Village of Tolono

South Bourne Street |Tolono, Illinois, 61880

WWTP Facility Improvements
Facility Plan

October 2019

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ABBREVIATIONS

BOD	Biological oxygen demand
DAF	Design average flow
DMF	Design maximum flow
GPD	gallons per day
GPM	gallons per minute
IEPA	Illinois Environmental Protection Agency
MG	Million gallon
MGD	Million gallons per day
NPDES	National Pollutant Discharge Elimination System
SCFM	Standard cubic feet per minute
SWD	Side water depth
TSS	Total suspended solids
WWTP	Wastewater treatment plant

1. EXECUTIVE SUMMARY

1.1 BACKGROUND

The Village of Tolono owns and operates the Wastewater Treatment Plant (WWTP) which treats wastewater generated by domestic and commercial customers. Currently the WWTP is operating above capacity. The Village is also considering taking waste from a nearby mobile home park, an elementary school, and some new residential developments. The Village has identified necessary wastewater treatment facility improvements required to bring age depreciated facilities up to date.

1.2 PURPOSE

The purpose of this Facility Plan Report is to document the basis of design for the improvements to the WWTP and to assess the cost of these upgrades, quantify the cost of financing the capital project and to estimate the impact of this financing on customer's sewer bills.

1.3 RECOMMENDED PROJECTS

The Recommended Project includes construction of an oxidation ditch system, forecasted at a total initial cost of $5.6 million. This is a substantial capital cost to the Village and will likely require some type of financing to make it affordable. Detailed financing scenarios for these improvement is excluded from this report because financing of these improvements is being analyzed along with other sewer system capital improvements as part of a rate study that is being completed in November 2019.

PROJECT PLANNING

1.4 FACILITY PLANNING AREA

The WWTP is located on South Bourne Street in Tolono, Illinois and discharges effluent to Hackett Branch under NPDES Permit No. IL0031453 (Appendix A), as shown in Figure 0-1.

The WWTP is currently rated for 0.235 MGD design average flow (DAF) and 0.588 MGD design maximum flow (DMF). The WWTP is located in Section 35, Township 18 North, Range 8 East, in the 3rd Principal Meridian. Figure 2 below shows the Tolono WWTP location on a USGS map.

Figure 0-1 Tolono Aerial

1.5 ENVIRONMENTAL RESOURCES

The recommended plant improvements are all located within the existing plant facility, therefore minimal environmental impact is expected. During full design Donohue will coordinate to solicit input from agencies associated with environmental issues such as wetlands; flood plains; unique plant or animal communities or other important fish and wildlife habitats; historic, archeological, and cultural features; and any other factors that would be significantly affected by the proposed improvements.

1.6 PLANNING PERIOD

The project planning period is a 20-year period, extending from 2020 to 2040. It is intended that all equipment (except SCADA–related hardware and software) proposed in this report have a design life of at least 20 years. User charge calculations within the aforementioned rate study will be compiled assuming a 20-year payback of the instruments of finance, such as Illinois EPA Water Pollution Control Loan Program loans.

1.7 POPULATION PROJECTION & INCOME STATISTICS

The City of Tolono is located in Champaign County in central Illinois. The Village currently occupies an area of approximately 2.14 square miles. The Village's population listed in the 2000 U.S. census was listed as 2,700 persons. The Village's population listed in the 2010 U.S. census was listed as 3,447 persons, approximately a 28% increase. The estimated 2017 population was 3,449, approximately no change from the 2010 census. The City currently serves a total of 1,158 homes. The Village is anticipates to increase the existing population served by expanding the service area to existing developments of nearby Oaks Mobile Home Park, Unity West Elementary School and Southside residential community, for an additional 841 users. The Village also anticipates future growth from new developments in the proposed service area from the Oaks Mobile Home Park, the Southside residential area, and other new residential developments near East Side Park and the existing Woodworth Property, west of Village Hall, for an additional 650 users. Based on an estimated growth of 1% per year paired the expanded service area growth stated above, the future service population is estimated to be 5,180 users in 2040. This corresponds to the design average daily flow of 500,000 gallons per day discussed in Chapter 2.

The U.S. Census Bureau reports that Tolono's average Median Household Income (MHI) for 2013-2017 was $69,712. The statewide average MHI for all of Illinois for 2013-2017 was reported to be $61,229.

1.8 FACILITY PLANNING AREA IMPACTS

The WWTP's proposed service area is currently within the Village of Tolono's existing Facilities Planning Area and this project proposes no changes to the Village's FPA boundaries.

1.9 ZONING AND LAND USES

The proposed upgrades will be constructed within the premises of the WWTP site. The current City Zoning Map as published by the Plan Commission shows the site to be zoned in the "AG-1" agriculture zoned district, in future corporate limits. The most recent zoning map can be found in Appendix B. These improvements are consistent with the other facilities currently on the site and the scope of this project does not intend to alter the zoning or nature of the land uses within the WWTP.

2. EXISTING CONDITIONS

2.1 DESIGN AND CURRENT FLOWS

The current and design treatment capacities, the additional flows and the future design flows are summarized in Table 2-1. Existing and new development growth was detailed in the previous section. The total average day flows from existing developments that could be incorporated immediately is estimated to be 0.044 MGD. The total average day flows from future developments is expected to be 0.065 MGD. The total increase in average flow would therefore be 0.110 MGD. The future flows shown in Table 2-1include the Oaks Park and new residential development flows.

Table 2-1 Design, Current and Future flows

Flows (MGD)	DAF	DMF
Original Design Flows	0.235	0.588
Current Flows*	0.333	0.767
Existing Development to be Incorporated	0.044	0.035
Future Development Growth	0.065	0.074
Population Growth (~1%)	0.058	0.124
Future Design Flows	0.500	1.00

***2019 Sanitary Sewer System Flow Study**

The following sub-sections details the unit process design criteria for the treatment system. A complete design basis spreadsheet for the treatment system is included in Appendix C. The next chapter provides more details for the recommended improvements.

