City of Saint Charles

Main Wastewater Treatment Facility

2015 Facility Plan Update

Continuity • Collaboration • Commitment

November, 2015

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LIST OF ABBREVIATIONS

ABBREVIATION	DESCRIPTION
AOR	actual oxygen requirement
avg	average
BNR	biological nutrient removal
BOD_5	5-day biochemical oxygen demand
C	Celsius
CCTV	closed-circuit television
cf	cubic feet
CIPP	cured-in-place pipe
CMAP	Chicago Metropolitan Agency for Planning
CMOM	Capacity, Management, Operation, and Maintenance
CSRP	Collection System Rehabilitation Program
DAF	design average flow
DMF	design maximum flow
DMR	discharge monitoring report
DNR	Department of Natural Resources
DO	dissolved oxygen
EcoCAT	Ecological Compliance Assessment Tool
EPA	Environmental Protection Agency
F	Fahrenheit
$FeCl_3$	ferric chloride
FOG	Fats, oils, and grease
FPA	Facility Planning Area
FPR	Facility Plan Report
fps	feet per second
ft	feet
FVFS	Fox Valley Fire and Safety
FY	fiscal year
gal	gallons
gcd	gallons per capita per day
gpd	gallons per day
gpm	gallons per minute
GIS	Geographical Information System
HDPE	high density polyethylene

LIST OF ABBREVIATIONS (CONTINUED)

ABBREVIATION	DESCRIPTION
HP	horsepower
hr.	hour
HSPF	Hydrological Simulation Program FORTRAN
IEPA	Illinois Environmental Protection Agency
I/I	infiltration and inflow
L	liter
LA	load allocation
lbs.	pounds
l.f.	lineal feet
in	inch
max	maximum
MCC	motor control center
mg/L	milligrams per liter
MGD	million gallons per day
min	minimum or minute
mL	milliliter
MLSS	mixed liquor suspended solids
mm	millimeter
MOP	Manual of Practice
MOS	margin of safety
NH_3-N	ammonia nitrogen
NO_2	nitrite
NO_3	nitrate
NPDES	National Pollutant Discharge Elimination System
NPW	non-potable water
O_2	oxygen
ORP	oxygen reduction potential
OTE	oxygen transfer efficiency
P	phosphorus
PAO	phosphorus accumulating organisms
PE	population equivalent
PHF	peak hourly flow
PSLRP	Private Sewer Lateral Rehabilitation Program

List of Abbreviations (continued)

Abbreviation	Description
POTW	Publically Owned Treatment Works
PVC	polyvinyl chloride
PWWF	peak wet weather flow
RAS	return activated sludge
RBC	rotating biological reactor
SCADA	Supervisory Control and Data Acquisition
sf	square feet
SOR	standard oxygen requirement
SOUR	specific oxygen uptake rate
sq	square
SS	suspended solids
SSES	Sanitary Sewer Evaluation Study
SSO	sanitary sewer overflow
SV	seasonal variation
TDH	total dynamic head
TMDL	total maximum daily load
TSS	total suspended solids
USEPA	United States Environmental Protection Agency
UV	ultraviolet
VFD	variable frequency drive
VSS	volatile suspended solids
WAS	waste activated sludge
WLA	waste load allocation
WQBEL	water quality based effluent limit
WWTP	wastewater treatment plant
yr	year

EXECUTIVE SUMMARY

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EXECUTIVE SUMMARY

INTRODUCTION AND BACKGROUND

The City of St. Charles is served by two wastewater treatment facilities, the Main Wastewater Treatment Facility (MWWTF) and the West Side Water Reclamation Facility (WSWRF). In 2014, the NPDES Permit for the Main Wastewater Treatment Facility established an annual average phosphorus discharge limit of 1.0 mg/L. At this time, the MWWTF's process design does not include phosphorus removal capabilities. The facility's capacity is 9.0 Million Gallons per Day (MGD) or 90,000 PE and is receiving approximately 4.66 MGD.

Recognizing the need for improvements to meet the new permit limits, the Illinois EPA incorporated a compliance schedule into the NPDES Permit as a Special Condition. The compliance schedule established a timeline for the City of St. Charles to plan, design and construct the necessary improvements. The compliance schedule submittal requirements were as follows:

Table 1 | NPDES Permit Compliance Schedule for the MWWTF

Description of Milestone	Date
Interim Report on Phosphorus Removal Feasibility Report	June-15
Phosphorus Removal Feasibility Report (1.0 and 0.5 mg/L) Submittal	December-15
Progress Report on Phosphorus Reductions / Implementation Plan	June-16
Progress Report on Recommendations of Implementation Plan	December-16
Plans and Specifications Submitted	June-17
Progress Report on Construction	December-17
Complete Construction	June-18
Progress Report on Optimizing Treatment System	December-18
Achieve Annual TP Concentration and Loading Effluent Limits	June-19

The first step in the process was the development of the 2015 Facility Plan Update. A Facility Plan is a management and planning document used to identify, evaluate, and plan required wastewater facility improvements required to address the facilities' needs for the next twenty years. It provides an assessment of the collection and treatment systems' abilities to meet both current and future loads, flows and regulatory requirements and provides critical information for improvements to correct current or projected deficiencies. Facility Plan Updates are required by the Illinois Environmental Protection Agency (IEPA) for any wastewater improvements that change the treatment process or expand the capacity of the wastewater treatment plant. These reports are typically updated every five to ten years, or when significant changes in growth or regulatory requirements have occurred or are expected.

In 2002, the City updated its Facility Plan which identified the need for nitrification capabilities. The 2002 Facility Plan Update was approved by the Illinois EPA in January of 2003 and construction of the MWWTF 2002 Nitrification Improvements project began in November of 2003. The 2002 Nitrification Improvements project scope included the construction of 2.5 million gallons of aeration capacity and blower building, rehabilitation of the existing aeration basins, expansion of the RAS/WAS pump station, conversion of the existing first flush holding tank to an excess flow clarifier and the construction of an ultra violet disinfection system. The project was completed in July of 2005.

The 2002 Facility Plan Update also identified several rehabilitation and upgrade projects for the City's lift stations. One of the projects identified in the report was the rehabilitation of East Side Lift Station to increase the lift station's capacity to meet Peak Wet Weather Flow and eliminate by-pass practices. A second project identified in the report was the replacement of the aging mechanical fine screens at Riverside Lift Station. The City elected to address the recommendations for both lift stations under one project titled the Rehabilitation of East Side and Riverside Lift Stations. This project required an update to the Facility Plan, which was completed in 2009.

In addition to recommending the rehabilitation of the East Side and Riverside lift stations, the 2009 Facility Plan Update also identified the need for improved sludge handling infrastructure. The City elected to pursue address this need with the 2012 Main and Sludge Handling Building Improvements, which was completed in the fall of 2014.

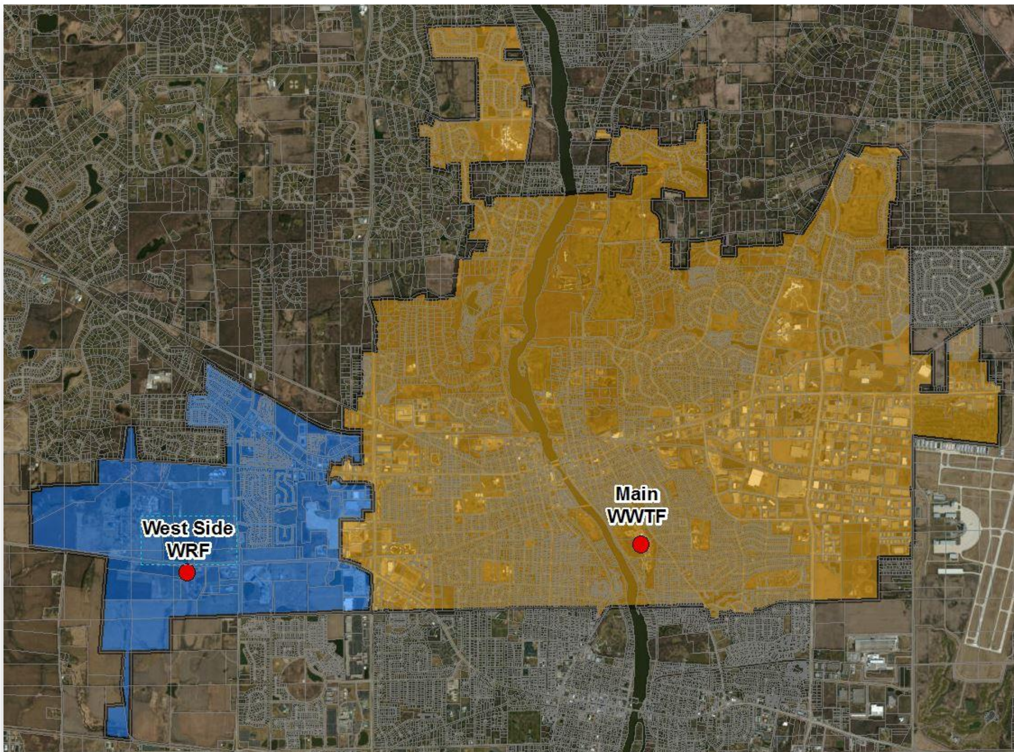
The purposes of this Facility Plan are to:

- Evaluate the adequacy of the existing collection and treatment facilities under the current flows, loads and regulatory requirements;
- Review the maintenance history and current condition of wastewater treatment units and lift stations and identify necessary repairs/replacements;
- Estimate the additional flows and loads associated with future growth within the planning area during the 20-year planning period;
- Summarize pending and potential future environmental regulations related to wastewater conveyance and treatment;
- Determine the impacts of future flows, loads and regulatory requirements on the existing system;
- Identify and evaluate alternatives to address both current and future deficiencies;
- Recommend cost effective alternatives; and
- Present costs, user fee analysis, implementation plans, cash flow projections and environmental impacts of the recommended alternatives.

THE COMMUNITY'S NEEDS

Wastewater treatment capacity is usually rated either in Millions of Gallons per Day (MGD), or Population Equivalents (PE). In order to estimate the industrial and commercial contributions to the wastewater load, we reviewed water usage records, the City's Comprehensive Land Use Plan, and the City's collection system maps. We also met with members of the City's Environmental Services, Utilities Billing, and Planning Divisions to arrive at a consensus for current, future and build-out population projections.

Exhibit 1 | Facility Planning Area (FPA)



The water usage within the Main WWTF's service area was 3.35 MGD. The residential users within the service area accounted for 2.02 MGD while the industrial/commercial users accounted for 1.34 MGD. Thus, the industrial portion of water usage is approximately 40%. Based on an estimated service area population of 29,924, the industrial/commercial contribution equates to an additional 19,841 PE, for a combined population equivalent of 49,764 PE.

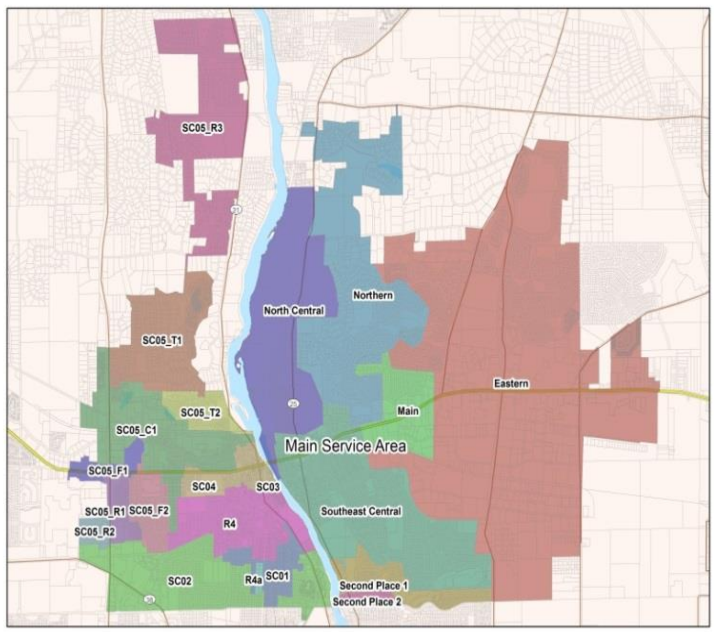
The future population equivalent, taking into account the projected population plus anticipated industrial and commercial loading, is 56,254 PE. Based on this number, it is not anticipated that the capacity needs of the service area will exceed 90,000 PE and therefore will not require expansion for the ultimate loading from the service area.

COLLECTION SYSTEM

The City of St. Charles wastewater collection system includes two service areas generally divided by Randall Road. The sanitary sewers west of Randall Road are tributary to the West Side Water Reclamation Facility (WRF). This service area is relatively new and the sewers have been constructed with modern materials, which minimize infiltration and inflow.

The sanitary sewer system east of Randall Road is tributary to the Main WWTF. The sewers within this collection system are of varying age and condition. As with many older collection systems, infiltration and inflow is a concern. The City of St. Charles has developed a rigorous maintenance program including flow monitoring, root cutting, grouting, sewer lining and other rehabilitation and replacement of the collection system. The City's goal is to eliminate basement back-ups and SSO's. The City has budgeted an additional $4.24 million for sanitary sewer projects within the five-year capital improvements program.

Exhibit 2 | Main WWTF Service Area Drainage Basins



The City of St. Charles' Finance Department maintains its GASB 34 Report, however, the collection system is not broken out by treatment facility. Therefore the actual value of this asset for the Main Service Area is not known. It has been estimated that the City currently maintains 172 miles of sanitary sewer mains (gravity and force main), as well as roughly 4,040 sanitary manholes and 13 lift stations in the Main Service Area. Using estimated replacement unit costs for sanitary sewer pipes and sanitary manholes, the City owns and maintains a $220 million dollar gravity collection system. Assuming 10% for contingency and 15% for design and administration, the replacement of the entire collection system is estimated to cost approximately $275 million. However, the majority of the collection system is not in need of replacement.

The service life of a collection system is approximately 75 years, and this life can be extended by approximately 25 years with ongoing maintenance and rehabilitation. Based on straight-line depreciation over this 100-year service life, it is recommended that the City reinvest $2,751,000 annually toward sanitary sewer collection system rehabilitation and replacement.

Approximately 20% of the collection system is already beyond its 75-year service life, and may be considered fully depreciated and in need of replacement. It is recommended that the City reinvest $1,403,000 annually toward the replacement of sewers that were installed before 1941 (as a portion of the annual reinvestment). It is also recommended that the remainder of the annual reinvestment be applied to the CMOM Program. There are several initial costs involved with starting up a program of this magnitude (within the first year of the program). This cost is estimated to be roughly $550,000. In order to sustain the long-term viability of the sewer utility, the City's sewer rehabilitation budget should be raised to the aforementioned level.

LIFT STATIONS

The City of St. Charles' Main Service Area includes thirteen lift stations, two of which are directly tributary to the headworks at the Main WWTF. The lift stations vary in age, however most were constructed between 1987 and 1997 as the City developed further north and east. The two main lift stations are Riverside Lift Station and East Side Lift Station. It should be noted that the figures in Table 2 do not include the engineering and contingencies that would be involved in a rehabilitation or replacement project.

Table 2 | Lift Station Asset Value

Lift Station	Equipment	Structure	Force Main	Totals
Riverside	$1,750,000	$2,000,000	$1,280,000	$5,030,000
East Side	$1,030,000	$1,500,000	$96,000	$2,626,000
7th & Division	$200,000	$145,000	$109,000	$454,000
Washington Ave.	$50,000	$50,000	$73,000	$173,000
Country Club	$200,000	$155,000	$129,000	$484,000
Pheasant Run Trails	$210,000	$185,000	$292,000	$687,000
Royal Fox #2	$220,000	$185,000	$498,000	$903,000
Royal Fox #1	$210,000	$165,000	$358,000	$733,000
Woods of Fox Glen	$210,000	$185,000	$566,000	$961,000
Kingswood	$210,000	$185,000	$197,000	$592,000
Wild Rose	$200,000	$160,000	$14,000	$374,000
Red Gate	$210,000	$185,000	$311,000	$706,000
Oak Crest	$200,000	$155,000	$74,000	$429,000
Totals	$4,900,000	$5,255,000	$3,997,000	$14,152,000
Design Life, Years	20	50	50	
Annual Replacement	$245,000	$105,100	$79,940	**$430,040**

The value of the City's lift station and force main assets is approximately $14,152,000. Based on a straight-line depreciation over the design life of the equipment, structures and force mains, the City should be reinvesting around $430,000 annually toward maintaining and replacing these assets within the Main Service Area. Operational staff has indicated that most of the recommended improvements could be accomplished utilizing in-house resources. The more significant improvements have been broken into capital projects and recommended budgets have been provided. These projects should be incorporated into the City's Capital Improvements Program.

Table 3 | Lift Station Capital Improvements Summary

RIVERSIDE LIFT STATION REPLACEMENT	$5,742,112
7TH & DIVISION LIFT STATION REPLACEMENT	$597,200
COUNTRY CLUB LIFT STATION REHABILITATION	$637,625
WILD ROSE LIFT STATION REPLACEMENT	$620,388
TOTAL LIFT STATION CAPITAL IMPROVEMENTS	$7,597,325

EXISTING MAIN WASTEWATER TREATMENT FACILITY

The City of St. Charles original wastewater treatment facility was located along the banks of the Fox River near the Riverside Lift Station. In the early 1930's, a new plant was constructed up the hill on what is now the wastewater treatment facility site. Since its construction, the facility has been upgraded numerous times. The City has traditionally performed rehabilitation and replacement of aging equipment through the operation and maintenance budget, and has performed major process upgrades through the Illinois State Revolving Fund program (SRF).

The 2002 Nitrification Improvements included the construction of a single stage nitrification process to meet the new ammonia nitrogen limits, excess flow improvements consisting of the conversion of the existing first flush holding tank to an excess flow clarifier, and construction of an ultra violet disinfection system for use with the normal process flow (to allow the chlorine contact tanks to be used for excess flow only). This project was funded through the Illinois SRF.

In 2009, the City of St. Charles upgraded the East Side Lift Station and Riverside Lift Station. The improvements to East Side Lift Station included replacement of all mechanical and electrical components including the fine screen, pumps, piping and controls. The rehabilitation to Riverside Lift Station was limited to screen, valve and variable frequency drive replacement. This project was funded through the Illinois SRF.

In late 2011, an assessment of the Main WWTF processes and infrastructure was completed. This assessment identified the need to structurally rehabilitate or replace the Main Sludge Handling Building. The City of St. Charles proceeded with replacement. The project was funded through the Illinois SRF, construction was completed in 2014.

The City has completed an audit of each unit process, its capacity, age and condition and developed a series of recommended capital improvements. It is recommended that these projects be incorporated into the City's Capital Improvements Program.

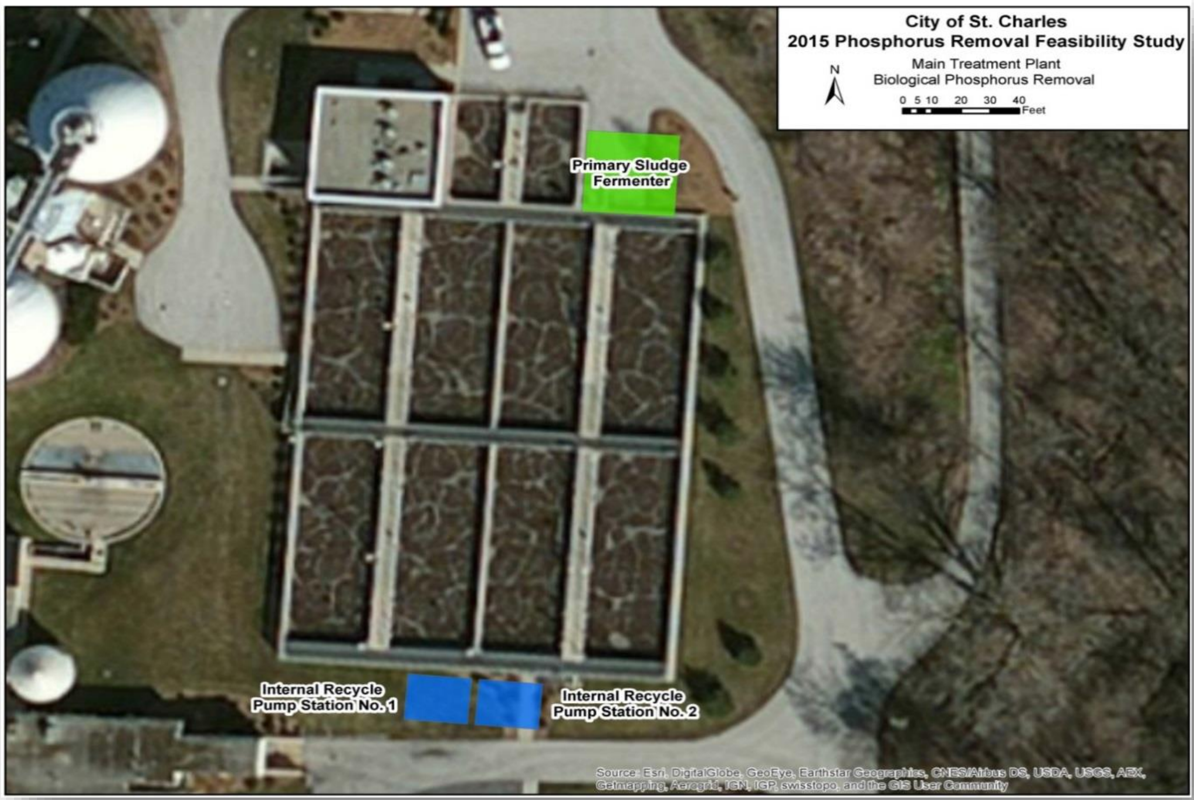
Table 4 | Main Wastewater Treatment Facility Capital Improvements Summary

PRIMARY CLARIFIER REHABILITATION	**$558,532**
ANAEROBIC DIGESTER REHABILITATION	**$7,960,605**
UV DISINFECTION REHABILITATION	**$2,576,218**
EXCESS FLOW FILTRATION	**$8,048,053**
PHOSPHORUS REMOVAL - BIO-P	**$7,370,208**
TOTAL MAIN WWTF CAPITAL IMPROVEMENTS	**$26,513,616**

Alternatives for Nutrient Removal

As identified on page 1, the City's revised NPDES permit required an evaluation and feasibility study of the alternatives for achieving an effluent phosphorus limit of 1.0 and 0.5 mg/L. During the development of this Facility Plan, numerous alternatives for biological phosphorus and nitrogen removal were evaluated. Preference was given to alternatives that accomplish removal of both nutrients while maximizing the strengths of the existing infrastructure. After careful consideration, the A^2/O process was selected as the preferred configuration for biological nutrient removal (Bio-P).

Exhibit 3 | Proposed Layout for A^2/O Process



Chemical precipitation of phosphorus (Chem-P) may be accomplished within either the primary or secondary treatment process. Jar testing was performed with ferric chloride and alum to determine the dosage requirements of these two metal salts. Based on the results, ferric chloride was the most efficient. The analysis included development of capital and operational costs for implementation of this alternative.

The alternatives for chemical and biological phosphorus removal at three possible effluent TP limits are compared below. The "increased annual operational costs" are in addition to the City's current budget for sludge disposal and chemical material. These values are therefore not representative of the total cost of operations for the Main WWTF. These costs were calculated over a 20-year period to project the net present value with an average influent of 7 MGD.

Table 5 | Probable Cost Analysis – 20-Year Period – 7 MGD

EFFLUENT TP	PROCESS	PRESENT VALUE OF CAPITAL COST	INCREASED ANNUAL OPERATIONAL COST	PRESENT VALUE OF OPERATIONAL COST	NET PRESENT VALUE
1.0 mg/L	CHEM-P	$1,946,349	$243,633	$4,872,656	$6,819,005
	BIO-P	$7,370,208	$42,280	$845,597	$8,215,805
0.5 mg/L	CHEM-P	$9,994,403	$358,608	$7,172,156	$17,166,559
	BIO-P	$15,418,261	$157,255	$3,145,097	$18,563,358
0.3 mg/L	CHEM-P	$16,099,800	$531,070	$10,621,406	$26,721,206
	BIO-P	$21,523,659	$329,717	$6,594,347	$28,118,006

The City has elected to pursue biological phosphorus removal to comply with its NPDES permit limit of 1.0 mg/L. Implementation of Bio-P will require a capital investment of approximately $7.4 Million. The City of St. Charles intends on funding the project through the Illinois SRF and to service the debt through user fees. During evaluation of the existing infrastructure, the City identified rehabilitation of the anaerobic digesters as a top priority. Therefore, it is recommended that the City pursue financing for implementation of phosphorus removal and anaerobic digester rehabilitation. It is also recommended that the improvements be designed, permitted and implemented as one construction project. The NPDES permit requires that the construction of the phosphorus removal improvements be completed by June of 2018. The following schedule is intended to meet those requirements.

Table 6 | Implementation Schedule

Description of Milestone	Date
Interim Report on Phosphorus Removal Feasibility Report	Completed
Phosphorus Removal Feasibility Report (1.0 and 0.5 mg/L) Submittal	Pending
Begin Design of Improvements	September-15
Plans and Specifications Submitted	March-16
IEPA Loan Application Submittal	March-16
Advertise for Bid	July-16
IEPA Loan Agreement Approval	September-16
Start Construction	October-16
Complete Construction	June-18

IMPLEMENTATION PLAN

The Main WWTF's NPDES permit requires that the City implement a CMOM, and upgrade the existing facility to comply with effluent phosphorus limits. Recommendations within Section 3 included budgets for sanitary sewer replacement and the CMOM program. The lift station O&M costs were identified in Section 4 for the Main and West Facility Plan Updates. The O&M costs for the Main WWTF and West Side WRF remain unchanged until after implementation of phosphorus removal and capacity expansion upgrades, respectively. The current need for O&M of the City's wastewater infrastructure is estimated to be $11.76 Million. The City currently has an O&M budget of approximately $8.46 Million.

Table 7 | Operation and Maintenance for Phased Implementation Plan

Description	'15-'16	'16-'17	'17-'18	'18-'19*	'19-'20	'20-'21** to '29-'30
COLLECTION SYSTEM – CMOM	$1.90	$1.35	$1.35	$1.35	$1.35	$13.48
LIFT STATIONS – WEST	$0.07	$0.07	$0.07	$0.07	$0.07	$0.65
LIFT STATIONS – MAIN	$0.43	$0.43	$0.43	$0.43	$0.43	$4.30
WEST SIDE WRF O&M	$0.72	$0.75	$0.77	$0.79	$0.81	$9.66
MAIN WWTF O&M	$7.24	$7.46	$7.68	$7.96	$8.20	$96.77
TOTAL PROPOSED O&M	$11.76	$11.45	$11.70	$11.99	$12.26	$124.86
CURRENT O&M BUDGET (3% increase)	$8.46	$8.72	$8.98	$9.25	$9.52	$112.16

Projected costs are in millions of dollars

* NOTE: In 2018, the operational cost increase for biological phosphorus removal at the Main WWTF will increase as projected in Section 6 of the Main Facility Plan Update.

** NOTE: In 2021, the operational cost increase for biological phosphorus removal at the West Side WRF will increase as projected in Section 6 of the West Side Facility Plan Update.

The complete list of all capital improvements recommended in this report, as well as the recommended capital improvements contained in the West Side WRF Facility Plan Update, is provided below.

Table 8 | Capital Improvements Summary

RIVERSIDE LIFT STATION REPLACEMENT	$5,742,112
7TH & DIVISION LIFT STATION REPLACEMENT	$597,200
COUNTRY CLUB LIFT STATION REHABILITATION	$637,625
WILD ROSE LIFT STATION REPLACEMENT	$620,388
WEST SIDE WRF EXPANSION - PHASE IIIA	$8,605,278
WEST SIDE WRF EXPANSION - PHASE IIIB	$3,607,067
PRIMARY CLARIFIER REHABILITATION	$558,532
ANAEROBIC DIGESTER REHABILITATION	$7,960,605
UV DISINFECTION REHABILITATION	$2,576,218
EXCESS FLOW FILTRATION	$8,048,053
BIOLOGICAL PHOSPHORUS REMOVAL	$7,370,208
TOTAL CAPITAL IMPROVEMENTS	$46,323,286

The City's existing debt service equates to approximately $1.73 Million. The existing debt service and recommended capital improvements are included in Table 8. City staff determined the priority and schedule for each capital project. It is recommended that the City conduct a study to address user rates and the revenue required to support operations and maintenance, as well as the capital improvements program.

Table 9 | Debt Service for Capital Improvements – Phased Implementation Plan

Description	'15-'16	'16-'17*	'17-'18	'18-'19	'19-'20	'20-'21** to '29-'30
EXISTING DEBT SERVICE						
WEST SIDE WRF PH. II EXPANSION	$0.47	$0.47	$0.47	$0.47	$0.47	$0.47
2002 NITRIFICATION IMPROVEMENTS	$0.65	$0.65	$0.65	$0.65	$0.65	$3.26
EAST SIDE & RIVERSIDE L.S. REHAB.	$0.10	$0.10	$0.10	$0.10	$0.10	$0.49
2012 MAIN AND S.H.B.	$0.51	$0.61	$0.61	$0.61	$0.61	$3.07
PROPOSED DEBT SERVICE						
COLL. SYSTEM – REPLACEMENT	$1.40	$1.40	$1.40	$1.40	$1.40	$7.02
RIVERSIDE LIFT STATION				$0.19	$0.38	$1.88
7TH & DIVISION LIFT STATION					$0.60	
COUNTRY CLUB LIFT STATION				$0.64		
WILD ROSE LIFT STATION						$0.62
WEST SIDE WRF PH. IIIA EXPANSION						$0.56
WEST SIDE WRF PH. IIIB EXPANSION						$0.12
PRIMARY CLARIFIER REHAB.		$0.56				
ANAEROBIC DIGESTER REHAB.		$0.00		$0.26	$0.52	$2.62
UV DISINFECTION REHAB.						$0.17
EXCESS FLOW FILTRATION						
PHOSPHORUS REMOVAL - BIO-P		$0.00		$0.24	$0.48	$2.42
TOTAL DEBT SERVICE	$3.13	$4.73	$3.23	$4.56	$5.21	$46.50

Projected costs are in millions of dollars

* NOTE: In 2016, the design engineering is projected to occur for the biological phosphorus removal and anaerobic digester rehabilitation project. This will require a projected cash flow of approximately $950,000 this year. The project may be funded with a SRF loan, which will include a repayment to the City for this cost. This repayment is projected to occur within the same fiscal year, resulting in a net cash flow of zero.

** NOTE: In 2021, the design engineering is projected to occur for the UV disinfection rehabilitation project. This will require a projected cash flow of approximately $160,000 this year. The project may be funded with a SRF loan, which will include a repayment to the City for this cost. This repayment is projected to occur within the same fiscal year, resulting in a net cash flow of zero.

SECTION 1

INTRODUCTION AND BACKGROUND

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1. INTRODUCTION AND BACKGROUND

1.1 GENERAL BACKGROUND

The City of St. Charles is located along the Fox River in central Kane County approximately 35 miles west of downtown Chicago. The City is bordered by the Village of South Elgin to the north, the City of West Chicago to the east, the City of Geneva to the south and the Village of Campton Hills to the west.

The City owns and operates a sanitary sewer collection system and two wastewater treatment facilities: the Main Wastewater Treatment Facility and the West Side Water Reclamation Facility. The collection system tributary to the Main Wastewater Treatment Facility (Main WWTF) consists of approximately 152 miles of sanitary sewers, 5 miles of force main and 13 lift stations. The Main WWTF is located at the Public Works Facility, 1405 S. 7th Avenue on the eastern shore of the Fox River, approximately nine-tenths of a mile south of the Illinois Route 64 Bridge. The St. Charles Facility Planning Area (FPA) is comprised of approximately 10,340 acres, of which 8,317 acres is tributary to the Main WWTF.

The Main WWTF plant has a design average treatment capacity of 9.0 million gallons per day (MGD). The facility generally serves the community's wastewater needs east of Randall Road and discharges to the Fox River.

The West Side Water Reclamation Facility (West Side WRF) is located at 3803 Illinois Route 38. The West Side WRF has a design average treatment capacity of 0.70 MGD. The facility generally serves the community's wastewater needs west of Randall Road and discharges to Mill Creek. The West Side WRF and the area that it serves is presented in a separate report titled "City of St. Charles 2015 Facility Plan Update – West Side Water Reclamation Facility".

The City's National Pollutant Discharge Elimination System (NPDES) Permit for the Main WWTF (Permit No. IL0022705), as administered by the Illinois Environmental Protection Agency (IEPA), was reissued on December 1st, 2014. The new permit incorporates special conditions, including the monitoring of effluent phosphorus and nitrogen and an annual average limit of 1 mg/L for effluent phosphorus. The NPDES permit is included as Appendix A.

1.2 STUDY PURPOSE AND SCOPE

The purpose of this study is two-fold and will include a comprehensive Facility Plan Report as well as a Phosphorus Removal Feasibility Report. The intent of the reports is to identify process upgrades and rehabilitation projects which should be incorporated into the City's five year Capital Improvements Program, as well as address long-range needs of the community.

1.2.1 Facility Plan Report

A Facility Plan Report (FPR) is a management and planning document used to identify, evaluate, and plan required wastewater facility improvements. It provides an assessment of the collection and treatment systems' abilities to meet both current and future loads, flows and regulatory requirements and provides critical information for improvements to correct current or projected deficiencies. FPRs are required by the Illinois Environmental Protection Agency (IEPA) for any wastewater improvements that change the treatment process or expand the capacity of the wastewater treatment plant.

FPRs are typically updated every five to ten years, or when significant changes in growth or regulatory requirements have occurred or are expected. In 2002, the City updated its FPR which identified the need for nitrification capabilities. In 2009, the City updated its FPR again which identified the need for improved sludge handling infrastructure.

The purposes of this FPR update are to:

- Evaluate the adequacy of the existing collection and treatment facilities under the current flows, loads and regulatory requirements;
- Review the maintenance history and current condition of wastewater treatment units and lift stations and identify requirement maintenance repairs/replacements;
- Estimate the additional flows and loads associated with future growth within the planning area during the 20-year planning period;
- Summarize pending and potential future environmental regulations related to wastewater conveyance and treatment;
- Determine the impacts of future flows, loads and regulatory requirements on the existing system;
- Identify and evaluate alternatives to address both current and future deficiencies;
- Recommend cost effective alternatives; and
- Present costs, user fee analysis, implementation plans, cash flow projections and environmental impacts of the recommended alternatives.

1.2.2 Phosphorus Removal Feasibility Study

The City of St. Charles has been a longstanding member of the Fox River Study Group, which has been evaluating the water quality impairments associated with the river since the early 2000's. In recent years, the IEPA has been receiving increased pressure from the USEPA to implement stricter nutrient standards on rivers and streams in Illinois which are impaired for dissolved oxygen. The Fox River Study Group, collectively with the municipalities, has negotiated language for the special conditions to be incorporated into the next round of NPDES permits.

The City of St. Charles was the first POTW to receive a draft permit with the special conditions incorporated. The City has reviewed and discussed these issues with the IEPA. The special conditions include language that requires the submittal of a Feasibility Study to lower the annual average effluent phosphorous concentration to 1.0 mg/L as well as 0.5 mg/L. This Feasibility Study must be completed and submitted to the IEPA within twelve months after issuance of the permit. Another special condition requires that the City of St. Charles study, design and construct improvements which will allow the plant to achieve a 1 mg/L effluent phosphorous limit within 54 months after issuance of the permit. In addition to these requirements, TAI will be considering the possible impacts of lowering the effluent phosphorous concentration to less than 0.5 mg/L.

The purposes of the Phosphorus Removal Feasibility Study are:

- Review the NPDES permit issued to the City for the Main WWTF
- Develop and evaluate chemical and biological phosphorus removal alternatives for achieving annual average effluent TP limits of 1.0 and 0.5 mg/L
- Present costs to implement and operate the selected alternative(s)

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SECTION 2

THE COMMUNITY'S NEEDS

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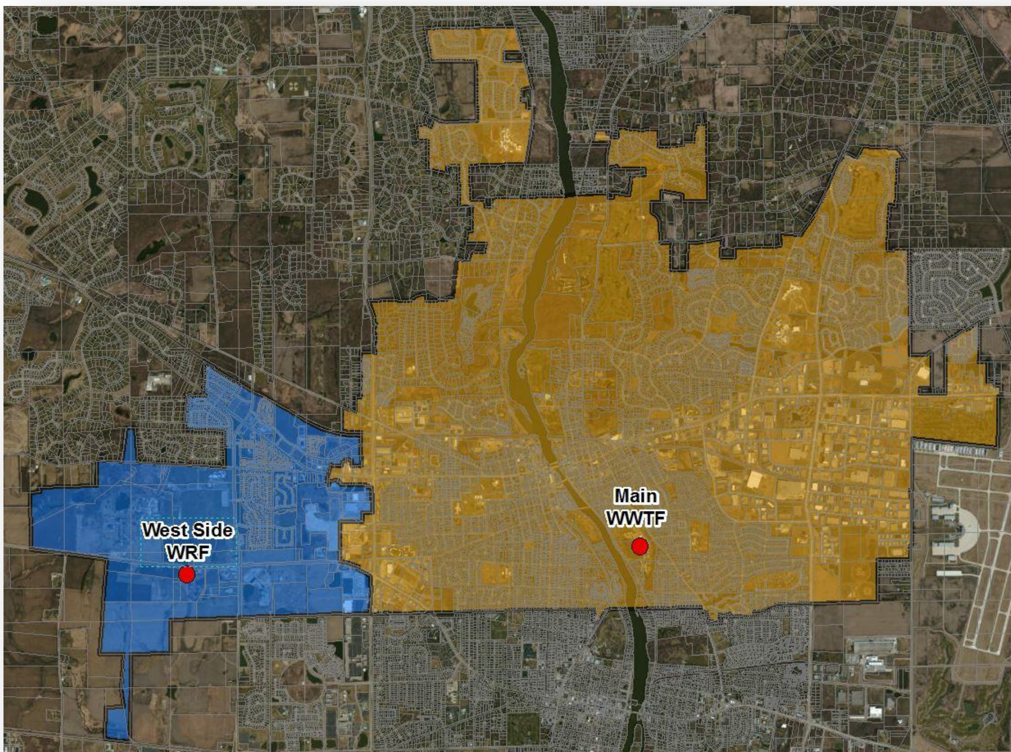
2. THE COMMUNITY'S NEEDS

2.1 GENERAL BACKGROUND

The City of St. Charles is situated along the Fox River and its location has made it attractive to residential, industrial and commercial development. The Main Wastewater Treatment Facility (WWTF) is located on the east bank of the Fox River and was originally built in the 1920's when the community was relatively small. Since that time the community has grown substantially. However, the treatment plant is restricted to its original site with limited room for expansion.

Exhibit 2-1 | Facility Planning Area (FPA)



The City of St. Charles FPA is bounded on the south by Geneva, on the north by South Elgin, and West Chicago to the east. The FPA is shown in the figure above, with the Main WWTF Service Area shown in orange. The City of St. Charles has grown from a community of 17,492 in 1980 to 27,910 people in 2001 to 32,974 people in 2010, of which 29,941 live in the Main WWTF's service area. The City Council has not approved any new developments within this service area for construction. The remaining undeveloped properties within the St. Charles FPA have been assigned a land use and density. Therefore, the projected population equivalents should be accurate as the City has a strong commitment to abiding by the Land Use Plan.

2.2 EXISTING POPULATION PROJECTIONS AND WATER DEMANDS

In order to accurately evaluate the current and future wastewater capacity needs, we established the current number of users, the users which are permitted or approved but not currently contributing and the potential population from the remaining open lands in the FPA.

The existing, approved, and future population equivalents were established by reviewing the City's records of building permits, water and sewer billing records, wastewater treatment plant flow monitoring records, approved development plans and the Land Use Plan. The City Planning Department provided support and information to establish the ultimate population equivalents.

In 2010, the U.S. Census Bureau estimated that the City of St. Charles served a total residential population of 32,974. The residential water usage based on billing records was 2,221,446 gallons per day (gpd). This residential usage consumed by an estimated 32,974 residents equates to 67.37 gallons per capita per day (gcd).

During 2011 and 2013, the City of St. Charles billed users an average of 3.35 MGD for water use, while the wastewater treatment facility received an average flow of 4.66 MGD (data from 2012 was disregarded due to drought conditions). The current population equivalents were estimated by breaking down water billing by classifications:

Table 2-1 | Current Population, Water Demands and Wastewater Flows

	Residential	Non-Residential	Total
Number of Customers	9,772	1,167	10,939
Population Equivalents	29,924 PE	19,841 PE	49,765 PE
Water Usage Billed	2.02 MGD	1.34 MGD	3.35 MGD
Water Usage / PE	67.37 gcd	67.37 gcd	67.37 gcd
Wastewater Received	2.80 MGD	1.86 MGD	4.66 MGD
Wastewater / PE	93.58 gcd	93.58 gcd	93.58 gcd

The future population projection, which is the ultimate buildout of properties within the FPA, was developed by assigning PE values to the planned development and remaining open lands in accordance with the Land Use Plan.

Future Population Equivalent

Total Current PE	49,765	PE
Additional PE at Build-Out of Service Area	6,489	PE
Total Future PE	56,254	PE

Projected 2030 Population Equivalent for the FPA is 56,254 PE. It should be noted that population equivalent resulting from the ultimate buildout will not exceed the present IEPA rated population equivalent of the Main WWTF which is 90,000 PE.

2.3 PREVIOUS STUDIES & REPORTS

In 2002, TAI completed a Facility Plan Update for the City of St. Charles. That report diagnosed the City's background, including population changes and a detailed history of the existing collection system and treatment facilities. As a result of the recommendations in that update, the City took on several projects: the addition of aeration tanks for nitrification in order to comply with more stringent nutrient limitations that would be enforced by the IEPA for the City's next NPDES permit; the expansion of the excess flow capacity to meet IEPA design standards; and the rehabilitation of the East Side and Riverside lift stations, which was combined into one project that was completed in 2010. The 2002 report delineated basins for the collection system on the east side of the Fox River that are a part of the Main WWTF's FPA. These same basins were used in the development of this report, and are detailed in Section 3.

In 2009, RJN Group submitted to the City a flow monitoring report for the areas on the west side of St. Charles tributary to the Main WWTF. The goal of that study was to identify areas of excessive infiltration and inflow, as well as to determine the average daily flows, peak dry-weather flows, and wet weather flows for 1-year and 5-year storms. Using the results from that study, the City plans to perform field studies on the sanitary sewer lines that were identified as the most critical first, and incorporate those studies into their Capital Improvements Program. The 2009 report delineated basins for the collection system on the west side of the Fox River that are a part of the Main WWTF's FPA. These same basins were used in the preparation of this report, and are detailed in Section 3.

2.3.1 Infiltration

The USEPA considers average annual infiltration to be excessive if it exceeds 50 gcd. The current estimated population equivalent within the Main WWTF's service area is 49,764 PE. We have estimated the average amount of infiltration by comparing the water usage records with the plant effluent records. The average water usage per population equivalent is 67.37 gcd. The average wastewater received per population equivalent is 93.58 gcd. The annual average I/I is approximately 26.21 gcd, which is half of the USEPA's criteria. This is a significant improvement from the previous studies in 2002 and 2009 which outlined an estimated infiltration of 49 and 46 gcd, respectively.

Based on wastewater flow data from the summer of 2012, the base flow during drought conditions where infiltration is minimized is about 63 gcd. Based on total current PE and the USEPA definition of excess infiltration (120 gcd during periods of high groundwater), the Main WWTF experiences excess infiltration when flows exceed 5.97 MGD.

2.3.2 Inflow

The issue of inflow has become more sensitive over the last few decades due to unusually heavy rainfall events that resulted in flooding of some residential basements. The 10-year peak wet weather flow presented in a system-wide capacity study prepared in 1996 was estimated to be 35.7 MGD, or 7.56 times the average daily flow. The estimated 5-year peak wet weather flow stated in the 2009 Report (from the west side of the Fox River alone) was 11.49 MGD, or 6.68 times the average daily flow from this area. The USEPA considers inflow to be excessive in separate sanitary sewer systems if the total flow, water usage plus infiltration plus inflow, exceeds 275 gcd. Based on the 1996 Report for the entire system, the estimated 10-year Peak Wet Weather Flow was almost 1,000 gallons per day per PE. Based on the 2009 RJN report for the west side of town, the estimated 5-year peak wet weather flow equates to 738 gallons per day per PE. Both were above the USEPA recommended standard.

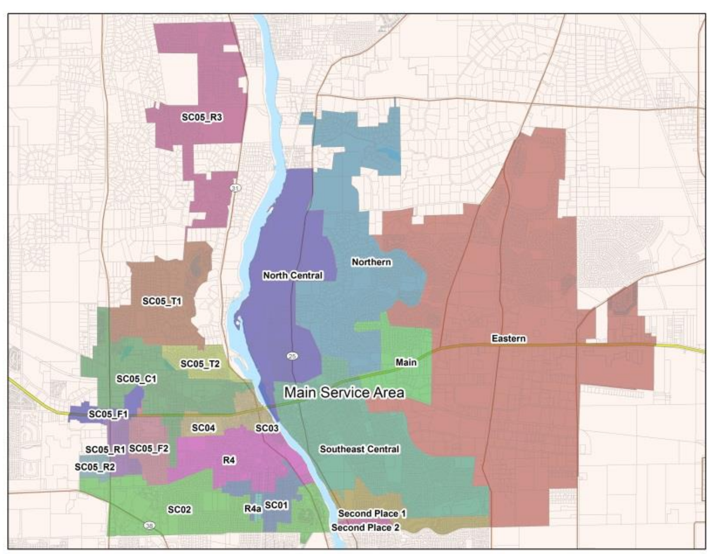
Based on wastewater flow data from the summer of 2012, the base flow during drought conditions where infiltration is minimized is about 63 gcd. Based on total current PE and the USEPA definition of excess infiltration (275 gcd during storm events where there are no basement back-ups), the Main WWTF experiences excess inflow when flows exceed 13.68 MGD.

2.4 FUTURE POPULATION PROJECTIONS

In order to accurately evaluate the City's current and future wastewater flows for the Main WWTF Service Area, the following data was reviewed and established:

- Current number of users.
- Estimated future users (determined from information regarding developments currently under construction or approved by the City of St. Charles).
- Potential number of users from the remaining undeveloped properties located within the boundaries of the City's current and future service area (based on the Land Use Plan).

Exhibit 2-2 | Main Service Area Basins



The existing, approved/permitted, and potential population equivalents were established by reviewing the City's detailed water and sewer billing records, wastewater treatment plant flow monitoring records, approved development plans, and the City's Comprehensive Land Use Plan. Analysis of the projected land use was the basis for developing future population projections. The City of St. Charles' Comprehensive Plan indicates future residential, commercial, and industrial uses.

The Main Service Area currently contains fourteen drainage basins. Each drainage basin was analyzed to establish the 2013 Conditions and Build-Out Population Equivalents.

2.4.1 Eastern Drainage Basin

The Eastern Basin includes 2,672 acres with a broad mix of institutional, office, industrial, commercial and residential development. Areas included within this basin are bordered by the Royal Fox subdivision to the north, by Division Street to the south, and include areas from roughly Dunham Road to the eastern City limits. The collection system in this area is tributary to the Southeast Central Basin and ultimately the East Side Lift Station.

The Eastern Drainage Basin is the largest in the City, and has been outlined by the City as a growth area. Both residential and commercial development has been slated for this area, as shown. Vacant residental lots are shown in red, potential residential development in yellow and orange, and industrial/commercial/retail shown in pink, purple, blue, and green. The development will add approximately 1,893 Non-Residential PE and 2,079 Residential PE, for a total additional development of 3,972 PE.

Exhibit 2-3 | Eastern Drainage Basin

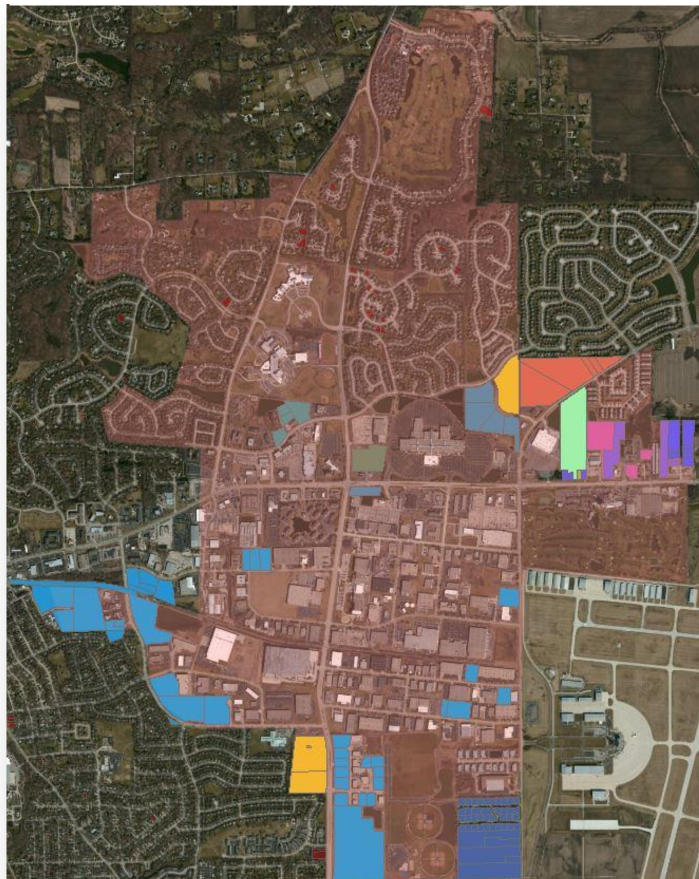


Table 2-2 | Eastern Basin Population and Flow Projections

Description	2013 PE	Build-Out PE
Eastern Service Area	17,643	**21,615**
Average Daily Flow (MGD)	1.65	**2.05**
***Existing Flows based on 93.58 gpd/ PE** ****Future Flows Based on IEPA 100 gpd/PE**		

2.4.2 North Central Drainage Basin

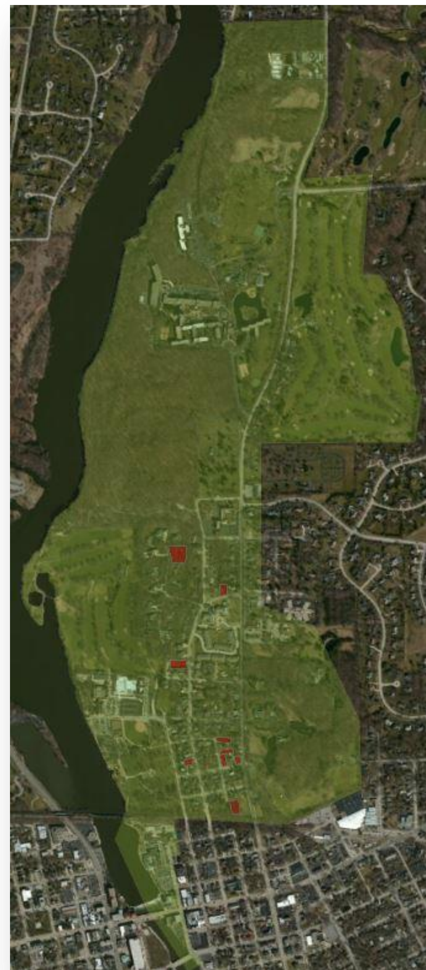
The North Central Basin is bordered on the west by the Fox River, from Illinois Route 64 to the northern limits of the City. The basin includes 598 acres of commercial, institutional and residential development. The collection system in this area was constructed from the early 1930's to the early 1990's.

The majority of the North Central Basin has been developed, no major development is anticipated. The North Central Basin has approximately ten residential lots that are vacant (shown in red) and have the potential for being developed. If the residential lots were to be built-out, an additional 32 PE is anticipated. The additional PE would bring the basin to a total of 1,832 PE.

Table 2-3 | North Central Drainage Basin Population and Flow Projections

Description	2013 PE	Build-Out PE
North Central Service Area	1,800	**1,832**
Average Daily Flow (MGD)	0.17	**0.17**
***Existing Flows based on 93.58 gpd/ PE** ****Future Flows Based on IEPA 100 gpd/PE**		

Exhibit 2-4 | North Central Drainage Basin



2.4.3 Southeast Central Drainage Basin

The Southeast Central Basin includes 763 acres of dense commercial, residential and light industrial development. The collection system is tributary to both of the lift stations serving the Main WWTF. 234 acres of the Southeast Basin are tributary to the Riverside Lift Station. The remaining 481 acres are tributary to the East Side Lift Station.

Exhibit 2-5 | Southeast Central Drainage Basin

The Southeast Central Basin has been identified as a growth area for Non-Residential development. The additional development outlined equates to an additional 761 PE, and is indicated below in blue. In addition to the additional development a few lots have been identified as vacant. The vacant lots (shown in red) have the potential of adding another 88 PE, bringing the total build-out to 7,362 PE.

Table 2-4 | Southeast Central Drainage Basin

Description	2013 PE	Build-Out PE
Southeast Central Service Area	6,514	**7,362**
Average Daily Flow (MGD)	0.61	**0.69**
***Existing Flows based on 93.58 gpd/ PE** ****Future Flows Based on IEPA 100 gpd/PE**		

2.4.4 Northern Drainage Basin

The Northern Basin includes 599 acres of circa 1980's residential development. The basin includes two pump stations and extends from Illinois Route 64 north to Army Trail Road, covering 822 total acres.

The Northern Drainage Basin is not anticipated to have any major development, the majority of the basin has been developed. The drainage basin does however have approximately four residential lots (shown in red) that could add a total of 14 additional PE, bringing the total equivalent population at buildout to 3,782 PE.

Exhibit 2-6 | Northern Drainage Basin

Table 2-5 | Northern Central Basin Population and Flow Projections

Description	2013 PE	Build-Out PE
Northern Service Area	3,768	**3,782**
Average Daily Flow (MGD)	0.35	**0.35**
*Existing Flows based on 93.58 gpd/ PE **Future Flows Based on IEPA 100 gpd/PE		

2.4.5 R4 Drainage Basin

R4 and R4a Basins are on the City's southwest side. R4 includes area around Prairie Street from 13th Street to the west bank of the Fox River. The basin includes 286 acres and serves 1,988 PE.

Exhibit 2-7 | R4 Drainage Basin

Future development in the R4 Basin is limited to vacant lots. Five lots have been identified as vacant (shown in red) within the basin and have the ablity to add an additional 18 PE. The additional PE would bring the total equivalent population at build-out to 2,006 PE.

Table 2-6 | R4 Basin Population and Flow Projections

Description	2013 PE	Build-Out PE
R4 Service Area	1,988	**2,006**
Average Daily Flow (MGD)	0.19	**0.19**
***Existing Flows based on 93.58 gpd/ PE** ****Future Flows Based on IEPA 100 gpd/PE**		

2.4.6 SC01 Drainage Basin

The SC-01 Basin is on the City's southwest side, and spans from the St. Charles/Geneva border to Mosedale Street on the north. The basin includes 99 acres and serves 726 PE.

Future development in the SC01 Basin is limited to vacant lots. Six lots have been identified as vacant (shown in red) within the basin and have the ability to add an additional 21 PE. The additional PE would bring the total equivalent population at build-out to 747 PE.

Exhibit 2-8 | SC01 Drainage Basin

Table 2-7 | SC01 Basin Population and Flow Projections

Description	2013 PE	Build-Out PE
SC01 Service Area	726	**747**
Average Daily Flow (MGD)	0.07	**0.07**
***Existing Flows based on 93.58 gpd/ PE** ****Future Flows Based on IEPA 100 gpd/PE**		

2.4.7 SC02 Drainage Basin

SC-02 is on the City's southwest side. The basin includes 458 acres and serves 4,040 PE. This basin is directly tributary to the South Siphon under the Fox River and ultimately to the Riverside Lift Station.

The City has identified the potential for additional growth within the SC-02 Basin. Both residential and commercial developments have been outlined for this area (shown in green and purple respectively). Nine lots have been identified as vacant in SC02 (shown in red). The nine vacant lots have the potential to add 32 additional PE, while the additional development outlined could contribute abother 516 PE, bringing the total equivalent population at build out to 4,556 PE.

Table 2-8 | SC02 Basin Population and Flow Projections

Description	2013 PE	Build-Out PE
SC02 Service Area	4,040	**4,556**
Average Daily Flow (MGD)	0.38	**0.43**
***Existing Flows based on 93.58 gpd/ PE** ****Future Flows Based on IEPA 100 gpd/PE**		

Exhibit 2-9 | SC02 Drainage Basin

2.4.8 SC05_C1 Drainage Basin

SC-05 is the designation provided to this drainage basin in the 1996 system-weide capacity study. This basin includes eight sub-basins, each of which were treated independently in the 2009 report.

