APPENDIX A

HYDRAULIC TECHNICAL MEMORANDUM

DISINFECTION COST STUDY
HYDRAULIC EVALUATION

FOR

METROPOLITAN WATER RECLAMATION DISTRICT OF GREATER CHICAGO

NORTH SIDE WATER RECLAMATION PLANT

TECHNICAL MEMORANDUM

OCTOBER 23, 2007

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MWRDGC Project No. 07-026-2P
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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1 INTRODUCTION

This technical memorandum has been developed as part of the Preliminary Cost Opinion for Ultraviolet (UV) Disinfection Facilities Study at the Metropolitan Water Reclamation District of Greater Chicago's (MWRDGC, or District) North Side Water Reclamation Plant (NSWRP) in Skokie, Illinois. This memorandum continues the preliminary hydraulic analysis that began in TM1-WQ and the NSWRP Master Plan, which were developed previously as part of the comprehensive Infrastructure and Process Needs Feasibility Study (Feasibility Study) for the NSWRP and a Water Quality (WQ) Strategy for affected Chicago Area Waterways.

The TM1-WQ documented the results of a Consoer Townsend Envirodyne Engineers (CTE) study of effluent disinfection alternatives for the District's North Side, Calumet and Stickney WRPs. Based on economic and non-economic evaluation of alternatives, ozone disinfection and UV disinfection were selected and preliminary basis of design and cost estimates were developed. Both alternatives were developed including three components: a low lift pump station, a tertiary filter facility, and a UV or ozone disinfection facility. The need for tertiary filtration to support disinfection was based on limited sampling that showed transmittance values less than the IEPA minimum of 65% and energy savings with a less turbid flow stream. Because of the limited available information, the estimates that were developed were broken into two alternatives for each disinfection technology: one with tertiary filters and one without tertiary filters. In both cases, a low lift pump station was included based on conceptual level evaluations of the available hydraulic driving head for the existing and proposed conditions.

Subsequent to the TM1-WQ evaluation, additional transmittance data was obtained and the District requested that the costs be further developed without including tertiary filtration. This additional evaluation is also based on the comments received from the United Stated Environmental Protection Agency (USEPA) as part of the Use Attainability Analysis (UAA) evaluations, and new information obtained since the previous work.

1.1 Objective

The primary objectives of the evaluation presented in this technical memorandum are:

- To update the hydraulic evaluation conducted during the preparation of TM-1WQ with subsequent work during the Master Plan that identified the proposed future expansion of the existing NSWRP
- To develop the hydraulic basis of design for further evaluation and development of the conceptual design of UV disinfection facilities
- To determine the need for a low lift pump station with the addition UV disinfection facilities both prior to and after the potential addition of tertiary filters

For the purposes of the Disinfection Cost Study, sound engineering judgment will be used to make assumptions regarding the most likely arrangement of the proposed facilities based on the current status of the future planned improvements to the NSWRP, including the proposed Battery E north of the existing Chicago Transit Authority (CTA) rail.

In the following discussion, the results of this evaluation are given. The sections that follow summarize the determination of the process flow through the proposed improvements including Battery E and the UV Disinfection Facilities, the hydraulic profile through the proposed UV Disinfection System, and the details of the Low Lift Pump Station.

2 PROPOSED FACILITIES

The proposed facilities considered in this study revolve around adding disinfection process facilities to the existing process train and all associated improvements required due to that addition. As such, the improvements will include a disinfection facility/building based on ultraviolet disinfection technology, additional effluent flow conduits, gate structures to redirect flow to the new facilities, and if necessary, a low lift pump station. Tertiary filters will not be included, although the proposed disinfection facilities will be designed to allow the future addition of tertiary filters. The decision to proceed with UV technology for disinfection was made by the District based on several factors including track-record of the technology, need to avoid release of additional chemicals to the environment such as chlorination byproducts, security concerns related to chlorine use and storage, and the cost comparison between the three short-listed disinfection technology alternatives (chlorination/dechlorination, ultraviolet treatment, and ozonzation) performed as part of TM-1WQ. UV technology was shown to be less costly than ozonation with substantially less concern regarding byproducts and security compared to chlorination/dechlorination.

2.1 Key Considerations for Design Development

In order to further develop the design for the UV Disinfection Facilities, CTE has reviewed the basis for the decisions that were incorporated into TM-1WQ in order to confirm the validity of those decisions. This review has identified several issues that must be addressed during the conceptual design of the facilities. These issues include: incorporation of the disinfection facilities into the NSWRP Master Plan for future improvements, the timing of the implementation of the Master Plan in relation to other proposed improvements that might influence the design of the disinfection facilities, and existing hydraulic constraints given the proposed future improvements.

2.1.1 Incorporation into Master Plan

The Master Plan evaluated numerous site alternatives for placement of needed facilities for current and future permit requirements. This evaluation also considered the allocation of space for future low lift pumping, disinfection, and filtration facilities.

The proposed disinfection facilities must fit with other proposed improvements identified as part of the NSWRP Master Plan. In addition to a broad range of proposed improvements to the NSWRP, the Master Plan includes planned improvements as follows that may influence design of the disinfection facilities:

1. Addition of Battery E to expand the existing activated sludge secondary treatment system to be located north of the Chicago Transit Authority (CTA) rail.
2. Modifications to the existing Batteries A through D and the proposed Battery E to accommodate future nutrient removal treatment to address future effluent limits for nitrogen and phosphorous.
3. Addition of tertiary filters to address future effluent limit reductions for suspended solids and biological oxygen demand as well as improving phosphorous removal.
4. Expansion, modification, and other improvements to the existing NSWRP facilities to accommodate future loading and tighter effluent limits that will increase the load on the existing electrical power distribution system.

These improvements create constraints on the design of the disinfection facility due to the need to plan for the allocation of available resources including space on the site, available hydraulic head to transport flow through the facilities, and the logical inclusion of the disinfection process into the existing and future process train to provide the most effective treatment.

Proposed Treatment Train

Disinfection facilities are always located at the farthest downstream point in the process treatment train for the obvious reason that the more treatment the effluent has received to remove both dissolved and suspended contaminants, the more effective the disinfection process. This is true for all disinfection technologies. It is also important to note that the Master Plan proposes to continue the current practice of operating the activated sludge batteries in parallel before recombining the flow prior to discharge. This plan will allow a more efficient approach to tertiary treatment processes, including filtration and disinfection, compared to separate facilities for the each Battery or two facilities, one for the existing site and one for the proposed Battery E site.

One major change from TM-1WQ is the relaxation of the assumed need for tertiary filtration as part of the disinfection facilities. TM-1WQ presented scenarios with and without filtration based on the lack of information to demonstrate that filtration was not required for effective disinfection. For the purposes of this study, it is assumed that tertiary filtration is not required. However, if tertiary filtration is implemented in the future, it would be beneficial for filtration to occur prior to disinfection to leverage the benefits lower suspended solids and BOD concentrations that would make disinfection both more effective and efficient.

The importance of this process flow diagram is highlighted when it is considered in conjunction with the space constraints on the site.

Space

Figure 1 shows the proposed future site plan from the Master Plan. The Master Plan allocated space in the northeast area of the existing site for disinfection and tertiary filtration because of the close proximity to the effluent conduit and outfall. The majority of the space needs are related to future tertiary filtration. The space allocated is based on conventional dual media filtration at 5 gpm/sf. Although other filtration technologies are available with smaller space requirements, it is prudent at this time to assume conventional filtration for planning purposes. As indicated in Figure 1, space is not available at other locations for filtration. Other possible locations for disinfection facilities include the following:

- Between existing primary settling tanks and proposed future Battery F
- At the north site adjacent to Battery E

Neither of these locations offers any potential cost savings because the proximity away from the outfall would require more extensive outside piping.

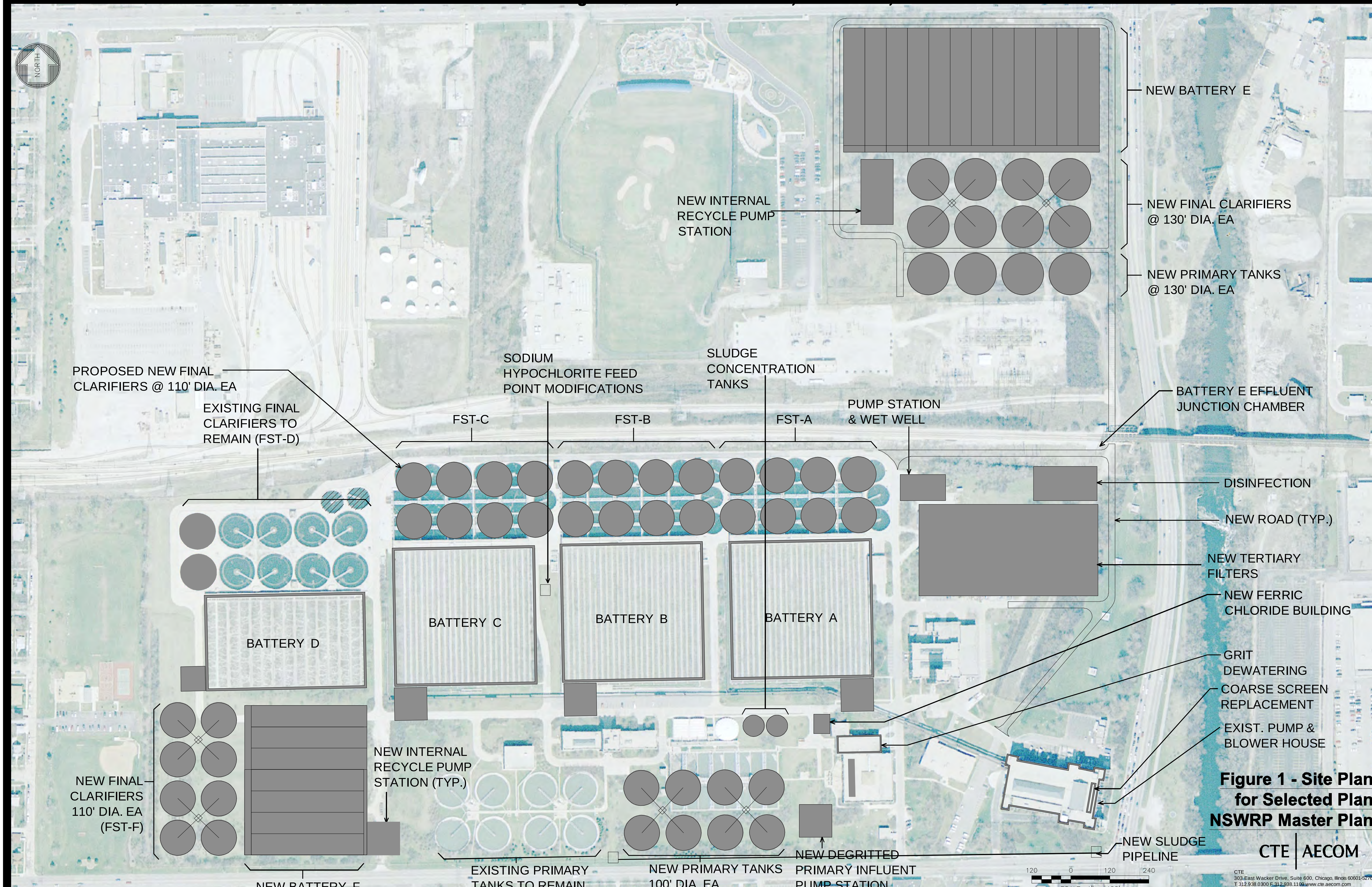


Figure 1 - Site Plan for Selected Plan
NSWRP Master Plan

Based on this review, it is clear that the basis for location and arrangement of the proposed facility is sound. However, it is also clear that the proposed disinfection facilities must accommodate the future addition of tertiary filters, which requires a significant amount of space.

2.1.2 Timing of Implementation

The second key consideration for the design of the disinfection facilities is the timing of implementation in relation to other proposed improvements. Proposed improvements that must be considered include the addition of Battery E and the addition of tertiary filters. Based on the current proposed timeline, Battery E will begin design in the next 3 to 6 months and be online by 2014 to 2016 (40 months for design and three to four years for construction and startup).

The disinfection facilities are not currently assumed to be necessary, but if implemented, it is unlikely that the facilities would be online sooner than 5 years from the date of this memorandum (1 year for planning, 1-2 years for design, and 2 years for construction and startup). It is also possible that the planning period could be extended to allow for a pilot facility or extended water quality sampling. In either case, the UV Disinfection Facilities would not be online prior to 2013.

Currently, there is no projected date for potential implementation of tertiary filters, if ever required. A reasonable assumption would be that nutrient removal is likely to be required in advance of tertiary filtration, which also has no actual implementation schedule. It is therefore conservative to assume that filter implementation would occur after 2020.

Therefore, this study will assume that the proposed disinfection facilities will be implemented in parallel to or after the construction of Battery E. It will also be assumed that tertiary filters will be constructed a minimum of five years after the disinfection facilities, potentially longer. However, the proposed disinfection facilities must be designed so that the tertiary filters can be added in the future.

2.1.3 Hydraulic Constraints/Need for Additional Pumping

The final key consideration for development of the potential disinfection facilities at NSWRP is the hydraulic constraints that may limit the ability to convey flow through the facilities by gravity. Currently, flow through the NSWRP is pumped into the treatment train at the Pump and Blower House at the upstream end of the process treatment train and flows by gravity through the plant and is discharged through the effluent conduit and outfall to the North Shore Channel (NSC). It is most desirable to maintain gravity flow through the plant to reduce capital, energy, operations, and maintenance costs by avoiding additional pumping.

Based on the hydraulic analyses completed as part of the Master Plan, CTE has completed additional hydraulic evaluations to estimate the headloss through the UV Disinfection Facilities including required connecting conduits to evaluate the ability to flow through the proposed Battery E improvements and potential disinfection facilities by gravity. Table 1 presents the results of that evaluation. The basis of this evaluation is discussed in more detail later in this memorandum with the following exceptions:

1. All flow is assumed to be by gravity flow following the Pump and Blower House.
2. The evaluation includes the implementation of Battery F on the existing site. Although Battery F is not scheduled until a later phase, for this evaluation this assumption reduces the total headloss through existing site facilities.
3. The unequal water surface elevation (WSE) between the two sites when the flows are recombined is ignored for the purposes of comparison only. If the system behaved in this manner, the flow through Battery E would be reduced or additional head loss would need to be created through the existing site.
4. The Master Plan recommended that Battery E be operated as a base loaded plant with a constant flow of 105 MGD. Although the design of Battery E will include provisions to convey flows greater than 105 MGD, this hydraulic evaluation will be based on the 105 MGD flow rate.

Table 1 – Theoretical WSE Assuming All Gravity Flow

	Existing Site Batteries A-F	Proposed Battery E
Grit Building Effluent Chamber	25.51	25.51
Battery A Effluent Channel U/S of Disinfection Facility	16.67	--
Battery E Effluent Channel U/S of Disinfection Facility	--	16.78
Effluent Conduit Surge Chamber	12.75	12.86
100-year Flood WSE in Surge Chamber	13.00	13.00

Note: All WSE in Chicago City Datum.

Without tertiary filters, the additional headloss through the UV disinfection facilities including associated flow splitting and control systems is approximately 3.36 feet. As shown by this table, gravity flow through the system would result in a WSE below the 100-year flood elevation. Additional pumping would be required for either flow path after the implementation of the UV disinfection facilities to meet the required peak flow rate of 450 MGD. Considering that this is a conceptual level evaluation, additional headlosses are possible and likely to be identified during final design as the details of flow splitting arrangements and other site constraints create less than ideal flow conditions. At this level, sound engineering judgment dictates that the assumption be that additional headloss will be expected and should be included in the analysis. Thus, it is concluded that additional pumping somewhere in the process train will be required for both flow paths.

In addition, it should be noted that pumping at static heads of less than 3 feet is a difficult application for pump selection and design. See Section 4.2 for a discussion of the pump selection criteria. In order to ensure proper operation of the pumps, additional static head will be added to the system to provide a safety factor to the evaluation and to ensure proper operation of the mechanical equipment.

2.2 Alternatives

With the above conclusion that pumping is required, various alternatives for locating the additional pumping for the disinfection facilities were considered and are described below.