2.2 DESIGN AND CURRENT CONCENTRATIONS AND LOADINGS

The original design and current concentrations and loadings are shown in Table 2-2. The design influent data was from the existing O&M manuals. The current conditions are based on one grab sample per month. Due to the minimal existing conditions data the facility design alternatives were based on the more conservative original design concentrations.

Before design it is recommended that the Village complete a special sampling campaign to better characterize the influent conditions including but not limited to concentrations and loading of BOD, TSS, ammonia and total phosphorus.

Table 2-2 Design and Current Concentrations and Loadings

		Design	Current Conditions *	
Monthly Average		**Influent**	**Influent**	**Effluent**
Flow	**MGD**	0.235	0.168	0.135
BOD	**mg/L**	277	134	6.7
	ppd	543	-	6.7
TSS	**mg/L**	323	154	8.8
	ppd	633	-	8.9
Ammonia	**mg/L**	19	-	1.07
	ppd	37	-	1.57
Phosphorus	**mg/L**	9	-	1.67
*** 2017-2019 DMR Data**				

2.3 TREATMENT PROCESS OVERVIEW

The Tolono WWTP was designed and built in the early 1970's with improvements in the 1980's and 2011. The 1984 update added a sludge storage lagoon, primary clarification and made other minor plant improvements. In 2011 a roughing filter was added to primary treatment to reduce the loading to the aeration basins. The majority of the process equipment is over thirty years old and has exceeded the original design life. Aerials of the facility are shown in Figure 2-1 and Figure 2-2.