The SC05_C1 Basin is anticipated to have future development. The growth includes buildout of vacant lots and residential/commercial development. Within the basin 11 lots have been identified as vacant (shown in red) and have the potential to add an additional 39 PE. In addition the residential and commercial (shown in blue and brown respectively) will add an additional 565 PE, bringing the total equivalent population at build-out to 3,157 PE.

Exhibit 2-10 | SC05_C1 Drainage Basin

Table 2-9 | SC05_C1 Central Basin

Description	2013 PE	Build-Out PE
SC05_C1 Service Area	2,553	**3,157**
Average Daily Flow (MGD)	0.24	**0.30**
***Existing Flows based on 93.58 gpd/ PE** ****Future Flows Based on IEPA 100 gpd/PE**		

2.4.9 SC05_F2 Drainage Basin

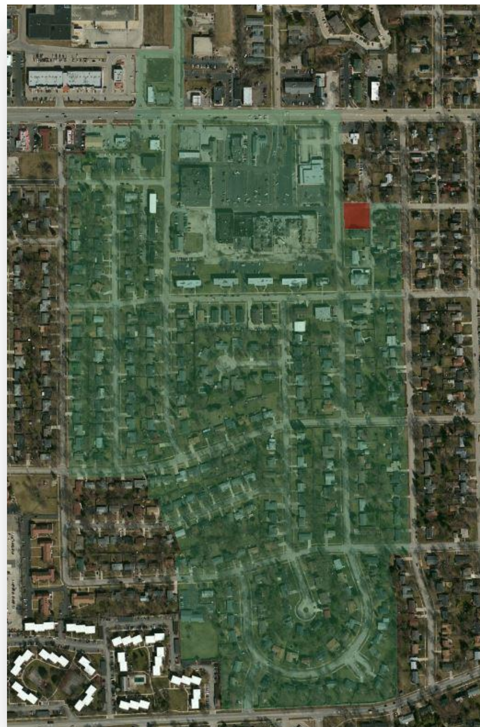
SC-05 is the designation provided to this drainage basin in the 1996 system-wide capacity study. This basin includes eight sub-basins, each of which were treated independently in the 2009 report.

Future development in the SC05_F2 Basin is limited to vacant lots. One lot has been identified as vacant (shown in red) within the basin and has the ability to add an additional 4 PE. The additional PE would bring the total equivalent population at build-out to 1,114 PE.

Table 2-10 | SC05_F2 Basin Population and Flow Projections

Description	2013 PE	Build-Out PE
SC05_F2 Service Area	1,110	**1,114**
Average Daily Flow (MGD)	0.10	**0.10**
***Existing Flows based on 93.58 gpd/ PE** ****Future Flows Based on IEPA 100 gpd/PE**		

Exhibit 2-11 | SC05_F2 Drainage Basin



2.4.10 SC05_R1 Drainage Basin

SC-05 is the designation provided to this drainage basin in the 1996 system-wide capacity study. This basin includes eight sub-basins, each of which were treated independently in the 2009 report.

Future development in the SC05_R1 basin is limited to vacant lots. Two lots have been identified as vacant (shown in red) within the basin and have the ability to add an additional 7 PE. The additional PE would bring the total equivalent population at build-out to 1,087 PE.

Table 2-11 | SC05_R1 Basin Population and Flow Projections

Description	2013 PE	Build-Out PE
SC05_R1 Service Area	1,080	**1,087**
Average Daily Flow (MGD)	0.10	**0.10**
***Existing Flows based on 93.58 gpd/ PE** ****Future Flows Based on IEPA 100 gpd/PE**		

Exhibit 2-12 | SC05_R1 Drainage Basin

2.4.11 SC05_R3 Drainage Basin

SC-05 is the designation provided to this drainage basin in the 1996 system-wide capacity study. This basin includes eight sub-basins, each of which were treated independently in the 2009 report

Future development in the SC05_R3 Basin is anticipated to include residential growth and the buildout of vacant lots. Residential growth (shown in purple) is could add an additional 641 PE to the basin. Thirteen vacant lots have been identified in the basin (shown in red) which could add an additional 46 PE. The additional PE would bring the total equivalent population at build-out to 2,788 PE.

Exhibit 2-13 | SC05_R3 Drainage Basin

Table 2-12 | SC05_R3 Basin Population and Flow Projections

Description	2013 PE	Build-Out PE
SC05_R3 Service Area	2,102	**2,788**
Average Daily Flow (MGD)	0.20	**0.27**
***Existing Flows based on 93.58 gpd/ PE** ****Future Flows Based on IEPA 100 gpd/PE**		

2.4.12 SC05_T1 Drainage Basin

SC-05 is the designation provided to this drainage basin in the 1996 system-wide capacity study. This basin includes eight sub-basins, each of which were treated independently in the 2009 report

Future development in the SC05_T1 basin is limited to vacant lots. One lot has been identified as vacant (shown in red) within the basin and has the ablity to add an additional 4 PE. The additional development would bring the total equivalent population at build-out to 947 PE.

Table 2-13 | SC05_T1 Basin Population and Flow Projections

Description	2013 PE	Build-Out PE
SC05_T1 Service Area	943	**947**
Average Daily Flow (MGD)	0.09	**0.09**
***Existing Flows based on 93.58 gpd/ PE** ****Future Flows Based on IEPA 100 gpd/PE**		

Exhibit 2-14 | SC05_T1 Drainage Basin

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SECTION 3

COLLECTION SYSTEM

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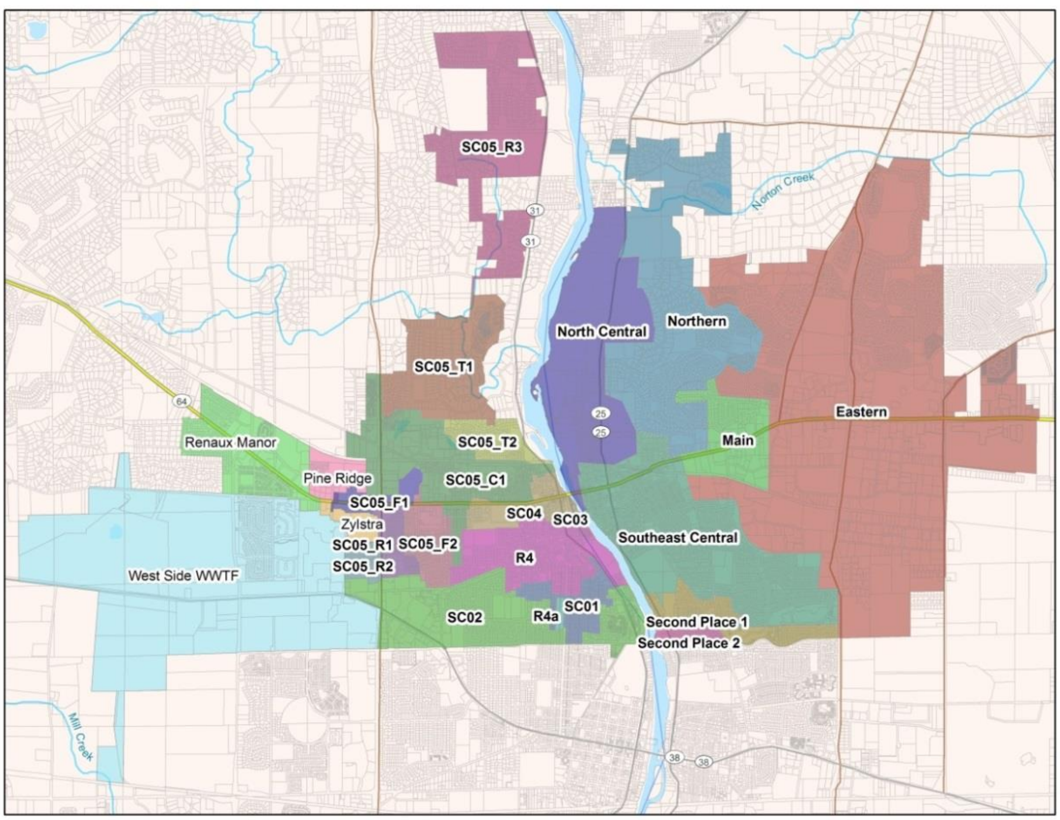
3. COLLECTION SYSTEM

3.1 GENERAL

The City of St. Charles wastewater collection system includes two service areas generally divided by Randall Road. The sanitary sewers west of Randall Road are tributary to the West Side Water Reclamation Facility (WRF). This service area is relatively new and the sewers have been constructed with modern materials, which minimize infiltration and inflow.

The sanitary sewer system east of Randall Road is tributary to the Main WWTF. The sewers within this collection system are of varying age and condition. As with many older collection systems, infiltration and inflow is a concern. Recognizing the importance of removing infiltration and inflow from the collection system, the City of St. Charles has developed a rigorous maintenance program including flow monitoring, root cutting, grouting, sewer lining and other rehabilitation and replacement of the collection system.

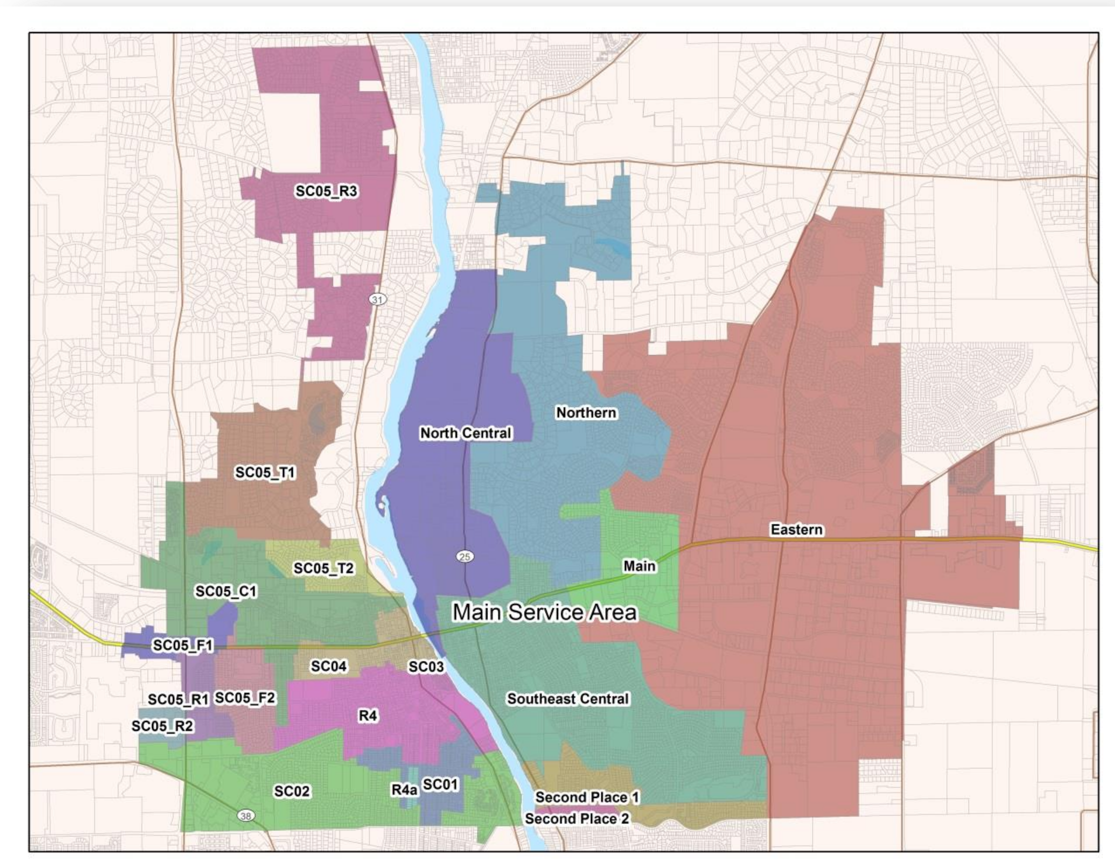
Exhibit 3-1 | Wastewater Drainage Basins



The City of St. Charles experienced the 500-year Storm Event in 1994 and the 100-year Storm Event in 1996 and 2007. These rain events led to widespread flooding in the area and caused Sanitary Sewer Overflows (SSO's). Recognizing the severity of the situation, the City also performed system-wide infiltration and inflow studies in 1994 and 1996 and again in 2006 and 2009, as well as Sanitary Sewer Evaluation Surveys (SSES's) on selected areas. The City's goal is to eliminate basement back-ups and SSO's. The recommendations contained in those reports have been incorporated into the City's annual sanitary sewer rehabilitation program. The City has budgeted an additional $4.24 million for sanitary sewer projects within the five-year capital improvements program.

While the City has made a commitment to improving the condition of the existing collection system, it also recognizes that infiltration and inflow cannot be completely eliminated. The collection system tributary to the Main WWTF consists of approximately 152 miles of sanitary sewers (in addition to the service laterals, which are similar in length), 5 miles of force main and 13 lift stations.

Exhibit 3-2 | Main WWTF Service Area Drainage Basins



During the 1996 Sanitary Sewer Study the entire collection system was divided into thirteen sub-basins to better define the extent and location of the infiltration and inflow issues. During the study flow monitoring was completed at ten locations. This data was correlated with flow meter data from the 1994 study to estimate 1-year, 5- year and 10- year Peak Wet Weather Flows for each basin. The City placed priority of service areas with the lowest levels of protection and plans to eventually provide a minimum of 10-year level of protection for the entire service area. The recommendations of the 1996 Phase II Sanitary Sewer System Evaluation estimated the 10-year peak wet weather flow tributary to the Main WWTF to be approximately 35.7 MGD.

In 2009, RJN Group prepared a flow monitoring report for the collection system on the west side of St. Charles tributary to the Main WWTF. The goal of that study was to identify areas of excessive infiltration and inflow, as well as to determine the average daily flows, peak dry-weather flows, and wet weather flows for the 1-year and 5-year storm events. During the study, flow monitoring was completed at 14 locations, which were delineated into 6 basins. This data was correlated with flow meter data from the 2006 study to estimate 1-year and 5-year Peak Wet Weather Flows from each basin. Using the results from that study, the City plans to perform field studies on the sanitary sewer lines that were identified as the most critical first, and incorporate these studies into their Capital Improvements Program. The six basins identified in this report will be used to describe the west side of the Main Service Area. The 2009 West Side Flow Monitoring Report estimated the 5-year peak wet weather flow to be 11.49 MGD.

The City has developed an extensive televising, cleaning and inspection program for the entire collection system. The collection system was divided into four quadrants (NE, SE, NW, and SW). The north/south dividing line is the Fox River that runs through the heart of downtown St. Charles and the east/west dividing line is Route 64. The City has televised, cleaned and inspected three out of the four quadrants to date, and the SW quadrant should be inspected in the near future. This is a continuous program that will repeat upon completion. This will assist the City in identifying problem locations for future projects.

In addition to the inspection and cleaning program, the City has identified that I/I is a large problem. The City has addressed the majority of the locations where spot repairs were needed. As a result of the repairs, the City has reduced a substantial amount of I/I. However, some locations continue to experience I/I. The City recognized that a large portion of I/I is a result of large private entities (schools, businesses and churches) not having the proper inspections during construction and the potential for illegal connections. The City plans on inspecting these locations to continue the reduction of I/I throughout the collection system.

The City of St. Charles' Finance Department maintains its GASB 34 Report, however, the collection system is not broken out by treatment facility. Therefore the actual value of this asset for the Main Service Area is not known. It has been estimated that the City currently maintains 172 miles of sanitary sewer mains (gravity and force main), as well as roughly 4,040 sanitary manholes in the Main and West Service Areas.

Using estimated replacement unit costs for sanitary sewer pipes, sanitary manholes and lift stations, the City owns and maintains a $220 million dollar collection system. Assuming 10% for contingency and 15% for design and administration, the replacement of the entire collection system is estimated to cost approximately $275 million. However, the majority of the collection system is not in need of replacement.

The service life of a collection system is approximately 75 years, and this life can be extended by approximately 25 years with ongoing maintenance and rehabilitation. Based on straight-line depreciation over this service life, it is recommended that the City reinvest $2,751,000 annually toward sanitary sewer collection system rehabilitation and replacement.

Approximately 20% of the collection system is already beyond its 75-year service life, and may be considered fully depreciated and in need of replacement. It is recommended that the City reinvest $1,403,000 annually toward the replacement of sewers that were installed before 1941 (as a portion of the annual reinvestment). It is also recommended that the remainder of the annual reinvestment be applied to the CMOM Program. There are several initial costs involved with starting up a program of this magnitude. This initial cost is estimated to be roughly $550,000. In order to sustain the long-term viability of the sewer utility, the City’s sewer rehabilitation budget should be raised to the aforementioned level.

This report section will revisit each of the basins, the recommendations of the 1996 and 2009 reports, and the proposed or completed solutions. This section will conclude with an overview of a CMOM Program for the City’s collection system. The following table provides a breakdown of the projected build-out population equivalent, Average Dry Weather Flow (ADDF) and calculated Peak Hourly Flow for each basin.

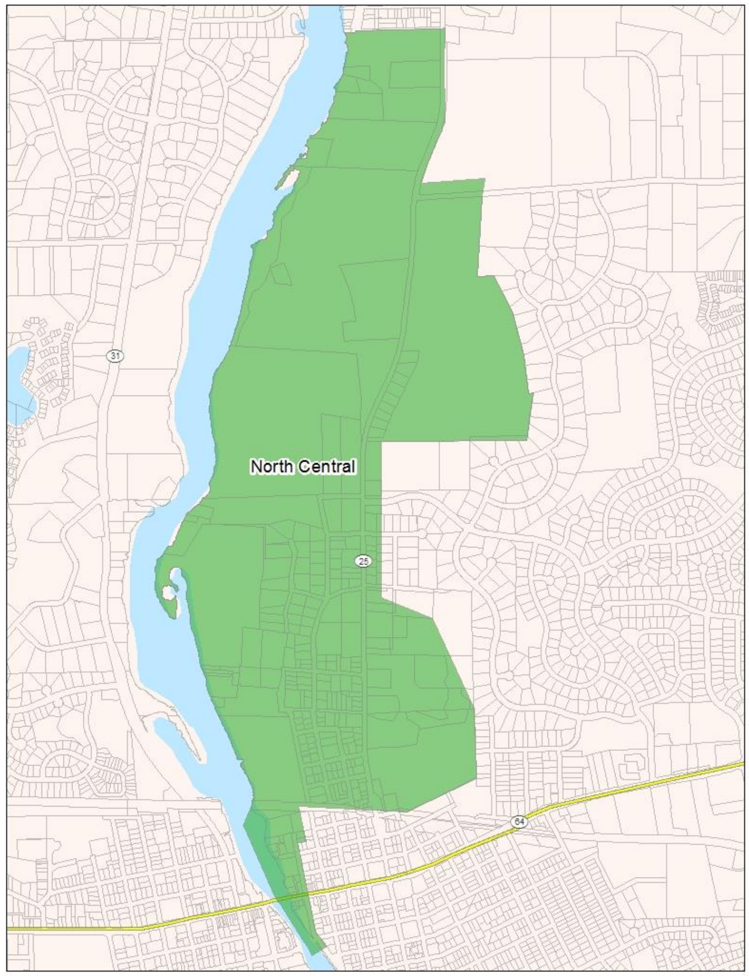
Table 3-1 | Main Service Area Population and Flow Projections

East Side Basins	PE	ADDF (MGD)	Peak Hourly Flow (MGD)
North Central	1,800	0.17	0.61
Second Place 1	1,064	0.10	0.38
Second Place 2	267	0.02	0.10
Eastern	17,643	1.65	4.47
Main	1,727	0.16	0.59
Northern	3,768	0.35	1.18
Southeastern Central	6,514	0.61	1.91
West Side Basins	**PE**	**ADDF (MGD)**	**Peak Hourly Flow (MGD)**
SC-01	726	0.07	0.26
SC-02	4,040	0.38	1.26
SC-03	72	0.01	0.03
SC-04	1,292	0.12	0.45
SC-05	8,805	0.82	2.48
R-4	1,988	0.19	0.67
R-4a	78	0.01	0.03

3.2 NORTH CENTRAL BASIN

The North Central Basin is bordered on the west by the Fox River, from Illinois Route 64 to the northern limits of the City. The basin includes 598 acres of commercial, institutional, and residential development. The collection system in this basin was constructed from the early 1930's to the early 1990's.

Exhibit 3-3 | North Central Basin



The Average Daily Dry Weather Flow in 1996 was 0.27 MGD, or 3,130 PE. Based on the new boundaries established in the 2009 Report, the basin size has been reduced and limited to the east side of the Fox River. As a result, the estimated population equivalents are reduced to 1,800 PE.

Portions of the SC-05 Basin are tributary to the North Central Basin, resulting in a Peak Wet Weather Flow in the 36-inch interceptor sewer of 13.9 MGD as estimated in the 1996 Report. This interceptor is tributary to a portion of the Southwest Central Basin and ultimately to the Riverside Lift Station.

The 1996 Report estimated that the sewers in this service area were adequate to convey the 10-year wet weather flow. While the sewers in the basin provide ten-year protection, the tributary areas do not have adequate capacity.

The City has developed an extensive sewer lining program that has been in place for several years. Recently, the majority of the sewers within the Norris Woos subdivision have been lined, approximately 2,200 lineal feet. The lining did not include any sewers within Pottawatomie Park.

The 1996 Report recommended increasing the size of one interceptor sewer segment within the basin from 18-inch to 27-inch. The project would extend from Manhole 5.3-174 on Route 31 north of the railroad bridge to Manhole 5.3-126 on Route 31 north of State Street, over 2,000 lineal feet. This interceptor serves the Timbers Basin and is located in a fully developed area of the community, which increases the cost of the project. The City currently does not have this project listed in its 10-year plan for capital improvements. Since the 1996 report other projects in the area have been completed that have reduced the urgency for the increased sewer capacity. It is recommended that this project be added to the capital projects; however it is a low priority project. This area was also revisited as part of the 2009 Report and is included in Basin SC-05.

The City has identified a site in this basin that may be contributing a large amount of I/I. The site is west of Route 31 and north of State Street, and was previously the site for Applied Composites. The potential source of I/I is due to failing sewers in the area that were previously abandoned. This site has the potential for future development. Upon development, the I/I sources should be removed. It is recommended that the City televise the sewers in this area to identify the exact locations of the leaks.

Table 3-2 | North Central Basin Population and Flow Projections

	PE	ADDF (MGD)	Peaking Factor	PHF (MGD)
Existing Conditions	1,800	0.17	3.62	0.61
Build-Out Conditions	1,832	0.17	3.62	0.62

3.3 SECOND PLACE 1 & 2

The Second Place 1 and Second Place 2 Basins experienced severe flooding during the 1994 and 1996 rain events. The basins were studied independently in 1996 to provide a more in-depth analysis and better define the cause of the back-ups.

Exhibit 3-5 | Second Place 1 Basin

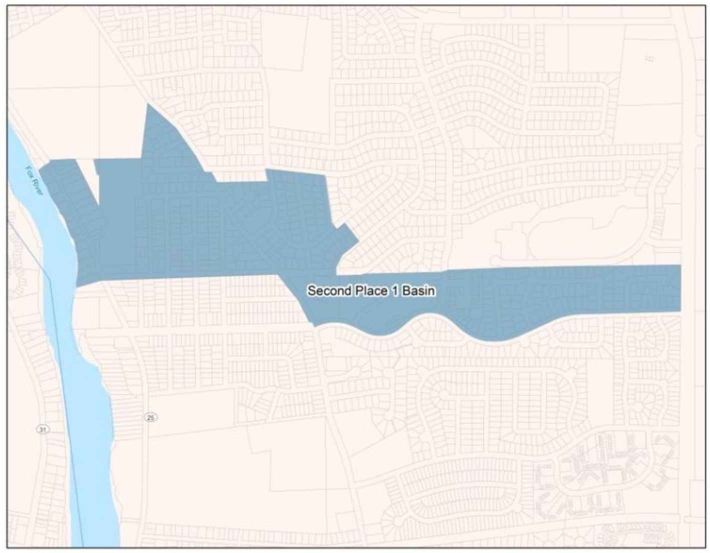


Exhibit 3-5 | Second Place 2 Basin

Second Place 1 Basin includes 150 acres of residential development along Division Street west of 7th Avenue and an older residential neighborhood east of Seventh Avenue Creek. The service area is tributary to the Beatrice Avenue sewer, the Seventh Avenue Creek Interceptor and ultimately the Riverside Avenue Interceptor. The Riverside Avenue Interceptor is tributary to the Riverside Pump Station.

Second Place 2 Basin includes 31 acres of established residential development south of Seventh Avenue Creek. This basin is tributary to the Riverside Avenue Interceptor Sewer and ultimately the Riverside Pump Station. The 1996 study determined that the main cause for the basement back-ups was overloading in the Seventh Avenue Creek and Riverside Avenue Interceptors. Both sewers were constructed of vitrified clay pipe in the naturally occurring limestone bedrock strata. Root intrusion and cracked pipe proved to be a major source of infiltration and inflow. The 1994 Report recommended rehabilitation of both these sewers. This has not been completed, and should be included in the 10-year capital improvements plan.

A couple of projects have been completed targeting I/I in the Second Place Basins. The first project included realigning the connection point between the Seventh Avenue Creek Interceptor and Riverside Avenue Interceptor, which improved the hydraulics of the system, and replacement of 620 feet of the Riverside Avenue Interceptor. The project was completed in 1995. The second project was completed in 1997, and included slip lining of the Seventh Avenue Creek Sewer. The remainder of the Riverside Avenue Interceptor sewer was rehabilitated with cured-in-place lining in 1998.

The City has developed an extensive televising and cleaning program over the past few years. The areas south of Route 64 are to be cleaned and televised by the City in the near future.

Table 3-3 | Second Place 1 Basin Population and Flow Projections

Second Place 1	PE	ADDF (MGD)	Peaking Factor	PHF (MGD)
Existing Conditions	1,064	0.10	3.78	0.38
Build-Out Conditions	1,064	0.10	3.78	0.38

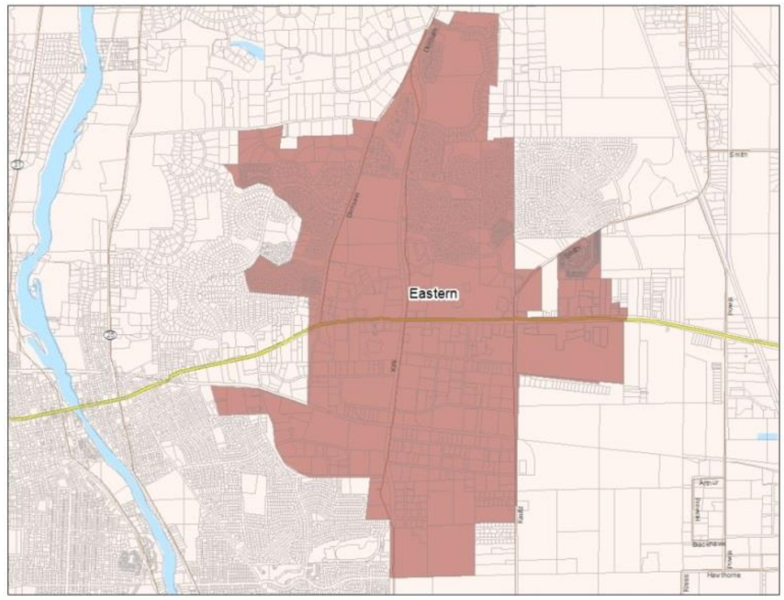
Table 3-4 | Second Place 2 Basin Population and Flow Projections

	PE	ADDF (MGD)	Peaking Factor	PHF (MGD)
Existing Conditions	267	0.02	4.10	0.10
Build-Out Conditions	267	0.02	4.10	0.10

3.4 EASTERN BASIN

The Eastern Basin includes 2,672 acres with a broad mix of institutional, office, industrial, commercial and residential development. The developments date from the 1960's to the present. Areas included within this basin are bordered by the Royal Fox subdivision to the north, by Division Street to the south, and include areas from roughly Dunham Road to the eastern City limits. The collection system in this area is tributary to the Southeast Central Basin and ultimately the East Side Lift Station.

Exhibit 3-6 | Eastern Basin



The 1996 Study determined the Average Daily Dry Weather Flow in the basin to be 0.223 MGD, or 2,690 PE. The service area has grown significantly since the 1995 flow monitoring and more up-to-date data is required to determine current loading. The current population equivalent for this basin is 17,689 PE or 1.19 MGD. The I/I during the 10-year Wet Weather event in the 1996 Report was estimated to be 1.41 MGD, which equals 1,500 gallons per day per acre. The collection system had capacity for over 2.5 times the 10-year event.

Table 3-5 | Eastern Basin Population and Flow Projections

	PE	ADDF (MGD)	Peaking Factor	PHF (MGD)
Existing Conditions	17,643	1.65	2.71	4.47
Build-Out Conditions	21,615	2.05	2.62	5.36

3.5 MAIN BASIN

The Main Basin serves a small area along Main Street (Illinois Route 64) from 14th Avenue to Dunham Road. The basin includes 201 acres of residential, commercial and light industrial development. The sanitary sewers in the area are of varying age and condition.

Exhibit 3-7 | Main Basin

The 1996 Report determined the Average Daily Dry Weather Flow to be 0.19 MGD, or 2,290 PE. Based on Illinois EPA peaking factors, the peak hourly flow for the basin can be estimated at 0.75 MGD.

The 1996 Report determined that sewers within the basin had less than a one-year level of protection, which was unacceptable. A series of sewer segments were identified along Illinois 64 for replacement to provide the desired 10-year protection level. This reach of sewer was replaced in 2011/2012 in conjunction with the Illinois DOT Route 64 Reconstruction Improvements. In addition, the City replaced all services as a part of that project.

The City has identified a manhole that is in need of rehabilitation and the associated sewer needs to be increased in size. The manhole and sewer is located near Route 64 and Tyler Road. Currently the sewer is an 8-inch pipe that should be increased to 12-inch. The City has experienced surcharging within the manhole during peak flows. However, the manhole is 30-feet deep and no overflow has been experienced. A 12-inch sewer would allow the system to convey all flows even during high flow events. This project is a low priority due to the location and depth of the existing sewer and manhole.

It should be noted that the water billing data provided from 2011 and 2013 indicates that the current population equivalent is closer to 1,727 PE instead of the previously estimated 2,290.

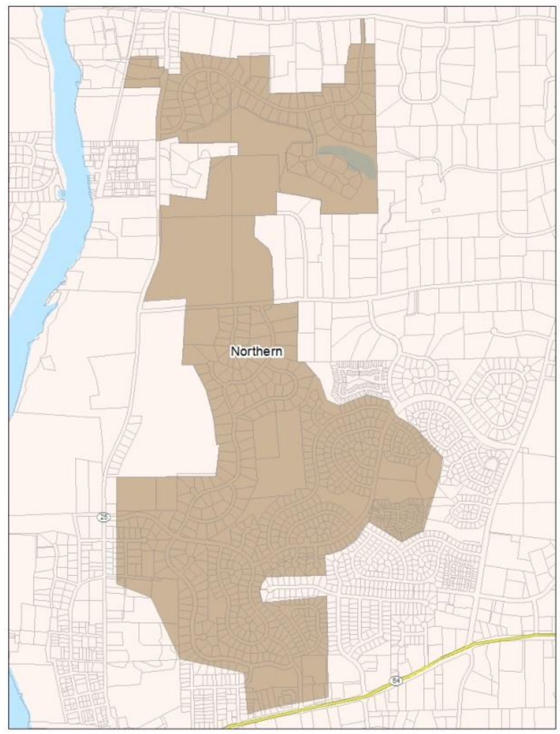
Table 3-6 | Main Basin Population and Flow Projections

.	PE	ADDF (MGD)	Peaking Factor	PHF (MGD)
Existing Conditions	1,727	0.16	3.63	0.59
Build-Out Conditions	1,727	0.16	3.63	0.59

3.6 NORTHERN BASIN

The Northern Basin includes 599 acres of (1980's) residential development. The basin includes two pump stations and extends from Illinois Route 64 north to Army Trail Road, encompassing a total of 822 acres. The Average Daily Dry Weather Flow from this Service Area was estimated to be 0.189 MGD, or 2,280 PE, in the 1996 study.

Exhibit 3-8 | Northern Basin



The Northern Basin is tributary to the Southeast Central Basin and ultimately the East Side Lift Station. The 1996 Report estimated the 10-year Peak Wet Weather Flow from the basin to be 1.17 MGD.

The collection system's capacity is nearly 3.0 MGD and provides the Northern Basin with a level of protection in excess of the 10-year Wet Weather Event. The 1996 Report recommended televising and dye testing of the area, but gave this recommendation a Level III priority based on the Northern Basin collection system's plentiful capacity and relatively young age. The City has completed the inspection and televising of the sewers in this area.

It should be noted that the water billing data provided from 2011 and 2013 indicates that this basin includes 3,768 PE in comparison with the 2,280 estimated as part of the 1996 Report.

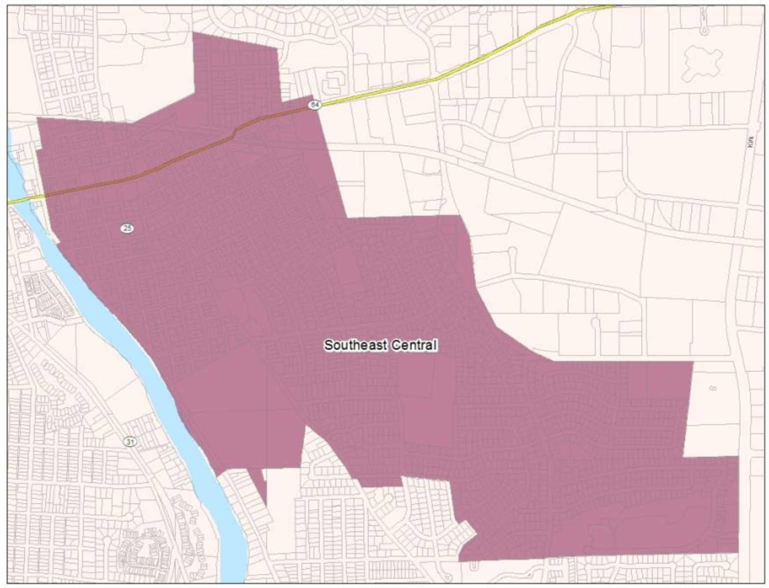
Table 3-7 | Northern Basin Population and Flow Projections

	PE	ADDF (MGD)	Peaking Factor	PHF (MGD)
Existing Conditions	3,768	0.35	3.36	1.18
Build-Out Conditions	3,782	0.35	3.36	1.19

3.7 SOUTHEAST CENTRAL

The Southeast Central Basin includes 763 acres of dense commercial, residential and light industrial development. The collection system is tributary to both of the lift stations tributary to the Main WWTF. 234 acres of the Southeast Basin are tributary to the Riverside Lift Station. The remaining 481 acres are tributary to the East Side Lift Station.

Exhibit 3-9 | Southeast Central Basin



The collection system in the Southeast Central Basin varies in age and condition, and contains some of the oldest sewers in the community. Infiltration and inflow problems in the Southeast Central Basin are severe. The 1996 Report estimated the Average Daily Dry Weather Flow from the basin to be 0.59 MGD, or 7,130 PE. Again, it should be noted that the population equivalents for this area have changed since the 1996 Report. The current estimate is 6,514 PE based on water billing records from 2011 and 2013.

Table 3-8 | Southeast Central Population and Flow Projections

	PE	ADDF (MGD)	Peaking Factor	PHF (MGD)
Existing Conditions	6,514	0.61	3.14	1.91
Build-Out Conditions	7,362	0.69	3.09	2.14

3.8 WEST SIDE BASIN SC-01

Basin SC-01 is on the southwest side of the Main Service Area, and spans from the St. Charles/Geneva border to Mosedale Street on the north. The basin includes 99 acres and serves 726 PE. As stated in the 2009 Flow Monitoring Report, this basin has a 5-year Peak Hourly Flow of 0.95 MGD. By comparing the measured ADDF in that report to the estimated 5-year Peak Wet Weather Flow, the I/I was approximately 0.80 MGD. This basin is tributary to the Park Shore (southern) Siphon under the Fox River and ultimately to the Riverside Lift Station.

Exhibit 3-10 | West Side Basin SC-01

Table 3-9 | West Side Basin SC-01 Population and Flow Projections

	PE	ADDF (MGD)	Peaking Factor	PHF (MGD)
Existing Conditions	726	0.07	3.89	0.26
Build-Out Conditions	747	0.07	3.88	0.27

3.9 WEST SIDE BASIN SC-02

Basin SC-02 is also on the southwest side of the Main Service Area. The basin includes 458 acres and serves 4,040 PE. As stated in the 2009 Flow Monitoring Report, this basin has a 5-year Peak Hourly Flow of 1.57 MGD. By comparing the measured ADDF in that report to the estimated 5-Year Peak Wet Weather Flow, the I/I was approximately 1.21 MGD. This basin is directly tributary South Siphon under the Fox River and ultimately to the Riverside Lift Station.

Exhibit 3-11 | West Side Basin SC-02

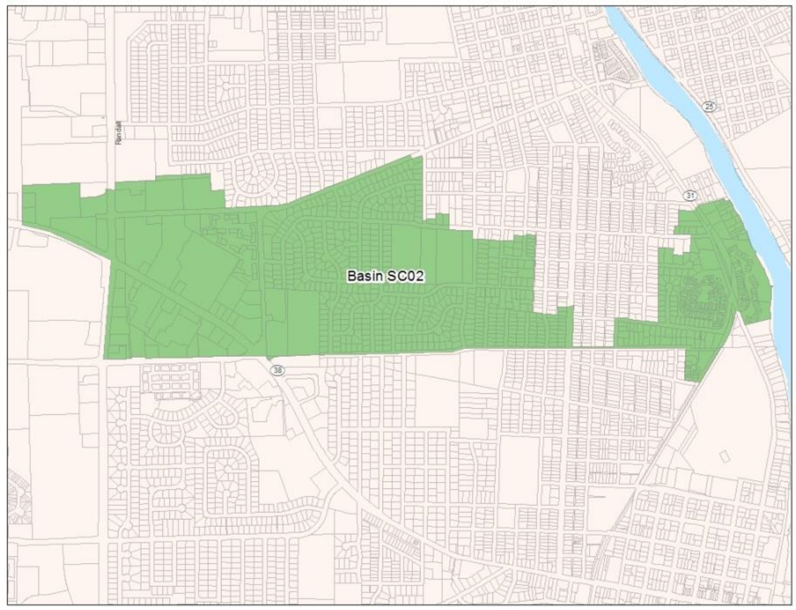


Table 3-10 | West Side Basin SC-02 Population and Flow Projections

	PE	ADDF (MGD)	Peaking Factor	PHF (MGD)
Existing Conditions	4,040	0.38	3.33	1.26
Build-Out Conditions	4,556	0.43	3.28	1.41

3.10 WEST SIDE BASIN SC-03

Basin SC-03 includes a very small portion of downtown St. Charles, only servicing 1.3 acres and two customers that equate to 72 PE. Despite its size, this basin has a 5-year Peak Hourly Flow of 1.57 MGD based on the 2009 Report. By comparing the measured ADDF in that report to the estimated 5-year Peak Wet Weather Flow, the I/I was approximately 1.51 MGD. This basin is tributary to Basin R-4 before being conveyed under the Fox River to the Riverside Lift Station via the Park Shore (south) Siphon.

Exhibit 3-12 | West Side Basin SC-03

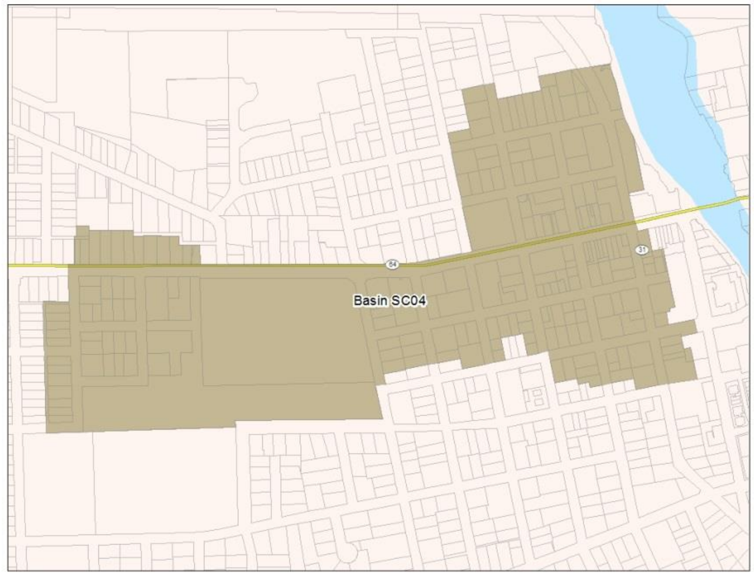
Table 3-11 | West Side Basin SC-03 Population and Flow Projections

	PE	ADDF (MGD)	Peaking Factor	PHF (MGD)
Existing Conditions	72	0.01	4.28	0.03
Build-Out Conditions	72	0.01	4.28	0.03

3.11 WEST SIDE BASIN SC-04

Basin SC-04 generally includes the west portion of downtown St. Charles, from 12th Street to the Fox River. The basin includes over 106 acres and serves 1,292 PE. This basin is tributary to Basins SC-03 and R-4 before being conveyed under the Fox River to the Riverside Lift Station via the Park Shore (south) Siphon.

Exhibit 3-13 | West Side Basin SC-04



Based on the 2009 Report, the 5-year Peak Hourly Flow was estimated to be 1.02 MGD. By comparing the measured ADDF in that report to the estimated 5-year Peak Wet Weather Flow, the I/I was approximately 0.97 MGD.

The collection system in this basin was constructed from the early 1930's to the early 1990's. Rehabilitation of this system was given a high priority in the 2009 Report because it contributed more than 4,000 gallons of I/I per inch diameter mile. The City conducted a Sanitary Sewer Evaluation Study (SSES), which included televising, manhole inspection, dye testing and building inspection.

Since inspection and televising of these sewers, a large amount of lining was performed along the Route 64 corridor. The majority of the lining consisted of point repairs and manhole rehabilitation. These improvements have removed a large amount of I/I in the basin. Although a reduction of I/I has been achieved, the City is expected to continue repairs in the basin.

Table 3-12 | West Side Basin SC-04 Population and Flow Projections

	PE	ADDF (MGD)	Peaking Factor	PHF (MGD)
Existing Conditions	1,292	0.12	3.73	0.45
Build-Out Conditions	1,292	0.12	3.73	0.45

3.12 WEST SIDE BASIN SC-05

Basin SC-05 is the designation provided to this basin in the 1996 Report. This basin includes eight sub-basins, each of which were treated independently in the 2009 report. The major basin includes approximately 1,586 acres and serves 8,805 PE.

Based on the 2009 Report, the 5-year Peak Wet Weather Flow for Basin SC-05 was estimated to be 6.56 MGD. This basin was analyzed for the 2-hour storm due to its size. By comparing the measured ADDF in that report to the estimated 5-year Peak Wet Weather Flow, the I/I was approximately 5.80 MGD.

Rehabilitation of the sewers in Basin SC05 was given a low priority in the 2009 Report because other areas of the community contributed more I/I per inch diameter mile. The 2009 Report recommended inspection and further monitoring of flows from the SC-05 sanitary sewers, but noted that areas should be studied and rehabilitated based on failure criticality. The City has budgeted for and anticipates the completion of the study within the next five years.

Exhibit 3-14 | West Side Basin SC-05

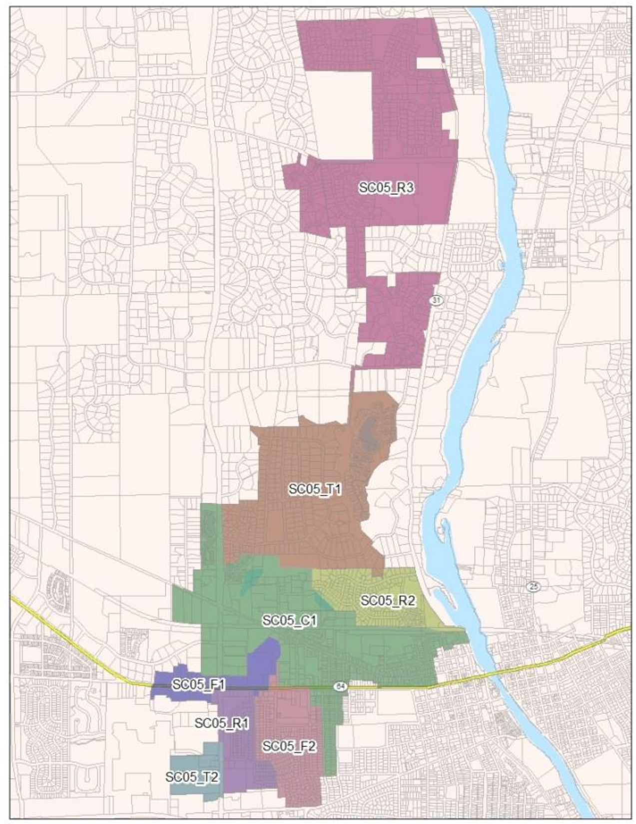


Table 3-13 | West Side Basin SC-05 Population and Flow Projections

Drainage Sub-Basin at Existing Conditions	PE	ADDF (MGD)	Peaking Factor	PHF (MGD)
SC-05_T2	696	0.07	3.90	0.25
SC-05_F2	1,110	0.10	3.77	0.39
SC-05_C1	2,553	0.24	3.50	0.84
SC-05_R3	2,102	0.20	3.57	0.70
SC-05_R2	69	0.01	4.28	0.03
SC-05_R1	1,080	0.10	3.78	0.38
SC-05_F1	252	0.02	4.11	0.10
SC-05_T1	943	0.09	3.82	0.34
Drainage Sub-Basin at Build-Out Conditions	**PE**	**ADDF (MGD)**	**Peaking Factor**	**PHF (MGD)**
SC-05_T2	696	0.07	3.90	0.25
SC-05_F2	1,114	0.10	3.77	0.39
SC-05_C1	3,157	0.30	3.42	1.02
SC-05_R3	2,788	0.27	3.48	0.92
SC-05_R2	69	0.01	4.28	0.03
SC-05_R1	1,087	0.10	3.78	0.38
SC-05_F1	252	0.02	4.11	0.10
SC-05_T1	947	0.09	3.82	0.34

Sub-basins SC-05_F1 and SC-05_F2 were given a high priority for rehabilitation in the 2009 Report because they contributed more than 4,000 gallons of I/I per inch diameter mile. It was recommended that they both be included in a Sanitary Sewer Evaluation Study (SSES), which would include manhole inspection, dye testing and building inspection. The City has completed inspection and performed some spot repairs and sewer lining in the basin. Over the next five years it is anticipated that the City will perform more work to further remove I/I within this basin.

3.13 WEST SIDE BASIN R4 AND R4A

Basins R4 and R4a are on the southwest side of the Main Service Area. Basin R4 includes area around Prairie Street from 13th Street to the west bank of the Fox River. This basin includes 286 acres and serves 1,988 PE. Basin R4a includes areas from Horne to Roosevelt Street, bound between 3rd and 4th Street. This basin includes about 8 acres and serves 78 PE. These basins exhibit a 5-year Peak Hourly Flow of 2.41 MGD based on the 2009 Report. By comparing the measured ADDF in that report to the estimated 5-year Peak Wet Weather Flow, the I/I was approximately 1.96 MGD.

Exhibit 3-16 | West Side Basin R4

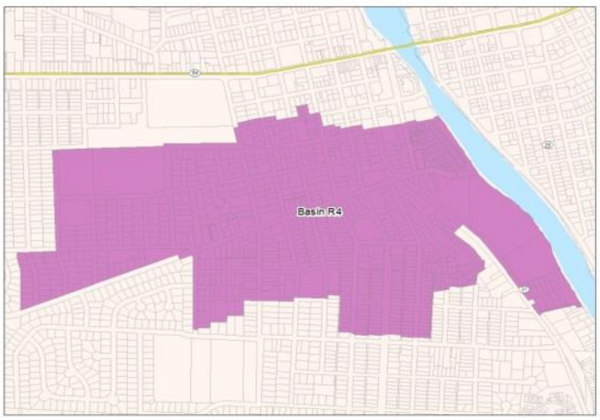


Exhibit 3-16 | West Side Basin R4a

Table 3-14 | West Side Basin R4 Population and Flow Projections

	PE	ADDF (MGD)	Peaking Factor	PHF (MGD)
Existing Conditions	1,988	0.19	3.59	0.67
Build-Out Conditions	2,006	0.19	3.58	0.67

Table 3-15 | West Side Basin R4a Population and Flow Projections

	PE	ADDF (MGD)	Peaking Factor	PHF (MGD)
Existing Conditions	78	0.01	4.27	0.03
Build-Out Conditions	78	0.01	4.27	0.03

3.14 CAPACITY, MANAGEMENT, OPERATION AND MAINTENANCE (CMOM) PROGRAM

The City of St. Charles has been investigating the I/I within the wastewater collection system for nearly 20 years; the City completed I/I and SSES projects in 1994, 1996, 2006 and 2009 as discussed throughout this Section of the report. As a result of these investigations and ongoing improvements, the City has successfully located and removed several sources of I/I.

An extensive televising, cleaning and inspection program has been implemented for the entire collection system. The City has completed approximately 75% of this program, and will complete the remaining 25% in the near future. The program will then start over with the intention of inspecting the entire system once every 10 years. The City has also addressed the majority of the locations where spot repairs were needed. As a result of these efforts, the City has reduced the amount of I/I considerably; I/I was reduced from roughly 46 gallons per day per PE in 2009 to roughly 26 gallons per day per PE in 2013.

The I/I currently accounts for approximately13% of the total influent flow received at the West Side WRF, and approximately 28% of the total influent flow received at the Main WWTF. During particularly severe wet weather events, the City must deploy trash pumps at certain locations in the collection system. These pumps relieve the hydraulic loading on the collection system by discharging raw sewage directly to nearby waterways, namely the Fox River. While these types of bypass pumping activities prevent basement back-ups, they are categorized as Sanitary Sewer Overflows (SSO's) by the IEPA and are prohibited by the City's NPDES Permit. While SSO's are rare, they have occurred in the past and the City is continuously striving to prevent them from happening in the future. Special Condition 21 of the City's renewed NPDES Permit for the Main WWTF outlines the efforts required of the City to eliminate basement back-ups and SSO's.

SPECIAL CONDITION 21. The Permittee shall work towards the goals of achieving no discharges from sanitary sewer overflows or basement backups and ensuring that overflows or backups, when they do occur do not cause or contribute to violations of applicable standards or cause impairment in any adjacent receiving water. In order to accomplish these goals, the Permittee shall develop, implement and submit to the IEPA a Capacity, Management, Operations, and Maintenance (CMOM) plan within twelve (12) months of the effective date of this Permit. The Permittee should work as appropriate, in consultation with affected authorities at the local, county, and/or state level to develop the plan components involving third party notification of overflow events. The Permittee may be required to construct additional sewage transport and/or treatment facilities in future permits or other enforceable documents should the implemented CMOM plan indicate that the Permittee's facilities are not capable of conveying and treating the flow for which they were designed.

The CMOM plan shall include the following elements:

a. Measures and Activities:

1. *A complete map of the collection system owned and operated by the Permittee;*
2. *Schedules, checklists, and mechanisms to ensure that preventative maintenance is performed on equipment owned and operated by the Permittee;*

3. *An assessment of the capacity of the collection and treatment system owned and operated by the Permittee at critical junctions and immediately upstream of locations where overflows and backups occur or are likely to occur; and*
4. *Identification and prioritization of structural deficiencies in the system owned and operated by the Permittee.*

b. Design and Performance Provisions:

1. *Monitor the effectiveness of CMOM;*
2. *Upgrade the elements of the CMOM plan as necessary; and*
3. *Maintain summary of CMOM activities.*

c. Overflow Response Plan:

1. *Know where overflows within the facilities owned and operated by the Permittee occur;*
2. *Respond to each overflow to determine additional actions such as clean up; and*
3. *Locations where basement back-ups and/or sanitary sewer overflows occur shall be evaluated as soon as practicable for excessive inflow /infiltration, obstructions or other causes of overflows or back-ups as set forth in the System Evaluation Plan.*

d. System Evaluation Plan.

e. Reporting and Monitoring Requirements.

f. Third Party Notice Plan:

1. *Describes how, under various overflow scenarios, the public, as well as other entities, would be notified of overflows within the Permittee's system that may endanger public health, safety or welfare;*
2. *Identifies overflows within the Permittee's system that would be reported, giving consideration to various types of events including events with potential widespread impacts;*
3. *Identifies who shall receive the notification;*
4. *Identifies the specific information that would be reported including actions that will be taken to respond to the overflow;*
5. *Includes a description of the lines of communication; and*
6. *Includes the identities and contact information of responsible POTW officials and local, county, and/or state level officials.*

In response to this Special Condition, the City will need to budget for the preparation and implementation of a CMOM program. It should be noted that GIS data is critical when evaluating the asset value of the collection system. The City's data was reviewed, and it was determined that several miles of sanitary sewers were dated incorrectly. Using historical aerial photography, these sewers were properly categorized as pre- or post-1941 to determine the recommended annual reinvestment for sanitary sewer replacement. We estimate that the City should be reinvesting $2,751,000 annually toward sanitary sewer collection system rehabilitation, and that $1,403,000 be put towards the replacement of sewers that were installed before 1941. It is also recommended that the remainder of the annual reinvestment be applied to the CMOM Program. There are several initial costs involved with starting up a program of this magnitude, which are shown to be included in the 2015/2016 fiscal year budget. This initial cost is estimated to be roughly $550,000. A recommended breakdown of necessary budget items is included on the following page.

