Recarbonation Basins of the West Plant are modified by extending them to the north to obtain additional contact time, there will be little room left for additional structures to the north of the basins due to the proximity of the existing Sludge Thickeners. Thus, the new West Settled Water Ozone Contact Basins would need to be constructed on the available area northwest of Basin 4, as shown in Figure 4-2. At the East Plant, the existing Recarbonation Basins have a contact time in excess of the required 20 minutes, so the proposed plan is to reduce them in size to make room to the north for the new ozone contact facilities. The new Basins 7 and 8 would match the footprint of the existing Basins 5 and 6. Thus, as shown in Figure 4-2, the new East Settled Water Ozone Contact Basins could be constructed to the north of Basins 5 and 6, leaving room to the north of the new Basins 7 and 8 potentially for the ozone generation facilities.

The proposed East and West Settled Water Ozone Contact Basin facilities would be underground reinforced concrete facilities, as shown conceptually in Figures 4-4, 4-5, 4-6, and 4-7. Each facility would consist of two 40-mgd ozone contact basins, each of which would be a baffled concrete structure with a horizontal serpentine flow pattern. This flow pattern is typical for a sidestream ozone injection system, which is recommended for these facilities. The sidestream ozone injection system is discussed in more detail in Section 5 of this report. An ozone solution would be injected at the inlet chamber of these basins and provisions for performing ozone residual quenching could be made at the basin outlet.

The hydraulics associated with including new, external, post-settling ozone contact basins in the existing treatment train were evaluated at a conceptual level. A WTP hydraulic profile that includes the new ozone contact basins is provided in Figure 4-8. It is anticipated that the proposed ozone contact facilities will require between 1.5 feet to 2.0 feet of additional hydraulic head. Given that the current additional hydraulic head availability is approximately 1 foot, it is proposed that a low-lift settled water pumping station be provided to boost the plant hydraulic gradient prior to the new ozone contact basins. As shown in Figures 4-5 and 4-6, provisions have been made in this conceptual design to include a low-lift pumping station in the central gallery of the new ozone contact facilities. It is anticipated that settled water pumping will only be required at flows near the maximum day demand. If during periods of lower demand the ozone contact basin trains are operated at less than its design capacity (i.e., 20 mgd), then pumping will likely be unnecessary as flow by gravity through the contact basins would become possible. This scenario has been taken into consideration for developing system operating costs in Section 6 of this report. If a HRT less than 10 minutes is determined to be appropriate during design and if baffling can be relaxed or optimized depending on the disinfection goals of the ozone facilities, then there is a possibility that contact basins that do not require settled water pumping could be designed. As part of such analysis, refinement of the current WTP hydraulic model, including validation and calibration of the model, would also be required. For the purposes of this conceptual study, a settled water pumping station will be included so that the space and cost requirements of a pumping station are evaluated and included.



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TOLEDO WTP REDUNDANT CAPACITY & HAB IMPROVEMENTS

SETTLED WATER OZONE CONTACT BASINS OZONE CONTACTOR - OVERALL PLAN

CITY OF TOLEDO

DATE: 02/05/15

B&V PROJECT 186505

FIGURE 4-4

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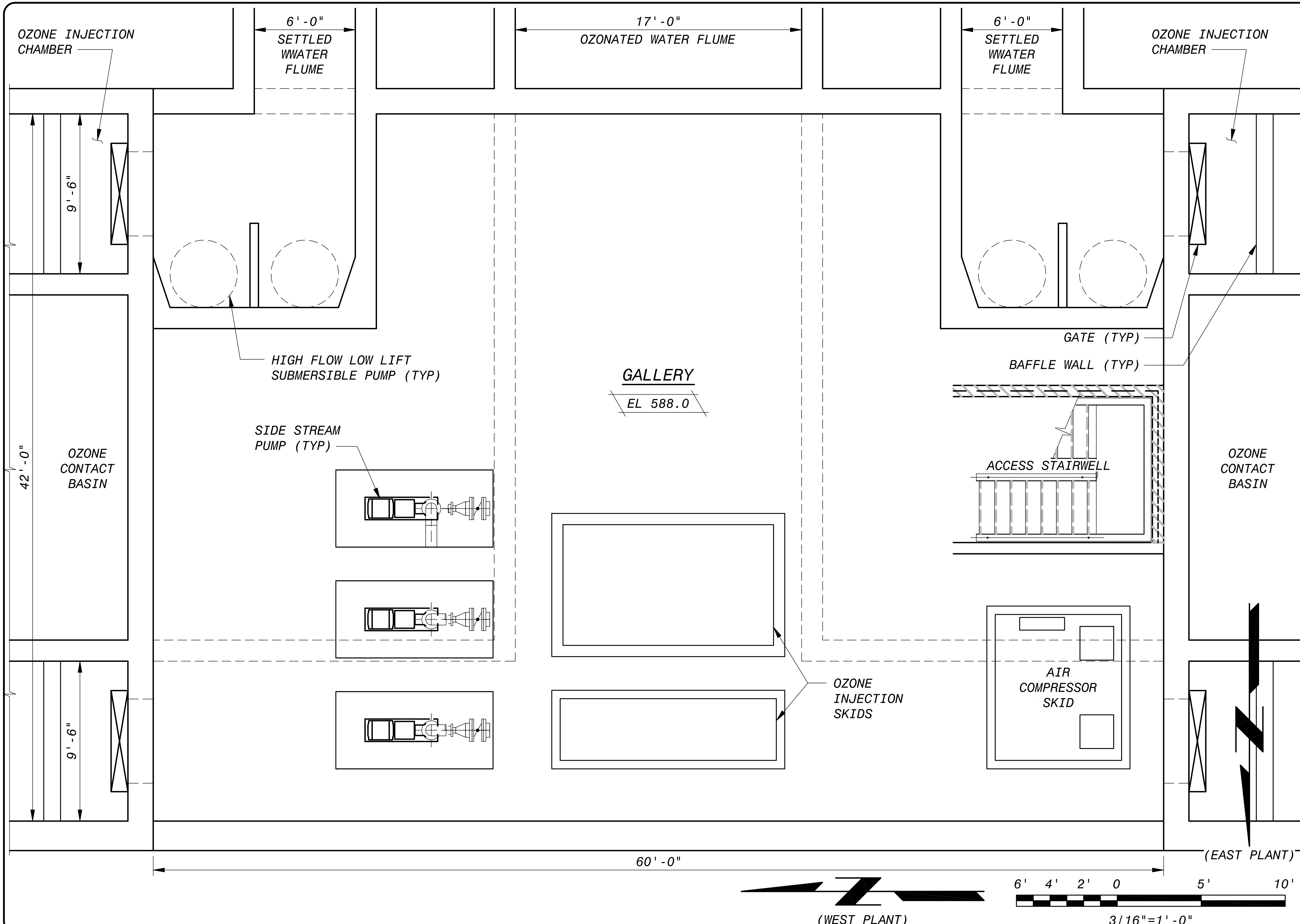

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THE CITY OF TOLEDO LABORARE EST ORARE SEAL OF THE JANUARY 7, 1837

TOLEDO WTP REDUNDANT CAPACITY & HAB IMPROVEMENTS

SETTLED WATER OZONE CONTACT BASINS OZONE CONTACTOR - GALLERY PLAN

CITY OF TOLEDO

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FIGURE 4-5

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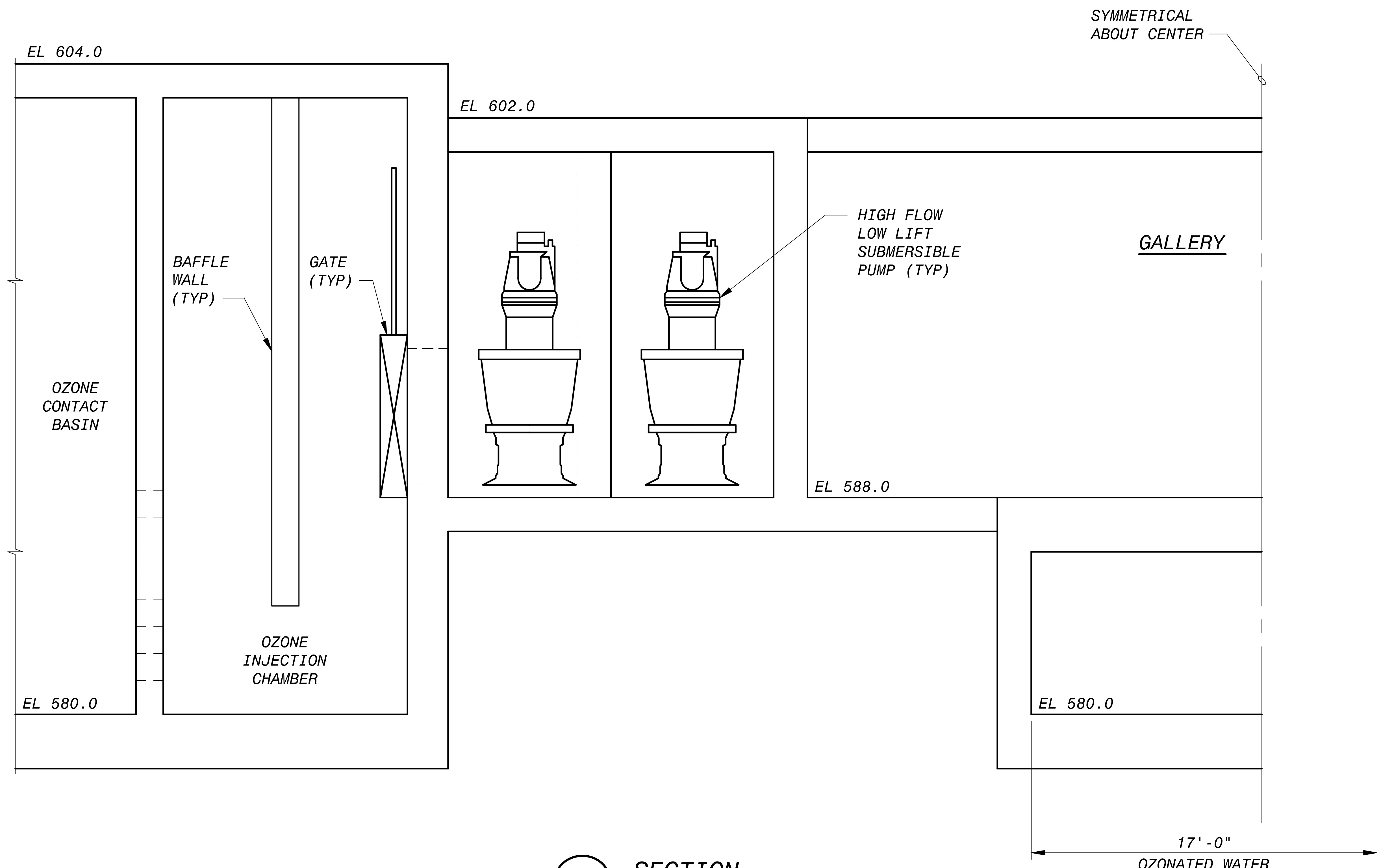

1 SECTION
4-4 1/4" = 1'-0"

4' 2' 0 4' 8'
1/4"=1'-0"

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TOLEDO WTP REDUNDANT
CAPACITY & HAB IMPROVEMENTS

SETTLED WATER
OZONE CONTACT BASINS
OZONE CONTACTOR - SECTION

CITY OF TOLEDO

DATE: 02/05/15

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FIGURE
4-6

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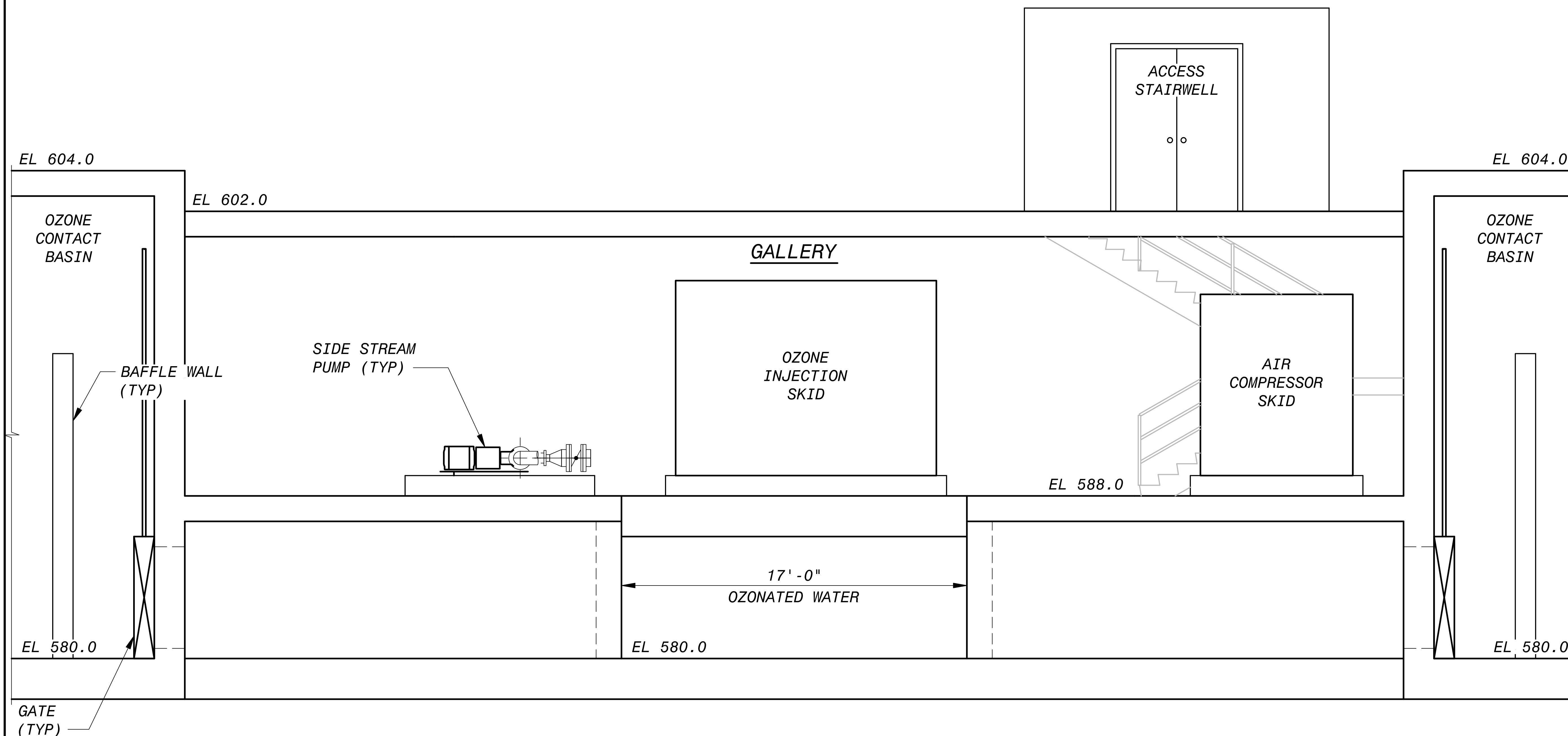

2 SECTION
4-4 3/16" = 1'-0"

6' 4' 2' 0 5' 10'
3/16"=1'-0"