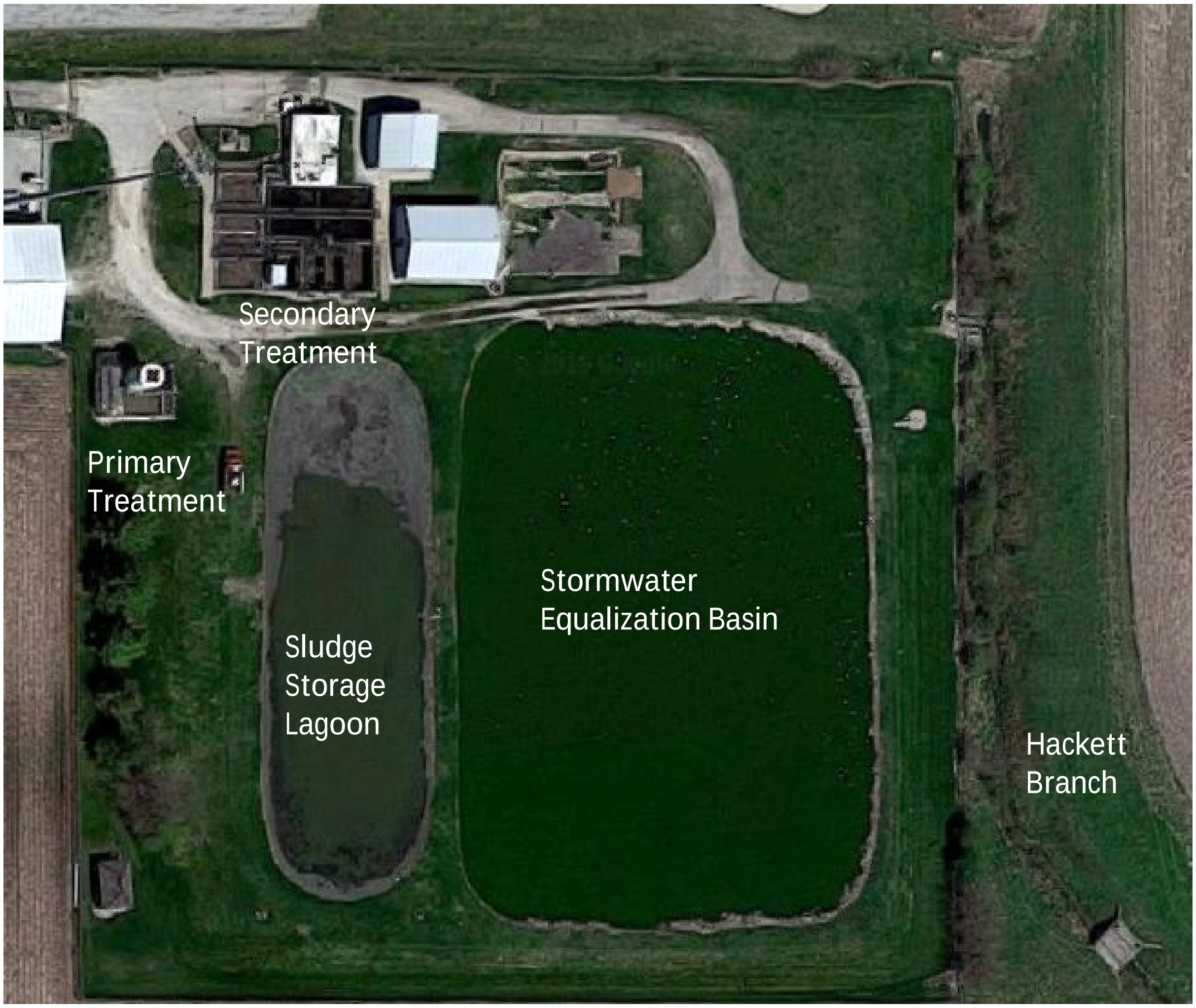

Figure 2-1 Tolono WWTP and Lagoon Aerial

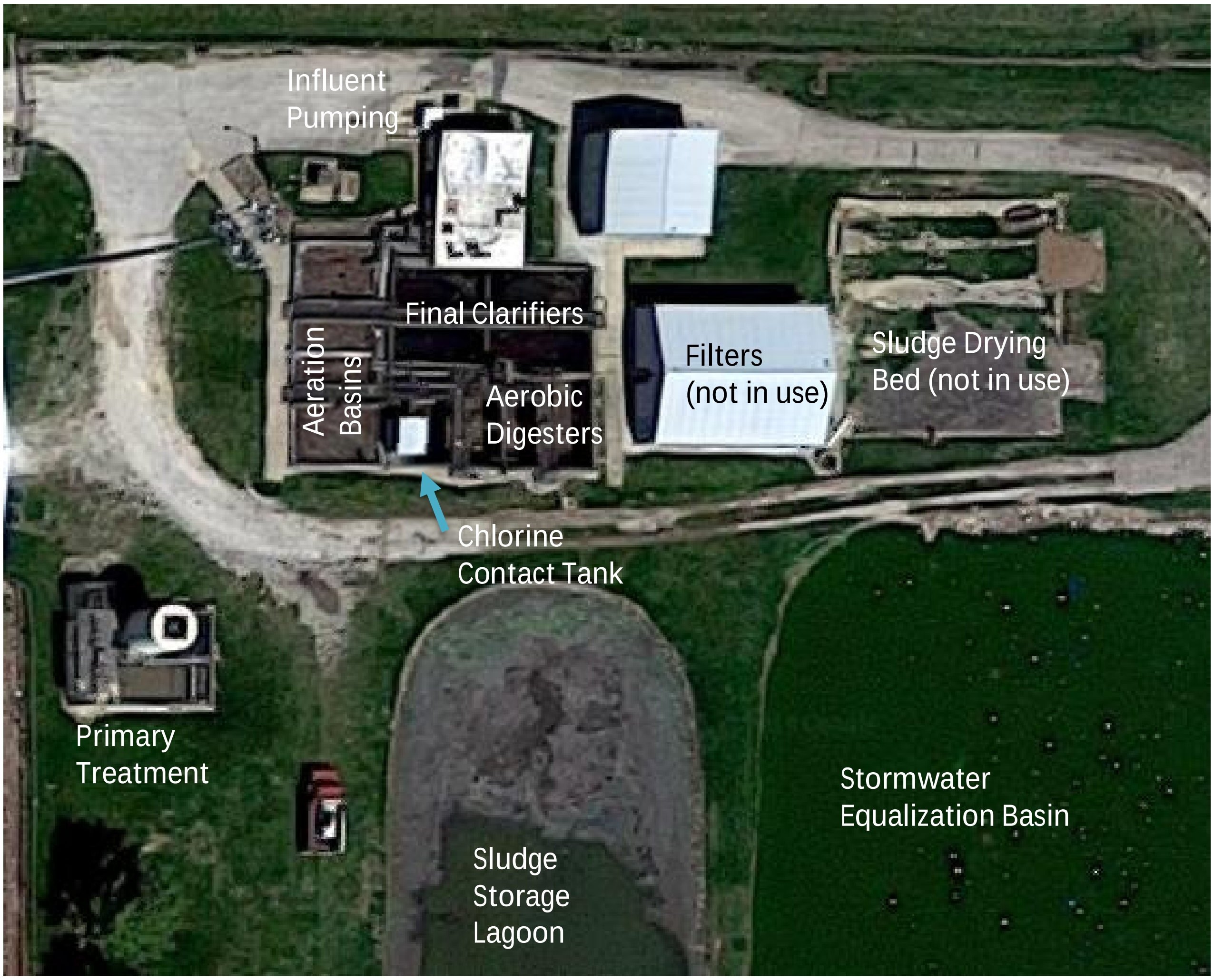

Figure 2-2 Tolono WWTP

Flow enters the raw influent wet well through a 12-inch influent sewer and a sewage comminutor, and is then pumped to either primary treatment (<0.588 MGD) or the stormwater equalization basin (>0.588 MGD). Stormwater can later flow back to the influent pumping well for full treatment or be discharged to the Hackett Branch after disinfection via gravity.

The raw influent pump discharge to two rectangular primary clarifier (10' x 31' each). During primary clarification flow is also pumped up through a primary roughing filter (11' diameter x 15' height) to reduce loading to the aeration basin. Flow exits the roughing filter and enters back into the one of the primary clarifiers. All flow then exits through the primary clarifier effluent troughs and flows via gravity to the two aeration basins (24' x 20' each), which are run in parallel.

Aeration basin effluent then enters two 24'x24' square final clarifiers. Flow can be sent to the chlorine contact tank (21' x 6.2') or discharged to the Hackett Branch. As the NPDES permit does not currently require seasonal disinfection flow is sent directly to the Hackett Branch. There are two tertiary sand filters that are not currently in use that were originally constructed for secondary effluent polishing.

Primary sludge from the primary clarifiers and waste activated sludge from secondary treatment is combined and digested in two aerobic digesters. There is currently no thickening at the plant. Digested sludge is then sent to the sludge storage lagoon. Sludge is typically dredged and land applied every other year. A summary of the existing unit process capacities, based on Ten State Standard Recommendations, is shown in Table 2-3. Any factor that would contribute to limiting the capacities of these unit processes will be discussed in the following sections. Loading to the plant has been defined by the current and future flows along with the original design influent BOD, TSS and ammonia.

Table 2-3 Existing Unit Process Capacities

	Treatment	Number	Capacity	Sizing
Headworks	Influent Pumping	3	3 - 410 gpm @ 37'	
	Comminutor	1		
Primary Treatment	Primary Clarifiers	2	0.62 MGD @ 1,000 gpd/sf 0.93 MGD @1,500 gpd/sf	31' x 10' x 6.69' SWD ea
	Roughing Filter Unit	1	0.68 MGD @ 5 gpm/sf	11' diameter x 15' depth
Secondary Treatment	Aeration Basins	2	13,920 cf	24' x 20' x 14.5 SWD ea
	Final Clarifiers	2	1.38 MGD @ 1,200 gpd/sf 2.76 MGD @ 20,000 gpd/f	24' x 24' x 11.5 SWD ea
	Tertiary Filters (out of service)	2		16 cells/filter 11" sand depth
	Blowers	3	767 scfm each	
Disinfection	Chlorine Contract Tank	1	0.90 MGD @ 15 min	21' x 6.2' x 9.75 SWD
Solids Handling	Aerobic Digesters	2	11,986 cf	21.2' x 20.2' x 14 SWD
	Sludge Lagoon	1		72,495 cuft
Stormwater	Stormwater Pumps	1	900 gpm	
	Stormwater Equalization Basin	1		2.7 MG