2.2.1 Alternative 1 – Gravity Influent to Battery E with Low Lift Pump Station (LLPS) Upstream of Disinfection for 450 MGD

Alternative 1 is shown in **Figure 2**. In this alternative, 105 MGD will flow by gravity through Battery E, combining with existing plant maximum day flow (345 MGD) upstream of a low lift pump station. The 450 MGD LLPS is to be located upstream of the disinfection facilities. The benefits of this alternative are the inclusion of pumping at only two locations (Pump and Blower House and Low Lift Pump Station) and the ability to easily reroute the pump discharge to the future tertiary filters in the future. The largest disadvantage is the lost available head through the existing site batteries when it must be combined with the Battery E flow (see Table 1) upstream of the proposed low lift pump station.

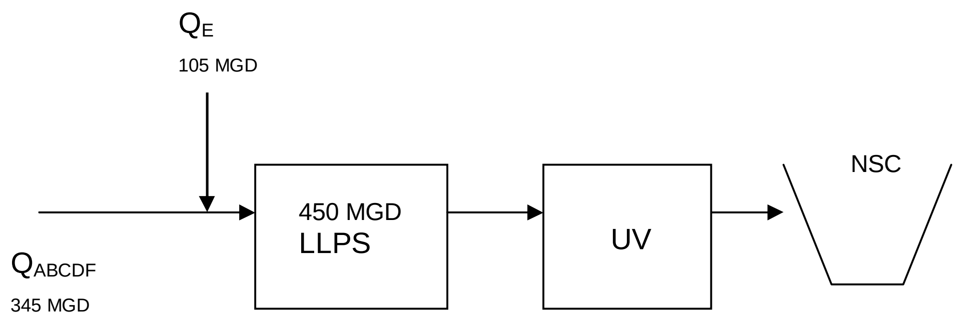


Figure 2 – Alternative 1-Gravity Influent Battery E, 450 MGD LLPS U/S of Disinfection

2.2.2 Alternative 2 – Intermediate Pump Station to Battery E with Low Lift Pump Station Upstream of Disinfection for 345 MGD

Alternative 2 is shown in **Figure 3**. In this alternative, 105 MGD through Battery E is pumped by an intermediate pump station located adjacent to the grit removal facility on the existing site. Existing plant flow (345 MGD) flows by gravity through the existing plant. The existing plant flow enters the LLPS, located upstream of the disinfection facilities. Existing plant flow (345 MGD) and Battery E flow (105 MGD) enter the disinfection facilities through separate conduits. This alternative requires pumping at three locations in the plant (Pump and Blower House, Battery E Influent Pump Station, and Low Lift Pump Station). Furthermore, to add tertiary filters in the future, the Battery E Influent Pump Station would have to be sized with additional head initially, or the Low Lift Pump Station would need to be designed to accommodate future expansion to handle all the plant flow.

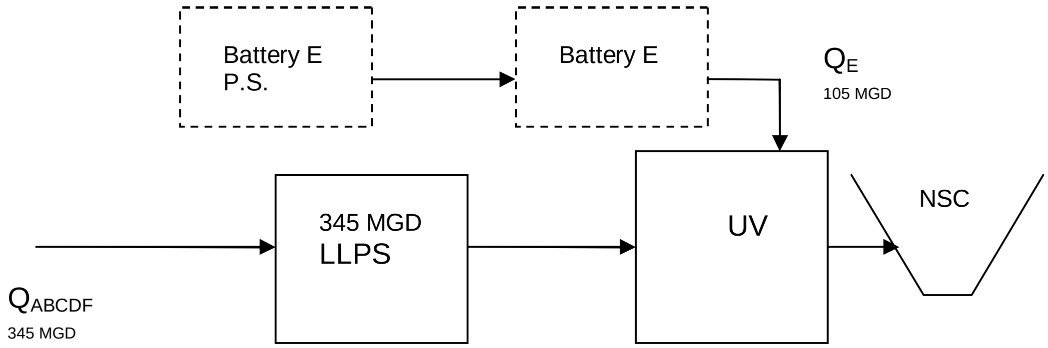


Figure 3 – Alternative 2 Intermediate. P.S. Battery E, LLPS 345 MGD U/S of Disinfection

2.2.3 Alternative 3 – Intermediate Pump Station to Battery E with Intermittent Low Lift Pump Station Downstream of Disinfection for 450 MGD

Alternative 3 is shown in **Figure 4**. In this alternative, 105 MGD through Battery E is pumped by the intermediate pump station located adjacent to the grit removal facility on the existing site. 345 MGD flows by gravity through the disinfection facilities. Total plant flow (450 MGD) is pumped by the LLPS into the North Shore Channel. The LLPS is located downstream of the disinfection facilities. The advantage of this alternative is the ability to use the LLPS only when required by high water levels in the NSC. The disadvantage is the need to replace the LLPS or UV Disinfection Facility when tertiary filters are added in the future.

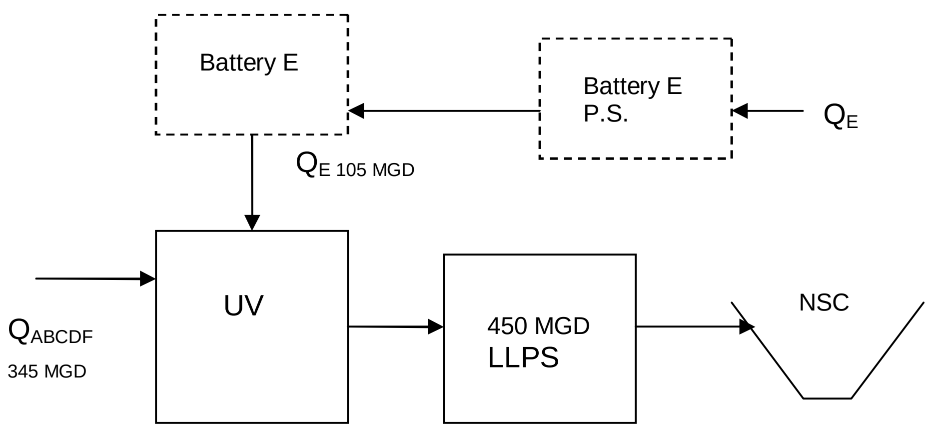


Figure 3 –Alternative 3 Intermediate. P.S. Battery E, LLPS 450 MGD D/S of Disinfection

2.2.4 Recommended Alternative for Disinfection Cost Study

After considering the various alternatives, CTE recommends Alternative 1 for the Disinfection Cost Study. This alternative minimizes the number of pumping facilities required and is the most easily modified to accommodate the future addition of tertiary filters. One of the other alternatives may result in a lower initial capital or operating cost, but is likely to be more costly over the full service life of the facility. For the purposes of this study, Alternative 1 will be used. Future review and more detailed analysis of these alternatives and the Master Plan may result in modifications to this recommendation based on other factors not considered here.

3 HYDRAULIC ANALYSIS OF THE UV DISINFECTION FACILITIES

3.1 Objectives

A preliminary hydraulic analysis was performed during the Master Plan to ensure its hydraulic feasibility. The objective is to identify any possible hydraulic bottlenecks for the recommended site plan indicating where detailed analysis will be required during the design phase. A hydraulic analysis was performed on the existing NSWRP in the Current Capacity and Future Treatment Evaluation Technical Memorandum, TM-5. For this study, modifications were made to this model in order to account for the addition of the UV Disinfection Facilities inclusive of the required additional effluent conduits, gate structures, UV channels and reactors, and Low Lift Pump Station.

3.2 Overview

The hydraulic analysis was completed using a spreadsheet utilizing standard open channel and closed conduit flow equations to represent the NSWRP. The hydraulics evaluated were for the year 2040 conditions, which include both infrastructure and permit-related improvements. A peak flow of 450 mgd was used. Flow in excess of 450 mgd is diverted to the TARP system. Return activated sludge flows were added to the influent where appropriate. In order to reflect the nutrient removal processes, internal mixed liquor recycled flows were used in the hydraulic analysis of the activated sludge aeration tanks.

Similar to the analysis performed in TM-5, critical flow paths were identified as those which would result in the greatest headloss. These critical flow paths were modeled from the North Shore Channel Outfall to immediately upstream of the coarse bar screens in the Pump and Blower House. The two flow paths identified as critical flow paths for this study are as follows:

1. Critical flow path through Battery A
2. Critical flow path through Battery E

3.3 Assumptions

Due to the preliminary nature of the selected site plan, assumptions were made in the development of the hydraulic model. These assumptions are as follows:

1. All NSWRP drawings obtained from MWRDGC are on the same datum, known as the Chicago City Datum (CCD).
2. The CCD has not changed since the plant was originally constructed in the 1920's.

3. Flow through Battery E is 105 MGD and it is treated as a base loaded plant. Flow through Batteries A, B, C, D, and F is the remainder and will be 345 MGD at peak flow. Flow over 450 MGD is diverted to TARP.
4. Return flow from the Grit Dewatering System and Scum Concentration Tanks as well as supernatant from the Sludge Concentration Tanks are negligible.
5. Flow reduction as a result of primary sludge removal is negligible.
6. The 100-year flood elevation is 12.30 CCD, as calculated in the Chicago Canal System Model, UNET. Appendix A provides selected pages from the USACE's Chicago Underflow Plan (CUP) Design Report presenting these results. Pre-Stage 1 (Stage 1 of McCook Reservoir Construction) values are used since the USACE's current estimate for completion of Stage 1 construction is 2020 or later.
7. Hydraulics through the existing Meter Building will control flow splits among Battery A, B, C, D, and F proportional to the battery volumes.
8. Flow splits evenly based on aeration tank volume within each battery.
9. Flow splits evenly among the aerated grit channels located in the Grit Building.
10. Return Activated Sludge (RAS) flows were calculated to be 55% of total influent flow.
11. Internal recycle flow for total nitrogen removal was calculated to be 150% of total influent flow per battery.
12. Baffle walls (for TN removal) were assumed to be mounted where mixed liquor flows from underneath one baffle wall to the top of the next baffle wall, creating a "up and down" flow pattern.
13. The longest flow path through each treatment process was used.
14. Tank geometry downstream of the aeration tank effluent weirs (Operating Gallery and Final Settling Tanks) in Battery A was assumed to be similar to that of existing Battery D.
15. Geometry of Batteries E and F were assumed to be similar to that of existing Battery D.
16. Proposed primary settling tank geometry was assumed to be similar to that of the existing circular primary settling tanks.
17. Velocity in Disinfection Influent and Effluent Distribution Chamber is zero
18. Battery E is to be pumped via the proposed low-lift pump station on the existing (southern) NSWRP site.
19. Battery E is gravity Fed from downstream of the Grit Building.
20. Disinfection channel effluent weir gate is assumed to be downstream WSE (WSE 4) + 0.5'
21. The following modeling equations were used:
    a. Pressure Flow – Hazen Williams Equation
    b. Open-Channel Flow – Manning's Equation
    c. Flow junctions – Pressure Momentum Analysis
22. Hydraulic coefficients used in developing this model include:
    a. Hazen Williams – 110 (concrete)
    b. Manning's
        i. Regular channel – 0.013
        ii. Aerated channel – 0.035

3.4 Results

Results are presented below. Tertiary filters are excluded from the hydraulic profile. The hydraulic profiles show the estimated WSEs at the maximum flow of 450 mgd. Flow that exceeds 450 mgd is diverted into the TARP system. **Table 2** presents the headlosses through various portions of the plant for Battery A and Battery E for comparison.

Table 2 – Summary of Headloss through NSWRP (Proposed)

Process/Flow Area	Battery A	Battery E
Pump and Blower House Discharge to Aerated Grit Discharge Chamber	2.03	2.03
Aerated Grit Discharge Chamber to PSTs	1.03	2.39
Primary Settling Tanks	1.83	2.44
Aeration Basins and Final Settling Tanks	5.98	2.72
Effluent Conduit to Low Lift Pump Station Wet Well	0.67	1.96
LLPS Discharge to UV Disinfection Effluent Chamber	3.36	3.36
UV Disinfection Effluent Chamber to Outfall	.66	.66
Total	15.56	15.56

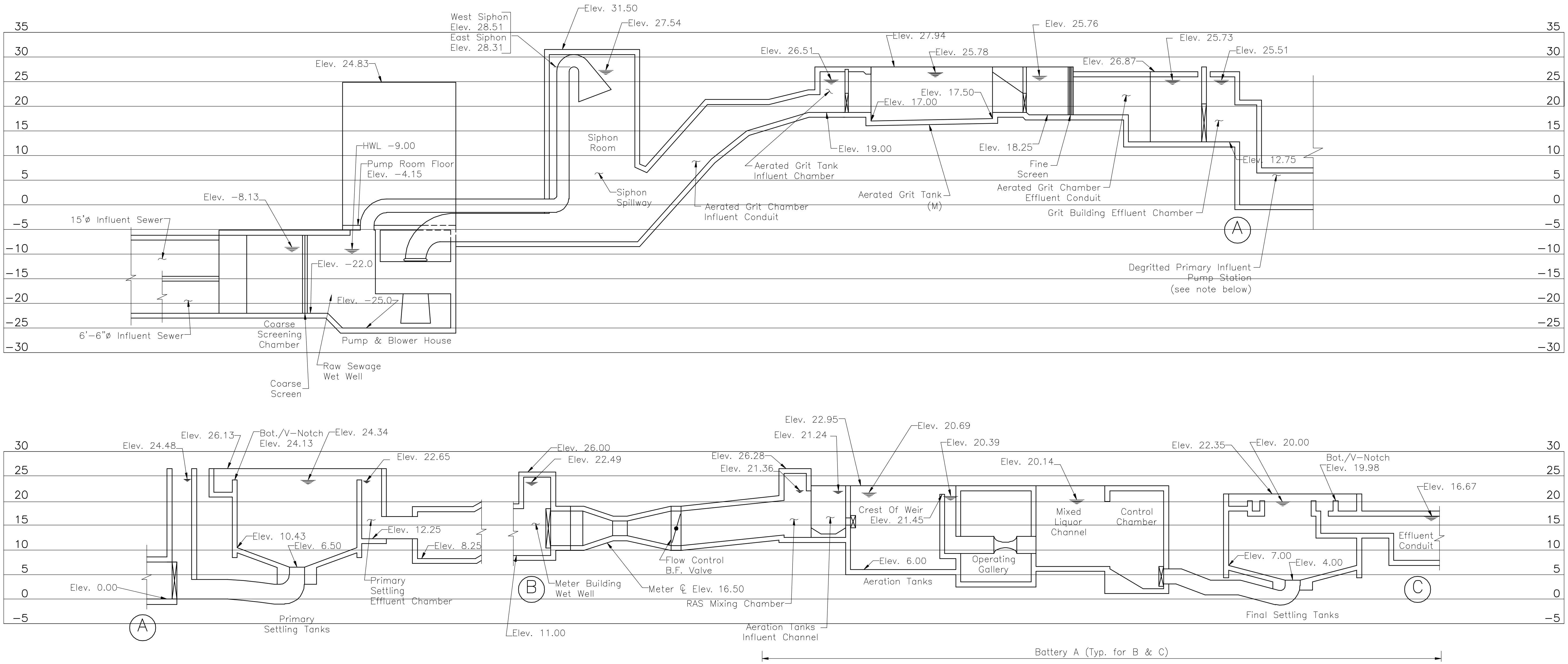
Notes: Values in feet of headloss.
Does not include head dissipated due to minimum pump head requirements.

Table 3 presents the final water surface elevations through the plant including the Low Lift Pump Station and UV Disinfection Building.

Table 3 – Summary of Proposed WSE including UV Disinfection Facilities

Location	Combined	Battery A	Battery E
North Shore Channel 100-yr Flood Elevation	12.30	--	--
D/S WSE @ New Surge Chamber	12.96	--	--
U/S WSE @ New Surge Chamber	15.96	--	--
WSE @ Disinfection Effl Channel	16.52	--	--
WSE just U/S of Weir Gate	18.03	--	--
WSE just D/S UV Reactor	18.08	--	--
WSE just U/S UV Reactor	18.83	--	--
WSE just D/S of influent gate	18.87	--	--
WSE in LLPS Discharge Channel	19.88	--	--
LLPS Wet Well	16.00	--	--
Final Settling Tank Effluent Chambers	--	16.67	17.96
Aeration Tank Effluent Chambers	--	20.39	18.88
Aeration Tanks	--	20.69	19.62
Primary Tank Effluent Chambers	--	22.65	20.68
Grit Building Effluent Chamber	25.51	--	--
U/S of Fine Screens	25.76	--	--
Aerated Grit Tank Influent Chamber	26.51	--	--
Siphon Room	27.54	--	--

Figure 5a and 5b contain hydraulic profiles of the two critical flow paths with the UV disinfection facilities and the available freeboard at the locations where water surface elevations (WSEs) were calculated at the maximum day flow.