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TOLEDO WTP REDUNDANT CAPACITY & HAB IMPROVEMENTS

SETTLED WATER OZONE CONTACT BASINS OZONE CONTACTOR - SECTION

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FIGURE 4-7

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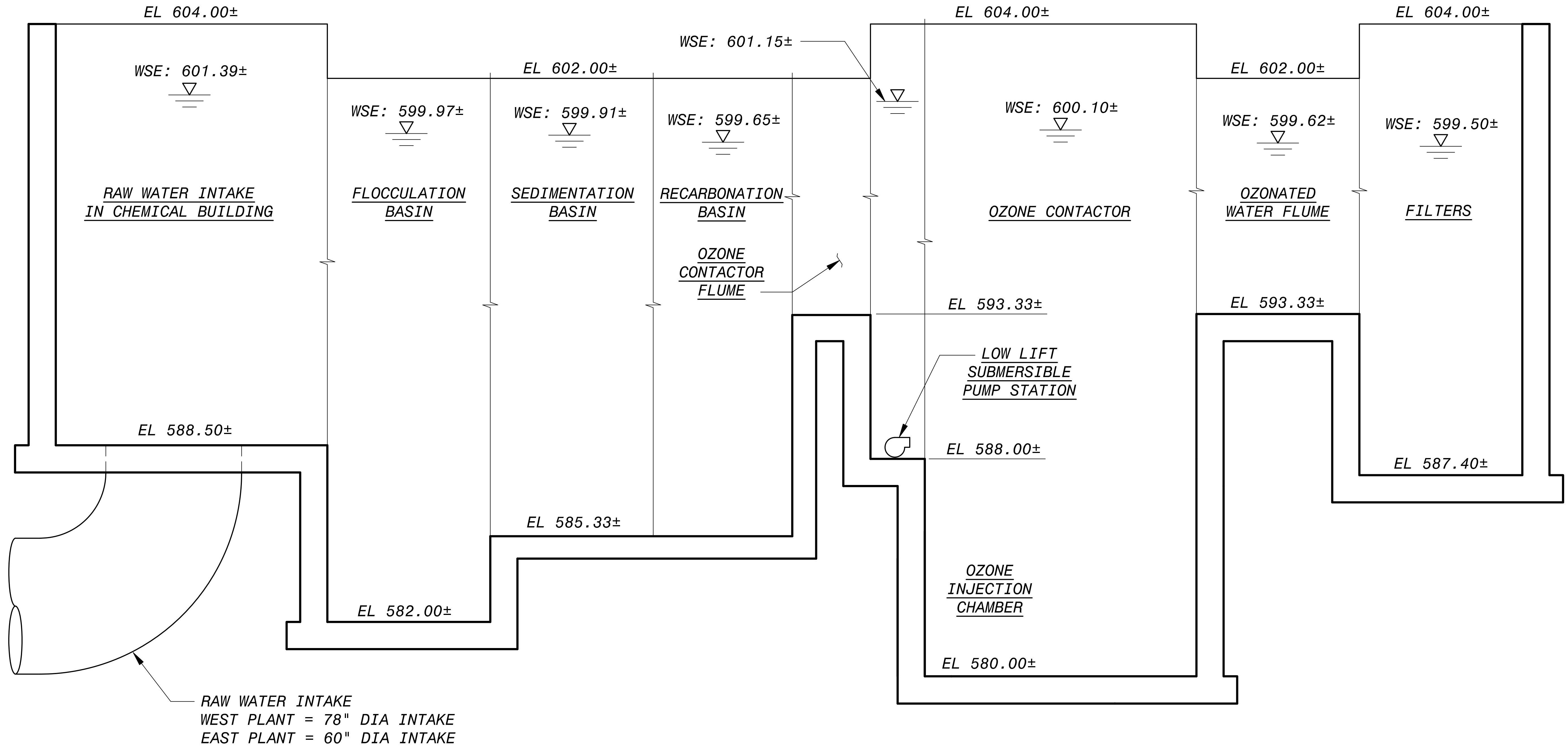

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TOLEDO WTP REDUNDANT CAPACITY & HAB IMPROVEMENTS

SETTLED WATER OZONE CONTACT BASINS HYDRAULIC PROFILE

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FIGURE 4-8

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Settled Water Flume Ozonation

Settled Water Flume Ozonation would include new ozone contact flumes installed directly downstream of the Recarbonation Basin outlets for ozone injection and initial contact and would make partial use of the existing settled water flumes for the remaining needed contact time. The same train sizing and arrangement considerations discussed above for the Settled Water Ozone Contact Basins would apply to sizing of flume ozonation. Separate sidestream injection facilities are proposed for the West and the East plants and the ozone contact facilities in each side of the WTP would consist of two (2) 40-mgd ozone contact flume trains with a common equipment and pipe gallery for housing ozone injection and destruct equipment and piping, and ozone sampling stations and associated instrumentation.

As shown in Figure 4-3, after the Recarbonation Basins of the West Plant are modified by extending them to the north to obtain additional contact time, there will be little room left for additional structures to the north of the basins due to the proximity of the existing Sludge Thickeners. Thus, the new West Settled Flume Ozone Contact Facilities would need to be laid out in a manner that makes efficient use of the available space. Figures 4-9 and 4-10 show a proposed conceptual arrangement of these facilities. It has been proposed that the existing Recarbonation Basins at the East Plant be reduced in size, which would make room to the north for the new flume ozone contact facilities. As shown in Figures 4-11 and 4-12, the new Basins 7 & 8 could be constructed in a similar manner. The new East Settled Water Flume Ozone Contact facilities could be constructed to the north of Basins 5, 6, 7, and 8.

The proposed East and West Settled Water Flume Ozone Contact Facilities would be underground reinforced concrete flumes, as shown conceptually in Figures 4-9 through 4-12. Each facility would consist of two new 40-mgd ozone contact flumes, each of which would consist of an injection chamber with down-and-up flow under a baffle followed by a relatively long-and-narrow concrete channel that would discharge into the existing Settled Water Flumes. An ozone solution would be injected into the ozone injection chamber from a sidestream-type ozone injection system, which is conceptually discussed in more detail in Section 5 of this report. The new flumes have been conceptually sized to match the same depth as that of the existing basins and to provide an ozone contact time (HRT) of 4 minutes. The 4 minutes of HRT added to the existing flume's capacity of a 6 minute HRT allows for a total HRT of 10 minutes. The ozone injection chambers shown in Figures 4-10 and 4-12 have been conceptually designed with dimensions and a flow pattern that will prevent ozone bubbles dispersed into the main stream from going back towards the Recarbonation Basins and that will allow collection of the off-gas at the high point of the flume by the underflow baffle provided in this chamber.

The hydraulics associated with the ozone contact flumes were evaluated at a conceptual level. A WTP hydraulic profile that includes the new ozone contact flumes is provided in Figure 4-13. It is anticipated that the proposed new ozone contact flumes will require approximately between 0.25 feet and 0.5 feet of additional hydraulic head. Given that the current additional hydraulic head availability of the existing WTP is likely about 1 foot, this option should be able to be implemented by gravity without the need for settled water pumping. If a reduced HRT as compared to the 10 minutes used for this study is determined to be appropriate during design and the condition of the concrete in the existing Settled Water Flumes is verified through inspection as being adequate, there is a possibility that the dimensions of the new ozone contact flumes could be reduced. Nevertheless, for the purposes of this conceptual study, the dimensions shown in this report have been assumed so that conservative spatial and cost requirements are evaluated and included.

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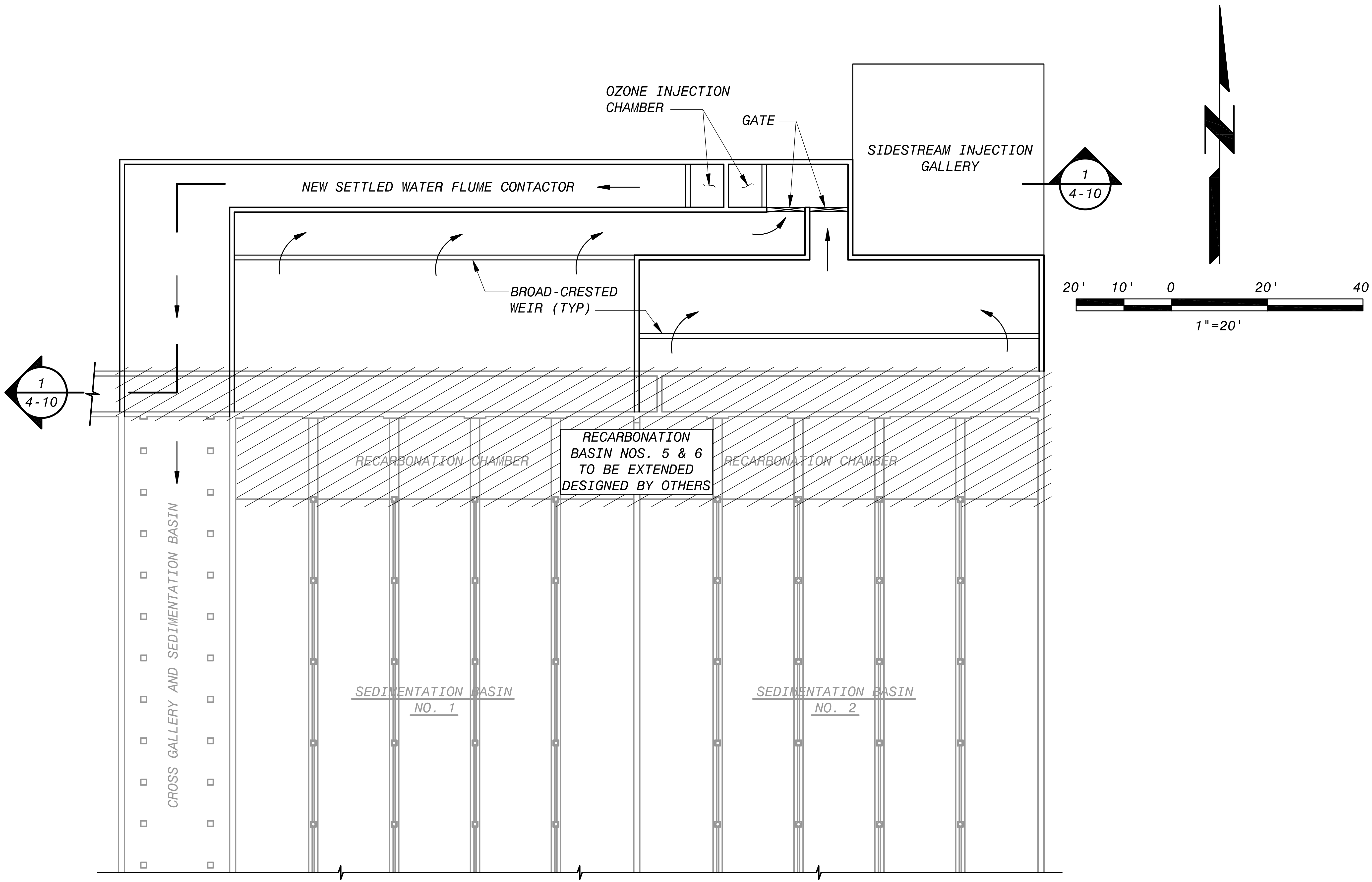

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SETTLED WATER FLUME OZONATION BASINS NOS. 1 & 2 PLAN

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FIGURE 4-9

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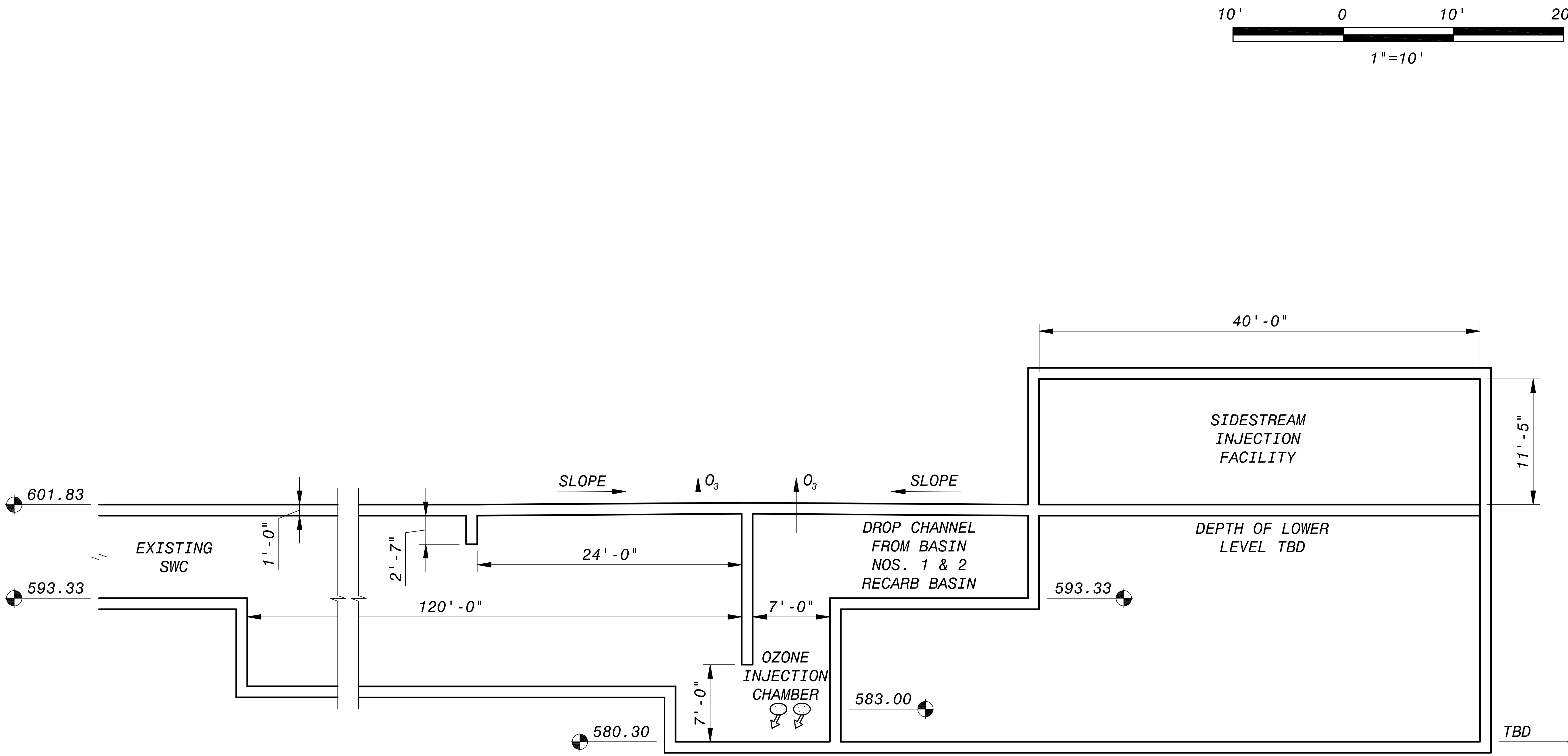

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TOLEDO WTP REDUNDANT
CAPACITY & HAB IMPROVEMENTS

SETTLED WATER FLUME OZONATION
BASIN NOS. 1 & 2
SECTIONS

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FIGURE
4-10

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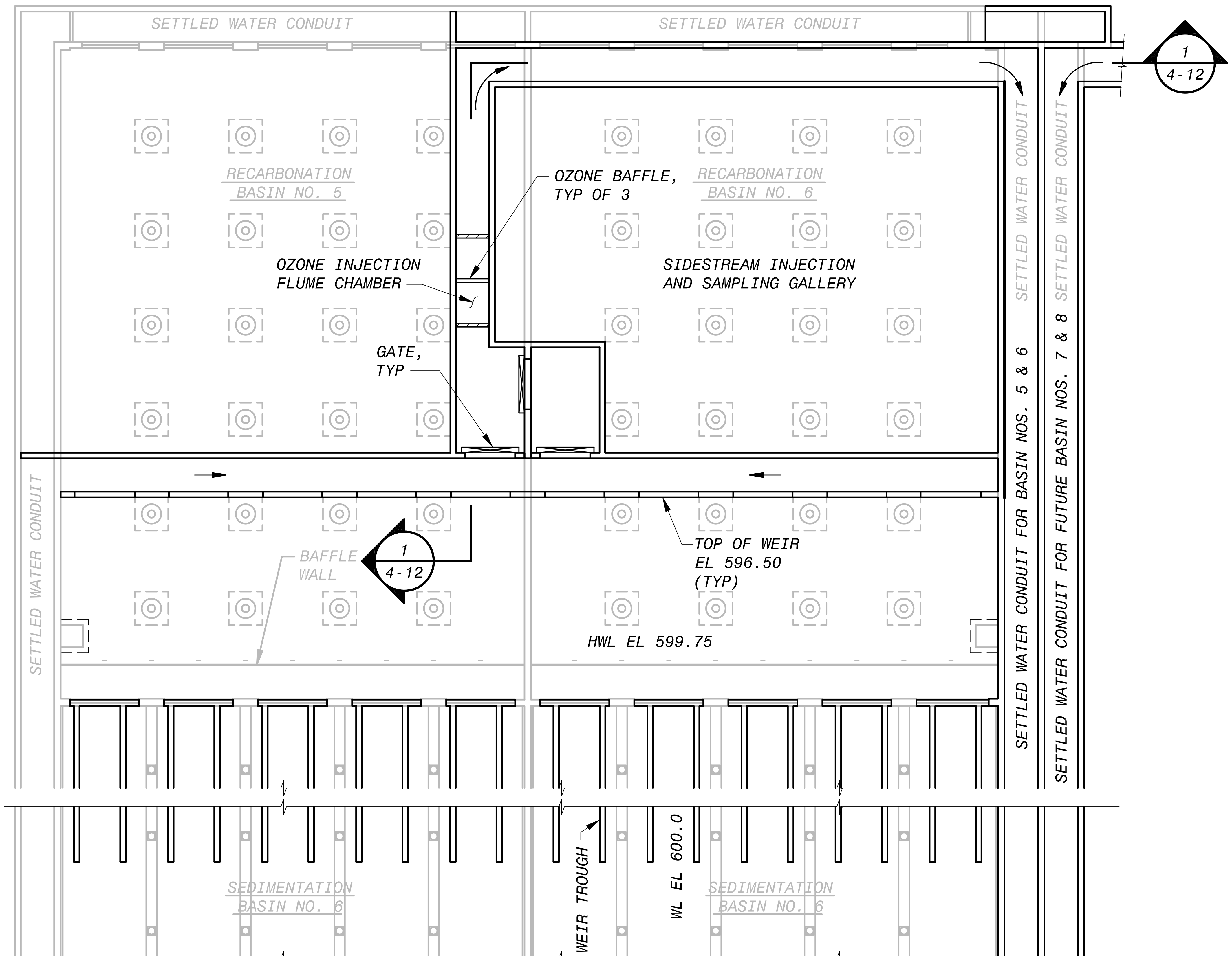