Figure 2-3 Existing Unit Capacities

*Primary clarifier capacity based on a 10' SWD, actual SWD 6.69'

*Secondary clarifier capacity based on 12' SWD, actual SWD 11.52'

2.4 HEADWORKS AND STORMWATER

The WWTP does not currently have a main headworks facility, instead influent sewage enters through a 12-inch line into muffin monster comminutor (sewage grinder) operated continuously and into the influent wet well. The raw wastewater can either be pumped to primary treatment, via three influent pumps located in the main control build, or pumped to the stormwater equalization basins, via one stormwater pump. All flows 0 – 0.588 MGD are pumped to primary treatment and all flows above 0.588 MGD are pumped to the stormwater equalization basin. The stormwater pump will turn on at a set wet well depth. Raw influent pumps are three influent Fairbank Morse 7.5 hp centrifugal pumps shown in Figure 2-4, located in the basement of the main control building east of the wet well.

The stormwater pump is located above ground, outside but covered south of the influent wet well and has a suction elevation higher than that of the influent pumps. The stormwater pump has a capacity of 900 gpm.

The plant currently lacks any influent screening or grit removal. In addition, due to age of the stormwater pump, the discharge piping leaks through a flange a large percentage of the pumped stormwater back into the influent wet well.

The influent pumps are shown in Figure 2-4. Influent wet well (grating), covered stormwater pump, and control building are shown in Figure 2-5.

Figure 2-4 Influent Pumps

Recommended upgrades are:

1. New screening/ headworks facilities
2. New stormwater pump and discharge piping
3. New flow metering

Figure 2-5 Influent Wet Well and Stormwater Pump and Cover

2.5 PRIMARY TREATMENT

Primary treatment consists of two rectangular primary clarifiers, shown in Figure 2-6, and one primary roughing filters shown in Figure 2-7. The two rectangular primary clarifiers have dimensions of 31' length, 10' width and 6.69' operating SWD each for a total volume of 0.031 MGD and total surface area of 620 sqft. Ten State Standards recommends a surface overflow rate of 1,000 gpd/ft^2 for design average flow and 1,500-2,000 gpd/ft^2 for peak hourly flow. Based on these standards the clarifiers should be sized for 0.62 MGD average day and 0.93 MGD peak hourly (at 1,500 gpd/ft^2); however, Ten States also recommends a minimum side water depth of 10 feet. As the existing depth of the primary clarifiers is shallower than 10' the primary clarifiers likely do not allow enough room between the sludge blanket and the effluent trough decreasing the removal efficiency. The roughing filter was likely added to aid in primary treatment and decrease the loading to the aeration basin. The roughing filter is limited to the peak pump capacity which is 0.63 MGD.

Figure 2-6 Primary Clarifier

Figure 2-7 Primary Clarifier and Roughing Filter

Recommended upgrades are:

1. Additional primary treatment

2. primary clarifier mechanisms
3. Larger primary roughing filter pumps

2.6 SECONDARY TREATMENT

Secondary treatment consists of two aeration basins run in parallel shown in Figure 2-8, and two square final clarifiers shown in Figure 2-10 and Figure 2-11. The two aeration basins have dimensions of 24' length, 20' width and 14.5' SWD, for a total volume of 0.104 MG. For a single-stage nitrification plant Ten State Standard recommends volumetric loading of 15 lb BOD/kcf, however Donohue has had success permitting plants up to 23 lb BOD/kcf. Based on conservative primary treatment BOD and TSS removal estimates, at 23 lb BOD/kcf the aeration basins are currently sized for an average daily flow of 0.22 MGD, and are likely currently experiencing average loading rates of 34 lb BOD/kcf. Future average day flows would increase loadings to 46 lb BOD/kcf. The estimated current and future air flow requirements are 254 scfm and 337 scfm respectively.

Blowers shown in Figure 2-9 feed the aeration basins, the aerobic digesters and the clarifier scum and sludge airlift pumps. There are currently three centrifugal blowers each with an output of 767 SCFM for a total output of 2301 SCFM.

The two square final clarifiers have dimensions of 24' by 24' and 11.52' operating SWD each for a total volume of 0.10 MG and total surface area of 1152 sqft. Ten State Standards recommends a surface overflow rate of 1,200 gpd/ft^2 for peak hourly flow. Based on these standards the clarifiers should be sized for 1.38 MGD peak hourly flows. To avoid the use of a tertiary filter or other polishing step the IEPA requires clarifier be sized for no more than 600 gpd/ft^2 average daily flow, which would limit the clarifier average daily flow to 0.691 MGD. In addition to the general age of the clarifier's mechanisms it was noted that the clarifiers tend to have a buildup of floatable solids in the corners.

The facility also has two out of service tertiary filters.

Figure 2-8 Aeration Basins

Figure 2-9 Aeration Basin Blowers

Figure 2-10 Final Clarifiers

Figure 2-11 Final Clarifier Effluent Weir

Recommended upgrades are:

1. New valves and piping
2. New diffusers
3. Additional aeration basins

4. Either expanded secondary clarifiers or rehabilitate tertiary filters
    a. New clarifier mechanisms
5. New blowers

2.7 DISINFECTION

Disinfection currently is only required on stormwater. The chlorine contact tank is located between the aeration basins and the aerobic digesters, shown under the white building in Figure 2-12. The chlorine contact tank has dimensions of 21’ length, 6.2’ width, and 9.75’ SWD for a total volume of 0.0094 MG. Based on the recommended 15 minute detention time hourly flow, the existing chlorine contact tank is sized for 0.89 MGD peak hourly flow.