Legend

- (A) – Flow diversion from Headworks to Batteries on Existing Site and North Site Battery E.
- (B) – Flow split at Meter Building Wet Well to Batteries A & F
- (C) – Flow junction from Batteries A, E, & F

FIGURE 5A
HYDRAULIC PROFILE FOR BATTERIES A–F
AFTER IMPLEMENTATION OF UV DISINFECTION FACILITIES

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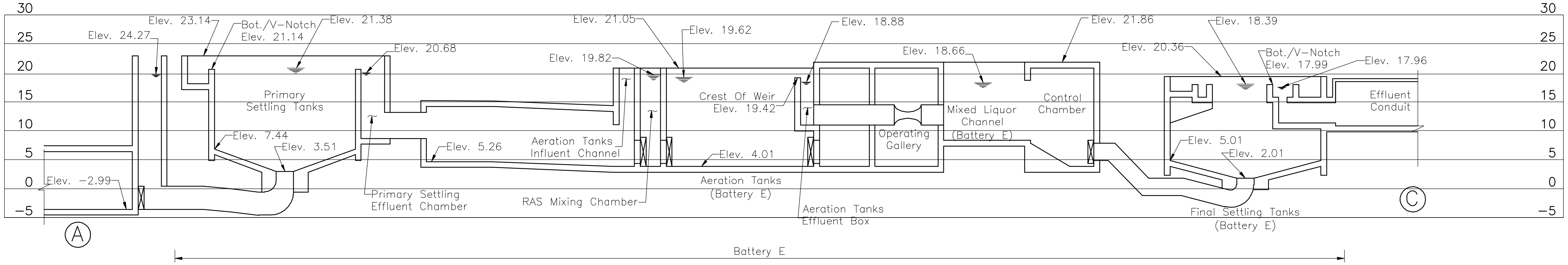

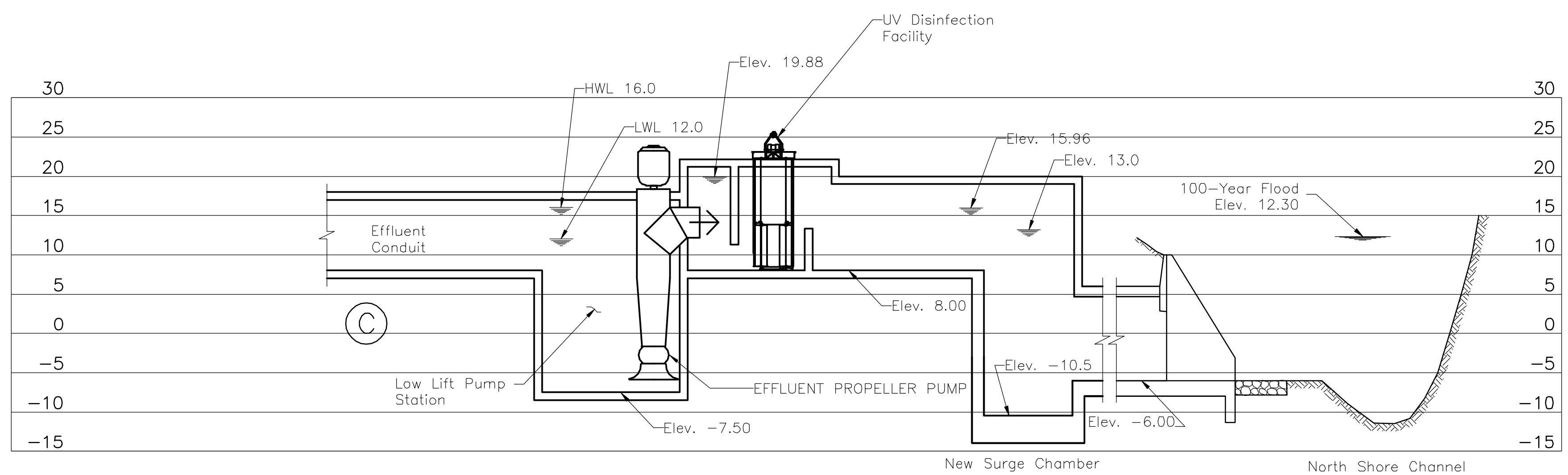

Legend

(A) – Flow diversion from Headworks to Battery E

(C) – Flow junction from Battries A, E, & F.

FIGURE 5B
HYDRAULIC PROFILE FOR BATTERIES E
AFTER IMPLEMENTATION OF UV DISINFECTION FACILITIES

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4 LOW LIFT PUMP STATION

This section will present the proposed arrangement and key characteristics of the proposed Low Lift Pump Station.

Based on the above analysis of hydraulics, it is estimated that the low lift pumps will raise the water approximately 7 feet (including static and friction losses) to the UV disinfection system influent, including estimated head to allow flow through the UV system. Should tertiary filtration become necessary in the future, these pumps can be modified to enable an increased head of approximately 11 feet.

Pumps will be axial flow, propeller type. The pumps will operate 24 hours a day, seven days per week. The level control will be automatic under normal conditions, with manual override possible.

4.1 Basis of Design

Table 4 provides a summary of the basis of design for the Low Lift Pump Station.

Table 4 – Low Lift Pump Station Basis of Design

Flow, MGD	450
Pumps	
Type	Axial Flow
Number	6 total (N+1+1)
Pumping Rates, gpm/pump	78,125
Total Dynamic Head, ft.	7
Motor, hp	250
Submergence, ft	16
Wet Well	
Length, ft.	86
Width, ft.	101

4.2 Pump Type

Several pump types were considered for this high flow (78,125 gpm) low head (7 feet TDH) application. Pump types considered included screw pumps, vertical turbine pumps, centrifugal pumps, and axial flow pumps. Many pump manufacturers found it difficult to recommend a pump that would operate efficiently for this application due primarily to the low head. Screw pumps and axial flow pumps appear to have the best operating performance for this condition.

Initially the Low Lift Pump Station will lift 450 MGD a total of 4 feet with a Total Dynamic Head (including station losses) of approximately 7 feet. However, if tertiary filtration is constructed in the future, the TDH will increase to approximately 11 feet (flow will remain the same). Screw pumps will not easily accommodate this change in head, without significant structural modifications to the pump station. However, axial pumps can be modified for future head conditions. Structural modifications to the pump station to accommodate these changes, if required, should be minimal. Therefore, axial flow, propeller type pumps are recommended.

4.3 Proposed Operational Description

The pump station will have a total of six pumps, with four duty pumps, one standby and one out of service (N+1+1). Four pumps will be driven by constant speed motors, two will be variable speed driven. In order to provide operational flexibility, the pump station will be divided into two wet wells, each containing three pumps. Design average flow (333 MGD) can be handled by two constant speed and one variable speed pumps, leaving three pumps on standby. Peak flow (450 MGD) can be handled by four pumps, leaving two on standby. Typically, at least one variable speed pump will operate at all times, to handle fluctuations in flow. **Table 5** illustrates an example of pump operation at design average flow and peak flow:

Table 5 – Summary of Pump Operation

Flow, MGD	Pump Drive Type	Pump Flow, gpm
250	Constant speed	78,125
	Constant speed	78,125
	Variable speed	46,875
333 (Design Average)	Constant speed	78,125
	Constant speed	78,125
	Variable speed	75,000
450 (Peak)	Constant speed	78,125
	Constant speed	78,125
	Constant speed	78,125
	Variable speed	78,125

In order to eliminate vortices, pumps require a minimum submergence as a function of pump suction bell diameter. For this flow condition, a 96-inch suction bell is required, which requires a minimum submergence of 168 inches, or 14 feet. Submergence requirements should be verified by the pump manufacturer during final design.

Level sensors in the wet well will relay a signal to turn pumps on and off. Other control inputs that need to be monitored include discharge pipe pressure, flap gate position, and motor alarms.

4.4 Proposed Layout

Flow will enter the pump station at the north end of the wet well, where it will be directed perpendicularly to the south through four 96-inch slide gates. Pumps are located at the south end of the pump station. Site constraints and pump station size appear to make this flow pattern necessary.

Available area on the site is insufficient for meeting Hydraulic Institute (HI) Standards directly. A trench type wet well was considered in order to meet HI standards, but its depth, in excess of fifty feet, precluded further study.

A rectangular wet well is shown in the plan and section. Design features, which have been shown to be effective in other installations, were incorporated in this design in order to meet HI standards. For example, perforated plates, curtain walls, and floor and back wall splitters have been incorporated into the conceptual design. (See Appendix B for a plan and section of the proposed layout). Sizing and details of these types of features are normally determined by physical scale modeling during detailed design.

Furthermore, based on the total flow and flow per pump, the Hydraulic Institute recommends physical scale modeling.

5 SUMMARY

This technical memorandum has been developed as part of the Preliminary Cost Opinion for Ultraviolet (UV) Disinfection Facilities Study at the Metropolitan Water Reclamation District of Greater Chicago's (MWRDGC, or District) North Side Water Reclamation Plant (NSWRP) in Skokie, Illinois. The study is advancing the previous work outlined in the NSWRP Master Plan and TM1-WQ based on the comments received from the United Stated Environmental Protection Agency (USEPA) as part of the Use Attainability Analysis (UAA) evaluations and new information obtained since the previous work.

CTE's efforts to date have identified several issues that must be addressed during the conceptual design of the disinfection facilities. These issues include: incorporation of the disinfection facilities into the NSWRP Master Plan for future improvements, the timing of the implementation of proposed improvements that might influence the design of the disinfection facilities, and existing hydraulic constraints given the needs of the proposed future improvements.

Through the work completed during the Master Plan, it has been determined that the disinfection facilities will be located in the northeast corner of the existing site due to the proximity to the existing outfall and effluent conduit as well as space needs for construction of other required future facilities (i.e. Battery E) at other available locations. The proposed disinfection facilities are assumed to be constructed after Battery E is online, but before the addition of tertiary filtration. The anticipated time frame for startup of the disinfection facilities is 2014 to 2016 for the purposes of the Disinfection Cost Study. This schedule should be considered conservative in the sense that the implementation schedule may be longer than assumed here due the complexity of the required planning and design efforts for facilities of this magnitude and the potential for delay due to the uncertainty inherent to the regulatory process.

Using the hydraulic analysis work completed for the NSWRP Master Plan, a preliminary evaluation of the hydraulic profile for the proposed facilities was completed assuming that all flow continued to be by gravity downstream of the influent Pump and Blower House. This evaluation shows that water surface elevation at peak flow at the surge chamber is below the 100-year flood elevation and therefore, the plant would not be capable of treating the peak design flow. Considering that this is a conceptual level study and additional losses are likely to be identified during final design, it is concluded that additional pumping for all flows from the existing site (Batteries A, B, C, D, and F) and from Battery E is required in order to convey and treat peak flows.

Several alternatives were considered regarding the layout and location of the pumping on the site. The recommended alternative is to provide a single low lift pump station downstream of the final clarifiers for all secondary treatment batteries but upstream of the disinfection facility. This arrangement minimizes the number of times that the flow is pumped and the number of locations of the major pumping equipment. It will also permit bypassing of the LLPS and disinfection facility during winter months when disinfection is not required. In addition, this alternative more easily allows diversion of the effluent to a tertiary filter facility in the future.

The hydraulic analysis was refined based on the proposed layout of the facilities to determine the specific needs for the LLPS. A proposed layout of the LLPS has been developed based on axial flow pumps. Axial flow pumps are recommended due to the low head conditions and the need to modify the discharge head when tertiary filters are added in the future. The primary alternative to axial flow pumps is screw pumps, but this pump type is not easily modified after installation to provide additional head.

The wet well layout, shown in Appendix B, is constrained by the available space and does not meet ideal Hydraulic Institute pump intake standards. However, pump intake flow improving features were incorporated into the layout of the wet well similar to other pump stations of similar size and application. Physical scale modeling during detailed design is strongly recommended due to the size of the pumps and to verify and size the hydraulic improvements.

In conclusion, this review has confirmed the primary assumptions of the NSWRP Master Plan in regards to the need for a low lift pump station, location of the facilities, and arrangement of the facilities to accommodate future facilities. The Disinfection Cost Study will proceed based on the assumption and the additional details provided in this report.

APPENDIX A
Selected Pages from USACE CUP DDR

US Army Corps
of Engineers®
CHICAGO DISTRICT

DESIGN DOCUMENTATION REPORT

CHICAGOLAND UNDERFLOW PLAN
McCOOK RESERVOIR, ILLINOIS

Volume I of VIII

NOVEMBER 1999

Table A-11. Canal System Observed and Modeled Maximum Water Surface Elevations

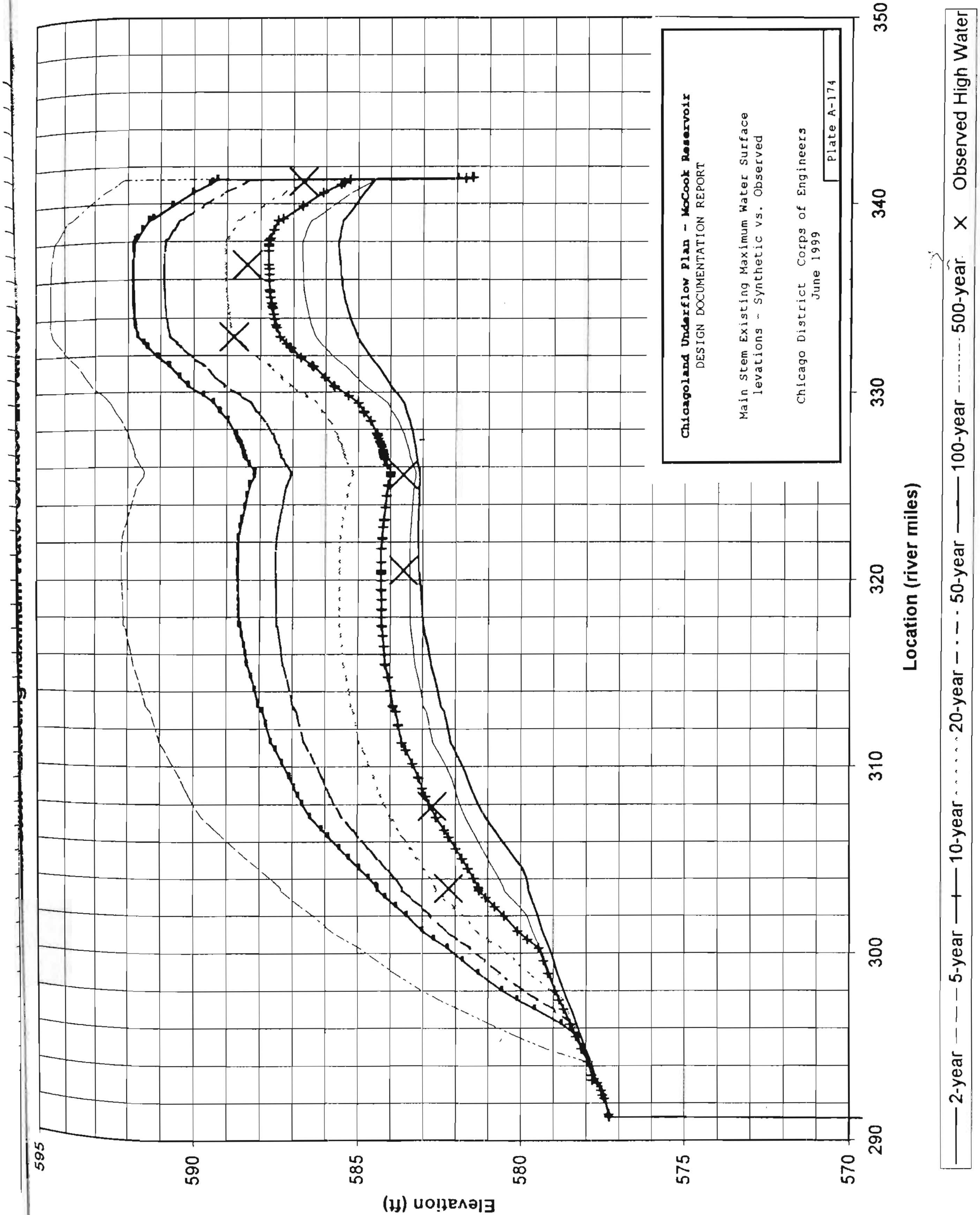
Location	Approx. River Mile	Maximum Water Surface Elevation (ft NGVD)						
			Modeled for Water Years 1951-1988			*Modeled 1% Chance Exceedance Event*		
		Observed, 1965 to present (Date)	Existing (Date)	Stage 1 Project (Date)	Stage 2 Project (Date)	Existing	Stage 1 Project	Stage 2 Project
Wilmette - NSC @ Sheridan Rd.	341.2	586.7 (4/18/75)	592.6 (7/57)	591.3 (7/57)	590.5 (7/57)	589.4	589.1	587.6
North Side SW - NSC @ Howard St.	336.8	588.4 (8/14/87)	594.9 (7/57)	593.1 (7/57)	592.6 (7/57)	591.8	590.9	589.5
North Branch PS - NSC @ Lawrence St.	333.0	588.8 (8/16/97)	594.6 (7/57)	592.2 (7/57)	592.2 (7/57)	591.7	589.8	588.4
Chicago River Controlling Works - Chicago River @ Lk Michigan*	325.6	583.6 (8/16/97)	589.1 (7/57)	585.3 (10/54)	583.9 (10/54)	588.2	585.0	583.2
31st & Western - CS&SC @ Willow Springs Rd.	320.5	583.6 (6/30/77)	589.6 (7/57)	585.4 (10/54)	583.9 (10/54)	588.7	585.1	583.0
Willow Springs - CS&SC @ Willow Springs Rd.	307.9	582.7 (7/18/96)	587.2 (7/57)	584.0 (10/54)	583.0 (10/54)	586.7	584.1	582.4
Sag Junction - Confluence of CS&SC and CSC	304.2	582.2 (7/18/96)	585.0 (7/57)	582.6 (10/54)	581.9 (10/54)	584.7	582.8	581.6
O'Brien Lock - Calumet River Downstream (south) of O'Brien Lock	325.8	583.8 (7/18/96)	585.0 (7/57)	584.6 (7/57)	584.6 (7/57)	584.7	584.0	583.8
Southwest Highway - CSC @ Southwest Hwy	310.8	583.7 (7/18/96)	585.0 (7/57)	584.3 (10/54)	584.3 (10/54)	585.0	583.5	583.1