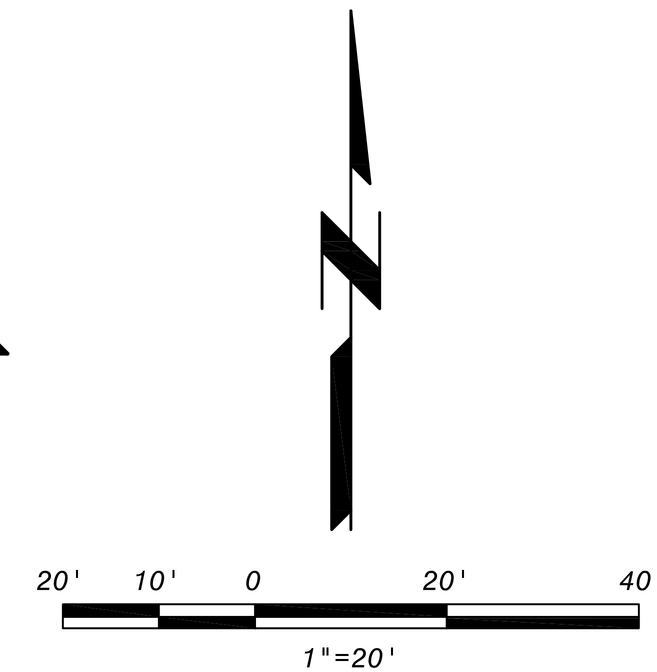

NOTE:
1. THE BASINS 7 & 8 LAYOUT WILL INCLUDE A SERPENTINE OZONE CONTACT FLUME TO MAXIMIZE SPACE ALLOCATIONS.

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SETTLED WATER FLUME OZONATION BASIN NOS. 5 & 6 PLAN

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FIGURE 4-11

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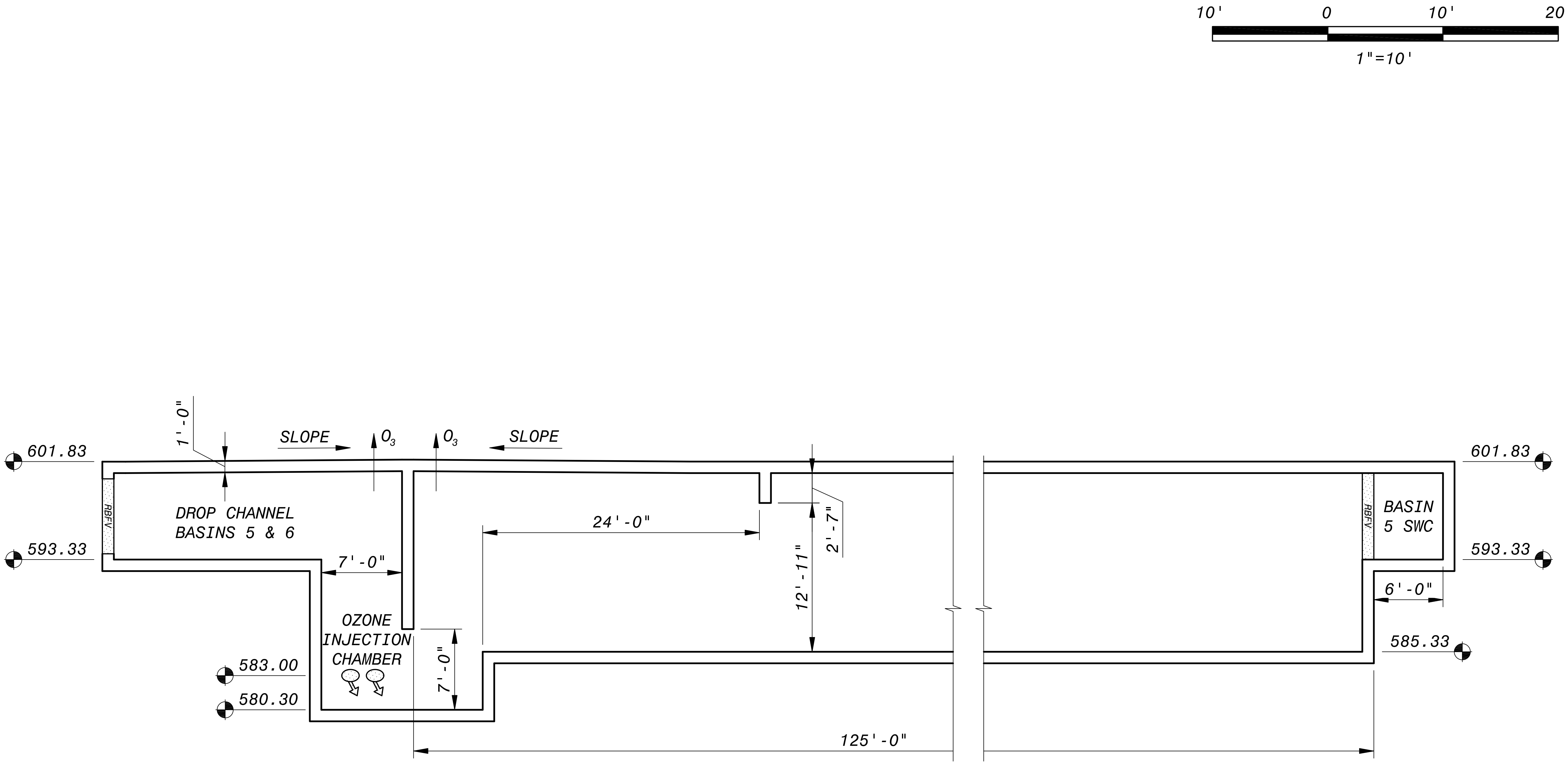

TOLEDO WTP REDUNDANT
CAPACITY & HAB IMPROVEMENTS

SETTLED WATER FLUME OZONATION
BASIN NOS. 5 & 6
SECTIONS

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FIGURE
4-12

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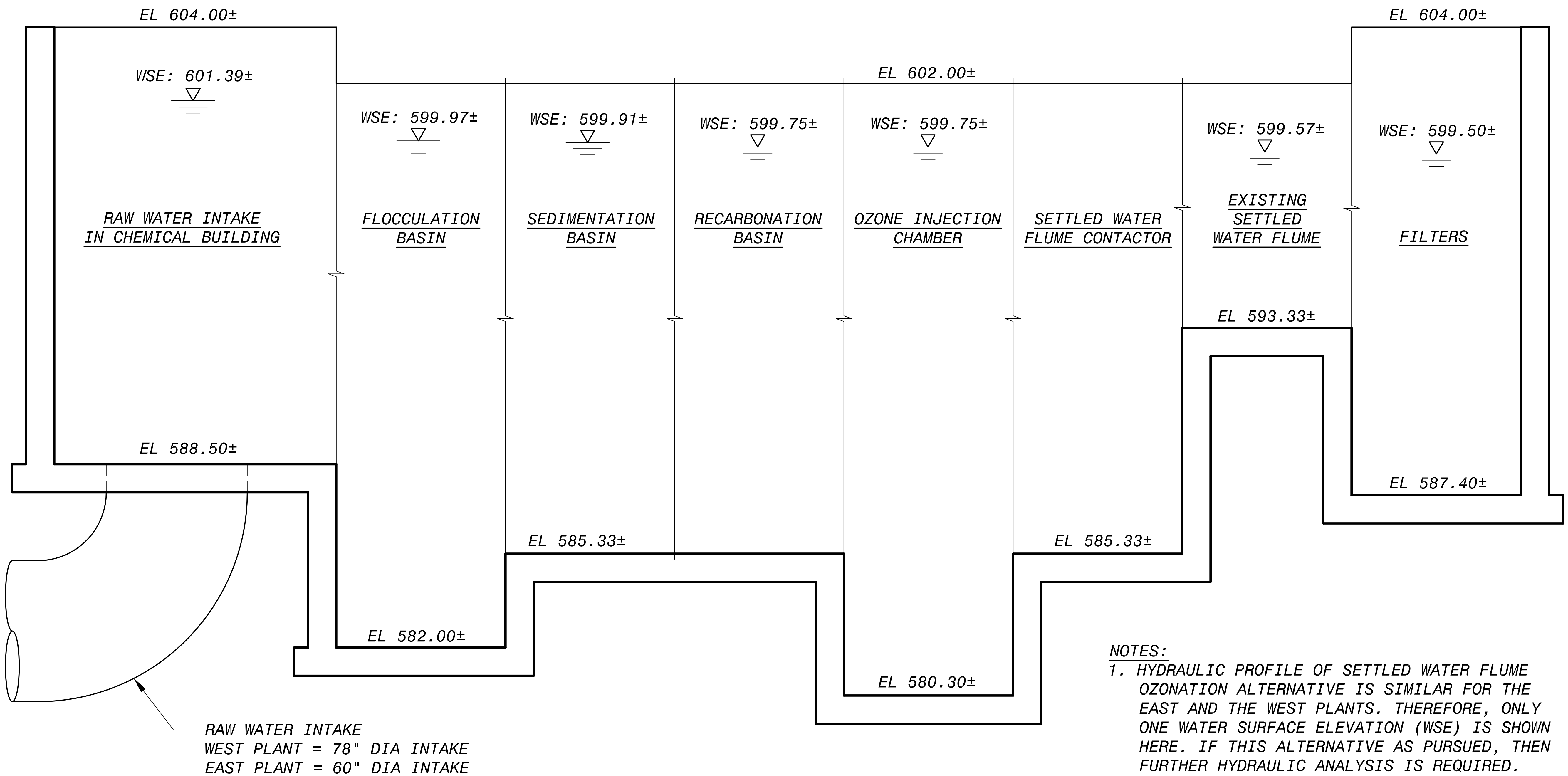

NOTES:

1. HYDRAULIC PROFILE OF SETTLED WATER FLUME OZONATION ALTERNATIVE IS SIMILAR FOR THE EAST AND THE WEST PLANTS. THEREFORE, ONLY ONE WATER SURFACE ELEVATION (WSE) IS SHOWN HERE. IF THIS ALTERNATIVE AS PURSUED, THEN FURTHER HYDRAULIC ANALYSIS IS REQUIRED.

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FIGURE 4-13

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5.0 Ozone Equipment

5.1 OVERVIEW

The following subsections describe equipment systems for oxygen storage and vaporization, ozone generation, dissolution, and destruction. The ozone will be generated within the ozone generator from gaseous oxygen which will be stored in a liquefied form within the liquid oxygen system. The generated ozone-in-oxygen gas will then be applied at the contact basins through the ozone dissolution system. Preliminary figures showing the proposed Ozone Generation Building and the Liquid Oxygen Storage Facility are contained in Figures 5-6 and 5-7 are the end of this section. The design criteria for the ozone system are contained in Table 5-2 at the end of this section.

5.2 OXYGEN SUPPLY

Ozone will be produced on-site using high purity oxygen stored on-site as liquid oxygen (LOX). The LOX will be delivered in semi-trailers in quantities of approximately 5,000 gallons per shipment. The LOX will be stored at a pressure of approximately 100 psi and maintained in a liquid form. The LOX will be converted to gaseous oxygen (GOX) through the use of ambient vaporizers. Heat gain will be provided from atmospheric air as the LOX flows through the vaporizing unit. The pressure of the storage tank will provide the motive force for the flow of oxygen out of the tank, through the vaporizers, to the ozone generators, and to the ozone dissolution system.

Other oxygen components include filters to remove fine particulates, ambient vaporizers to convert the oxygen from liquid to gaseous form, pressure regulation, valves, and instrumentation.

Figure 5-1 Vertical LOX Storage Tanks

5.2.1 LOX Storage Tank

LOX will be stored on-site in a vertical bulk cryogenic storage tank. The LOX storage tank will consist of an inner pressure vessel and an outer jacket. The space between the pressure vessel and outer jacket will be filled with Perlite or another similar insulating material and placed under vacuum to insulate the tank. The tank and LOX lines will be equipped with pressure relief valves between every pair of isolation valves to prevent over-pressurization of the tanks or lines due to vaporization of the liquid. Relief valves will be provided in the gaseous oxygen (GOX) lines and the tank will include rupture discs. The LOX storage tank will be located near the Ozone Generation Building within a gravel containment area.

There are several arrangements to provide LOX equipment, but the two most common approaches are to lease the equipment from the LOX supplier or the City own the LOX system. The approach used in this memo is to lease the equipment from the LOX supplier since it results in a lower capital cost and all maintenance on cryogenic equipment is provided by the LOX supplier. The primary disadvantages are that the annual cost is higher as a result of the leasing fee and the City is in a long-term, generally 7 to 10 year lease agreement. The cost of oxygen is generally the same regardless if the equipment is leased or owned.

During detailed design, the decision to lease verses own should be evaluated. Other evaluations could include on-site oxygen generation through vacuum swing adsorption or pressure swing adsorption. Onsite oxygen generation for this size facility is generally not advantageous and results in a much higher capital cost.

5.2.2 Vaporizers

The vaporizers will be located outside on concrete pads, adjacent to the LOX storage tanks.

The LOX will be converted to gaseous oxygen in the vaporizers. Three units will be provided: one for vaporization service, one for defrosting and one for standby. Each vaporizer will be sized to deliver 110 percent of the total future system demand.

Ambient vaporizers utilize heat from ambient air to raise the temperature of the LOX and convert it to a gaseous stage starting at a temperature of about -297°F and finishing at a temperature about 30 degrees below ambient. The vaporizer size (area available for heat transfer) and operation are dependent on the ambient air temperatures. Ice forms on the exterior of the vaporizers, increasing equipment weight and requiring each one to periodically cycle from on-line to off-line to defrost.

In the vaporizer, LOX will be vaporized to a gaseous state and heated to an operating temperature that approaches the ambient air temperature. Automatic open-close valves will be provided downstream from the vaporizers in the GOX piping to open as required when an ozone generator is placed on line and to sequence the vaporizers for duty, defrosting, and standby service.

Figure 5-2 Ambient LOX Vaporizers

5.2.3 Ancillary Gas Equipment

The GOX will flow through orifice plate flow meters and oxygen particulate filters. An initial pressure reduction of the GOX will take place outside in the LOX area. The initial pressure reduction will be done by mechanical pressure reducing valve packages. A second pressure regulation step will occur using pilot actuated regulators to the desired ozone generator inlet pressure of approximately 15 to 20 psig.