Figure 2-12 Chlorine Contact Tank with Chlorine Building

2.8 SOLIDS HANDLING

Solids handling is completed through aerobic digestion. The plant does not have thickening so WAS and primary sludge are sent directly to digestion. There are two aerobic digesters sized 21.2’ length, 20.2’ width and 14’ SWD, for a total volume of 11,986 cubic feet. The original design required 9,871 cubic feet, current average flows and future average flows require 13,987 cubic feet and 18,574 cubic feet of volume, respectively. The max required airflow for the future average day flows is 830 SCFM, a little over a third of the total blower capacity.

Digested sludge is stored in the sludge lagoon, with a storage volume of 0.52 MG shown in Figure 2-14. The lagoon was originally designed for 120 days of sludge storage.

Figure 2-13 Aerobic Digester

Figure 2-14 Sludge Storage Lagoon

Recommended upgrades are:

1. Additional blowers
2. Additional aerobic digesters

2.9 GENERAL UPGRADES

General recommended upgrades are:

1. National Fire Protection Agency (NFPA) 820 Code compliance upgrades
    a. Additional HVAC to 6 air changes per hour to declassify spaces
2. Grating and handrail safety upgrades
3. General concrete repairs on basins and clarifiers
4. Demolishing or rehabilitating the existing chemical storage building
5. New valves and piping

3. WATER QUALITY & EFFLUENT LIMITS

3.1 GENERAL

Illinois EPA has indicated that this receiving stream segment, Water Body Segment BERB-TO-C1 of the Hackett Branch, as being as an impaired water for aquatic life. The impairment for aquatic life potential causes is identified as total phosphorus or dissolved oxygen.

3.2 EXISTING EFFLUENT LIMITS

The discharge from the WWTP is regulated by NPDES Permit No. IL0031453. The draft reissued permit public notice ended on October 14th 2016. The discharge limits for BOD_5 at outfall 001 (Hackett Branch) are set by IEPA at 10 mg/L on a monthly average and a 20 mg/L daily maximum. The discharge limits at outfall 001 for total suspended solids (TSS) are set at 12 mg/L on a monthly average and a 24 mg/L daily maximum. The discharge limits for both BOD_5 and TSS at outfall 002 (Excess Flow) are set by IEPA at 30 mg/L on a monthly average and a 45 mg/L weekly average.

For outfall 001, ammonia limits for April through October are currently set for a monthly average of 1.5 mg/l and a daily maximum of 3.0 mg/l. Ammonia limits for November through February are currently set at a monthly average of 4.0 mg/L with a daily maximum concentration of 4.7 mg/l. Ammonia limits for the month of March are set at a monthly average of 1.6 mg/L and a daily maximum of 6.9 mg/L. Ammonia is monitor only on outfall 002.

The plant currently requires phosphorus monitoring only at both outfalls 001 and 002. Fecal coliform is monitored May through October at outfall 001. At outfall 002 fecal coliforms cannot exceed a monthly average of 400 per 100 ML. Permit limits for outfall 001 and 002 are shown in Table 3-1 and Table 3-2.

Table 3-1 NPDES Permit Limits Outfall 001

	Monthly Average (mg/L)	Daily Maximum (mg/L)
CBOD5	10	20
TSS	12	24
pH	6-9	
Fecal Coliform	Monitor only	
Total Phosphorus	Monitor only	
Ammonia		
April – Oct.	1.5	3.0
Nov. – Feb.	4.0	4.7
March	1.6	6.9
DO		
March – July	Daily min 5.0 mg/L, Weekly avg. 6.0	
Aug. – Feb.	Daily min 4.5 mg/L, Weekly avg. 4.0	

Table 3-2 NPDES Permit Limits Outfall 002

	Monthly Average (mg/L)	Weekly Average (mg/L)

CBOD5	30	45
TSS	30	45
pH	6-9	
Fecal Coliform	Daily maximum 400 per 100 ML	
Total Phosphorus	Monitor only	
Ammonia	Monitor only	
Dissolved Oxygen	Monitor only	
Chlorine Residual	0.75	

3.3 FUTURE EFFLUENT LIMITS

This project assumes there will be no major changes to the new NPDES permit. However, it is likely that over the 20 year planning period phosphorus limits will be added. The plant currently has a disinfection exemption that is based on the physical characteristics of the receiving stream. There is a possibility that IEPA may remove the exemption in the coming years.

4. WWTP IMPROVEMENTS

4.1 INTRODUCTION

At the WWTP, improvements will be constructed to achieve compliance with current design standards and to update processes at the end of their design life. It has been determined that not only are most of the process equipment and infrastructures at the end of their useful life, almost all of the existing processes are under sized for the current flows and loadings. The future estimated flows are estimated to be almost double the original design. The current processes are undersized for the future estimated flows and loadings:

- Primary clarifiers
- Aeration basins
- Secondary clarifiers
- Aerobic digestion

The following are past or quickly approaching the end of their useful life:

- Influent pumps
- Stormwater pump
- Blowers
- Clarifiers mechanisms
- Most valves and metering

The following upgrades are required for safety and to meet National Fire Protection Agency (NFPA) 820 code:

- Improve ventilation to 6 air changes per hour to declassify the space
- General concrete repairs
- New grating and handrails

Based on the needed improvements, the two main update options are to either update and expand all the existing processes; new headworks, primary, secondary, disinfection and solids handling; or to invest in a new treatment system and reuse the existing infrastructure. Updating and reusing the existing infrastructure was determined to be both prohibitively expensive and technically difficult for construction and rehabilitation due to existing infrastructure age and condition, therefore the evaluated alternatives are new treatment facilities that utilizes minimal existing infrastructure. All alternative will reuse the existing headworks (with upgrades), sludge lagoon, excess flow lagoon and chlorine contact.

4.2 EVALUATION OF ALTERNATIVES

Each alternative is briefly described in the following sections including planning level capital costs, recommended basin sizing and potential advantages and disadvantages of each technology. A breakdown of the planning level cost estimates can be found in Appendix D.