Capacity Maintenance, Operation and Management (CMOM)

A. *Capacity Maintenance and Operation (70% of Budget)*
   1. *Sanitary Sewer Evaluation Survey (SSES) and Rehabilitation Program*
      a) *Interceptor Sewer Cleaning and Inspection Project*
         (1) *STC Staff – $20,000 – FY 2015/2016*
         (2) *Engineering – $20,000 – FY 2015/2016*
         (3) *Construction – $360,000 – FY 2015/2016*
      b) *Interceptor Sewer Capacity Management Program*
         (1) *STC Staff – $12,000 Annually*
         (2) *Engineering – ($25,000) Annually*
         (3) *Construction – ($180,000) Annually*
      c) *Sanitary Sewer Inspection Program*
         (1) *STC Staff – $131,000 Annually*
      d) *Sanitary Sewer Spot Repair Program*
         (1) *STC Staff – $13,000 Annually*
         (2) *Engineering – ($28,000) Annually*
         (3) *Construction – ($166,000) Annually*
      e) *Sanitary Sewer Lining Program*
         (1) *STC Staff – $16,000 Annually*
         (2) *Engineering – ($33,000) Annually*
         (3) *Construction – ($340,000) Annually*

B. *Capacity Management (20% of Budget)*
   1. *Flow Metering Project*
      a) *Flow Metering Program Development*
         (1) *STC Staff – $10,000 – FY 2015/2016*
         (2) *Engineering – $10,000 – FY 2015/2016*
         (3) *Meter Purchase – $60,000 – FY 2015/2016*
      b) *Annual Metering Program*
         (1) *STC Staff – $120,000*
         (2) *Engineering – $20,000*
   2. *Sanitary Sewer Modeling/ Atlas Program*
      a) *Program Development*
         (1) *STC Staff – $8,000 – FY 2015/2016*
         (2) *Engineering – $20,000 – FY 2015/2016*
      b) *Annual Modeling Program*
         (1) *STC Staff – $10,000*
         (2) *Engineering – $120,000*

C. *Private Service Lateral Rehabilitation Program (10% of Budget)*
   1. *Policy Initiation*
      a) *Public Education*
         (1) *STC Staff – $11,000 – FY 2015/2016*
         (2) *Outside Consultants – $25,000 – FY 2015/2016*
   2. *Program Implementation*
      a) *Annual Inspection Program*
         (1) *STC Staff – $90,000*
      b) *Annual Lateral Rehabilitation Program*
         (1) *STC Staff – $18,000*
         (2) *Construction – $0 – i.e. All homes sold to be inspected and repaired if necessary, paid for by seller*

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SECTION 4

LIFT STATIONS

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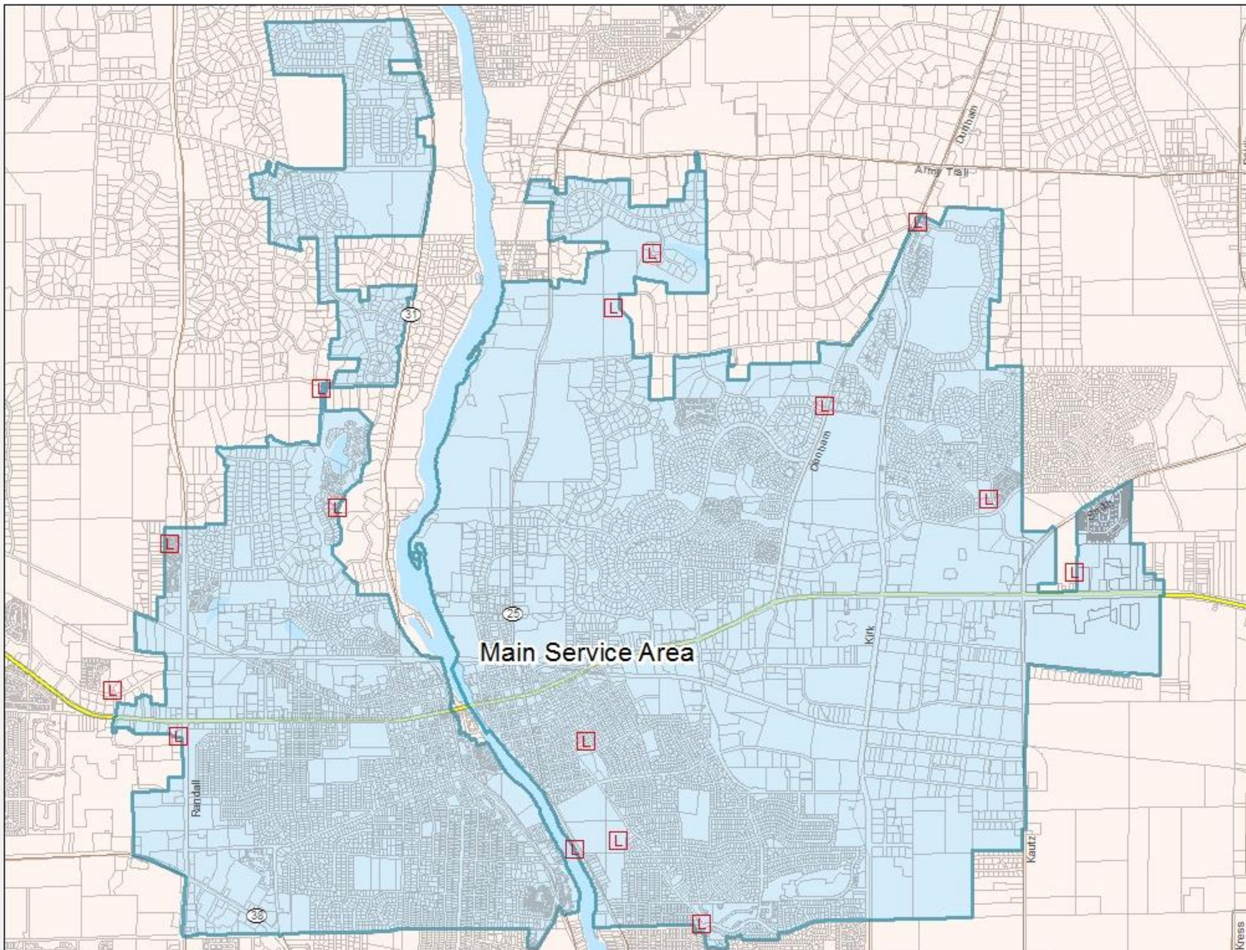
4. LIFT STATIONS

The City of St. Charles' Main Service Area includes thirteen lift stations, two of which are directly tributary to the headworks at the Main WWTF. Locations of the lift stations are shown on the exhibit below:

Exhibit 4-1 | Main Service Area Lift Station Locations



The lift stations vary in age and condition, however most were constructed between 1987 and 1997 as the City developed further north and east. The two main lift stations are Riverside Lift Station and East Side Lift Station. City staff has assisted in the development of this Section of the Facility Plan Update and has provided input with respect to improvements needed at each station.

Table 4-1 | Lift Station Asset Value

Lift Station	Equipment	Structure	Force Main	Totals
Riverside	$1,750,000	$2,000,000	$1,280,000	$5,030,000
East Side	$1,030,000	$1,500,000	$96,000	$2,626,000
7th & Division	$200,000	$145,000	$109,000	$454,000
Washington Ave.	$50,000	$50,000	$73,000	$173,000
Country Club	$200,000	$155,000	$129,000	$484,000
Pheasant Run Trails	$210,000	$185,000	$292,000	$687,000
Royal Fox #2	$220,000	$185,000	$498,000	$903,000
Royal Fox #1	$210,000	$165,000	$358,000	$733,000
Woods of Fox Glen	$210,000	$185,000	$566,000	$961,000
Kingswood	$210,000	$185,000	$197,000	$592,000
Wild Rose	$200,000	$160,000	$14,000	$374,000
Red Gate	$210,000	$185,000	$311,000	$706,000
Oak Crest	$200,000	$155,000	$74,000	$429,000
Totals	$4,900,000	$5,255,000	$3,997,000	$14,152,000
Design Life, Years	20	50	50	
Annual Replacement	$245,000	$105,100	$79,940	**$430,040**

It should be noted that the above figures do not include the engineering and contingencies that would be involved in a rehabilitation or replacement project. The value of the City's lift station and force main assets is approximately $14,152,000. Based on a straight-line depreciation over the design life of the equipment, structures and force mains, the City should be reinvesting around $430,000 annually toward maintaining and replacing these assets within the Main Service Area.

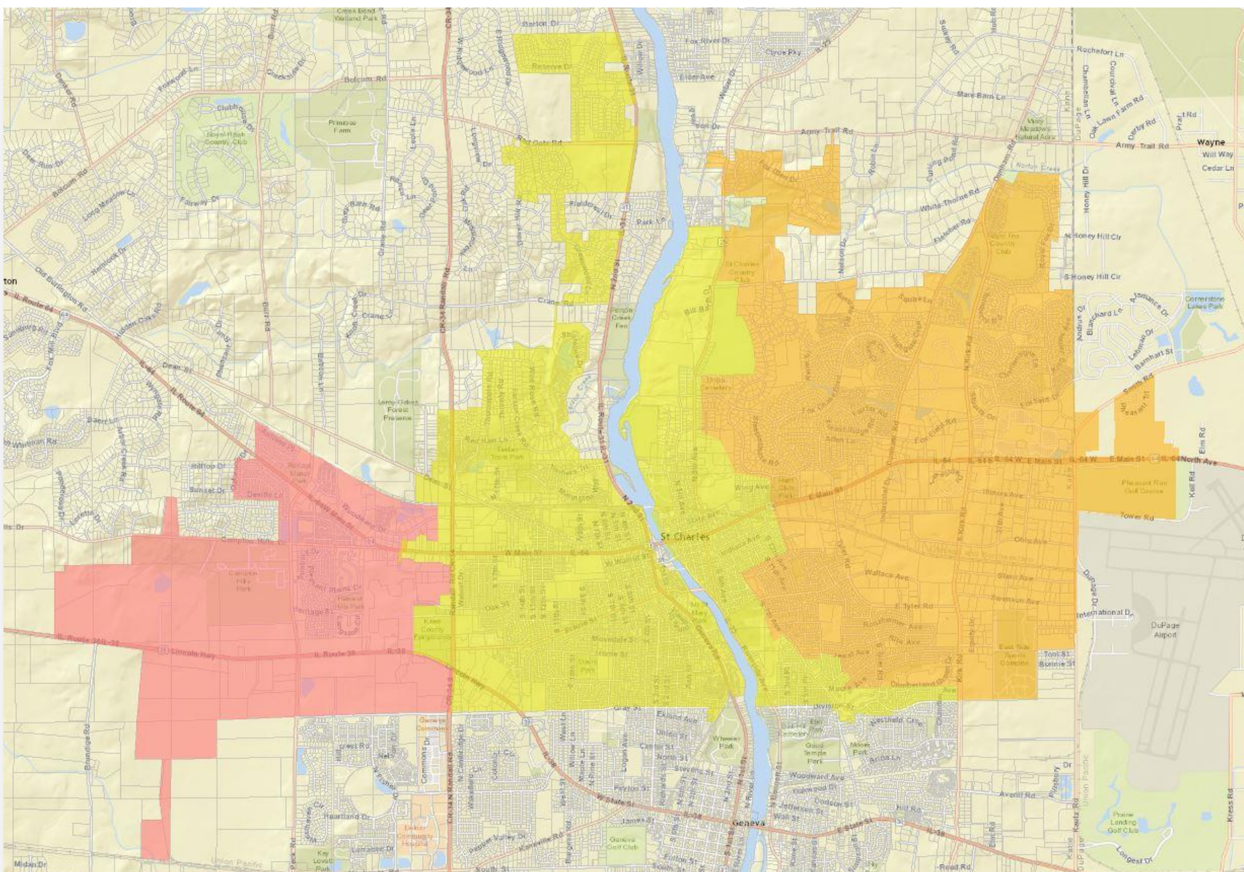
This section will discuss each lift station's strengths, deficiencies, and future needs independently. Operational staff has indicated that most of the recommended improvements could be accomplished utilizing in-house resources. The more significant improvements have been broken into capital projects and recommended budgets have been provided. These projects should be incorporated into the City's Capital Improvements Program.

4.1 RIVERSIDE LIFT STATION:

4.1.1 General Description

Riverside Lift Station is located at the intersection of Riverside Avenue (Illinois Route 25) and Devereaux Way. The lift station is located at the site of what was the City of St. Charles' first wastewater treatment facility, which was an Imhoff tank. When the wastewater treatment facility was relocated up the hill to the east in the 1930's, this site remained as a collection point for the City of St. Charles' wastewater infrastructure.

Exhibit 4-2 | Riverside Lift Station Service Area (Yellow)



The Riverside Lift Station serves the majority of downtown area and west to Randall Road. The lift station service area, shown above in yellow contains roughly 26,143 PE. A mixture of land uses is served, including residential, commercial and light industrial. The lift station's Design Average Flow and the Peak Wet Weather Flow are 1.97 MGD and 28.2 MGD respectively.

Table 4-2 | Riverside Lift Station – Pump and Force Main Data

Number of Pumps	Pump Manuf. & Type	Pump Motor (HP)	Pump Rated Capacity (GPM)	Force Main Dia. (inch)	Rated TDH (feet)	Installation Date	Rehab Date
4	Flygt Submersible	2 @ 75 & 2 @ 180	3,480 (75 HP) & 6,700 (180 HP)	16 & 24	59 & 74	1930	2010

The lift station has a four pump system which discharges to dual 16-inch and 24-inch force mains. There is also an 8-inch force main that was previously utilized by a 20 HP pump, which has been removed from service. The two active force mains are tributary to the influent channel at the Main WWTF just upstream of the influent flow meter (Parshall flume). The channel is approximately 1600 feet from Riverside Lift Station. This lift station utilizes two mechanical fine screens to separate non-biological solids from the raw sewage prior to conveyance to the Main WWTF. The trapped solids are then sent to a washer and compactor that deposit the resulting debris into a dumpster for disposal. The screening system is served by a protected water system.

4.1.2 Strengths and Deficiencies

The Riverside Lift Station is in fair condition. The control panel is located in the northern section of the building in a separate room, which allows for easier access as well as isolation from the corrosive lift station environment.

The two 75 HP pumps were serviced in 2010 (one had the motor stator replaced, one had the stator rewound), but these pumps are in need of replacement. The mechanical fine screens do not operate efficiently; when both screens are running, significantly less material is captured than when only one of the screens is in use. This issue should be investigated, as screening prevents deterioration of the pumps and prevents ragging at the treatment facility.

There have been leaks reported in the pump check valves, which may need to be serviced or replaced. The large ball-check valves on the 24-inch discharge force main slam closed and actually bounce within the valve body when the 180 HP pumps shut off, despite the fact that these pumps are equipped with reduced voltage drives. The resulting impacts have caused pipe breaks in the past, and are likely causes for pipe damage and leaks. The ball check valves should be replaced with slow-closing swing check valves to reduce the water hammer caused by hydraulic transients.

The bridge crane hoist needs to be replaced, and the integrity of the structure itself should be evaluated. The mechanical fine screens cannot rotate completely out of the influent channels without hitting the roof of the station. The roof of the station should therefore be replaced and raised with any future rehabilitation that includes the replacement of the bridge crane and/or screens. Proper steel supports for rotating the mechanical fine screens up out of the channels for maintenance are not in place and should be installed with any future rehabilitation

The generator room floor slab has settled approximately 2 inches from its original elevation, which must be investigated. The old underground diesel tank south of the station should be removed and the generator itself is beyond its service life and should be replaced with any future rehabilitation.

The HVAC system should be replaced, and programming within SCADA should be modified to utilize the data from the magnetic flow meter in this lift station to record its contribution to the Main WWTF influent flow (in addition to the Parshall flume).

Table 4-3 | Riverside Lift Station – Strengths and Deficiencies

Category	Strength	Deficiency
Operations		• Screening inefficiency • Needs new crane system and pumps
Maintenance		• Leaking check valves
Aesthetics		• Must be improved
Mechanical & Electrical	• Control panel in separate room • Generator in separate room	• Old diesel generator should be replaced with natural gas, tank removed
Miscellaneous		• HVAC • Needs a new roof • Generator room floor slab

4.1.3 Pump Performance

The drawdown test was conducted on June 19, 2014. Two trials were conducted using all four pumps. A follow-up drawdown test was conducted on August 15, 2014 for the individual 180 horsepower pumps. The tested flow rates and pressures are all below design, and are outside of the +/- 8% variance allowed by the Hydraulic Institute. It is recommended that the pumps be serviced and the force main be televised to determine the cause of these low flow rates. If necessary, the pumps should be replaced. It is also recommended that Riverside Lift Station be considered for replacement. An estimate for this work is included below:

Table 4-4 | Riverside Lift Station – Pump Drawdown Test Results

Pump	Rated Capacity	Test Flow Rate and Pressure	Tolerance (flow and pressure)
No. 1 (75 HP)	3,480 gpm @ 59.0' TDH	1,980 gpm @ 52.2' TDH	-43% and -12%
No. 2 (75 HP)	3,480 gpm @ 59.0' TDH	2,280 gpm @ 53.3' TDH	-34% and -10%
No. 3 (180 HP)	6,700 gpm @ 73.5' TDH	5,150 gpm @ 46.4' TDH	-23% and -37%
No. 4 (180 HP)	6,700 gpm @ 73.5' TDH	6,710 gpm @ 46.4' TDH	0% and -37%
Firm Capacity		9,325 gpm	

Table 4-5 | Riverside Lift Station Replacement – Probable Costs

GENERAL CONDITIONS	$622,500
SITEWORK	$747,500
LIFT STATION REPLACEMENT	$2,269,150
ELECTRICAL & CONTROLS	$558,300
CONSTRUCTION SUB-TOTAL	**$4,197,450**
CONTIGENCY @ 20%	**$839,490**
ESTIMATED CONSTRUCTION COST	**$5,036,940**
ENGINEERING (14%)	**$705,172**
PROJECT TOTAL	**$5,742,112**

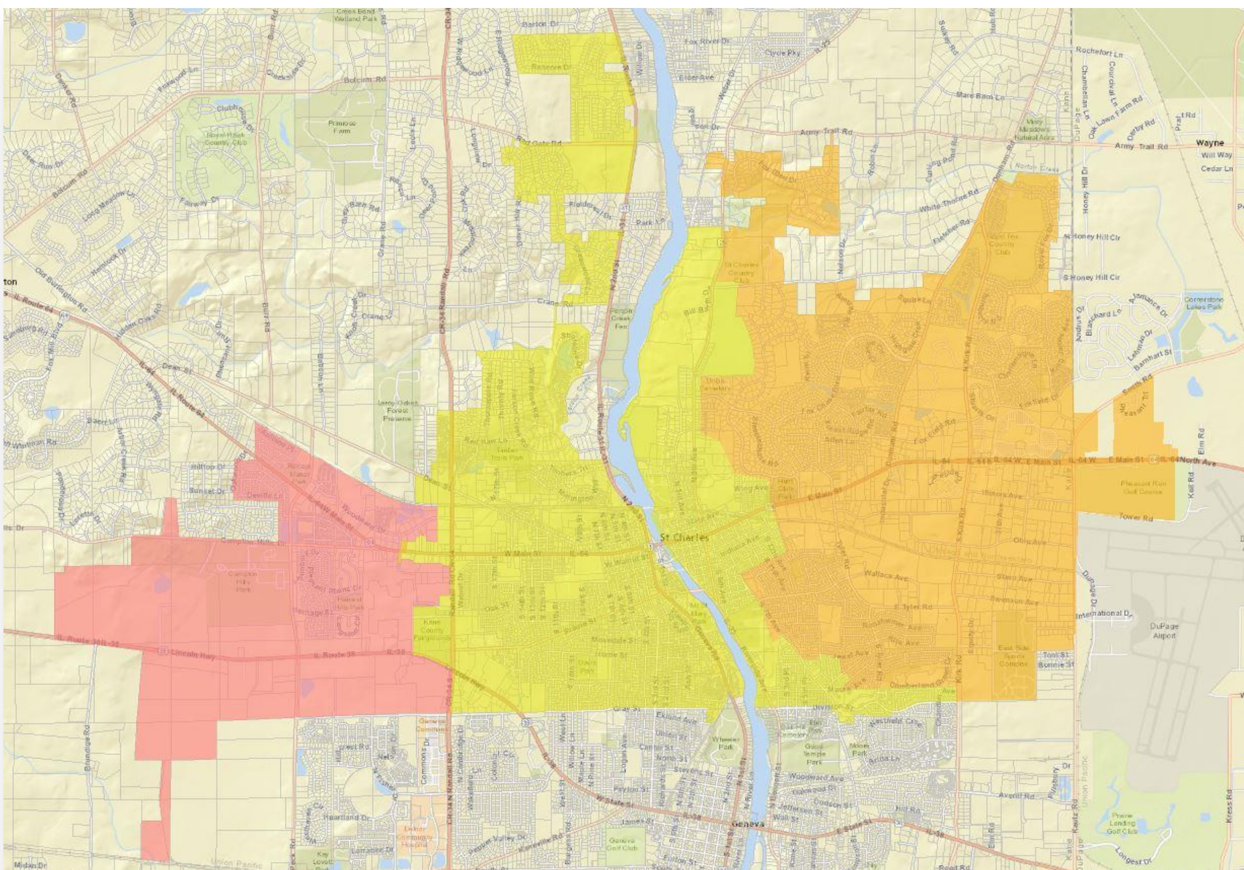
The scope of work involved with replacing the Riverside Lift Station is substantial. Factors that must be taken into account in the design of this work include the site constraints posed by the Fox River and Route 25, groundwater concerns, disposal of contaminated soils, and unknown bedrock conflicts. There will also be challenges for traffic control, material and equipment storage, parking and access. However, the resulting lift station would have adequate capacity, improved layout with bypass capabilities, better access for operation and maintenance, and confidence in the life of the structure. The existing emergency generator room and underground storage tank would be removed first. Then the influent channels, wet well, valve room and superstructure of the new lift station would be constructed to the south of the existing structure. Once new piping connections are made and start-up is complete, the remainder of the existing lift station would be removed and the new generator and odor control system room will be constructed.

4.2 EAST SIDE LIFT STATION

4.2.1 General Description

East Side Lift Station, originally constructed in 1973, is located along Seventh Avenue Creek at the northeast corner of the Main WWTF property. Prior to construction of this lift station, the service area was tributary to the Riverside Lift Station via an interceptor sewer along Seventh Avenue Creek. The interceptor is currently maintained as an emergency overflow in the event that the East Side Lift Station is unable to handle peak wet weather flows. The service area is shown below in orange.

Exhibit 4-3 | East Side Lift Station Service Area (Orange)



The capacity of the existing wet well is 11,445 gallons at high water level. The range in the wet well is only two feet from "Pump On" to "High Level Alarm". As a result, flow pacing over the entire flow range is vital to minimize start and stop operations, which is attained by level transducers and VFD's on the pumps.

Table 4-6 | East Side Lift Station – Pump and Force Main Data

Number of Pumps	Pump Manuf. & Type	Pump Motor (HP)	Pump Rated Capacity (GPM)	Force Main Dia. (inch)	Rated TDH (feet)	Installation Date	Rehab Date
4	Flowserve Submersible	100	4,345	16	62	1973	2010

The East Side Lift Station currently serves approximately 23,039 PE. The 2010 rehabilitation of the lift station included the replacement of the existing 50 horsepower dry-well pumps with 100 horsepower submersible pumps, the installation of variable frequency drives and controls, and the replacement of the bar screen with a mechanical fine screen and washer/grinder/compactor. This rehabilitation expanded the lift station's rated capacity to 14.0 MGD.

4.2.2 Strengths and Deficiencies

The station does not have any major maintenance issues. There was a slight difference in flow rate readings between the control panel and MAG meter during the drawdown test which may require a calibration of the flowmeter and transmitter. The on-site generator ensures the station will remain operational during a power outage, but the generator has recently failed and is in the process of being replaced with the City's emergency funds.

The conditions in the wet area of the lift station promote mold growth, as seen above on the stainless steel walls of the fine screen. The temperature in the control room is high and cannot be maintained by the existing HVAC system. The replacement of the current system with improved climate control is recommended.

Description	Elevation (ft.)	Height Above Wet Well Floor (ft.)
Top of Diverter Gates	697.00	8.50
Top of Influent Pipe	696.60	8.10
High Water Level Upstream of Screen*	695.51	7.01
Invert of Influent Pipe	694.60	6.10
High Water Level Downstream of Screen*	694.11	5.61
Current Level Setting in Wet Well*	694.00	5.50
Floor of Influent Channels	693.00	4.50
Floor of Sump Prior to Wet Well	691.50	3.00
Low Water Level	690.22	1.72
Floor of Wet and Dry Wells	688.50	0.00

*NOTE: These numbers are from as-built drawings, and may have been modified.

Due to the constraints of the lift station influent channel, the depth of flow at the end of the channel must be approximately 1.06 feet to convey 14 MGD (PWWF and the lift station's rated capacity). The depth of flow immediately downstream of the fine screen is calculated to be 1.11 feet. Design head loss through the screen is approximately 1.41 feet, which makes a depth of flow upstream of the fine screen 2.51 feet. The existing fine screen, even when run in HAND, surcharges the upstream collection system and overflows the four-foot-tall diversion gates and bypasses to the 7th Avenue Creek sewer and ultimately to the Riverside Lift Station.

During flow surges, the mechanical fine screen does not permit enough throughput and the collection system surcharges (this reportedly occurs when the flows exceed 4 MGD). When the surcharge gets to be 0.4 feet above the top of the influent pipe, flow is able to go over the diverter gates. This means that the fine screen is currently creating more than 2.895 feet of head loss. This might be due to the perforated plates on the screen not be getting cleared of debris by the spray nozzles and brushes at the top section of the equipment. It is recommended that the condition of the fine screen be evaluated by the manufacturer to determine the cause for this increased headloss. It is also recommended that the City determine what impact it will have on the upstream collection system if the water level downstream of the fine screen is brought back to manufacturer's recommendation (3.09 ft above channel bottom). If it is determined that the water levels may be increased, the set point in the wet well should be raised and the diversion gates should be provided with taller plates or extensions for the existing plates, accordingly.

Maintenance of the washer/grinder/compactor requires the removal of the auger rotor assembly every year, but has not been performed since installation in 2010. This work is difficult due to space constraints in the station, but can be done according to the manufacturer's representatives. Also, the discharge chute from this equipment makes it extremely difficult to remove pumps from the station for servicing. Consideration should be given to modifying the walls of the station (either to the east or through the existing overhead door) and/or the orientation of the washer/grinder/compactor. The position of the trolley beam above the pumps makes it difficult to get pumps completely out of the wet well for servicing. If the walls of the station are to be modified, consideration of a realignment of this beam should be given.

The roadway to the lift station should be widened for better access, if possible. Programming of the VFDs should be modified to "soften" the ramp-up and slow-down of the submersible pumping equipment to better maintain a constant level in the wet well. Programming within SCADA should be modified to utilize the data from the magnetic flow meter in this lift station to record its contribution to the Main WWTF influent flow (in addition to the Parshall flume).

Table 4-7 | East Side Lift Station – Strengths and Deficiencies

Category	Strength	Deficiency
Operations		• Difference in flow readings • VFD, SCADA programming
Maintenance		• Difficult to get the pumps out of wet well
Aesthetics	• Enclosed in a building on treatment plant property	• Access Road
Mechanical & Electrical		• Old diesel generator and tank
Miscellaneous		• High temperature in the control room

4.2.3 Pump Performance

The drawdown test was conducted on June 19, 2014. Two trials were conducted running each of the four pumps. Two tests were then performed with all pumps running together to determine the lift station capacity. Based on the test results, the pumps appear to be operating above the design point and are adequately sized for the tributary flow. However, the tested flow rate for pump No. 1 and No. 4 are outside of the +/- 8% variance allowed by the Hydraulic Institute. It is recommended that the discharge force main be fitted with pressure gages to ensure that the pumps are operating within the safe operating range of the manufacturer's pump curve.

Table 4-8 | East Side Lift Station – Pump Drawdown Test Results

Pump	Rated Capacity	Test Flow Rate (gpm)	Tolerance
No. 1	4,345 gpm @ 62' TDH	4,804	11%
No. 2	4,345 gpm @ 62' TDH	4,405	1%
No. 3	4,345 gpm @ 62' TDH	4,353	0%
No. 4	4,345 gpm @ 62' TDH	4,737	9%

4.3 7TH & DIVISION LIFT STATION

4.3.1 General Description

The 7th & Division Lift Station's service area is generally bound by Moore Avenue on the north, Division Street on the south, 7th Avenue on the west and Kirk Road on the east. The lift station serves approximately 500 PE. The lift station discharges into the collection system at the intersection of 7th Avenue and Moore Avenue and is tributary to Riverside Lift Station.

The pre-engineered lift station was originally constructed in 1974 with a wet well/dry well configuration. Since its construction, the dry well has been converted to a second wet well and submersible pumps were installed.

The design life for lift stations of this type is twenty-five years. The lift station was rehabilitated in 2007, and this work included the installation a new pre-cast concrete lids, pumps, flow meter, controls, generator and transfer switch. The pumps were installed in 2007, replaced in 2009 and replaced once again in 2014.

Table 4-9 | 7th & Division Lift Station – Pump and Force Main Data

Number of Pumps	Pump Manuf. & Type	Pump Motor (HP)	Pump Rated Capacity (GPM)	Force Main Dia. (inch)	Rated TDH (feet)	Installation Date
2	Gorman-Rupp Submersible	4	220	6	35	2014

Table 4-10 | 7th & Division Lift Station – Strengths and Deficiencies

Category	Strength	Deficiency
Operations	• Bypass capabilities	• I&I issues • Access
Maintenance		• Severe ragging and grease issues
Aesthetics	• Partially hidden by vegetation	• Deteriorating concrete and fence
Mechanical & Electrical	• New pumps	• Service problems with generator, deteriorating control panel
Miscellaneous		• Not connected to SCADA

4.3.2 Strengths and Deficiencies

The 7th & Division Lift Station has the ability to bypass flow. Ragging and grease are a problem at this station. The control panel is in poor condition, as it is old and deteriorating. A rehabilitation project should include converting the current two wet well system to a single wet well with a valve vault. Improving vehicle access to the station, control systems and integration into the City's SCADA system is also recommended. :

4.3.3 Pump Performance

No drawdown test was conducted, as the pumps were in the process of being replaced. It is recommended that the 7th & Division Lift Station be a priority for rehabilitation/replacement, and an estimate for the cost of this work is included below:

Table 4-11 | 7th & Division Lift Station Replacement – Probable Costs

GENERAL CONDITIONS	$92,900
SITEWORK	$75,500
LIFT STATION REPLACEMENT	$136,650
ELECTRICAL & CONTROLS	$131,500
CONSTRUCTION SUB-TOTAL	**$436,550**
CONTIGENCY @ 20%	**$87,310**
ESTIMATED CONSTRUCTION COST	**$523,860**
ENGINEERING (14%)	**$73,340**
PROJECT TOTAL	**$597,200**

4.4 WASHINGTON AVENUE LIFT STATION

4.4.1 General Description

The Washington Avenue Lift Station serves only seven houses in an area between Seventh Avenue and Ninth Avenue. The lift station discharges to a gravity sewer along Ninth Avenue and is tributary to the East Side Lift Station.

The Washington Avenue Lift Station was constructed in 1987 and has two small submersible pumps. The lift station is in generally sound condition. The only rehabilitation to the station since its construction included replacement of one pump and replacement of the guiderail system.

Table 4-12 | Washington Avenue Lift Station – Pump and Force Main Data

Number of Pumps	Pump Manuf. & Type	Pump Motor (HP)	Pump Rated Capacity (GPM)	Force Main Dia. (inch)	Rated TDH (feet)	Installation Date
2	Meyers Submersible Grinder	2	22	6	15	1987

4.4.2 Strengths and Deficiencies

Due to the small amount of flow that is tributary to this lift station, the equipment has little wear and does not require rehabilitation. Washington does not have a generator, nor does it have bypass pumping capabilities. There is no transducer, the access hatch is in need of replacement, and one pump needs cleaning.

Table 4-13 | Washington Avenue Lift Station – Strengths and Deficiencies

Category	Strength	Deficiency
Operations		• No transducer, no bypass capabilities
Maintenance	• No major issues, one pump needs cleaning	
Aesthetics	• Covered in a front lawn	
Mechanical & Electrical		• No on-site generator
Miscellaneous		• Needs a new hatch • Not connected to SCADA

4.4.3 Pump Performance

No drawdown test was conducted at the Washington Avenue Lift Station due to low influent flow.

4.5 COUNTRY CLUB LIFT STATION

4.5.1 General Description

Country Club Lift Station was constructed in 1988 when the St. Charles Country Club moved its clubhouse to the east side of Illinois Route 25. The lift station serves only the Country Club and is located north of the club house. The 4-inch force main discharges to the collection system at the intersection of Persimmon Drive and Country Club Road. From there, the flow is tributary to the East Side Lift Station.

Table 4-14 | Country Club Lift Station – Pump and Force Main Data

Number of Pumps	Pump Manuf. & Type	Pump Motor (HP)	Pump Rated Capacity (GPM)	Force Main Dia. (inch)	Rated TDH (feet)	Installation Date
2	Meyers Submersible	5	80	4	47	1988

4.5.2 Strengths and Deficiencies

The lift station was recently equipped with a new generator, however the City would prefer a natural gas generator at this location. The existing wet well cover is fiberglass and has deteriorated from exposure to ultraviolet sunlight. A new cover is highly recommended for safety and liability purposes. The valve vault is unusually shallow and routinely fills with ground water. Standing water has been noted by the staff in the valve vault above the piping. As a result, maintenance requires installation of a sump pump prior to access and working room is limited.

Country Club Lift Station is one of the older lift stations in the system. The nature of the influent and the age of the pumps have resulted in increased maintenance time and expense. There are rags and large amounts of grease sent to the station from the Country Club kitchens, which have constricted the collection system upstream of this lift station and affected equipment and the discharge force main as well. The force main needs to be cleaned out and it is recommended that a grease trap be added prior to the station to reduce future damage. The lift station should be connected to the City's SCADA system. This remote location will require installation of a fiber-optic cable from Country Club Road or from the Well 9 site.

Table 4-15 | Country Club Lift Station – Strengths and Deficiencies

Category	Strength	Deficiency
Operations		• Aging pumps
Maintenance		• Ragging and severe amounts of grease
Aesthetics	• Hidden by vegetation	• On country club property
Mechanical & Electrical		• Old control panel • Natural gas generator preferred
Miscellaneous		• Not connected to SCADA

4.5.3 Pump Performance

No drawdown test was performed at the Country Club Lift Station due to low influent flow.

It is recommended that the lift station be considered for a rehabilitation project, and an estimate for the cost of this work is included below. Please note that this does not include any easement acquisition costs as may be necessary for the work.

Table 4-16 | Country Club Lift Station Rehabilitation – Probable Costs

GENERAL CONDITIONS	$112,600
SITEWORK	$66,900
LIFT STATION REHABILITATION	$155,100
ELECTRICAL & CONTROLS	$131,500
CONSTRUCTION SUB-TOTAL	**$466,100**
CONTIGENCY @ 20%	**$93,220**
ESTIMATED CONSTRUCTION COST	**$559,320**
ENGINEERING (14%)	**$78,305**
PROJECT TOTAL	**$637,625**

4.6 PHEASANT RUN TRAILS LIFT STATION

4.6.1 General Description

The Pheasant Run Trails Lift Station serves multi-family development north of Illinois Route 64 and south of Smith Road. The lift station was constructed in 1997 and serves approximately 925 PE including the Hilton Inn & Gardens Hotel. The 6-inch force main extends to the intersection of Illinois Route 64 and Kautz Road. The flow is tributary to the East Side Lift Station. There is potential for residential and commercial development in the lift station'.

Table 4-17 | Pheasant Run Trails Lift Station – Pump and Force Main Data

Number of Pumps	Pump Manuf. & Type	Pump Motor (HP)	Pump Rated Capacity (GPM)	Force Main Dia. (inch)	Rated TDH (feet)	Installation Date	Rehab Date
2	Hydromatic Submersible	15	468	6	42	1997	2009

4.6.2 Strengths and Deficiencies

The influent line to the wet well deposits flow, including rags and grease, directly on top of one of the pumps. This builds up debris over time, making the pumps difficult to remove for maintenance. This issue may be addressed by physically rotating the pumps and corresponding rails within the wet well. To address the excessive ragging, the City may install a pump that will pass larger solids (i.e. screw centrifugal pumps), install a chopper pump to reduce the solid size, or require that the tributary users provide pretreatment screening. Vehicle access is a concern, and relocating the vehicle entrance to the south will allow easier access for maintenance and emergency repairs. The fencing is broken in certain places and does not provide adequate protection for the station, and should be replaced. Bypassing the station is difficult, as the bypass connection is located in a confined space. Finally, the lift station should be connected to the City's SCADA as part of any rehabilitation. The Pheasant Run Trails Lift Station is a candidate for rehabilitation, but the work recommended above will be performed by the City through the maintenance budget.

Table 4-18 | Pheasant Run Trails Lift Station – Strengths and Deficiencies

Category	Strength	Deficiency
Operations		• Access to bypass
Maintenance		• Ragging, some grease
Aesthetics		• Broken fence
Mechanical & Electrical	• On-Site Generator	
Miscellaneous		• Pump location • Not connected to SCADA

4.6.3 Pump Performance

The drawdown test was conducted on June 19, 2014. Two trials were conducted running each pump individually, and also running both pumps simultaneously. Based on the test results, the pumps appear to be adequately sized for the tributary flow. However, the tested flow rate is outside of the +/- 8% variance allowed by the Hydraulic Institute. It is recommended that the discharge force main be fitted with pressure gages to ensure that the pumps are operating within safe operating range of the manufacturer's pump curve. It is also recommended that the orientation of the pumps be modified as part of the rehabilitation of the station. When the more critical lift stations are rehabilitated or replaced, this station should be considered for rehabilitation.

Table 4-19 | Pheasant Run Trails Lift Station – Pump Drawdown Test Results

Pump	Rated Capacity	Test Flow Rate (gpm)	Tolerance
No. 1	468 gpm @ 42' TDH	532	14%
No. 2	468 gpm @ 42' TDH	648	38%

4.7 ROYAL FOX LIFT STATION NO. 2

4.7.1 General Description

Royal Fox Lift Station No. 2 is located along Dunham Road immediately north of St. Charles East High School. The lift station was constructed in 1988 and serves approximately 2,500 PE, which equates to a peak hourly flow of 570 gpm. The lift station firm capacity is 650 gpm and is nearing build-out. The 8-inch force main extends south to a 15-inch gravity sanitary sewer along Dunham Road. The flow from this service area is tributary to the East Side Lift Station.

Table 4-20 | Royal Fox Lift Station No. 2 – Pump and Force Main Data

Number of Pumps	Pump Manuf. & Type	Pump Motor (HP)	Pump Rated Capacity (GPM)	Force Main Dia. (inch)	Rated TDH (feet)	Installation Date	Rehab Date
2	4-inch ABS XFP 100G CB1 Submersible	28	650	8	95	1988	2013

4.7.2 Strengths and Deficiencies

The lift station was rehabilitated in 2013. This work included replacement of pumps, valves, and piping within the station, replacement of the control systems and traffic box, rehabilitation of the lift station structure with a spray-applied structural lining, installation of a magnetic flow meter and bypass pump connection vault, installation of a new concrete lid, connection to the City SCADA, and site improvements.

There are no major outstanding operational or maintenance issues. Occasionally, a pump will fail to start in automatic mode. The operational staff currently fixes this problem by rebooting the power on the control panel after being called out for a high level alarm.

Table 4-21 | Royal Fox Lift Station No. 2 – Strengths and Deficiencies

Category	Strength	Deficiency
Operations	• New pumps	
Maintenance	• No issues noted	
Aesthetics	• Fresh paint and new pavement	
Mechanical & Electrical	• New control panel	
Miscellaneous		• Large amounts of tributary I&I

4.7.3 Pump Performance

The drawdown test was conducted on June 26, 2014. Two trials were conducted running each pump individually and also running both pumps simultaneously. Based on the test results, the pumps appear to be adequately sized for the tributary flow. However, the tested flow rate is outside of the +/- 8% variance allowed by the Hydraulic Institute. It is recommended that the discharge force main be fitted with pressure gages to ensure that the pumps are operating within the safe operating range of the manufacturer’s pump curve.

Table 4-22 | Royal Fox Lift Station No. 2 – Pump Drawdown Test Results

Pump	Rated Capacity	Test Flow Rate (gpm)	Tolerance
No. 1	650 gpm @ 95’ TDH	761	17%
No. 2	650 gpm @ 95' TDH	762	17%

4.8 ROYAL FOX LIFT STATION NO. 1

4.8.1 General Description

Royal Fox Lift Station No. 1, constructed in 1988, is located at the intersection of Royal Fox Drive and Dunham Road. The lift station serves the northern part of Royal Fox Subdivision, approximately 500 PE. The lift station contains two submersible pumps each rated for 200 gpm and receives a peak hourly flow of approximately 111 gpm. Therefore, the lift station has plenty of remaining capacity. The lift station was rehabilitated in the summer of 2014.

Table 4-23 | Royal Fox Lift Station No. 1 – Pump and Force Main Data

Number of Pumps	Pump Manuf. & Type	Pump Motor (HP)	Pump Rated Capacity (GPM)	Force Main Dia. (inch)	Rated TDH (feet)	Installation Date	Rehab Date
2	4-inch ABS XFPD 100E-CB1 Submersible	10	200	6	60	1988	2014

4.8.2 Strengths and Deficiencies

The lift station only experiences capacity issues when the pool is drained at the Country Club. This is exacerbated by grease issues in the discharge force main, which has been known to reduce the effective pipe size to 2-inch at the discharge manhole. The rehabilitation of this lift station was completed in the fall of 2014, and included replacement of pumps, valves, and piping within the station, replacement of the control systems and traffic box, rehabilitation of the lift station structure with a spray-applied structural lining, installation of an above-grade bypass pump connection, installation of a new concrete lid, and site improvements. In order to address the grease issues, cleanout connections and structures were installed along the discharge force main. As seen below, heavy grease is received at this lift station and builds up on the guide rails, pumps and wet well walls.

4.8.3 Pump Performance

The drawdown test was conducted on March 16, 2015. Two trials were conducted running each pump individually and also running both pumps simultaneously. Based on the test results, the pumps appear to be adequately sized for the tributary flow. However, the tested flow rate is below the design point and is outside of the +/- 8% variance allowed by the Hydraulic Institute. It is recommended that the discharge force main be fitted with pressure gages to ensure that the pumps are operating within the safe operating range of the manufacturer's pump curve.

Table 4-24 | Royal Fox Lift Station No. 1 – Pump Drawdown Test Results

Pump	Rated Capacity	Test Flow Rate (gpm)	Tolerance
No. 1	200 gpm @ 60' TDH	175	-13%
No. 2	200 gpm @ 60' TDH	175	-13%

4.9 WOODS OF FOX GLEN LIFT STATION

4.9.1 General Description

The Woods of Fox Glen Lift Station is located in the center of Glenbriar Court in the Woods of Fox Glen Subdivision on the north edge of St. Charles. The 6-inch force main extends through the St. Charles Country Club property and is tributary to the sanitary sewer at the intersection of Country Club Road and Persimmon Drive, and eventually to the East Side Lift Station. The lift station was constructed in 1989 and serves approximately 350 PE. There is not a large amount of future development to add to the basin. The lift station piping has a unique design with the check valves installed in the discharge riser within the wet well. Installation of the check valves in vertical piping is not sound engineering practice, is not code compliant, and solids will settle atop the closed check valve disc.

City staff recommends relocating the check valves to a valve vault where they can be installed horizontally and will be more accessible for maintenance. In addition, the lift station should be upgraded with a flow meter and connection the City's SCADA system. It has been determined that this work will be performed with in-house resources.

Table 4-25 | Woods of Fox Glen Lift Station – Pump and Force Main Data

Number of Pumps	Pump Manuf. & Type	Pump Motor (HP)	Pump Rated Capacity (GPM)	Force Main Dia. (inch)	Rated TDH (feet)	Installation Date	Rehab Date
2	Myers Submersible	20	180	6	111	1989	2009, 2013

4.9.2 Strengths and Deficiencies

The Woods of Fox Glen Lift Station is in good condition. There are no issues that would require immediate attention or rehabilitation. New guide rails and new floats were installed within the last two years. The control panel is old, however the operational staff says that it is in good working condition.

Table 4-26 | Woods of Fox Glen Lift Station – Strengths and Deficiencies

Category	Strength	Deficiency
Operations	• Bypass capabilities	
Maintenance	• No issues noted	
Aesthetics	• Hidden by vegetation	
Mechanical & Electrical	• On-site generator	
Miscellaneous	• New rails and new floats	• Not connected to SCADA

4.9.3 Pump Performance

A drawdown test could not be performed at the Woods of Fox Glen Lift Station due to low influent flow.

4.10 KINGSWOOD LIFT STATION

4.10.1 General Description

The Kingswood Lift Station is a duplex submersible lift station located north of Foxfield Drive on King Edwards Street. The lift station serves approximately 830 PE, which equates to a peak hourly flow of 193 gpm. The existing pumps provide a firm capacity of 400 gpm. The force main discharges to the sanitary sewer system near the intersection of Indian Way and Foxfield Drive. The flow is tributary to the East Side Lift Station.

Table 4-27 | Kingswood Lift Station – Pump and Force Main Data

Number of Pumps	Pump Manuf. & Type	Pump Motor (HP)	Pump Rated Capacity (GPM)	Force Main Dia. (inch)	Rated TDH (feet)	Installation Date
2	Hydromatic Submersible	15	400	8	50	1996

4.10.2 Strengths and Deficiencies

The Kingswood Lift Station is in excellent working condition. There is a bypass connection available; however, access to the bypass location is difficult. The presence of an on-site generator allows the lift station to continue to operate during a power outage. The control panel is in good condition. Ragging is an issue at this station, and consideration should be given to installing a pump that will pass larger solids (i.e. screw centrifugal pumps), or a chopper pump to reduce the solid size. Piping issues have been reported by the operational staff, specifically the flange on Pump 1. The Kingswood Lift Station should be connected to the SCADA system when the City's budget allows.

Table 4-28 | Kingswood Lift Station – Strengths and Deficiencies

Category	Strength	Deficiency
Operations		• Difficult to bypass
Maintenance		• Ragging
Aesthetics	• Hidden by vegetation	
Mechanical & Electrical	• On-site generator	
Miscellaneous		• Not connected to SCADA

4.10.3 Pump Performance

The drawdown test was conducted on June 18, 2014. Two trials were conducted running each pump individually and also running both pumps simultaneously. Based on the test results, the pumps appear to be operating above the design point and are adequately sized for the tributary flow. However, the tested flow rate is outside of the +/- 8% variance allowed by the Hydraulic Institute. It is recommended that the discharge force main be fitted with pressure gages to ensure that the pumps are operating within the safe operating range of the manufacturer's pump curve.

Table 4-29 | Kingswood Lift Station – Pump Drawdown Test Results

Pump	Rated Capacity	Test Flow Rate (gpm)	Tolerance
No. 1	400gpm @ 50' TDH	501	25%
No. 2	400gpm @ 50' TDH	502	26%

4.11 WILD ROSE LIFT STATION

4.11.1 General Description

The Wild Rose Lift Station was constructed in 1980. The lift station is located along Wild Spring Drive and serves approximately 420 PE. The peak hourly flow to the lift station is 94 gpm while the pumps provide a firm capacity of 106 gpm. The force main is tributary to the 18-inch Interceptor, the North Siphon, and ultimately to the Riverside Lift Station.

Table 4-30 | Wild Rose Lift Station – Pump and Motor Data

Number of Pumps	Pump Manuf. & Type	Pump Motor (HP)	Pump Rated Capacity (GPM)	Force Main Dia. (inch)	Rated TDH (feet)	Installation Date	Rehab Date
2	Hydromatic Submersible	5	106	4	25	1980	2011

4.11.2 Strengths and Deficiencies

The grading around the lift station directs surface water into the wet well. The pumps are still in good shape but are at the end of their service life and are in need of replacement. The station is nearing its capacity and the pumps should be upsized. The piping and valves should be replaced. The generator and control panel are both old and deteriorating, and should be replaced along with the lift station lid, access hatches, and control system. The lift station should also be connected to the City's SCADA. The City staff has expressed interest in paving the road leading to the station and raising the grade of the station. Finally, the wet well is rusting through; over the last 30 years, the corrosive atmosphere within the wet well has deteriorated the steel walls of the structure.

Table 4-31 | Wild Rose Lift Station – Strengths and Deficiencies

Category	Strength	Deficiency
Operations	• Pumps are in good condition	• No level transducer • Needs new valves
Maintenance		• Ragging and grease
Aesthetics	• Good distance from roadway	• No vegetation surrounding generator or control panel • Needs a paved road to the station
Mechanical & Electrical		• Old deteriorating control panel & generator
Miscellaneous		• Candidate for rehabilitation • Not connected to SCADA

4.11.3 Pump Performance

A drawdown test could not be performed at the Wild Rose Lift Station due to low influent flow. Based on staff's recommendations, the lift station was to be replaced in 2008. In 2011, the height of the wet well was raised, but the other improvements have not yet been addressed. It is recommended that the Wild Rose Lift Station be considered for a replacement project, and an estimate for the cost of this work is included below.

Table 4-32 | Wild Rose Lift Station Replacement – Probable Costs

GENERAL CONDITIONS	$95,100
SITEWORK	$85,500
LIFT STATION REPLACEMENT	$141,400
ELECTRICAL & CONTROLS	$131,500
CONSTRUCTION SUB-TOTAL	**$453,500**
CONTIGENCY @ 20%	**$90,700**
ESTIMATED CONSTRUCTION COST	**$544,200**
ENGINEERING (14%)	**$76,188**
PROJECT TOTAL	**$620,388**

4.12 RED GATE LIFT STATION

4.12.1 General Description

The Red Gate Lift Station, constructed in 1988 and upgraded in 1999, is located on the south side of Crane Road across from Crane Road Estates. The lift station currently serves approximately 2,000 students at St. Charles North High School and 196 residential lots, which equates to approximately 1,150 PE with a calculated peak hourly flow of 262 gpm. There is projected future development in the collection system that will be tributary to this lift station. However, with a firm capacity of 506 gpm, the lift station has a significant amount of reserve capacity.

The lift station was replaced in 2006, and is a packaged design including two submersible pumps, valve vault and stand-by generator. The 6-inch force main extends southward across Ferson Creek and discharges into the collection system in the Wild Rose Subdivision. From there, flow is conveyed by gravity to the Riverside Lift Station.

The City has serious concerns regarding the capacity of the downstream collection system under peak wet weather flow conditions. As discussed in Section 3, the downstream collection system is beyond its conveyance capacity during the 10-year Peak Wet Weather Flow. Projects have been identified in Section 3 to alleviate the surcharged conditions.

Table 4-33 | Red Gate Lift Station – Pump and Force Main Data

Number of Pumps	Pump Manuf. & Type	Pump Motor (HP)	Pump Rated Capacity (GPM)	Force Main Dia. (inch)	Rated TDH (feet)	Installation Date
2	Hydromatic Submersible	20	506	6	66	2005

4.12.2 Strengths and Deficiencies

Red Gate is currently in good operating condition. Ragging and grease are an issue, and may decrease the service lift of the pumping equipment. There is a crack in the body of one of the check valves in the valve vault. At this time, the crack is not causing any operational issues, however it needs to be monitored closely by City staff. It is recommended that this check valve be replaced as soon as possible. The control panel will occasionally shut off and will send an alarm to the treatment plant, requiring an operator to reboot the power at the station.

Table 4-34 | Red Gate Lift Station – Strengths and Deficiencies

Category	Strength	Deficiency
Operations	• Bypass capabilities	• Crack in a check valve
Maintenance		• Ragging and grease
Aesthetics		• Close to road, needs traffic protection
Mechanical & Electrical	• On-site generator	• Control panel shuts off
Miscellaneous	• Pumps in good condition	• Not connected to SCADA

4.12.3 Pump Performance

The drawdown test was conducted on June 20, 2014. Two trials were conducted running each pumps individually and also running both pumps simultaneously. Based on the test results, the pumps appear to be operating above the design point and are adequately sized for the tributary flow. However, the tested flow rate is above the design point and pump No. 1 is outside of the +/- 8% variance allowed by the Hydraulic Institute. It is recommended that the discharge force main be fitted with pressure gages to ensure that the pumps are operating within the safe operating range of the manufacturer's pump curve.

Table 4-35 | Red Gate Lift Station – Pump Drawdown Test Results

Pump	Rated Capacity	Test Flow Rate (gpm)	Tolerance
No. 1	506 gpm	578	14%
No. 2	506 gpm	543	7%

4.13 OAK CREST LIFT STATION

4.13.1 General Description

The Oak Crest Lift Station was constructed in 2000. The lift station is located on Crestwood Lane in the Oak Crest Subdivision. The lift station includes a two-pump system, back-up generator and auto-transfer switch.

Table 4-36 | Oak Crest Lift Station – Pump and Force Main Data

Number of Pumps	Pump Manuf. & Type	Pump Motor (HP)	Pump Rated Capacity (GPM)	Force Main Dia. (inch)	Rated TDH (feet)	Installation Date
2	Hydromatic Submersible	7.5	100	6	43	2000

4.13.2 Strengths and Deficiencies

The Oak Crest Lift Station is in good operating condition. The presence of an on-site generator allows the station to operate under power outages. There is a reported seal failure in one of the pumps. Despite the seal fail alarm, the pump continues to operate properly. There is also a replacement pump available in stock in case of failure. However, the pumps, piping and control system are nearing the end of their service life and should be replaced with any future rehabilitation. The lift station is close to the road, and traffic protection improvements should be included with future rehabilitation of the lift station. Finally, the Oak Crest Lift Station should be connected to the City's SCADA.

Table 4-37 | Oak Crest Lift Station – Strengths and Deficiencies

Category	Strength	Deficiency
Operations	• Bypass capabilities	• Pump seal failure, spare pump in stock
Maintenance	• No issues noted	
Aesthetics		• Close to road, needs traffic protection
Mechanical & Electrical	• On-site generator	
Miscellaneous		• Not connected to SCADA

4.13.3 Pump Performance

No drawdown test was performed at the Oak Crest Lift Station due to low influent flow.

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SECTION 5

EXISTING MAIN WASTEWATER TREATMENT FACILITY

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5. EXISTING MAIN WASTEWATER TREATMENT FACILITY

5.1 GENERAL BACKGROUND AND EXPANSION HISTORY

The City of St. Charles original wastewater treatment facility was located along the banks of the Fox River near the Riverside Lift Station. In the early 1930's, a new plant was constructed up the hill on what is now the wastewater treatment facility site. The first plant on this site consisted of an Imhoff tank. Shortly after construction of the first structure, a new sludge management process was introduced referred to as the Putnam process. At this time, the first section of the existing sludge handling building was constructed in addition to several sludge storage tanks, two of which remain to this day. The Putnam Process was abandoned during later expansions, but the building that housed it was incorporated into the Sludge Handling Building.

In the early 1950's, the plant was expanded to include two primary clarifiers, a 130-foot diameter trickling filter and final clarifier. The existing Imhoff tank was converted to sludge digestion. In 1966, the City constructed a contact stabilization process and rectangular final clarifiers. The existing final clarifier was converted to a chlorine contact tank. Once the improvements were completed the existing trickling filter was abandoned.

Due to growth within the community the wastewater treatment facility was expanded again in 1972. The Stage One project was funded through the newly implemented Federal Grants Program brought about by the Clean Water Act. The project included new headworks, primary clarifiers, final clarifier revisions and sludge pumping improvements.

The new headworks included a Parshall flume for flow monitoring, chemical addition for phosphorous removal and two aerated grit chambers for removal of inorganic settable solids. The two primary clarifiers were 100 feet long by 40 feet wide. They were fitted with a traveling bridge collector mechanism, which spanned both tanks. The improvements to the final clarifiers enhanced the sludge return capabilities of the existing biological process. The sludge pumping improvements allowed the existing primary clarifiers to be converted to waste activated sludge holding.

The second phase was constructed in 1973. The project scope included construction of the East Side Lift Station, two new 65-foot diameter roughing filters, two additional final clarifiers, and two chlorine contact tanks. The project also included renovation of the laboratory/ sludge handling building. The improvements including pumps, piping, a coil filter press and conversion of the old chlorine contact tank to sludge holding.

In 1981, Riverside Lift Station was rehabilitated and expanded to serve the communities increasing demands. The rehabilitation included conversion of the dry well to additional wet well capacity, installation of submersible pumps, fine mechanical screens and backup generator. The project also included construction of the brick and block building to allow the operators above grade access to the station. This project was also funded through the Illinois EPA Grant program.

In 1985, the City of St. Charles upgraded the wastewater treatment facility to include excess flow treatment facilities. The project included construction of two 120-foot diameter final clarifiers and conversion of the existing rectangular units to first flush and excess flow clarifiers. In addition, a new chlorine building was constructed.

In 1986, the City increased the plant's sludge dewatering capacity by installing a 2-meter belt filter press (BFP). The project included sludge pumping improvements and a serpentine conveyor, which collected the sludge from both the BFP and existing coil filter and discharged it to a truck dock.

In 1987, the City expanded the capacity of the East Side Lift Station. The lift station included a wet well/ dry well configuration with four centrifugal pumps in parallel. The pumps discharge directly to the wastewater treatment facility headworks through a 16” force main.

In 1989, the Sludge Handling and Excess Flow Improvements were prompted by the USEPA 503 Regulations, which were pending at that time, and applicable excess flow treatment requirements. The sludge handling facilities improvements included construction of an egg-shaped anaerobic digestion complex. The improvements also included sludge pumping and storage modifications. The existing first flush tanks were converted to waste activated sludge holding, while one of the excess flow clarifiers was converted to first flush holding. The project was completed in 1991.

In 1994, the Illinois Pollution Control Board began the promulgation of Rule 94-1, which addressed ammonia nitrogen discharge limits for communities along the Fox and Rock River. After receiving public comment from several of the impacted POTW’s and interested citizen groups the Pollution Control Board implemented the Rule.

In 1996, the City's NPDES permit was under review for re-issuance. The IEPA incorporated both ammonia nitrogen standards and revised chlorine residual limits into the permit. The permit included a compliance schedule for the necessary improvements to meet the new limits. The City completed the construction of the dechlorination facilities in 1996 utilizing in-house resources. This project included equipment installation and piping modifications.

The new ammonia nitrogen limits were 9.4 mg/L monthly during summer month and 8.0 during winter. The City reviewed their existing infrastructure, made minor improvements and adjusted their operational approach to meet the proposed limits. Concurrently the City was receiving odor complaints. Due to limited capital funds the City elected to break projects into phases, which could be implemented on an annual basis. The 1996 Odor and Ammonia Control Project included covering of the 65-foot trickling filters and conversion of the WAS holding tanks to side stream treatment aeration basins. The newly created aeration basins treat the high strength filtrate from sludge dewatering operations prior to the flow being returned to the head of the plant. Covering the trickling filters enhanced the units' performance during winter operation and contained the odors from the units.

The City recognized that the coil filter installed in 1973 was nearing the end of its service life and investigated available sludge dewatering technologies. The City determined that centrifuge technology was the most cost-effective solution. The 1997 Sludge Dewatering Improvements included the removal of the coil filter and installation of the first centrifuge, a new conveyor, pump, and polymer unit. The project design and layout provided for installation of a second unit in the future.

The next phase of the odor control program was completed in 1998. The project included covering the existing Parshall flume, aerated grit basins and primary clarifier launders. The atmosphere under the covers was evacuated through an exhaust system and the odors treated by oxidation with ozone.