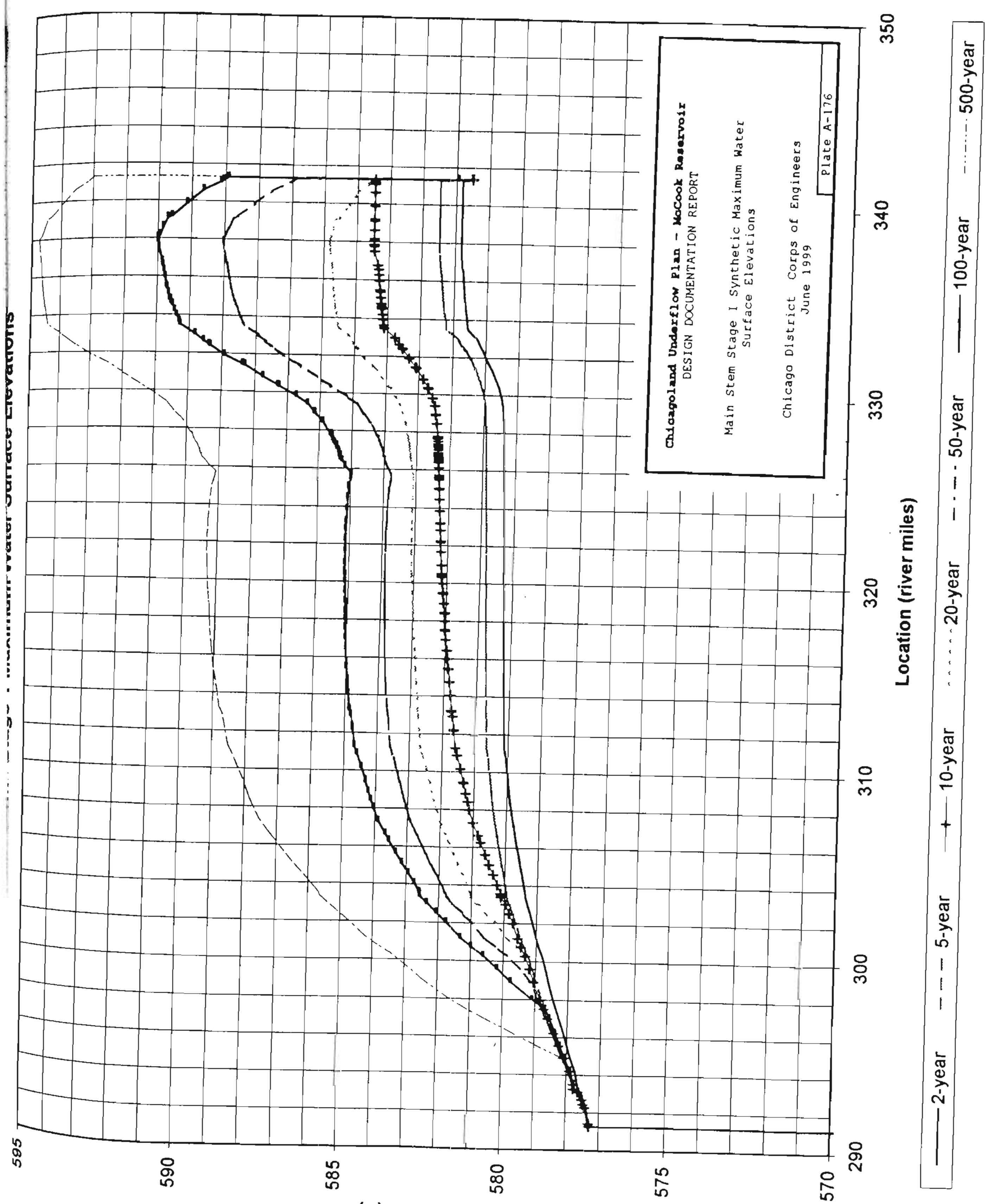
*The approximated river mile is for the junction of the Chicago River and its North and South Branch.

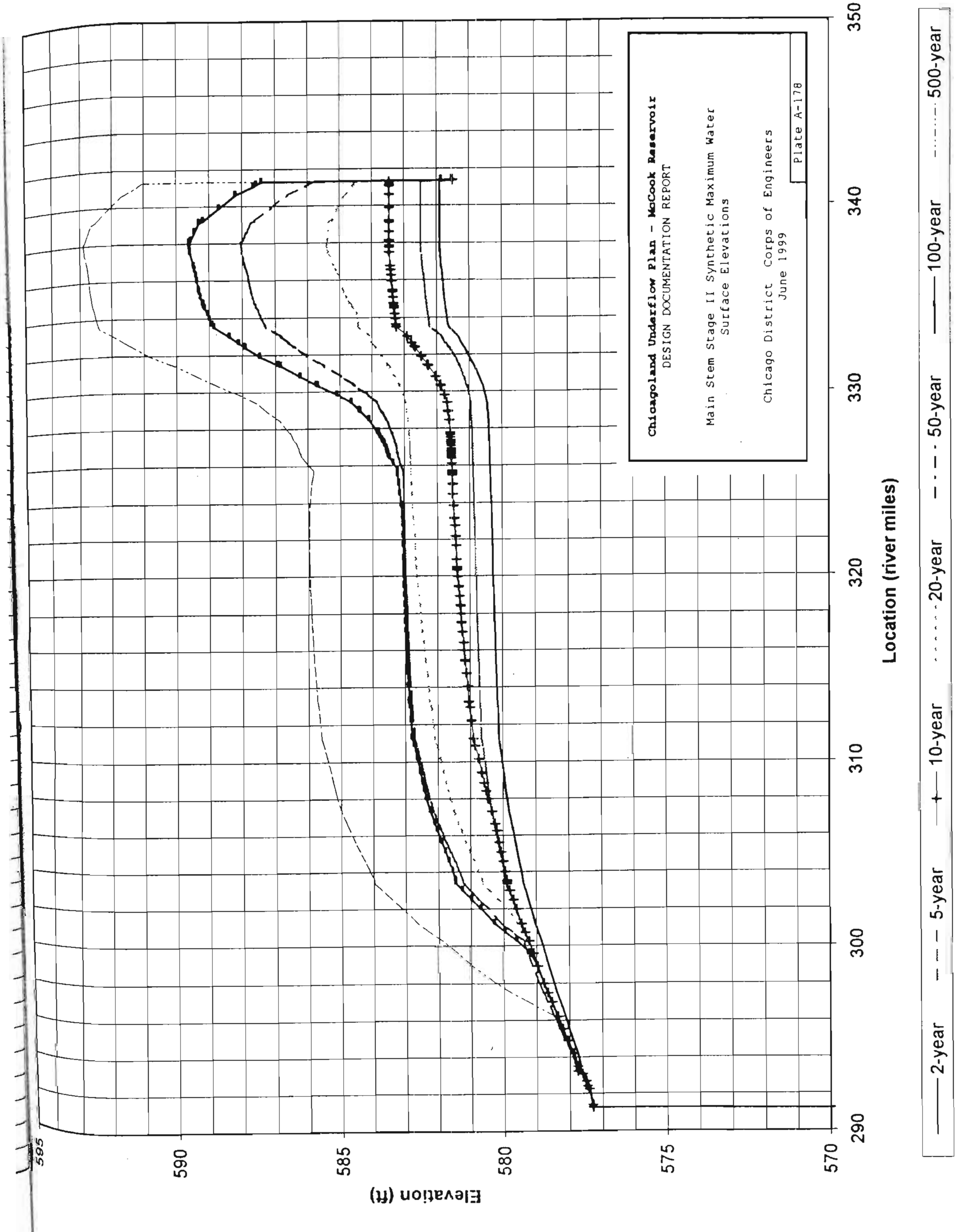
NSC = North Shore Channel
CS&SC = Chicago Sanitary and Ship Canal
CSC = Calumet Sag Channel

Table A-12. Index of Major Bridges and Confluences for Chicago Canal Model

Reach Scheme (Canal Model)	Tributary Stream	Bridge Name	River Mile
2	North Shore Channel	Sheridan Road Lock	341.2 1/
2	"	Central Street	340.4
2	"	Green Bay Road	339.8
2	"	Church Street	338.7
2	"	Dempster, Il 58	338.2
2	"	Oakton Street	337.2
2	"	Touhy Avenue	336.2
2	"	Devon Avenue	335.2
2	"	Peterson, US 14	334.7
2	"	Foster Avenue	333.6
2	"	Jct. North Branch	333.5
1	North Branch	Touhy	51.4 2/
1		(05536000 gage)	
1	"	Devon Avenue	49.2
1	"	Edens Expwy.	46.2
1	"	Cicero Avenue	46.1
1	"	Foster Avenue	44.5
1	"	Kimball Avenue	43.9
1	"	Kedzie Avenue	43.6
1	"	Jct. North Shore Channel	43.3
3	"	Jct. North Shore Channel	333.5
3	"	Lawrence Ave.	333.1
3	"	Montrose Ave.	332.5
3	"	Irving Park Rd.	332.0
3	"	Addison Street	331.4
3	"	Belmont Ave.	330.9
3	"	Western Ave.	330.6
3	"	Diversy Ave.	330.2
3	"	Damen Ave.	329.9
3	"	Fullerton Ave.	329.5
3	"	Ashland Ave.	329.1
3	"	Cortland Street	328.6
3	"	North Ave.	327.9
4	North Br. (Goose Island West)	Division Street	327.4
4	"	Ogden Ave.	326.9
4	"	Halsted Street	326.6
5	North Br. (Goose Island East)	Division Street	327.0
5	"	Ogden Ave.	326.9
5	"	Halsted Street	326.85
6	North Branch	Chicago Ave.	326.4
6	"	Ohio/Kennedy Expwy.	326.1
6	"	Grand Ave.	326.0
6	"	Kinzie Street	325.8
6	"	Jct. South Branch	325.6
7	Chicago River	Franklin Street	325.65
7	"	Wells Street	325.7
7	"	LaSalle Street	325.8
7	"	Clark Street	325.9
7	"	Dearborn Street	326.0
7	"	State Street	326.1
7	"	Wabash Ave.	326.3
7	"	Michigan Ave.	326.4
7	"	Lake Shore Drive	326.9
8	South Branch	Lake Street	325.6
8	"	Randolph Street	325.5
8	"	Washington Street	325.4
8	"	Madison Street	325.3
8	"	Monroe Street	325.1
8	"	Adams Street	325.0

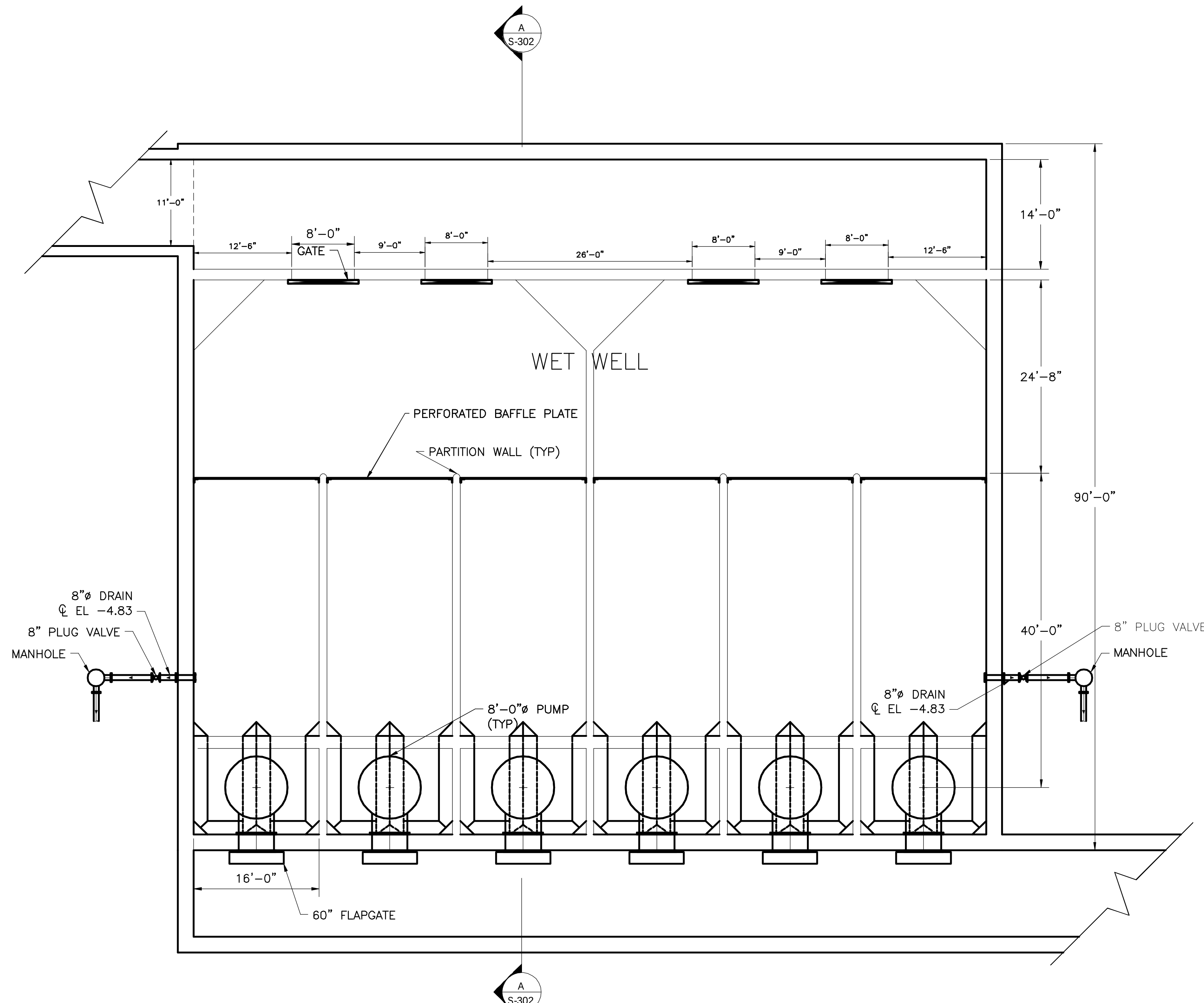

Chicagoland Underflow Plan - McCook Reservoir
DESIGN DOCUMENTATION REPORT
Main Stem Existing Maximum Water Surface
levations - Synthetic vs. Observed
Chicago District Corps of Engineers
June 1999
Plate A-174
Elevation (ft)
595
590
585
580
575
570
Location (river miles)
290
300
310
320
330
340
350
2-year
5-year
10-year
20-year
50-year
100-year
500-year
Observed High Water