Oxygen supply to the ozone generators will be monitored for moisture.

5.2.4 Supplemental Air Sub-System

A supplemental compressed air system will be included as part of the ozone generation system. Generators that use liquid oxygen as the source gas operate more efficiently and reliably over a wider operating range when a small amount of nitrogen is added to the gas stream. An inexpensive method of adding nitrogen is to add clean, dry compressed air since air is greater than 70 percent nitrogen.

The supplemental air system will include the following components: two oil-less rotary scroll air compressors (lead and lag units), receiver tank, regenerative desiccant dryers, pre-filters and after-filters. The supplemental air system will be sized to deliver nitrogen to meet the ozone system supplier's requirements, which range from 0.1 to 2 percent of the oxygen gas flow. The system is also used to supply air to pneumatic valve actuators. One ozone system supplier contacted does not require supplemental air. Therefore the size and even need for the system is entirely based on the selected ozone system supplier.

5.3 OZONE GENERATION SYSTEM

To provide the transferred ozone dosage of 1.7 mg/L, three generators, each capable of producing 1,200 ppd of ozone at 12 percent by weight and 82 degree F cooling water, will be provided.

Figure 5-3 Ozone Generator (1,200 ppd)

The ozone generator is composed of two units: the generating shell and the power supply unit (PSU) where the electrical current and frequency of the power applied to the generator shell is manipulated.

Much of the power applied is rejected in the form of heat. Heat rejection from the PSU cabinet is regulated using a three way valve or other mechanism. The generator shell is cooled using water as well. The ozone generator and PSU will be skid mounted and bolted together in the field for grounding.

A production concentration of 12 percent weight was selected as it minimizes the LOX use and the operating cost difference between 10 and 12 percent is small ($0.54/lb ozone versus $0.49/lb ozone for 10 and 12 percent weight respectively).

The cooling water temperature selected was based on the maximum water temperature plus a 3 degree approach temperature that would be achieved across the closed loop heat exchanger.

A single ozone generating facility is proposed although the ozone contactors are in two separate complexes. The reasons that providing a single generating complex is recommended include:

- Fewer generators would be required since one redundant generator is provided for the single facility verses two if there were two generating facilities.
- Lower capital cost because ancillary systems (HVAC, electrical, cooling system, etc.) do not need to be provided.
- Decay of ozone gas occurs but the retention time between the generating facility and the application point is quite low, less than 30 seconds at design flow, because of the high velocities used in the design of gas piping. The contact time assumes ozone generation near the chlorine building and new ozone contactors being constructed to the West of Basin 4.
- Previous investigations that utilize non-adjacent ozone generating facilities (Southern Nevada Water Authority and North Texas Municipal Water District) indicate that the decay of ozone in well-designed conveyance pipelines is low.

5.4 OZONE CONTACTOR

The settled water ozone system will be located downstream from the sedimentation and recarbonation basins and upstream from filtration. Ozone will be applied at the basin inlet for the oxidation of algal toxins, primarily microcystin-LR. It is also highly effective for the oxidation of taste- and odor-causing compounds, oxidation of organic compounds, and inactivation of bacteria (*Giardia*) and viruses.

The ozone gas will be transferred into solution using sidestream injection. Sidestream injection has a higher capital cost than diffusers but the maintenance of the system is moved to the exterior of the contactor greatly reducing the need to enter the ozone contactor. Higher ozone transfer efficiency is generally achieved as well.

Two ozone contactor complexes will be provided: the west complex will treat water from Basins 1 through 4 and the east complex will treat water from Basins 5 through 8. Each complex will have two contactors, each rated for 40 mgd. One sidestream pump and one injector will be dedicated to each contactor and a common spare will be provided. The sidestream pumps will draw water from the inlet of the contactor and convey it through an injector that draws in the ozone gas. The gas water solution will then be injected directly into the main process water flow at the beginning of the Settled Water Ozone Contactor. Two manufacturers were contacted regarding the project and the design values are listed in Table 5-1.

The contact basin will have a concrete top slab to capture the ozone off-gas. An ozone destruct system will create a vacuum in the contactor headspace and will collect ozone off-gas for catalytic destruction prior to release to the atmosphere. The ozone destruct units will be a dedicated set of duty and standby units for the Settled Water Ozone Contactor.

Table 5-1 Ozone Contactor Design Criteria

DESCRIPTION	VALUE	
Ozone Contact Basins		
Number of Contactors	4	
Nominal Design Flow per Contactor, mgd	40	
Minimum Design Flow per Contactor, mgd	15	
Type	Concrete, baffled	
Ozone Gas Train Manifold		
Number	6 (4 duty, 2 standby), 3 in each facility	
Design gas flow, scfm each	46	
Minimum gas flow, scfm each	9	
Turndown	5:1	
Ozone Dispersion System		
Dispersion Method	Sidestream Injection	
Minimum Transfer Efficiency %	95	
Sidestream Pumps		
Number	6 (4 duty, 2 standby), 3 in each facility	
Type	Horizontal end suction centrifugal	
	Supplier A	Supplier B
Rated Flow, gpm	400	935
Rated Head, ft	95	95
Motor power, HP	25	40

5.5 OZONE DESTRUCT SYSTEM

The concentration of the off-gas in the contact basin head space can be as high as 1% wt (6,600 ppm(v)) and as such cannot be released into the atmosphere. Safe levels are less than 0.10 ppm(v). After leaving the contact basin, the off-gas will pass through a pre-heater in order to decrease relative humidity, as condensed moisture will damage the catalyst. The off-gas is heated to 30°F above ambient temperature. The off-gas then passes through the catalyst chamber resulting in a thermal catalytic reaction to reduce the concentration of the ozone gas to approximately 0.08 ppm(v). The discharge gas stream leaving the ozone destruct system will be monitored to verify that the vent gas has ozone levels below the permissible limit to discharge to atmosphere.

One of the contactor options utilizes a portion of the existing flumes to provide contact time. In that approach, the new portion of the ozone contactor will include provisions to trap all of the off-gas and not allow the gas to be conveyed into the existing flume sections. The gas from the transfer portion of the new contactor will be sent to destruct. If the water entering the existing flume sections contains a dissolved ozone residual, the water will off-gas ozone to unsafe levels which should be collected, treated, and discharged. Failure to do so could result in unsafe conditions within the filter gallery and exposure of non-compatible materials to ozone off-gas.

It is desirable to have separate off-gas treatment systems for the new and older contactors for the following reasons:

1) The concentration of off-gas in the existing flume (1 to 150 ppm(v)) is much lower than the new (3,300 to 6,600 ppm(v)) section. Migration of the higher concentration off-gas to the older flume section increases the rate of corrosion in the existing flume sections.

2) The off-gas in the new section will contain elevated levels of oxygen (approximately 90 to 99 percent oxygen) which support combustion. Separating that off-gas from off-gas that is roughly the composition of air is (21% wt) is desirable. Blending ambient air, which could contain contaminants, into enriched oxygen gas is undesirable from a safety standpoint.

The (existing) flume destruct system would be located in the ozone contactor building adjacent to the off-gas destruct units. Ambient air would be allowed into the existing flume near the filter gallery and the headspace of the flume refreshed back to the contactor building through use of the flume destruct blower.

Figure 5-4 Catalytic Ozone Destruct Unit

5.5.1 Cooling Water Sub-System

Three types of cooling water systems are generally considered:

1. Open Loop: In an open loop system, plant water is passed through the ozone generator and returned back to the full-scale process.

2. Closed Loop: In a closed loop system, the water in contact with the ozone generator is recirculated and the heat added to the system is rejected through a heat exchanger into once through plant water.

3. Chilled Water: A chilled water system is a closed loop system, but includes air or water cooled chillers to drive the cooling water temperature down to 50 ^{o}F. One ozone system supplier contacted requires the use of chillers to meet the design requirements.

A closed loop cooling water system was assumed in the design to cool the ozone generators as it has nearly become industry standard practice as a result of corrosion that can occur while equipment is offline. A closed loop system includes plate and frame type heat exchangers, an expansion tank and re-circulation pumps. The closed loop water is treated with corrosion and microbial inhibitors.

Figure 5-5 Closed Loop Cooling Water System

5.5.2 Piping and Other Appurtenances

LOX piping is generally copper, valves are brass (cryogenic type for valves) with extended stems, and all piping insulated to prevent incidental contact with pipes that may be -297°F and heat gain to the tank that would waste LOX. Gaseous oxygen and ozone piping are 316L stainless steel and valves are 316L stainless steel and include ozone resistant gaskets such as Polytetrafluoroethylene (PTFE).

All ozone and oxygen piping and valves must be oxygen cleaned for the removal of hydrocarbons that could become flammable in the presence of oxygen and particles that could also become flammable or foul the ozone generator. Valves and skidded equipment are generally cleaned in the factory. Field installed pipe is oxygen cleaned by a specialty cleaning company.

5.5.3 Ambient Monitors

Ambient ozone and oxygen monitors are provided to ensure safe operating conditions within enclosed spaces that contain oxygen or ozone. They are generally tied to the HVAC system that is used to evacuate the elevated oxygen or ozone gas from the room and the ozone control system to stop the flow of gas into that space. Ambient ozone concentration analyzers would be located in the ozone generating building and at each contactor building.

5.5.4 Ozone Analyzers

Each ozone generator has a high concentration ozone analyzer that is used for feedback control of the power applied to the generator. In addition, for each contactor building, medium concentration off-gas monitors measure the ozone concentration in the off-gas collection pipes to determine ozone transfer efficiency. Low concentration vent-gas ozone analyzers are included in the discharge of each destruct unit to monitor the system performance.

5.5.5 Ozone Residual Monitoring

Monitoring the ozone residual will be accomplished with the direct measurement of the residual with sampling stations. Each ozone contactor will have three monitoring stations that are are piped to six or more locations inside the contactor. The sample locations are selected based on seasonal, flow, and water quality variations to monitor treatment performance.

5.6 OZONE SYSTEM DESIGN CRITERIA

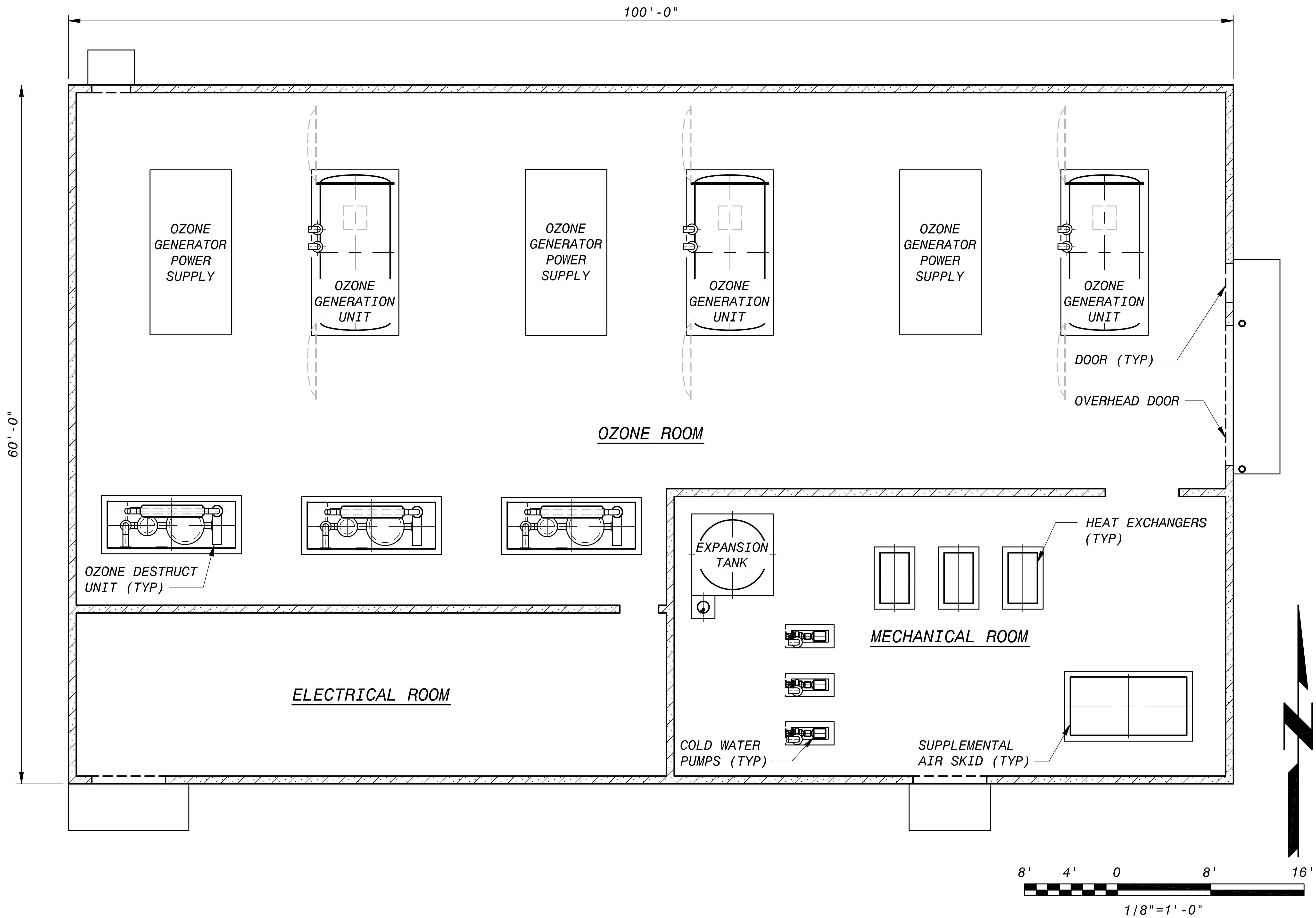
A summary of the ozone system design criteria is provided in Table 5-2 below.

Table 5-2 Settled Water Ozone System Design Criteria

DESCRIPTION	VALUE
Applied Ozone Dosage	
Minimum, mg/L	1.0
Average, mg/L	1.3
Maximum, mg/L	1.8
Plant Flow	
Minimum, mgd	60
Average, mgd	79
Maximum, mgd	139
Peak, mgd	160
Ozone Feed Rate	
Minimum Flow/Minimum Dose, ppd	500
Average Flow/Average Dose, ppd	850
Maximum Flow/Maximum Dose, ppd	2,400
Turndown Ratio	5:1
Design Parameters	
Liquid Oxygen Storage and Feed Sub-system	
Delivered Chemical	99.5% Liquid Oxygen
Liquid Oxygen Storage Tanks	
Number of Tanks	2
Nominal Tank Capacity, each, gal	11,000
Type	Vertical
Location	Outside
Dimensions	9’-5” diameter x 44’ tall
Days of Storage (at Average Flow, Average Dose)	32
Days of Storage (at Maximum Flow, Maximum Dose)	10
Oxygen Vaporizer	
Number	3
Type	Ambient
Design gas flow rate, scfm	184
Oxygen Particulate Filter	
Number	2 (1 duty, 1 standby)
Design flowrate, each scfm	184

Supplemental Air Sub-system	
Number of compressors	2 (1 lead, 1 lag)
Type	Rotary Scroll, oilless
Discharge pressure, psig	115
Estimated motor size, hp	10
Number of dryers	2 (packaged)
Type	Regenerative desiccant
Ozone Generators	
Number of generators	3 (2 duty, 1 standby)
Design capacity, each, ppd	1,200
Ozone gas concentration, minimum percent by weight at design capacity	12
Ozone gas concentration range, percent by weight	4 to 12
Cooling water temperature range, ^{o}F	33 to 82
Power Supply Units	
Number	3 (1 per generator)
Type	Medium Voltage
Cooling Water	
Power Supply Cooling Water Flow, gpm, each	40
Generator Cooling Water Flow, gpm, each	400
Closed-Loop Water System	
Open loop cooling water temperature range, degrees F	33-79
Heat exchanger approach temperature, ^{o}F	3
Maximum closed loop cooling water temperature entering generator, degrees F	82
Total estimated closed loop water requirement, gpm	880
Total estimated open loop water requirement, gpm	880
Closed Loop Pumps	
Number closed loop pumps	3
Type	Centrifugal
Flow rate, gpm each	440
Head, ft	40
Estimated pump motor size, each, hp	15
Heat Exchangers	
Number heat exchangers	2 (1 duty; 1 standby)
Type	Plate and Frame