In 1999, the traveling bridge primary clarifier mechanism, installed in 1972, was nearing the end of its service life and becoming a maintenance liability. The 2000 Primary Clarifier Improvements included demolition of the existing equipment and installation of traditional chain and scraper collector mechanisms. The existing primary clarifiers were 40 feet wide, however the chain and scraper mechanisms can only span twenty feet. Intermediate walls and drive pads were constructed allowing installation of two mechanisms in each clarifier. The sludge hoppers and pumping system remained the same.

The City installed the second centrifuge, conveyor and polymer unit during the 2001 Sludge Dewatering Improvements. The belt filter press (installed in 1987) was removed shortly after start-up of the second centrifuge.

In May of 2001, the Illinois EPA issued the City's new NPDES Permit. The updated permit included more stringent ammonia nitrogen limits. Recognizing that the City would be forced to complete a major renovation to achieve the new limits the Illinois EPA incorporated a compliance schedule into the permit, which states that the new limits became effective June of 2004. The City commissioned a facility plan update in 2002, which provided a series of recommendations to maintain regulatory compliance and rehabilitate the existing facilities. The City completed construction of the 2002 Nitrification Improvements project in 2005.

The project scope included demolition of the existing trickling filters and salt storage building. The process upgrades included construction of 2.5 million gallons of aeration capacity, blower building, rehabilitation of the existing aeration basins and expansion of the RAS/WAS pump station. Upgrades to the excess flow facilities included conversion of the existing first flush holding tank to an excess flow clarifier. The excess flow treatment process continues to be tributary to the chlorine contact tank. An ultra violet disinfection system was constructed for use with the process flow.

In 2009, the City of St. Charles upgraded the East Side Lift Station and Riverside Lift Station. The improvements to East Side Lift Station included replacement of all mechanical and electrical components including the fine screen, pumps, piping and controls. The rehabilitation to Riverside Lift Station was limited to screen, valve and variable frequency drive replacement.

The City removed the chemical (ferric chloride) phosphorous removal system and replaced the primary sludge pumping system, primary clarifier cross-collector drive mechanism, odor control system, and associated MCC in the 2010 Headworks Rehabilitation. Other improvements included replacement of the existing rolling grit unit draft tubes, suction lift pumps, grit and primary sludge piping.

In late 2011, an assessment of the Main WWTF was completed. This included an evaluation of all processes and infrastructure, including the Main Sludge Handling Building. The functions within the building included the main switch gear; sludge pumping, holding, thickening and dewatering operations; maintenance facilities; inventory; and offices. The electrical systems, thickening and dewatering equipment had reached the end of their service life and the building required significant structural rehabilitation.

The City of St. Charles proceeded with replacement of the Main Sludge Handling Building in 2011, which included a Facility Plan Update. The improvements were designed in such a configuration that future treatment processes or sludge stabilization upgrades were not negatively impacted. Furthermore, the City and TAI evaluated several centrifuge manufacturers and selected two to perform on-site pilot testing of their equipment. Both manufacturers demonstrated the capability to meet the City's performance requirements. In addition, the existing infrastructure needed to remain in service during construction. The project therefore included construction of the new building in two phases.

The first phase included electrical/ control, sludge thickening and dewatering facilities. The waste activated sludge improvements included WAS holding, sludge feed pumps, polymer unit, gravity belt thickener, TWAS holding and pumping systems. The sludge dewatering improvements included sludge feed pumps, polymer units, two centrifuges and a conveyor in a loading dock. The second phase included construction of an operations building that contains an office, break room, locker room, inventory, and maintenance garage. The Facility Plan Update was submitted in July 2011 and the project was funded through the Illinois State Revolving Fund (SRF) program. Design was completed in December 2011, and construction was completed in the fall of 2014.

The City's existing Main WWTF infrastructure is of varying age and condition. The City has completed a brief audit of each unit process, its capacity, age and condition and developed a series of recommended improvements. The existing NPDES permit limits are included in the next section, followed by the analysis of each unit process starting at the Headworks.

5.2 NPDES PERMIT LIMITS:

Flow

Design Average Flow, MGD	9.0
Design Maximum Flow, MGD	18.35

$CBOD_5$

Monthly Average, mg/L	20
Monthly Average, lbs.	1,501
Weekly Average, mg/L	40
Weekly Average, lbs.	3,002

Suspended Solids

Monthly Average, mg/L	25
Monthly Average, lbs.	1,877
Weekly Average, mg/L	45
Weekly Average, lbs.	3,378

Fecal Coliform

Monthly Maximum (Geometric Mean)	200 per 100 ml

pH

Range	6 - 9

Chlorine Residual

Daily Maximum, mg/L	0.05

Ammonia Nitrogen

March-May, Sept.-Oct.

Daily Maximum, mg/L	1.8
Daily Maximum, lbs.	135
Monthly Average, mg/L	1.5
Monthly Average, lbs.	113

June through August

Daily Maximum, mg/L	1.4
Daily Maximum, lbs.	105
Monthly Average, mg/L	1.3
Monthly Average, lbs.	98

November through February

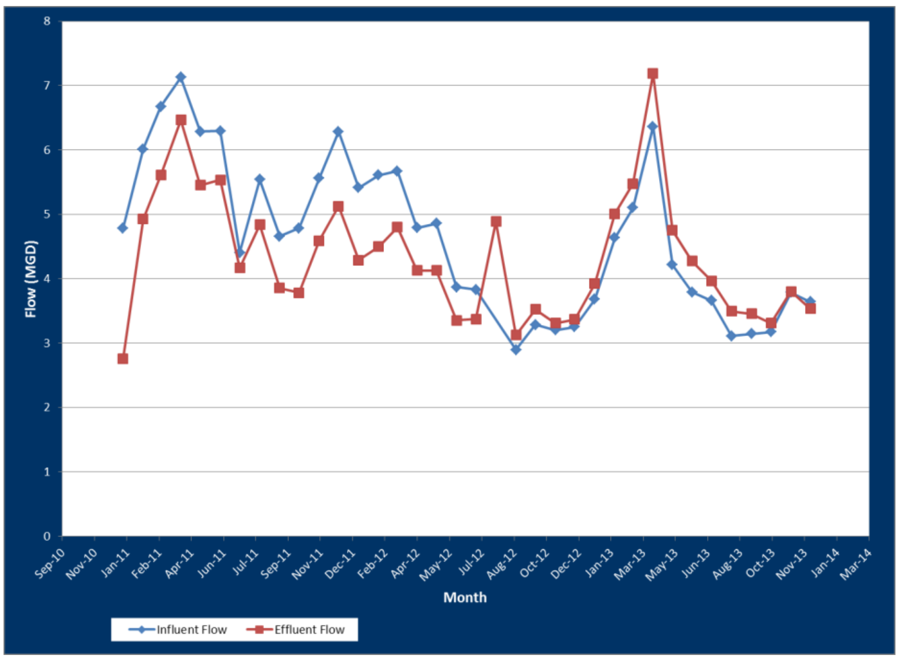
Daily Maximum, mg/L	3.4
Daily Maximum, lbs.	255

5.3 PLANT PERFORMANCE

5.3.1 Influent Flow

The Design Average Flow for the Main Facility is 9.0 MGD. The Illinois EPA reviews the three low flows months for any twelve-month period. The average of the three low flow months is compared to the design average flow to determine the remaining capacity for connecting additional load and sewer extensions. Below is a chart showing the Design Average Flow and the monthly average flow from 2011 through 2013. Based on the consistency of the flow, it is evident that the collection system is subject to infiltration and inflow.

Figure 5-1 | Main WWTF – Influent and Effluent Flows



The monthly average flow from 2011 through 2013 ranged from 2.90 MGD up to 7.13 MGD. The chart on the below shows the annual average flow and three low flow months for calendar years 2011 through 2013.

Table 5-1 | Main WWTF – Average and Low Flow Data

	Annual Average Daily Flow	Three Low Flow Months	Three Low Flow Months Average
2011	5.701 MGD	July, September & October	4.62 MGD
2012	4.244 MGD	September, November & December	3.12 MGD
2013	4.024 MGD	August, September & October	3.14 MGD

5.3.2 Influent and Effluent $CBOD_5$

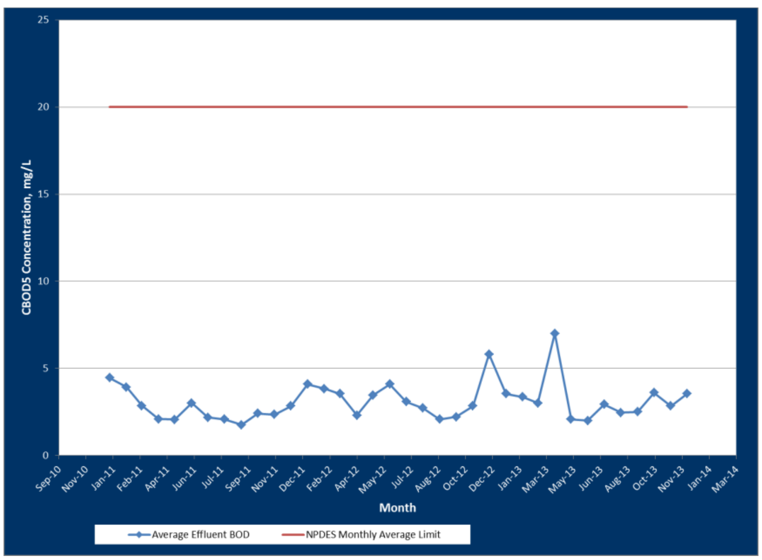
In determining the proper $CBOD_5$ design loading, we reviewed the monthly average, the average monthly maximum and the highest annual maximum for the three-year period. The average monthly maximum is the average of the highest concentration received in each month, while the highest annual maximum represents the highest concentration within the twelve-month period.

Year	***Monthly Average***	***Average Monthly Maximum***	***Highest Annual Avg.***
2011	161 mg/L	205 mg/L	370 mg/L
2012	200 mg/L	257 mg/L	500 mg/L
2013	182 mg/L	254 mg/L	450 mg/L
Average	181 mg/L	239 mg/L	440 mg/L

The facility should be designed with adequate biological reduction capacity to meet the effluent limits on a continuous basis. The influent concentrations should be evaluated based on 2011, 2012 and 2013 data. The 2002 design calculations were based around existing influent $CBOD_5$ of 183 mg/L. This is consistent with the current monthly average design condition. While this design parameter is adequate to determine basin sizing, it is recommended that the aeration system capacity be able to treat the 239 mg/L average monthly maximum. Furthermore, in 2002 the primary clarifiers included ferric chloride addition. In-plant testing at the time demonstrated that the primary clarifiers were removing 57% of $CBOD_5$. This capability has been removed, and primary clarifier $CBOD_5$ removal efficiency is closer to 32%. This combination, among other factors, significantly increases the potential oxygen demand within the biological process.

The Daily Monitoring Reports were reviewed to document the efficiency of the existing process. The average influent and effluent CBOD5 for the period were 181 mg/L and 3.08 mg/L, respectively. This reflects an efficiency of 98.3%.

Figure 5-2 | Main WWTF – $CBOD_5$ Performance



5.3.3 Total Suspended Solids Concentration

Total Suspended Solids (TSS) loadings was were analyzed by comparing the monthly average, the average monthly maximum, and the highest annual maximum for the last three years DMR's

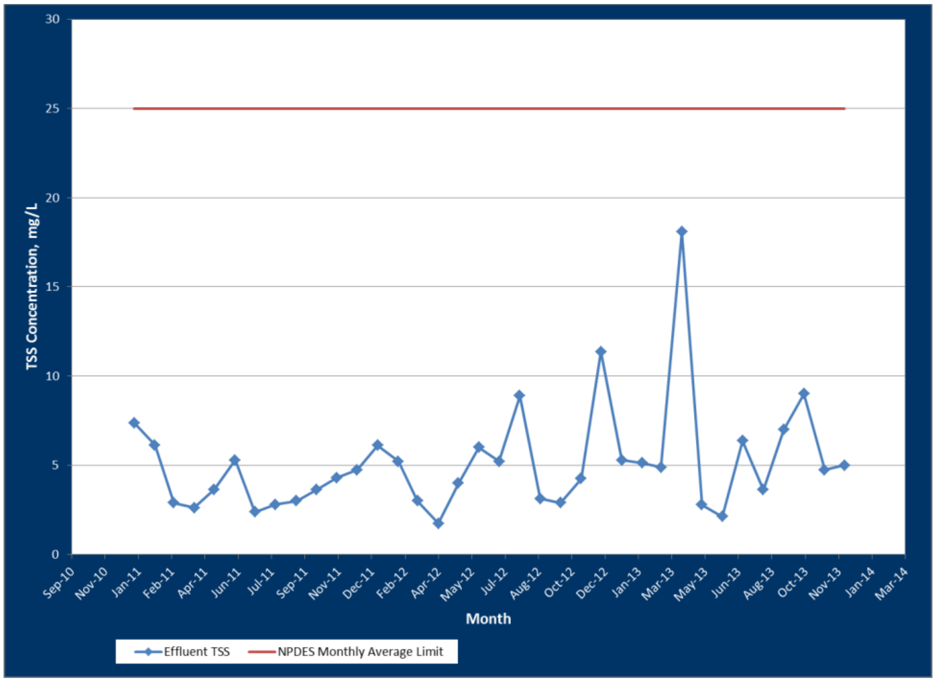
Year	*Monthly Average*	*Average Monthly Maximum*	*Highest Annual Max.*
2011	181 mg/L	315 mg/L	760 mg/L
2012	229 mg/L	323 mg/L	472 mg/L
2013	212 mg/L	267 mg/L	635 mg/L
Average	207 mg/L	302 mg/L	622 mg/L

The facility should be designed with adequate solids handling capacity to meet the bio-solids reduction needs on a continuous basis. However, solids reduction is a continuous process in excess of 24 days detention time. Therefore it is not adversely effected by increased solids loading from a single day and the monthly average loading will be utilized for design.

TSS = 9.0 MGD x 207 mg/l x 8.34 lb./gal = 15,537 lb. /day

The NPDES Permit Limit for TSS is 25 mg/L monthly average and 45 mg/L weekly average. A similar analysis of the DMR's was completed for this limit. The plant's overall performance from 2011 through 2013 was 97.5% effective with an average effluent concentration of 5.13 mg/L. The plant has been able to meet its permit limits on a continuous basis over the past three years. The graph below demonstrates the plant's performance on a monthly basis.

Figure 5-3 | Main WWTF – TSS Performance



5.3.4 Ammonia Concentration

Influent Ammonia should be considered similar to BOD5 loading to the biological process by comparing the monthly average, the average monthly maximum and the highest annual maximum for the last three years DMR's

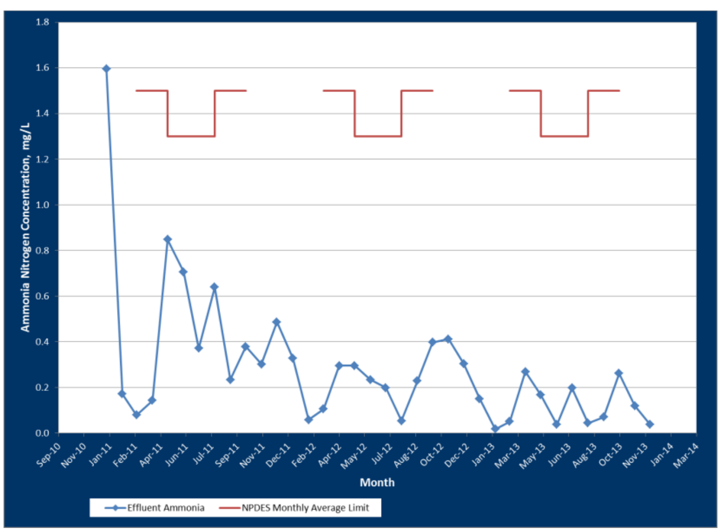
Year	***Monthly Average***	***Average Monthly Maximum***	***Highest Annual Avg.***
2011	20 mg/L	26 mg/L	34 mg/L
2012	23 mg/L	27 mg/L	35 mg/L
2013	20 mg/L	25 mg/L	33 mg/L
Average	21 mg/L	26 mg/L	34 mg/L

The facility should be designed with adequate nutrient removal capacity to meet the effluent limits needs on a continuous basis. However, designing around the highest monthly maximum for each year seems too conservative. Therefore, it is recommended that the design be based around the Average Monthly Maximum.

NH3-N = 9.0 MGD x 26 mg/L x 8.34 lb./gal. = 1,952 lb./day

The current NPDES Permit includes stringent ammonia nitrogen limits. Based on a decision from the Illinois Pollution Control Board, the permit was revised in 2015. The current NPDES permit limits provide three seasons: winter, spring/fall, and summer. The effluent ammonia concentrations were compared to the current and proposed monthly effluent limits. The plant's overall efficiency from 2011 through 2013 was 98.7% effective with an average effluent concentration of 0.29 mg/L. The plant has been able to meet its permit limits on a continuous basis over the past three years. The graph below demonstrates the plant's performance on a monthly basis. Note that there is not a monthly average limit from November through February included in the current NPDES permit.

Figure 5-4 | Main WWTF – Ammonia Nitrogen Performance



5.4 INFLUENT CHANNEL

5.4.1 Process Description

Flow is received at the head of the influent channel from three active force mains; a 16" and 24" force main from the Riverside Lift Station and a 16" force main from the East Side Lift Station. There is also an 8" force main from the Riverside Lift Station that has been abandoned in place. Outside of the influent channel, there are isolation gate valves with extended bonnets and hand-wheel operators on each of these force mains.

Flow measurement is obtained through a Parshall flume. Prior to flow measurement, flow in excess of 18.35 MGD is diverted over a weir trough and through a gravity drain to the excess flow clarifiers. This gravity drain is equipped with an isolation gate valve with an extended bonnet and hand-wheel operator. Downstream of flow measurement and upstream of the grit tanks, a 6" force main conveys flow from the Recycle Pump Station after an excess flow event to drain the excess flow clarifiers. This force main is equipped with an isolation plug valve outside of the influent channel.

5.4.2 IEPA Regulatory Requirements

Following is an excerpt from Title 35: Subtitle C: Chapter II: Part 370.550 Illinois Recommended Standards for Sewage Works.

Flow Measurement
Flow measurement facilities shall be provided so as to measure the following flows:

B) Plant influent flow, if significantly different from plant effluent flow, such as for lagoons and plants with excess flow storage or flow equalization.

5.4.3 Design Data

Number of Tributary Force Mains	3 (1 abandoned)
Design Average Flow (DAF), MGD	9.00
Peak Hourly Flow (PHF), MGD	18.35
Excess Flow Capacity, MGD	17.35
Excess Flow Fixed Weir Length, feet	40
Head over Fixed Weir @ 17.35 MGD, feet	0.146

5.4.4 Performance

The influent channel conveys process and excess flow adequately for the City.

5.4.5 Deficiencies

The gate valves on the force mains from East Side and Riverside Lift Stations do not operate properly. The actuator for the excess flow gravity drain gate valve appears to operate properly, but the valve itself should be inspected and replaced if necessary. The heat tape and insulation on the exposed 6" recycle force main outside of the influent channel has deteriorated and no longer appears to be effective.

5.4.6 Recommendations

The gate valves on the force mains from East Side and Riverside Lift Stations and the excess flow gravity drain valve should be replaced. The heat tape and insulation on the exposed 6" recycle force main outside of the influent channel should also be replaced. Finally, the City should continue to have the influent flow measurement devices checked and calibrated regularly to ensure accurate data is being collected for submission to the EPA. It is recommended that this work be funded through the City's maintenance budget. If the screening unit processes are not improved at the tributary lift stations, the possibility of a screening unit within the influent channel should be considered.

5.5 GRIT TANKS

5.5.1 IEPA Regulatory Requirements

Following is an excerpt from Title 35: Subtitle C: Chapter II: Part 370.620 Illinois Recommended Standards for Sewage Works.

Inlet
The inlet shall be located and arranged to prevent short-circuiting to the outlet and oriented to the unit flow pattern so as to provide for adequate scouring segregation of organic and grit materials prior to discharge.

Detention
A detention time of at least 3 minutes at design peak flow should be provided.

Air Supply
Air should be supplied at 5 cubic feet per minute (cfm) per foot of tank length. The rate of air supplied shall be widely variable so as to maximize unit process effectiveness.

5.5.2 Design Data

Number of Units	2
Design	Aerated
Design Average Flow (DAF), MGD	9.00
Peak Hourly Flow (PHF), MGD	18.35
Length, ft.	22
Width, ft.	20
Side Water Depth, ft.	18
Total Volume, gallons	99,858
Total Volume, cu. ft.	13,350
Detention Time at DAF, min	16.0
Detention Time at PHF, min	7.8

The equipment in the Grit Handling process consists of the following:

- ✓ Two rectangular concrete tanks 22 feet long by 20 feet wide with a side water depth of 18'. The mechanisms within the tanks include Walker Rolling Grit System which consists of a draft tube, air lift pump, and head box. The system uses low head compressed air to develop a roll pattern within the basin. Due to entrained air, the grit is less buoyant and settles quickly. The air lift pump draws the grit from the bottom of the tank and transfers it to the grit classifier. These units were replaced in-kind in 2011.

- ✓ One Walker Process screw type grit classifier, Size GW350, driven by a 1 HP, 1,800 rpm, General Electric motor operates on 240 volt, 3-phase, 60 cycle current. The grit classifier was replaced in-kind in 2001.

- ✓ Two Hoffman centrifugal blowers with a rated capacity of 375 scfm at 7.0 psig for each blower. Each blower is driven by a General Electric 30 HP, 3,600 rpm, electric motor operating on 240 volt, 3-phase, 60 cycle current. The blowers were replaced in kind in 2005 and are located in the Blower Building.

5.5.3 Performance

The Design Peak Flow (DPF) is defined by the IEPA Section 370.211 as the instantaneous maximum flow rate to be received, which is 18.35 MGD. The Hydraulic Retention Time (HRT) at the DPF is 7.84 minutes which meets the IEPA requirements and manufacturer recommendations.

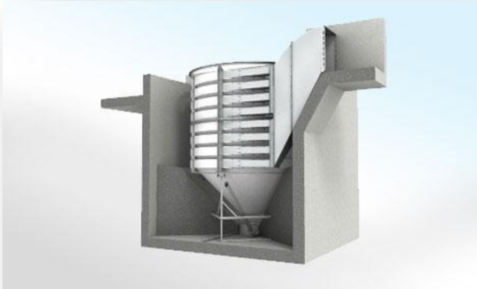
5.5.4 Deficiencies

The HVAC system for the grit room does not operate effectively, and is beyond its service life. The effluent weir baffles within the grit tanks were removed during one of the headworks rehabilitation projects, which may allow lighter grit particles to bypass the grit removal system. Also, the grit classifier is currently hydraulically washing out the grit and recycling it back into the southern grit tank.

The City has noticed a decline in the performance of these units over the last couple years. The City is currently removing 2-4 yards per week. While the quantity of grit received by a plant varies greatly, the City staff believes that the system should be more efficient.

The City has expressed interest in new technologies for grit removal. Several different options are discussed below:

- Plate Settler (i.e. Eutek HeadCell)
    - Probably most efficient design in terms of grit removal
    - Existing system can be retrofitted to incorporate this technology
    - Significant concerns regarding ragging exist and may negate the advantages of this type of system due to the potential for increased labor, maintenance.



- Vortex Grit Separation (i.e. Eutek Grit King)
    - Good resistance to negative effects of ragging
    - Does not require use of air, therefore more energy efficient with pump system
    - Not as effective as the plate settler
    - Settles 106 microns and larger

- Grit Washer (i.e. Eutek TeaCup)
    - No moving parts, resistance to negative effects of ragging
    - Can remove 95% of all grit 75 microns and larger
    - Can treat up to 8 MGD each
    - May present issue with head conditions on existing air-lift pump system, as well as existing grit room height, due to unit height and need to discharge into grit classifier, snail or decanter

- Grit Decanter (i.e. Eutek Decanter)
  - Grit dewatering through wedge wire screen
  - No moving parts, upgrade from current storage dumpster
  - Requires a grit washer/classifier upstream to remove organic material

5.5.5 Recommendations

A more detailed study of the grit system needs to be performed if and when the City elects to reinvest in grit removal. The study should evaluate the benefits of each additional or modified component of the system, and should consider special limitations, head conditions and energy/labor cost comparisons. If possible, it is recommended that the existing scum pump wet well be considered for a partial repurposing for the grit removal system. It is also recommended that the existing grit classifier and dumpster be replaced with a grit washer and grit decanter. At a minimum, the HVAC system and the baffles on the grit unit effluent weirs should be reinstalled.

5.6 PRIMARY CLARIFIERS

5.6.1 IEPA Regulatory Requirements

Following is an excerpt from Title 35: Subtitle C: Chapter II: Part 370.710 Illinois Recommended Standards for Sewage Works.

Surface Settling Rates (Overflow Rates)
The hydraulic design of settling tanks shall be based on the anticipated peak hourly flow. Some indication of BOD removals may be obtained by reference to Appendix E, Figure No. 2. The figure should not be used to design units which receive wastewaters which have BOD and total suspended solids concentrations which are substantially different from normal domestic sewage. The operating characteristics of such units should be established by appropriate field and laboratory tests. If activated sludge is wasted to the primary settling unit, then the design surface settling rate shall not exceed 1,000 gallons per day per square foot based on design peak hourly flow, including all flows to the unit. Refer to subsection (b)(3) and Section 370.820.

Weir loadings
Weir loadings shall not exceed 20,000 gallons per day per lineal foot based on design peak hourly flows for plants having design average flows of 1.0 mgd or less. Overflow rates shall not exceed 30,000 gallons per day per lineal foot based on design peak hourly flow for plants having design average flow of greater than 1.0 mgd. Higher weir overflow rates may be allowed for bypass settling tanks. If pumping is required, weir loadings should be related to pump delivery rates to avoid short circuiting. Refer to Section 370.410(c)(8).

5.6.2 Design Data

Number	4
Length, ft.	100
Width, ft.	20
Surface Area, sf/clarifier	2,000
Total Surface Area, sf	8,000
Overflow Rate at DAF, gpd/sf	1,125
Weir Loading Rate, gpd/ft	15,000
Primary Influent BOD, lbs./day	17,939
Primary Influent TSS, lbs./day	19,816
Primary Influent NH_3-N, lbs./day	1,952
Removal Efficiency - BOD, %	32
Removal Efficiency - SS, %	56
BOD Removed, lbs./day	5,741
Suspended Solids Removed, lbs./day	11,097
Primary Effluent BOD, lbs./day	12,198
Primary Effluent TSS, lbs./day	8,719
Sludge Volume at 5%, gpd	26,612
VSS Solids to Digestion (85%), lbs.	

Two primary settling tanks are provided at the plant. The settling tanks and associated equipment consist of:

- ✓ Two concrete tanks, each tank is 100’ long by 40’ wide with an average side water depth (SWD) of 8’3". A 5' deep sludge hopper is provided at one end for storage, thickening and withdrawal of sludge.
- ✓ Each tank is served by two chain and scraper mechanisms, which are supported by a concrete wall that bisects each tank lengthwise. This wall is only used to support the chain and scraper mechanisms, and is open at the west end of the tanks to hydraulically connect the two tank halves. These units were installed in 2001.
- ✓ Two Type RP Helithickener Cross Collectors as manufactured by Walker Process. Equipment is provided for sludge conveyance to the pump draw-off. One collector is located in the sludge hopper of each tank half, and is equipped with a 1 HP, 1750 RPM electric motor operating on 240 volt, 60-cycle, 3 phase current. The worm gear reducer has a ratio of 64:1 providing a rotational speed of 5 RPM for the collectors. These units were replaced in 2001, rehabilitated in 2011, and the worm gear reducer was modified in 2014 to cut the rotational speed for the collectors in half to 2.5 RPM.
- ✓ Each tank is equipped with inlet weirs, inlet baffles and outlet weir troughs. An effluent collection channel and drop box is located at the end of the settling tanks.

5.6.3 Performance

The primary clarifiers are an important part of the complete treatment process. The 2001 Facility Plan design criteria was based on an evaluation of actual performance data. A regression curve analysis was performed, and predicted that the expected TSS and BOD removal at design flow should be 63% and 36%, respectively. At that time, the City of St. Charles was adding ferric chloride to aid in the settling of solids as well as address odor control issues and control of struvite formation. This capability has since been removed.

Based on recent DMR data, the existing clarifiers average TSS removal has been 56% and the average BOD_5 removal has been 38%. The facility is operating at roughly 50% capacity. The current performance is comparable to traditional removal efficiencies. These traditional values should be used for future design loading criteria.

The surface overflow rate at the DAF (9 MGD), is 1,125 gpd/sf. The surface overflow rate at the anticipated peak hourly flow (18.35 MGD) is 2,294 gpd/sf. A limit on surface overflow rate is not stipulated in the Section 370 unless waste activated sludge (WAS) is returned to the primary tanks for thickening. Based on the WEF Manual of Practice Number 8, the primary clarifier suspended solids removal at roughly 1,100 gpd/sf can be estimated to be 56%. Using Figure 2, in Appendix E of Title 35 Section 370, the BOD_5 removal would be 32% at DAF. This BOD_5 removal through the primaries affects the design of the downstream unit processes.

5.6.4 Deficiencies

The primary clarifiers were rehabilitated in 2000, and again in 2011. The equipment and tankage has an expected service life of twenty years. Based on the design calculation, treatment capacity is at an acceptable level.

However, the City has been experiencing problems with the Helithickener Cross Collectors. The Helithickener Cross Collectors are an auger located within the sludge hopper. Their function is to transfer sludge in the hopper to the sludge draw-off pipe.

During the 2011 Headworks Rehabilitation, the mechanisms were modified to utilize a chain and sprocket drive system, which included replacement of the drives and modifications to one section of the torque tube in each clarifier. Due to mechanical failures after this rehabilitation, the gear reducer was replaced to provide a slower, steadier rotation and conveyance of primary sludge. This should reduce the wear on the equipment, as well as equipment shutdowns due to tripped limit switches.

If the new gear reducer does not have the desired effect on the Cross Collectors, replacing the bearing hanger between each pair of collectors should be considered. The bearing hanger may be misaligned, which would cause over-torqueing on the drive system. A concrete stub wall, built down from the existing web wall above the trough, would allow for a fixed connection point for the collectors. It would leave a gap of about 6" between the bottom of the new wall and the bottom of the trough, which would maintain the hydraulic connection between each pair of tanks. This option would require that each Cross Collector be provided with its own chain and sprocket drive system. Finally, the effluent weirs, the aluminum decking over these weirs and the drain valves at the east end of the clarifiers do not appear structurally sound and should be repaired or replaced.

5.6.5 Recommendations

The primary clarifiers were rehabilitated in 2011, but several items were not included in that rehabilitation and are in need of replacement. A cost estimate for this work is included below.

Table 5-2 | Primary Clarifier Rehabilitation – Probable Costs

GENERAL CONDITIONS	$78,200
PRIMARY CLARIFIER REHABILITATION	$353,200
ELECTRICAL & CONTROLS	$14,000
CONSTRUCTION SUB-TOTAL	**$445,400**
CONTIGENCY @ 10%	**$44,540**
ESTIMATED CONSTRUCTION COST	**$489,940**
ENGINEERING (14%)	**$68,592**
TOTAL PROJECT COST	**$558,532**

5.7 HEADWORKS ODOR CONTROL

5.7.1 General Description

The City has continuously attempted to provide odor control at the wastewater treatment facility. Some of the major areas of concern include odors generated in the influent flume, the aerated grit tanks and effluent launders of the primary clarifiers.

In the mid 90's, the City researched the several odor control systems for these areas. The selected alternative was ozone oxidation. In 1997, the City covered the areas of concern, constructed ductwork for capture and conveyance of the exhaust and installed a simple pre-manufactured ozone system. While the system was effective, the system lasted approximately 18 months before mechanical failure.

In 2005, the City elected to replace the failed system with a custom designed system, which was installed as part of the larger Nitrification Improvements project. The new system included an air compressor, expansion tank, purification filter, oxygen generator, condenser, receiver and ozone unit. In accordance with the manufacturer's recommendations, a separate room was constructed with a fresh air supply to improve the atmosphere and longevity of the ozone system as part of the 2005 improvements.

The manufacturer of the custom system provided start-up and training on operation as well as maintenance of this system, and offered an annual maintenance contract to ensure proper operation. The City elected not to execute the maintenance contract. Unfortunately, the door between the chemical feed room and the ozone room was regularly left open, allowing fumes from the chemical feed room to enter the ozone room which caused severe corrosion. Once again, this unit worked effectively for approximately 24 months before being shut down.

In 2011, this building was rehabilitated, the ferric chloride tanks were removed, and new primary sludge pumping systems were installed in this area. The upgrades significantly improved the atmosphere within the building, and a new air compressor, oxygen unit, ozone unit and control system were installed as part of the overall improvements. Once again, the system manufacturer provided a quote for an annual maintenance contract which the City elected not to execute. The system was shut down after approximately 12 months.

5.7.2 Design Data

Number	1
Design Run Time, hrs/day	24/7
Capacity, ppd	7

5.7.3 Performance

The system installed in 2011 performed well, but required maintenance that was not provided. Therefore, the system was shut down rather than allowing it to run to failure.

The City does not currently have a maintenance contract for the equipment, and is not confident with working on the equipment with in-house staff. Furthermore, the City has not received odor complaints related to this part of the treatment process.

5.7.4 Deficiencies

Since the unit has not been in operation for a couple years, the manufacturer would likely need to refurbish the ozone unit if it was to be put back into operation.

5.7.5 Recommendations

After several attempts to operate this equipment over the years with in-house staff, the same course of action cannot be recommended. The system should be decommissioned and removed.

5.8 PRIMARY SLUDGE SCUM PUMPING

5.8.1 IEPA Regulatory Requirements

Following is an excerpt from Title 35: Subtitle C: Chapter II: Part 370.720 Illinois Recommended Standards for Sewage Works.

Sludge and Scum Removal (Sludge Removal Piping)
Each hopper shall have an individually valved sludge withdrawal line at least 6 inches in diameter. The static head available for withdrawal of sludge shall be 30 inches or greater, as necessary to maintain a 3 feet per second velocity in the withdrawal pipe. Clearance between the end of the withdrawal line and the hopper walls shall be sufficient to prevent "bridging" of the sludge. Adequate provisions shall be made for rodding or back-flushing individual pipe runs. Piping shall also be provided to return waste sludge from secondary and tertiary processes to primary clarifiers where they are used. Refer to Section 370.820.

Sludge and Scum Removal (Sludge Removal Control)
Sludge wells equipped with telescoping valves or swing pipes are recommended for primary sludge and fixed film sludges where periodic withdrawal is proposed. Air lift type of sludge removal will not be approved for removal of primary sludges.

5.8.2 Design Data

Number	3
Run Time, hrs./day	24
Capacity, gpm	35

- ✓ Primary sludge pumping consists of three rotary lobe pumps which were installed during the 2011 Headworks Rehabilitation. These pumps are provided with upstream in-line grinders, temperature and pressure switches for run-dry protection, and variable frequency drives.
- ✓ Primary sludge flow to the digesters is measured with a magnetic flow meter within the primary sludge pumping room.
- ✓ Two hand-wheel operated skimmers are located on the discharge end of each primary clarifier. These units were installed in 2001.

5.8.3 Performance

The existing rotary lobe pumps draw sludge directly from the primary clarifiers and transfer it to the anaerobic digesters for stabilization. The pumping system has adequate capacity to transfer primary sludge.

The hand-wheel operated skimmers convey scum from the east end of the primary clarifiers by gravity to a wet well located in the grit classifier room. From here, scum is pumped directly to the digesters within the primary sludge pipe with a submersible scum pump. This pump was installed in 2005, and has performed well over the past 9 years.

5.8.4 Deficiencies

The HVAC system for the primary sludge pump room is beyond its service life, and should be replaced. The existing primary sludge pumps went through several iterations of operating speeds and rotor materials before being able to operate efficiently and continuously. There were several occasions where the lobes were wearing at the edges and losing suction. An operational change was made so that the pumps conveyed sludge at a greater rate for a reduced duration. This, along with the addition of grit boxes on the pumps themselves, has provided adequate capacity for the primary sludge pumping system.

5.8.5 Recommendations

The HVAC system should be replaced, and it is recommended that this be funded through the City's maintenance budget.

5.9 BIOLOGICAL PROCESS

5.9.1 Process Description

In 2005, the City of St. Charles Main WWTP secondary treatment process was upgraded for single stage nitrification. The process control variables used are sludge age and feed to mass ratio. The primary effluent is blended with RAS in the inlet box to form MLSS. The design includes two pre-mix basins (Basins 301 and 302) that are 19.25 feet wide by 40 feet long by 16 feet deep. The MLSS then enters a distribution channel to the first bank of four aeration basins (Basins 303 and 306). The aeration basins are 90 feet long by 30’-6” wide by 16 feet deep. At the effluent end of the aeration basins, the MLSS flow over a fixed weir to the collection channel. The collection channel also serves as the distribution channel to the second bank of four aeration basins (Basins 307 and 310), which is identical in size to the first bank. The MLSS from the second bank of aeration basins flows over a second fixed weir to a collection channel. From the new aeration basins the MLSS flows to the final clarifiers.

The City still has the ability to send flow from the second bank of aeration basins to the original aeration basins (Basins 401 through 404) if additional detention time is needed for the biological process. However, current plant loading does not require that these basins remain in operation. The diffuser membranes were replaced in the fall of 2014. The average service life for a diffuser membrane is approximately seven years, so the City should not need to replace these membranes until 2021.

5.9.2 Design Data

Number of Tanks	14
Side Water Depth, ft.	15 – 16
Aeration Basin 301 & 302, total cf	24,640
Aeration Basin 303 Thru 310, total cf	351,360
Aeration Basin 401 & 402, total cf	67,200
Aeration Basin 403 & 404, total cf	35,280
Existing Volume, cf	478,480
Existing Volume, gal.	3,579,030
Detention Time at 9.00 MGD, hrs.	9.5
Organic Loading, lbs/day BOD	12,198
Organic Loading, mg/L BOD	163
Organic Loading Rate, lbs/day BOD/1,000 cf	25.49
MLSS, mg/L	3,500
Solids Inventory, lbs.	104,471
RAS Return Rate, MGD	7.51
WAS, lbs/day	7,929
WAS Volume at 1% TS, gpd	95,072
Air Required Reduction , scfm	7,230
Air Provided, scfm	4,800
Sludge Age, days	13.18
Feed to Mass Ratio	0.117

5.9.3 Performance

The existing system is performing very well. The City has not had any ammonia or BOD_5 violation related to performance of the biological process since the system was placed into operation in 2005.

5.9.4 Deficiencies

The upper aeration basin blowers have been operating for almost a decade, and should be replaced. The City has expressed interest in positive displacement blowers, turbo-blowers and screw compressors. The northeast isolation valve on the 16" RAS pipe also needs to be replaced. The bronze seating of the mud valves in all of the aeration basins has deteriorated over time and prevents the valves from operating properly. These valves should be replaced. The diverter gates in front of the lower aeration basins are overtaken during high-flow periods, but this will not be an issue if the lower basins are reutilized for biological and/or chemical phosphorus removal.

The 2002 design calculations were based around existing influent $CBOD_5$ of 183 mg/L. This is consistent with the current monthly average design condition. While this design parameter is adequate to determine basin sizing, it is recommended that the aeration system capacity be able to treat the 239 mg/L average monthly maximum. Furthermore, in 2002 the primary clarifiers included ferric chloride addition. In-plant testing at the time demonstrated that the primary clarifiers were removing 57% of $CBOD_5$. This capability has been removed, and primary clarifier removal efficiency is closer to 32%.

This combination, among other factors, significantly increases the potential oxygen demand within the biological process. At design flow of 9 MGD, the aeration basins provide 9.6 hours detention time and an MCRT of 12 days. While the plant is currently operating at 4.7 MGD, the plant may struggle to meet NPDES permit limits if it approaches the 9 MGD design flow. The City has received a new NPDES permit from the IEPA that includes nutrient limits for total phosphorus and monitoring requirements for total nitrogen in the plant effluent. The City cannot meet these new limits with the current biological process.

5.9.5 Recommendations

The City is also enacting a preventive maintenance program to operate and grease the slide gates to ensure proper operation. The rehabilitation of the existing biological process will likely coincide with the phosphorus removal alternative selected by the City. Therefore, the replacement of the upper aeration basin blowers, 16" RAS isolation valve and any slide gates determined to be inoperable should take place at that time. The mud valves typically have a longer service life, and should be replaced with valves that utilize a stainless steel valve seat to extend its service life during the rehabilitation of the biological process.

The samplers and probes should be replaced, and consideration should be given to online phosphorus and ammonia monitoring to enhance plant performance and assist in maintaining nutrient limits. Several options for additional tankage and repurposing of existing tankage for phosphorus removal are discussed in depth in Section 6.

5.10 FINAL CLARIFIERS

5.10.1 IEPA Regulatory Requirements

Following is an excerpt from Title 35: Subtitle C: Chapter II: Part 370.710 Illinois Recommended Standards for Sewage Works.

Surface Settling Rates (Overflow Rates)
The hydraulic loadings shall not exceed 1000 gallons per day per square foot based on design peak hourly flow, and 800 gallons per day per square foot based on peak hourly flow for separate activated sludge nitrification stage. Refer to Section 370.1210(c)(4).

Solids Loading Rate
The solids loading shall not exceed 50 pounds solids per day per square foot at the design peak hourly rate.

Weir Loading
Weir loadings shall not exceed 20,000 gallons per day per lineal foot based on design peak hourly flows for plants having design average flows of 1.0 mgd or less. Overflow rates shall not exceed 30,000 gallons per day per lineal foot based on design peak hourly flow for plants having design average flow of greater than 1.0 mgd. Higher weir overflow rates may be allowed for bypass settling tanks. If pumping is required, weir loadings should be related to pump delivery rates to avoid short circuiting. Refer to Section 370.410(c)(8).

5.10.2 Design Data

Number	2
Design	Hydraulic Differential
Average Flow, MGD	9.00
Peak Hourly Flow, MGD	18.35
Diameter, ft.	120
Sidewater Depth, ft.	12.75
Surface Area – Each, sf	11,310
Surface Area – Total, sf	22,620
Weir Length – Each, lin. ft.	343
Weir Length – Total, lin. ft.	686
Surface Loading Rate at PHF, gpd/sf	811
Solids Loading Rate at PHF, lbs./day/sf	23.7
Weir Loading Rate, gpd/lf	26,749

The two 120 foot diameter clarifiers were constructed in 1987. The existing design includes peripheral feed and take-off. The existing mechanism is an Envirex To-Bro and operates on a hydraulic differential principal. The existing To-Bro header is designed to remove sludge from the entire clarifier floor evenly, instead of raking the bio-solids to a center hopper. The design capacity of the units is within the Illinois EPA guidelines, and should continue to serve the City well.

5.10.3 Performance

The removal of TSS has been very effective, with a range of 95% to 99% removal.

5.10.4 Deficiencies

Flow splitting between the two clarifiers is controlled by inverted slide gates at the flow diversion structure to the north. This method of flow splitting is difficult to control, and one clarifier typically sees the majority of flow from the aeration basins. Consideration should be given for removing this structure and replacing it with a Tee and two gate valves.

When algae build up on the effluent weirs, they must be washed down with a hose. The non-potable water piping around the clarifiers that was installed as a part of the 2002 Nitrification Improvements was unintentionally left filled during the winter months, which froze and caused the pipes to burst. Therefore, City staff must haul a 1.5” hose to around the final clarifiers and utilize the non-potable water yard hydrant for wash down and cleaning of the equipment. Options to address this build-up include lining of the effluent weirs with fiberglass, installing covers over the weirs, and installing a method to flood the outer trough with large quantities of non-potable water at one time.

The central column and scum skimmer in each unit was recently replaced and are performing well. However, the To-Bro headers are from the original installation and were stripped and painted in 1995. The walls of these headers may be rusted through and drawing unevenly from the bottom of the clarifiers. The existing units have sufficient capacity to serve the future design loadings, but may need to be replaced in kind.

City staff requested that the effluent weirs be checked for level. This was completed and confirmed that they were properly installed. However, it was noted that the weirs were showing significant wear and that distribution of flow through the weirs was no longer consistent particularly at low flows. The weirs are from original construction, and are therefore 27 years old.

5.10.5 Recommendations

The 120 foot diameter final clarifiers were constructed in 1987 and have been rehabilitated on a routine basis. The service life of the clarifiers should be 25 to 30 years. It is recommended that the clarifier mechanisms, To-Bro headers and weirs be replaced within the next five years. It is also recommended that the flow diversion structure with inverted weirs be replaced. Finally, the effluent weir should be protected from algae buildup by the installation of covers around the perimeter of the clarifiers. It is recommended that the City Budget $1.5 Million for this work.

5.11 UV DISINFECTION SYSTEM

5.11.1 Process Description

During the Nitrification Improvements, the plant upgrades included installation of an Ultra Violet Disinfection system. The system was designed to handle the peak hourly flow through the treatment facility. In addition, a second channel was constructed for installation of a parallel system in the future if the facility was upgraded to utilize UV disinfection on excess flow. Finally, a non-potable water system was installed just downstream of the UV disinfection channels to provide wash water from the treated effluent.

5.11.2 Design Data

Peak Design Flow, MGD	20
UV Transmission, % (Field measured transmissivity = 80%)	65
TSS, mg/L	45
Disinfection Limit, fecal count	400
Design Intensity, mW	40,125
Number of Channels	1
Number of Reactors per channel	1
Number of Banks/ Reactor	2
Number of Modules per Bank	4
Total Number of UV Lamps	72
Type of level control	Fixed Weir
Automatic Mechanical Cleaning	Yes

5.11.3 Performance

The existing system is performing very well. The City has not had any violation related to the performance of the system since it was placed into operation in 2005.

5.11.4 Deficiencies

The UV System provides water to the non-potable water (NPW) system, which is used for several purposes around the wastewater treatment plant site. The non-potable water system includes three vertical turbine pumps and a filtration system. The UV system tends to generate a significant amount of algae, due to the combination of high intensity light and nutrients (phosphorus and nitrogen) being available in the effluent. The algae enters the NPW system and plugs pumps and the filter system. The exposure of this system to weather year-round has taken its toll on the system components. The controllers for this system, as well as hydraulic lines and pumps, should be replaced.

5.11.5 Recommendations

There are several alternative solutions to the algae issue including removal of the nutrients, installation of a different UV system or modification of the NPW system. The plant will need to be upgraded to remove total nitrogen and phosphorus, both of which are discussed at length in Section 6 of this report. The non-potable water system for the plant is nearing the limits of its capacity, and must be upgraded. The UV disinfection technology has advanced over the last ten years, and it is recommended that the existing Trojan 4000 system be replaced simultaneously with the installation of the newer Trojan Sigma system in the open channel. Finally, it is recommended that a structure be built around the equipment to extend its service life. A cost estimate for this work is provided below.

Table 5-3 | Ultraviolet Rehabilitation – Probable Costs

GENERAL CONDITIONS	$101,000
SITE WORK	$26,200
ULTRAVIOLET REHABILITATION	$1,756,000
CONSTRUCTION SUBTOTAL	$1,883,200
CONTINGENCY @ 20%	$376,640
CONSTRUCTION TOTAL	$2,259,840
PROJECT ENGINEERING (14%)	$316,378
TOTAL PROJECT COST	$2,576,218

5.12 ANAEROBIC DIGESTION

5.12.1 IEPA Regulatory Requirements

Following is an excerpt from Title 35: Subtitle C: Chapter II: Part 370.830 Illinois Recommended Standards for Sewage Works.

Tank Capacity

1) Rational Design

The total digestion tank capacity shall be determined by rational calculations based upon such factors as volume of sludge added, its percent solids, and character, the temperature to be maintained in the digesters, the degree or extent of mixing to be obtained, the degree of volatile solids reduction required, method of sludge disposal, and the size of the installation with appropriate allowances for gas, scum, supernatant and digested sludge storage. Secondary digesters of two-stage series digestion systems that are used for digested sludge storage and concentration shall not be credited in the calculations for volumes required for sludge digestion. Calculations should be submitted to justify the basis of design.

2) Empirical Design

When such calculations are not submitted to justify the design based on the above factors, the minimum combined digestion tank capacity outlined below will be required. Such requirements assume that the raw sludge is derived from ordinary domestic wastewater, a digestion temperature is to be maintained in the range of 85 to 95 F (29 to 35 C), 40 to 50 percent volatile matter in the digested sludge, and that the digested sludge will be removed frequently from the process. (See also subsection (a)(1) above and Section 370.860(a)(1).)

A) Completely Mixed Systems

For digestion systems providing for intimate and effective mixing of the digester contents, the system may be loaded up to 80 pounds of volatile solids per 1000 cubic feet of volume per day in the active digestion units.

B) Moderately Mixed Systems

For digestion systems where mixing is accomplished only by circulating sludge through an external heat exchanger, the system may be loaded up to 40 pounds of volatile solids per 1000 cubic feet of volume per day in the active digestion units. This loading may be modified upward or downward depending upon the degree of mixing provided.

5.12.2 Design Data

Number	2
Design	Egg-Shaped
Volume, cf each	64,171
Total Volume, cf	128,342
Total Volume, gal	960,000
TWAS VSS, lbs/day	4,497
Primary VSS, lbs/day	10,209
Volatile Solids Loading Rate, lbs VSS/day	14,706
Loading Rate, lbs VSS/1000 cf	114
Loading Rate, gpd	42,506
Detention Time, days	22.6

5.12.3 Performance

The anaerobic digesters were constructed as part of the 1991 Sludge Handling Improvements. The egg-shaped digesters were the second system of its kind constructed in the United States. When in operation, the digesters have continuously met the volatile solids reduction requirements for Class B land application. The major components have adequate detention time and capacity to effectively treat the bio-solids produced by the 9.0 MGD treatment facility.

5.12.4 Deficiencies

The service conditions created within the anaerobic digestion process commonly are very corrosive which negatively impacts valves, piping equipment and controls. Over the last few years, the major components of the anaerobic digesters have begun to break down resulting in expensive, emergency repairs.

In the summer of 2010, the glass access cover to the south digester was shattered due to excess pressure within the digester and the failure of the pressure relief components for this digester. The lid has been replaced with a temporary cover, but the City is still without proper gas pressure vacuum/relief device on this unit. In the winter of 2013/2014, a hole in the steel shell was discovered in the north digester, which was the only operational digester at that time. The hole was located in the cradle (basement, or underground portion) of the digester and this required removal of the digester contents, cleaning of the tank and repair of the steel tank walls. During these repairs, the City had to pay for the temporary dewatering of primary and waste activated sludge as the facility had no digestion capabilities. Also during this time, the rehabilitation of certain critical piping systems was required to allow for the inoperable digester to return to service. Finally, the City had to pay for seed sludge from a neighboring facility in order to start-up the digesters.

The City is currently utilizing both anaerobic digesters. However, the existing mechanicals systems have been in operation for over twenty years. While some the components have been repaired or replaced, the system overall is becoming less mechanically reliable.

The digested sludge storage tank was constructed in 1951 as a primary clarifier. The tank was repurposed in 1966 for chlorine disinfection, and again in 1973 for sludge holding. The tank currently provides approximately 2 days of storage, which is only enough for a typical weekend without the need to dewater sludge; weekends of three days or more require operators to work during the weekend or modify operations before and after the weekend to compensate. The mechanical mixer in the tank is in need of replacement. Also, the ground around the tank perimeter slopes sharply up to the curb to the east and south which hinders tank access. The tank is without any level control or monitoring, and City staff must manually check the depth of sludge in the tank several times a day. The digesters produce gas that may be processed and used by the boilers that heat the sludge to maintain proper temperatures within the anaerobic digestion system. However, the City does not currently have enough digester gas storage to service both of the boilers and must supplement their boilers with natural gas.

5.12.5 Recommendations

The existing boilers, hydronic piping and heat exchangers have reached the end of their service life and should be replaced. The existing controls are now defunct and have not been replaced because the valves which they control are unreliable. It is recommended that all piping and valves be replaced, and possibly redesigned in its entirety. For example, the Primary Sludge and TWAS combine at a 6”x6” Tee southeast of the digester operations building. It is suspected that this piping configuration creates a slug of inorganic material to coagulate near this location and create undue head pressure on the TWAS and Primary Sludge pumps. Consideration will be given to installing separate feed lines for the two sludge types into the digester operations building, where there will be a clean-out for each line prior to heating the sludge and sending it to the digesters.

The exterior coating of the digesters, all grating, the grinder on the influent sludge line, and all gas control and relief equipment should also be replaced. New electrical wiring to the control panel is necessary along with the replacement of the Motor Control Center. Finally, the digested sludge storage tank should be replaced and equipped with gas storage and a new mixing system. At this time, it is recommended that this work be considered as a single project and that the City budget $8 Million dollars for the project. The City has requested that the project be broken up into phases to minimize the financial impact. A breakdown is provided below.

Table 5-4 | Anaerobic Digestion Complex Rehabilitation – Probable Costs

	Phase I	Phase IIA	Phase IIB
GENERAL CONDITIONS	$137,000	$358,000	$335,000
SITEWORK	$5,000	$65,000	$154,500
ANAEROBIC DIGESTION COMPLEX REHABILITATION	$0	$2,155,135	$2,226,135
DIGESTED SLUDGE STORAGE TANK	$763,900	$186,500	$0
CONSTRUCTION SUB-TOTAL	**$905,900**	**$2,764,635**	**$2,715,635**
CONTIGENCY @ 10%	**$90,590**	**$276,464**	**$271,564**
ESTIMATED CONSTRUCTION COST	**$996,490**	**$3,041,099**	**$2,987,199**
ENGINEERING (14%)	**$139,509**	**$425,754**	**$418,208**
PROJECT TOTAL	**$1,135,999**	**$3,466,852**	**$3,405,406**

5.13 SLUDGE HANDLING BUILDING

5.13.1 Process Description

This building was constructed in two phases during the 2012 Main and Sludge Handling Building Improvements. The first phase included electrical/ control, sludge thickening and dewatering facilities. The sludge thickening facilities include WAS holding, sludge feed pumps, polymer unit, gravity belt thickener, TWAS holding and TWAS pumping systems. The sludge dewatering facilities include digested sludge feed pumps, polymer units, two centrifuges and a conveyor in a loading dock. The second phase included an operations building that contains an office, break room, locker room, inventory and maintenance garage.

For the sludge thickening process, waste activated sludge (WAS) is drawn from either the existing WAS holding tanks or the WAS holding tank within the new Sludge Handling Building by progressive-cavity pumps, and conveyed to the gravity belt thickener (GBT). The process utilizes the polymer feed system to assist in thickening the sludge. Thickened sludge (TWAS) from the GBT is received by the TWAS holding tank within the new Sludge Handling Building, then conveyed to the egg-shaped anaerobic digesters by another set of progressive-cavity pumps. This process is similar to the sludge thickening operation utilized by the City prior to this project, However, it is much easier and cleaner to operate, maintain and control thanks to the functional and practical design of the unit processes and the building itself.

The sludge dewatering process utilizes digested sludge pumping systems, two centrifuges and a conveyor in a loading dock. Sludge is drawn from the digested sludge storage tank and sent to the centrifuges via progressive cavity pumps. The centrifuges dewater the sludge from about 2.5% solids to approximately 22%, which greatly reduces the volume of the sludge for disposal. Dewatered sludge is sent to trucks by a shaftless screw conveyor through one of five different locations in the new truck dock, and then hauled away for land application. Again, this process is similar to the sludge dewatering process previously utilized by the City, but is much easier and cleaner to operate, maintain and control.

5.13.2 Design Data

<u>Sludge Thickening</u>	
<u>Gravity Belt Thickeners</u>	
Number of Gravity Belt Thickeners	1
Belt Width, meters	2
Solids Loading, lbs. DWS/day	5,714
Solids Loading, gallons/day	91,351
Maximum Loading Rate, lbs. DS/hr.	2,000
Operation, hrs./week	20
Thickened Sludge Volume at 5% TS, gpd	13,703
<u>WAS Storage Tank</u>	
Number	1
Volume, gal.	83,711
Storage, days	0.9
<u>TWAS Storage Tank</u>	
Number	1
Volume, gal.	73,462
Storage, days	5.4

Sludge Dewatering

Centrifuges

Number of units	2
Hydraulic Loading, gpm	150
Solids Loading, lbs. TS/hr.	1,875
Operation, hrs./centrifuge/week	16.5

5.13.3 Deficiencies

The unit processes contained in this building are all operating as designed with the exception of the thickened WAS pumping and mixing systems. The pumps convey TWAS to the anaerobic digesters by connecting to the original 6” TWAS force main just southeast of the digested sludge storage tank. This existing 6” force main created operational difficulties with the previous pumping system.