Chicagoland Underflow Plan - McCook Reservoir
DESIGN DOCUMENTATION REPORT
Main Stem Stage I Synthetic Maximum Water Surface Elevations
Chicago District Corps of Engineers
June 1999
Plate A-176
Elevation (ft)
595
590
585
580
575
570
Location (river miles)
290
300
310
320
330
340
350
2-year
5-year
10-year
20-year
50-year
100-year
500-year


Chicagoland Underflow Plan - McCook Reservoir
DESIGN DOCUMENTATION REPORT
Main Stem Stage II Synthetic Maximum Water Surface Elevations
Chicago District Corps of Engineers
June 1999
Plate A-178
Elevation (ft)
595
590
585
580
575
570
Location (river miles)
290
300
310
320
330
340
350
2-year
5-year
10-year
20-year
50-year
100-year
500-year

APPENDIX B
LLPS Proposed Layout

Electronic Filing - Received, Clerk's Office, October 20, 2008



1 LOWER PLAN
SCALE: 1"=8'-0"

PRELIMINARY

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

Seal

Rev.	Description	Appr.	Date

METROPOLITAN WATER RECLAMATION DISTRICT OF GREATER CHICAGO

Approved: MWRD Assistant Chief Engineer

Correct: ANTHONY BOUCHARD

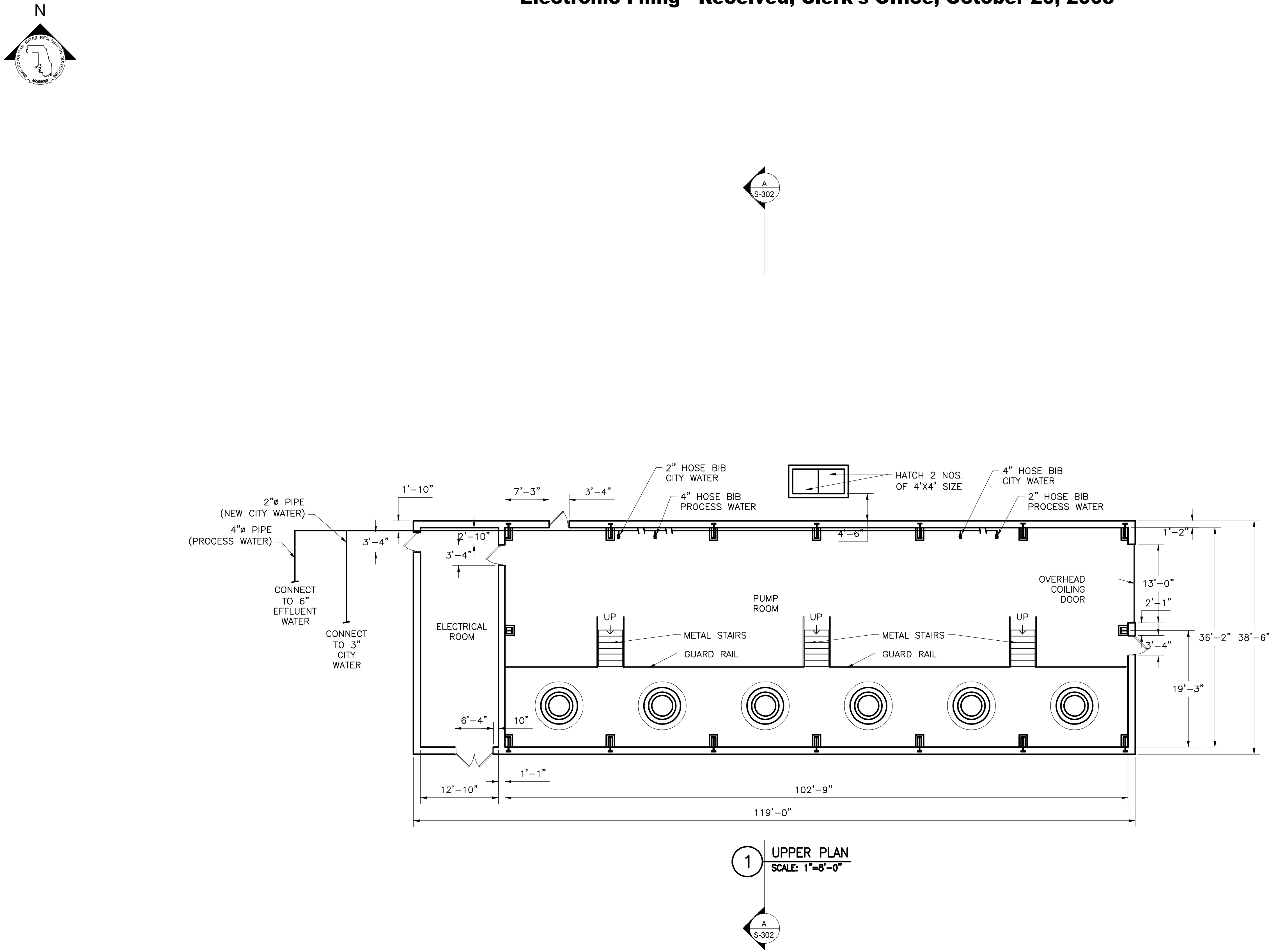
CTE AECOM

Designed by: CMB
Checked by: XX
Drawn by: MB
Reviewed by: XX
Date: XX
Scale: 1/8"=1'-0"

CONTRACT 07-026-2P
NORTH SIDE WATER RECLAMATION PLANT
ULTRAVIOLET DISINFECTION FACILITIES

LOW LIFT PUMP STATION
LOWER LEVEL PLAN

Sheet Number: P-301
Page Number: XX



1 UPPER PLAN
SCALE: 1"=8'-0"

A
S-302

PRELIMINARY

Rev.	Description	Appr.	Date

METROPOLITAN WATER RECLAMATION DISTRICT OF GREATER CHICAGO

Approved: MWRD Assistant Chief Engineer

Correct: ANTHONY BOUCHARD

CTE AECOM

Designed by: CMB
Checked by: XX
Reviewed by: XX
Drawn by: MB
Date: XX
Scale: 1/8"=1'-0"

CONTRACT 07-026-2P
NORTH SIDE WATER RECLAMATION PLANT
ULTRAVIOLET DISINFECTION FACILITIES

LOW LIFT PUMP STATION
UPPER LEVEL PLAN

4 0 4 8 12
SCALE: 1/8" = 1'-0"

Seal

Sheet Number: P-302
Page Number: XX

Electronic Filing - Received, Clerk's Office, October 20, 2008

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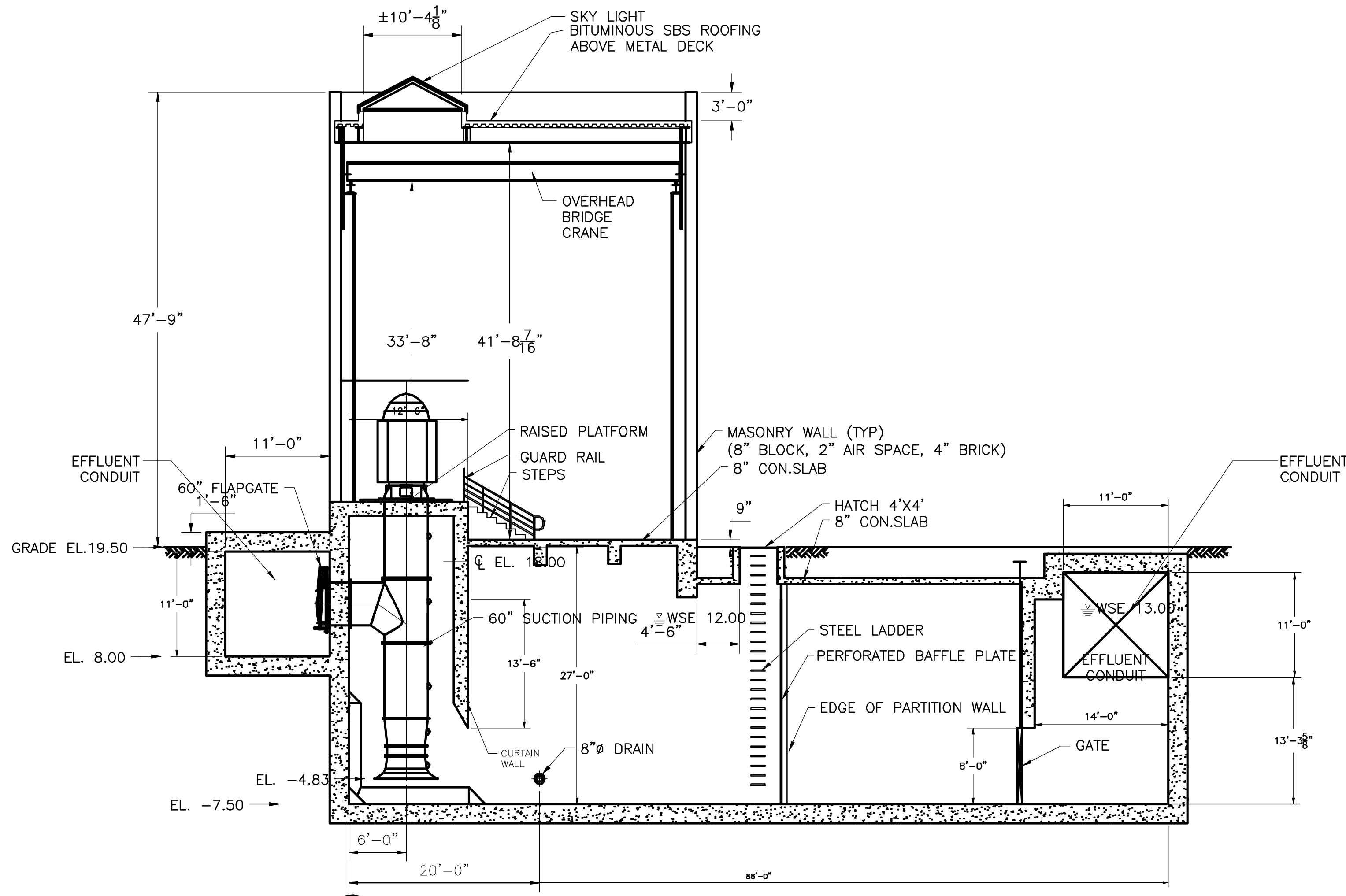

1 SECTION AA
SCALE: 1"=8'-0"

Rev. | Description | Appr. | Date

METROPOLITAN WATER RECLAMATION DISTRICT
OF GREATER CHICAGO

Designed by: CMB
Checked by: XX
Correct: ANTHONY BOUCHARD
Approved: MWRD Assistant Chief Engineer
Drawn by: MB
Reviewed by: XX
Date: XX
Scale: 1/8"=1'-0"

CTE AECOM

CONTRACT 07-026-2P
NORTH SIDE WATER RECLAMATION PLANT
ULTRAVIOLET DISINFECTION FACILITIES

LOW LIFT PUMP STATION
SECTION

PRELIMINARY

4 0 4 8 12
SCALE: 1/8" = 1'-0"

Seal

Sheet Number: P-303
Page Number: XX

PLOT DATE: 9/26/2007 10:40 AM PLOTTED BY: COCKERILL, ERIC

APPENDIX B

UV TECHNOLOGY TECHNICAL MEMORANDUM

Electronic Filing - Received, Clerk's Office, October 20, 2008

DISINFECTION COST STUDY

ULTRAVIOLET DISINFECTION TECHNOLOGY EVALUATION

FOR

METROPOLITAN WATER RECLAMATION DISTRICT OF GREATER CHICAGO

NORTH SIDE WATER RECLAMATION PLANT

TECHNICAL MEMORANDUM

OCTOBER 23, 2007

Prepared By

CTE | AECOM

303 EAST WACKER DRIVE, SUITE 600
CHICAGO, ILLINOIS 60601

MWRDGC Project No. 07-026-2P
CTE Project No. 60026610

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

INTRODUCTION

Background

This technical memorandum has been developed as part of the Preliminary Cost Opinion for Ultraviolet (UV) Disinfection Facilities Study at the Metropolitan Water Reclamation District of Greater Chicago's (MWRDGC, or District) North Side Water Reclamation Plant (NSWRP) in Skokie, Illinois. This memorandum continues the work began in TM1-WQ, which was developed previously as part of the comprehensive Infrastructure and Process Needs Feasibility Study (Feasibility Study) for the NSWRP and a Water Quality (WQ) Strategy for affected Chicago Area Waterways.

The TM1-WQ documented the results of a CTE study of effluent disinfection alternatives for the District's North Side, Calumet and Stickney WRPs. In that study, a task force of national experts (referred to as the Blue Ribbon Panel) reviewed different disinfection technologies and their range of pathogen destruction efficiency, disinfection byproducts and impacts upon aquatic life and human health. Their investigation also included an examination of the environmental and human health impacts of the energy required for the operation of the facility and for the processing and production of process chemicals. Based on economic and non-economic evaluation of alternatives, ozone disinfection and UV disinfection were selected and preliminary basis of design and cost estimates were developed. The UV disinfection system using medium pressure high intensity lamps provided by Trojan Technologies, Inc. was used as a basis of design and cost estimates for the UV system.

Objective

Per the District's request, further evaluation of the UV disinfection technology is required. This additional evaluation is based on the TM-1WQ, the comments received from the EPA as part of the UAA evaluations, and new information obtained since the previous work. The primary objectives of the evaluation presented in this technical memorandum are:

- To describe the current UV technologies being used to disinfect wastewater treatment plant effluent and to find if changes have occurred in the selected UV technology
- To get updated recommendations and costs from different vendors for the selected technology
- To incorporate information available from literature
- To provide references of experience in UV disinfection at other facilities

In the following discussion, the results of this evaluation are given. The sections that follow summarize the currently available UV technologies for disinfection and the experience of using such systems in WWTPs, and provide an updated basis of design for the selected UV disinfection system at the NSWRP.

AVAILABLE UV DISINFECTION TECHNOLOGIES

In the past 20 years, UV disinfection has gained popularity as it is becoming more feasible to implement due its advantages over alternate disinfection methods (i.e. chlorination/dechlorination, ozonation, etc) as noted in TM-1WQ. The UV disinfection systems have also become more sophisticated, reliable, and cost-effective. The currently available technologies of UV disinfection used are shown in Figure 1 (common configurations for municipal wastewater applications are shown bold).