OZONE OFF-GAS DESTRUCT SUB-SYSTEM	
Ozone Destruct Units	
Number of units	6 (4 duty, 2 standby)
Type	Thermal catalytic
Blower	
Type	Centrifugal
Design flowrate, each scfm	101
Estimated motor size, each, hp	5
Estimated preheater power requirement, each, kw	5.0



TOLEDO WTP REDUNDANT CAPACITY & HAB IMPROVEMENTS

OZONE GENERATION BUILDING PLAN

CITY OF TOLEDO

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FIGURE 5-6

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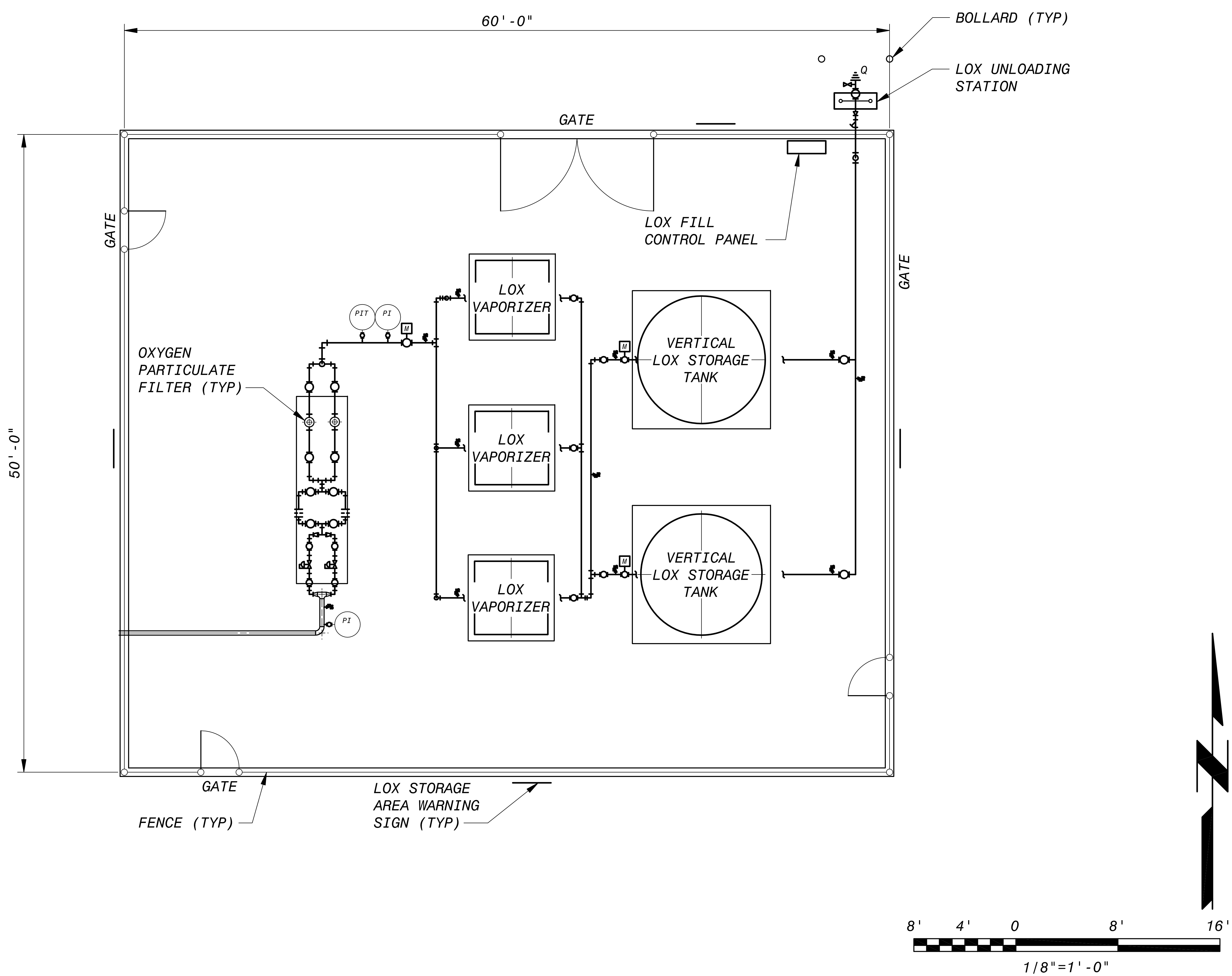

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TOLEDO WTP REDUNDANT CAPACITY & HAB IMPROVEMENTS

LIQUID OXYGEN STORAGE FACILITY PLAN

CITY OF TOLEDO

DATE: 02/05/15

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FIGURE 5-7

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6.0 Engineer's Opinion of Conceptual Level Cost

6.1 CAPITAL COST

The Engineers Opinion of Probable Cost as presented herein was developed during the preparation of this Conceptual Report. Ozone equipment vendors were contacted, and budget cost information was obtained for the larger items of equipment. Ozone Equipment cost was the median value of four supplier quotes. Where possible, quantities were calculated from the conceptual engineering work. Unit costs were based on budget costs from equipment vendors, historical bidding data from similar projects by Black & Veatch, and Means Building Construction Cost Data. Costs were adjusted for inflation, where required. An additional 30 percent contingency was added to the capital cost estimate to reflect the conceptual design of the work. Engineering, legal, and administrative costs were estimated as 20% of the project cost. See Table 6-1 for the Engineers Opinion of Conceptual Level Cost of the Settled Water Ozone alternatives.

Table 6-1: Collins Park WTP Ozone System Conceptual Cost Estimate

		SETTLED WATER FLUME CONTACTOR	EXTERNAL SETTLED WATER OZONE CONTACTOR
Capacity, mgd		160	160
Generator Size, ppd		1,200	1,200
Number Generators Installed		3	3
Firm Dosage, mg/L		1.8	1.8
Number Contactors		4	4
Contactor Hydraulic Retention Time, min		4 in new contactor, 6 in existing flume	10 in new contactor
		Settled Water Flume Contactor	**External Settled Water Ozone Contactor**
Estimated Equipment Cost		$4,116,250	$4,116,250
Installation	25%	$1,029,063	$1,029,063
Mechanical (process piping, supports)	25%	$1,029,063	$1,029,063
Installed Equipement[1]		**$6,200,000**	**$6,200,000**
Ozone Building		$1,200,000	$1,200,000
LOX Storage Area		$40,000	$40,000
Injection Buildings		$960,000	$960,000
Contactor		$1,000,000	$3,500,000
Misc. Work		$2,250,000	$2,970,000
Pump Station		$0	$1,043,000

		SETTLED WATER FLUME CONTACTOR	EXTERNAL SETTLED WATER OZONE CONTACTOR
Electrical and I&C	20%	$2,300,000	$3,200,000
Civil / Site Work	3%	$300,000	$500,000
Construction Direct Cost		**$14,250,000**	**$19,613,000**
Installation Contractor's O & P	10%	$1,400,000	$2,000,000
Contractor Bonding and Insurance	5%	$800,000	$1,100,000
General Conditions	7%	$1,100,000	$1,500,000
Construction Cost w/o Contingency		**$17,600,000**	**$24,200,000**
Contingency	30%	$5,300,000	$7,300,000
Total Construction Cost		**$22,900,000**	**$31,500,000**
Construction Midpoint - June 2018	3%	$1,400,000	$1,900,000
Total Construction Cost at Midpoint		**$24,300,000**	**$33,400,000**
Engineering, Legal, and Admin.	20%	$4,900,000	$6,700,000
Budgetary Estimate of Project Cost		**$29,000,000**	**$40,000,000**

[1] Installed equipment includes ozone generators, injection equipment, ozone destruct units, controls, and loose valves and instruments, all process mechanical piping, supports, Electrical and I&C for Ozone equipment is captured as a percentage in the item category below.

6.2 OPERATING & MAINTENANCE COST

The engineer's opinions of probable O&M cost were developed based on the following categories:

- Power Costs. These costs were determined based on the anticipated plant loads and an average power cost of $0.056/KWh.
- Equipment Repair and Miscellaneous O&M. These costs were assumed to represent a constant 2% each year of the project's equipment cost.
- Chemical Costs. These costs were determined based on the chemicals and dosages described in Chapter 5. The chemical unit prices were established based on vendor quotes and recent historical data.
- The LOX Leasing fee was quoted at $3,000 per month.
- A quenching chemical system was not included in this estimate. Provisions will be made for a future quenching system, which would need to be installed if disinfection credit is needed.
- Cooling Water costs are estimated based on a 30 psi, running open loop system.
- Staffing costs are included and the value was based on a burdened cost of $35.50/hr. A summary is shown below in Table 6-2.

Table 6-2: Annual Cost Estimate

	SETTLED WATER FLUME CONTACTOR	EXTERNAL SETTLED WATER OZONE CONTACTOR
Electricity, $/yr	$124,000	$136,000
LOX, $/yr	$86,000	$86,000
Cooling Water	$1,000	$1,000
LOX Leasing Fee ($3,000/month)	$36,000	$36,000
Staff Salary	$74,000	$74,000
Estimated Annual Operating Cost, $/yr	**$321,000**	**$333,000**
Annual Maintenance @2%, $/yr	**$82,000**	**$82,000**

6.3 PRESENT WORTH EVALUATION

The present worth cost was determined using the above capital and O&M costs in an evaluation period of 20 years after construction of the project. An effective interest rate (i.e., resulting from discounting inflation from the desired rate of return) of 3% was used for calculating the present worth cost. The capital and O&M present worth costs for the two ozonation alternatives are summarized in Table 6-3.

Table 6-3: Present Worth Cost Estimate

	SETTLED WATER FLUME CONTACTOR	EXTERNAL SETTLED WATER OZONE CONTACTOR
Capital Cost	$29,000,000	$40,000,000
Annual Operating Cost	$321,000	$333,000
Annual Maintenance at 2 % equipment cost	$82,000	$82,000
Effective interest rate	3.00%	3.00%
20 Year Present Worth Cost	**$35,000,000**	**$46,200,000**

7.0 Construction Considerations

7.1 OVERVIEW

Many factors are utilized to determine the constructability of the work described herein. Of those, a major driving factor is the production capacity of the treatment facility versus system demands. Provided flow data stated the minimum day flow is 60 mgd, the average day flow is 79 mgd, the maximum day flow is 135 mgd, and the peak hour flow is 160 mgd.

The historical flow data provides the requirement of a minimum treatment plant capacity of more than 60 mgd to be maintained at all times during low demand period (i.e., winter months) and a treatment capacity of at least 80 mgd to be maintained during the early spring and late fall. With this information in mind and considering conservative approaches to construction, the following conceptual construction sequence is proposed.

- Congruent with the submittal stage of construction, the contractor may begin parallel work activities on the construction of the Basins 7 & 8, the construction of the Ozone Generation Facility, the construction of the Liquid Oxygen Storage Facility, and the construction of the external contactors—if the latter item is pursued. During this time, it is expected 120 mgd of WTP capacity could be maintained, except for a short duration to interconnect the existing WTP structures to the Basin 7 & 8 trains. The short connection duration would truncate WTP flow capacity to 80 mgd; that is, only flow from the West Plant.

- After Basins 7 & 8 are substantially complete, Basins 5 & 6 may be taken offline to incorporate improvements—thereby limiting WTP capacity to 120 mgd; that is, Basins 1, 2, 3, 4, 7, & 8 operating each at 20 mgd per basin. It would be preferable to take Basins 5 & 6 out of service for upgrades right after Basins 7 & 8 become substantially complete so that major construction activities are limited to one side of the WTP. If the Basins 5 & 6 work is scheduled at a time that the WTP can meet production objectives with 80 mgd of total capacity, then Basins 1 & 2 improvements may begin in parallel to Basins 5 & 6 improvements. Otherwise, Basins 5 & 6 work will need to be substantially complete prior to improving the West Plant.

  Upon substantial completion of the East Plant (Basins 5, 6, 7, & 8), work may commence on the West Plant. It is suggested that Basins 1 & 2 be improved first to allow ample area for external contactor construction to begin—if this option is pursued. During Basins 1 & 2 improvements, WTP capacity is expected to be 120 mgd; that is, Basins 3, 4, 5, 6, 7, & 8 in full operations. Near the substantial completion of the Basins 1 & 2 improvements, WTP capacity would reduce to 80 mgd for a short duration to facilitate Basins 1 & 2's interconnection with Basins 3 & 4. If the Basins 1 & 2 work is scheduled at a time that the WTP can meet production objectives with 80 mgd, then Basins 3 & 4 improvements may begin in parallel to Basins 1 & 2 improvements.
- If 120 mgd of capacity were required to be maintained throughout the West Plant improvements duration, then the Basins 3 & 4 improvements would require the substantial completion of the Basins 1 & 2 work.

- During the Basin 3 & 4 improvements, it is expected that all equipment be delivered, installed, and tested, as is practicable. WTP capacity would reduce to 80 mgd during the short Basins 3 & 4 and Basins 1 & 2 interconnection period. Additionally, short duration basin shutdowns may be required to test plant equipment.

In summary, it appears the proposed facilities can be built while keeping the WTP production capacity generally at 120 mgd with short periods of 80-mgd production to allow for interconnections. Design and construction should be able to be completed conservatively within 12 months and 30 to 36 months, respectively. If the construction sequence is specified carefully with completion milestones on specific dates, construction may be accelerated by allowing longer periods of 80-mgd WTP capacity to occur. There are also opportunities to advance the schedule through alternative project delivery methods (i.e., Design-Build and Construction Management at Risk) or pre-procurement of ozone equipment.

Based on the assumption that the construction procedure will follow a traditional design-bid-build model and a conservative approach will be adopted, the schedule presented in Figure 7-1 is anticipated. Preliminary assessment suggests that pre-procurement of equipment may be able to reduce the construction schedule by about 6 months.

7.2 SCHEDULE

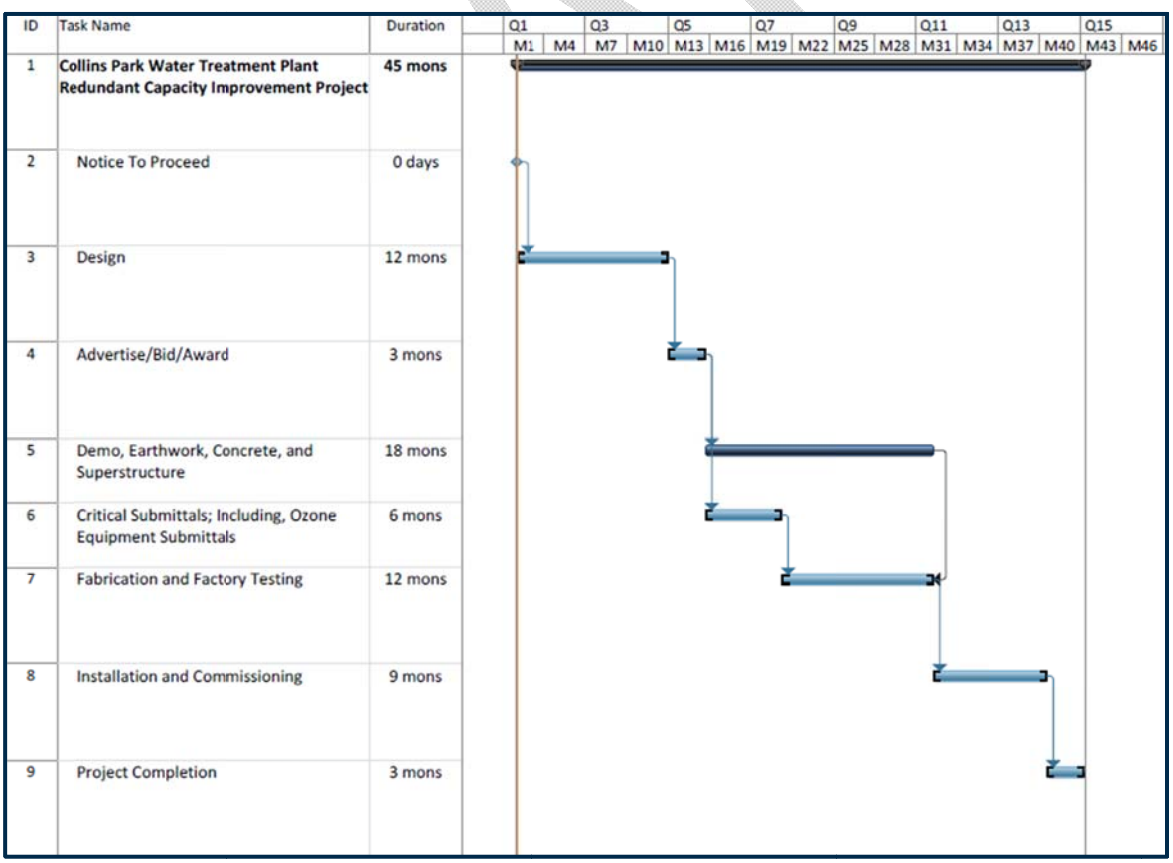

Figure 7-1 Proposed Construction Schedule

Appendix D

Evaluation of GAC Treatment for Microcystin Removal

TECHNICAL MEMORANDUM

To:
City of Toledo, OH

ARCADIS U.S., Inc.
One SeaGate
Suite 700
Toledo, OH 43604
Tel 419 473 1121
Fax 419 473 2108

From:
ARCADIS U.S., Inc.