The Primary Sludge and TWAS combine at a 6”x6” Tee southeast of the digester operations building. It is suspected that this piping configuration creates a slug of inorganic material to coagulate near this location and create undue head pressure on the TWAS and Primary Sludge pumps.

City staff has reported that the TWAS mixing system, which utilizes the TWAS pumps to recycle the tank contents through two nozzles above the high water level in the tank, is not effective. As a result, the TWAS settles out into three layers: water on the bottom, saturated TWAS in the middle, and a mat of dried TWAS on top. This dried mat of sludge is not broken up by flow from the mixing system or from the GBT discharge, which compounds the issue.

5.13.4 Recommendations

As discussed in 5.12, it is recommended that the Primary/TWAS blending will take place within the digester operations building, where there will be a clean-out for each line to assist in preventative maintenance.

As a minimum, an additional valve should be installed between the existing TWAS mixing system riser and TWP-1102 so that TWAS may be mixed and sent to the digester simultaneously. Consideration for implementation of a different TWAS mixing system should also be given, and options for this are included below:

- Air pulse pump(s) within the TWAS tank
- Microbiology addition to TWAS tank

If the tank is utilized for TWAS storage, consideration should be given for adding microbiology to the tank to prevent the sludge mat from forming. This may be implemented from within the Sludge Thickening Room with a small mixing tank and feed pump. If the City wishes to repurpose TWAS tank for side stream filtrate treatment, the GBT discharge may be piped directly to a TWAS pump, which may be relocated to the Sludge Thickening Room.

5.14 CONSOLIDATED DESIGN CALCULATIONS OF EXISTING FACILITY

POPULATION EQUIVALENT

Existing Population Equivalent, PE	49,764
Build-out of Service Area, PE	6,489
Total Service Area, PE	56,253

56,253 PE x 93.58 gal/day/PE = 5,264,156 gallons/day

DESIGN FLOWS

Design Average Flow, MGD	9.00
Peak Hourly (Dry Weather) Flow, MGD	18.35
Peak Wet Weather Flow, MGD	35.70
PWWF through WWTP, MGD	18.35
PWWF through Excess Flow, MGD	17.35

DRY WEATHER WASTEWATER CHARACTERISTICS

BOD5 = 9.0 MGD x 181 mg/l x 8.34 lb./gal. = 13,586 lb./day
TSS = 9.0 MGD x 207 mg/l x 8.34 lb./gal = 15,537 lb./day
NH_3-N = 9.0 MGD x 21 mg/l x 8.34 lbs./gal. =1,576 lb./day

NPDES PERMIT LIMITS:

Flow	
Design Average Flow, MGD	9.0
Design Maximum Flow, MGD	18.35
CBOD5	
Monthly Average, mg/L	20
Monthly Average, lbs.	1,501
Weekly Average, mg/L	40
Weekly Average, lbs.	3,002
Suspended Solids	
Monthly Average, mg/L	25
Monthly Average, lbs.	1,877
Weekly Average, mg/L	45
Weekly Average, lbs.	3,378
Fecal Coliform	
Monthly Maximum (Geometric Mean)	200 per 100 ml
pH	
Range	6 - 9
Chlorine Residual	
Daily Maximum, mg/L	0.05
Ammonia Nitrogen	
March-May, Sept.-Oct.	
Daily Maximum, mg/L	1.8
Daily Maximum, lbs.	135
Monthly Average, mg/L	1.5
Monthly Average, lbs.	113
June through August	
Daily Maximum, mg/L	1.4
Daily Maximum, lbs.	105
Monthly Average, mg/L	1.3
Monthly Average, lbs.	98
November through February	
Daily Maximum, mg/L	3.4
Daily Maximum, lbs.	255

EAST SIDE LIFT STATION

Screens	Perforated Plate Mechanical
Number of Units	1
Opening Size, mm	3
Max flow through screen: MGD	14
Channel width: ft.	3.50
Channel depth: ft.	13.00
Discharge height: ft.	5.00
Perforation size: mm	3
Water level downstream: ft.	1.105
Screen Headloss: ft.	1.410
Max water level upstream: ft.	2.515
Screenings Washer/ Compactor	
Number of Units	1
Grinder, HP	5
Auger, HP	3
Pumps	Submersible
Number of Units	4
Horsepower, HP	100
Design Condition 1 – 100% One Pump	
Flow, GPM	4345
TDH, Ft	62
Speed, RPM	1,200
Design Condition 2 – 100% Three Pumps	
Flow, GPM (each)	3,240
TDH, Ft	80.6
Speed, RPM	1,200
Design Condition 3 – 60% One Pump	
Flow, GPM (each)	700
TDH, Ft	33.2
Speed, RPM	700

RIVERSIDE LIFT STATION

Screens	Perforated Plate Mechanical
Number of Units	2
Opening Size, mm	3
Max flow through screen: MGD (FS201/ FS202)	14/15.5
Channel width: ft.	3.50/ 4.00
Channel depth: ft.	8.33/ 9.50
Discharge height: ft.	5.00 / 5.00
Water level downstream: ft.	1.066 / 1.056
Screen Headloss: ft.	1.426 / 1.346
Max water level upstream: ft.	2.42 / 2.40
Screenings Washer/ Compactor	
Number of Units	1
Grinder, HP	5
Auger, HP	3
Pumps	Submersible
Number of Units	5
~~Pump #1~~ – Removed from Service	
~~Horsepower, HP~~	~~20~~
~~Flow, GPM~~	~~800~~
~~Force main, in.~~	~~8~~
Pump #2	
Horsepower, HP (w/ VFD)	77
Flow, GPM	3,100
Force main, in.	16
Pump #3	
Horsepower, HP (w/ VFD)	77
Flow, GPM	3,100
Force main, in.	16
Pump #4	
Horsepower, HP (w/ VFD)	180
Flow, GPM	6,700
Force main, in.	24
Pump #5	
Horsepower, HP (w/ VFD)	180
Flow, GPM	6,700
Force main, in.	24

PROCESS/EXCESS FLOW DIVERSION:

Fixed Weir Flow Splitting:	
Weir Length, feet	40
Head over Weir @ 17.35 MGD, feet	0.146

EXCESS FLOW FACILITIES:

Excess Flow Clarifier:	
Number of Units	2
Peak Wet Weather Flow, MGD	17.35
BOD5 Influent (estimated), mg/l	181
BOD5 Influent (est.), lb./day	26,191
TSS Influent (est.), mg/l	207
TSS Influent (est.), lb./day	29,953
NH3-N Influent (est.), mg/l	21
NH3-N Influent (est.), lb./day	3,039
Length, ft.	110
Width, ft.	44
Depth (average), ft.	9.43
Volume, $ft.^3$	91,282
Volume, gallons	682,800
Surface Area, $ft.^2$	9,680
Weir Length, ft.	640
Surface Loading Rate, $gal/day/ft.^2$	1,792
Solids Loading Rate, $lb./day/ft.^2$	3.94
Weir Overflow Rate, gal/day/ft.	27,109
Detention Time, minutes	58
BOD5 Removal (efficiency)	24%
BOD5 Effluent, mg/l	138
BOD5 Effluent (est.), lb./day	19,905
TSS Removal (est.)	39%
TSS Effluent (est.), mg/l	126
TSS Effluent (est.), lb./day	18,271
NH3-N Removal (est.)	0%
NH3-N Effluent (est.), mg/l	21
NH3-N Effluent (est.), lb./day	3,039
Fecal Count	1 x 107

EXCESS FLOW FACILITIES: (CONT.)

Chlorine Contact Tank:

Number of Units	2
Length, feet	96
Width, feet	21
Depth, feet	7
Volume (total), cu. ft.	28,224
Volume (total), gallons	211,116
Detention Time, minutes	17.5

PRELIMINARY TREATMENT

Grit Tank

Number of Units	2
Design	Aerated
Design Average Flow (DAF), MGD	9.00
Peak Wet Weather Flow (PHF), MGD	18.35
Length, ft.	22
Width, ft.	20
Sidewater Depth, ft.	18
Total Volume, gallons	99,858
Total Volume, cu. ft.	13,350
Detention Time at DAF, min	16.0
Detention Time at PHF, min	7.8

PRIMARY TREATMENT

Primary Settling Tanks

Number	4
Length, ft.	100
Width, ft.	20
Surface Area, sf/clarifier	2,000
Total Surface Area, sf	8,000
Overflow Rate at DAF, gpd/sf	1,125
Weir Loading Rate, gpd/ft.	15,000
Primary Influent BOD, lbs./day	13,586
Primary Influent TSS, lbs./day	15,537
Primary Influent NH_3-N, lbs./day	1,576
Removal Efficiency - BOD, %	32
Removal Efficiency - SS, %	56
BOD Removed, lbs./day	4,348
Suspended Solids Removed, lbs./day	8,701
Primary Effluent BOD, lbs./day	9,238
Primary Effluent TSS, lbs./day	6,836
Sludge Volume at 4%, gpd	26,082
VSS Solids to Digestion (85%), lbs.	12,122

Primary Sludge Pumps

Number	3
Run Time, hr./day	24
Capacity, gpm	35

SECONDARY TREATMENT

Existing Aeration	
Number of Tanks	14
Sidewater Depth, ft.	15 – 16
Aeration Basin 301 & 302, total cu. ft.	24,640
Aeration Basin 303 Thru 310, total cu. ft.	351,360
Aeration Basin 401 & 402, total cu. ft.	67,200
Aeration Basin 403& 404, total cu. ft.	35,280
Existing Volume, cu. ft.	478,480
Existing Volume, gal.	3,579,030
Detention Time at 9.00 MGD, hrs.	9.5
Organic Loading, lbs./day BOD	9,238
Organic Loading Rate, lbs./day BOD/1,000 cu. ft.	19.31
MLSS, mg/l	3,500
Solids Inventory, lbs.	104,471
RAS Return Rate, MGD	7.51
WAS, lbs./day	6,005
WAS Volume at 1% TS, gpd	72,002
Air Required Reduction , scfm	5,630
Air Provided, scfm	6,300
Sludge Age, days	17.40
F/M Ratio	0.088

Final Clarifiers	
Number	2
Design	Hydraulic Differential
Average Flow, MGD	9.00
Peak Hourly Flow, MGD	18.35
Diameter, ft.	120
Sidewater Depth, ft.	12.75
Surface Area – Each, sf	11,310
Surface Area – Total, sf	22,620
Weir Length – Each, lin. ft.	343
Weir Length – Total, lin. ft.	686
Surface Loading Rate at PHF, gpd/sf	811
Solids Loading Rate at PHF, lbs./day/sf	23.68
Weir Loading Rate, gpd/lf	26,750

RAS Pump Station	
Design	Submersible
Number of Pumps	*4*
RAS Pump Capacity	2,666 gpm @ 68.3 ft. TDH
RAS Force Main Size	16”

ULTRAVIOLET DISINFECTION

Peak Design Flow, MGD	20
UV Transmission, %(Field measured transmissivity = 80%)	*65*
TSS, mg/L	45
Disinfection Limit, fecal count	400
Design Intensity, mW	40,125
Number of Channels	1
Number of Reactors per channel	1
Number of Banks/ Reactor	2
Number of Modules per Bank	4
Total Number of UV Lamps	72
Type of level control	Fixed Weir
Automatic Mechanical Cleaning	Yes

SLUDGE HANDLING FACILITY

Sludge Thickening - Gravity Belt Thickeners	
Number of GBT’s	1
Belt Width, meters	2
Solids Loading, lbs. DS/day	6,005
Solids Loading, gallons/day	72,002
Maximum Loading Rate, lbs. DS/hr.	2,000
Operation, hrs./week	21
Thickened Sludge Volume at 5% TS, gpd	14,400

TWAS Storage Tank	
Number	1
Volume, gal.	73,462
Storage, days	5.1

SLUDGE HANDLING FACILITY (CONT.)

Anaerobic Digestion	
Number	2
Design	Egg-Shaped
Volume, cu. ft. each	64,171
Total Volume, cu. ft.	128,342
Total Volume, gpd	960,000
TWAS VSS, lbs./day	4,726
Primary VSS, lbs./day	7,396
Volatile Solids Loading Rate, lbs. VSS/day	12,122
Loading Rate, lbs. VSS/1000 cu. ft.	94
Loading Rate, gpd	40,782
Detention Time, days	24
Gas Production	
Actual Gas Production:	
Low End Gas Production, cu. ft./day	91,644
High end Gas Production, cu. ft./day	137,466
Minimum Per EPA	
VSS Reduction, %	38
VSS Reduction, lbs.	4,606
Low End Gas Production, cu. ft./day	55,272
High End Gas Production, cu. ft./day	82,908
Low End Heating Range, btu/day	33,163,200
High End Heating Range, btu/day	49,744,800
Sludge Decant/Storage Tank	
Number	1
Diameter, ft.	45
Sidewater Depth, ft.	7.0
Volume, cf	11,133
Volume, gal.	83,274
Storage, days	2
Centrifuges	
Number of units	2
Hydraulic Loading, gpm	150
Solids Loading, lbs. TS/hr.	1.875
Operation, hrs./centrifuge/week	16.5

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SECTION 6 (A.K.A. PHOSPHORUS REMOVAL FEASIBILITY STUDY)
ALTERNATIVES ANALYSIS FOR NUTRIENT REMOVAL

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6. ALTERNATIVES ANALYSIS FOR NUTRIENT REMOVAL

6.1 PROJECT BACKGROUND

The City of St. Charles' Main Wastewater Treatment Facility (MWWTF) discharges to the Fox River. According to the Illinois EPA Clean Water Act Section 303(d) List, the Fox River does not meet water quality standards for its intended use in the majority of the segments, including the segments immediately downstream of the St. Charles MWWTF. The impairment on the river for aquatic life is based on a low dissolved oxygen concentration. This low dissolved oxygen content is due to algal growth and exacerbated by the presence of pools upstream of the low head dams along the river.

In 2001, the Illinois EPA was contemplating performing a Total Maximum Daily Load (TMDL) study on the Fox River in an attempt to address the impairment. At that time, there was insufficient data available to support a TMDL and therefore would simply be a modeling exercise which would not reflect actual environmental conditions. Many of the communities along the Fox River (including St. Charles) joined forces with other stakeholders, including Friends of the Fox and Sierra Club, to form the Fox River Study Group (FRSG). The FRSG determined that it was in the best interest of all the stakeholders if a comprehensive solution was developed and that solution was based on comprehensive river-monitoring data and modeling. The FRSG, in concert with the POTWs along the river, have monitored the river for numerous constituents including phosphorus, nitrogen, fecal coliform and chlorophyll a. This water quality data provided the basis for development of QUAL2K and HSPF models.

In 2004, the Illinois EPA implemented statewide nutrient removal criteria for wastewater treatment facilities that were proposing expansion of their hydraulic capacity. Two nutrients of concern were total nitrogen and phosphorus. The NPDES Permits issued for these facilities typically contained an interim 1 mg/L annual average phosphorus limit and requirement to monitor total nitrogen.

In 2011, the Illinois EPA was receiving increased pressure by the USEPA and environmental stakeholders to address nutrient criteria on all POTWs, not only treatment plants undergoing expansion. Several NPDES permits along the Fox River had expired and were due to be reissued by the Illinois EPA. However, the Illinois EPA elected to delay reissuance so the NPDES permits could incorporate language agreed upon in ongoing discussions on nutrient criteria.

In January 2012, in an attempt to build consensus among all stakeholders, the Illinois EPA presented the FRSG with special conditions in draft form for nutrient criteria. The FRSG had not yet completed the low flow monitoring required to calibrate the HSPF and QUAL2K models. Therefore, determination of a water quality based phosphorus limit could not be determined at that time. The FRSG in conjunction with the Illinois EPA worked to develop a schedule for completion of the modeling effort and determination of water quality based phosphorus standards. During the drought in the summer of 2012, the FRSG was able to obtain low flow monitoring for the Fox River and further calibrate the model.

In January 2013, the Illinois EPA and FRSG were able to agree on special conditions for all dischargers greater than 1 MGD. These conditions included a 1 mg/L interim phosphorus standard and a schedule for completion of the water quality modeling for the development of permanent phosphorus criteria. The permit language requires the FRSG to complete analysis of the alternatives and provide recommendations by December 2015. The permit also requires the POTWs to perform a study and determine the cost for compliance of phosphorus removal for a 1 mg/L standard as well as a 0.5 mg/L standard. It is the intent of the special conditions that all dischargers along the Fox River will meet the recommended standards by 2030.

The City of St. Charles received a final NPDES permit in December of 2014. The special conditions are outlined below:

> *SPECIAL CONDITION 14. This Permit may be modified to include alternative or additional final effluent limitations pursuant to either an approved Total Maximum Daily Load (TMDL) Study or an approved Fox River Implementation Plan.*
>
> *SPECIAL CONDITION 17. The Permittee shall monitor the wastewater effluent for Total Phosphorus, Dissolved Phosphorus, Nitrate/Nitrite, Total Kjeldahl Nitrogen (TKN), Ammonia, Total Nitrogen (calculated), Alkalinity and Temperature at least once a month beginning on the effective date of this permit. The results shall be submitted on Discharge Monitoring Report (DMR) Forms or NetDMRs to IEPA unless otherwise specified by the IEPA.*
>
> *SPECIAL CONDITION 18. The Permittee shall participate in the Fox River Study Group (FRSG). The Permittee shall work with other watershed members of the FRSG to determine the most cost effective means to remove dissolved oxygen (DO) and offensive condition impairments in the Fox River. This Permit may be modified to include additional conditions and effluent limitations to include implementation measures based on the Fox River Implementation Plan (Implementation Plan). The following tasks will be completed during the life of this permit:*
>
> 1. *The Permittee shall prepare a phosphorus removal feasibility report specific to its plant(s) on the method, time frame and costs for reducing its loading of phosphorus to levels equivalent to monthly average discharges of 1 mg/L and 0.5 mg/L on a seasonal basis and on a year round basis. The feasibility report shall be submitted to the I EPA twelve (12) months from the effective date of the Permit. The feasibility report shall also be shared with the FRSG.*
> 2. *The Permittee shall submit the Fox River Study Group Watershed Investigation Phase Ill Report, which includes stream modeling, to the I EPA within 1 month of the effective date of this Permit.*
> 3. *The FRSG will complete an Implementation Plan that identifies phosphorus input reductions by point source discharges, non-point source discharges and other measures necessary to remove DO and offensive condition impairments in the Fox River. The Implementation Plan shall be submitted to the I EPA by December 31, 2015. The Permittee shall initiate the recommendations of the Implementation Plan that are applicable to said Permittee during the remaining term of this Permit. This Permit may be modified to include additional pollutant reduction activities necessary to implement the Implementation Plan.*
> 4. *In its application for renewal of this permit, the Permittee shall consider and incorporate recommended FRSG phosphorus input reduction implementation projects that the Permittee will implement during the next permit term.*
> 5. *The Permittee shall operate the existing facilities to optimize the removal of phosphorus.*

SPECIAL CONDITION 19. A phosphorus limit of 1.0 mg/L (Annual Average) shall become effective four and one-half (4 1/2) years from the effective date of this Permit.

In order for the Permittee to achieve the above limit, it will be necessary to modify existing treatment facilities to include phosphorus removal, reduce phosphorus sources or explore other ways to prevent discharges that exceed the limit. The Permittee must implement the following compliance measures consistent with the schedule below:

1. *Interim Report on Phosphorus Removal Feasibility Report*

   6 months from the effective date of this Permit
2. *Phosphorus Removal Feasibility Report submitted*

   12 months from the effective date of this Permit
3. *Progress Report on Phosphorus Input Reductions and Implementation Plan*

   18 Months from the effective date of this Permit
4. *Progress Report on Recommendations of Implementation Plan*

   24 months from the effective date of this Permit
5. *Plans and specifications submitted*

   30 months from the effective date of this Permit
6. *Progress Report on Construction*

   36 months from the effective date of this Permit
7. *Complete Construction*

   42 months from the effective date of this Permit
8. *Progress Report on Optimizing Treatment System*

   48 months from the effective date of this Permit
9. *Achieve Annual Concentration and Loading Effluent Limitations for Total Phosphorus*

   54 months from the effective date of this Permit

Compliance dates may be modified based on the results of the Phosphorus Removal Feasibility Report required by Special Condition 18 of this Permit. All modifications of this Permit must be in accordance with 40 CFR 122.62 or 40 CFR 122.63.

In summary, the St. Charles MWWTF must comply with a 1 mg/L annual average phosphorus limit. It is likely that the Facility will need to achieve lower phosphorus effluent limits prior to 2030. Phosphorous removal in wastewater treatment plants was common in the 1970's. The most widespread method of phosphorous removal used at that time was the addition of chemical coagulants that cause phosphate compounds to settle out of solution. Phosphorous removal is also possible through biological processes, but the amount of phosphorous that can be removed through such processes is limited. Both biological and chemical phosphorus removal options will be evaluated in this section.

6.2 BIOLOGICAL NUTRIENT REMOVAL

All life forms utilize a food source and a source of oxidative potential, usually oxygen or nitrite, to absorb phosphates into their bodies as the molecule adenosine tri-phosphate (ATP). This process is known as metabolism. Phosphorous is released from ATP to provide energy for cellular growth and activities. When activated sludge is produced and collected, phosphates absorbed within the cells of microorganisms as ATP and other cellular components are removed from the wastewater flow. This is the basis for biological phosphorous removal, a small amount of which occurs in all activated sludge processes in which activated sludge is wasted.

Greater amounts of phosphorous can be removed through biological methods by creating an anaerobic zone, in which no oxygen or nitrate is available, within a treatment facility's suspended biological growth processes. Most microorganisms are not capable of storing large amounts of ATP and rely on a constant rate of metabolism to maintain cellular activity. Certain microorganisms known as Phosphorus Accumulating Organisms (PAOs) can store significantly more phosphorous than other heterotrophic bacteria. PAOs are capable of survival in an anaerobic environment absent of nitrate and oxygen. As such, the percentage of PAOs within the microbiological community increases when the process includes an anaerobic zone. The larger PAO population ensures a higher concentration of phosphorus within the sludge wasted from the process.

Biological Phosphorus Removal (BPR) requires rigid operational control in order to maximize the efficiency of the process. The process is sensitive to changes in temperature, flow and feed concentration. BPR may not be able to continuously meet the interim 1 mg/L effluent standard set by the IEPA. Therefore, chemical polishing capabilities would be incorporated into a biological phosphorus removal design.

It is important to note that the phosphorous captured in the BPR process is simply stored in the bodies of microorganisms and can easily be returned to solution. The high phosphorus sludge is wasted from the biological process to a sludge stabilization process. Once stabilized, the sludge is then dewatered and disposed of through land application or land filling operations.

The existing biological process may be modified to reduce the concentration of phosphorus for the new NPDES permit limit. Consideration also must be given for the biological reduction of nitrogen for possible future limits. This approach to wastewater treatment is called Biological Nutrient Removal (BNR).

For the consideration of a BNR alternative, the overall system was modeled to identify potential operational issues and boundary conditions. The use of these models has become standard industry practice for evaluation and design of biological treatment plant processes, especially in phosphorus removal applications. The model was developed utilizing existing dimensions of the biological process basins, and was calibrated by data obtained during an intensive sampling and lab testing process. The protocol for the intensive sampling is included as Appendix B.

It has been documented that anaerobic zones are needed to provide an environment where the PAOs are allowed to metabolize influent organic material with limited competition from other organisms. In this environment, the PAOs release phosphorus and metabolize the readily biodegradable Chemical Oxygen Demand (rbCOD). In downstream aerobic zones, the PAOs enter an endogenous state and perform luxury uptake of phosphorus. The following excerpt from the 4th Edition of Wastewater Engineering: Treatment and Reuse (Metcalf and Eddy) further explains the zones within a typical Biological Phosphorus Removal (BPR) system:

> *"Wastewater characterization, including rbCOD measurements, is essential to evaluate fully the design and performance of BPR systems. Biological phosphorus removal is initiated in the anaerobic zone where acetate (and propionate) is taken up by phosphorus-storing bacteria and converted to carbon storage products that provide energy and growth in the subsequent anoxic and aerobic zones. The rbCOD is the primary source of volatile fatty acids (VFAs) for the phosphorus-storing bacteria ... The more acetate, the more cell growth, and, thus, more phosphorus removal."*

Most BNR processes also address nitrogen removal. Raw wastewater is anaerobic and therefore the majority of nitrogen is in the form of ammonia. The nitrogen cycle includes four forms; ammonia → nitrite → nitrate → nitrogen gas. Ammonia, nitrite and nitrate are all soluble, whereas nitrogen gas is released to the atmosphere. Therefore, removal of nitrogen from wastewater requires a process which produces nitrogen gas. Nitrification is an aerobic process where organisms oxidize ammonia to nitrite and nitrate. Nitrosomonas and similar microorganisms oxidize ammonia (NH_3) to nitrite (NO_2). Nitrite is oxidized to nitrate (NO_3) by nitrobacter and similar microorganisms. Denitrification is an anoxic process where organisms reduce nitrate to nitrogen gas (N_2). The driving mechanism for denitrification is the microorganisms need to obtain the oxygen molecule for respiration. This process is more efficient when microorganisms have a readily available carbon source.

The alternation from anaerobic, anoxic and aerobic zones have been modified, enhanced and utilized in several different configurations. As the influent to each wastewater treatment facility and the desired effluent quality is different, the configuration of BPR or BNR processes must be carefully evaluated.

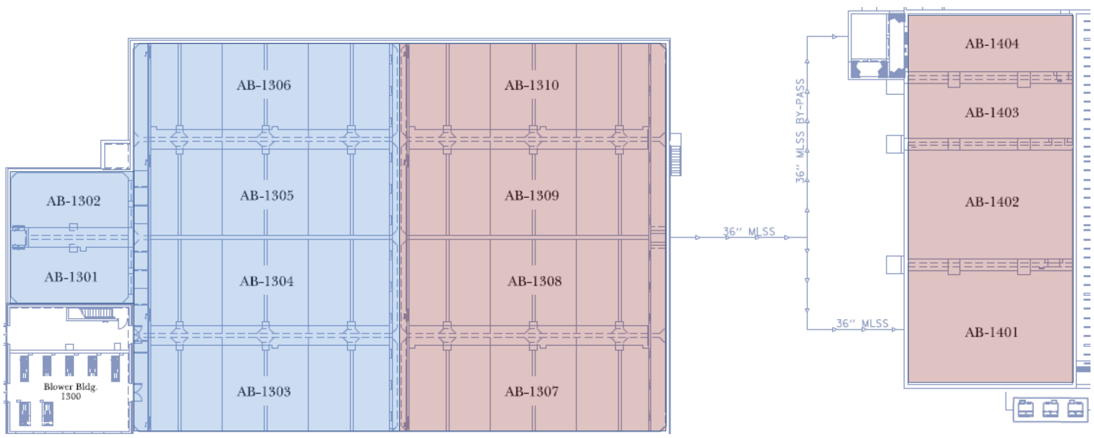
During the study, BioWin™ was utilized to model the existing plant and alternative process configurations for phosphorus and nitrogen removal. The alternatives modeled only utilized the existing upper and lower aeration basins. The models were based on a design average flow rate of 9 MGD. Based on site-specific data, it was determined that the model should utilize a MLSS temperature of 9^oC and an influent rbCOD concentration of 78 mg/L. Other parameters included the maximum monthly average $CBOD_5$ (239 mg/L), ammonia (26 mg/L) and TKN (40 mg/L) concentrations. The model was calibrated and validated by comparing the results to existing flows and loads. Once calibrated, four BNR alternatives were evaluated.

6.2.1. A/O Process

The A/O configuration of the biological process is named for its anaerobic and aerobic ("oxic") zones. This is a BPR process and only addresses phosphorus removal. As shown below, basins 1301 through 1306 would be converted from aerobic zones to anaerobic zones. The remainder of the process would remain aerobic.

The model indicated that this configuration would be able to address the 1.0 mg/L annual average phosphorus limit. Also, implementation would only require the conversion of roughly 42% of the existing aeration basins to anaerobic basins, which is relatively inexpensive. However, this configuration lacks the ability to denitrify and therefore does not have the flexibility to address any future nitrogen limits without substantial modifications.

Exhibit 6-1 | A/O Process



Blue = Anaerobic, Red = Aerobic (i.e. "Oxic")

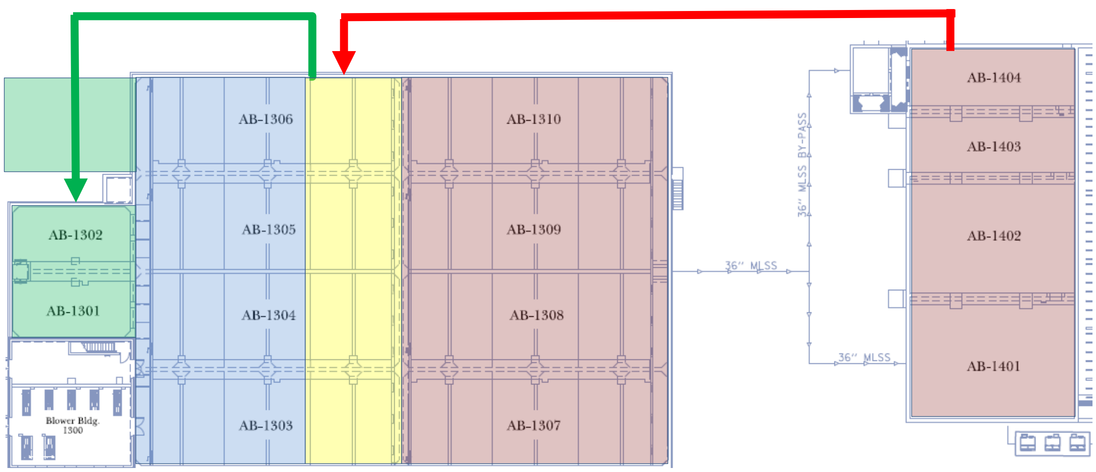
6.2.2. Modified Johannesburg Process

This configuration of the biological process originated in Johannesburg, South Africa as an alternative to the UCT (University of Cape Town) process. The process utilizes four different zones; pre-anoxic, anaerobic, anoxic, and aerobic.

The pre-anoxic zone is designed to denitrify the RAS to minimize nitrate interference in the downstream anaerobic process. Basins 1301 and 1302 would be converted to pre-anoxic zones. An internal recycle of 0.1 times the design flow from the end of the anaerobic zones would provide organic loading to facilitate denitrification. The modeling suggested that the pre-anoxic zone may require construction of additional basins similar in size to 1301 and 1302.

Basins 1303 through 1306 would be divided into anaerobic and anoxic zones. MLSS from the pre-anoxic zones would be blended with primary effluent and introduced at the head of the anaerobic zones. Within the anaerobic zones, the PAOs metabolize rbCOD and release polyphosphates. At the head of the anoxic zone, MLSS from the anaerobic zones is blended with a second internal recycle (2 times the design flow) from the end of the aerobic zones to facilitate denitrification. The remainder of the basins would be aerobic to provide nitrification and phosphorus uptake.

Exhibit 6-2 | Modified Johannesburg Process



Green = Pre-Anoxic, Blue = Anaerobic, Yellow = Anoxic, Red = Aerobic (i.e. “Oxic”)

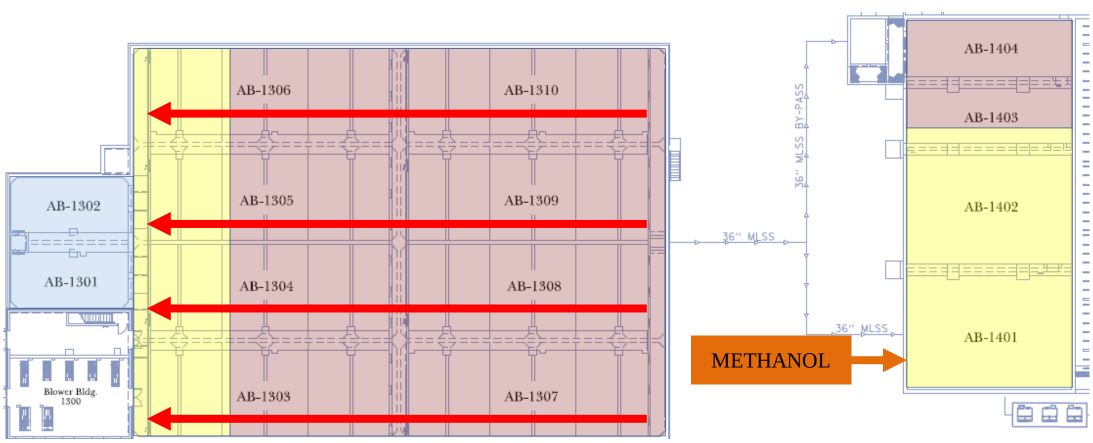
The model indicated that this configuration would be able to address the 1.0 mg/L annual average phosphorus limit as well as nitrogen removal. This configuration would require two internal recycle pump stations. One may be done with in-pipe mixers (green arrow). The other (red arrow) would require a more substantial pumping system to overcome the static head between the upper and lower basins. This configuration would require construction of additional tankage, baffle walls, and conversion of roughly 42% of the existing aeration basins to anoxic/anaerobic basins.

6.2.3. Five-Stage Bardenpho Process

Originally developed by Dr. James Barnard, this configuration provides denitrification and phosphorus removal, which is the basis for the name of the process (Bar-den-pho). The head of the process is an anaerobic zone, followed by the first set of anoxic and aerobic zones. An internal recycle of approximately 4 times the design flow from the end of the first aerobic zones is conveyed to the head of the first anoxic zones. This internal recycle will denitrify approximately 80% of the flow. The configuration ends with a second set of anoxic and aerobic zones. The second anoxic zones provide additional denitrification by utilizing nitrate from the first aerobic zones in combination with the organic carbon to create nitrogen gas, which is stripped from the water in the final aerobic zone. The zones would be split by the construction of baffle walls within the existing basins. Roughly 43% of the existing aeration basins would be converted to anoxic/anaerobic basins.

The typical Five-stage Bardenpho process requires approximately 14 hours of hydraulic retention. The existing basins provide less than 10 hours at design flow, and may not be able to achieve the effluent nutrient limits consistently within the biological process. The BioWin™ model demonstrated that 10 hours detention time is insufficient. Implementation of this process would require improvements to provide sufficient capacity within the first aerobic zones. This may be done by constructing additional basins or by employing an Integrated Fixed-Film Activated Sludge (IFAS) system within the process. The cost of implementation of either alternative would be significant.

Exhibit 6-3 | 5-Stage Bardenpho Process



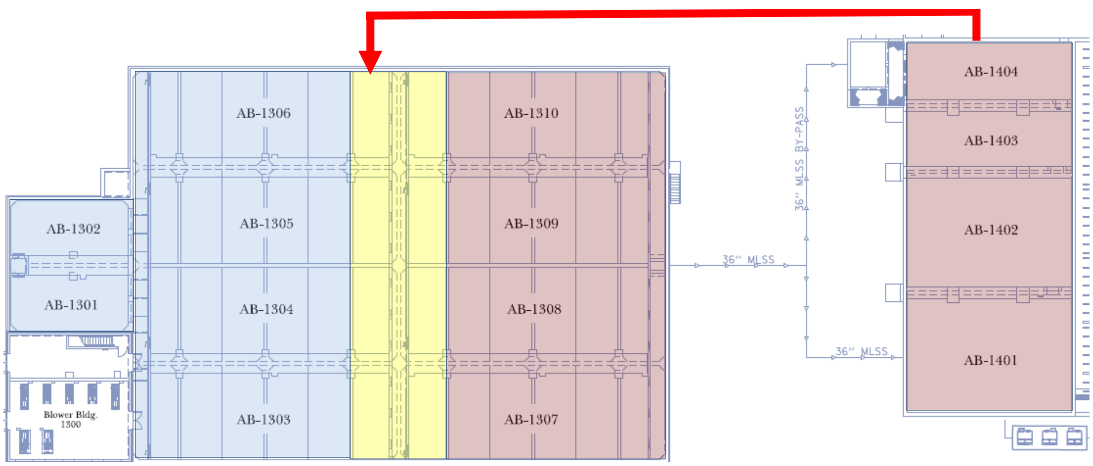
Blue = Anaerobic, Yellow = Anoxic, Red = Aerobic

6.2.4. A^2/O Process

The A^2/O configuration of the biological process utilizes three zones. The head of the process is an anaerobic zone, followed by an anoxic and an aerobic zone. An internal recycle of approximately 2 times the design flow from the end of the aerobic zones is conveyed to the head of the anoxic zones. This internal recycle will denitrify approximately 66% of the flow.

The major cost of implementation for this process would be the construction of the internal recycle pump station. However, this configuration can be implemented within the existing basins, requiring only the construction of baffle walls for zone isolation and the conversion of roughly 46% of the existing aeration basins to anoxic/anaerobic basins. In addition, operation and maintenance would be simplified by having all internal recycle pumping equipment in one location.

Exhibit 6-4 | A^2/O Process

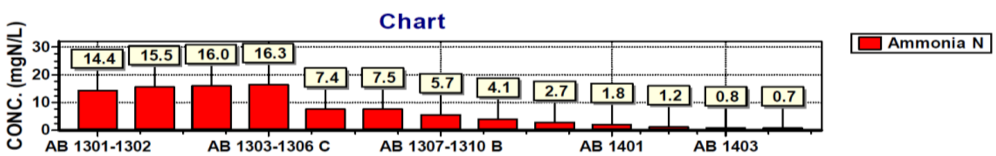


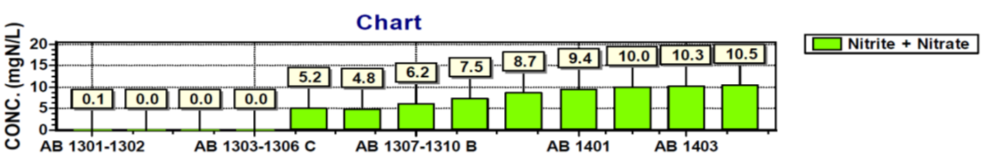
Blue = Anaerobic, Yellow = Anoxic, Red = Aerobic (i.e. "Oxic")

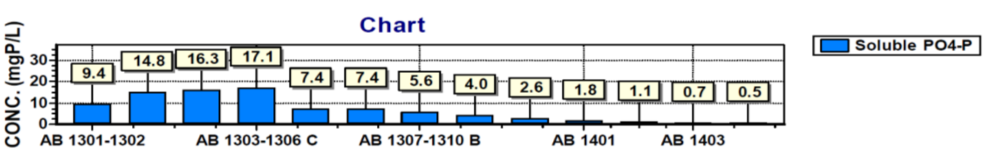
6.2.5. BNR Process Recommendations

The four alternatives were presented to the City during a one-day work session. The team members presented the strengths and limitations of each alternative. A consensus was reached during this meeting that the A^2/O configuration was the simplest and most stable process for biological nutrient removal.

Based on this conclusion, further analysis of the A^2/O process was performed to identify and confirm the initial findings. Specific adjustments included increasing the MLVSS concentration, optimization of basin sizing, and a review of internal recycle rates. The projected influent rbCOD concentration remained 78 mg/L and the MLVSS concentration increased to 2,300 mg/L. At these conditions, the A^2/O process was able to satisfy the new NPDES permit limits. Projected phosphorus and ammonia effluent concentrations were projected to be 0.79 mg/L and 0.66 mg/L, respectively. In addition, total nitrogen removal is expected to be 47%. Under these conditions, the model predicted the following concentrations throughout the biological process:







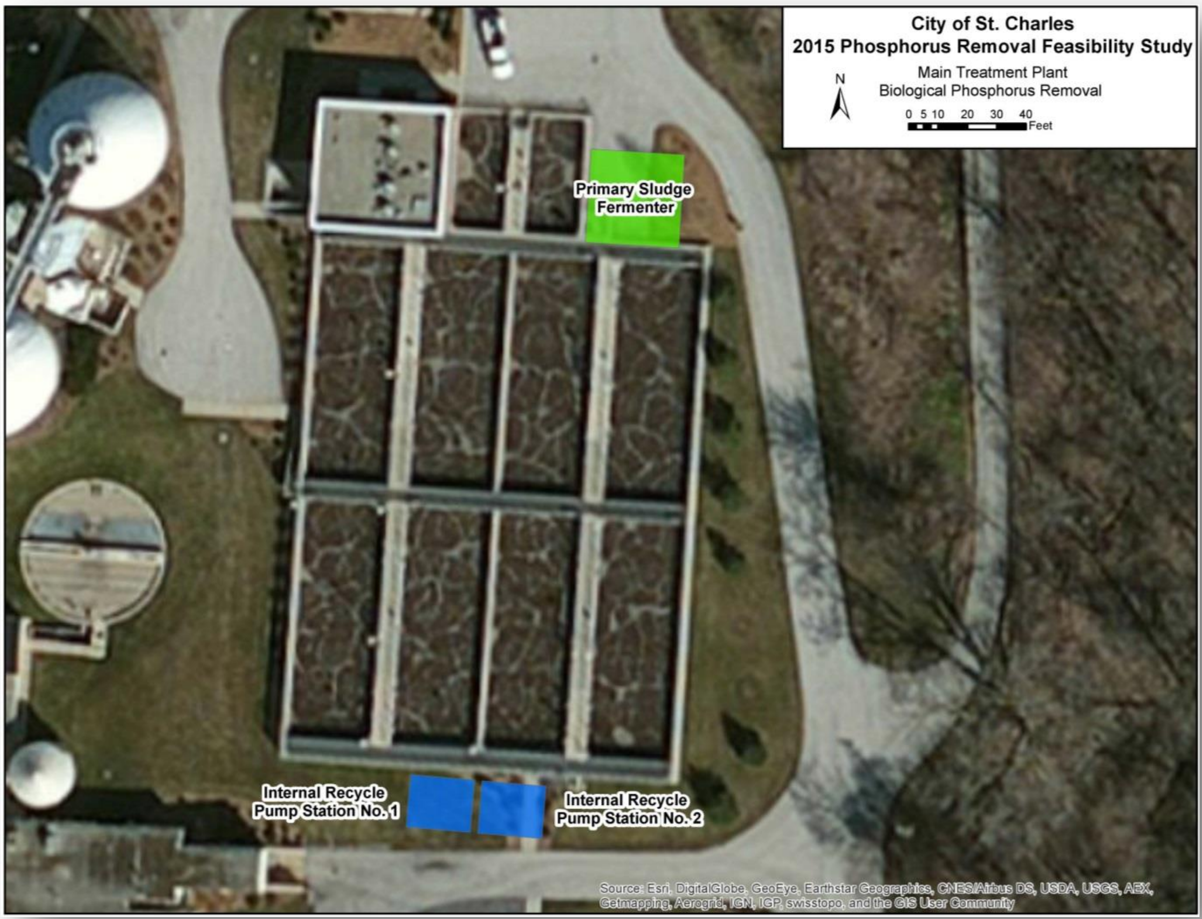
The final output from the model describes concentrations in the secondary clarifier supernatant, prior to disinfection. The output data is shown below:

Table 6-1 | Effluent Concentrations from A^2/O Model

Total COD mg/L	Total CBOD mg/L	Total P mgP/L	Soluble PO_4-P mgP/L	Total N mgN/L	Ammonia N mgN/L	Nitrate N mgN/L	Nitrite N mgN/L	Total SS mgTSS/L	Volatile SS mgVSS/L
28.48	2.61	0.79	0.48	12.93	0.66	8.57	1.89	5	3.5

The results from the model analysis for the A^2/O configuration are included as Appendix C of this report. Based on the analysis, it is recommended that the improvements incorporate a primary sludge fermenter for carbon augmentation. A recommended location of the proposed primary sludge fermenter and internal recycle pump stations is shown in Exhibit 6-1.

Exhibit 6-5 | Proposed Layout for A^2/O Process



Under average loading conditions, the existing process is able to meet effluent standards at and above 50° F (10° C). Under maximum day demand loading, the existing process is only able to meet effluent standards with wastewater at and above 57° F (14° C). The model indicates that additional detention time is necessary under high loading and low temperature conditions.

In order to address this issue, the process could be extended from 9.6 hours detention time to 12.5 hours with additional tankage to achieve nitrification, denitrification and biological phosphorus removal. A second option would allow the biological process to revert back to single-stage nitrification under these loadings and temperatures. In this mode, the City would perform chemical phosphorus removal to achieve the proposed NPDES permit limits.

A third option includes implementation of an Integrated Fixed-film Activated Sludge (IFAS) process. IFAS systems have been implemented in over 50 facilities worldwide. The system utilizes aeration basins that contain media with high specific surfaces, in addition to standard anaerobic, anoxic and aerobic zones. The media is suspended in the flow and biomass attaches itself to the media surfaces. The biomass attached to the media within the IFAS zones would include a high percentage of nitrifying bacteria (nitrosomonas, nitrobacter and similar microorganisms). The media is retained within the zone by screening assemblies, and the resulting process has a lower MLSS and hydraulic retention time required for nitrification.

Implementation of a BNR process must consider effects on downstream processes. The sludge stabilization process is anaerobic digestion. Orthophosphate tied up in PAOs from the BNR process is released under anaerobic conditions, increasing concerns regarding struvite formation. Struvite is a compound made up of magnesium, ammonium and phosphorus. Alkaline conditions increase the potential for struvite crystallization, which can attach to the mixing systems, heat exchangers, sludge recirculation pumps and sludge transfer pipes. Struvite may be controlled by minimizing the concentrations of the three main soluble ions or chemical addition to reduce the pH level.

Two alternatives were evaluated to address the struvite issue. The first is a struvite harvesting process which includes a pretreatment step to promote phosphorus release prior to thickening. Recovering phosphorus prior to and following anaerobic digestion significantly decreases the uncontrolled formation of struvite within the solids handling system. The phosphorus recovered is in pellet form and suitable for the fertilizer market.

The project team has evaluated struvite harvesting. The probable cost for a struvite harvesting system is approximately $8 Million. Evaluation of the system has determined that it is more cost effective for wastewater treatment facilities 15 MGD or greater.

The second alternative is implementation of a chemical buffering system and chemical storage facility. An on line pH monitoring system would dose the digestion contents with a buffering agent, such as a weak acid, to maintain pH levels between 6.5 and 7.5 and avoid struvite formation. If BNR is the selected alternative, it is recommended that a chemical buffering system be added to the overall process.

Cost estimates for all three alternatives were prepared. The first alternative includes implementation of a BNR process as the normal operational mode, with single-stage nitrification during high loading and low temperatures.

Table 6-2 | Cost Estimate for Biological Nutrient Removal (A^2/O) – 9 MGD

GENERAL CONDITIONS	$552,000
SITE WORK	$520,000
PRIMARY SLUDGE FERMENTER	$575,652
A^2/O PROCESS	$2,713,537
CHEMICAL FEED SYSTEM	$764,770
CHEMICAL BUFFER SYSTEM	$261,620
CONSTRUCTION SUBTOTAL	$5,387,579
CONTINGENCY @ 20%	$1,077,516
CONSTRUCTION TOTAL	$6,465,095
PROJECT ENGINEERING (14%)	$905,113
TOTAL PROJECT COST	$7,370,208

The second alternative expands the biological process to provide 12.5 hours detention time and would function over the entire range of operational conditions. This alternative would include construction of additional 1.5 million gallons of detention time within the biological process. The most logical location for this additional tankage is immediately south of the upper aeration basins

Table 6-3 | Cost Estimate for BNR with Expanded Capacity – 9 MGD

GENERAL CONDITIONS	$937,000
SITE WORK	$2,353,340
PRIMARY SLUDGE FERMENTER	$575,652
A^2/O PROCESS	$4,357,556
CHEMICAL FEED SYSTEM	$764,770
CHEMICAL BUFFER SYSTEM	$261,620
CONSTRUCTION SUBTOTAL	$9,249,937
CONTINGENCY @ 20%	$1,849,987
CONSTRUCTION TOTAL	$11,099,925
PROJECT ENGINEERING (14%)	$1,553,989
TOTAL PROJECT COST	$12,653,914

The third alternative includes implementation of an IFAS system in lieu of expanded tankage, and would function over the entire range of operational conditions.

Table 6-4 | Cost Estimate for BNR with IFAS System – 9 MGD

GENERAL CONDITIONS	$926,000
SITE WORK	$520,000
PRIMARY SLUDGE FERMENTER	$575,652
IFAS BARDENPHO	$6,143,000
CHEMICAL FEED SYSTEM	$764,770
CHEMICAL BUFFER SYSTEM	$261,620
CONSTRUCTION SUBTOTAL	$9,191,042
CONTINGENCY @ 20%	$1,838,208
CONSTRUCTION TOTAL	$11,029,250
PROJECT ENGINEERING (14%)	$1,544,095
TOTAL PROJECT COST	$12,573,345

The analysis for the service area to the Main WWTF provided in Section 2 indicates that at build-out, the facility will serve 56,254 PE. The average flow is 93.58 gcd, which converts to an average daily flow of 5.26 MGD rather than 9 MGD. Using the modeled conditions (MLSS at 9^{o}C, 239 mg/L $CBOD_5$, 26 mg/L ammonia and 40 mg/L TKN), the first alternative is able to meet effluent standards at 5.26 MGD.

Without a driving force to expand the biological process, and recognizing that the existing infrastructure is able to maintain the desired effluent quality, it is apparent that that the first alternative is the most cost effective solution for BNR. If the first alternative is implemented, it should be done in a manner that would allow the City to implement either alternative two or three in the future. For the purposes of this study, the alternative analysis will focus on comparing the A^2/O process with chemical phosphorus removal.

6.3 CHEMICAL PHOSPHORUS REMOVAL

Chemical precipitation of phosphorus can be accomplished within either the primary or secondary treatment process. The City has several options for chemical selection. Lime addition is effective but produces a considerable amount of sludge. Alum and iron salts are more commonly recommended. The locally available iron salts include ferric chloride ($FeCl_3$) and ferrous sulfate ($FeSO_4$). Both are highly corrosive and should be stored in a separate, well-ventilated area.

It is estimated that the sludge production from chemical precipitation in the primary clarifiers will yield four times the influent pounds of phosphorus removed, which would increase overall primary sludge production by roughly 50%. Other more conservative estimates indicate sludge yields increasing by 100%. The actual yield should be field verified. Benefits of adding iron salt or alum to the primary clarifiers include increased efficiency in solids and BOD_5 removal and precipitation of copper ions.

Chemical precipitation within the secondary process is slightly more predictable. Application points vary from site to site. Some facilities introduce the chemical to the RAS prior to entering the basins while others add the iron salt or alum in the MLSS diversion structure. Advantages of precipitation in the secondary process include lower chemical requirements, increased settling ability of the flocculation within the clarifiers, and lower sludge production. However, the sludge produced is a waste activated sludge and can reduce the efficiency of the anaerobic digestion system.

The average influent phosphorus concentration is approximately 6 mg/L, which was verified during the intensive sampling conducted for the calibration of the A^2/O BNR model. The chemical precipitation required for phosphorus removal is estimated to be one mole of iron (Fe) for one mole phosphorus (P). However, an additional one to five moles of iron is required to satisfy competing reactions, such as hydroxide formation. The anticipated chemical relationship between chemical dose and phosphorus removal was determined for the City of St. Charles by jar testing.

Jar testing was performed on raw influent, secondary clarifier influent, double-dosing of secondary clarifier influent (assuming the addition of filters for solids removal of the secondary clarifier effluent) and the centrate/filtrate side stream (assuming improvements are made to treat this flow before it is sent to the collection system). Testing utilized ferric chloride and alum to determine the dosage requirements of these two metal salts for phosphorus removal. The protocol for this jar testing is included as Appendix D.

One minor change made to the protocol was to lower the dosage range for ferric chloride based on the anticipated chemical relationship of the facility influent. This change was made based on the assumption that competing reactions would require less than 4 moles of chemical per mole of phosphorus for treatment. While additional data is needed to more accurately predict exact chemical dosage requirements for phosphorus removal, this testing data will provide direction as to the most efficient dosing location (where the effluent limit may be reached using the least amount of chemical).

The existing biological process reduces the phosphorus from 6 mg/L to about 3.5 mg/L. Results from the primary clarifier jar testing indicated that the dosage requirements for phosphorus removal from 3.5 mg/L to 1 mg/L may be cost prohibitive if utilized only in this location (approximately 70 gallons per hour). This would also increase primary sludge production significantly, which increases solids handling costs.

In order to size the chemical feed system for side stream treatment of centrate and filtrate, additional jar testing is required for both ferric chloride and alum; the effective dose appears to be well above the dosing range of the jar testing. Side stream treatment would also require the construction of additional tankage to retain the flow so that the effectiveness of the chemical dosing may be field verified and modified if necessary. Results for the double dosing of secondary clarifier influent were also inconclusive, and would require more testing as well as the construction of filters to remove the solids prior to disinfection.

The jar testing indicated that approximately 40 mg/L dosage of ferric chloride is required for secondary treatment to consistently reduce effluent phosphorus concentration below 1 mg/L. The calculations for ferric chloride ($FeCl_3$) addition at 35% solution strength are as follows:

Estimate Phosphorus Loading

6 mg/L x 8.34 lbs. /gal x 9.0 MGD = 450.4 lbs. PO_4/day

Estimate Chemical Relationship from Jar Testing Data

40 mg/L x 8.34 lbs. /gal x 9.0 MGD = 3,002 lbs. $FeCl_3$/day

3,002 lbs. $FeCl_3$/day / (11.23 lbs. /gal x 35%) = 764 gallons $FeCl_3$/day

764 gallons $FeCl_3$/day / 450.4 lbs. PO_4/day = 1.7 gallons $FeCl_3$/lb. PO_4

1.7 gal $FeCl_3$/lb. PO4 * 10.9 mol $FeCl_3$/ gal $FeCl_3$= 18.489 mol $FeCl_3$/ lb. PO_4

18.489 mol $FeCl_3$ / lb. PO_4 / 4.768 mol PO_4 / lb. PO_4 = 3.9 mol $FeCl_3$ per mole of PO_4

This indicates that the dosage for secondary treatment at the St. Charles MWWTF will utilize a chemical relationship of approximately 4 moles $FeCl_3$ per mole PO_4 to overcome competing reactions.

Estimate FeCl3 dosage for Secondary Treatment (use 4 moles $FeCl_3$ / mole PO_4)

4 mol $FeCl_3$ per mole of PO_4 x 4.768 mol PO_4 / lb. PO_4 = 19.072 mol $FeCl_3$ /lb. PO_4

19.072 mol $FeCl_3$ / lb. PO_4 / 10.9 mol $FeCl_3$ / gal $FeCl_3$ = 1.75 gals $FeCl_3$ / lb. PO_4

Use 1.8 gallons of $FeCl_3$ per pound PO_4

Estimate Total Volume of $FeCl_3$ required for Secondary Treatment

At a DAF of 9.0 MGD

6 mg/L x 9.0 MGD x 8.34 = 450.4 lbs. PO_4 / day

1.8 gallons $FeCl_3$ / lb. PO4 x 450.4 lbs. PO_4 / day = 810.7 gallons $FeCl_3$ / day

810.7 gallons $FeCl_3$ / day = approx. 30 gph $FeCl_3$

The calculated dosing requirements for secondary treatment are much more reasonable (approximately 30 gallons per hour). This equates to a daily usage of approximately 811 gallons. For 30 day's storage, the City would need to have approximately 24,330 gallons onsite.

This single dosing location would also require the least amount of capital cost to implement in order to attain a 1 mg/L annual average phosphorus limit. Therefore, it is recommended that installation of a chemical feed system at the splitter box upstream of the secondary clarifiers be considered by the City for chemical phosphorus removal. A recommended location of the proposed chemical feed building is shown in Exhibit 6-2.

Exhibit 6-6 | Proposed Layout of Chemical Phosphorus Removal

The capital cost estimate for implementing chemical phosphorus removal to attain a 1.0 mg/L annual average phosphorus limit is shown below.

Table 6-5 | Cost Estimate for Chemical Phosphorus Removal to 1.0 mg/L – 9 MGD

GENERAL CONDITIONS	$204,000
SITE WORK	$454,000
CHEMICAL FEED SYSTEM	$764,770
CONSTRUCTION SUBTOTAL	$1,422,770
CONTINGENCY @ 20%	$284,554
CONSTRUCTION TOTAL	$1,707,324
PROJECT ENGINEERING (14%)	$239,025
TOTAL PROJECT COST	$1,946,349

Based on a dosage from the jar testing of 40 mg/L, the annual cost analysis for implementing chemical phosphorus removal to attain a 1.0 mg/L annual average phosphorus limit is shown below.