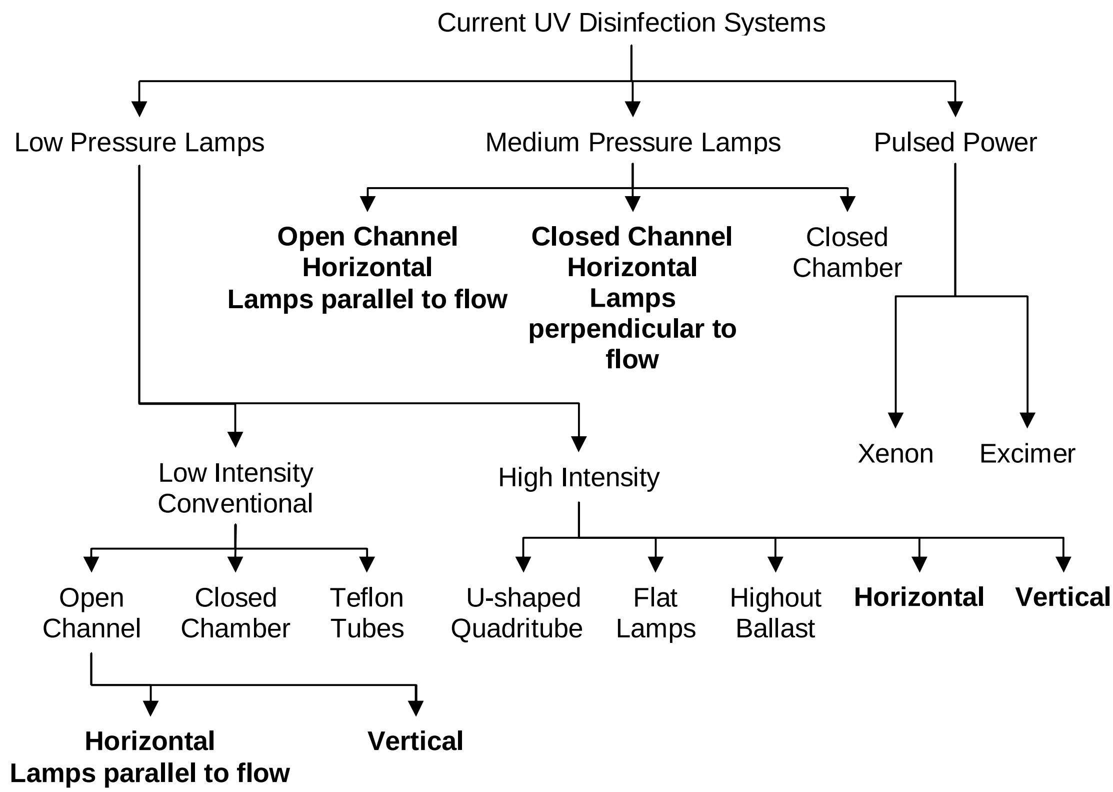

Figure 1 – Categories of Currently Available UV Disinfection Systems (Hunter, et al., 2006b)

To maximize the efficiency of the system, the light source must emit at the wavelength range where DNA and RNA molecules in the microorganisms exhibit a maximum absorbance of UV light (254 nM). Hence, the most important element of UV systems is the light source or lamp. Based on the source of UV, these disinfection systems are categorized into three categories. The important characteristics of these categories are given in Table 1. Here, “Pressure” refers to the pressure of gasses inside the lamp. “Intensity” refers to the energy output.

Low Pressure – Low Intensity (LP-LI)

Available for more than 20 years, low-pressure lamps are arranged in horizontal or vertical configurations submerged in relatively shallow flow channels. Enclosed and Teflon-tube systems are also available. Lamp control is limited to "on" and "off." These lamps are the most energy efficient lamps used for UV disinfection because 85% of their

Table 1 – Typical UV Technology Categories (Bazzazieh, 2005)

UV System	**Low Pressure, Low Intensity**	**Low Pressure, High Intensity**	**Medium Pressure, High Intensity**
Lamp mercury pressure, torr	10^{-3} to 10^{-2}	10^{-3} to 10^{-2}	10^{2} to 10^{3}
Lamp operating temperature, degrees C	40	90 to 250	600-900
Typical power use per lamp, watts	70 to 85	170 to 1,600	2,000 to 5,000
Cleaning	Manual	Automatic wipers	Automatic wipers

total emissions are near the peak for germicidal effectiveness (NYSERDA, 2004). The estimated lifetime of the lamp is approximately 13,000 hours. They are typically used at facilities where the design flow is less than 5 MGD (Hunter, et al., 2006b). Because more lamps are needed as flow increases, the related maintenance costs at large facilities may be higher than those for other UV systems.

Low Pressure – High Intensity (LP-HI)

Introduced within the last several years, early installations of low-pressure, high-intensity lamp systems were deliberately overdesigned, involving multiple banks of lamps and cumbersome hydraulic diversion controls designed to turn lamp banks on and off as operating conditions dictated. When these systems were on, all lamps in the bank or channel operated at full intensity. Newer improvements allow the lamp's wattage output to be varied to optimize dose delivery. These systems also include an automatic cleaning system. These lamps have an average lifetime of about 8,000 hours, with gradually falling lamp intensities (NYSERDA, 2004). These systems use about one-third the lamps of low-pressure systems but also about three times more than medium-pressure systems (Hunter, et al., 2006b).

Medium Pressure – High Intensity (MP-HI)

Medium-pressure lamps became available in open-channel and closed-pipe configurations during the last decade. They use more power and generate higher head losses than the low-pressure systems (Bazzazieh, 2005). An automatic cleaning system that periodically removes the solids that coat the quartz sleeves is also required. The lamps have an average lifetime of about 8,000 hours with intensity gradually declining over time (NYSERDA, 2004). Because they have higher UV output, medium-pressure systems use about one-tenth the number of lamps that a low-pressure system requires (Hunter, et al., 2006b). Medium pressure UV lamps are mostly recommended for larger wastewater treatment plants where the provisions for head requirements could be incorporated in the design, and where a smaller footprint and lower maintenance is needed.

Thus, the technologies are distinguished by the germicidal intensity given off by each lamp type, which correlates to the number of lamps required and the overall UV system size in order to provide a specified dose of energy to the target media (pathogens within the plant effluent). The lamp type selected is determined on a site-specific basis. For the NSWRP, the District has selected the MP-HI system of UV disinfection based on their interest in minimizing the total number of lamps required and the recommendations of the Blue Ribbon Panel during the NSWRP Master Plan. Further investigation of this technology is discussed in the following sections.

LITERATURE REVIEW OF SELECTED MP-HI UV TECHNOLOGY

Information on the latest developments and experience in using the MP-HI UV disinfection system was researched in literature including technical proceedings from Water Environment Federation (WEF), Water Environment Research Foundation (WERF), proceedings from the latest Disinfection conference series undertaken by WEF, American Water Works Association (AWWA), and International Water Association (IWA). In the following discussion, a description of the latest MP-HI technology is provided. This section also summarizes the experiences of some of the wastewater treatment facilities that have successfully implemented UV disinfection.

Typical MP-HI System Configuration

The MP-HI system involves sending the secondary or tertiary effluent through a confined space containing banks of MP-HI UV lamps. A typical MP-HI UV system currently consists of a power supply, an electrical system, a reactor, MP-HI lamps, a mechanical and/or chemical cleaning system, and a control system. The MP-HI UV lamps are enclosed in individual quartz sleeves for protection against dirt and breakage. Reactor chambers (open or enclosed channels) hold the lamps in either a horizontal or vertical configuration. In an open channel system, effluent weirs or automatic level control devices are used to keep the lamps submerged under the effluent water to ensure that the lamps to not overheat, which can reduce lamp life or result in lamp burnout. The whole UV system is also sometimes enclosed in a building to protect it from the natural elements.

The MP-HI UV systems can be divided into several key components for design and troubleshooting purposes including the quality of the influent to the UV system, hydraulics and headloss, the level of disinfection that must be attained for compliance with the regulatory requirements, the reactor configuration, the quartz sleeves, frames, the cleaning mechanisms, the lamps, ballasts or transformers, wiring, and the electrical control system. Brief descriptions of the important process, mechanical, and some of the electrical components are discussed in this section.

Influent Characteristics

The water quality characteristics that affect UV transmittance include iron, hardness, suspended solids, humic materials and organic dyes (NYSERDA, 2004). Dissolved iron can absorb UV light and precipitate on the UV system quartz tubes. Hardness affects the solubility of metals that absorb UV light and can precipitate carbonates on quartz tubes. Organic humic acids and dyes also absorb UV light. Depending on the disinfection system used, the UV transmittance needs to be above a certain level. The generally accepted minimum transmittance is 65%. However, some commercially available MP-HI systems claim to disinfect wastewater with UV transmittance as low as 15-percent.

Reactor Configuration and Hydraulics

An open channel or closed conduit is used as a reactor. One or more than one reactor may be necessary to disinfect the total amount of effluent. UV disinfection systems employ a variety of physical configurations but the most common ones have lamps arranged in linear configuration to increase intensity along the linear axis by avoiding UV emission losses due to self absorption, reflection or refraction that can occur if a UV lamp were twisted into loops or spirals.

The hydraulic characteristics of a reactor can strongly influence disinfection effectiveness. The optimum hydraulic scenario for UV disinfection involves turbulent flow with mixing while minimizing head loss. To maximize effectiveness, UV reactors are preferred to operate at a Reynolds Number of greater than 5,000 (NYSERDA, 2004). Reactor design, including inlet and outlet flow distribution, determines how close the unit operates to a plug flow. Inlet conditions are designed to distribute the flow and equalize velocities. UV system outlets are designed to control the water level at a constant level with little fluctuation within the UV disinfection reactor.

Lamps and UV Intensity Control

The MP-HI lamps contain mercury vapor and argon gas that produce polychromatic radiation, which is concentrated at select peaks throughout the germicidal wavelength region. Most commercially available MP-HI lamps look similar to a fluorescent tube light bulb, but they are made of quartz glass because quartz has the ability to transmit UV light.

The intensity of the lamp is unstable for the first 100 hours of operation and decreases more rapidly during that period. Hence the 100% intensity of the lamp is usually measured after this 100-hour time period. These lamps have a germicidal output of about 16 W/cm, which is about 80 times higher than LP-LI lamps (NYSERDA, 2004). Electronic ballasts for each lamp are used to control the power to the lamp. If the UV dose is to be reduced, variable output electronic ballast can regulate the power to the lamp from 100% to 30%. Entire banks can also be turned off if there is no flow. This allows dose-pacing based on the secondary or tertiary effluent flow and quality, which helps save power and lamp life.

Lamp Fouling and Cleaning

The MP-HI lamps operate at a temperature range of 600 to 900 degree C. The warm temperatures produced by UV lamps promote the precipitation of an inorganic, amorphous film (scale) on the surface of the quartz sleeves when the lamps are placed directly within the wastewater stream. Iron is the most abundant metal in these scales along with other mineral salts and oil, grease, suspended solids deposits, and biofilms (NYSERDA, 2004). If no tertiary treatment is provided, physical debris may contribute to fouling as well.

Lamp fouling significantly reduces the effectiveness of UV disinfection by blocking the UV rays. The MP-HI UV disinfection systems must be cleaned on a regular basis. Researchers have found that the lamp fouling increases linearly with the time elapsed after last cleaning, but the dependency of the cleaning frequency on the quality of effluent is not well predicted (NYSERDA, 2004). So, pilot testing is usually done to determine cleaning frequency. Most of the commercially available MP-HI UV disinfection systems require mechanical as well as chemical cleaning. The latest technology uses a system of mechanical wipers and sleeves containing cleaning chemicals surrounding the lamp. The cleaning solution usually contains some acidic solution that prevents fouling (Darby et al., 1995). This cleaning system can be programmed to clean at a set frequency without the need for disrupting the disinfection process. The cleaning solution needs to be replaced periodically depending on the type of solution used and characteristics of the site specific effluent water quality.

Process Control

The need to pace the dose in the MP-HI UV disinfection system is important because too much dosing wastes electricity and too little dosing would not meet the disinfection regulatory requirements and goals. Several process control options are available to control the dosing. Although manual control of the dosing is possible, an automated process control facilitates online pacing of the dose and also allows it to be interfaced with the plant's overall supervisory control and data acquisition (SCADA) system. The flow, lamp output, and water conditions are measured in pacing of the dose, and an algorithm is developed based on long-term measurements to predict necessary system adjustments, maintenance, and component replacements.

Programmable logic control (PLC) technology is the latest available process control technology for dose pacing in the MP-HI UV disinfection system (Hunter et al, 2006b). The PLC interacts with the ballasts, sensors, and online monitoring technology for each disinfection unit. The PLC then interacts with the plant's overall control system to allow remote monitoring and adjustment of the system. The PLC is usually supplied by the manufacturer of the unit.

Safety

The UV disinfection systems are one of the safest technologies available for disinfection. The high voltage power supplies for the MP-HI UV disinfection system may pose some issue as the lamps are submerged in the water most of the time, but compliance with normal electrical safety codes should mitigate the hazardous conditions. Submerging a lamp in water, even if it is just a few inches below the surface, will greatly reduce the intensity (NYSERDA, 2004). Thus, the MP-HI UV reactors should be designed to ensure constant water levels to minimize the risk of UV exposure.

Sudden or prolonged exposure to ultraviolet (UV) light can result in eye injury, skin burns, premature skin aging, or skin cancer. Individuals who work with UV disinfection systems – or in any area where UV light is used - are at risk of UV exposure if the appropriate protective equipment is not used. The UV radiation should be confined to a restricted area, and an interlocked access system should be in place so that the UV light is shut off when the protective enclosure is opened (Prentiss, 2004). A UV safety program for operators is usually undertaken to make them aware of the effects of UV exposure.

REVIEW OF AVAILABLE TECHNOLOGIES FROM MANUFACTURERS

As discussed previously, the Blue Ribbon Panel recommended medium pressure, high intensity technology based on the size of the proposed facilities and the District's interest in minimizing the total number of bulbs. Two commercially available medium pressure, high intensity systems are available for the municipal wastewater market. For comparison, low pressure, high intensity system manufacturers were also contacted. A review of the information available from the UV technology manufacturers has been summarized in Table 2 and discussed below.

Trojan Technologies – Trojan UV4000™Plus

Trojan Technologies recommends their Trojan UV4000™Plus model for disinfection of the effluent at the North Side WRP. The system is especially designed for large scale applications of 10 MGD or more, and uses MP-HI lamps horizontal and parallel with the flow incorporating an automatic chemical/mechanical cleaning system. Trojan claims that this system is capable of treating wastewater effluents with UV transmittance as low as 15-percent when appropriately sized. It has a PLC-based system to monitor and control all UV functions, and has automated dose delivery based on lamp age, and other water parameters such as flow rate, UV transmittance, and turbidity. The system has high efficiency ballasts that can vary output from 30% to 100% per bank to match the UV dose with effluent quality and flow rate. Trojan claims to have over 375 installations of this system worldwide.

Figure 2 – UV4000+ System
(Courtesy of Trojan Technologies)

Aquionics – InLine50,000+

Aquionics has recommended their InLine50,000+ system for disinfection of the effluent at the North Side WRP. The system uses horizontal high output medium pressure lamps aligned perpendicular to the flow in a closed conduit reactor, which enables treatment of high flows without bypass. The manufacturer claims the compact design achieves a low pressure drop even for gravity fed flows, although reported headloss is approximately 5-6 times that of an open channel system. It comes with advanced "fail-safe" UV monitors with all functions controlled by microprocessors.

Figure 3 – InLine50,000+ System
(Courtesy of Aquionics)

Calgon Carbon – C^{3}500™

The C^{3}500™ wastewater disinfection system recommended by Calgon Carbon employs low pressure, high intensity UV lamp technology with electronic ballasts to effectively disinfect wastewater plant effluent. The modular design can be quickly installed in an open channel parallel to the flow of wastewater. The C^3 Series™ is designed for simple operation and trouble-free maintenance. It has a control system that allows dose or flow pacing. The system has only automatic mechanical cleaning and does not utilize any automatic chemical cleaning. Other manufacturers that supply this type of system include ITT/Wedeco, and Infilco-Degremont/Ozonia.

Figure 4 – TAK25 System
(Courtesy of ITT/Wedeco)

Severn Trent Services (STS)/Quay – MicroDynamics™

STS/Quay has recommended their MicroDynamics™ system for disinfection of the final effluent at the North Side WRP. Their microwave ballast technology uses microwaves to energize low-pressure, high-output bulbs for wastewater disinfection. The bulbs light instantly and lamps can be switched on and off to match the flow. According to the manufacturer, the main advantage of the system is better control of power to the lamps, which significantly increases the lamp life. The system is based on a relatively new concept and no information is available on its application and experience at large wastewater treatment facilities.