Date:
February 5, 2015

Subject:
Evaluation of GAC Treatment for Microcystin Removal

ARCADIS Project No.:
TO000747.M002

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1. Introduction

A third alternative treatment barrier against microcystin and other harmful algal byproducts is granular activated carbon (GAC). Previous research has shown that GAC can effectively remove certain microcystin variants, and a study was conducted as part of this project to determine whether GAC treatment is a viable option for protecting against microcystin at the Collins Park WTP. Notably, GAC captures dissolved contaminants through a process called adsorption, where contaminants migrate into tiny pores within the GAC and become trapped there. This process is not effective for removing cell-bound microcystin. Therefore, GAC treatment could only serve to capture extracellular microcystin that passes through the filters. GAC treatment could take the form of filter adsorbers, where the GAC is installed in the filter basins, or post-filter contactors, where the GAC is contained in basins or vessels that are located downstream of filtration.

A key criterion in designing GAC filter adsorbers or post-filter contactors is empty-bed contact time (EBCT). This is the time it takes for water to move through the zone where GAC is contained, when the zone is empty. It directly reflects the time available for adsorption to occur, and strongly influences the contaminant removal performance of a GAC treatment system. The EBCT that can be achieved in a filter adsorber is often less than 10 minutes, due to the limited bed depth available in most filter basins. Post-filter contactors are usually designed with an EBCT of between 10 and 20 minutes. The optimum EBCT for a particular application is the lowest EBCT that allows for reasonable GAC change-out frequencies. GAC cannot remove contaminants indefinitely, and once its adsorption capacity is exhausted, it must either be replaced or reactivated. It is advantageous to select a low EBCT because this reduces the GAC bed depth requirements for filter adsorbers, and decreases the size requirements (i.e., capital cost) for post-filter contactors. However, shortening the EBCT also increases the rate at which contaminants break through the GAC (i.e., the rate at which the GAC becomes spent); meaning, it leads to higher GAC replacement rates. A GAC replacement rate of more than 1-2 times per year would in most cases be deemed impractical.

The main goals of the GAC evaluation conducted as part of this project were to:

- Determine whether reasonable microcystin breakthrough times are achievable at typical EBCTs
- Identify the best-performing GAC type among commercially-available GACs
- Determine whether filter adsorbers are a viable option
- Develop preliminary design and operating recommendations for implementing GAC treatment at the Collins Park WTP

This evaluation also included an assessment of whether onsite GAC reactivation would be a cost-effective alternative to offsite reactivation or replacement with new GAC. In some cases, it can be cheaper to build a thermal reactivation facility and reactivate spent GAC on site, rather than having a vendor reactivate the material or purchasing new GAC for each change-out.

2. Background

2.1. Microcystin Removal by Activated Carbon

Research on the removal of microcystin by activated carbon has shown varying effectiveness across carbon types and microcystin variants. For example, Donati et al (1993) evaluated the microcystin-LR (mLR) removal performance of eight different powdered activated carbons (PACs). Wood-based PAC achieved the highest removal rates, followed by coal-based PAC. Coconut shell- and peat moss-based PAC were the least effective. Further analysis indicated that mesopore volume was the leading indicator of removal performance. Wood-based carbons exhibited the highest mesopore volume, correlating to the greatest removal of mLR, whereas coconut and peat moss-based products exhibited the lowest mesopore volumes of the carbons tested. It should be noted that no data on the effectiveness of lignite-based activated carbon were discovered during the literature review for this evaluation.

With respect to microcystin removal by GAC, prior research has shown that microcystin can rapidly break through a GAC contactor, and that breakthrough times may be influenced by the presence of background organic matter. Newcombe (2002) evaluated the mLR and mLA breakthrough performance of Picacarbon, a chemically activated wood-based GAC. In this study, the mLR and mLA levels in the GAC effluent exceeded the World Health Organization (WHO) recommended guideline of 1 µg/L in little more than a month of operation. This rapid breakthrough was attributed to competition for adsorption sites with naturally-occurring organic matter in the influent water. A study by Alvarez et al. (2010) showed that a GAC mini-column that simulated an EBCT of 5 minutes processed 15% fewer bed volumes when removing mLR from a source water containing about 5 mg/L total organic carbon (TOC), as compared to a source water with about 2 mg/L TOC. Interestingly, this difference was not observed for mini-columns that simulated an EBCT of 10 or 15 minutes. That is, the number of bed volumes processed was roughly the same for both the high and low TOC waters.

Microcystin variants are not equally removed by activated carbon. Cooke and Newcombe (2002) demonstrated the following trend concerning microcystin uptake by PAC (shown from most easily to least easily removed): mRR > mYR > mLR > mLA. The study by Newcombe (2002) noted above showed similar results, in that the GAC columns employed in that project removed mLA less effectively than mLR. Much of the research to date (involving activated carbon) has focused on the removal of mLR. This is not surprising, as mLR is often the most prevalent microcystin variant, and it is not that well removed (meaning it allows for a relatively conservative assessment of activated carbon removal performance).

2.2 Microcystin Trends in Lake Erie

A series of microcystin outbreaks in the western basin of Lake Erie during the 1990s spurred efforts to better understand this algal toxin. The National Oceanic and Atmospheric Administration (NOAA) funded the Monitoring and Event Response to Harmful Algal Blooms in the Lower Great Lakes (MERHAB-LGL) project in 2002 to develop an approach for dealing with harmful algal blooms (Boyer, 2007). Upwards of 2500 samples were collected from New York state lakes between 2000 and 2004. Of the samples gathered from Lake Erie:

- 40% contained measurable levels of particulate microcystin (>0.01 µg/L);
- 29% contained levels exceeding 0.1 µg/L; and
- 12% contained levels exceeding the WHO advisory limit of 1.0 µg/L.

Dr. Gregory Boyer, acting director of the Great Lakes Research Consortium, has indicated that the microcystin present in Lake Erie during the harmful algal bloom that occurred in 2014 was comprised of the following variants:

- mLR (60% – 80%);
- mRR (10% - 25%); and
- mYR (5% - 15%).

It is important to note that the microcystin variant profile in Lake Erie has not been carefully studied. Thus, it is unknown whether the profile observed in 2014 would be representative of future microcystin episodes.

3. Testing Approach

A series of rapid small-scale columns tests (RSSCTs) were conducted to evaluate the impact of EBCT and GAC type on microcystin removal. Rapid small-scale column tests involve using miniature GAC columns to simulate the performance of full-scale GAC systems. It has repeatedly been shown that properly designed RSSCTs can accurately predict full-scale breakthrough performance. Notably, there are two primary methods for designing RSSCTs: the constant diffusivity approach that assumes contaminant diffusion rates within GAC pores are independent of GAC grain size, and the proportional diffusivity approach that assumes contaminant diffusion rates within GAC pores are a function of GAC grain size. These methodologies have been carefully described elsewhere (Crittenden et al., 1989). For the RSSCTs conducted during this evaluation, the constant diffusivity approach was employed. In a recent study by Summers et al. (2014), it was shown that constant diffusivity RSSCTs more accurately predicted the time to initial breakthrough for a range of micropollutants, as compared to proportional diffusivity RSSCTs. Although microcystin was not included in that study, the chemicals that were tested are not vastly different from microcystin in terms of molecular weight, pK_a and log K_{ow}. Unfortunately, a literature search did not uncover any previous studies examining the applicability of constant and proportional diffusivity RSSCTs for predicting microcystin breakthrough performance.

Four commercially-available GACs were included in this evaluation, and these are identified in Table 3-1.

Table 3-1. Commercially-Available GACs Included in This Evaluation

Product Name	Manufacturer	Source Material
AquaCarb CX	Evoqua	Coconut shells
Filtrasorb 400	Calgon Carbon	Bituminous coal
WV-B 30	MeadWestvaco	Wood
HYDRODARCO 4000	Cabot	Lignite

Notably, each of the carbon types that was tested is manufactured from a different source material. As discussed above, wood-based carbons have previously performed well for removing microcystin. Also, there are no readily available data on the performance of lignite-based activated carbons for removing microcystin; and thus it was deemed important to include a representative lignite-based GAC in this study.

Combined filter effluent from the Collins Park WTP served as the influent for the RSSCTs, given that full-scale post-filter GAC contactors would process this same water. Samples of combined filter effluent were collected during the week of 12/01/14, and the total organic carbon (TOC) concentration in these samples was about 1.4 mg/L. The combined filter effluent samples did not contain measureable amounts of microcystin, and thus the influent to the RSSCTs was spiked with mLR, to simulate a microcystin event in which extracellular microcystin appears in the filter effluent. Microcystin-LR was selected for these tests because it was the most prevalent form of microcystin during the harmful algal bloom of 2014, and because it is more difficult to remove with activated carbon than the other variants that have appeared in Lake Erie (mRR and mYR). Thus, using mLR as a surrogate for naturally-occurring microcystin mixtures allows for a conservative assessment of activated carbon removal performance.

4. Results

Figure 4-1 shows the microcystin breakthrough profiles from the initial round of RSSCTs. Four RSSCTs were conducted, each simulating a full-scale GAC contactor with an EBCT of 10 minutes, and each containing a different GAC type. An EBCT of 10 minutes was selected because it is both the shortest EBCT normally selected for post-filter GAC systems, as well as the longest EBCT (roughly) that could be achieved in the Collins Park WTP filters (if GAC was installed in the filters). The influent microcystin level for these tests was approximately 8 µg/L; and this likely simulates a worst-case microcystin episode. Notably, the coconut shell- and bituminous coal-based GACs exhibited fairly rapid microcystin breakthrough, with the effluent microcystin level exceeding the WHO standard of 1.0 µg/L within 25 and 50 days,[1] respectively. By comparison, the effluent microcystin level for the wood-based GAC had not exceeded 1.0 µg/L after nearly 150 days[1] of operation, and the effluent microcystin level for the lignite-based GAC was still below 1.0 µg/L after nearly 300 days[1] of operation. The breakthrough performance of each GAC type is summarized in Table 4-1.

[1] Simulated full-scale operating time

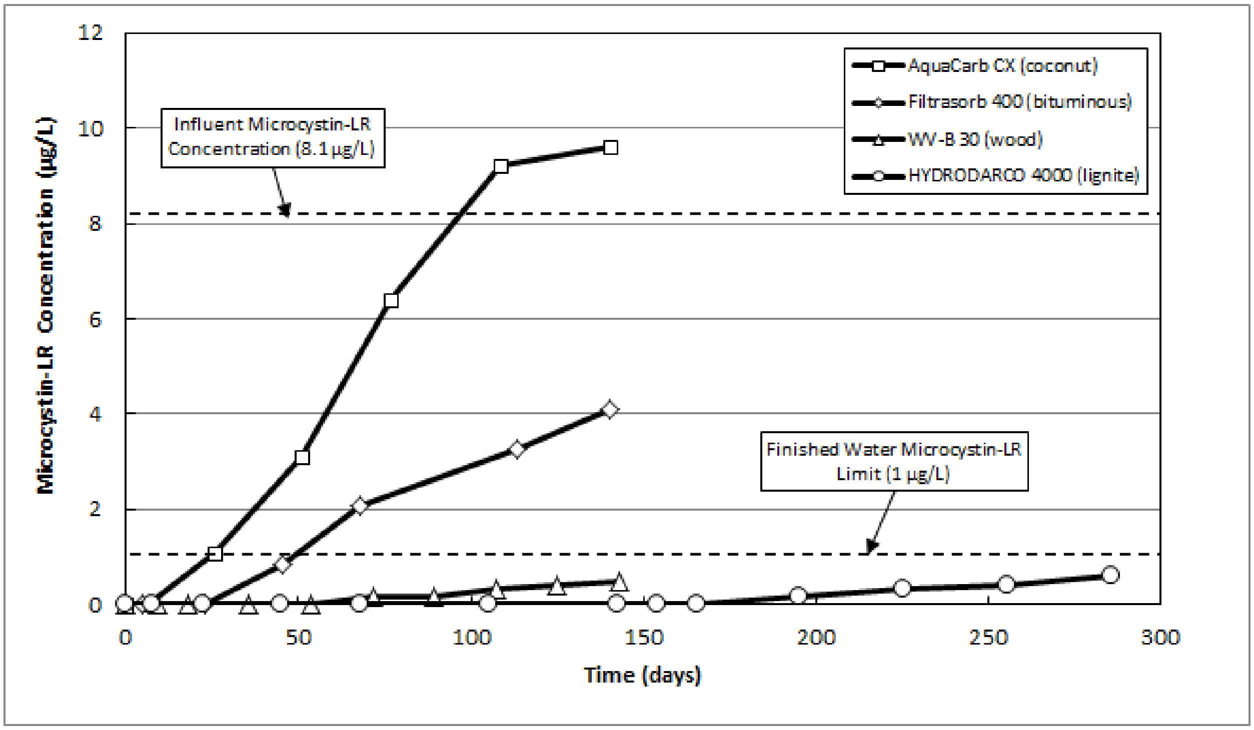

Figure 4-1. Microcystin breakthrough profiles (EBCT = 10 minutes, background TOC concentration = 1.4 mg/L)

Table 4-1. Microcystin Breakthrough Times for Various GAC Types (EBCT = 10 minutes; influent microcystin level = 8.1 µg/L; background TOC concentration = 1.4 mg/L)

GAC Type	Time to Initial Breakthrough	Time to Exceed 1.0 µg/L
Coconut shell-based	10 days	25 days
Bituminous coal-based	23 days	50 days
Wood-based	54 days	225 days (estimated)
Lignite-based	165 days	350 days (estimated)

This vast difference in removal performance highlights the importance of GAC selection in designing/operating a GAC treatment system. Clearly, the wood- and lignite-based GACs are far superior in terms of removing microcystin from the Collins Park combined filter effluent. These results indicate that post-filter contactors with an EBCT of 10 minutes could continuously remove an influent microcystin level of 8 µg/L down to less than 1.0 µg/L for 300 days or more, if a lignite-based GAC was in use.

It is important to note that virgin (new) GAC was used in generating the breakthrough profiles shown in Figure 4-1. The GAC samples had not previously been exposed to water containing natural organic matter (NOM). However, in a full-scale system, the GAC contactors will likely be online before microcystin appears in the combined filter effluent (GAC influent). Thus, the GAC will be partially loaded with NOM before it begins removing microcystin; and this will likely shorten the time to microcystin breakthrough (as compared to the breakthrough times for virgin GAC, as shown in Figure 1). To evaluate the impact of NOM preloading on microcystin removal, a fifth RSSCT was conducted, in which lignite GAC (again with an EBCT of 10 minutes) first processed combined filter effluent that did not contain microcystin, for a period of 140 days,[1] after which the influent microcystin level was increased to about 7 µg/L. In other words, this test simulated a full-scale operating scenario in which the GAC contactors are online for roughly 4.5 months before microcystin appears in the GAC influent. The results are presented in Figure 4-2, which indicate that the effluent microcystin level reached 1.0 µg/L about 145 days after the microcystin appeared.