Table 6-6 | Chemical Cost Analysis for TP = 1.0 mg/L

DAF (MGD)	Phosphorus (Lbs./day)	Phosphorus (Lbs./ Year)	$FeCl_3$ (Gallons/ Year)	Estimated Annual Cost
5	250	91,250	164,250	$164,250
7	350	127,750	229,950	$229,950
9	450	164,250	295,650	$295,650

It is anticipated that the City of St. Charles may need to lower their effluent phosphorus concentrations even further to comply with future permit limits. The current NPDES permit requires the City to investigate the feasibility of achieving 0.5 mg/L. At this level, the City must consider the impacts of both soluble and particulate phosphorus. Therefore, the City would need to limit the TSS in the effluent to maintain compliance with 0.5 mg/L total phosphorus.

<u>Estimate Phosphorus Content in Effluent Suspended Solids</u>

BOD_5 to Biological Process from Primaries

9.0 MGD x 181 mg/L x 8.34 lb. / gal. = 13,586 lb. / day (WWTF influent)

13,586 lbs. BOD_5 / day x 0.68 = 9,238 lbs. BOD_5 / day

WAS Production = 0.80 x 9,238 lbs. BOD_5 / day = 7,390 lbs. /day

Solids Retention Time (SRT)

MLSS = 3.579 MGD x 3,500 mg/L x 8.34 = 104,471 lbs. under aeration

Sludge Age = 104,471 lbs. / 7,390 lbs. / day = 14.14 days

Solids within Biological Process

Phosphorus = 14.14 days * 450.4 lbs. PO_4 / day = 6,367 lbs. PO_4

6,367 lbs. PO_4 / 104,471 lbs. MLSS = 6.1% PO_4

If the City implements chemical phosphorus removal only, and the existing single-stage nitrification process continues to be utilized, it is anticipated that secondary clarifier effluent TSS would continue to be 5 mg/L. Based on the calculations above, it is anticipated that phosphorus represents 6.1% of the solids. The effluent TSS would contain 0.305 mg/L phosphorus without consideration of soluble reactive or non-reactive phosphorus.

According to the jar testing results, the total non-reactive phosphorus is estimated to be 0.1 mg/L. At a dosage of 60 mg/L $FeCl_3$, the remaining soluble reactive phosphorus is 0.15 mg/L.

Therefore, the design should target 0.15 mg/L of phosphorus within the TSS in order to maintain 0.4 mg/L TP and meet the 0.5 mg/L effluent TP limit.

Estimate Required Effluent TSS Concentration from Filters

0.15 mg/L PO_4 / (6.1% PO_4 to TSS) = 2.46 mg/L TSS

Therefore, the filters would need to reduce the effluent TSS from 5 mg/L to about 2 mg/L. The capital cost estimate for implementing chemical phosphorus removal and filtration to attain a 0.5 mg/L limit are shown below.

Table 6-7 | Cost Estimate for Chemical Phosphorus Removal to 0.5 mg/L – 9 MGD

GENERAL CONDITIONS	$933,500
SITE WORK	$1,423,560
FILTRATION SYSTEM	$4,184,020
CHEMICAL FEED SYSTEM	$764,770
CONSTRUCTION SUBTOTAL	$7,305,850
CONTINGENCY @ 20%	$1,461,170
CONSTRUCTION TOTAL	$8,767,020
PROJECT ENGINEERING (14%)	$1,227,383
TOTAL PROJECT COST	$9,994,403

The annual cost of chemical associated with this system would require a 50% increase to approximately 1,150 gallons per day (40 mg/L increased to 60 mg/L). For 30 day's storage, the City would need to have approximately 34,500 gallons onsite. The annual cost analysis for implementing chemical phosphorus removal to attain a 0.5 mg/L limit is shown below.

Table 6-8 | Chemical Cost Analysis for TP = 0.5 mg/L

DAF (MGD)	Phosphorus (Lbs./day)	Phosphorus (Lbs./ Year)	$FeCl_3$ (Gallons/ Year)	Estimated Annual Cost
5	250	91,250	246,375	$246,375
7	350	127,750	344,925	$344,925
9	450	164,250	443,475	$443,475

If the City is required to reduce their effluent phosphorus concentrations to 0.3 mg/L, and non-reactive phosphorus is estimated to be 0.1 mg/L, then nearly all of the phosphorus in the TSS would have to be physically removed.

Estimate Ratio of Reactive to Non-Reactive Phosphorus for 0.3 mg/L Effluent

Reactive Phosphorus = Total – Non-Reactive = 0.3 mg/L – 0.1 mg/L = 0.2 mg/L

It would therefore be necessary to implement an enhanced tertiary filtration system for polishing, as well as increase the volume of chemical added. Assuming TSS is 6% phosphorus, and the filtration system can reduce TSS effluent concentration to 1 mg/L, the estimated contribution from solids is roughly 0.06 mg/L. Therefore, the maximum reactive phosphorus concentration must be less than 0.14 mg/L.

There is insufficient data to accurately predict how much additional chemical is required to reduce reactive phosphorus to below 0.15 mg/L. Initial testing suggested that the dosage is between 80 and 100 mg/L. Therefore, it is recommended that another series of jar tests be performed if the City is required to meet this limit. Until more data is available, it may be assumed that approximately 90 mg/L will be required. This would increase the cost of chemical to about $665,000 per year at design conditions. This will bring the overall capital cost to approximately $16.1 Million.

Table 6-9 | Cost Estimate for Chemical Phosphorus Removal to 0.3 mg/L – 9 MGD

GENERAL CONDITIONS	$1,397,500
SITE WORK	$1,471,560
FILTRATION SYSTEM CONSTRUCTION	$8,135,030
CHEMICAL FEED SYSTEM	$764,770
CONSTRUCTION SUBTOTAL	$11,768,860
CONTINGENCY @ 20%	$2,353,772
CONSTRUCTION TOTAL	$14,122,632
PROJECT ENGINEERING (14%)	$1,977,168
TOTAL PROJECT COST	$16,099,800

Table 6-10 | Chemical Cost Analysis for TP = 0.3 mg/L

DAF (MGD)	Phosphorus (Lbs./day)	Phosphorus (Lbs./ Year)	$FeCl_3$ (Gallons/ Year)	Estimated Annual Cost
5	250	91,250	369,563	$369,563
7	350	127,750	517,388	$517,388
9	450	164,250	665,213	$665,213

Other recent NPDES permits have required the POTW to investigate the feasibility of meeting 0.1 mg/L total phosphorus. Jar testing completed during this study indicated the non-reactive component was approximately 0.1 mg/L. Prior to jar testing, it would be anticipated that the non-reactive phosphorus concentration would be in the range of 0.03-0.05 mg/L. It is recommended that further jar testing be completed to verify the value of this non-reactive component.

Assuming further jar testing confirms that the non-reactive TP is close to 0.1 mg/L, then an additional process must be incorporated to utilize either granular activated carbon, a chelating agent in concert with co-precipitation, ion exchange, or membrane / nano-filtration technology.

Granular activated carbon would have a modest capital cost, but a significant operational cost as the spent carbon would need to be replaced routinely. Chelating agents are often used in processes where conventional precipitation is not able to achieve the desired levels. A chelating agent would need to be identified and bench tested for further analysis. It is anticipated that the capital cost would not be a significant addition to what has been previously proposed, but that the operational costs could be significant. Cation exchange would require salt addition, which would result in the discharge of chlorides which is counterproductive as high chloride concentrations in receiving streams is already a contaminant of concern. Nano-filtration would require a high capital costs, as well as operational concerns with respect to power consumption. While the nano-filtration technology results in high purity water, the reject rate is between 10% and 20% of forward flow. Return of the reject to the WWTF may result in high concentrations or build-up of metal ions and phosphorus within the existing process. Alternative methods for disposal of the reject would likely include land application, which is not available at the Main WWTF. In summary, further investigation would be required. The first step in this process would be further testing to validate the non-reactive component in the waste stream.

If it is found that the non-reactive component is less than 0.05 mg/L, then it is possible that the solution provided under 0.3 mg/L would be able to achieve 0.1 mg/L.

6.4 ALTERNATIVES FOR SOLIDS HANDLING

For proper comparison of phosphorus removal alternatives, the impact on the solids handling operations of the Main WWTF must be considered. With chemical and biological phosphorus removal considerations, the volume of sludge produced will increase by 28% and 23%, respectively.

Table 6-11 | WAS Production Analysis for TP = 1.0 mg/L

DAF (MGD)	Type of Phosphorus Removal	WAS Production (lbs./day)*	Total
5	No Phosphorus Removal	4,405	100%
	Chem-P in Secondary	5,636	128%
	Biological P Removal	5,422	123%
7	No Phosphorus Removal	6,167	100%
	Chem-P in Secondary	7,891	128%
	Biological P Removal	7,590	123%
9	No Phosphorus Removal	7,929	100%
	Chem-P in Secondary	10,145	128%
	Biological P Removal	9,759	123%

**NOTE: These values assume effluent TP at 1 mg/L and average monthly maximum influent BOD_5*

The increased solids loading would have a minimal effect on operations. Currently, the City has capacity for almost 4 days of WAS storage at design conditions. The increased WAS loading from the chemical phosphorus removal would lower this capacity to about 3.0 days, and the BPR process would reduce capacity to 3.2 days.

The additional loading would also increase the hours of operation of the gravity belt thickener (5.6 hours per day at design conditions). The GBT operations would increase to 7.1 hours per day for the chemical phosphorus removal process and to 6.8 hours per day for the BPR process. As the facility approaches design flows, the City should consider installation of the second GBT (provisions for this equipment were made in the 2012 Main and Sludge Handling Building Improvements).

The subsequent solids handling processes would also be marginally effected (digester loading would increase, digested sludge storage would decrease, required hours of centrifugal dewatering operations would increase, etc.). The anaerobic digesters currently provide approximately 24 days detention time. The additional sludge produced would reduce the detention time to 22 days, which is more than sufficient for VSS reduction.

The chemical vs. biological phosphorus removal alternatives produce similar quantities of sludge. However, the non-economic impacts to be considered include potential for struvite formation within the digesters and the ability to dewater the digested sludge.

The City has experienced minor struvite issues in the past. Increased available phosphorus may result in the formation of additional struvite. The existing biological process removes roughly 50% of the influent phosphorus. The proposed biological phosphorus removal process would increase this to 85%, or 170% of the current conditions. The previous estimates for biological phosphorus removal included a chemical buffering system to address struvite formation.

Implementation of chemical phosphorus removal will create a 10% increase in sludge production. Implementation of BPR will create an 8% increase in sludge production, and sludge from BPR will be more difficult to dewater. At this time, it is estimated that the dewatered cake solids will decrease from 22% to 20% with the implementation of BPR. Overall, BPR is estimated to increase the volume of sludge by 19%. Therefore, sludge production from BPR is estimated to be 9% greater than from chemical phosphorus removal. Based on a disposal cost of $30/wet ton, it is estimated that at 7 MGD the increased disposal cost of BPR to chemical phosphorus removal to be approximately $29,000 per year.

Table 6-12 | Sludge Disposal Probable Cost Analysis – 7 MGD

	Sludge (lbs./day)	Sludge (dry tons/ year)	Sludge (wet tons/ year)	Estimated Annual Cost
Current	5,498	1,003	4,561	$136,828
Chem-P	6,048	1,104	5,017	$150,511
Bio-P	6,543	1,194	5,970	$179,108

The alternatives for chemical and biological phosphorus removal at three possible effluent TP limits are compared below. The "increased annual operational costs" are in addition to the City's current budget for sludge disposal and chemical material. These values are therefore not representative of the total cost of operations for the Main WWTF. These costs were calculated over a 20-year period to project the net present value with an average influent of 7 MGD.

Table 6-13 | Probable Cost Analysis – 20-Year Period – 7 MGD

EFFLUENT TP	PROCESS	PRESENT VALUE OF CAPITAL COST	INCREASED ANNUAL OPERATIONAL COST	PRESENT VALUE OF OPERATIONAL COST	NET PRESENT VALUE
1.0 mg/L	CHEM-P	$1,946,349	$243,633	$4,872,656	$6,819,005
	BIO-P	$7,370,208	$42,280	$845,597	$8,215,805
0.5 mg/L	CHEM-P	$9,994,403	$358,608	$7,172,156	$17,166,559
	BIO-P	$15,418,261	$157,255	$3,145,097	$18,563,358
0.3 mg/L	CHEM-P	$16,099,800	$531,070	$10,621,406	$26,721,206
	BIO-P	$21,523,659	$329,717	$6,594,347	$28,118,006

The high operational cost of chemical phosphorus removal does not outweigh the high capital cost of biological phosphorus removal in this comparison. However, the estimated inflation was based on the Construction Cost Index (CCI), which has been roughly 3% annually over the last 20 years. With increased chemical demand due to enforcement of effluent phosphorus limits on POTWs, it is possible that the commodity value of ferric chloride may exceed the CCI. If the chemical cost increases by 6% per year, for example, the BPR alternative becomes marginally more economical; chemical phosphorus removal is projected to cost roughly $220,000 more than BPR over a 20-year period. If the chemical cost increases by 5.65% per year, the alternatives have equal 20-year cost projections.

6.5 RECOMMENDATIONS

The following matrix was developed to determine the best alternative for the City of St. Charles Main WWTF. Economic and non-economic factors were listed and weighted. The alternative that was in the best interest of the City for each factor was awarded those points, and a total score was tallied. In some instances, the factor was found to be approximately the same for both alternatives. In these cases, points were awarded to both alternatives.

Table 6-14 | Phosphorus Removal Decision Matrix

Description	Weight Factor (1-18)	Bio-P	Bio-P Score	Chem-P	Chem-P Score
Economic Factors					
Capital	17		0	1	17
O&M	16	1	16		0
Life Cycle Cost	18	1	18	1	18
Staffing Requirements	13	1	13	1	13
Long-Term Maintenance	12	1	12		0
20-Year Residual Value	15	1	15		0
Operational Simplicity	11		0	1	11
Inflationary Risk	14	1	14		0
Non-Economic Factors					
Quality of Sludge	2	1	2		0
Dewaterability	1		0	1	1
Effluent Quality	10	1	10		0
Future Nitrogen	9	1	9		0
Future Chlorides	8	1	8		0
Training Requirements	6		0	1	6
Disruption in Operations	7		0	1	7
Public Acceptance	3	1	3		0
Environmental	4	1	4		0
Innovation	5	1	5		0
Total Score			**129**		**73**

The City has elected to pursue biological phosphorus removal to comply with its annual average NPDES permit limit of 1.0 mg/L. Implementation of Bio-P will require a capital investment of approximately $7.4 Million. The City of St. Charles intends on funding the project through the Illinois SRF and to service the debt through user fees. During evaluation of the existing infrastructure, the City identified rehabilitation of the anaerobic digesters as a top priority. Therefore, it is recommended that the City pursue financing for implementation of phosphorus removal and anaerobic digester rehabilitation. It is also recommended that the improvements be designed, permitted and implemented as one construction project. The NPDES permit requires that the construction of the phosphorus removal improvements be completed by June of 2018. The following schedule is intended to meet those requirements.

Table 6-15 | Implementation Schedule

Description of Milestone	Date
Interim Report on Phosphorus Removal Feasibility Report	Completed
Phosphorus Removal Feasibility Report (1.0 and 0.5 mg/L) Submittal	Pending
Begin Design of Improvements	September-15
Plans and Specifications Submitted	March-16
IEPA Loan Application Submittal	March-16
Advertise for Bid	July-16
IEPA Loan Agreement Approval	September-16
Start Construction	October-16
Complete Construction	June-18

SECTION 7

IMPLEMENTATION PLAN

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7. IMPLEMENTATION PLAN

7.1 SELECTED ALTERNATIVE

The alternatives for phosphorus removal at the Main WWTF were analyzed in Section 6 of this report. After careful consideration of these alternatives, the City has elected to pursue biological phosphorus removal to comply with its annual average NPDES permit limit of 1.0 mg/L. The City of St. Charles intends on funding the project through the Water Pollution Control Loan Program administered by the Illinois EPA with the intention of servicing the debt through user fees.

It is recommended that the City pursue IEPA Low-Interest Loan financing for implementation of the recommended improvements for both phosphorus removal and for the anaerobic digester rehabilitation. It is also recommended that the improvements be designed, permitted and implemented as one construction project. The overall project will require that most unit processes in these two systems be shut down and brought back into operation in phases to maintain compliance with effluent limits.

For the biological process, each of the basins will need to be isolated, drained, cleaned and outfitted with new equipment prior to being brought back into operation. The internal recycle pump stations and primary sludge fermenter described in Section 6 may be constructed concurrently with this work. The digestion system rehabilitation is proposed to be completed in parallel with the biological process improvements, and would therefore be done in one phase as opposed to the phased approach discussed in Section 5. The benefit would be the reduction of overall project time for the rehabilitation, as there would be no turnover time between phases for loan administration, project closeout, and possible change of contractors. However, the City may be required to operate without digestion capabilities for a number of months.

7.2 RECOMMENDATIONS

The City currently has an operations and maintenance budget of approximately $8.46 Million, which is shown in Table 7-1 to increase 3% annually. The lift station O&M costs in Table 7-1 have been updated to the recommended levels from Section 4 for the Main and West Facility Plan Updates. The O&M costs for the Main WWTF and West Side WRF consume the majority of the remaining budget. The costs of the CMOM program as recommended in Section 3 are also included, but are in addition to the other (existing) budget items. Therefore, a $3.3 Million O&M budget shortfall is currently projected for the next fiscal year. A more detailed user rate study will be required to assess how the City should cover this shortfall.

Table 7-1 | Operation and Maintenance for Phased Implementation Plan

Description	'15-'16	'16-'17	'17-'18	'18-'19*	'19-'20	'20-'21** to '29-'30
COLLECTION SYSTEM – CMOM	$1.90	$1.35	$1.35	$1.35	$1.35	$13.48
LIFT STATIONS – WEST	$0.07	$0.07	$0.07	$0.07	$0.07	$0.65
LIFT STATIONS – MAIN	$0.43	$0.43	$0.43	$0.43	$0.43	$4.30
WEST SIDE WRF O&M	$0.72	$0.75	$0.77	$0.79	$0.81	$9.66
MAIN WWTF O&M	$7.24	$7.46	$7.68	$7.96	$8.20	$96.77
TOTAL PROPOSED O&M	$11.76	$11.45	$11.70	$11.99	$12.26	$124.86
CURRENT O&M BUDGET (3% increase)	$8.46	$8.72	$8.98	$9.25	$9.52	$112.16

Projected costs are in millions of dollars
* NOTE: In 2018, the operational cost increase for biological phosphorus removal at the Main WWTF will increase as projected in Section 6 of the Main Facility Plan Update.
** NOTE: In 2021, the operational cost increase for biological phosphorus removal at the West Side WRF will increase as projected in Section 6 of the West Side Facility Plan Update.

The complete list of all capital improvements recommended in this report, as well as the recommended capital improvements contained in the West Side WRF Facility Plan Update, is provided below.

Table 7-2 | Capital Improvements Summary

RIVERSIDE LIFT STATION REPLACEMENT	$5,742,112
7TH & DIVISION LIFT STATION REPLACEMENT	$597,200
COUNTRY CLUB LIFT STATION REHABILITATION	$637,625
WILD ROSE LIFT STATION REPLACEMENT	$620,388
WEST SIDE WRF EXPANSION - PHASE IIIA	$8,605,278
WEST SIDE WRF EXPANSION - PHASE IIIB	$3,607,067
PRIMARY CLARIFIER REHABILITATION	$558,532
ANAEROBIC DIGESTER REHABILITATION	$7,960,605
UV DISINFECTION REHABILITATION	$2,576,218
EXCESS FLOW FILTRATION	$8,048,053
BIOLOGICAL PHOSPHORUS REMOVAL	$7,370,208
TOTAL CAPITAL IMPROVEMENTS	$46,323,286

The City currently has a capital improvements budget of approximately $1.73 Million. This cost represents the existing debt service on previously completed improvements that were funded through the Illinois SRF, and are labeled in Table 7-3 as “Existing Debt Service”. The additional costs of the recommended capital improvements recommended in Sections 3 – 6 are included in Table 7-2 as “Proposed Debt Service”. These projects were discussed with City staff to gain concurrence on the desired start and completion dates for each recommended improvement. A detailed user rate study is recommended to assess how the City should cover the recommended capital improvements.

Table 7-3 | Debt Service for Capital Improvements – Phased Implementation Plan

Description	'15-'16	'16-'17*	'17-'18	'18-'19	'19-'20	'20-'21** to '29-'30
EXISTING DEBT SERVICE						
WEST SIDE WRF PH. II EXPANSION	**$0.47**	**$0.47**	**$0.47**	**$0.47**	**$0.47**	**$0.47**
2002 NITRIFICATION IMPROVEMENTS	**$0.65**	**$0.65**	**$0.65**	**$0.65**	**$0.65**	**$3.26**
EAST SIDE & RIVERSIDE L.S. REHAB.	**$0.10**	**$0.10**	**$0.10**	**$0.10**	**$0.10**	**$0.49**
2012 MAIN AND S.H.B.	**$0.51**	**$0.61**	**$0.61**	**$0.61**	**$0.61**	**$3.07**
PROPOSED DEBT SERVICE						
COLL. SYSTEM – REPLACEMENT	**$1.40**	**$1.40**	**$1.40**	**$1.40**	**$1.40**	**$7.02**
RIVERSIDE LIFT STATION				**$0.19**	**$0.38**	**$1.88**
7TH & DIVISION LIFT STATION					**$0.60**	
COUNTRY CLUB LIFT STATION				**$0.64**		
WILD ROSE LIFT STATION						**$0.62**
WEST SIDE WRF PH. IIIA EXPANSION						**$0.56**
WEST SIDE WRF PH. IIIB EXPANSION						**$0.12**
PRIMARY CLARIFIER REHAB.		**$0.56**				
ANAEROBIC DIGESTER REHAB.		**$0.00**		**$0.26**	**$0.52**	**$2.62**
UV DISINFECTION REHAB.						**$0.17**
EXCESS FLOW FILTRATION						
PHOSPHORUS REMOVAL - BIO-P		**$0.00**		**$0.24**	**$0.48**	**$2.42**
TOTAL DEBT SERVICE	**$3.13**	**$4.73**	**$3.23**	**$4.56**	**$5.21**	**$46.50**

Projected costs are in millions of dollars
* NOTE: In 2016, the design engineering is projected to occur for the biological phosphorus removal and anaerobic digester rehabilitation project. This will require a projected cash flow of approximately $950,000 this year. The project may be funded with a SRF loan, which will include a repayment to the City for this cost. This repayment is projected to occur within the same fiscal year, resulting in a net cash flow of zero.
** NOTE: In 2021, the design engineering is projected to occur for the UV disinfection rehabilitation project. This will require a projected cash flow of approximately $160,000 this year. The project may be funded with a SRF loan, which will include a repayment to the City for this cost. This repayment is projected to occur within the same fiscal year, resulting in a net cash flow of zero.

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SECTION 8
ANTI-DEGRADATION & ENVIRONMENTAL IMPACTS

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8. ANTI-DEGRADATION AND ENVIRONMENTAL IMPACT ANALYSIS

8.1 GENERAL DISCUSSION

The City of St. Charles is responsible for providing sanitary service and treatment for the communities within the Facility Planning Area (FPA). Sections 1 through 6 describe the basins of the FPA that are tributary to the Main WWTF (a.k.a. the Main Service Area), the anticipated development, collection system, and treatment facility improvement needs in detail. As the designated management agency, the City is also responsible for meeting the long-range goals of the Clean Water Act and to minimize the environmental impacts of pollution from the sanitary waste generated within the Facility Planning Area and specifically within the Main Service Area.

The City has and continues to work with each of the affected communities by providing sanitary service, encouraging responsible development practices, and working with state and local agencies to protect the Fox River from pollutants.

In addition to actively pursuing solutions to the communities wastewater collection needs, the City has invested in upgrading the Main WWTF with newer technologies to meet the needs of the Fox River Watershed. Some of the improvements to protect the environment incorporated into the recent projects include:

- Installation of mechanical fine screens in the tributary lift stations
- Rehabilitation of the headworks
- Replacement of the sludge handling facilities

As shown in Section 5, the performance of the Main WWTF has been outstanding. The BOD_5, suspended solids, and ammonia loadings are continuously well below the NPDES Permit Limits.

The City is committed to upgrading the wastewater treatment facility in a manner that will be a benefit to both the communities served and the ecosystem surrounding the Fox River.

8.2 ENVIRONMENTAL AREAS OF CONCERN

Areas of environmental concern include not only the Fox River, but the wetlands and nature preserves within the area. The wildlife habitat and open space represent a significant portion of the Facility Planning Area. The comprehensive plan prepared by the City within the FPA recognizes the importance of preserving open space and incorporating responsible development. Ordinances and development practices to minimize urban run-off from impacting the environment is encouraged.

The most significant concern for the Main WWTF includes the quality of the final effluent. The facility's current effluent quality is exceptional. However, concerns over impacts on the surrounding environment including wetlands, wildlife habitat, and endangered species must be considered.

8.2.1. Water Quality Concerns

The Clean Water Act was established to protect and revive the lakes, rivers, and streams throughout the United States. Restoring their quality is crucial in maintaining a healthy environment and ensuring the sustainability of these waters for all to use and enjoy.

Title 35, Section 302 of the Illinois Administrative Code establishes the method for determining, implementing, and regulating Water Quality Standards. Section 302.105 – Antidegradation has been added to protect existing uses of all water, maintain the quality of waters, and prevent unnecessary deterioration of the waterways.

The Clean Water Act also established the NPDES Permitting program managed by the individual state agencies. The program establishes effluent limits that the Publically Owned Treatment Works (POTWs) must meet. The Main WWTF has consistently been in accordance with its NPDES permit limits.

There are two methods of determining effluent limits. The first is Water Quality Based Effluent Limits (WQBEL's). WQBEL's have historically been used throughout Illinois to establish the NPDES Permit Limits for POTW Discharges.

The second method is to study a particular body of water and establish Total Maximum Daily Loads (TMDL's) based on the ecosystem's ability to receive pollutants without having an adverse effect on the streams ability to support its designated uses. By taking a watershed approach, a TMDL considers all potential sources of pollutants, both point and non-point sources. It also takes into account a margin of safety, which reflects scientific uncertainty and future growth. The effects of seasonal variation are also included.

In short, a TMDL is calculated using the following equation:

$$TMDL = WLA + MOS + SV$$

Such that:

WLA = Waste Load Allocation (point sources)
LA = Load Allocation (non-point sources)
MOS = Margin of Safety
SV = Seasonal Variation

Section 303(d) of the Clean Water Act requires each state to prepare a list of waters of the state that are considered to be impaired for their intended uses. In 2014, the Illinois EPA issued a revised Integrated Water Quality report and Section 303(d) List. Portions of the Fox River have been placed on this list.

The City's Main WWTF discharges to segment DT-58, which includes 3.76 miles of the Fox River. This segment has been identified as impaired but at a low priority. The assessment was based on site-specific data and concluded that segment DT-58 was not supporting aquatic life, fish consumption, or primary contact recreation. A summary of these impairments and their causes are shown below:

Table 8-1 | Excerpt from Illinois' 2014 303(d) List and Prioritization: IL_DT-58

Order	Priority	Hydrologic Unit Code	Water Name	Water Size	Designated Use	Cause
1481	Low	0712000701	Fox River	3.76	Aquatic Life	Dissolved Oxygen
1482	Low	0712000701	Fox River	3.76	Fish Consumption	Mercury, PCBs
1483	Low	0712000701	Fox River	3.76	Primary Contact Recreation	Fecal Coliform

The Illinois EPA defines the potential causes and sources of impairment for given water bodies. Specific assessment information was provided by the IEPA for segment DT-58 in 2014, and the causes of these impairments are listed as codes which are summarized below:

Table 8-2 | Excerpt 1 from Specific Assessment Info. for Streams, 2014: DT-58

Cause ID	Description
84	Alteration in stream-side or littoral vegetative covers
274	Mercury
319	Other flow regime alterations
322	Oxygen, Dissolved
348	Polychlorinated Biphenyls (PCBs)
400	Fecal Coliform

The sources of the impairments were also listed as codes in the 2014 specific assessment, which are summarized below:

Table 8-3 | Excerpt 2 from Specific Assessment Info. for Streams, 2014: DT-58

Source ID	Potential Source Description	Potential Source Guidelines for Identification*
10	**Atmospheric Deposition – Toxics**	Atmospheric deposition of nutrients, minerals, etc. based upon actual observation and/or other existing data.
58	**Impacts from Hydrostructure Flow Regulation / Modification**	Alteration of normal flow regimes (e.g., dams, channelization, impervious surfaces, water withdrawal) based upon actual observation and/or other existing data.
125	**Streambank Modifications / Destabilization**	Shoreline modification/destabilization activities (e.g., bank erosion, rip rap, loss of habitat) based upon actual observation and/or other existing data.
140	**Source Unknown**	No identifiable source based upon available information
177	**Urban Runoff / Storm Sewers**	Urban and storm sewer runoff based upon actual observation and/or other existing data

**NOTE: Excerpt from Integrated Water Quality Report and Section 303(d) List – Volume I: Surface Water – 2014*

Interestingly, neither "municipal point source discharges" nor "on-site treatment systems" were listed as sources of impairment. As such, it can be concluded that the City's Main WWTF does not contribute any substantial harmful pollutants to segment DT-58 of the Fox River. However, it is still important to address any at-risk species in the vicinity that could be affected by future pollutant loadings.

8.2.2. Threatened and Endangered Species

The Illinois Department of Natural Resources offers an Ecological Compliance Assessment Tool (EcoCAT) that analyzes a given area and provides a list of protected resources in the vicinity of the project location. An EcoCAT was conducted for the areas surrounding the treatment facility and determined that the Illinois Natural Heritage Database contains no record of State-listed threatened or endangered species, Illinois Natural Area Inventory sites, dedicated Illinois Nature Preserves, or registered Land and Water Reserves in the vicinity of the project location. This report is included as Appendix E.

8.2.3. Input from Stakeholders

The USEPA, along with the IEPA, is currently considering alternatives to limit nutrient concentrations in an effort to reduce or eliminate local water quality impairments as well as hypoxia in the Gulf of Mexico. As discussed in Section 6, the Illinois EPA is focused on statewide nutrient removal criteria for wastewater treatment facilities. The Illinois EPA, along with the Fox River Study Group and other stakeholders, are developing solutions to address the impairments found along the Fox River.

For many years, the IEPA has enforced nutrient removal criteria for treatment facilities seeking to expand their hydraulic capacity. The IEPA revised the water quality standards in Illinois which resulted in lower treatment plant effluent limits for ammonia-nitrogen and phosphorus at Illinois POTWs. The City received a new NPDES permit in December of 2014 which included a 1.0 mg/L monthly average phosphorus limit, as well as lower ammonia nitrogen levels. This new permit is included as Appendix A.

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SECTION 9

SUMMARY AND RECOMMENDATIONS

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9. SUMMARY AND RECOMMENDATIONS

9.1 GENERAL DISCUSSION

The background, contents and purpose of this Facility Plan are discussed in Section 1. The City owns and operates a sanitary sewer collection system and two wastewater treatment facilities: the Main WWTF and the West Side WRF. The collection system tributary to the Main WWTF consists of approximately 152 miles of sanitary sewers, 5 miles of force main and 13 lift stations. The Main WWTF is located at the Public Works Facility, 1405 S. 7th Avenue on the eastern shore of the Fox River, approximately nine-tenths of a mile south of the Illinois Route 64 Bridge. The St. Charles Facility Planning Area (FPA) is comprised of approximately 10,340 acres, of which 8,317 acres is tributary to the Main WWTF.

The Main WWTF plant has a design average treatment capacity of 9.0 million gallons per day (MGD). The facility generally serves the community's wastewater needs east of Randall Road and discharges to the Fox River.

The City's National Pollutant Discharge Elimination System (NPDES) Permit for the Main WWTF (Permit No. IL0022705), as administered by the Illinois Environmental Protection Agency (IEPA), was reissued on December 1st, 2014. The new permit incorporates special conditions, including the monitoring of effluent phosphorus and nitrogen and an annual average concentration limit of 1 mg/L for effluent phosphorus. The NPDES permit is included as Appendix A.

Recognizing the need for improvements to meet the new permit limits, the Illinois EPA incorporated a compliance schedule into the NPDES Permit as a Special Condition. The compliance schedule established a timeline for the City of St. Charles to plan, design and construct the necessary improvements. The compliance schedule submittal requirements were as follows:

Table 9-1 | NPDES Permit Compliance Schedule for the MWWTF

Description of Milestone	Date
Interim Report on Phosphorus Removal Feasibility Report	June-15
Phosphorus Removal Feasibility Report (1.0 and 0.5 mg/L) Submittal	December-15
Progress Report on Phosphorus Reductions / Implementation Plan	June-16
Progress Report on Recommendations of Implementation Plan	December-16
Plans and Specifications Submitted	June-17
Progress Report on Construction	December-17
Complete Construction	June-18
Progress Report on Optimizing Treatment System	December-18
Achieve Annual TP Concentration and Loading Effluent Limits	June-19

9.2 POPULATION EQUIVALENTS AND WASTEWATER FLOWS

Section 2 of this report included an evaluation of the current and projected population equivalents, wastewater flows and pollutant loadings. The City of St. Charles has grown from a community of 17,492 in 1980 to 27,910 people in 2001 to 32,974 people in 2010, of which 29,941 live in the Main WWTF's service area. The City Council has not approved any new developments within this service area for construction. The remaining undeveloped properties within the St. Charles FPA were been assigned a land use and density.

In 2010, the U.S. Census Bureau estimated that the City of St. Charles served a total residential population of 32,974. The residential water usage based on billing records was 2,221,446 gallons per day (gpd). This residential usage consumed by an estimated 32,974 residents equates to 67.37 gallons per capita per day (gcd).

During 2011 and 2013, the City of St. Charles billed users an average of 3.35 MGD for water use, while the wastewater treatment facility received an average flow of 4.66 MGD (data from 2012 was disregarded due to drought conditions). The current population equivalents were estimated by breaking down water billing by classifications

Table 9-2 | Current Population, Water Demands and Wastewater Flows

	Residential	Non-Residential	Total
Number of Customers	9,772	1,167	10,939
Population Equivalents	29,924 PE	19,841 PE	49,765 PE
Water Usage Billed	2.02 MGD	1.34 MGD	3.35 MGD
Water Usage / PE	67.37 gcd	67.37 gcd	67.37 gcd
Wastewater Received	2.80 MGD	1.86 MGD	4.66 MGD
Wastewater / PE	93.58 gcd	93.58 gcd	93.58 gcd

The future population projection, which is the ultimate buildout of properties within the FPA, was developed by assigning PE values to the planned development and remaining open lands in accordance with the Land Use Plan.

Future Population Equivalent

Total Current PE	49,765	PE
Additional PE at Build-Out of Service Area	6,489	PE
Total Future PE	56,254	PE

Projected 2030 Population Equivalent for the FPA is 56,254 PE. It should be noted that population equivalent resulting from the ultimate buildout will not exceed the present IEPA rated population equivalent of the Main WWTF which is 90,000 PE.

9.3 COLLECTION SYSTEM

An assessment of the City's collection system was presented in Section 3. The wastewater collection system includes two service areas generally divided by Randall Road. The sanitary sewer system east of Randall Road is tributary to the Main WWTF. The sewers within this collection system are of varying age and condition. As with many older collection systems, infiltration and inflow is a concern. Recognizing the importance of removing infiltration and inflow from the collection system, the City of St. Charles has developed a rigorous maintenance program including flow monitoring, root cutting, grouting, sewer lining and other rehabilitation and replacement of the collection system. The City has budgeted $4.24 million for sanitary sewer projects within the five-year capital improvements program.

The City of St. Charles' Finance Department maintains its GASB 34 Report, however, the collection system is not broken out by treatment facility. Therefore the actual value of this asset for the Main Service Area is not known. It has been estimated that the City currently maintains 172 miles of sanitary sewer mains (gravity and force main), as well as roughly 4,040 sanitary manholes in the Main and West Service Areas.

Using estimated replacement unit costs for sanitary sewer pipes, sanitary manholes and lift stations, the City owns and maintains a $220 million dollar collection system. Assuming 10% for contingency and 15% for design and administration, the replacement of the entire collection system is estimated to cost approximately $275 million. However, the majority of the collection system is not in need of replacement. The service life of a collection system is approximately 75 years, and this life can be extended by approximately 25 years with ongoing maintenance and rehabilitation. Based on straight-line depreciation over this 100-year service life, the City should be reinvesting about $2,751,000 annually toward sanitary sewer collection system rehabilitation.

Approximately 20% of the collection system is already beyond its initial 75-year service life, and may be considered fully depreciated and in need of replacement. The City should be reinvesting $1,403,000 annually toward the replacement of sewers that were installed before 1941 (as a portion of the annual reinvestment). The remaining $1,348,000 should be put towards the annual costs of the CMOM program. There are several initial costs involved with starting up a program of this magnitude, which are shown to be included in the 2015/2016 fiscal year budget. This initial cost is estimated to be roughly $550,000.

In order to sustain the long-term viability of the sewer utility, the City's sewer rehabilitation budget should be raised to the aforementioned level if it has not been already. If the money is not used, it should be placed into a replacement account for future use.

9.4 LIFT STATIONS

An assessment of the City's lift stations was presented in Section 4. The City of St. Charles' Main Service Area includes thirteen lift stations, two of which are directly tributary to the headworks at the Main WWTF. The lift stations vary in age and condition, however most were constructed between 1987 and 1997 as the City developed further north and east. The two main lift stations are Riverside Lift Station and East Side Lift Station.

Table 9-3 | Lift Station Asset Value

Lift Station	Equipment	Structure	Force Main	Totals
Riverside	$1,750,000	$2,000,000	$1,280,000	$5,030,000
East Side	$1,030,000	$1,500,000	$96,000	$2,626,000
7th & Division	$200,000	$145,000	$109,000	$454,000
Washington Ave.	$50,000	$50,000	$73,000	$173,000
Country Club	$200,000	$155,000	$129,000	$484,000
Pheasant Run Trails	$210,000	$185,000	$292,000	$687,000
Royal Fox #2	$220,000	$185,000	$498,000	$903,000
Royal Fox #1	$210,000	$165,000	$358,000	$733,000
Woods of Fox Glen	$210,000	$185,000	$566,000	$961,000
Kingswood	$210,000	$185,000	$197,000	$592,000
Wild Rose	$200,000	$160,000	$14,000	$374,000
Red Gate	$210,000	$185,000	$311,000	$706,000
Oak Crest	$200,000	$155,000	$74,000	$429,000
Totals	$4,900,000	$5,255,000	$3,997,000	$14,152,000
Design Life, Years	20	50	50	
Annual Replacement	$245,000	$105,100	$79,940	**$430,040**

It should be noted that the above figures do not include the engineering and contingencies that would be involved in a rehabilitation or replacement project. The value of the City's lift station and force main assets is approximately $14,152,000. Based on a straight-line depreciation over the design life of the equipment, structures and force mains, the City should be reinvesting around $430,000 annually toward maintaining and replacing these assets within the Main Service Area.

Operational staff has indicated that most of the recommended improvements could be accomplished utilizing in-house resources. The more significant improvements have been broken into capital projects and recommended budgets have been provided. These projects should be incorporated into the City's Capital Improvements Program.

Table 9-4 | Lift Station Capital Improvements Summary

RIVERSIDE LIFT STATION REPLACEMENT	$5,742,112
7TH & DIVISION LIFT STATION REPLACEMENT	$597,200
COUNTRY CLUB LIFT STATION REHABILITATION	$637,625
WILD ROSE LIFT STATION REPLACEMENT	$620,388
TOTAL LIFT STATION CAPITAL IMPROVEMENTS	$7,597,325

9.5 EXISTING WASTEWATER FACILITY

The existing Main WWTF was discussed in Section 5. The City of St. Charles' original wastewater treatment facility was located along the banks of the Fox River near the Riverside Lift Station. In the early 1930's, a new plant was constructed up the hill on what is now the Main WWTF site. Over the following 80 years, the facility was expanded and upgraded numerous times to address capacity and regulatory concerns. Therefore, the existing Main WWTF infrastructure is of varying age and condition. The City has completed a brief audit of each unit process, its capacity, age and condition and developed a series of recommended improvements.

Table 9-5 | Main WWTF Capital Improvements Summary

PRIMARY CLARIFIER REHABILITATION	$558,532
ANAEROBIC DIGESTER REHABILITATION	$7,960,605
UV DISINFECTION REHABILITATION	$2,576,218
EXCESS FLOW FILTRATION	$8,048,053
TOTAL MAIN WWTF CAPITAL IMPROVEMENTS	$19,143,408

9.6 WASTEWATER FACILITY UPGRADE PLAN

The City of St. Charles received a final NPDES permit in December of 2014. The special conditions included a 1 mg/L interim phosphorus standard, requires the POTWs to perform a study and determine the cost for compliance of phosphorus removal for a 1 mg/L standard as well as a 0.5 mg/L standard. The projected costs and alternatives analysis for biological and chemical phosphorus removal are discussed in Section 6.

The alternatives for chemical and biological phosphorus removal at three possible effluent TP limits are compared below. The "increased annual operational costs" are in addition to the City's current budget for sludge disposal and chemical material. These values are therefore not representative of the total cost of operations for the Main WWTF. These costs were calculated over a 20-year period to project the net present value with an average influent of 7 MGD.

Table 9-6 | Probable Cost Analysis – 20-Year Period – 7 MGD

EFFLUENT TP	PROCESS	PRESENT VALUE OF CAPITAL COST	INCREASED ANNUAL OPERATIONAL COST	PRESENT VALUE OF OPERATIONAL COST	NET PRESENT VALUE
1.0 mg/L	CHEM-P	$1,946,349	$243,633	$4,872,656	$6,819,005
	BIO-P	$7,370,208	$42,280	$845,597	$8,215,805
0.5 mg/L	CHEM-P	$9,994,403	$358,608	$7,172,156	$17,166,559
	BIO-P	$15,418,261	$157,255	$3,145,097	$18,563,358
0.3 mg/L	CHEM-P	$16,099,800	$531,070	$10,621,406	$26,721,206
	BIO-P	$21,523,659	$329,717	$6,594,347	$28,118,006

The City has elected to pursue biological phosphorus removal to comply with its NPDES permit limit of 1.0 mg/L. Implementation of Bio-P will require a capital investment of approximately $7.4 Million. The City of St. Charles intends on funding the project through the Illinois SRF and to service the debt through user fees. During evaluation of the existing infrastructure, the City identified rehabilitation of the anaerobic digesters as a top priority. Therefore, it is recommended that the City pursue financing for implementation of phosphorus removal and anaerobic digester rehabilitation. It is also recommended that the improvements be designed, permitted and implemented as one construction project. The NPDES permit requires that the construction of the phosphorus removal improvements be completed by June of 2018. The following schedule is intended to meet those requirements.

Table 9-7 | Implementation Schedule

Description of Milestone	Date
Interim Report on Phosphorus Removal Feasibility Report	Completed
Phosphorus Removal Feasibility Report (1.0 and 0.5 mg/L) Submittal	Pending
Begin Design of Improvements	September-15
Plans and Specifications Submitted	March-16
IEPA Loan Application Submittal	March-16
Advertise for Bid	July-16
IEPA Loan Agreement Approval	September-16
Start Construction	October-16
Complete Construction	June-18

9.7 IMPLEMENTATION PLAN

The Main WWTF's NPDES permit requires that the City implement a CMOM, and upgrade the existing facility to comply with effluent phosphorus limits. Recommendations within Section 3 included budgets for sanitary sewer replacement and the CMOM program. The lift station O&M costs were identified in Section 4 for the Main and West Facility Plan Updates. The O&M costs for the Main WWTF and West Side WRF remain unchanged until after implementation of phosphorus removal and capacity expansion upgrades, respectively. The current need for O&M of the City's wastewater infrastructure is estimated to be $11.76 Million. The City currently has an O&M budget of approximately $8.46 Million.

Table 9-8 | Operation and Maintenance for Phased Implementation Plan

Description	'15-'16	'16-'17	'17-'18	'18-'19*	'19-'20	'20-'21** to '29-'30
COLLECTION SYSTEM – CMOM	$1.90	$1.35	$1.35	$1.35	$1.35	$13.48
LIFT STATIONS – WEST	$0.07	$0.07	$0.07	$0.07	$0.07	$0.65
LIFT STATIONS – MAIN	$0.43	$0.43	$0.43	$0.43	$0.43	$4.30
WEST SIDE WRF O&M	$0.72	$0.75	$0.77	$0.79	$0.81	$9.66
MAIN WWTF O&M	$7.24	$7.46	$7.68	$7.96	$8.20	$96.77
TOTAL PROPOSED O&M	$11.76	$11.45	$11.70	$11.99	$12.26	$124.86
CURRENT O&M BUDGET (3% increase)	$8.46	$8.72	$8.98	$9.25	$9.52	$112.16

Projected costs are in millions of dollars
* NOTE: In 2018, the operational cost increase for biological phosphorus removal at the Main WWTF will increase as projected in Section 6 of the Main Facility Plan Update.
** NOTE: In 2021, the operational cost increase for biological phosphorus removal at the West Side WRF will increase as projected in Section 6 of the West Side Facility Plan Update.

The complete list of all capital improvements recommended in this report, as well as the recommended capital improvements contained in the West Side WRF Facility Plan Update, is provided below.

Table 9-9 | Capital Improvements Summary

RIVERSIDE LIFT STATION REPLACEMENT	$5,742,112
7TH & DIVISION LIFT STATION REPLACEMENT	$597,200
COUNTRY CLUB LIFT STATION REHABILITATION	$637,625
WILD ROSE LIFT STATION REPLACEMENT	$620,388
WEST SIDE WRF EXPANSION - PHASE IIIA	$8,605,278
WEST SIDE WRF EXPANSION - PHASE IIIB	$3,607,067
PRIMARY CLARIFIER REHABILITATION	$558,532
ANAEROBIC DIGESTER REHABILITATION	$7,960,605
UV DISINFECTION REHABILITATION	$2,576,218
EXCESS FLOW FILTRATION	$8,048,053
BIOLOGICAL PHOSPHORUS REMOVAL	$7,370,208
TOTAL CAPITAL IMPROVEMENTS	$46,323,286

The City's existing debt service equates to approximately $1.73 Million. The existing debt service and recommended capital improvements are included in Table 9-10. City staff determined the priority and schedule for each capital project. It is recommended that the City conduct a study to address user rates and the revenue required to support operations and maintenance, as well as the capital improvements program.

Table 9-10 | Debt Service for Capital Improvements – Phased Implementation Plan

Description	'15-'16	'16-'17*	'17-'18	'18-'19	'19-'20	'20-'21** to '29-'30
EXISTING DEBT SERVICE						
WEST SIDE WRF PH. II EXPANSION	$0.47	$0.47	$0.47	$0.47	$0.47	$0.47
2002 NITRIFICATION IMPROVEMENTS	$0.65	$0.65	$0.65	$0.65	$0.65	$3.26
EAST SIDE & RIVERSIDE L.S. REHAB.	$0.10	$0.10	$0.10	$0.10	$0.10	$0.49
2012 MAIN AND S.H.B.	$0.51	$0.61	$0.61	$0.61	$0.61	$3.07
PROPOSED DEBT SERVICE						
COLL. SYSTEM – REPLACEMENT	$1.40	$1.40	$1.40	$1.40	$1.40	$7.02
RIVERSIDE LIFT STATION				$0.19	$0.38	$1.88
7TH & DIVISION LIFT STATION					$0.60	
COUNTRY CLUB LIFT STATION				$0.64		
WILD ROSE LIFT STATION						$0.62
WEST SIDE WRF PH. IIIA EXPANSION						$0.56
WEST SIDE WRF PH. IIIB EXPANSION						$0.12
PRIMARY CLARIFIER REHAB.		$0.56				
ANAEROBIC DIGESTER REHAB.		$0.00		$0.26	$0.52	$2.62
UV DISINFECTION REHAB.						$0.17
EXCESS FLOW FILTRATION						
PHOSPHORUS REMOVAL - BIO-P		$0.00		$0.24	$0.48	$2.42
TOTAL DEBT SERVICE	$3.13	$4.73	$3.23	$4.56	$5.21	$46.50

Projected costs are in millions of dollars

* NOTE: In 2016, the design engineering is projected to occur for the biological phosphorus removal and anaerobic digester rehabilitation project. This will require a projected cash flow of approximately $950,000 this year. The project may be funded with a SRF loan, which will include a repayment to the City for this cost. This repayment is projected to occur within the same fiscal year, resulting in a net cash flow of zero.

** NOTE: In 2021, the design engineering is projected to occur for the UV disinfection rehabilitation project. This will require a projected cash flow of approximately $160,000 this year. The project may be funded with a SRF loan, which will include a repayment to the City for this cost. This repayment is projected to occur within the same fiscal year, resulting in a net cash flow of zero.

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APPENDIX A
STC MAIN PLANT NPDES PERMIT

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NPDES Permit No. IL0022705

Illinois Environmental Protection Agency

Division of Water Pollution Control

1021 North Grand Avenue East

Post Office Box 19276

Springfield, Illinois 62794-9276

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM

Reissued (NPDES) Permit

Expiration Date: November 30, 2017

Issue Date: November 21, 2014
Effective Date: December 1, 2014

Name and Address of Permittee:

City of St. Charles
Two East Main Street
St. Charles, Illinois 60174

Facility Name and Address:

City of St. Charles - Eastside WWTF
East end of Devereaux Way
St. Charles, Illinois
(Kane County)

Receiving Waters: Fox River

In compliance with the provisions of the Illinois Environmental Protection Act, Title 35 of the Ill. Adm. Code, Subtitle C, Chapter I, and the Clean Water Act (CWA), the above-named Permittee is hereby authorized to discharge at the above location to the above-named receiving stream in accordance with the standard conditions and attachments herein.

Permittee is not authorized to discharge after the above expiration date. In order to receive authorization to discharge beyond the expiration date, the Permittee shall submit the proper application as required by the Illinois Environmental Protection Agency (IEPA) not later than 180 days prior to the expiration date.

Alan Keller

Alan Keller, P.E.
Manager, Permit Section
Division of Water Pollution Control

SAK:AAH:11020301.bah

NPDES Permit No. IL0022705

Effluent Limitations, Monitoring, and Reporting

FINAL

Discharge Number(s) and Name(s): B01 STP Internal Outfall

Load limits computed based on a design average flow (DAF) of 9.0 MGD (design maximum flow (DMF) of 18.35 MGD).

From the effective date of this Permit until the expiration date, the effluent of the above discharge(s) shall be monitored and limited at all times as follows:

	LOAD LIMITS lbs/day DAF (DMF)*			CONCENTRATION LIMITS mg/L				
Parameter	Monthly Average	Weekly Average	Daily Maximum	Monthly Average	Weekly Average	Daily Maximum	Sample Frequency	Sample Type
Flow (MGD)							Continuous	
$CBOD_5$**[1]	1501 (3061)	3002 (6122)		20	40		2 Days/Week	Composite
Suspended Solids[1]	1877 (3826)	3378 (6887)		25	45		2 Days/Week	Composite
pH	Shall be in the range of 6 to 9 Standard Units						2 Days/Week	Grab
Fecal Coliform***	The monthly geometric mean shall not exceed 200 per 100 mL (May through October)						5 Days/Week	Grab
Chlorine Residual						0.05	***	Grab
Ammonia Nitrogen: As (N)								
March-May/Sept.-Oct.	113 (230)		135 (275)	1.5		1.8	2 Days/Week	Composite
June-August	98 (199)		105 (214)	1.3		1.4	2 Days/Week	Composite
November-February	----		255 (520)	----		3.4	2 Days/Week	Composite
Total Nitrogen****	Monitor Only						1 Day/Month	Composite
Dissolved Phosphorus	Monitor Only						1 Day/Month	Composite
Nitrate/Nitrite	Monitor Only						1 Day/Month	Grab
Total Kjeldahl Nitrogen (TKN)	Monitor Only						1 Day/Month	Grab
Alkalinity	Monitor Only						1 Day/Month	Grab
Temperature	Monitor Only						1 Day/Month	Grab
		Annual Average			Annual Average			
Total Phosphorus (as P)*****		75 (153)			1.0		1 Day/Week	Composite
				Monthly Average not less than	Weekly Average not less than	Daily Minimum		
Dissolved Oxygen March-July				N/A	6.0	5.0	2 Days/Week	Grab
August-February				5.5	4.0	3.5	2 Days/Week	Grab

*Load limits based on design maximum flow shall apply only when flow exceeds design average flow.
**Carbonaceous BOD_5 ($CBOD_5$) testing shall be in accordance with 40 CFR 136.
***See Special Condition 10. During the weeks of Memorial Day, July Fourth and Labor Day, the sampling frequency shall be 3 Days/Week.
****See Special Condition 15. Total Nitrogen shall be reported on the DMR as a daily maximum value.
***** See Special Condition 19. The annual phosphorus limit has been included in the permit pending the completion of the Fox River Implementation Plan.

NPDES Permit No. IL0022705

Effluent Limitations, Monitoring, and Reporting

FINAL

Discharge Number(s) and Name(s): B01 STP Internal Outfall (continued)

Flow shall be reported on the Discharge Monitoring Report (DMR) as monthly average and daily maximum.

Fecal Coliform shall be reported on the DMR as a monthly geometric mean. No more than 10% of the samples during the month shall exceed 400 per 100 ml.

Chlorine Residual shall be reported on the DMR as a daily maximum value.

pH shall be reported on the DMR as minimum and maximum value.

The rolling annual monthly average total phosphorus values shall be computed monthly beginning 12 months after the effective date of the permit and shall include the previous 12 months of data. The rolling annual monthly average, monthly average and daily maximum values for total phosphorus shall be reported on the DMR. The rolling annual monthly average shall be calculated by adding the sum of the total phosphorus monitoring values from the previous 12 months of data expressed in milligrams/liter and divided by the number of samples collected.

Dissolved Oxygen shall be reported on DMR as Minimum value.

[1]BOD_5 and Suspended Solids (85% removal required): In accordance with 40 CFR 133, the 30-day average percent removal shall not be less than 85 percent except as provided in Sections 133.103 and 133.105. The percent removal need not be reported to the IEPA on DMRs but influent and effluent data must be available, as required elsewhere in this Permit, for IEPA inspection and review. For measuring compliance with this requirement, 5 mg/L shall be added to the effluent $CBOD_5$ concentration to determine the effluent BOD_5 concentration.

NPDES Permit No. IL0022705

Effluent, Limitations, Monitoring, and Reporting

FINAL

Discharge Number(s) and Name(s): A01 Excess Flow Outfall (Flow in excess of 18.35 MGD)

These flow facilities shall not be utilized until the main treatment facility is receiving its design maximum flow (DMF)* (flow in excess of 18.35 MGD).

From the effective date of this Permit until the expiration date, the effluent of the above discharge(s) shall be monitored and limited at all times as follows:

	CONCENTRATION LIMITS (mg/L)			
Parameter	Monthly Average	Weekly Average	Sample Frequency	Sample Type
Total Flow (MG)			Daily When Discharging	Continuous
BOD_5	Monitor Only		Daily When Discharging	Grab
Suspended Solids	Monitor Only		Daily When Discharging	Grab
Ammonia Nitrogen (as N)	Monitor Only		Daily When Discharging	Grab
Total Phosphorus (as P)	Monitor Only		Daily When Discharging	Grab

*An explanation shall be provided in the comment section of the DMR should these facilities be used when the main treatment facility is not receiving Design Maximum Flow (DMF). The explanation shall identify the reasons the main facility is at a diminished treatment capacity. Additionally, the Permittee shall comply with the provisions of Special Condition 7.

The duration of each A01 discharge and rainfall event (i.e., start and ending time) including rainfall intensity shall be provided in the comment section of the DMR.

Total flow in million gallons shall be reported on the Discharge Monitoring Report (DMR) in the quantity maximum column.

Report the number of days of discharge in the comments section of the DMR.

BOD_5 and Suspended Solids shall be reported on the DMR as a daily maximum value.

Ammonia Nitrogen shall be reported on the DMR as a daily maximum value.

Total Phosphorus shall be reported on the DMR as a daily maximum value.

NPDES Permit No. IL0022705

Effluent, Limitations, Monitoring, and Reporting

FINAL

Discharge Number(s) and Name(s): 001 Combined Discharge from A01 and B01 Outfall

From the effective date of this Permit until the expiration date, the effluent of the above discharge(s) shall be monitored and limited at all times as follows:

	CONCENTRATION LIMITS (mg/L)			
Parameter	Monthly Average	Weekly Average	Sample Frequency	Sample Type
Total Flow (MG)			Daily When A01 is Discharging	Continuous
BOD_5**	30	45	Daily When A01 is Discharging	Grab
Suspended Solids**	30	45	Daily When A01 is Discharging	Grab
pH	Shall be in the range of 6 to 9 Standard Units		Daily When A01 is Discharging	Grab
Fecal Coliform	The monthly geometric mean shall not exceed 200 per 100 mL		Daily When A01 is Discharging	Grab
Chlorine Residual	0.75		Daily When A01 is Discharging	Grab
Ammonia Nitrogen (as N)***	Monitor only		Daily When A01 is Discharging	Grab
Total Phosphorus (as P)	Monitor only		Daily When A01 is Discharging	Grab
Dissolved Oxygen	Monitor only		Daily When A01 is Discharging	Grab

*An explanation shall be provided in the comment section of the DMR should these facilities be used when the main treatment facility is not receiving Design Maximum Flow (DMF). The explanation shall identify the reasons the main facility is at a diminished treatment capacity. Additionally, the Permittee shall comply with the provisions of Special Condition 7.