Figure 5 – MicroDynamics System (Courtesy of STS/Quay)

Table 2. Summary of Manufacturer-recommended UV Technologies for NSWRP

	Trojan Technologies	**Aquionics**	**Calgon Carbon**	**STS/Quay**
Recommended model	UV4000™Plus	InLine50000+	$C^3$500™	MicroDynamics™
Lamp type	MP-HI	MP-HI	LP-HI amalgam	LP-HI energized by microwaves
Channel dimensions LxWxD	40’6” x 8’10” x 14’4”	N/A	38’6” x 7’2.25” x 6’4”	N/A
Channels	5 (4 + 1 for redundancy)	18	15	N/A
Reactors/channel	1	1	1	N/A
Banks/reactor	2	1	2	N/A
Modules/bank	7	1	15 racks/bank	N/A
Lamps/module	24	32	8 lamps/rack	N/A
Total lamps	1680	576	3600	N/A
Lamp life, hours	5,000	8,000	12,000	27,000
Lamp configuration	Horizontal, parallel to flow	Horizontal, perpendicular to flow	Horizontal, parallel to flow	N/A
Headloss through Reactor	9”	56”	N/A	N/A
Cleaning system	Automatic mechanical and chemical	Automatic mechanical and chemical	Automatic mechanical, non-chemical	N/A
Price (excluding taxes)	$ 7,986,000	$ 5,221,000	$ 7,455,000	N/A

N/A – Not available

REFERENCE INFORMATION FROM OTHER OPERATING FACILITIES

Case Study: Clayton Water Reclamation Center (WRC), Atlanta, GA
Source: Goodman and Mills, 2002

The Clayton WRC is a biological nutrient removal plant serving portions of Fulton, DeKalb, and Gwinnett counties and much of the City of Atlanta, Georgia. The plant discharges into the Chattahoochee River. It has a maximum monthly flow of 122 MGD, with a permit limit of 30 mg/L of monthly average TSS in the final effluent. The maximum allowable Fecal Coliform in the final effluent is 200 counts/100 mL monthly maximum average and 400 counts/100 mL weekly maximum average.

The plant uses an open channel, gravity-flow MP-HI UV disinfection system consisting of medium-pressure vapor UV lamps, oriented horizontally and parallel to flow, arranged in modules, and installed inside enclosed reactors in open channels. The basis of design of the UV system is given in Table 3. At this facility, flow from the filters initially enters the influent channel of the disinfection structure, then flows over a weir into a common influent channel, and finally flows through four individual channels. Each of these channels is equipped with a UV lamp system. In order for the UV lamp system to work properly, a specified level of liquid must be maintained in the channel to ensure that the lamps are always submerged when in operation. To maintain the desired liquid level in each channel, downstream weirs are used prior to the flow entering the clearwell. Plant reuse pumps are located downstream of the UV system.

Table 3. Basis of Design – Clayton WRC

Number of channels	4 operational/1 future
Number of banks/channel	2
Number of modules/bank	9
Number of lamps/module	10
Total number of lamps	720
UV dose, mJ/cm^2	24

Before the design, installation and operation of the UV system, a collimated-beam dose-response testing was done to estimate the sensitivity of the in-situ fecal coliform to UV. Once the dose was determined using the pilot tests, the system was installed and came into operation. The initial operational data is given in Table 4.

Table 4. Operational Data – Clayton WRC (April to September, 2001)

Normal Daily Dose Range	24 to 49 $mW\text{-}sec/cm^2$
Overall Dose Range	18 to 100 $mW\text{-}sec/cm^2$
Normal Daily Transmittance Range	74% to 78%
Overall Transmittance Range	65% to 83%
Days of Coliform Data	182
Days Count was Below 400 per 100 mL	174
Days Where Fecal Count was Below 200 per 100 mL	170
Days Where Fecal Count was Below 23 per 100 mL	141

During the initial phase, the facility operated on a UV dose exceeding the one established during the dose-response testing. In the first couple of months of operation after the startup of the UV system, the Clayton operational staff fed a small dose of sodium hypochlorite downstream of the UV system, until they became comfortable with the system and its reliability. During initial operation, it was found that the normal transmittance range was 74% to 78%, which exceeded the conservative average design value of 68% established using unfiltered samples. The UV system was found to meet the Georgia state standards for reuse 77% of the time, and monthly averages 95% of the time.

Telephone Survey of Experience at Other Facilities

A telephone survey was done by calling relevant personnel at facilities that have been using UV technology to disinfect their secondary or tertiary effluent. Priority was given based on the following criteria for selection of the facility for the telephone survey.

- Facility should preferably be in the Midwest or other areas that treat hard water and may be prone to calcium fouling
- Facility should have a high treatment capacity, possibly greater than 100 MGD
- Facility should be using a MP-HI UV disinfection system

Five facilities were contacted and the personnel responsible for the operation and maintenance of the UV equipment were interviewed. A summary of the results of this telephone survey is given in Table 5. The facilities contacted were Racine WWTP in Racine (WI), R.L. Sutton WRF in Cobb County (GA), Grand Rapids WWTP in Grand Rapids (MI), Jacksonville WWTP in Buckman (FL), and Valley Creek WWTP in Valley Creek (AL). All these facilities have peak influent flows close to or above 100 MGD.

Following observations are made based on the telephone interview of facilities using a MP-HI UV system for disinfection of their secondary or tertiary effluent.

- Four out of the five facilities use a system provided by Trojan Technologies, Inc.
- The Jacksonville WWTP has low UV transmittance, sometimes as low as 8% during high industrial discharge to the plant. They have had a few permit violations, but otherwise their disinfection system helps them meet the permit limits.
- Calcium fouling due to hardness in the source water is not a significant problem because of the automatic mechanical/chemical cleaning system that dissolves and wipes away any scales. This was observed in all five plants including the Racine and Grand Rapids utilities which have Lake Michigan source water.
- Fouling due to iron in the effluent has been a problem at the Racine, Sutton, and Grand Rapids facilities. The iron in the effluent at all three plants was primarily from the chemical phosphorus removal using Ferric Chloride. At Grand Rapids WWTP, the chemical addition is upstream of the secondary treatment process; staining of sleeves was found only when the chemical addition was in the secondary clarifiers. At the Sutton WRF, fouling of lamps due to iron is observed although chemical addition is upstream of secondary process and sand filters are used upstream of the UV disinfection system. At the Racine WWTP, fouling may be due to ferric chloride addition and/or due to the additional iron brought by the ferric sludge from another water treatment plant, although operational controls are used to prevent both sources from occurring simultaneously.

- The Trojan ActiClean gel was found to be ineffective at the Racine and Grand Rapids plants experiencing fouling due to iron. These utilities and Sutton WRF used alternate chemicals to clean the lamp sleeves.
- The frequency of cleaning and changing of the cleaning solution is specific to the utility and would have to be determined only by experience.
- The facilities typically replace lamps after the lamps' rated service life of 5000 to 6000 hours, but many times the operators used the lamps until they failed (shorter lamp life) or burn out (lamp life up to 9000 hours).
- Labor requirements varied amongst facilities, with some facilities requiring more manhours to handle the fouling. The Jacksonville WWTP required more labor to mitigate the algal growth caused by high temperatures.
- Storage requirements were not significant at all the facilities. Only a few gallons of the cleaning solution were stored at a time. Lamps were also not stored on a large scale.
- None of the facilities had done an on-site pilot testing. Only collimated beam testing (by the manufacturer, at Grand Rapids and Jacksonville WWTPs) was done to analyze the UV dose-response. At Valley Creek WWTP, one of the smaller facilities had a functioning UV system by Trojan Technologies, and that prompted them to install the system at their larger plant without any pilot testing.

As long as other processes in the plant are performing as desired, all five facilities were satisfied with the UV disinfection system because it met their disinfection goals.

Table 5. Summary of Telephone Interviews of Utilities Using MP-HI UV Disinfection Systems

Facility	Racine WWTP	R.L.Sutton WRF	Grand Rapids WWTP	Jacksonville WWTP	Valley Creek WWTP
Location	Racine, WI	Cobb County, GA	Grand Rapids, MI	Buckman, FL	Valley Creek, AL
UV disinfection system	Trojan UV4000+	Aquionics	Trojan UV4000+	Trojan UV4000 with custom modifications	Trojan UV4000+
Startup date	2005	Dec 2005	Feb 2005	2001	Jul 5, 2005
Disinfection goals met	Yes	Yes	Yes	Yes	Yes
Plant maximum flow	108 mgd	120 mgd design	90 mgd	105 mgd	240 mgd
UV transmittance, %	60%-85%	N/A	60 to 65%	48% to 55%	80% to 85%
Coliforms, current (monthly permit)	N/A (400) E. Coli count/100 mL	1 (200) F. Coli count/100 mL	80 to 140 (200) F. Coli count/100 mL	200 (800) F. Coli count/100 mL	15 (1000) F. Coli count/100 mL
Target UV dose	~29 mJ/cm^2	50 mJ/cm^2	30 to 40 mJ/cm^2	N/A	32 mJ/cm^2
Tertiary filtration	No	Yes, sand filters.	No	No	Yes, sand filters
Chemical Phosphorus removal - Ferric Chloride addition	Yes, additional ferric sludge from water treatment plant.	Yes, addition before secondary treatment.	Yes, addition before secondary treatment.	No	No
Fouling – iron (staining of sleeves)	Yes	Yes, sleeves replaced 1.5 to 2 yr	When chemicals added to secondary clarifiers	N/A	N/A
Water hardness	Lake Michigan source	Not significant	Lake Michigan source	Well water	River water
Fouling – hardness	Yes, but insignificant	Negligible	Yes	Yes	Negligible
Cleaning Chemical Used	Lime-Away	Phosphoric acid	Lime-Away plus 10% phosphoric acid	Trojan ActiClean gel	Trojan ActiClean gel
Additional cleaning other than automatic cleaning and its frequency	Manual once/ week only if necessary. Change cleaning solution per 6-8 weeks	Once after shutting down a channel and once before startup.	Check for fouling every 2 weeks and replace the cleaning solution once a month.	Check and replace cleaning solution every 2 months.	Manual, if necessary
Storage of cleaning solution	7-8 cases with 1-gal container/case	Buy 5-gal acid crystals Make phosphoric acid in a storage tank.	1-gal container at North side and 1 gallon at South side.	2 to 3 cases with 4 gal/case.	4 cases, 16 bottles/case.
Lamp replacement frequency	~ 6000 hrs, or after burnoff at ~9000 hrs.	~ 5000 to 6000 hrs. About 1 lamp/week.	~ 5000 to 6000 hrs, or after failure.	~ 5000 hrs, or after failure.	~ 6200 hrs, or after failure or burnoff.
Lamp storage	N/A	Very few.	Very few (Trojan ships new lamps on time)	~100 lamps at a time.	Few new lamps. Partially used lamps stored for reuse.
Pilot testing on site	None	None	None	None	None
Other testing	Collimated beam	N/A	Collimated beam by Trojan	Collimated beam by Trojan	None
Labor requirement	8 hrs/ week	7-8 hrs/ week	8 hrs/week	18 to 20 hrs/week	12 hrs/bank to replace cleaning gel twice/yr. 25 hrs/bank to replace bulbs.

N/A – Not Available

DISTRICT UV EQUIPMENT TRIALS PROJECT AND SUPPORTING WATER QUALITY INFORMATION

Currently, the District is planning an ultraviolet disinfection technology disinfection trial at the Hanover Park WRP. The trial is intended to provide real world operating and performance data on several available UV systems. The trials will allow District staff to become familiar with design, implementation, operation, and monitoring of a UV disinfection system through a small scale application.

Due to the site and time limitations, the UV technologies to be tested are limited to low pressure, high intensity technology to match the low flows available for testing. Currently, the District has invited Trojan Technologies, ITT/Wedeco, Severn Trent Services/Quay, and Infilco-Degremont/Ozonia to set up small-scale pilot installations for startup and operation during the winter of 2007-2008.

In preparation for this testing and to support the District's ongoing investigations into the potential need for UV disinfection implementation, additional water quality data testing related specifically to UV disinfection has been completed at Hanover Park WRP, North Side WRP, and Calumet WRP in 2006-2007. Water quality data was collected once every two weeks on plant effluent grab samples for Fecal Coliform counts, Escherichia Coliform counts, Total Coliform counts, COD, and UV transmittance. This data was tested pre-filtered, post-laboratory filtered, and post-full scale filtered (Hanover Park WRP samples only). In addition, the District collected hourly grab sample UV transmittance data at Hanover Park for two days in June of 2007. Appendix A includes the complete data collected to date.

Table 6 below presents a summary of the unfiltered data at the NSWRP and CWRP sites.

Table 6. Summary of 2006/2007 Water Quality Testing

Site	Fecal[1]	E.Coli	Total Coliform	COD	UV Transmittance
	CFU/100 ml	CFU/100 ml	CFU/100 ml	mg/L	%
NSWRP					
Average	13,254	11,825	147,140	26	76.7
Std Dev	8,213	5,818	59,619	12	3.54
CWRP					
Average	10,804	9,878	120,321	27	71.3
Std Dev	7,292	5,270	55,471	9	2.22

[1] Prior to 2006, WRP outfall sampling indicated maximum fecal coliform counts of 200,000.

While additional data is suggested to increase the level of confidence in the maximum day data (98% confidence level), this information does provide a good indication of the UV transmittance data and normal range of the bacteria levels. This information can be used to develop appropriate assumptions for the UV disinfection sizing criteria.

Need for Pilot Testing

Although many manufacturers suggest that collimated beam testing of water samples is sufficient for design, full-scale pilot testing is useful for demonstrating the effectiveness and performance of the UV systems as well as establishing critical design parameters.

In this case, the proposed UV disinfection systems will be among the largest ever constructed in North America and none of the UV systems have been applied at this scale in their current configuration. In particular, the following three issues could be addressed during full-scale piloting:

1. In-situ determination of fouling factors and lamp aging factors based on actual site specific conditions. This data is critical to optimize the lamp dose calculations and system sizing.
2. In-situ determination of fouling potential with and without iron salt addition. The phone survey has indicated that Lake Michigan source water combined with iron salt addition creates more rapid fouling than other applications.
3. Actual development of maintenance and operating frequencies required for the specific system to be implemented including preventative maintenance, bulb replacement, sensor maintenance, operating modes, power optimization, etc. This data may influence system sizing if individual lamps are not replaced if they burn out early.

Additional site-specific data such as UV transmittance, optimum UV dose requirements, and effluent quality information could be obtained from a carefully designed pilot testing program. This data might permit the District to collect a body of data by which to present the case for a lower UV dose to more closely match the required log removal of bacteria.

BASIS OF DESIGN OF UV SYSTEM FOR NORTH SIDE WRP

Per the District's recommendation, the MP-HI UV disinfection system has been selected for disinfection of the final effluent at the North Side WRP. Based on a review of the information provided by the UV equipment manufacturers and the experience of five other facilities, it is observed that Trojan Technologies provides a widely-used low-maintenance solution for final effluent disinfection. The design of the MP-HI UV disinfection system for the North Side WRP is based on the Trojan UV4000™Plus equipment provided by Trojan Technologies. The basis of design is given in Table 7.

Table 7. Design Parameters for UV Disinfection Unit at NSWRP

Parameter	Design Value
Design flow, mgd	450
Average flow, mgd	333
Maximum TSS[a], mg/L	15
Pre-Disinfection Effluent E.Coli Count)[b], cfu/100 mL, maximum (Assumed)	200,000
Post-Disinfection Effluent E.Coli Count Target[c], cfu/100 mL	1030
Effluent hardness[d], mg/L as $CaCO_3$	270
UV transmittance, minimum, %	65
UV dosing	
UV intensity[e], W/lamp	4,000
Fouling Factor, %	90
Lamp Aging Factor, %	89
Lamp Age, hours	5,000
UV dose[f], mW-s/cm^2	40
Hydraulics	
Channel dimensions, WxD	106" x 172"
Number of channels	5 (4 plus 1 standby)
Number of reactors per channel	1
Number of banks per reactor	2
Number of modules per bank	7
Number of lamps per module	24
Total number of lamps	1680
Liquid level control in channel	Motorized Weir Gate
Headloss, UV reactor only	9"
Velocity in each channel, V, ft/s	1.74
Total power requirement, kW	5376
Average power requirement, kW	2903

[a] Monthly TSS permit limit, 12 mg/L
[b] Annual average
[c] Future requirement (monthly geometric average)
[d] Mean value
[e] 100% intensity at 100 hours of lamp use
[f] IEPA requirement

The lamp aging and fouling factors are based on recommendations of manufacturers. Trojan Technologies generally recommends a fouling factor of 95%, which was determined using Bioassay validation required by the State of California. USEPA's UVdis program (UV Dosing Modeling Software) recommends a fouling factor of 100% for a system that incorporates automatic mechanical and chemical cleaning, such as Trojan's UV4000™Plus. The IEPA accepts the results of the UVdis program to size the system to meet the IEPA's 40 mJ/cm2 dose requirement. Other UV disinfection systems' fouling factors range from approximately 80 to 85%, though these systems do not incorporate chemical cleaning systems into their design.