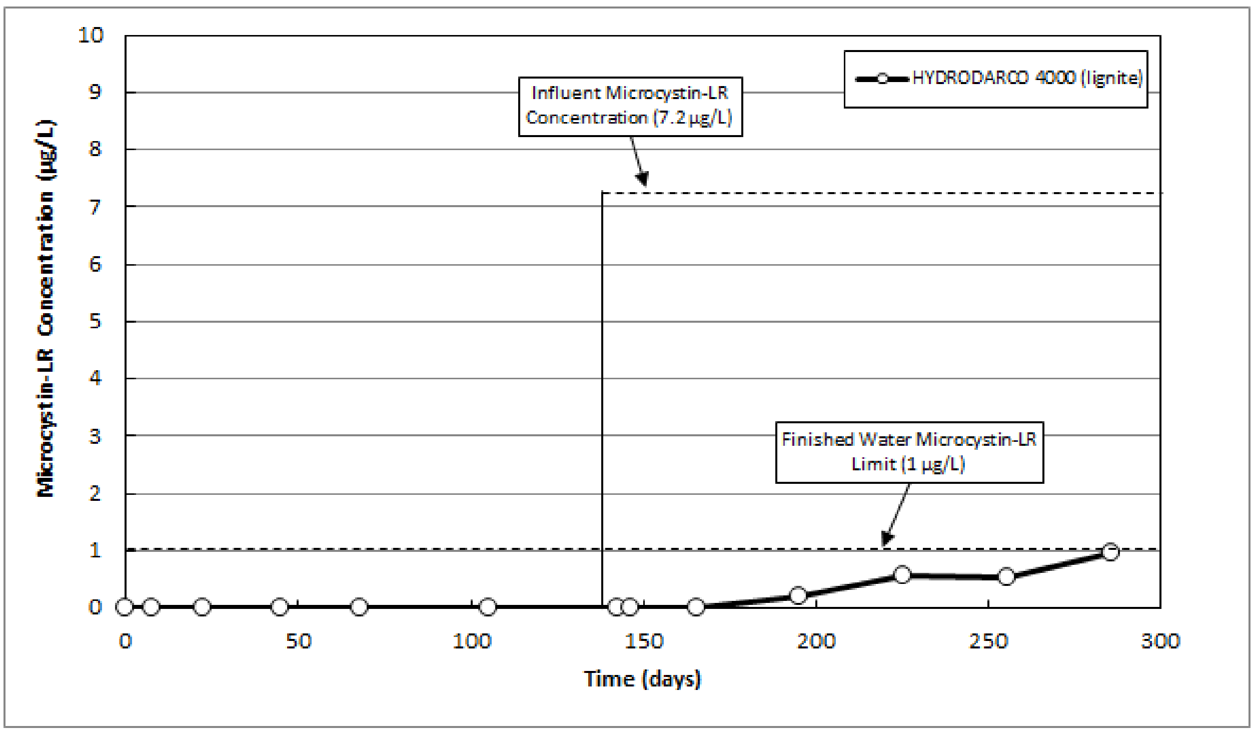

Figure 4-2. Microcystin breakthrough profile for lignite-based GAC following an initial period of NOM preloading (EBCT = 10 minutes; background TOC concentration = 1.4 mg/L)

Figure 4-3 includes both of the microcystin breakthrough profiles developed for lignite-based GAC, and these data show that NOM preloading can substantially shorten the breakthrough time for microcystin. Thus, it would be advantageous to bypass a post-filter GAC treatment system during periods when there is little or no risk of microcystin appearing in the combined filter effluent, so as to preserve the microcystin adsorption capacity of the GAC. Also, these results suggest that installing GAC in the Collins Park filters may not be a viable approach to removing extracellular microcystin, as it would probably require that the GAC be replaced on a yearly basis. Leaving the GAC in the filters for more than a year could result in NOM buildup within the GAC that prevents adequate microcystin removal.

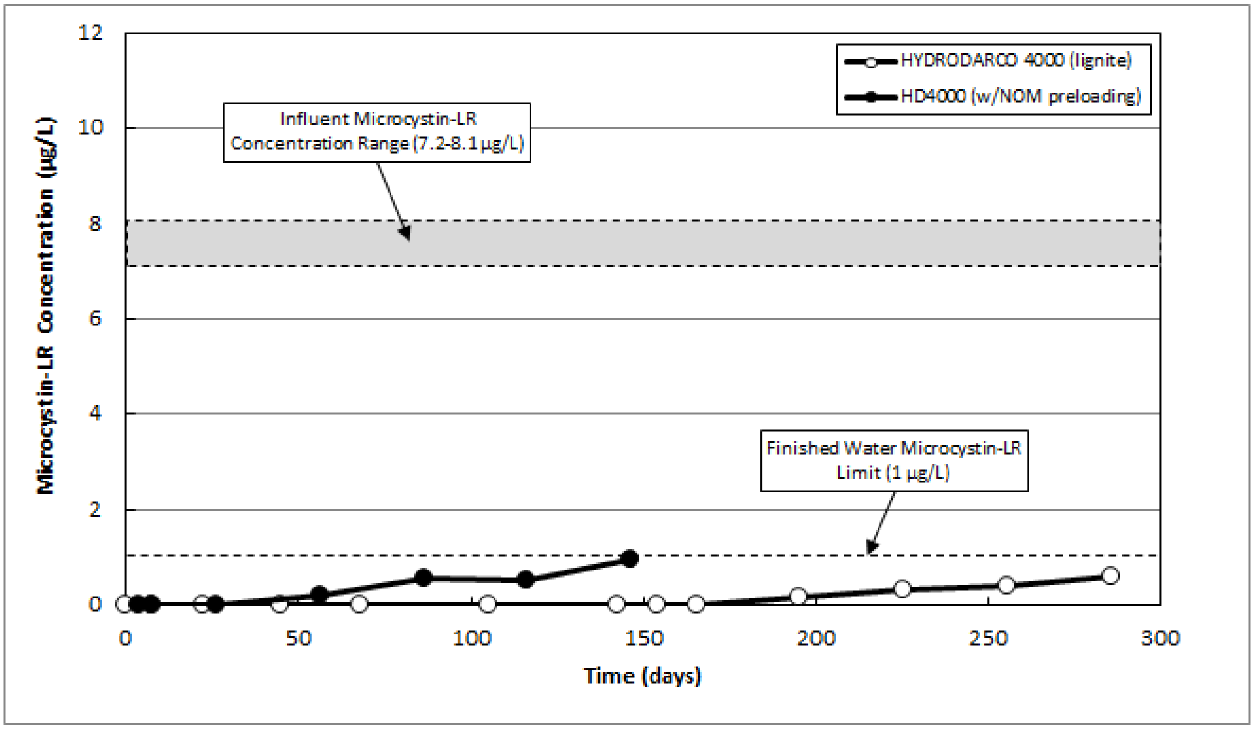

Figure 4-3. Impact of NOM preloading on the microcystin breakthrough time for lignite-based GAC (EBCT = 10 minutes; background TOC concentration = 1.4 mg/L)

5. Preliminary Design/Operating Recommendations

5.1 Empty-Bed Contact Time and GAC Type

As discussed above, a key design criterion for post-filter GAC contactors is EBCT, as it strongly influences the performance of a GAC system and also defines the GAC bed volume requirement. The overall size (cost) of a post-filter GAC system is largely determined by the EBCT. Previous GAC cost analyses have shown that the net present worth of post-filter GAC systems is primarily a function of the capital cost. A larger system with a longer EBCT will generally have a higher net present worth than a smaller system with a shorter EBCT. Thus, the most cost-effective approach to designing a post-filter GAC system is to select the shortest EBCT (smallest system) that allows for reasonable GAC change-out frequencies. Post-filter contactors for potable water applications are normally designed with EBCTs in the range of 10 to 20 minutes. The data gathered during this evaluation indicate that a post-filter GAC system with an EBCT of 10 minutes would serve as a reliable microcystin barrier at the Collins Park WTP, as long as a lignite-based GAC (equivalent to HYDRODARCO 4000) was installed. Such a system would provide adequate protection against extracellular microcystin that could appear in the combined filter effluent during harmful algal blooms.

It is recommended that the post-filter GAC system be designed to provide an EBCT of 10 minutes at the maximum anticipated flow at the Collins Park WTP, which is assumed to be 160 MGD. This way the full-scale GAC system can provide the level of microcystin performance exhibited by the RSSCTs across the full range of flow rates expected at Collins Park. Aiming for an EBCT of less than 10 minutes is not recommended for the following reasons:

- Although the RSSCTs that were conducted as part of this evaluation included an influent microcystin level that could be considered worst-case, the background TOC concentration for these tests (1.4 mg/L) was lower than the levels that are normally observed during the summer (i.e., when microcystin episodes typically occur). As the background TOC concentration goes up, microcystin uptake by activated carbon is likely to become less efficient, due to the effects of competitive adsorption. Thus, full-scale microcystin breakthrough times could be shorter than was predicted by the RSSCTs.

- Microcystin variants that are less adsorbable than mLR (e.g., mLA) may appear in future harmful algal blooms.

- Reducing the EBCT below 10 minutes may limit the usefulness of the post-filter contactors for removing other unwanted algal byproducts such as 2-methylisoborneol and geosmin.

It is strongly recommended that a lignite-based GAC be used in operating a GAC system at the Collins Park WTP. The RSSCTs conducted during this evaluation indicated that lignite-based GAC is far superior to other GAC types for removing microcystin. Wood-based GAC could serve as an alternative to lignite (e.g., if lignite-based is not readily available), as it achieved relatively long breakthrough times. However, using a wood-based GAC would almost certainly increase the GAC replacement frequency. Prior to considering lignite- or wood-based GACs other than those listed in Table 1 (i.e., during the GAC purchasing process), additional RSSCTs should be conducted to confirm that these alternate products can achieve roughly the same microcystin removal performance.

5.2 Filter Adsorbers vs. Post-Filter Contactors

Given that an EBCT of 10 minutes would be suitable in this application, installing GAC in the existing filter basins is a potential option. In cases where longer EBCTs are required, the allowable bed depth in the filters is often insufficient. However, it is likely that an EBCT of 10 minutes could be achieved (at the average, or even maximum flow rates) in the Collins Park filters. Nevertheless, utilizing GAC filters in place of post-filter contactors may not be practical. For one thing, the filters cannot be bypassed, meaning the GAC would continually be adsorbing NOM and thereby losing microcystin adsorption capacity. Thus, the GAC would likely need to be replaced on a yearly basis, so as not to risk microcystin breakthrough. Also, replacing the GAC (media) in all of the filters could be fairly disruptive from an operational standpoint. By comparison, the GAC change-out for post-filter contactors can be conducted during a time when the contactors are offline (e.g., during the winter), meaning the operational impacts would be minimal.

5.3 Partial Stream Treatment

In some cases it is possible to lower the capital cost of a post-filter GAC system by sizing it for less than full flow. In other words, only a portion of the combined filter effluent would pass through the contactors. The GAC-treated water is ultimately blended with the stream that bypasses the GAC system. However, partial stream treatment is not a good option for GAC systems designed to protect against microcystin. This is because the finished water microcystin level must always remain below 1.0 µg/L; and for a system where some portion of the flow is not treated with GAC, there is a maximum influent (combined filter effluent) microcystin level beyond which it is no longer possible to achieve a finished water concentration of 1.0 µg/L. For example, if 50% GAC treatment is provided, it becomes impossible to achieve a finished water microcystin level of 1.0 µg/L if the level in the combined filter effluent exceeds 2.0 µg/L. At 75% GAC treatment, this threshold increases to 4.0 µg/L. Given that the microcystin level in the combined filter effluent could exceed 4.0 µg/L during a future harmful algal bloom, partial stream GAC treatment is not recommended.

5.4 Seasonal Usage

As discussed in the Results section, NOM uptake by the GAC can substantially shorten the breakthrough time for microcystin. This is because the NOM will occupy adsorption sites within the GAC that would otherwise be available for microcystin uptake. Thus, the useful life of the GAC can be extended by taking the contactors offline (i.e., bypassing the GAC system) during periods when no there is little or no risk of a microcystin episode. In all likelihood, the GAC system could be safely bypassed for six months of the year (e.g., November through April).

5.5 GAC Replacement Frequency

The RSSCTs showed that a post-filter GAC contactors containing a lignite-based GAC, and operating with an EBCT of 10 minutes could reduce an influent microcystin level of 8 µg/L down to non-detectable levels for about 165 days and to less than 1.0 µg/L for about 350 days. Given that the microcystin concentration in the combined filter effluent will generally be well below 8 µg/L, even during severe microcystin episodes, and that the EBCT in the contactors will often be more than 10 minutes (since the plant flow will usually be less than 160 MGD), it would be overly conservative to assume that the GAC would require replacement after 165 days (or even

350 days) of operation. Furthermore, if the contactors are bypassed for six months of the year (i.e., from November through April, when there is little or no risk of harmful algal blooms), the useful life of the GAC will be further extended. For the purpose of this evaluation, it was assumed that the GAC change-out frequency would be once every two years. In practice, the need for GAC replacement will be determined by sampling for microcystin within the GAC beds and/or conducting bench-scale adsorptions tests to assess the microcystin adsorption capacity remaining within the GAC.

5.6 Onsite GAC Reactivation

One approach to reducing the cost of GAC replacement is to thermally reactivate the GAC after it becomes exhausted/spent, rather than disposing of it and purchasing virgin (new) carbon. Thermal reactivation involves heating spent GAC in an oxygen-starved environment to temperatures in the range of 800-1000 degrees Celsius, and then exposing the GAC to steam and/or carbon dioxide. Under these conditions, steam and carbon dioxide will oxidize adsorbed organics within the GAC, thus reopening pores and restoring adsorption capacity. Several GAC vendors in the U.S. provide reactivation services, where spent GAC is taken to an offsite reactivation facility (there are facilities located in Columbus, OH and Pittsburgh, PA), and then returned to the utility. For large utilities, where millions of pounds of GAC are in use, it can be cheaper to construct a reactivation facility onsite (i.e., on property owned by the utility), and reactivate the GAC in-house. Onsite reactivation was considered as part of this evaluation; however, the GAC types that are best suited for microcystin removal, namely lignite- and wood-based GACs, are incompatible with existing thermal reactivation protocols. Lignite- and wood-based GACs are highly susceptible to oxidation by steam and carbon dioxide. Meaning, these carbons are likely to be destroyed during a typical thermal reactivation process. It may be that in the future a thermal reactivation process is developed that can effectively reactivate lignite- and wood-based GACs. In the meantime, the best approach to operating a post-filter GAC system at the Collins Park WTP is to purchase lignite-based GAC on an as-needed basis, and dispose of the spent carbon. Vendors that provide lignite-based GAC will normally include the cost of disposal in the overall price for GAC replacement.

5.7 Additional Considerations

Pressure Vessels vs. Concrete Basins – Post-filter GAC contactors can take the form of prefabricated steel pressure vessels or concrete basins (similar to traditional granular media filter basins). However, for a large facility such as the Collins Park WTP, pressure vessels are not a practical alternative. The largest pressure vessels currently available can contain roughly 2260 ft^3 of GAC. The total volume of GAC required to provide a 10-minute EBCT at a flow of 160 MGD is 148,544 ft^3. Thus, about 65 of the largest available vessels would be necessary at the Collins Park WTP. Given that each vessel would include motorized valves, sampling ports, various gauges and other appurtenances, such a large number of vessels would necessitate a tremendous amount of maintenance. A post-filter GAC system comprised of concrete basins would have far fewer “moving parts.” For example, the post-filter GAC system operated by the

Greater Cincinnati Water Works, which was designed to provide an EBCT of 15 minutes at a flow of 175 MGD, includes only 12 concrete basins. Concrete basins are the recommended approach for implementing GAC treatment at the Collins Park WTP.