**BOD_5 and Suspended Solids (85% removal required): In accordance with 40 CFR 133, the 30-day average percent removal shall not be less than 85 percent except as provided in Sections 133.103 and 133.105. The percent removal need not be reported to the IEPA on DMRs but influent and effluent data must be available, as required elsewhere in this Permit, for IEPA inspection and review. For measuring compliance with this requirement, 5 mg/L shall be added to the effluent $CBOD_5$ concentration to determine the effluent BOD_5 concentration.

***See Special Condition 20.

Total flow in million gallons shall be reported on the Discharge Monitoring Report (DMR) in the quantity maximum column. Report the number of days of discharge in the comments section of the DMR.

Fecal Coliform shall be reported on the DMR as a monthly geometric mean. No more than 10% of the samples during the month shall exceed 400 per 100 ml.
Chlorine Residual shall be reported on the DMR as monthly average value.
pH shall be reported on the DMR as a minimum and a maximum value.
BOD_5 and Suspended Solids shall be reported on the DMR as a monthly and weekly average concentration.

A monthly average value for ammonia shall be computed for each month that A01 discharges beginning one month after the effective date of the permit. A monthly average concentration shall be determined by combining data collected from 001 and B01 (only B01 data from days when A01 is not discharging) for the reporting period. These monitoring results shall be submitted to the Agency on the DMR. Ammonia Nitrogen shall also be reported on the DMR as a maximum value.

A monthly and weekly average value for Dissolved Oxygen (DO) shall be computed for each month that A01 discharges beginning one month after the effective date of the permit. The monthly and weekly average concentrations for 001 shall be determined by combining data collected from 001 and B01 (only B01 data from days when A01 is not discharging) for the reporting period. These monitoring results shall be submitted to the Agency on the DMR. DO shall also be reported on the DMR as a minimum value.

Total Phosphorus shall be reported on the DMR as a maximum value.

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Influent Monitoring, and Reporting

The influent to the plant shall be monitored as follows:

Parameter	Sample Frequency	Sample Type
Flow (MGD)	Continuous	
BOD_5	2 Days/Week and Daily When Outfall A01 is Discharging	Composite
Suspended Solids	2 Days/Week and Daily When Outfall A01 is Discharging	Composite

Influent samples shall be taken at a point representative of the influent.

Flow (MGD) shall be reported on the Discharge Monitoring Report (DMR) as monthly average and daily maximum.

BOD_5 and Suspended Solids shall be reported on the DMR as a monthly average concentration.

Special Conditions

SPECIAL CONDITION 1. This Permit may be modified to include different final effluent limitations or requirements which are consistent with applicable laws and regulations. The IEPA will public notice the permit modification.

SPECIAL CONDITION 2. The use or operation of this facility shall be by or under the supervision of a Certified Class1 operator.

SPECIAL CONDITION 3. The IEPA may request in writing submittal of operational information in a specified form and at a required frequency at any time during the effective period of this Permit.

SPECIAL CONDITION 4. The IEPA may request more frequent monitoring by permit modification pursuant to 40 CFR § 122.63 and Without Public Notice.

SPECIAL CONDITION 5. The effluent, alone or in combination with other sources, shall not cause a violation of any applicable water quality standard outlined in 35 Ill. Adm. Code 302.

SPECIAL CONDITION 6. The Permittee shall record monitoring results on Discharge Monitoring Report (DMR) Forms using one such form for each outfall each month.

In the event that an outfall does not discharge during a monthly reporting period, the DMR Form shall be submitted with no discharge indicated.

The Permittee may choose to submit electronic DMRs (NetDMRs) instead of mailing paper DMRs to the IEPA. More information, including registration information for the NetDMR program, can be obtained on the IEPA website, http://www.epa.state.il.us/water/net-dmr/index.html.

The completed Discharge Monitoring Report forms shall be submitted to IEPA no later than the 25th day of the following month, unless otherwise specified by the permitting authority.

Permittees not using NetDMRs shall mail Discharge Monitoring Reports with an original signature to the IEPA at the following address:

Illinois Environmental Protection Agency
Division of Water Pollution Control
Attention: Compliance Assurance Section, Mail Code # 19
1021 North Grand Avenue East
Post Office Box 19276
Springfield, Illinois 62794-9276

SPECIAL CONDITION 7. The provisions of 40 CFR Section 122.41(m) & (n) are incorporated herin by reference.

SPECIAL CONDITION 8.

A. For Outfall Number B01: Samples for all effluent limitations and monitoring parameters applicable to Outfall B01 shall be taken at a point representative of the flows from Outfall B01 but prior to entry into the receiving stream. On days when there are discharges from Outfall A01, samples for all effluent limitations and monitoring parameters applicable to Outfall B01 shall be representative of discharges from B01 and shall be taken at a point prior to admixture with discharges from Outfall A01.

B. For Outfall Number A01: Samples for all effluent limitations and monitoring parameters applicable to Outfall A01 shall be taken at a point representative of the discharge from Outfall A01 and shall be taken at a point prior to admixture with discharges from Outfall B01.

C. For Outfall Number 001: Samples for all effluent limitations and monitoring parameters applicable to Outfall 001 shall be taken at a point representative of the discharge from Outfall 001 but prior to entry into the receiving stream and shall include all flow from Outfalls A01 and B01. On days when there are no discharges through Outfall A01, samples for discharges through Outfall 001 can be taken at the location of sampling for Outfall B01. When there are discharges from Outfall A01, samples for all effluent limitations and monitoring parameters applicable to Outfall 001 shall be representative of the discharge from Outfall 001 and shall be taken at a point after flows from Outfalls A01 and B01 are mixed.

SPECIAL CONDITION 9. This Permit may be modified to include requirements for the Permittee on a continuing basis to evaluate and detail its efforts to effectively control sources of infiltration and inflow into the sewer system and to submit reports to the IEPA if necessary.

SPECIAL CONDITION 10. Fecal Coliform limits for Discharge Number B01 are effective May thru October. Sampling of Fecal Coliform is only required during this time period.

Any use of chlorine to control slime growths, odors or as an operational control, etc. shall not exceed the limit of 0.05 mg/L (daily maximum) total residual chlorine in the effluent. Sampling is required on a daily grab basis during the chlorination process. Reporting

Special Conditions

shall be submitted on the DMR's on a monthly basis.

SPECIAL CONDITION 11.

A. Publicly Owned Treatment Works (POTW) Pretreatment Program General Provisions

1. The Permittee shall implement and enforce its approved Pretreatment Program which was approved on September 18, 1985 and all approved subsequent modifications thereto. The Permittee shall maintain legal authority adequate to fully implement the Pretreatment Program in compliance with Federal (40 CFR 403), State, and local laws and regulations. All definitions in this section unless specifically otherwise defined in this section, are those definitions listed in 40 CFR 403.3. USEPA Region 5 is the Approval Authority for the administration of pretreatment programs in Illinois. The Permittee shall:

   a. Develop and implement procedures to ensure compliance with the requirments of a pretreatment program as specified in 40 CFR 403.8 (f) (2).

   b. Carry out independent inspection and monitoring procedures at least once per year, which will determine whether each significant industrial user (SIU) is in compliance with applicable pretreatment standards;

   c. Evaluate whether each SIU needs a slug control plan or other action to control slug discharges. If needed, the SIU slug control plan shall include the items specified in 40 CFR 403.8(f)(2)(vi). For Industrial Users (IUs) identified as significant prior to November 14, 2005, this evaluation must have been conducted at least once by October 14, 2006; additional SIUs must be evaluated within 1 year of being designated an SIU;

   d. Update its inventory of Industrial Users (IUs) at least annually and as needed to ensure that all SIUs are properly identified, characterized, and categorized;

   e. Receive and review self monitoring and other IU reports to determine compliance with all pretreatment standards and requirements, and obtain appropriate remedies for noncompliance by any IU with any pretreatment standard and/or requirement;

   f. Investigate instances of noncompliance, collect and analyze samples, and compile other information with sufficient care as to produce evidence admissible in enforcement proceedings, including judicial action;

   g. Require development, as necessary, of compliance schedules by each industrial user to meet applicable pretreatment standards; and,

   h. Maintain an adequate revenue structure and staffing levels for continued operation of the Pretreatment Program.

2. The Permittee shall issue/reissue permits or equivalent control mechanisms to all SIUs prior to expiration of existing permits or prior to commencement of discharge in the case of new discharges. The permits at a minimum shall include the elements listed in 40 CFR § 403.8(f)(1)(iii).

3. The Permittee shall develop, maintain, and enforce, as necessary, local limits to implement the general and specific prohibitions in 40 CFR § 403.5 which prohibit the introduction of any pollutant(s) which cause pass through or interference and the introduction of specific pollutants to the waste treatment system from any source of nondomestic discharge.

4. In addition to the general limitations expressed in Paragraph 3 above, applicable pretreatment standards must be met by all industrial users of the POTW. These limitations include specific standards for certain industrial categories as determined by Section 307(b) and (c) of the Clean Water Act, State limits, or local limits, whichever are more stringent.

5. The USEPA and IEPA individually retain the right to take legal action against any industrial user and/or the POTW for those cases where an industrial user has failed to meet an applicable pretreatment standard by the deadline date regardless of whether or not such failure has resulted in a permit violation.

6. The Permittee shall establish agreements with all contributing jurisdictions, as necessary, to enable it to fulfill its requirements with respect to all IUs discharging to its system.

7. Unless already completed, the Permittee shall within one (1) year of the effective date of this Permit submit to USEPA and IEPA a proposal to modify and update its approved Pretreatment Program to incorporate Federal revisions to the general pretreatment regulations. The proposal shall include all changes to the approved program and the sewer use ordinance which are necessary to incorporate the revisions of the Pretreatment Streamlining Rule (which became effective on November 14, 2005), which are considered required changes, as described in the Pretreatment Streamlining Rule Fact Sheet 2.0: Required changes, available at: http://cfpub.epa.gov/npdes/whatsnew.cfm?program_id=3. This includes any necessary revisions to the Permittee's Enforcement Response Plan (ERP).

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8. Within 1 year from the effective date of this permit, the Permittee shall conduct a technical re-evaluation of its local limitations consistent with U.S. EPA's Local Limits Development Guidance (July 2004), and submit the evaluation and any proposed revisions to its local limits to IEPA and U.S. EPA Region 5 for review and approval. U.S. EPA Region 5 will request Permittee to submit the evaluation and any proposed revisions to its local limits on the spreadsheet found at http://www.epa.gov/region5/water/npdestek/Locallmt.XLS. To demonstrate technical justification for new local industrial user limits or justification for retaining existing limits, the following information must be submitted to U.S. EPA:

   a. Total plant flow
   b. Domestic/commercial pollutant contributions for pollutants of concern
   c. Industrial pollutant contributions and flows
   d. Current POTW pollutant loadings, including loadings of conventional pollutants
   e. Actual treatment plant removal efficiencies, as a decimal (primary, secondary, across the wastewater treatment plant)
   f. Safety factor to be applied
   g. Identification of applicable criteria:
      i. NPDES permit conditions
         •Specific NPDES effluent limitations
         •Water-quality criteria
         •Whole effluent toxicity requirements
         •Criteria and other conditions for sludge disposal
      ii. Biological process inhibition
         •Nitrification
         •Sludge digester
      iii. Collection system problems
   h. The Permittee's sludge disposal methods (land application, surface disposal, incineration, landfill)
   i. Sludge flow to digester
   j. Sludge flow to disposal
   k. % solids in sludge to disposal, not as a decimal
   l. % solids in sludge to digester, not as a decimal
   m. Plant removal efficiencies for conventional pollutants
   n. If revised industrial user discharge limits are proposed, the method of allocating available pollutants loads to industrial users
   o. A comparison of maximum allowable headworks loadings based on all applicable criteria listed in g, above
   p. Pollutants that have caused:
      i. Violations or operational problems at the POTW, including conventional pollutants
      ii. Fires and explosions
      iii. Corrosion
      iv. Flow obstructions
      v. Increased temperature in the sewer system
      vi. Toxic gases, vapors or fumes that caused acute worker health and safety problems
      vii. Toxicity found through Whole Effluent Toxicity testing
      viii. Inhibition
   q. Pollutants designated as "monitoring only" in the NPDES permit
   r. Supporting data, assumptions, and methodologies used in establishing the information a through q above.

9 The Permittee's Pretreatment Program has been modified to incorporate a Pretreatment Program Amendment approved by USEPA on October 1, 1996. The amendment became effective on the date of approval and is a fully enforceable provision of your Pretreatment Program.

Modifications of your Pretreatment Program shall be submitted in accordance with 40 CFR § 403.18, which established conditions for substantial and nonsubstantial modifications. All requests should be sent in electronic format to r5npdes@epa.gov, attention: NPDES Programs Branch.

B. Reporting and Records Requirements

1. The Permittee shall provide an annual report briefly describing the permittee's pretreatment program activities over the previous calendar year. Permittees who operate multiple plants may provide a single report providing all plant-specific reporting requirements are met. Such report shall be submitted no later than April 28th of each year to USEPA, Region 5, 77 West Jackson Blvd., Chicago, Illinois 60604, Attention: Water Enforcement and Compliance Assurance Branch, and shall be in the format set forth in IEPA's POTW Pretreatment Report Package which contains information regarding:

   a. An updated listing of the Permittee's significant industrial users, indicating additions and deletions from the previous year, along with brief explanations for deletions. The list shall specify which categorical Pretreatment standards, if any, are applicable to each Industrial User.

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b. A descriptive summary of the compliance activities including numbers of any major enforcement actions, (i.e., administrative orders, penalties, civil actions, etc.), and the outcome of those actions. This includes an assessment of the compliance status of the Permittee's industrial users and the effectiveness of the Permittee's Pretreatment Program in meeting its needs and objectives.

c. A description of all substantive changes made to the Permittee's Pretreatment Program. Changes which are "substantial modifications" as described in 40 CFR § 403.18(c) must receive prior approval from the USEPA.

d. Results of sampling and analysis of POTW influent, effluent, and sludge.

e. A summary of the findings from the priority pollutants sampling. As sufficient data becomes available the IEPA may modify this Permit to incorporate additional requirements relating to the evaluation, establishment, and enforcement of local limits for organic pollutants. Any permit modification is subject to formal due process procedures pursuant to State and Federal law and regulation. Upon a determination that an organic pollutant is present that causes interference or pass through, the Permittee shall establish local limits as required by 40 CFR § 403.5(c).

2. The Permittee shall maintain all pretreatment data and records for a minimum of three (3) years. This period shall be extended during the course of unresolved litigation or when requested by the IEPA or the Regional Administrator of USEPA. Records shall be available to USEPA and the IEPA upon request.

3. The Permittee shall establish public participation requirements of 40 CFR 25 in implementation of its Pretreatment Program. The Permittee shall at least annually, publish the names of all IU's which were in significant noncompliance (SNC), as defined by 40 CFR § 403.8(f)(2)(viii), in a newspaper of general circulation that provides meaningful public notice within the jurisdictions served by the Permittee or based on any more restrictive definition of SNC that the POTW may be using.

4. The Permittee shall provide written notification to the USEPA, Region 5, 77 West Jackson Blvd., Chicago, Illinois 60604, Attention: NPDES Programs Branch and to the Deputy Counsel for the Division of Water Pollution Control, IEPA, 1021 North Grand Avenue East, P.O. Box 19276, Springfield, Illinois 62794-9276 within five (5) days of receiving notice that any Industrial User of its sewage treatment plant is appealing to the Circuit Court any condition imposed by the Permittee in any permit issued to the Industrial User by Permittee. A copy of the Industrial User's appeal and all other pleadings filed by all parties shall be mailed to the Deputy Counsel within five (5) days of the pleadings being filed in Circuit Court.

C. Monitoring Requirements

1. The Permittee shall monitor its influent, effluent and sludge and report concentrations of the following parameters on monitoring report forms provided by the IEPA and include them in its annual report. Samples shall be taken at semi-annual intervals at the indicated reporting limit or better and consist of a 24-hour composite unless otherwise specified below. Sludge samples shall be taken of final sludge and consist of a grab sample reported on a dry weight basis.

STORET CODE	PARAMETER	Minimum reporting limit
01097	Antimony	0.07 mg/L
01002	Arsenic	0.05 mg/L
01007	Barium	0.5 mg/L
01012	Beryllium	0.005 mg/L
01027	Cadmium	0.001 mg/L
01032	Chromium (hex) (grab not to exceed 24 hours)*	0.01 mg/L
01034	Chromium (total)	0.05 mg/L
01042	Copper	0.005 mg/L
00718	Cyanide* (grab) (available **** or amenable to chlorination)	5.0 ug/L
00720	Cyanide (total) (grab)	5.0 ug/L
00951	Fluoride*	0.1 mg/L
01045	Iron (total)	0.5 mg/L
01046	Iron (Dissolved)*	0.5 mg/L
01051	Lead	0.05 mg/L
01055	Manganese	0.5 mg/L
71900	Mercury (effluent grab)***	1.0 ng/L**
01067	Nickel	0.005 mg/L
00556	Oil (hexane soluble or equivalent) (Grab Sample only)*	5.0 mg/L
32730	Phenols (grab)	0.005 mg/L
01147	Selenium	0.005 mg/L
01077	Silver (total)	0.003 mg/L
01059	Thallium	0.3 mg/L
01092	Zinc	0.025 mg/L

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Minimum reporting limits are defined as - (1) The minimum value below which data are documented as non-detects. (2) Three to ten times the method detection limit. (3) The minimum value of the calibration range.

All sample containers, preservatives, holding times, analyses, method detection limit determinations and quality assurance/quality control requirements shall be in accordance with 40 CFR 136.

* Influent and effluent only
**1 ng/L = 1 part per trillion.
***Utilize USEPA Method 1631E and the digestion procedure described in Section 11.1.1.2 of 1631E, other approved methods may be used for influent (composite) and sludge.
**** USEPA Method OIA-1677.

Unless otherwise indicated, concentrations refer to the total amount of the constituent present in all phases, whether solid, suspended or dissolved, elemental or combined including all oxidation states. Where constituents are commonly measured as other than total, the phase is so indicated.

2. The Permittee shall conduct an analysis for the one hundred and ten (110) organic priority pollutants identified in 40 CFR 122 Appendix D, Table II as amended. This monitoring shall be done annually and reported on monitoring report forms provided by the IEPA and shall consist of the following:

   a. The influent and effluent shall be sampled and analyzed for the one hundred and ten (110) organic priority pollutants. The sampling shall be done during a day when industrial discharges are expected to be occurring at normal to maximum levels.

      Samples for the analysis of acid and base/neutral extractable compounds shall be 24-hour composites.

      Five (5) grab samples shall be collected each monitoring day to be analyzed for volatile organic compounds. A single analysis for volatile pollutants (Method 624) may be run for each monitoring day by compositing equal volumes of each grab sample directly in the GC purge and trap apparatus in the laboratory, with no less than one (1) mL of each grab included in the composite.

      Wastewater samples must be handled, prepared, and analyzed by GC/MS in accordance with USEPA Methods 624 and 625 of 40 CFR 136 as amended.

   b. The sludge shall be sampled and analyzed for the one hundred and ten (110) organic priority pollutants. A sludge sample shall be collected concurrent with a wastewater sample and taken as final sludge.

      Sampling and analysis shall conform to USEPA Methods 624 and 625 unless an alternate method has been approved by IEPA.

   c. Sample collection, preservation and storage shall conform to approved USEPA procedures and requirements.

3. In addition, the Permittee shall monitor any new toxic substances as defined by the Clean Water Act, as amended, following notification by the IEPA.

4. Permittee shall report any noncompliance with effluent or water quality standards in accordance with Standard Condition 12(f) of this Permit.

5. Analytical detection limits shall be in accordance with 40 CFR 136. Minimum detection limits for sludge analyses shall be in accordance with 40 CFR 503.

D. Pretreatment Reporting

USEPA Region 5 is the Approval Authority for administering the pretreatment program in Illinois. All requests for modification of pretreatment program elements should be submitted in redline/strikeout electronic format and must be sent to USEPA at r5npdes@epa.gov.

Permittee shall upon notice from USEPA, modify any pretreatment program element found to be inconsistent with 40 CFR 403.

SPECIAL CONDITION 12. During January of each year the Permittee shall submit annual fiscal data regarding sewerage system operations to the Illinois Environmental Protection Agency/Division of Water Pollution Control/Compliance Assurance Section. The Permittee may use any fiscal year period provided the period ends within twelve (12) months of the submission date.

Submission shall be on forms provided by IEPA titled "Fiscal Report Form For NPDES Permittees".

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SPECIAL CONDITION 13. For the duration of this Permit, the Permittee shall determine the quantity of sludge produced by the treatment facility in dry tons or gallons with average percent total solids analysis. The Permittee shall maintain adequate records of the quantities of sludge produced and have said records available for IEPA inspection. The Permittee shall submit to the IEPA, at a minimum, a semi-annual summary report of the quantities of sludge generated and disposed of, in units of dry tons or gallons (average total percent solids) by different disposal methods including but not limited to application on farmland, application on reclamation land, landfilling, public distribution, dedicated land disposal, sod farms, storage lagoons or any other specified disposal method. Said reports shall be submitted to the IEPA by January 31 and July 31 of each year reporting the preceding January thru June and July thru December interval of sludge disposal operations.

Duty to Mitigate. The Permittee shall take all reasonable steps to minimize any sludge use or disposal in violation of this Permit.

Sludge monitoring must be conducted according to test procedures approved under 40 CFR 136 unless otherwise specified in 40 CFR 503, unless other test procedures have been specified in this Permit.

Planned Changes. The Permittee shall give notice to the IEPA on the semi-annual report of any changes in sludge use and disposal.

The Permittee shall retain records of all sludge monitoring, and reports required by the Sludge Permit as referenced in Standard Condition 23 for a period of at least five (5) years from the date of this Permit.

If the Permittee monitors any pollutant more frequently than required by the Sludge Permit, the results of this monitoring shall be included in the reporting of data submitted to the IEPA.

The Permittee shall comply with existing federal regulations governing sewage sludge use or disposal and shall comply with all existing applicable regulations in any jurisdiction in which the sewage sludge is actually used or disposed.

The Permittee shall comply with standards for sewage sludge use or disposal established under Section 405(d) of the CWA within the time provided in the regulations that establish the standards for sewage sludge use or disposal even if the permit has not been modified to incorporate the requirement.

The Permittee shall ensure that the applicable requirements in 40 CFR Part 503 are met when the sewage sludge is applied to the land, placed on a surface disposal site, or fired in a sewage sludge incinerator.

Monitoring reports for sludge shall be reported on the form titled "Sludge Management Reports" to the following address:

Illinois Environmental Protection Agency
Bureau of Water
Compliance Assurance Section
Mail Code #19
1021 North Grand Avenue East
Post Office Box 19276
Springfield, Illinois 62794-9276

SPECIAL CONDITION 14. This Permit may be modified to include alternative or additional final effluent limitations pursuant to either an approved Total Maximum Daily Load (TMDL) Study or an approved Fox River Implementation Plan.

SPECIAL CONDITION 15. Monitoring for Total Nitrogen is required to document the actual total nitrogen effluent concentration. The Permittee shall monitor the effluent for total nitrogen one/month. The monitoring shall be a composite sample and the results reported as a daily maximum on the Permittee's Discharge Monitoring Forms.

SPECIAL CONDITION 16. The Permittee shall conduct biomonitoring of the effluent from Discharge Number(s) B01.

Biomonitoring

1. Acute Toxicity - Standard definitive acute toxicity tests shall be run on at least two trophic levels of aquatic species (fish, invertebrate) representative of the aquatic community of the receiving stream. Testing must be consistent with Methods for Measuring the Acute Toxicity of Effluents and Receiving Waters to Freshwater and Marine Organisms (Fifth Ed.) EPA/821-R-02-012. Unless substitute tests are pre-approved; the following tests are required:

    a. Fish - 96 hour static LC_{50} Bioassay using fathead minnows (Pimephales promelas).

    b. Invertebrate 48-hour static LC_{50} Bioassay using Ceriodaphnia.

Special Conditions

2. Testing Frequency - The above tests shall be conducted using 24-hour composite samples unless otherwise authorized by the IEPA. Samples must be collected in the 18th, 15th, 12th, and 9th month prior to the expiration date of this Permit.

3. Reporting - Results shall be reported according to EPA/821-R-02-012, Section 12, Report Preparation, and shall be submitted to IEPA, Bureau of Water, Compliance Assurance Section within one week of receipt from the laboratory. Reports are due to the IEPA no later than the 16th, 13th, 10th, and 7th month prior to the expiration date of this Permit.

4. Toxicity - Should a bioassay result in toxicity to >20% of organisms test in the 100% effluent treatment, the IEPA may require, upon notification, six (6) additional rounds of monthly testing on the affected organism(s) to be initiated within 30 days of the toxic bioassay. Results shall be submitted to IEPA within (1) week of becoming available to the Permittee. Should any of the additional bioassays result in toxicity to ≥50% of organisms tested in the 100% effluent treatments, the Permittee shall immediately notify IEPA in writing of the test results.

5. Toxicity Reduction Evaluation and Identification - Should the biomonitoring program identify toxicity and result in notification by IEPA, the permittee shall develop a plan for toxicity reduction evaluation and identification. The plan shall be developed and implemented in accordance with Toxicity Reduction Evaluation Guidance for Municipal Wastewater Treatment Plants, EPA/833B-99/002, and shall include an evaluation to determine which chemicals have a potential for being discharged in the plant wastewater, a monitoring program to determine their presence or absence and to identify other compounds which are not being removed by treatment, and other measures as appropriate. The Permittee shall submit to the IEPA its plan within ninety (90) days following notification by the IEPA. The Permittee shall implement the plan within ninety (90) days of notification of the permittee above or other such date as is received by letter from IEPA.

   The IEPA may modify this Permit during its term to incorporate additional requirements or limitations based on the results of the biomonitoring. In addition, after review of the monitoring results and toxicity reduction evaluation, the IEPA may modify this Permit to include numerical limitations for specific toxic pollutants and additional whole effluent toxicity monitoring to confirm the results of the evaluation. Modifications under this condition shall follow public notice and opportunity for hearing.

SPECIAL CONDITION 17. The Permittee shall monitor the wastewater effluent for Total Phosphorus, Dissolved Phosphorus, Nitrate/Nitrite, Total Kjeldahl Nitrogen (TKN), Ammonia, Total Nitrogen (calculated), Alkalinity and Temperature at least once a month beginning on the effective date of this permit. The results shall be submitted on Discharge Monitoring Report (DMR) Forms or NetDMRs to IEPA unless otherwise specified by the IEPA.

SPECIAL CONDITION 18. The Permittee shall participate in the Fox River Study Group (FRSG). The Permittee shall work with other watershed members of the FRSG to determine the most cost effective means to remove dissolved oxygen (DO) and offensive condition impairments in the Fox River. This Permit may be modified to include additional conditions and effluent limitations to include implementation measures based on the Fox River Implementation Plan (Implementation Plan). The following tasks will be completed during the life of this permit:

1. The Permittee shall prepare a phosphorus removal feasibility report specific to its plant(s) on the method, time frame and costs for reducing its loading of phosphorus to levels equivalent to monthly average discharges of 1 mg/L and 0.5 mg/L on a seasonal basis and on a year round basis. The feasibility report shall be submitted to the IEPA twelve (12) months from the effective date of the Permit. The feasibility report shall also be shared with the FRSG.
2. The Permittee shall submit the Fox River Study Group Watershed Investigation Phase III Report, which includes stream modeling, to the IEPA within 1 month of the effective date of this Permit.
3. The FRSG will complete an Implementation Plan that identifies phosphorus input reductions by point source discharges, non-point source discharges and other measures necessary to remove DO and offensive condition impairments in the Fox River. The Implementation Plan shall be submitted to the IEPA by December 31, 2015. The Permittee shall initiate the recommendations of the Implementation Plan that are applicable to said Permittee during the remaining term of this Permit. This Permit may be modified to include additional pollutant reduction activities necessary to implement the Implementation Plan.
4. In its application for renewal of this permit, the Permittee shall consider and incorporate recommended FRSG phosphorus input reduction implementation projects that the Permittee will implement during the next permit term.
5. The Permittee shall operate the existing facilities to optimize the removal of phosphorus.

SPECIAL CONDITION 19. A phosphorus limit of 1.0 mg/L (Annual Average) shall become effective four and one-half (4 1/2) years from the effective date of this Permit.

In order for the Permittee to achieve the above limit, it will be necessary to modify existing treatment facilities to include phosphorus removal, reduce phosphorus sources or explore other ways to prevent discharges that exceed the limit. The Permittee must implement the following compliance measures consistent with the schedule below:

1. Interim Report on Phosphorus Removal Feasibility Report	6 months from the effective date of this Permit

Special Conditions

2.	Phosphorus Removal Feasibility Report submitted	12 months from the effective date of this Permit
3.	Progress Report on Phosphorus Input Reductions and Implementation Plan	18 Months from the effective date of this Permit
4.	Progress Report on Recommendations of Implementation Plan	24 months from the effective date of this Permit
5.	Plans and specifications submitted	30 months from the effective date of this Permit
6.	Progress Report on Construction	36 months from the effective date of this Permit
7.	Complete Construction	42 months from the effective date of this Permit
8.	Progress Report on Optimizing Treatment System	48 months from the effective date of this Permit
9.	Achieve Annual Concentration and Loading Effluent Limitations for Total Phosphorus	54 months from the effective date of this Permit

Compliance dates may be modified based on the results of the Phosphorus Removal Feasibility Report required by Special Condition 18 of this Permit. All modifications of this Permit must be in accordance with 40 CFR 122.62 or 40 CFR 122.63.

Reporting shall be submitted on the DMR's on a monthly basis.

REPORTING

The Permittee shall submit progress reports for items 1, 2, 3, 4, 6, 7, 8 and 9 of the compliance schedule indicating: a) the date the item was completed, or b) that the item was not completed, the reasons for non-completion and the anticipated completion date to the Agency Compliance Section.

SPECIAL CONDITION 20. The Agency shall consider all monitoring data submitted by the discharger in accordance with the monitoring requirements of this permit for all parameters, including but not limited to data pertaining to ammonia and dissolved oxygen for discharges from Discharge Number 001, to determine whether the discharges are at levels which cause, have the reasonable potential to cause or contribute to exceedances of water quality standards; and, if so, to develop appropriate water quality based effluent limitations. If the discharger wants the Agency to consider mixing when determining the need for and establishment of water quality based effluent limitations, the discharger shall submit a study plan on mixing to the Agency for the Agency's review and comment within two (2) months of the effective date of this Permit.

SPECIAL CONDITION 21. The Permittee shall work towards the goals of achieving no discharges from sanitary sewer overflows or basement backups and ensuring that overflows or backups, when they do occur do not cause or contribute to violations of applicable standards or cause impairment in any adjacent receiving water. In order to accomplish these goals, the Permittee shall develop, implement and submit to the IEPA a Capacity, Management, Operations, and Maintenance (CMOM) plan within twelve (12) months of the effective date of this Permit. The Permittee should work as appropriate, in consultation with affected authorities at the local, county, and/or state level to develop the plan components involving third party notification of overflow events. The Permittee may be required to construct additional sewage transport and/or treatment facilities in future permits or other enforceable documents should the implemented CMOM plan indicate that the Permittee's facilities are not capable of conveying and treating the flow for which they were designed.

The CMOM plan shall include the following elements:

a. Measures and Activities:

1. A complete map of the collection system owned and operated by the Permittee;
2. Schedules, checklists, and mechanisms to ensure that preventative maintenance is performed on equipment owned and operated by the Permittee;
3. An assessment of the capacity of the collection and treatment system owned and operated by the Permittee at critical junctions and immediately upstream of locations where overflows and backups occur or are likely to occur; and
4. Identification and prioritization of structural deficiencies in the system owned and operated by the Permittee.

b. Design and Performance Provisions:

1. Monitor the effectiveness of CMOM;
2. Upgrade the elements of the CMOM plan as necessary; and

Special Conditions

3. Maintain summary of CMOM activities.

c. Overflow Response Plan:

1. Know where overflows within the facilities owned and operated by the Permittee occur;
2. Respond to each overflow to determine additional actions such as clean up; and
3. Locations where basement back-ups and/or sanitary sewer overflows occur shall be evaluated as soon as practicable for excessive inflow /infiltration, obstructions or other causes of overflows or back-ups as set forth in the System Evaluation Plan.

d. System Evaluation Plan.

e. Reporting and Monitoring Requirements.

f. Third Party Notice Plan:

1. Describes how, under various overflow scenarios, the public, as well as other entities, would be notified of overflows within the Permittee's system that may endanger public health, safety or welfare;
2. Identifies overflows within the Permittee's system that would be reported, giving consideration to various types of events including events with potential widespread impacts;
3. Identifies who shall receive the notification;
4. Identifies the specific information that would be reported including actions that will be taken to respond to the overflow;
5. Includes a description of the lines of communication; and
6. Includes the identities and contact information of responsible POTW officials and local, county, and/or state level officials.

SPECIAL CONDITION 22. The Permittee may collect data in support of developing site-specific effluent limitations for ammonia nitrogen. In-stream monitoring for pH and temperature would be required. Samples should be taken downstream at a point representative of substantial mixing with the receiving stream and below the surface. A monitoring plan must be submitted to the Agency for approval which indicates the location, sample frequency and the duration of the monitoring program.

Attachment H

Standard Conditions

Definitions

Act means the Illinois Environmental Protection Act, 415 ILCS 5 as Amended.

Agency means the Illinois Environmental Protection Agency.

Board means the Illinois Pollution Control Board.

Clean Water Act (formerly referred to as the Federal Water Pollution Control Act) means Pub. L 92-500, as amended. 33 U.S.C. 1251 et seq.

NPDES (National Pollutant Discharge Elimination System) means the national program for issuing, modifying, revoking and reissuing, terminating, monitoring and enforcing permits, and imposing and enforcing pretreatment requirements, under Sections 307, 402, 318 and 405 of the Clean Water Act.

USEPA means the United States Environmental Protection Agency.

Daily Discharge means the discharge of a pollutant measured during a calendar day or any 24-hour period that reasonably represents the calendar day for purposes of sampling. For pollutants with limitations expressed in units of mass, the "daily discharge" is calculated as the total mass of the pollutant discharged over the day. For pollutants with limitations expressed in other units of measurements, the "daily discharge" is calculated as the average measurement of the pollutant over the day.

Maximum Daily Discharge Limitation (daily maximum) means the highest allowable daily discharge.

Average Monthly Discharge Limitation (30 day average) means the highest allowable average of daily discharges over a calendar month, calculated as the sum of all daily discharges measured during a calendar month divided by the number of daily discharges measured during that month.

Average Weekly Discharge Limitation (7 day average) means the highest allowable average of daily discharges over a calendar week, calculated as the sum of all daily discharges measured during a calendar week divided by the number of daily discharges measured during that week.

Best Management Practices (BMPs) means schedules of activities, prohibitions of practices, maintenance procedures, and other management practices to prevent or reduce the pollution of waters of the State. BMPs also include treatment requirements, operating procedures, and practices to control plant site runoff, spillage or leaks, sludge or waste disposal, or drainage from raw material storage.

Aliquot means a sample of specified volume used to make up a total composite sample.

Grab Sample means an individual sample of at least 100 milliliters collected at a randomly-selected time over a period not exceeding 15 minutes.

24-Hour Composite Sample means a combination of at least 8 sample aliquots of at least 100 milliliters, collected at periodic intervals during the operating hours of a facility over a 24-hour period.

8-Hour Composite Sample means a combination of at least 3 sample aliquots of at least 100 milliliters, collected at periodic intervals during the operating hours of a facility over an 8-hour period.

Flow Proportional Composite Sample means a combination of sample aliquots of at least 100 milliliters collected at periodic intervals such that either the time interval between each aliquot or the volume of each aliquot is proportional to either the stream flow at the time of sampling or the total stream flow since the collection of the previous aliquot.

(1) **Duty to comply.** The permittee must comply with all conditions of this permit. Any permit noncompliance constitutes a violation of the Act and is grounds for enforcement action, permit termination, revocation and reissuance, modification, or for denial of a permit renewal application. The permittee shall comply with effluent standards or prohibitions established under Section 307(a) of the Clean Water Act for toxic pollutants within the time provided in the regulations that establish these standards or prohibitions, even if the permit has not yet been modified to incorporate the requirements.

(2) **Duty to reapply.** If the permittee wishes to continue an activity regulated by this permit after the expiration date of this permit, the permittee must apply for and obtain a new permit. If the permittee submits a proper application as required by the Agency no later than 180 days prior to the expiration date, this permit shall continue in full force and effect until the final Agency decision on the application has been made.

(3) **Need to halt or reduce activity not a defense**. It shall not be a defense for a permittee in an enforcement action that it would have been necessary to halt or reduce the permitted activity in order to maintain compliance with the conditions of this permit.

(4) **Duty to mitigate**. The permittee shall take all reasonable steps to minimize or prevent any discharge in violation of this permit which has a reasonable likelihood of adversely affecting human health or the environment.

(5) **Proper operation and maintenance**. The permittee shall at all times properly operate and maintain all facilities and systems of treatment and control (and related appurtenances) which are installed or used by the permittee to achieve compliance with conditions of this permit. Proper operation and maintenance includes effective performance, adequate funding, adequate operator staffing and training, and adequate laboratory and process controls, including appropriate quality assurance procedures. This provision requires the operation of back-up, or auxiliary facilities, or similar systems only when necessary to achieve compliance with the conditions of the permit.

(6) **Permit actions**. This permit may be modified, revoked and reissued, or terminated for cause by the Agency pursuant to 40 CFR 122.62 and 40 CFR 122.63. The filing of a request by the permittee for a permit modification, revocation and reissuance, or termination, or a notification of planned changes or anticipated noncompliance, does not stay any permit condition.

(7) **Property rights**. This permit does not convey any property rights of any sort, or any exclusive privilege.

(8) **Duty to provide information**. The permittee shall furnish to the Agency within a reasonable time, any information which the Agency may request to determine whether cause exists for modifying, revoking and reissuing, or terminating this permit, or to determine compliance with the permit. The permittee shall also furnish to the Agency upon request, copies of records required to be kept by this permit.

(9) **Inspection and entry**. The permittee shall allow an authorized representative of the Agency or USEPA (including an authorized contractor acting as a representative of the Agency or USEPA), upon the presentation of credentials and other documents as may be required by law, to:
(a) Enter upon the permittee's premises where a regulated facility or activity is located or conducted, or where records must be kept under the conditions of this permit;
(b) Have access to and copy, at reasonable times, any records that must be kept under the conditions of this permit;
(c) Inspect at reasonable times any facilities, equipment (including monitoring and control equipment), practices, or operations regulated or required under this permit; and
(d) Sample or monitor at reasonable times, for the purpose of assuring permit compliance, or as otherwise authorized by the Act, any substances or parameters at any location.

(10) **Monitoring and records**.
(a) Samples and measurements taken for the purpose of monitoring shall be representative of the monitored activity.
(b) The permittee shall retain records of all monitoring information, including all calibration and maintenance records, and all original strip chart recordings for continuous monitoring instrumentation, copies of all reports required by this permit, and records of all data used to complete the application for this permit, for a period of at least 3 years from the date of this permit, measurement, report or application. Records related to the permittee's sewage sludge use and disposal activities shall be retained for a period of at least five years (or longer as required by 40 CFR Part 503). This period may be extended by request of the Agency or USEPA at any time.
(c) Records of monitoring information shall include:
(1) The date, exact place, and time of sampling or measurements;
(2) The individual(s) who performed the sampling or measurements;
(3) The date(s) analyses were performed;
(4) The individual(s) who performed the analyses;
(5) The analytical techniques or methods used; and
(6) The results of such analyses.
(d) Monitoring must be conducted according to test procedures approved under 40 CFR Part 136, unless other test procedures have been specified in this permit. Where no test procedure under 40 CFR Part 136 has been approved, the permittee must submit to the Agency a test method for approval. The permittee shall calibrate and perform maintenance procedures on all monitoring and analytical instrumentation at intervals to ensure accuracy of measurements.

(11) **Signatory requirement**. All applications, reports or information submitted to the Agency shall be signed and certified.
(a) **Application**. All permit applications shall be signed as follows:
(1) For a corporation: by a principal executive officer of at least the level of vice president or a person or position having overall responsibility for environmental matters for the corporation:
(2) For a partnership or sole proprietorship: by a general partner or the proprietor, respectively; or
(3) For a municipality, State, Federal, or other public agency: by either a principal executive officer or ranking elected official.
(b) **Reports**. All reports required by permits, or other information requested by the Agency shall be signed by a person described in paragraph (a) or by a duly authorized representative of that person. A person is a duly authorized representative only if:
(1) The authorization is made in writing by a person described in paragraph (a); and
(2) The authorization specifies either an individual or a position responsible for the overall operation of the facility, from which the discharge originates, such as a plant manager, superintendent or person of equivalent responsibility; and
(3) The written authorization is submitted to the Agency.
(c) **Changes of Authorization**. If an authorization under (b) is no longer accurate because a different individual or position has responsibility for the overall operation of the facility, a new authorization satisfying the requirements of (b) must be submitted to the Agency prior to or together with any reports, information, or applications to be signed by an authorized representative.
(d) **Certification**. Any person signing a document under paragraph (a) or (b) of this section shall make the following certification:

I certify under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations.

(12) **Reporting requirements**.
(a) **Planned changes**. The permittee shall give notice to the Agency as soon as possible of any planned physical alterations or additions to the permitted facility.
Notice is required when:
(1) The alteration or addition to a permitted facility may meet one of the criteria for determining whether a facility is a new source pursuant to 40 CFR 122.29 (b); or
(2) The alteration or addition could significantly change the nature or increase the quantity of pollutants discharged. This notification applies to pollutants which are subject neither to effluent limitations in the permit, nor to notification requirements pursuant to 40 CFR 122.42 (a)(1).
(3) The alteration or addition results in a significant change in the permittee's sludge use or disposal practices, and such alteration, addition, or change may justify the application of permit conditions that are different from or absent in the existing permit, including notification of additional use or disposal sites not reported during the permit application process or not reported pursuant to an approved land application plan.
(b) **Anticipated noncompliance**. The permittee shall give advance notice to the Agency of any planned changes in the permitted facility or activity which may result in noncompliance with permit requirements.
(c) **Transfers**. This permit is not transferable to any person except after notice to the Agency.
(d) **Compliance schedules**. Reports of compliance or noncompliance with, or any progress reports on, interim and final requirements contained in any compliance schedule of this permit shall be submitted no later than 14 days following each schedule date.
(e) **Monitoring reports**. Monitoring results shall be reported at the intervals specified elsewhere in this permit.
(1) Monitoring results must be reported on a Discharge Monitoring Report (DMR).

(2) If the permittee monitors any pollutant more frequently than required by the permit, using test procedures approved under 40 CFR 136 or as specified in the permit, the results of this monitoring shall be included in the calculation and reporting of the data submitted in the DMR.

(3) Calculations for all limitations which require averaging of measurements shall utilize an arithmetic mean unless otherwise specified by the Agency in the permit.

(f) **Twenty-four hour reporting**. The permittee shall report any noncompliance which may endanger health or the environment. Any information shall be provided orally within 24-hours from the time the permittee becomes aware of the circumstances. A written submission shall also be provided within 5 days of the time the permittee becomes aware of the circumstances. The written submission shall contain a description of the noncompliance and its cause; the period of noncompliance, including exact dates and time; and if the noncompliance has not been corrected, the anticipated time it is expected to continue; and steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance. The following shall be included as information which must be reported within 24-hours:

(1) Any unanticipated bypass which exceeds any effluent limitation in the permit.

(2) Any upset which exceeds any effluent limitation in the permit.

(3) Violation of a maximum daily discharge limitation for any of the pollutants listed by the Agency in the permit or any pollutant which may endanger health or the environment.

The Agency may waive the written report on a case-by-case basis if the oral report has been received within 24-hours.

(g) **Other noncompliance**. The permittee shall report all instances of noncompliance not reported under paragraphs (12) (d), (e), or (f), at the time monitoring reports are submitted. The reports shall contain the information listed in paragraph (12) (f).

(h) **Other information**. Where the permittee becomes aware that it failed to submit any relevant facts in a permit application, or submitted incorrect information in a permit application, or in any report to the Agency, it shall promptly submit such facts or information.

(13) **Bypass**.

(a) Definitions.

(1) Bypass means the intentional diversion of waste streams from any portion of a treatment facility.

(2) Severe property damage means substantial physical damage to property, damage to the treatment facilities which causes them to become inoperable, or substantial and permanent loss of natural resources which can reasonably be expected to occur in the absence of a bypass. Severe property damage does not mean economic loss caused by delays in production.

(b) Bypass not exceeding limitations. The permittee may allow any bypass to occur which does not cause effluent limitations to be exceeded, but only if it also is for essential maintenance to assure efficient operation. These bypasses are not subject to the provisions of paragraphs (13)(c) and (13)(d).

(c) Notice.

(1) Anticipated bypass. If the permittee knows in advance of the need for a bypass, it shall submit prior notice, if possible at least ten days before the date of the bypass.

(2) Unanticipated bypass. The permittee shall submit notice of an unanticipated bypass as required in paragraph (12)(f) (24-hour notice).

(d) Prohibition of bypass.

(1) Bypass is prohibited, and the Agency may take enforcement action against a permittee for bypass, unless:

(i) Bypass was unavoidable to prevent loss of life, personal injury, or severe property damage;

(ii) There were no feasible alternatives to the bypass, such as the use of auxiliary treatment facilities, retention of untreated wastes, or maintenance during normal periods of equipment downtime. This condition is not satisfied if adequate back-up equipment should have been installed in the exercise of reasonable engineering judgment to prevent a bypass which occurred during normal periods of equipment downtime or preventive maintenance; and

(iii) The permittee submitted notices as required under paragraph (13)(c).

(2) The Agency may approve an anticipated bypass, after considering its adverse effects, if the Agency determines that it will meet the three conditions listed above in paragraph (13)(d)(1).

(14) **Upset**.

(a) Definition. Upset means an exceptional incident in which there is unintentional and temporary noncompliance with technology based permit effluent limitations because of factors beyond the reasonable control of the permittee. An upset does not include noncompliance to the extent caused by operational error, improperly designed treatment facilities, inadequate treatment facilities, lack of preventive maintenance, or careless or improper operation.

(b) Effect of an upset. An upset constitutes an affirmative defense to an action brought for noncompliance with such technology based permit effluent limitations if the requirements of paragraph (14)(c) are met. No determination made during administrative review of claims that noncompliance was caused by upset, and before an action for noncompliance, is final administrative action subject to judicial review.

(c) Conditions necessary for a demonstration of upset. A permittee who wishes to establish the affirmative defense of upset shall demonstrate, through properly signed, contemporaneous operating logs, or other relevant evidence that:

(1) An upset occurred and that the permittee can identify the cause(s) of the upset;

(2) The permitted facility was at the time being properly operated; and

(3) The permittee submitted notice of the upset as required in paragraph (12)(f)(2) (24-hour notice).

(4) The permittee complied with any remedial measures required under paragraph (4).

(d) Burden of proof. In any enforcement proceeding the permittee seeking to establish the occurrence of an upset has the burden of proof.

(15) **Transfer of permits**. Permits may be transferred by modification or automatic transfer as described below:

(a) Transfers by modification. Except as provided in paragraph (b), a permit may be transferred by the permittee to a new owner or operator only if the permit has been modified or revoked and reissued pursuant to 40 CFR 122.62 (b) (2), or a minor modification made pursuant to 40 CFR 122.63 (d), to identify the new permittee and incorporate such other requirements as may be necessary under the Clean Water Act.

(b) Automatic transfers. As an alternative to transfers under paragraph (a), any NPDES permit may be automatically transferred to a new permittee if:

(1) The current permittee notifies the Agency at least 30 days in advance of the proposed transfer date;

(2) The notice includes a written agreement between the existing and new permittees containing a specified date for transfer of permit responsibility, coverage and liability between the existing and new permittees; and

(3) The Agency does not notify the existing permittee and the proposed new permittee of its intent to modify or revoke and reissue the permit. If this notice is not received, the transfer is effective on the date specified in the agreement.

(16) All manufacturing, commercial, mining, and silvicultural dischargers must notify the Agency as soon as they know or have reason to believe:

(a) That any activity has occurred or will occur which would result in the discharge of any toxic pollutant identified under Section 307 of the Clean Water Act which is not limited in the permit, if that discharge will exceed the highest of the following notification levels:

(1) One hundred micrograms per liter (100 ug/l);

(2) Two hundred micrograms per liter (200 ug/l) for acrolein and acrylonitrile; five hundred micrograms per liter (500 ug/l) for 2,4-dinitrophenol and for 2-methyl-4,6 dinitrophenol; and one milligram per liter (1 mg/l) for antimony.

(3) Five (5) times the maximum concentration value reported for that pollutant in the NPDES permit application; or

(4) The level established by the Agency in this permit.

(b) That they have begun or expect to begin to use or manufacture as an intermediate or final product or byproduct any toxic pollutant which was not reported in the NPDES permit application.

(17) All Publicly Owned Treatment Works (POTWs) must provide adequate notice to the Agency of the following:

(a) Any new introduction of pollutants into that POTW from an indirect discharge which would be subject to Sections 301 or 306 of the Clean Water Act if it were directly discharging those pollutants; and

(b) Any substantial change in the volume or character of pollutants being introduced into that POTW by a source introducing pollutants into the POTW at the time of issuance of the permit.

(c) For purposes of this paragraph, adequate notice shall include information on (i) the quality and quantity of effluent introduced into the POTW, and (ii) any anticipated impact of the change on the quantity or quality of effluent to be discharged from the POTW.

(18) If the permit is issued to a publicly owned or publicly regulated treatment works, the permittee shall require any industrial user of such treatment works to comply with federal requirements concerning:

(a) User charges pursuant to Section 204 (b) of the Clean Water Act, and applicable regulations appearing in 40 CFR 35;

(b) Toxic pollutant effluent standards and pretreatment standards pursuant to Section 307 of the Clean Water Act; and

(c) Inspection, monitoring and entry pursuant to Section 308 of the Clean Water Act.

(Rev. 7-9-2010 bah)

(19) If an applicable standard or limitation is promulgated under Section 301(b)(2)(C) and (D), 304(b)(2), or 307(a)(2) and that effluent standard or limitation is more stringent than any effluent limitation in the permit, or controls a pollutant not limited in the permit, the permit shall be promptly modified or revoked, and reissued to conform to that effluent standard or limitation.

(20) Any authorization to construct issued to the permittee pursuant to 35 Ill. Adm. Code 309.154 is hereby incorporated by reference as a condition of this permit.

(21) The permittee shall not make any false statement, representation or certification in any application, record, report, plan or other document submitted to the Agency or the USEPA, or required to be maintained under this permit.

(22) The Clean Water Act provides that any person who violates a permit condition implementing Sections 301, 302, 306, 307, 308, 318, or 405 of the Clean Water Act is subject to a civil penalty not to exceed $25,000 per day of such violation. Any person who willfully or negligently violates permit conditions implementing Sections 301, 302, 306, 307, 308, 318 or 405 of the Clean Water Act is subject to a fine of not less than $2,500 nor more than $25,000 per day of violation, or by imprisonment for not more than one year, or both.
Additional penalties for violating these sections of the Clean Water Act are identified in 40 CFR 122.41 (a)(2) and (3).

(23) The Clean Water Act provides that any person who falsifies, tampers with, or knowingly renders inaccurate any monitoring device or method required to be maintained under this permit shall, upon conviction, be punished by a fine of not more than $10,000, or by imprisonment for not more than 2 years, or both. If a conviction of a person is for a violation committed after a first conviction of such person under this paragraph, punishment is a fine of not more than $20,000 per day of violation, or by imprisonment of not more than 4 years, or both.

(24) The Clean Water Act provides that any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit, including monitoring reports or reports of compliance or non-compliance shall, upon conviction, be punished by a fine of not more than $10,000 per violation, or by imprisonment for not more than 6 months per violation, or by both.

(25) Collected screening, slurries, sludges, and other solids shall be disposed of in such a manner as to prevent entry of those wastes (or runoff from the wastes) into waters of the State. The proper authorization for such disposal shall be obtained from the Agency and is incorporated as part hereof by reference.

(26) In case of conflict between these standard conditions and any other condition(s) included in this permit, the other condition(s) shall govern.

(27) The permittee shall comply with, in addition to the requirements of the permit, all applicable provisions of 35 Ill. Adm. Code, Subtitle C, Subtitle D, Subtitle E, and all applicable orders of the Board or any court with jurisdiction.

(28) The provisions of this permit are severable, and if any provision of this permit, or the application of any provision of this permit is held invalid, the remaining provisions of this permit shall continue in full force and effect.

APPENDIX B
MODELING SAMPLING PROTOCOL

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Memorandum

To: *Scott Trotter P.E., TAI Inc.*

From: *Lucas Botero, P.E., BCEE*

Date: *October 10, 2014*

Subject: *Process Simulation Modeling Sampling and Analysis Protocol - Final*
City of Saint Charles, IL
Main Wastewater Treatment Plant

CDM Smith's scope for the project includes the development of a plant process simulation model using BioWin. A level 2 calibration of the model will be conducted which is sufficient to complete the evaluation for the phosphorus removal study. The influent and effluent characterization would include two weeks of composite sampling and analysis to develop the appropriate site-specific influent COD, phosphorus, and applicable nitrogen fractions. This memorandum provides a protocol for the recommended sampling and analysis program.

Composite Sampling Locations and Analyses

The City will collect and analyze 24-hour flow-weighted composite samples from the *influent*, primary, and final effluent. The following parameters will be analyzed:

Influent (24-hour composites)	***Test Method***
▪ *Total chemical oxygen demand (COD)*	*SM 5220/EPA 410.4*

Primary Effluent (24-hour composites)	**Test Method**
▪ Total suspended solids (TSS)	SM 2540
▪ Volatile suspended solids (VSS)	SM2540
▪ Total chemical oxygen demand (COD)	SM 5220/EPA 410.4
▪ Acetate (VFA)	Ion chromatography (*)
▪ Glass fiber (1.2 µm) filtered chemical oxygen demand (gfCOD)	SM 5220/EPA 410.4

(*) IC / AM23G. Recommend diluting the samples by half to cover a 0.1 to 20 mg/L range. Collect and ship only 3 samples per each 7 day campaign.

- Flocculated and Filtered (0.45 µm) COD (ffCOD) (see below)
- Total (uninhibited) biochemical oxygen demand (BOD_5) SM 5210
- Carbonaceous biochemical oxygen demand ($cBOD_5$) SM 5210B
- Glass fiber (1.2 µm) filtered (uninhibited) BOD_5 ($gfBOD_5$) SM 5210
- Total phosphorus (TP) SM 4500P
- Orthophosphate (Ortho-P) SM 4500P
- Total Kjeldahl Nitrogen (TKN) SM 4500N
- Filtered TKN SM 4500N
- Ammonia-N SM 4500N
- Nitrate EPA 353.2
- pH SM 4500H^+
- Alkalinity SM 2320

Final Effluent (24-hr composites): **Test Method**

- Total suspended solids SM 2540
- Glass fiber filtered (1.2 µm) chemical oxygen demand SM 5220/EPA 410.4
- Total Kjeldahl Nitrogen SM 4500N
- Filtered TKN SM 4500N
- Ammonia-N SM 4500N
- Nitrite/Nitrate-N EPA 353.2
- Total phosphorus SM 4500P
- Orthophosphate SM 4500P
- pH SM 4500H^+

- Alkalinity SM 2320

Primary effluent results will be used in the process modeling to simplify the model configuration. Primary effluent will be used since it includes all in-plant sidestreams. Secondary effluent results will be used to complete the COD fractionation for the model.