These values were taken into consideration when choosing a fouling factor for NSWRP's design. A value of 90% was settled upon to incorporate both Trojan's recommendations and good engineering judgement.

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APPENDIX A
2006 UV TRIAL WATER QUALITY DATA
NSWRP, CWRP, AND HPWRP

Table 1: HOURLY PERCENT UV TRANSMITTANCE DATA ON SECONDARY EFFLUENT SAMPLES COLLECTED AT HANOVER PARK WRP FROM 6/5/07 TO 6/8/07
Secondary Effluent Grab Samples Collected Hourly[1]

Date	Time	Percent UV Transmittance
6/5/2007	12:15	71
6/5/2007	13:15	73
6/5/2007	15:15	70
6/5/2007	15:15	71
6/5/2007	16:15	70
6/5/2007	17:15	67
6/5/2007	18:15	70
6/5/2007	19:15	70
6/5/2007	20:15	72
6/5/2007	21:15	69
6/5/2007	22:15	72
6/5/2007	23:15	74
6/6/2007	00:15	72
6/6/2007	01:15	68
6/6/2007	02:15	71
6/6/2007	03:15	72
6/6/2007	04:15	73
6/6/2007	05:15	73
6/6/2007	06:15	72
6/6/2007	07:15	74
6/6/2007	08:15	75
6/6/2007	09:15	75
6/6/2007	10:15	77
6/6/2007	11:55	73
6/6/2007	12:55	70
6/6/2007	13:55	71
6/6/2007	14:55	70
6/6/2007	15:55	76
6/6/2007	16:55	NS
6/6/2007	17:55	72
6/6/2007	18:55	72
6/6/2007	19:55	72
6/6/2007	20:55	75
6/6/2007	21:55	71

Table 1 (Continued): HOURLY PERCENT UV TRANSMITTANCE DATA ON SECONDARY EFFLUENT SAMPLES COLLECTED AT HANOVER PARK WRP FROM 6/5/07 TO 6/8/07
Secondary Effluent Grab Samples Collected Hourly[1]

Date	Time	Percent UV Transmittance
6/6/2007	22:55	72
6/6/2007	23:55	71
6/7/2007	00:55	71
6/7/2007	01:55	71
6/7/2007	02:55	71
6/7/2007	03:55	73
6/7/2007	04:55	74
6/7/2007	05:55	69
6/7/2007	06:55	71
6/7/2007	07:55	71
6/7/2007	08:55	70
6/7/2007	09:55	72
6/7/2007	10:30	77
6/7/2007	10:55	76
6/7/2007	11:30	78
6/7/2007	12:30	77
6/7/2007	13:30	77
6/7/2007	14:30	76
6/7/2007	15:30	77
6/7/2007	16:30	76
6/7/2007	17:30	76
6/7/2007	18:30	77
6/7/2007	19:30	76
6/7/2007	20:30	76
6/7/2007	21:30	76
6/7/2007	22:30	76
6/7/2007	23:30	76
6/8/2007	00:30	77
6/8/2007	01:30	77
6/8/2007	02:30	68
6/8/2007	03:30	76
6/8/2007	04:30	78
6/8/2007	05:30	76
6/8/2007	06:30	76

Table 1 (Continued): HOURLY PERCENT UV TRANSMITTANCE DATA ON SECONDARY EFFLUENT SAMPLES COLLECTED AT HANOVER PARK WRP FROM 6/5/07 TO 6/8/07
Secondary Effluent Grab Samples Collected Hourly[1]

Date	Time	Percent UV Transmittance
6/8/2007	07:30	75
6/8/2007	08:30	75
6/8/2007	09:30	75
Minimum		65.0
Maximum		69.0
Mean		66.7

NS = No sample.
[1]Samples collected from a manhole wherein the effluent from all eight final tanks mingle.

TABLE 1: COMPARISON OF PRE-FILTER, LAB-FILTERED, AND FULL-SCALE POST-FILTER SECONDARY EFFLUENT (PLANT OUTFALL) SAMPLES COLLECTED FROM 11/16/05 TO 5/17/06 AT HANOVER PARK WRP

	Fecal Coliform[1] CFU per 100 mL			*Escherichia* Coliform[1] CFU per 100 mL			Total Coliforms[1] CFU per mL			COD[2] mg/L			Absorbance[2] abs_unit			Transmittance[2] %		
Date	Pre-Filter	Full-Scale Post-Filter	Lab-Filtered	Pre-Filter	Full-Scale Post-Filter	Lab-Filtered	Pre-Filter	Full-Scale Post-Filter	Lab-Filtered	Pre-Filter	Full-Scale Post-Filter	Lab-Filtered	Pre-Filter	Full-Scale Post-Filter	Lab-Filtered	Pre-Filter	Full-Scale Post-Filter	Lab-Filtered
11/16/2005	22,000	9,500	3,300	17,330	8,160	1,920	155,300	92,100	6,870	n/a	n/a	n/a	n/a	0.153	n/a	n/a	70.31	n/a
11/30/2005	60,000	38,000	5,600	46,100	24,190	9,210	>241,900	>241,500	36,500	n/a	n/a	n/a	n/a	0.167	n/a	n/a	68.03	n/a
12/14/2005	14,000	9,400	4,500	18,500	15,530	4,610	241,900	173,300	24,190	n/a	n/a	n/a	n/a	0.149	n/a	n/a	70.92	n/a
1/11/2006	1,200	4,900	1,100	12,033	7,700	1,220	130,000	77,010	8,660	41	28	33	0.160	0.144	0.143	69.14	71.82	71.99
1/25/2006	4,700	3,800	380	6,870	2,610	450	77,000	46,100	2,050	49	37	36	0.158	0.145	0.136	69.50	71.57	73.11
2/8/2006	4,200	2,500	200	3,650	2,360	430	20,500	15,300	1,840	50	36	27	0.149	0.136	0.133	70.92	73.16	73.71
2/22/2006	6,700	2,900	700	6,130	4,610	771	61,300	32,600	2,760	59	36	27	0.134	0.131	0.129	73.41	74.00	74.34
3/8/2006	2,500	1,700	230	2,610	1,550	404	24,200	13,500	1,260	54	38	35	0.155	0.151	0.140	70.06	70.67	72.53
3/22/2006	3,800	2,000	280	3,260	3,450	504	43,500	19,900	1,270	79	29	29	0.156	0.150	0.143	69.78	70.84	71.99
4/5/2006	4,500	200	260	2,760	2,360	374	32,600	19,900	1,450	46	29	20	0.151	0.151	0.138	70.63	70.67	72.78
4/19/2006	5,800	3,100	440	4,880	3,650	520	51,700	32,600	2,190	57	42	29	0.159	0.148	0.144	69.34	71.20	71.78
5/3/2006	<100	<100	<20	20	<10	<10	355	97	10	48	31	40	0.172	0.138	0.142	67.34	72.82	72.19
5/17/2006	200	<100	<20	63	<10	<10	1,090	135	10	48	35	27	0.166	0.138	0.140	68.27	72.86	72.40
Average	10,800	7,091	1,545	9,534	6,923	1,856	69,954	43,729	6,851	53	34	30	0.156	0.146	0.139	69.84	71.46	72.68
Std. Dev	16,627	10,673	1,963	12,493	7,011	2,742	72,069	49,564	11,011	11	5	6	0.010	0.009	0.005	1.64	1.53	0.82

[1]Grab samples.

[2]24-hour composite plant samples taken for NPDES permit.

TABLE 3: CHARACTERIZATION OF PRE-FILTER AND LAB-FILTERED NORTH SIDE WRP OUTFALL SAMPLES COLLECTED FROM 11-15-05 TO 11-28-06

Date	Fecal Coliform[1] CFU per 100mL		*Escherichia* Coliform[1] CFU per 100mL		Total Coliforms[1] CFU per 100mL		COD[2] mg/L		Absorbance[2] abs_unit		Transmittance[2] %	
	Pre-Filter	Lab-Filtered	Pre-Filter	Lab-Filtered	Pre-Filter	Lab-Filtered	Pre-Filter	Lab-Filtered	Pre-Filter	Lab-Filtered	Pre-Filter	Lab-Filtered
11/15/2005	9,300	700	15,530	910	198,600	4,350	n/a	n/a	n/a	0.114	n/a	77.00
11/29/2005	10,000	760	5,170	560	173,300	3,450	n/a	n/a	0.152	0.149	70.51	71.00
12/13/2005	11,000	640	8,660	600	241,900	2,910	n/a	n/a	0.104	0.104	78.75	78.70
1/10/2006	8,800	280	8,160	320	241,900	2,280	30	22	0.108	0.098	78.03	79.75
1/24/2006	7,700	1,100	11,200	1,070	>241,900	6,130	51	23	0.106	0.098	78.39	79.89
2/7/2006	10,000	1,500	15,500	1,270	141,000	5,790	29	25	0.098	0.093	79.85	80.68
2/21/2006	5,200	500	7,270	581	77,000	2,250	25	16	0.102	0.097	79.11	80.08
3/7/2006	9,400	620	13,000	697	173,000	4,110	46	46	0.152	0.152	69.66	70.43
3/21/2006	4,400	390	4,110	399	105,000	1,660	18	24	0.102	0.098	79.11	79.80
4/4/2006	2,400	100	2,760	155	24,200	1,080	22	14	0.111	0.102	77.40	79.07
4/18/2006	8,700	620	9,210	860	105,000	1,920	20	29	0.131	0.121	74.00	75.77
5/2/2006	20,000	2,100	19,900	798	141,000	4,880	20	38	0.118	0.115	76.21	76.78
5/16/2006	5,500	330	4,880	384	64,900	1,900	22	22	0.121	0.117	75.73	76.34
5/30/2006	7,800	860	8,660	1,320	98,000	7,270	20	18	0.108	0.100	78.07	79.43
6/13/2006	16,000	1,600	14,100	2,480	199,000	7,270	57	31	0.125	0.115	74.99	76.74
6/27/2006	7,600	440	6,870	759	130,000	3,870	41	18	0.180	0.172	66.07	67.30
7/11/2006	16,000	2,800	15,500	3,870	155,000	10,500	16	18	0.113	0.108	77.18	78.03
7/25/2006	26,000	2,300	12,000	2,220	120,000	9,210	14	14	0.103	0.099	78.98	79.62
8/8/2006	22,000	3,800	17,300	4,110	141,000	14,100	15	11	0.097	0.097	80.08	80.03
8/22/2006	20,000	3,900	24,200	3,870	242,000	12,000	31	5	0.112	0.103	77.36	78.89
9/5/2006	15,000	2,400	14,100	3,450	105,000	8,160	24	20	0.130	0.139	74.13	72.57
9/19/2006	27,000	5,600	19,900	5,170	242,000	14,100	16	16	0.114	0.121	76.96	75.64
10/3/2006	28,000	2,400	19,900	2,600	173,000	9,210	16	12	0.107	0.110	78.12	77.71
10/17/2006	32,000	2,800	17,300	2,380	173,000	10,500	18	12	0.107	0.111	78.12	77.49
10/31/2006	8,100	940	6,490	1,040	72,700	4,160	20	26	0.095	0.097	80.40	80.08
11/28/2006	6,700	600	5,790	624	141,000	1,860	26	24	0.095	0.105	80.40	78.48
Average	13,254	1,542	11,825	1,635	147,140	5,958	26	21	0.116	0.113	76.70	77.20
Std. Dev	8,213	1,374	5,818	1,417	59,619	3,946	12	9	0.020	0.020	3.54	3.40

[1]Grab samples.

[2]24-hour composite plant samples taken for NPDES permit.

TABLE 6: CHARACTERIZATION OF PRE-FILTER AND LAB-FILTERED CALUMET WRP OUTFALL SAMPLES COLLECTED FROM 11-17-05 TO 11-02-06

Date	Fecal Coliform[1] CFU per 100mL		*Escherichia* Coliform[1] CFU per 100mL		Total Coliforms[1] CFU per 100mL		COD[2] mg/L		Absorbance[2] abs_unit		Transmittance[2] %	
	Pre-Filter	Lab-Filtered	Pre-Filter	Lab-Filtered	Pre-Filter	Lab-Filtered	Pre-Filter	Lab-Filtered	Pre-Filter	Lab-Filtered	Pre-Filter	Lab-Filtered
11/17/2005	20,000	2,000	19,860	2,720	241,900	20,100	n/a	n/a	0.155	0.157	69.98	69.66
12/1/2005	8,000	1,400	9,800	2,060	155,300	11,200	n/a	n/a	0.146	0.148	71.53	71.20
12/15/2005	7,000	440	7,700	670	173,300	3,080	n/a	n/a	0.130	0.127	74.09	74.69
1/12/2006	6,000	1,100	7,270	880	120,300	7,270	n/a	30	0.141	0.137	72.24	72.99
1/26/2006	3,900	300	3,450	380	51,700	2,250	24	25	0.134	0.129	73.41	74.30
2/9/2006	7,600	820	8,160	712	77,000	3,260	31	27	0.140	0.135	72.40	73.28
2/23/2006	5,900	520	6,490	759	92,100	3,650	31	21	0.140	0.134	72.40	73.49
3/9/2006	23,000	2,500	26,000	3,450	173,000	13,000	53	42	0.149	0.146	71.00	71.41
3/23/2006	9,400	1,200	10,500	860	105,000	5,170	20	29	0.136	0.135	73.07	73.37
4/6/2006	9,300	880	10,500	1,530	130,000	12,000	31	12	0.121	0.114	75.64	76.87
4/20/2006	6,000	720	6,870	833	155,000	4,880	35	25	0.180	0.164	66.15	68.59
5/4/2006	2,900	340	4,350	504	61,300	2,140	38	35	0.168	0.164	67.96	68.55
5/18/2006	n/s	n/s	n/s	n/s	n/s	n/s	27	27	0.166	0.161	68.23	69.02
6/1/2006	15,000	1,200	10,500	1,400	173,000	9,800	40	18	0.154	0.143	70.15	71.94
6/15/2006	3,500	460	3,870	565	48,800	3,080	27	35	0.157	0.149	69.70	71.04
6/29/2006	21,000	2,400	17,300	4,350	173,000	24,200	24	22	0.167	0.163	68.16	68.71
7/13/2006	6,800	840	6,870	1,050	112,000	5,790	20	22	0.154	0.151	70.15	70.71
7/27/2006	4,700	400	3,610	464	77,000	3,870	24	24	0.148	0.139	71.12	72.65
8/10/2006	13,000	2,400	12,000	1,510	98,000	8,160	20	18	0.142	0.143	72.15	71.94
8/24/2006	6,600	700	8,660	798	81,600	3,450	26	16	0.156	0.151	69.90	70.71
9/7/2006	12,000	1,300	11,200	1,560	130,000	5,170	22	20	0.141	0.145	72.36	71.57
9/21/2006	5,700	480	8,660	670	64,900	2,490	23	20	0.131	0.138	74.00	72.74
10/5/2006	30,000	1,700	14,100	1,250	242,000	14,100	20	16	0.138	0.140	72.82	72.49
10/19/2006	22,000	720	11,200	867	86,600	6,870	16	16	0.138	0.142	72.78	72.11
11/2/2006	10,000	660	8,160	644	64,900	3,260	23	19	0.142	0.147	72.11	71.29
Average	10,804	1,062	9,878	1,270	120,321	7,427	27	24	0.147	0.144	71.34	71.81
Std. Dev	7,292	683	5,270	986	55,471	5,797	9	7	0.014	0.012	2.22	2.02

[1]Grab samples.

[2]24-hour composite plant samples taken for NPDES permit.