Biological Microcystin Removal – Post-filter GAC contactors (and GAC filters) inevitably become biologically active. In some cases, the microbial population that develops includes bacteria that can metabolize microcystin. In the study by Newcombe (2002) referenced above, a GAC filter reduced influent mLR and mLA concentrations of approximately 20 g/L down to less than 1 g/L; and this was after exhibiting microcystin breakthrough earlier in the study. These results were attributed to biological degradation of the incoming mLR and mLA. Newcombe (2002) stressed that biological microcystin removal is largely dependent on background water quality, and thus will vary considerably from one location to another. It is unknown whether biological microcystin removal would occur in a GAC system operating at the Collins Park WTP. However, there is a chance that it will occur, and it could reduce the frequency at which the GAC must be replaced. A longer term pilot test (lasting at least several months) and/or a full-scale evaluation would be necessary to fully evaluate the potential for biological microcystin removal.

6. Conceptual Design

The key facilities required for a post-filter GAC system include: a) the GAC contactors and associated building, b) a pump station that serves to convey water from the filters to the contactors, and provides flow for backwashing the GAC, and c) the structures (and equipment) necessary to facilitate GAC replacement. The main design criteria for implementing a post-filter GAC system at the Collins Park WTP are summarized in Table 3. Notably, the existing hydraulic grade line does not allow for gravity flow from the filters to a new GAC system. Therefore, the conceptual design includes an intermediate pump station to convey water from the filters to the GAC contactors. This intermediate pumping will involve collecting filtered water in a wet well. Water from the wet well will be pumped to the necessary elevation in the contactors, after which it will flow by gravity through the GAC and into the existing clearwell. There will be a seal well downstream of the contactors that will prevent them from draining when the system is offline.

Table 6-1. Conceptual Design Criteria for Post-Filter GAC Contactors

Empty Bed Contact Time	10 minutes at 160 MGD (treat 100% of combined filter effluent)
GAC Type	Lignite-based (HYDRODARCO 4000 or equivalent)
Design Flow Rate	160 MGD
GAC Bed Depth	7 ft
Number of Basins	12 (11 in service + 1 spare)
Surface Loading Rate	5 gpm/SF @ 160 MGD
Total Volume of GAC	162,048 ft^3
Total Weight of GAC	3,484,032 lb of lignite-based GAC
Filtered Water Pump Requirements	7 pumps, 250 HP each
Backwash Supply Pump Requirements	2 pumps, 300 HP each
Backwash Supply Rate	8-15 gpm/ft^2
Backwash Duration	Minimum 30 minutes (or until water is clear)
Filter-to-Waste (following backwash)	5 gpm/ft^2 for 30 minutes

6.1 GAC Contactor Configuration

The GAC system would include 12 contactors, sized such that 11 contactors could provide an EBCT of 10 minutes at a flow of 160 MGD (meaning one contactor would serve as a spare). Each GAC contactor would be 30 ft wide and 65 ft long, with a GAC bed depth of 7 ft. Further optimization of the contactor dimensions and GAC bed depth may occur during the detailed design phase.

6.2 GAC Backwashing

With lignite-based GAC installed, the necessary backwash rate will be in the range of 8-15 gpm/ft^2 (depending on water temperature). This backwash rate will expand the GAC bed by at least 20-30 percent. Each backwash is followed by a (roughly) 30-minute contactor-to-waste cycle at a loading rate of 5 gpm/ft^2. The contactors must be backwashed following GAC replacement, to remove fines. Additionally, when the contactors are in service they will likely be backwashed once every 1-2 months, to keep the GAC from becoming overly compacted. For this evaluation it is assumed that the waste streams produced by backwashing the GAC would be directed to the existing residuals handling systems at the Collins Park WTP. Potential impacts to the residuals handling facilities would be further evaluated during the detailed design phase.

6.3 GAC Replacement

As discussed above, use of lignite-based GAC precludes thermal reactivation and requires that spent GAC be disposed of. Installation of fresh GAC and disposal of spent carbon would be carried out by a GAC vendor or a third-party contractor. The tank trailers used to transport GAC can typically carry 40,000 lbs of carbon. It is estimated that approximately 15 trailers would be required to remove the spent GAC from one contactor; and this same number of trailers could deliver the fresh GAC necessary to fill a contactor.

The GAC vendor (or third-party contractor) will provide connections from the carbon trailer to the GAC contactor. Once the connections are in place, the spent GAC in the contactor will be fluidized with water, thus causing it to flow into the trailer (by gravity). Water will be continuously drained from the trailer during the transfer process. Fresh GAC is transferred to the contactor in a similar fashion, with fluidized GAC being pumped from the trailers into the contactor.

6.4 Conceptual Layout

A conceptual site plan for the GAC facilities was developed and is presented in Figure 6-1. Figure 6-2 includes a conceptual layout of the GAC contactors and the filtered water pump station.

7. Conceptual Cost Opinion

7.1 Capital Cost

The capital cost opinion for the implementing a post-filter GAC system at the Collins Park WTP meets the requirements of a Class 5 Estimate, as defined by the Association for Advancement of Cost Engineering (AACE). A Class 5 Estimate is considered a budget-level estimate, with an accuracy of -30% to +50%. This type of estimate is appropriate for a project that is between 0 and 2% defined.

This cost opinion presented herein is based on a combination of historical cost data from similar projects and stochastic (i.e., factors, allowances, $/square foot) methods. Costs were adjusted for inflation as required. All prices were determined in January 2015 dollars, and are escalated to the mid-point of construction. A 30 %contingency was added to the construction cost opinion to account for the inherent inaccuracies in developing cost estimates during the conceptual design phase (i.e., the current phase of this project). Engineering, legal, and administrative costs were estimated as 20 % of the project cost. The entire capital cost opinion is presented in Table 7-1.

Table 7-1. Post-Filter GAC Alternative – Capital Cost Opinion

Component		**Cost Opinion**
Site work & yard piping		$2,810,000
GAC pump station and wetwell		$12,140,000
GAC building and contactors		$28,620,000
Electrical and instrumentation	10%	$4,357,000
General conditions and mobilization	10%	$4,792,700
Subtotal		**$52,719,700**
Contingency	30%	$15,816,000
Subtotal with Contingency		**$68,536,000**
Contractor insurance/bonding	5%	$3,427,000
Contractor overhead/profit/general	15%	$10,280,000
Total Construction Costs		**$82,243,000**
Construction costs inflation to midpoint of construction	3%	$2,467,000
Subtotal		**$84,700,000**
Engineering/Legal/Administrative	20%	$16,940,000
Total Project Cost		**$101,600,000**

The following assumptions and reference were used to develop the opinion of probable cost:

1. All final opinions are rounded to the nearest $100,000.
2. Costs are in January 2015 dollars and escalated to the mid-point of construction.
3. This cost opinion meets the requirements of an AACE Class 5 Cost Estimate, which is a budget-level estimate that is considered to be -30% to +50% accurate.
4. A point estimate, along with low and high estimates are provided. It is recommended that a cost range be used for budgeting.

7.2 Annual Operating and Maintenance Cost

The annual operating and maintenance cost opinion includes:

- Power Costs for major equipment (feed pumps and backwash pumps) – An average power cost of $0.056/KWh was assumed.

- Equipment repair and maintenance – These costs were assumed to be 2% of the capital cost for major equipment.

- Carbon cost – The average annual carbon replacement cost was based on an estimated GAC usage rate of 1.75 million pounds of carbon per year (i.e., replacing all of the GAC once every two years), and a GAC cost $2/lb (Cabot Corporation recently indicated that this is the current unit cost for fresh lignite GAC, including supply, freight, and disposal). Note that the annual replacement cost will vary depending on the actual GAC bed life.

- Labor – Labor costs assume one full-time employee at a burdened rate of $39.50/hr.

The entire operation and maintenance cost opinion is presented in Table 7-2.

Table 7-2. Post-Filter GAC Alternative – Operation and Maintenance Cost Opinion

Element	Annual Cost
Power	$492,000/yr
Carbon	$3,500,000/yr
Labor	$80,000/yr
Repair/Maintenance	$490,000/yr
Total	**$4,562,000/yr**

7.3 Present Worth Evaluation

The 20-year present worth of the GAC alternative is shown in Table 7-3.

Table 7-3. Present Worth Estimate

Capital Cost	$101,600,000
Annual Operating and Maintenance Cost	$4,562,000
Effective Interest Rate	3.00%
20-Year Present Worth	**$169,500,000**

8. References

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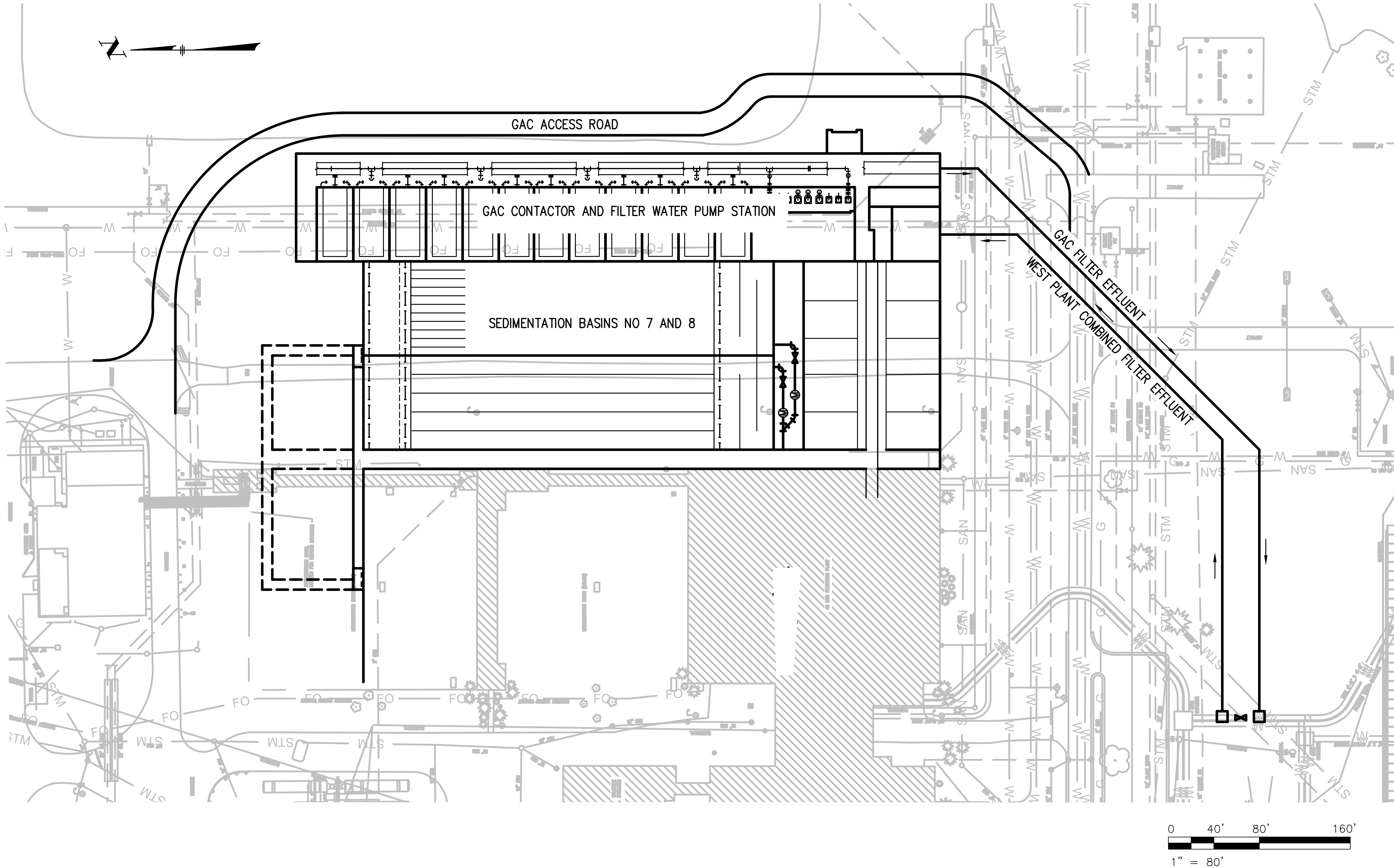

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HAB GAC ALTERNATIVE 2

GAC FACILITY SITE PLAN

FEB 2015

FIGURE 6-1

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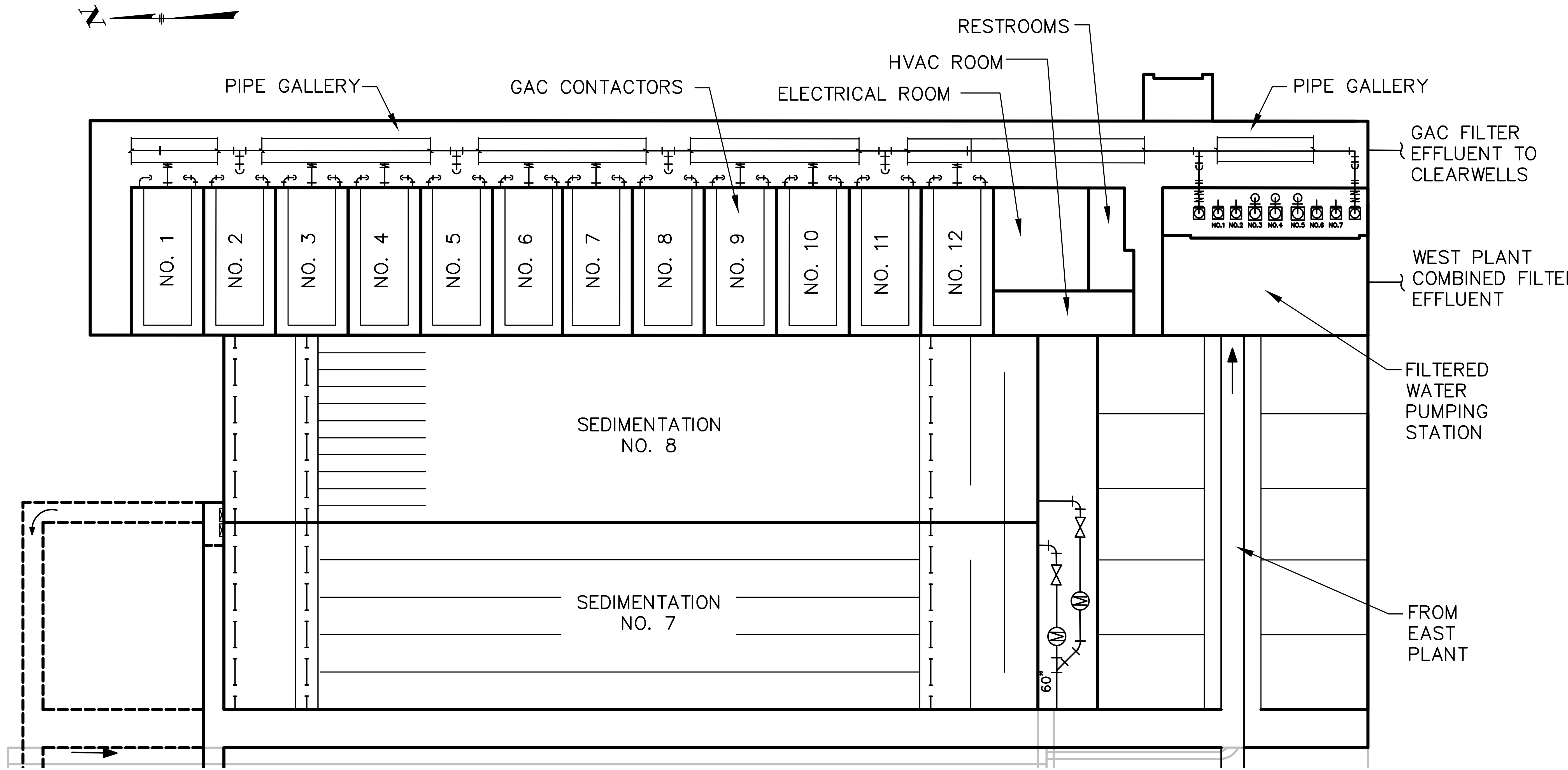

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GAC CONTACTORS AND FILTERED WATER PUMPING STATION PLAN

FEB 2015

FIGURE 6–2

User: CKIRSCH Spec: AUS–NCSMOD File: G:\PROJECT\TO000747.M002\DRAWING\GAC\3 PLAN AT EL 118.50.DWG Scale: 1:1 SavedDate: 2/4/2015 Time: 16:21
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