The following WERF (2003) procedure shall be used for the ffCOD analyses in the primary effluent:

1. *1 mL of 100 g/L zinc sulfate solution is added to 100 mL of wastewater.*
2. *The sample is then mixed vigorously for approximately 1 minute.*
3. *The sample pH is adjusted to approximately 10.5 using 6 M sodium hydroxide solution.*
4. *The sample then is allowed to settle, and a sample of the supernatant is withdrawn*
5. *The supernatant sample is filtered using a 0.45 μm membrane filter, and the filtrate COD is analyzed (according to SM 5220 or EPA 410.4).*

Composite Sampling Procedure

Composite sampler samples should be collected in clean composite containers, rinsed of growth and accumulated solids. Sampler hoses should be checked for buildup and replaced if fouled, and should be of sufficient diameter. Hoses should also be placed at a representative location within a well-mixed channel or launder. Solids can be stratified within a channel with insufficient velocity or mixing. Composite containers should be shaken vigorously immediately prior to pouring to create the combined primary effluent composite and prior to pouring into each laboratory bottle. Solids can settle quickly following shaking but before pouring, and may bias results. Composite containers should be refrigerated.

Process Control Data

The City will compile facility process control data in Excel format which will include:

- Primary sludge flow and concentration.
- Mixed liquor suspended solids concentration (MLSS) and mixed liquor volatile suspended solids (MLVSS) data.
- Return and waste activated sludge flows.
- Return activated sludge concentration.
- Thickened sludge flow and concentrations.
- Digester volatile suspended solids reduction.

- Dewatered sludge concentrations.

Supplemental Grab Sampling

Supplemental grab sampling of sidestreams would be beneficial to assist in the evaluation. Samples are recommended to be taken twice per day during periods when biosolids are being processed. Recommended sample type, location, and frequency are denoted below:

WAS (*)

- Total phosphorus — SM 4500P

(*) If WAS grab sampling is not possible, then take the sample from the MLSS from the effluent channel at the aeration tanks.

WAS Storage Tank (close to effluent)

- Total phosphorus — SM 4500P
- Orthophosphate — SM 4500P

Anaerobic Digester

- Ammonia-N — SM 4500N
- Total phosphorus — SM 4500P
- Orthophosphate — SM 4500P

APPENDIX C
ST CHARLES A^2/O PROCESS BIOWIN MODEL REPORT

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St Charles WWTP

BioWin User and Configuration Data

Project details

Project name: St Charles **Project Number: 105971**

Plant name: Main WWTP **User name: Lucas Botero/Timur Deniz**

Created: 12/23/2014 **Saved: 2/26/2015**

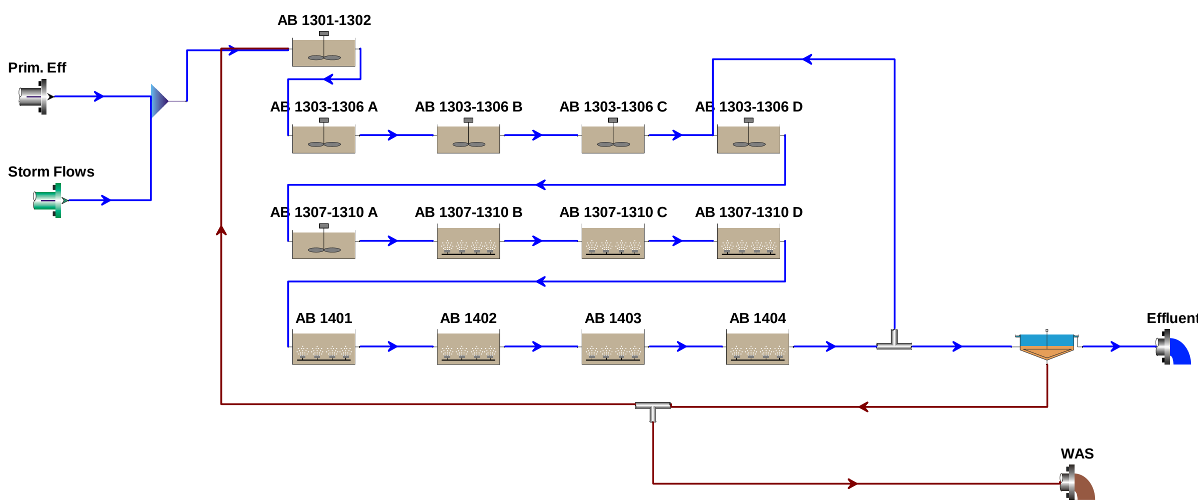
Steady state solution

AerSRT : 9.5 days

Total SRT : 17.4 days

Temperature: 9.0°C

Flowsheet



Configuration information for all Bioreactor units

Physical data

Element name	Volume [m3]	Area [m2]	Depth [m]	Aeration
AB 1301-1302	698.0000	140.1606	4.980	Un-aerated
AB 1303-1306 A	1271.0000	262.0619	4.850	Un-aerated
AB 1303-1306 B	1271.0000	262.0619	4.850	Un-aerated
AB 1307-1310 A	493.0000	102.7083	4.800	Un-aerated
AB 1307-1310 B	1494.0000	311.2500	4.800	Aerated
AB 1303-1306 C	1271.0000	262.0619	4.850	Un-aerated
AB 1303-1306 D	1160.0000	239.1753	4.850	Un-aerated
AB 1307-1310 C	1494.0000	311.2500	4.800	Aerated
AB 1307-1310 D	1494.0000	311.2500	4.800	Aerated
AB 1401	950.0000	193.4827	4.910	Aerated
AB 1402	950.0000	193.8776	4.900	Aerated
AB 1403	499.0000	101.8367	4.900	Aerated
AB 1404	499.0000	103.3126	4.830	Aerated

Operating data Average (flow/time weighted as required)

Element name	Average DO Setpoint [mg/L]
AB 1301-1302	0
AB 1303-1306 A	0
AB 1303-1306 B	0
AB 1307-1310 A	0
AB 1307-1310 B	2.0
AB 1303-1306 C	0
AB 1303-1306 D	0
AB 1307-1310 C	2.0
AB 1307-1310 D	2.0
AB 1401	2.0
AB 1402	2.0
AB 1403	2.0
AB 1404	0.5

Aeration equipment parameters

Element name	k1 in C = k1(PC)^0.25 + k2	k2 in C = k1(PC)^0.25 + k2	Y in Kla = C Usg ^ Y - Usg in [m3/(m2 d)]	Area of one diffuser	% of tank area covered by diffusers [%]
AB 1301-1302	2.5656	0.0432	0.8200	0.0410	10.0000
AB 1303-1306 A	2.5656	0.0432	0.8200	0.0410	10.0000
AB 1303-1306 B	2.5656	0.0432	0.8200	0.0410	10.0000
AB 1307-1310 A	2.5656	0.0432	0.8200	0.0410	10.0000
AB 1307-1310 B	2.5656	0.0432	0.8200	0.0410	10.0000
AB 1303-1306 C	2.5656	0.0432	0.8200	0.0410	10.0000
AB 1303-1306 D	2.5656	0.0432	0.8200	0.0410	10.0000
AB 1307-1310 C	2.5656	0.0432	0.8200	0.0410	10.0000
AB 1307-1310 D	2.5656	0.0432	0.8200	0.0410	10.0000
AB 1401	2.5656	0.0432	0.8200	0.0410	10.0000
AB 1402	2.5656	0.0432	0.8200	0.0410	10.0000
AB 1403	2.5656	0.0432	0.8200	0.0410	10.0000
AB 1404	2.5656	0.0432	0.8200	0.0410	10.0000

Configuration information for all Effluent units

Configuration information for all Ideal clarifier units

Physical data

Element name	Volume [m3]	Area [m2]	Depth [m]
Sec. Clarifiers	8164.9062	2101.0000	3.886

Operating data Average (flow/time weighted as required)

Element name	Split method	Average Split specification
Sec. Clarifiers	Flow paced	50.00 %

Element name	Average Temperature	Reactive	Percent removal	Blanket fraction
Sec. Clarifiers	Uses global setting	No	99.90	0.05

Configuration information for all COD Influent units

Operating data Average (flow/time weighted as required)

Element name	Prim. Eff
Time	0
Flow	34065
Total COD mgCOD/L	243.50
Total Kjeldahl Nitrogen mgN/L	27.74
Total P mgP/L	5.45
Nitrate N mgN/L	0
pH	7.47
Alkalinity mmol/L	8.74
ISS Influent mgISS/L	10.00
Calcium mg/L	80.00
Magnesium mg/L	15.00
Dissolved oxygen mg/L	0

Element name	Prim. Eff
Fbs - Readily biodegradable (including Acetate) [gCOD/g of total COD]	0.3210
Fac - Acetate [gCOD/g of readily biodegradable COD]	0.1420
Fxsp - Non-colloidal slowly biodegradable [gCOD/g of slowly degradable COD]	0.7300
Fus - Unbiodegradable soluble [gCOD/g of total COD]	0.0820
Fup - Unbiodegradable particulate [gCOD/g of total COD]	0.0750
Fna - Ammonia [gNH3-N/gTKN]	0.7300
Fnox - Particulate organic nitrogen [gN/g Organic N]	0.2500
Fnus - Soluble unbiodegradable TKN [gN/gTKN]	0.0200
FupN - N:COD ratio for unbiodegradable part. COD [gN/gCOD]	0.0350
Fpo4 - Phosphate [gPO4-P/gTP]	0.7980
FupP - P:COD ratio for unbiodegradable part. COD [gP/gCOD]	0.0110
FZbh - OHO COD fraction [gCOD/g of total COD]	0.0100
FZbm - Methylotroph COD fraction [gCOD/g of total COD]	1.000E-4
FZaob - AOB COD fraction [gCOD/g of total COD]	1.000E-4
FZnob - NOB COD fraction [gCOD/g of total COD]	1.000E-4
FZaao - AAO COD fraction [gCOD/g of total COD]	1.000E-4

FZbp - PAO COD fraction [gCOD/g of total COD]	1.000E-4
FZbpa - Propionic acetogens COD fraction [gCOD/g of total COD]	1.000E-4
FZbam - Acetoclastic methanogens COD fraction [gCOD/g of total COD]	1.000E-4
FZbhm - H2-utilizing methanogens COD fraction [gCOD/g of total COD]	1.000E-4
FZe - Endogenous products COD fraction [gCOD/g of total COD]	0

Configuration information for all Sludge units

Configuration information for all Splitter units

Operating data Average (flow/time weighted as required)

Element name	Split method	Average Split specification
WAS Splitter	Flowrate [Side]	260
Splitter44	Flow paced	200.00 %

Configuration information for all Stream (SV) Influent units

Operating data Average (flow/time weighted as required)

Element name	Storm Flows
Ordinary heterotrophic organisms (OHO) mgCOD/L	0
Methylotrophs mgCOD/L	0
Ammonia oxidizing biomass (AOB) mgCOD/L	0
Nitrite oxidizing biomass (NOB) mgCOD/L	0
Anaerobic ammonia oxidizers (AAO) mgCOD/L	0
Polyphosphate accumulating organisms (PAO) mgCOD/L	0
Propionic acetogens mgCOD/L	0
Methanogens - acetoclastic mgCOD/L	0
Methanogens - hydrogenotrophic mgCOD/L	0
Endogenous products mgCOD/L	0
Slowly bio. COD (part.) mgCOD/L	0
Slowly bio. COD (colloid.) mgCOD/L	0
Part. inert. COD mgCOD/L	0

Part. bio. org. N mgN/L	0
Part. bio. org. P mgP/L	0
Part. inert N mgN/L	0
Part. inert P mgP/L	0
Stored PHA mgCOD/L	0
Releasable stored polyP mgP/L	0
Fixed stored polyP mgP/L	0
Readily bio. COD (complex) mgCOD/L	0
Acetate mgCOD/L	0
Propionate mgCOD/L	0
Methanol mgCOD/L	0
Dissolved H2 mgCOD/L	0
Dissolved methane mg/L	0
Ammonia N mgN/L	0
Sol. bio. org. N mgN/L	0
Nitrous Oxide N mgN/L	0
Nitrite N mgN/L	0
Nitrate N mgN/L	0
Dissolved nitrogen gas mgN/L	0
PO4-P (Sol. & Me Complexed) mgP/L	0
Sol. inert COD mgCOD/L	0
Sol. inert TKN mgN/L	0
ISS Influent mgISS/L	0
Struvite mgISS/L	0
Hydroxy-dicalcium-phosphate mgISS/L	0
Hydroxy-apatite mgISS/L	0
Magnesium mg/L	0
Calcium mg/L	0
Metal mg/L	0
Other Cations (strong bases) meq/L	0
Other Anions (strong acids) meq/L	0
Total CO2 mmol/L	0
User defined 1 mg/L	0
User defined 2 mg/L	0
User defined 3 mgVSS/L	0

User defined 4 mgISS/L	0
Dissolved oxygen mg/L	0
Flow	1

BioWin Album

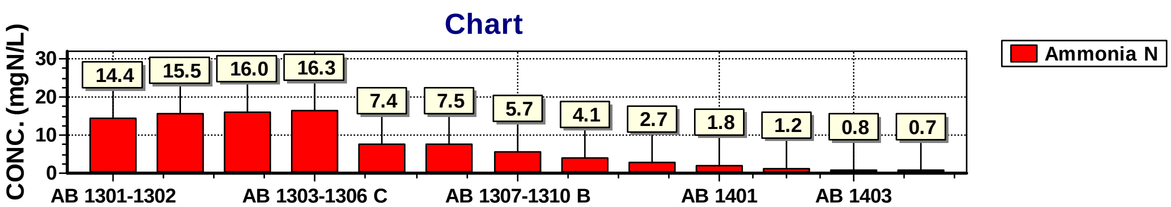
Album page - Tables

Elements	Total suspended solids [mgTSS/L]	Volatile suspended solids [mgVSS/L]
AB 1404	3324.71	2329.44

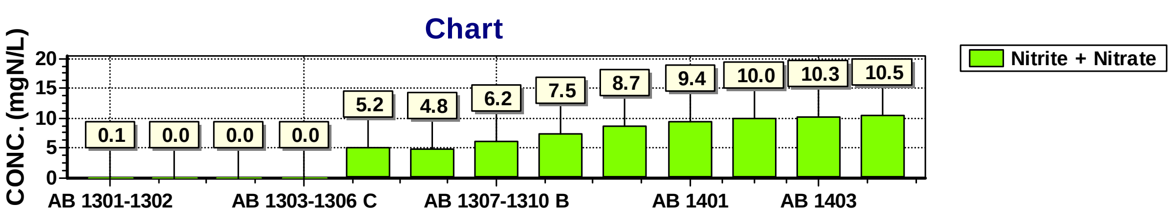
Album page - Tables

Elements	Total suspended solids [kg TSS/d]	Volatile suspended solids [kg VSS/d]
WAS	2577.55	1805.95

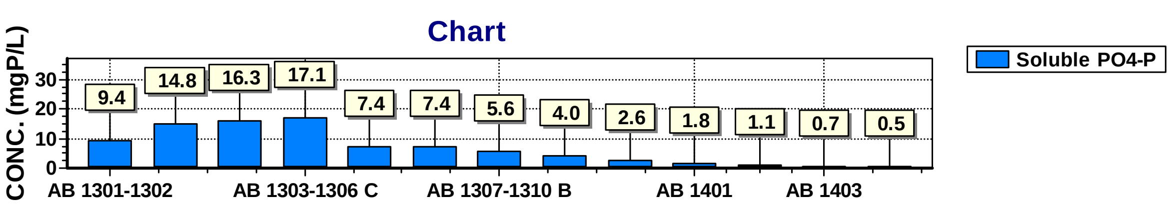
Album page - Nut Prof



Album page - Nut Prof



Album page - Nut Prof



Album page - Eff Conc.

Elements	Total COD [mg/L]	Total Carbonaceous BOD [mg/L]	Total P [mgP/L]	Soluble PO4-P [mgP/L]	Total N [mgN/L]	Ammonia N [mgN/L]	Nitrate N [mgN/L]	Nitrite N [mgN/L]	Total suspended solids [mgTSS/L]	Volatile suspended solids [mgVSS/L]
Effluent	28.48	2.61	0.79	0.48	12.93	0.66	8.57	1.89	5.00	3.50

Global Parameters

Common

Name	Default	Value	
Hydrolysis rate [1/d]	2.1000	2.1000	1.0290
Hydrolysis half sat. [-]	0.0600	0.0600	1.0000
Anoxic hydrolysis factor [-]	0.2800	0.2800	1.0000
Anaerobic hydrolysis factor (AS) [-]	0.0400	0.0400	1.0000
Anaerobic hydrolysis factor (AD) [-]	0.5000	0.5000	1.0000
Adsorption rate of colloids [L/(mgCOD d)]	0.1500	0.1500	1.0290
Ammonification rate [L/(mgN d)]	0.0400	0.0400	1.0290
Assimilative nitrate/nitrite reduction rate [1/d]	0.5000	0.5000	1.0000
Endogenous products decay rate [1/d]	0	0	1.0000

AOB

Name	Default	Value	
Max. spec. growth rate [1/d]	0.9000	0.9000	1.0720
Substrate (NH4) half sat. [mgN/L]	0.7000	0.7000	1.0000
Byproduct NH4 logistic slope [-]	50.0000	50.0000	1.0000
Byproduct NH4 inflection point [mgN/L]	1.4000	1.4000	1.0000
AOB denite DO half sat. [mg/L]	0.1000	0.1000	1.0000
AOB denite HNO2 half sat. [mgN/L]	5.000E-6	5.000E-6	1.0000
Aerobic decay rate [1/d]	0.1700	0.1700	1.0290
Anoxic/anaerobic decay rate [1/d]	0.0800	0.0800	1.0290
KiHNO2 [mmol/L]	0.0050	0.0050	1.0000

NOB

Name	Default	Value

Max. spec. growth rate [1/d]	0.7000	0.7000	1.0600
Substrate (NO2) half sat. [mgN/L]	0.1000	0.1000	1.0000
Aerobic decay rate [1/d]	0.1700	0.1700	1.0290
Anoxic/anaerobic decay rate [1/d]	0.0800	0.0800	1.0290
KiNH3 [mmol/L]	0.0750	0.0750	1.0000

AAO

Name	Default	Value	
Max. spec. growth rate [1/d]	0.2000	0.2000	1.1000
Substrate (NH4) half sat. [mgN/L]	2.0000	2.0000	1.0000
Substrate (NO2) half sat. [mgN/L]	1.0000	1.0000	1.0000
Aerobic decay rate [1/d]	0.0190	0.0190	1.0290
Anoxic/anaerobic decay rate [1/d]	0.0095	0.0095	1.0290
Ki Nitrite [mgN/L]	1000.0000	1000.0000	1.0000
Nitrite sensitivity constant [L / (d mgN)]	0.0160	0.0160	1.0000

OHO

Name	Default	Value	
Max. spec. growth rate [1/d]	3.2000	3.2000	1.0290
Substrate half sat. [mgCOD/L]	5.0000	5.0000	1.0000
Anoxic growth factor [-]	0.5000	0.5000	1.0000
Denite N2 producers (NO3 or NO2) [-]	0.5000	0.5000	1.0000
Aerobic decay rate [1/d]	0.6200	0.6200	1.0290
Anoxic decay rate [1/d]	0.2330	0.2330	1.0290
Anaerobic decay rate [1/d]	0.1310	0.1310	1.0290
Fermentation rate [1/d]	1.6000	1.6000	1.0290
Fermentation half sat. [mgCOD/L]	5.0000	5.0000	1.0000
Fermentation growth factor (AS) [-]	0.2500	0.2500	1.0000
Free nitrous acid inhibition [mmol/L]	1.000E-7	1.000E-7	1.0000

Methylotrophs

Name	Default	Value	
Max. spec. growth rate [1/d]	1.3000	1.3000	1.0720

Methanol half sat. [mgCOD/L]	0.5000	0.5000	1.0000
Denite N2 producers (NO3 or NO2) [-]	0.5000	0.5000	1.0000
Aerobic decay rate [1/d]	0.0400	0.0400	1.0290
Anoxic/anaerobic decay rate [1/d]	0.0300	0.0300	1.0000
Free nitrous acid inhibition [mmol/L]	1.000E-7	1.000E-7	1.0000

PAO

Name	Default	Value	
Max. spec. growth rate [1/d]	0.9500	0.9500	1.0000
Max. spec. growth rate, P-limited [1/d]	0.4200	0.4200	1.0000
Substrate half sat. [mgCOD(PHB)/mgCOD(Zbp)]	0.1000	0.1000	1.0000
Substrate half sat., P-limited [mgCOD(PHB)/mgCOD(Zbp)]	0.0500	0.0500	1.0000
Magnesium half sat. [mgMg/L]	0.1000	0.1000	1.0000
Cation half sat. [mmol/L]	0.1000	0.1000	1.0000
Calcium half sat. [mgCa/L]	0.1000	0.1000	1.0000
Aerobic/anoxic decay rate [1/d]	0.1000	0.1000	1.0000
Aerobic/anoxic maintenance rate [1/d]	0	0	1.0000
Anaerobic decay rate [1/d]	0.0400	0.0400	1.0000
Anaerobic maintenance rate [1/d]	0	0	1.0000
Sequestration rate [1/d]	4.5000	4.5000	1.0000
Anoxic growth factor [-]	0.3300	0.3300	1.0000

Acetogens

Name	Default	Value	
Max. spec. growth rate [1/d]	0.2500	0.2500	1.0290
Substrate half sat. [mgCOD/L]	10.0000	10.0000	1.0000
Acetate inhibition [mgCOD/L]	10000.0000	10000.0000	1.0000
Anaerobic decay rate [1/d]	0.0500	0.0500	1.0290
Aerobic/anoxic decay rate [1/d]	0.5200	0.5200	1.0290

Methanogens

Name	Default	Value	
Acetoclastic max. spec. growth rate [1/d]	0.3000	0.3000	1.0290

H2-utilizing max. spec. growth rate [1/d]	1.4000	1.4000	1.0290
Acetoclastic substrate half sat. [mgCOD/L]	100.0000	100.0000	1.0000
Acetoclastic methanol half sat. [mgCOD/L]	0.5000	0.5000	1.0000
H2-utilizing CO2 half sat. [mmol/L]	0.1000	0.1000	1.0000
H2-utilizing substrate half sat. [mgCOD/L]	0.1000	0.1000	1.0000
H2-utilizing methanol half sat. [mgCOD/L]	0.5000	0.5000	1.0000
Acetoclastic propionic inhibition [mgCOD/L]	10000.0000	10000.0000	1.0000
Acetoclastic anaerobic decay rate [1/d]	0.1300	0.1300	1.0290
Acetoclastic aerobic/anoxic decay rate [1/d]	0.6000	0.6000	1.0290
H2-utilizing anaerobic decay rate [1/d]	0.1300	0.1300	1.0290
H2-utilizing aerobic/anoxic decay rate [1/d]	2.8000	2.8000	1.0290

pH

Name	Default	Value
OHO low pH limit [-]	4.0000	4.0000
OHO high pH limit [-]	10.0000	10.0000
Methylotrophs low pH limit [-]	4.0000	4.0000
Methylotrophs high pH limit [-]	10.0000	10.0000
Autotrophs low pH limit [-]	5.5000	5.5000
Autotrophs high pH limit [-]	9.5000	9.5000
PAO low pH limit [-]	4.0000	4.0000
PAO high pH limit [-]	10.0000	10.0000
OHO low pH limit (anaerobic) [-]	5.5000	5.5000
OHO high pH limit (anaerobic) [-]	8.5000	8.5000
Propionic acetogens low pH limit [-]	4.0000	4.0000
Propionic acetogens high pH limit [-]	10.0000	10.0000
Acetoclastic methanogens low pH limit [-]	5.0000	5.0000
Acetoclastic methanogens high pH limit [-]	9.0000	9.0000
H2-utilizing methanogens low pH limit [-]	5.0000	5.0000
H2-utilizing methanogens high pH limit [-]	9.0000	9.0000

Switches

Name	Default	Value
Aerobic/anoxic DO half sat. [mgO2/L]	0.0500	0.0500

Anoxic/anaerobic NOx half sat. [mgN/L]	0.1500	0.1500
AOB DO half sat. [mgO2/L]	0.2500	0.2500
NOB DO half sat. [mgO2/L]	0.5000	0.5000
AAO DO half sat. [mgO2/L]	0.0100	0.0100
Anoxic NO3(->NO2) half sat. [mgN/L]	0.1000	0.1000
Anoxic NO3(->N2) half sat. [mgN/L]	0.0500	0.0500
Anoxic NO2(->N2) half sat. (mgN/L)	0.0100	0.0100
NH3 nutrient half sat. [mgN/L]	0.0050	0.0050
PolyP half sat. [mgP/mgCOD]	0.0100	0.0100
VFA sequestration half sat. [mgCOD/L]	5.0000	5.0000
P uptake half sat. [mgP/L]	0.1500	0.1500
P nutrient half sat. [mgP/L]	0.0010	0.0010
Autotroph CO2 half sat. [mmol/L]	0.1000	0.1000
H2 low/high half sat. [mgCOD/L]	1.0000	1.0000
Propionic acetogens H2 inhibition [mgCOD/L]	5.0000	5.0000
Synthesis anion/cation half sat. [meq/L]	0.0100	0.0100

Common

Name	Default	Value
Biomass volatile fraction (VSS/TSS)	0.9200	0.9200
Endogenous residue volatile fraction (VSS/TSS)	0.9200	0.9200
N in endogenous residue [mgN/mgCOD]	0.0700	0.0700
P in endogenous residue [mgP/mgCOD]	0.0220	0.0220
Endogenous residue COD:VSS ratio [mgCOD/mgVSS]	1.4200	1.4200
Particulate substrate COD:VSS ratio [mgCOD/mgVSS]	1.6000	1.7400
Particulate inert COD:VSS ratio [mgCOD/mgVSS]	1.6000	1.7400

AOB

Name	Default	Value
Yield [mgCOD/mgN]	0.1500	0.1500
AOB denite NO2 fraction as TEA [-]	0.5000	0.5000
Byproduct NH4 fraction to N2O [-]	0.0025	0.0025
N in biomass [mgN/mgCOD]	0.0700	0.0700
P in biomass [mgP/mgCOD]	0.0220	0.0220

Fraction to endogenous residue [-]	0.0800	0.0800
COD:VSS ratio [mgCOD/mgVSS]	1.4200	1.4200

NOB

Name	Default	Value
Yield [mgCOD/mgN]	0.0900	0.0900
N in biomass [mgN/mgCOD]	0.0700	0.0700
P in biomass [mgP/mgCOD]	0.0220	0.0220
Fraction to endogenous residue [-]	0.0800	0.0800
COD:VSS ratio [mgCOD/mgVSS]	1.4200	1.4200

AAO

Name	Default	Value
Yield [mgCOD/mgN]	0.1140	0.1140
Nitrate production [mgN/mgBiomassCOD]	2.2800	2.2800
N in biomass [mgN/mgCOD]	0.0700	0.0700
P in biomass [mgP/mgCOD]	0.0220	0.0220
Fraction to endogenous residue [-]	0.0800	0.0800
COD:VSS ratio [mgCOD/mgVSS]	1.4200	1.4200

OHO

Name	Default	Value
Yield (aerobic) [-]	0.6660	0.6660
Yield (fermentation, low H2) [-]	0.1000	0.1000
Yield (fermentation, high H2) [-]	0.1000	0.1000
H2 yield (fermentation low H2) [-]	0.3500	0.3500
H2 yield (fermentation high H2) [-]	0	0
Propionate yield (fermentation, low H2) [-]	0	0
Propionate yield (fermentation, high H2) [-]	0.7000	0.7000
CO2 yield (fermentation, low H2) [-]	0.7000	0.7000
CO2 yield (fermentation, high H2) [-]	0	0
N in biomass [mgN/mgCOD]	0.0700	0.0700
P in biomass [mgP/mgCOD]	0.0220	0.0220

Endogenous fraction - aerobic [-]	0.0800	0.0800
Endogenous fraction - anoxic [-]	0.1030	0.1030
Endogenous fraction - anaerobic [-]	0.1840	0.1840
COD:VSS ratio [mgCOD/mgVSS]	1.4200	1.4200
Yield (anoxic) [-]	0.5400	0.5400
Yield propionic (aerobic) [-]	0.6400	0.6400
Yield propionic (anoxic) [-]	0.4600	0.4600
Yield acetic (aerobic) [-]	0.6000	0.6000
Yield acetic (anoxic) [-]	0.4300	0.4300
Yield methanol (aerobic) [-]	0.5000	0.5000
Adsorp. max. [-]	1.0000	1.0000
Max fraction to N2O at high FNA over nitrate [-]	0.0500	0.0500
Max fraction to N2O at high FNA over nitrite [-]	0.1000	0.1000

Methylotrophs

Name	Default	Value
Yield (anoxic) [-]	0.4000	0.4000
N in biomass [mgN/mgCOD]	0.0700	0.0700
P in biomass [mgP/mgCOD]	0.0220	0.0220
Fraction to endogenous residue [-]	0.0800	0.0800
COD:VSS ratio [mgCOD/mgVSS]	1.4200	1.4200
Max fraction to N2O at high FNA over nitrate [-]	0.1000	0.1000
Max fraction to N2O at high FNA over nitrite [-]	0.1500	0.1500

PAO

Name	Default	Value
Yield (aerobic) [-]	0.6390	0.6390
Yield (anoxic) [-]	0.5200	0.5200
Aerobic P/PHA uptake [mgP/mgCOD]	0.9300	0.9300
Anoxic P/PHA uptake [mgP/mgCOD]	0.3500	0.3500
Yield of PHA on sequestration [-]	0.8890	0.8890
N in biomass [mgN/mgCOD]	0.0700	0.0700
N in sol. inert [mgN/mgCOD]	0.0700	0.0700
P in biomass [mgP/mgCOD]	0.0220	0.0220

Fraction to endogenous part. [-]	0.2500	0.2500
Inert fraction of endogenous sol. [-]	0.2000	0.2000
P/Ac release ratio [mgP/mgCOD]	0.5100	0.5100
COD:VSS ratio [mgCOD/mgVSS]	1.4200	1.4200
Yield of low PP [-]	0.9400	0.9400

Acetogens

Name	Default	Value
Yield [-]	0.1000	0.1000
H2 yield [-]	0.4000	0.4000
CO2 yield [-]	1.0000	1.0000
N in biomass [mgN/mgCOD]	0.0700	0.0700
P in biomass [mgP/mgCOD]	0.0220	0.0220
Fraction to endogenous residue [-]	0.0800	0.0800
COD:VSS ratio [mgCOD/mgVSS]	1.4200	1.4200

Methanogens

Name	Default	Value
Acetoclastic yield [-]	0.1000	0.1000
Methanol acetoclastic yield [-]	0.1000	0.1000
H2-utilizing yield [-]	0.1000	0.1000
Methanol H2-utilizing yield [-]	0.1000	0.1000
N in acetoclastic biomass [mgN/mgCOD]	0.0700	0.0700
N in H2-utilizing biomass [mgN/mgCOD]	0.0700	0.0700
P in acetoclastic biomass [mgP/mgCOD]	0.0220	0.0220
P in H2-utilizing biomass [mgP/mgCOD]	0.0220	0.0220
Acetoclastic fraction to endog. residue [-]	0.0800	0.0800
H2-utilizing fraction to endog. residue [-]	0.0800	0.0800
Acetoclastic COD:VSS ratio [mgCOD/mgVSS]	1.4200	1.4200
H2-utilizing COD:VSS ratio [mgCOD/mgVSS]	1.4200	1.4200

General

Name	Default	Value
Molecular weight of other anions [mg/mmol]	35.5000	35.5000
Molecular weight of other cations [mg/mmol]	39.1000	39.1000
Mg to P mole ratio in polyphosphate [mmolMg/mmolP]	0.3000	0.3000
Cation to P mole ratio in polyphosphate [meq/mmolP]	0.1500	0.1500
Ca to P mole ratio in polyphosphate [mmolCa/mmolP]	0.0500	0.0500
Cation to P mole ratio in organic phosphate [meq/mmolP]	0.0100	0.0100
Bubble rise velocity (anaerobic digester) [cm/s]	23.9000	23.9000
Bubble Sauter mean diameter (anaerobic digester) [cm]	0.3500	0.3500
Anaerobic digester gas hold-up factor []	1.0000	1.0000
Tank head loss per metre of length (from flow) [m/m]	0.0025	0.0025

Mass transfer

Name	Default	Value	
Kl for H2 [m/d]	17.0000	17.0000	1.0240
Kl for CO2 [m/d]	10.0000	10.0000	1.0240
Kl for NH3 [m/d]	1.0000	1.0000	1.0240
Kl for CH4 [m/d]	8.0000	8.0000	1.0240
Kl for N2 [m/d]	15.0000	15.0000	1.0240
Kl for N2O [m/d]	8.0000	8.0000	1.0240
Kl for O2 [m/d]	13.0000	13.0000	1.0240

Henry's law constants

Name	Default	Value	
CO2 [M/atm]	3.4000E-2	3.4000E-2	2400.0000
O2 [M/atm]	1.3000E-3	1.3000E-3	1500.0000
N2 [M/atm]	6.5000E-4	6.5000E-4	1300.0000
N2O [M/atm]	2.5000E-2	2.5000E-2	2600.0000
NH3 [M/atm]	5.8000E+1	5.8000E+1	4100.0000
CH4 [M/atm]	1.4000E-3	1.4000E-3	1600.0000
H2 [M/atm]	7.8000E-4	7.8000E-4	500.0000

Physico-chemical rates

Name	Default	Value	
Struvite precipitation rate [1/d]	3.000E+10	3.000E+10	1.0240
Struvite redissolution rate [1/d]	3.000E+11	3.000E+11	1.0240
Struvite half sat. [mgTSS/L]	1.0000	1.0000	1.0000
HDP precipitation rate [L/(molP d)]	1.000E+8	1.000E+8	1.0000
HDP redissolution rate [L/(mol P d)]	1.000E+8	1.000E+8	1.0000
HAP precipitation rate [molHDP/(L d)]	5.000E-4	5.000E-4	1.0000

Physico-chemical constants

Name	Default	Value
Struvite solubility constant [mol/L]	6.918E-14	6.918E-14
HDP solubility product [mol/L]	2.750E-22	2.750E-22
HDP half sat. [mgTSS/L]	1.0000	1.0000
Equilibrium soluble PO4 with Al dosing at pH 7 [mgP/L]	0.0100	0.0100
Al to P ratio [molAl/molP]	0.8000	0.8000
Al(OH)3 solubility product [mol/L]	1.259E+9	1.259E+9
AlHPO4+ dissociation constant [mol/L]	7.943E-13	7.943E-13
Equilibrium soluble PO4 with Fe dosing at pH 7 [mgP/L]	0.0100	0.0100
Fe to P ratio [molFe/molP]	1.6000	1.6000
Fe(OH)3 solubility product [mol/L]	0.0500	0.0500
FeH2PO4++ dissociation constant [mol/L]	5.012E-22	5.012E-22

Aeration

Name	Default	Value
Alpha (surf) OR Alpha F (diff) [-]	0.5000	0.5000
Beta [-]	0.9500	0.9500
Surface pressure [kPa]	101.3250	101.3250
Fractional effective saturation depth (Fed) [-]	0.3250	0.3250
Supply gas CO2 content [vol. %]	0.0350	0.0350
Supply gas O2 [vol. %]	20.9500	20.9500
Off-gas CO2 [vol. %]	2.0000	2.0000
Off-gas O2 [vol. %]	18.8000	18.8000
Off-gas H2 [vol. %]	0	0

Off-gas NH3 [vol. %]	0	0
Off-gas CH4 [vol. %]	0	0
Surface turbulence factor [-]	2.0000	2.0000
Set point controller gain []	1.0000	1.0000

Modified Vesilind

Name	Default	Value
Maximum Vesilind settling velocity (Vo) [m/d]	170.000	170.000
Vesilind hindered zone settling parameter (K) [L/g]	0.370	0.370
Clarification switching function [mg/L]	100.000	100.000
Specified TSS conc.for height calc. [mg/L]	2500.000	2500.000
Maximum compactability constant [mg/L]	15000.000	15000.000

Double exponential

Name	Default	Value
Maximum Vesilind settling velocity (Vo) [m/d]	410.000	410.000
Maximum (practical) settling velocity (Vo') [m/d]	270.000	270.000
Hindered zone settling parameter (Kh) [L/g]	0.400	0.400
Flocculent zone settling parameter (Kf) [L/g]	2.500	2.500
Maximum non-settleable TSS [mg/L]	20.0000	20.0000
Non-settleable fraction [-]	0.0010	0.0010
Specified TSS conc. for height calc. [mg/L]	2500.0000	2500.0000

Emission factors

Name	Default	Value
Carbon dioxide equivalence of nitrous oxide	296.0000	296.0000
Carbon dioxide equivalence of methane	23.0000	23.0000

APPENDIX D
JAR TEST PROTOCOL TP

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CDM Smith

Memorandum

To: *Scott Trotter, TAI Inc.*

From: *Lucas Botero, P.E. BCEE*

Date: *October 10, 2014*

Subject: *Chemical Phosphorus Removal Jar Testing Protocol - Final*
City of St. Charles
Wastewater Treatment Facilities

1.0 Jar Testing Objectives

Bench-scale jar tests will be performed in the laboratory to determine the following items for the evaluation of phosphorus removing alternatives and the future detail design of the recommended alternative.

- Confirm chemical usage with respect to optimizing phosphorus removal.
- Confirm the solids production of the tested metal salts.
- Determine the SUP (soluble unreactive phosphorus) fraction of the St. Charles WWTF influent stream
- Determine possible impacts of ferrous salts to the UV transmittance

It is anticipated that a total of 12 jar tests will be performed at each facility to develop the information necessary to evaluate TP as a function of metal salt (coagulant) dose. Both aluminum sulfate and ferric chloride are to be evaluated for their phosphorus removal effectiveness. The jar testing is to be conducted by TAI staff and the CITY will arrange for and cover expenses for any laboratory analyses.

It is important to note that this jar testing protocol has not been designed to evaluate mixing kinetics as they relate to phosphorus removal at either facility. It is recommended that mixing is evaluated during the detail design phase of the phosphorus removal implementation because in poorly mixed systems phosphorus removal can be decreased as much as 25-percent compared to well-mixed systems (Smith et al., 2008). Clearly, this factor has an impact on both chemical costs, and sludge production and handling.

2.0 Metal Salts

There are several metals salts that are effective removing phosphorus in domestic wastewater streams. Selection of metal salt is project specific with great importance given to the operation cost of the selected salt but there are also other importance non-economic considerations including:

- Metal salt handling hazards
- Commonality with other plant or Utility processes
- Storage requirements (especially in cold climates)
- Metal salts shelf life (important if seasonal TP limits are in effect)
- Impact to plant's processes

The selection of the metal salt to be used at the St. Charles WWTF will be discussed in the study report. However, CDM Smith recommends testing the most commonly used metals salts for TP removal: ferric chloride and alum, as they have proven to be the most cost beneficial alternatives for most plants as a starting point.

3.0 Jar Test Protocol

Metal salts (including ferric chloride, aluminum sulfate, sodium aluminate, polyaluminum chloride, etc.) have been used for decades to reliably remove phosphorus from wastewater. In the use of chemical precipitation targets soluble orthophosphate removal, the removal occurs through coagulation from solution into a solid, and that solid is removed via clarification or filtration. This solid can be removed in primary clarifiers with primary solids; it can be removed in secondary clarifiers as part of the mixed liquor (with the associated improved sludge settling properties); it can be removed in tertiary clarifiers or processes; or it can be removed in any combination of the three, with removal efficiencies generally increasing with the number of chemical addition points. Despite the widespread acceptance and use of this approach, the solid conversion mechanisms of orthophosphate have not been well understood and the success of chemical precipitation of phosphorus is highly dependent upon site specific conditions.

As a result it is important to conduct jar testing to determine the optimal chemical and dose of metal salt to be used for this process. This section outlines the sample location, chemical preparation, specific jar testing procedures, and analytical requirements that will be used at each facility.

3.1 Sample Locations

Jar tests will be conducted for wastewater samples collected at various locations at each facility, selected because they are potential chemical feed points for phosphorus removal.

The most common locations for chemical addition are the primary clarifiers or the mixed liquor at or upstream of the splitter box feeding the secondary clarifiers. This latter feed point allows for chemical sludge to be formed and mostly removed in the clarifiers. Because the RAS flow recycles most of the secondary clarifier solids to the aeration basins, chemical addition at the secondary clarifiers will also result in chemical solids in the aeration basins as well. For improved removal efficiency, chemical feed to the clarifier effluent should also be considered. The solids formed in the secondary clarifier would be removed in a subsequent solids separation step. The planned jar testing allows for evaluation of the optimized chemical dose to achieve maximum removal in a single step.

Literature reports support the theory that multiple points of application can result in better removal efficiencies at lower overall coagulant doses. Therefore, additional two-stage chemical addition will be evaluated to provide a better estimate of the chemical dose needed for application upstream of filter, because that would be done following chemical addition at the secondary clarifier influent. For this test, 50-percent of the optimized dose from tests on secondary clarifier influent will be run in all jars; the samples will be decanted to remove settled solids. The supernatant will be retested with second stage chemical addition with doses that are approximately 10, 20, 30, 40, 50 and 60-percent of dose determined in the optimized single-stage test.

A summary of the jar testing locations that are recommended are shown in **Table 3-1**.

Table 3-1
Jar Testing Locations

Sample Location	Comments
Influent	-
Secondary Clarifier Influent	Mixed liquor characteristics
Secondary Clarifier Influent (two-stage testing)	Supernatant from optimized dose at Secondary Clarifier Influent
Filtrate/Centrate (Main WWTP)	High P side stream

Side stream recycle within a WWTP are also potential chemical addition points because of the high phosphorus concentrations often found in these waste streams. Thickener or dewatering sidestreams are likely sources as exhibited by the high phosphorus concentrations that have been reported in these flows.

3.2 Preparation of Metal Salt Stock Solutions

Stock metal salt solutions will be prepared from the metal salt samples obtained from either the CITY or from chemical vendors no more than 24 hours prior to jar testing. The material safety data sheet will be used to determine the percent solution (by weight) and bulk solution density. This information will be used to prepare 4,000 mg/L (as metal salt) stock solution. The 4,000 mg/L stock solution allows the two liter beakers for the jar test to be dosed with concentrations as high as 400 mg/L with no more than a 10 percent change in test volume. Additionally, this stock concentration allows doses to be determined in the field with no field calculations which can significantly reduce errors in dosing; thus, for a 50 mg/L dose, a stock volume of 25 mL would be added to the 2L jar and for 100 mg/L, a stock volume of 50 mL would be added to the 2L jar, and so forth.

The appropriate volume of vendor/CITY supplied stock solution will be transferred into a two liter volumetric flask using volumetric, graduated pipets. The flask will then be filled to volume with purified, deionized water and mixed. The 4000 mg/L stock solutions will be transferred to clean glass or plastic storage bottles with a tight fitting lid.

3.3 Jar Test Procedures

The sample will be collected immediately before each jar test is run. A screening level jar test will be conducted to target more focused coagulant dose ranges, prior to running the jars from which analytical samples will be collected. The screening level testing allows field personnel to use surrogate field parameters to target a series of concentrations that will provide more accurate information on chemical dose optimization. Surrogate field parameters will include turbidity, ultraviolet transmittance (UVT) and field orthophosphate (ortho-P) as indicators of the effectiveness of the coagulation/flocculation reactions for each chemical and dose. A matrix of the doses to be used for screening tests is shown in **Table3-2**.

Table 3-2
Screening Jar Test Matrix

Parameter	**Jar 1 Control**	**Jar 2**	**Jar 3**	**Jar 4**	**Jar 5**	**Jar 6**
Ferric Chloride (mg/L as $FeCl_3$)	0	10	30	60	100	150
Alum (mg/L as $Al_2(SO_4)_3$)	0	25	75	125	150	200

Once the screening jar tests have been completed, field personnel should immediately run the jar testing for analytical testing. The chemical dose for each of these tests will be determined in the field based on the results of the screening jar test. Using the UVT, turbidity and field ortho-P results to determine the best performance of the wide range of metal salt doses, field personnel will select a dose range that brackets the best jars. A sample jar test worksheet is provided in **Appendix A**.

All experiments will be performed in a six paddle, Phipps-Bird jar test unit with 2L square beakers. This configuration will allow up to five concentrations of a metal salt to be tested, in addition to a control sample for each test. The procedure that will be used for each jar test follows:

1. Transfer 2 liters of sample from a collection bucket into each of the six square jar testing beakers.

2. Simultaneously dose each beaker with the appropriate volumes of stock solution (stock solution preparation will be described in **Section 3.2**) using either graduated cylinders or syringes, depending upon the volume required.

3. Stir the beakers at 300 rotations per minute (rpm) for 45 seconds to simulate a rapid mix or chemical induction phase.

4. Stir the beakers an additional two minutes at 60 rpm to allow coagulated materials to flocculate.

5. Turn the stirrers off and allow the samples to settle for 30 minutes.

6. Collect supernatant from each jar and pour into a small beaker so that field measurements of temperature, pH, ORP, conductivity and turbidity can be measured; also collect a sample for UVT measurement. Measure field parameters and record data in a field log book.

7. Select the dose range for the jar testing to be conducted for laboratory analysis by identifying the best two jars; those are the jars that have the lowest turbidity and highest UVT. Use this information to develop a targeted dose range for the second set of jar testing.

8. For the jar test to be submitted for laboratory analysis, repeat steps one through six. In addition to the field parameters, collect settled water samples to be submitted for analytical testing (laboratory parameters and methods will be provided in **Section 3.4**)

9. Once settled water samples have been collected, filter sufficient sample from the three best jars using a 0.45µm filter and collect filtrate to be submitted for analytical testing(laboratory parameters and methods will be provided in Section 3.4)

10. Each sample will be identified by plant, sample location, the metal salt and dose used, whether the sample is settled or filtered water and the date as follows: PLANT-LOC-CHEMDOSE-FRAC-DATE10.

For a sample of settled water from an influent sample jar test at the Main WWTF that was dosed with 50 mg/L of ferric chloride, tested on August 26 would be noted as follows: MAIN-INF-FER50-SET-082610. For a sample of filtered water from an effluent sample jar test that was dosed with 100 mg/L of alum, tested on August 28 would be noted as follows: MAIN-EFF-ALUM100-FIL-082810

11. All laboratory samples should be packed in ice and delivered to the laboratory within 24 hours along with a sample Chain of Custody (COC) form. A sample jar test worksheet that may be used in the field is provided in **Appendix B**.

3.4 Laboratory Parameters and Methods

The purpose of this evaluation is to select and optimize the metal salt to be used for chemical precipitation of phosphorus was discussed in **Section 1.0**. Laboratory analyses will be conducted on samples collected from the jar tests to aid in characterizing phosphorus speciation. The phosphorus parameters include total phosphorus, hydrolysable phosphorus and ortho-phosphorus. Both filtered and unfiltered samples (filtration will be conducted in the field) will be submitted to the laboratory for these analyses.

Addition of metal salts for chemical precipitation can consume alkalinity and depress the solution pH. The NPDES permit limits on pH is between 6.0 and 9.0. During wastewater treatment using activated sludge processes, substantial alkalinity can be consumed during nitrification and alkalinity could remain low if a possible denitrification process cannot produce adequate alkalinity. Therefore, both pH and alkalinity will be measured, in the laboratory.

The St. Charles WWTF uses UV disinfection as the method for achieving bacterial discharge limits, thus, it is important to understand if the process used for phosphorus removal could have potential impacts on these systems. It is widely known that the chemical precipitation process used to remove phosphorus from wastewater effluents can also have positive impacts on the color, organic carbon, and total suspended solids in wastewater effluent; TSS and UVT samples will be submitted for laboratory analysis to evaluate the impact of this process on these indicator parameters. Conversely, residual iron from ferric coagulants may adversely impact UV performance because of increased inorganic fouling of quartz sleeves, which is also a function of hardness in the effluent. Additionally, because UV disinfection inactivates microorganisms at specific UV wavelengths, residual iron can have a negative impact on the transmittance of UV through wastewater. These issues are typically associated with ferric iron, although ferrous iron can also have an impact; impact threshold concentrations, which are concentrations that result in UVT decreases from 91 to 90 percent have been reported at 0.057 mg/L for ferric iron (Fe^{3+}) and 9.6 mg/L for ferrous iron (Fe^{2+}) (Bolton et al. 2001). Thus, residual dissolved iron concentrations will be evaluated in this study.

The analytical parameters to be collected for each jar test and the total sample numbers are provided in **Tables 3-3**. The analytical methods, sample container, and storage requirements for the parameters discussed in this section are provided in **Appendix C**.

Table 3-3
Analytical Matrix
(matrix will be repeated for each coagulant tested)

Sample	Laboratory pH	Akalinity	Hardness	Iron*	TSS	TP	TRP	TAHP
	(s.u.)	(mg/L as CaCO3)		(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)
Influent								
Control (untreated)	**1**	**1**	**1**	—	**1**	**1**	**1**	**1**
Field Filtered Control	—	—	—	**1**	—	**1**	**1**	**1**
Best Jar-A	**2**	**2**	**2**	—	**2**	**2**	—	—
Field Filtered Jar-A	—	—	—	**1**	—	—	—	—
Influent Splitter Box to Secondary Clarifiers								
Control (untreated)	**1**	**1**	**1**	—	**1**	**1**	**1**	**1**
Field Filtered Control	—	—	—	**1**	—	**1**	**1**	**1**
Best Jar-A	**2**	**2**	**2**	—	**2**	**2**	**2**	**2**
Field Filtered Jar-A	—	—	—	**1**	—	**2**	**2**	**2**
Influent Splitter Box to Secondary Clarifier –Two Stage P Removal								
Control (untreated)	**1**	**1**	**1**	—	**1**	**1**	**1**	**1**
Field Filtered Control	—	—	—	**1**	—	**1**	**1**	**1**
Best Jar-A	**2**	**2**	**2**	—	**2**	**2**	**2**	**2**
Field Filtered Jar-A	—	—	—	**1**	—	**2**	**2**	**2**
Filtrate/Centrate								
Control (untreated)	**1**	**1**	**1**	**1** (filtered)	**1**	**1**	—	—
Best Jar-A	**2**	**2**	**2**	**1** (filtered)	**2**	**2**	—	—
Total Samples	12	12	12	8	12	15	14	14

Iron will be run for the jar tests for the ferric chloride testing only

References

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Neethling, J.B., and A. Gu. (2007) Phosphorus Speciation Provides Direction to Produce 10 ug/l. Nutrient Removal Specialty Conference, Baltimore, 2007.

Gehr, R.; Wright, H. (1998) UV Disinfection of Wastewater Coagulated with Ferric Chloride: Recalcitrance and Fouling Problems. *Water Science & Technology, 38(3):15-23*.

S. Smith, S., Taka, I., Murthy, S., Daigger, G.T., Szabo, A. (2008) Phosphate Complexation Model and Its Implications for Chemical Phosphorus Removal. *Water Enironment Research, 80(5): 407-416.*

Szabo, A., Taka, I., Murthy, S., Daigger. G.T., Licsko, I., Smith, S. (2008) Significance of Design and Operational Variables in Chemical Phosphorus Removal. *Water Enironment Research, 80(5): 428-438.*

Leong , L.Y.C. , Kuo , J. and Tang , C. C. (2008). Disinfection of wastewater effluent – comparison of alternative technologies. WERF Report 04 - HHE - 4, Water Environment Research Foundation, Alexandria, Virginia

Appendix A
Jar Test Worksheet for Screening Test

SAMPLE LOCATION	Jar 1 Control	Jar 2	Jar 3	Jar 4	Jar 5	Jar 6
Ferric Chloride (mg/L as $FeCl_3$)	0	10	30	60	100	150
Temperature (Degrees C)						
pH (s.u.)						
Conductivity (µS/cm)						
ORP (mV)						
UVT (%T at 254nm)						
Turbidity (NTU)						
Ortho-P (mg/L)						
Alum (mg/L as $Al_2(SO_4)_3$)	0	25	75	125	150	200
Temperature (Degrees C)						
pH (s.u.)						
Conductivity (µS/cm)						
ORP (mV)						
UVT (%T at 254nm)						
Turbidity (NTU)						
Ortho-P (mg/L)						

Appendix B
Jar Test Worksheet for Laboratory Analysis

SAMPLE LOCATION	Jar 1 Control	Jar 2	Jar 3	Jar 4	Jar 5	Jar 6
Ferric Chloride (mg/L as $FeCl_3$)	0					
Temperature (Degrees C)						
pH (s.u.)						
Conductivity (µS/cm)						
ORP (mV)						
UVT (%T at 254nm)						
Turbidity (NTU)						
Ortho-P (mg/L)						
Alum (mg/L as $Al_2(SO_4)_3$)	0					
Temperature (Degrees C)						
pH (s.u.)						
Conductivity (µS/cm)						
ORP (mV)						
UVT (%T at 254nm)						
Turbidity (NTU)						
Ortho-P (mg/L)						

Appendix C

Analytical Methods and Storage Requirements

Analyte	Preferred Analytical Method*	Sample Volume & Container	Maximum Holding Time	Preservation Method
pH	SM 4500-H	100 mL Plastic or glass	24 hours	refrigerate at 4C
Alkalinity	SM 2340	100 mL Plastic or glass	14 days	refrigerate at 4C
Hardness	SM 2320	100 mL Plastic or glass	14 days	refrigerate at 4C
Iron	SM 3500	100 mL Plastic or glass	At pH < 2 6 months	concentrated HNO_3, refrigerate at 4C
Total Phosphorus (TP)	SM 4500-P B.5. Persulfate Digestion, followed by SM 4500-P E. Ascorbic Acid Method	100 mL Pre-cleaned glass	48 hours	refrigerate at 4C
Total Reactive Phosphorus (TRP)	SM 4500-P E. Ascorbic Acid Method	See TP	48 hours	refrigerate at 4C
Total Acid Hydrolyzable Phosphorus (TAHP)	SM 4500-P B.2. Acid Hydrolysis, followed by SM 4500-P E. Ascorbic Acid Method	See TP	48 hours	refrigerate at 4C
Total Suspended Solids (TSS)	SM 2540	100 mL Plastic or glass	7 days	refrigerate at 4C

A substitute method is acceptable provided that detection limits are provided prior to analysis.

APPENDIX E
ECOCAT_1512192

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EcoCAT
Ecological Compliance Assessment Tool

Applicant: Trotter and Associates, Inc.
Contact: Jerry Ruth
Address: 40W201 Wasco Road, Suite D
St. Charles, IL 60175

IDNR Project Number: 1512192
Date: 05/07/2015
Alternate Number: STC-093

Project: 2015 Facility Plan Update
Address: 200 Devereaux Way, St. Charles

Description: A Facility Plan Report (FPR) is a management and planning document used to identify, evaluate, and plan required wastewater facility improvements. It provides an assessment of the collection and treatment systems' abilities to meet both current and future loads, flows and regulatory requirements and provides critical information for improvements to correct current or projected deficiencies. FPRs are required by the Illinois Environmental Protection Agency (IEPA) for any wastewater improvements that change the treatment process or expand the capacity of the wastewater treatment plant.
FPRs are typically updated every five to ten years, or when significant changes in growth or regulatory requirements have occurred or are expected. In 2002, the City updated its FPR which identified the need for nitrification capabilities. In 2009, the City updated its FPR again which identified the need for improved biosolids handling infrastructure

Natural Resource Review Results

This project was submitted for information only. It is not a consultation under Part 1075.

The Illinois Natural Heritage Database contains no record of State-listed threatened or endangered species, Illinois Natural Area Inventory sites, dedicated Illinois Nature Preserves, or registered Land and Water Reserves in the vicinity of the project location.

Location

The applicant is responsible for the accuracy of the location submitted for the project.

County: Kane

Township, Range, Section:
40N, 8E, 34
40N, 8E, 35

IL Department of Natural Resources Contact
Impact Assessment Section
217-785-5500
Division of Ecosystems & Environment

Disclaimer

The Illinois Natural Heritage Database cannot provide a conclusive statement on the presence, absence, or condition of natural resources in Illinois. This review reflects the information existing in the Database at the time of this inquiry, and should not be regarded as a final statement on the site being considered, nor should it be a substitute for detailed site surveys or field surveys required for environmental assessments. If additional protected resources are encountered during the project's implementation, compliance with applicable statutes and regulations is required.

Terms of Use

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1. The IDNR EcoCAT website was developed so that units of local government, state agencies and the public could request information or begin natural resource consultations on-line for the Illinois Endangered Species Protection Act, Illinois Natural Areas Preservation Act, and Illinois Interagency Wetland Policy Act. EcoCAT uses databases, Geographic Information System mapping, and a set of programmed decision rules to determine if proposed actions are in the vicinity of protected natural resources. By indicating your agreement to the Terms of Use for this application, you warrant that you will not use this web site for any other purpose.

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Unauthorized use, tampering with or modification of this system, including supporting hardware or software, may subject the violator to criminal and civil penalties. In the event of unauthorized intrusion, all relevant information regarding possible violation of law may be provided to law enforcement officials.

Privacy

EcoCAT generates a public record subject to disclosure under the Freedom of Information Act. Otherwise, IDNR uses the information submitted to EcoCAT solely for internal tracking purposes.

City of Saint Charles

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