4.2.1 OXIDATION DITCH

An oxidation ditch is a modified activated sludge system that typically utilizes oval shaped basins divided into multiple channels. Oxidation ditches can be a complete mix system or modified for plug flow, and typically operate with higher SRTs. Oxidation ditches can also be modified with anaerobic and anoxic zones for nutrient removal and/or simultaneous nitrification-denitrification. Oxidization ditches can typical function without primary treatment but still require headworks (screening and possibly grit removal) and secondary clarification. Possible advantages of an oxidation ditch include: simple operation, well known technology, nutrient removal, and longer SRT to minimize shock loading. Possible disadvantages include: requiring secondary clarification and general larger footprint than other technologies. A proposed oxidation ditch by Evoqua is shown in Figure 4-1, and an aerial layout is shown in Figure 4-2. A planning level capital cost for an oxidation ditch is $5.6 million.

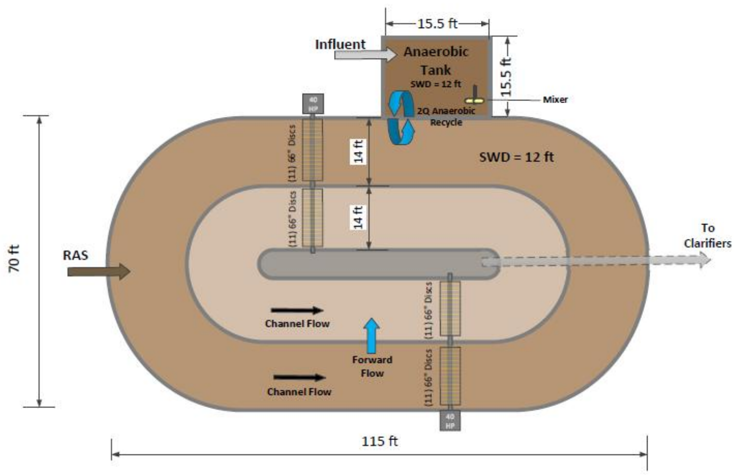

Figure 4-1 Evoqua Oxidization Ditch Proposal Layout

Figure 4-2 Oxidation ditch Aerial Layout

4.2.2 SBR

A sequence batch reactor (SBR) uses a single basin to complete all phases of a typical activated sludge treatment process: primary treatment, aeration/activated sludge and final clarification. A system is usually designed with multiple basins to allow for cycling between the two. A figure of an example SBR plant from Aqua-Aerobics is shown in Figure 4-3. An aerial layout of the proposed SBR system is shown in Figure 4-4. Potential advantages of an SBR system include: no recycling or separate clarifiers, simple construction and O&M. Potential disadvantages include: requires tertiary filtration to meet effluent limits, and requires very deep basins for the required reactor volume. A planning level capital cost for a SBR is $5.8 million.

Figure 4-3 SBR Example Reactors Aqua-Aerobics

Figure 4-4 Proposed Layout Aqua-Aerobics

4.2.3 MBR

Membrane bioreactors (MBR) is a treatment process that uses membranes for solids separation instead of traditional clarifiers. Membranes operate at a much higher solids removal than traditional clarifiers allowing for significantly higher MLSS in the aerobic tanks (8,000 – 10,000 mg/L). The higher solids allow for greater treatment in a smaller footprint. MBRs can be operated with or without traditional primary treatments but do require 2 mm screens, in addition to traditional headworks, upstream of reactor basins to ensure the longevity and operation of the membranes. A schematic of a GE MBR is shown in Figure 4-5. An aerial layout of the proposed MBR system is shown in Figure 4-6. The potential advantages of an MBR system are the small footprint and the ability to handle load fluctuations. The potential disadvantages of an MBR system are the high recycle rates and the higher O&M costs. A planning level capital cost for a MBR is $7.0 million.

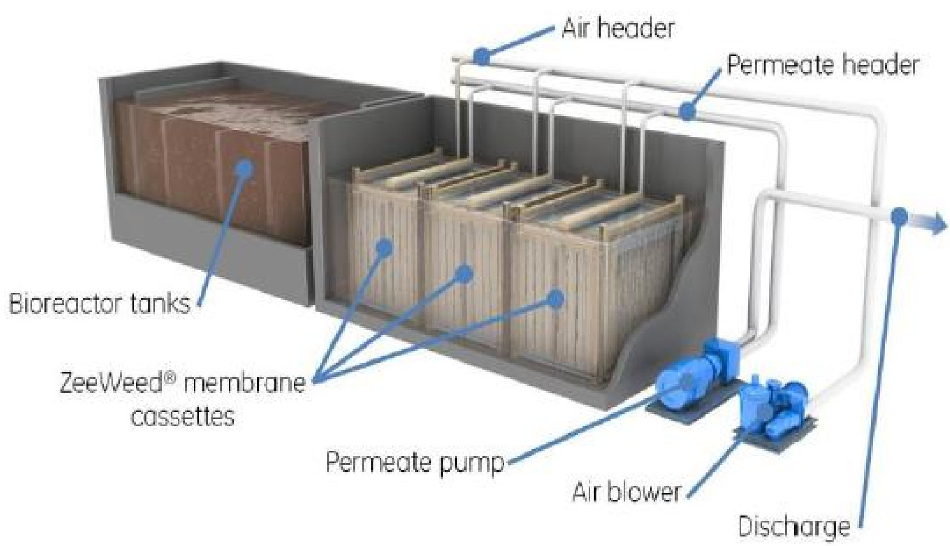

Figure 4-5 MBR Layout Schematic

Figure 4-6 MBR Aerial Layout

4.2.4 PACKAGE PLANT

A package plant is a pre-manufactured process plant, typically used for lower flows. Package plant can refer to any treatment process that comes fully manufactured including: SBRs, oxidation ditches and contact stabilization. For the purposes of comparison and costs estimating an extended aeration system from Smith & Loveless was reviewed. The proposed system contains flow equalization, traditional activated sludge and clarification, filtration and sludge storage. A flow schematic of the proposed package plant is shown in Figure 4-7. An aerial layout of the package plant system is shown in Figure 4-8. Possible advantages of a typical extended aeration package plant are easy O&M, no primary treatment, and responding well to load variations. Possible disadvantages of a package plant are typically higher aeration costs, limited future flexibility and larger footprint. A planning level capital cost for package plant is $6.4 million.

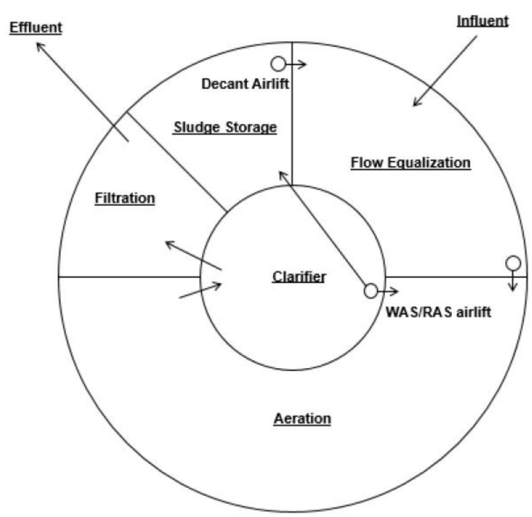

Figure 4-7 Package Plant Flow Schematic

Figure 4-8 Package Plant Aerial Layout

4.2.5 ALGAEWHEEL

Algaewheel is a rotating algal contactor (RAC) which combines agal biofilms and moving bed bioreactors (MBBR) technology for a small footprint treatment process. Algaewheel was a considered alternative as it is a small footprint, low O&M and effective treatment process; however, it was determined that the future flows and loadings are currently too high for the current Algaewheel technology, which is currently designed for smaller flows and loadings. An Algaewheel schematic is shown in Figure 4-9.

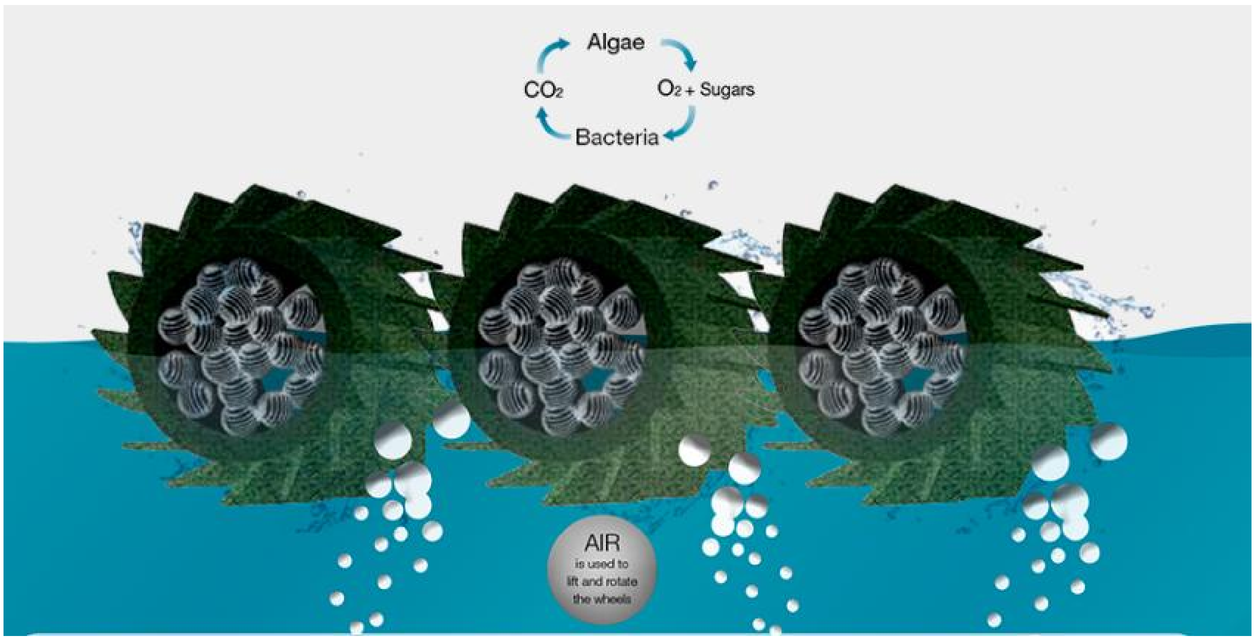

Figure 4-9 Aglaewheel

4.2.6 LEMTEC LAGOON

LemTec is a lagoon based treatment process which utilizes aerated lagoons, and typically 2 to 3 stages/lagoons to complete aeration and aerobic treatment to remove BOD, solids management for solids separation, reduction and storage, and tertiary treatment for ammonia polishing. Possible advantages of the lagoon system are no primary treatment, simplest operation, and nutrient removal. Possible disadvantage of a lagoon system is the large footprint. An example process flow of a LemTec lagoon is shown in Figure 4-10. The proposed LemTec layout is shown in Figure 4-11. A planning level capital cost for a LemTec lagoon system is $6.4 million.

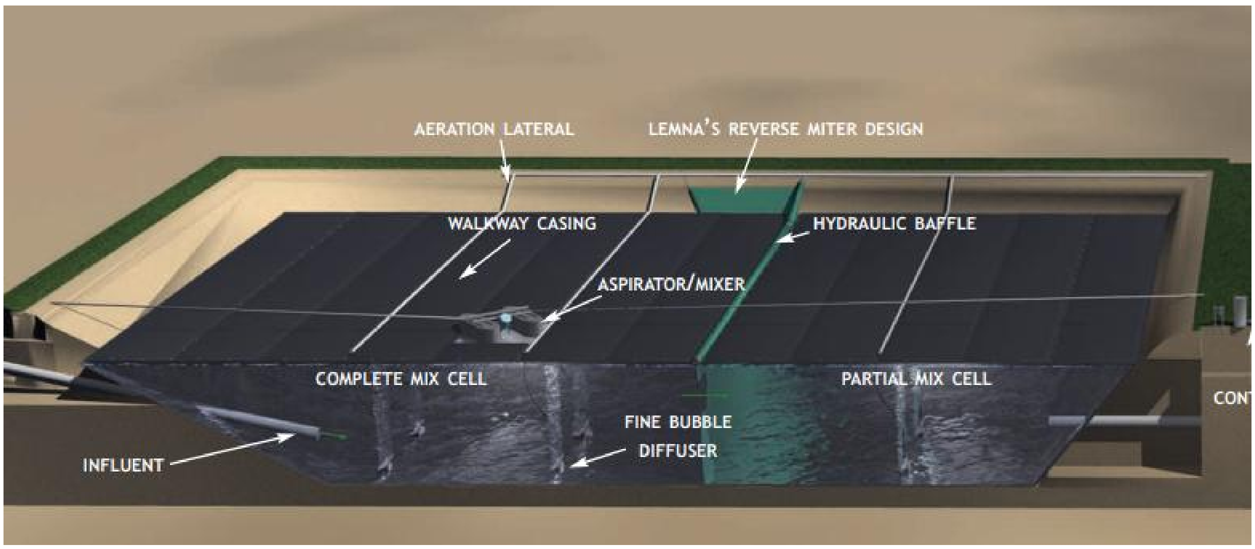

Figure 4-10 LemTec Lagoon Example

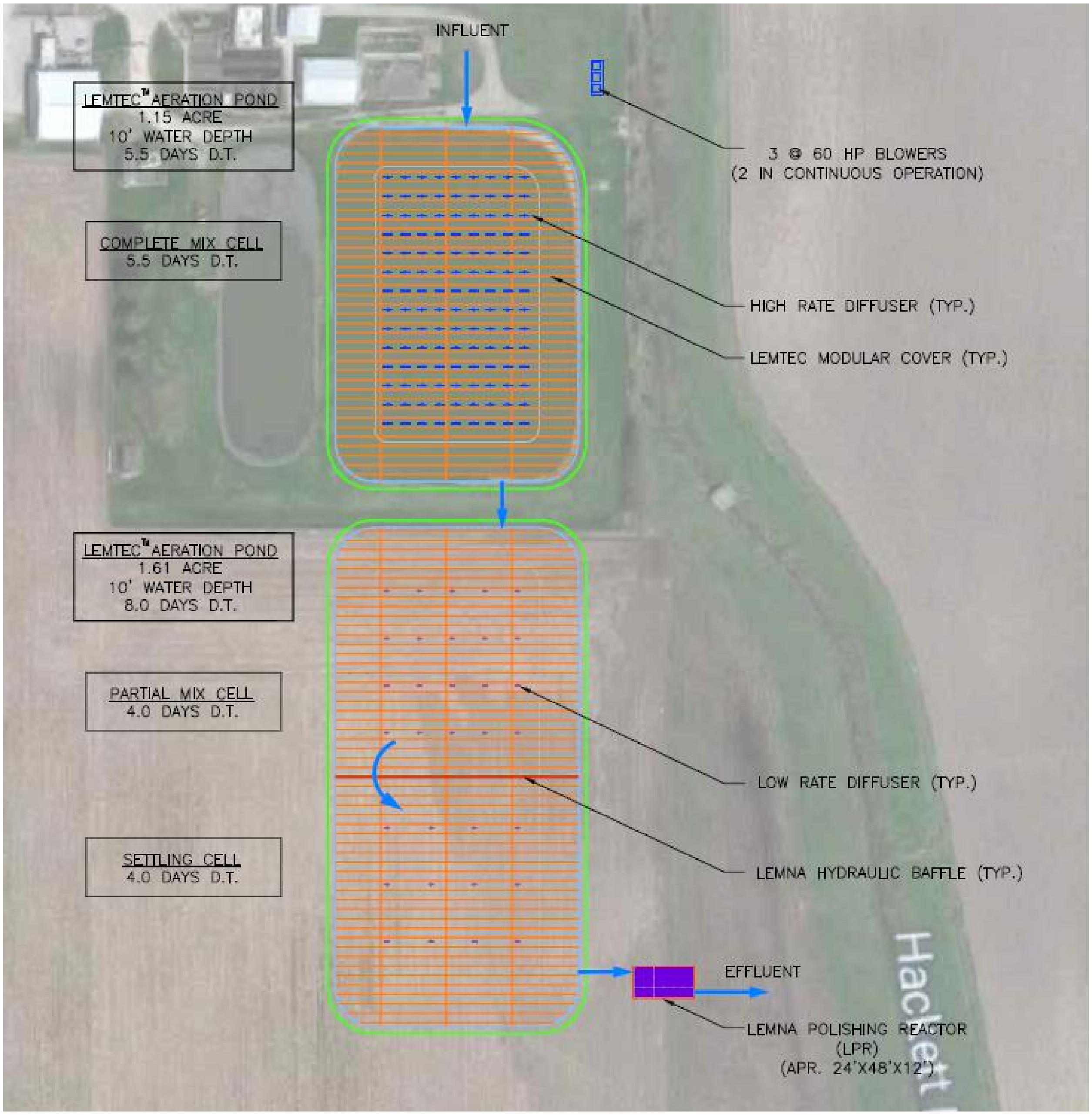

Figure 4-11 LemTec Proposed Aerial layout

4.3 RECOMMENDATION

The oxidation ditch is the recommended alternative due to the low capital costs, the ability to more easily meet future nutrient removal requirements, the flexibility to treat higher future flows, and the relative ease of installation and operation.

5. FINANCIAL AND RATE IMPACTS

A specific recommended financial plan and impacts study on both the water and wastewater systems, which includes the capital costs of the recommended alternative, is detailed in a separate Rate Study completed in November 2019.