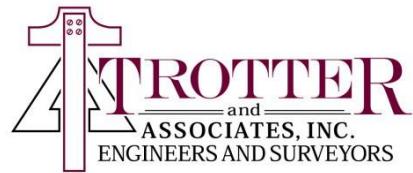


**NORTHWEST REGIONAL WATER
RECLAMATION FACILITY**

2015 FIVE-YEAR FACILITY ASSESSMENT

Prepared By:



PREFACE

The Village of Fox Lake has entered into an inter-governmental agreement with Lake County related to wastewater services provided to several communities throughout northwest Lake County. The inter-governmental agreement requires that the Village of Fox Lake conduct an assessment of the Northwest Regional Water Reclamation Facility (NWRWRF), and that Lake County complete a similar assessment of the wastewater collection facilities, every 5 years.

This report includes a comprehensive assessment of the condition of each treatment process at the NWRWRF with respect to performance, and to identify improvements that are needed to improve the effectiveness and/or efficiency of the facility. Several projects are identified that accomplish these objectives. Opinions of probable capital costs are presented for the highest priority projects that are recommended for implementation over the next 5-year period. Those improvements include: improvements to the aeration blowers; increasing the capacity of the return activated sludge pumping system; construction of a new sludge handling and storage building; and preparation of a phosphorus removal feasibility study. The total capital cost for these projects is currently estimated at approximately \$25M.

The report also includes recommendations for future improvements beyond the current 5-year planning period, including: replacement of the UV disinfection control panel; sidestream treatment facilities; excess flow tanks on the Prairie property; a W3 water filtration system; and rehabilitation of the outfall sewer. The total capital costs to implement these improvements have not been studied in detail but it is currently estimated they in total will amount to another \$25M.

Operational and maintenance costs typically increase at a rate similar to the Construction Cost Index, roughly 3 percent per year. User charges have not been adjusted annually to address these increased costs. Therefore, the adjustment has been made by lowering the amount placed in reserves for replacement of equipment.

Furthermore, expansion of the treatment capacity at the NWRWRF from 12 million gallons per day (MGD) to 15 MGD is currently estimated to cost between \$45M and \$60M. The current facilities have capacity to serve an additional 48,000 PE. Planning and design of the expansion would be triggered after the connection of about 34,000 PE. It is not anticipated that the expansion project will be required for many years.

There are changing regulatory requirements that must be addressed in the future including removal of nutrients (phosphorus and nitrogen) as well as infiltration and inflow. It is unlikely that connection fees will be available to completely fund these capital improvements, as development throughout the service area has slowed. The Illinois EPA is conducting a TMDL study of the Fox River, which may impact NPDES permit limits for nutrients. The communities served by the NWRWRF are currently preparing CMOM Programs and Lake County will commission its findings for the 5-year assessment of the collection system.

It is recommended that the Village update its Facility Plan for the NWRWRF to address rehabilitation, capacity, and regulatory requirements. The plan should also include a User Charge and Connection Fee Analysis to ensure funding is available to implement the recommended improvements.

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1. INTRODUCTION AND BACKGROUND

1.1 GENERAL BACKGROUND

The Northwest Regional Water Reclamation Facility (NWRWRF) is a wastewater treatment facility located in the Village of Fox Lake at 200 Industrial Avenue. The treatment facility is owned and operated by the Village of Fox Lake, and treats the wastewater generated from a total service area of approximately 60,000 acres which includes parts of Fox Lake and several neighboring communities including Ingleside, Long Lake, Round Lake, Round Lake Park, Round Lake Heights, Round Lake Beach, Hainesville, Lake Villa, Antioch, Volo, parts of unincorporated Lake County, and the Lakes Region Sanitary District.

It is estimated that a population equivalent (PE) over 101,000 PE is served by the NWRWRF, which discharges treated effluent just upstream of the mouth of the Fox River, as well as into Pistakee Lake under high-flow events.

The Village of Fox Lake has entered into an inter-governmental agreement with Lake County to provide service to several communities. The agreement requires that the Village conduct an assessment of the NWRWRF every five years. The purpose of this report is to satisfy this requirement and present a summary of the facility assessments performed.



The five-year assessment is intended to address the condition of each treatment process with respect to performance efficiency and effectiveness, and to identify improvements that are needed to improve the effectiveness and/or efficiency of the facility. The existing processes and equipment were inspected and treatment facility operating data, including wastewater flows, pollutant loadings, and effluent water quality was analyzed. The assessments and analyses were performed in conjunction with NWRWRF Staff and with information provided by the Illinois Environmental Protection Agency (IEPA).

1.2 NORTHWEST REGIONAL WATER RECLAMATION FACILITY

The NWRWRF was first constructed in 1979 with capacity to treat a design average flow (DAF) of 6 million gallons per day (MGD). The original treatment facility employed fixed film secondary biological treatment via rotating biological contactors (RBCs). In the early 1990's, the plant was expanded to treat an average flow of 9 MGD with capacity to treat a peak flow of 22.5 MGD. The fixed film RBC process was replaced by an activated sludge biological process.

The treatment facility was most recently expanded in 2012 to increase the average flow treatment capacity to 12 MGD with the ability to treat a peak flow of 30 MGD. The 2012 expansion included conversion of the secondary treatment process from single-stage nitrification to a Modified Lutz-Eddinger (MLE) process.

1.2.1 Existing Facility Basis of Design

The NWRWRF employs preliminary, primary, secondary, and tertiary treatment processes and is designed to treat influent wastewater flows and pollutant loadings as listed in Table 1. These design flows and loadings do not include those attributable to treatment sidestream flows (i.e. sludge thickening filtrate and sludge dewatering centrate) that are returned to the process, which in the case of ammonia ($\text{NH}_3\text{-N}$) is significant and impacts (reduces capacity) the MLE Process.

Table 1: NWRWRF Basis of Design

Parameter	Basis of Design
Design Average Flow (MGD)	12
Design Maximum Flow (MGD)	30
BOD ₅ Loading (lbs/day)	21,520
TSS Loading (lbs/day)	28,520
TKN Loading (lbs/day)	3,300
NH ₃ -N Loading (lbs/day)	2,200
Total Phosphorus Loading (lbs/day)	600

1.2.2 Existing Treatment Processes

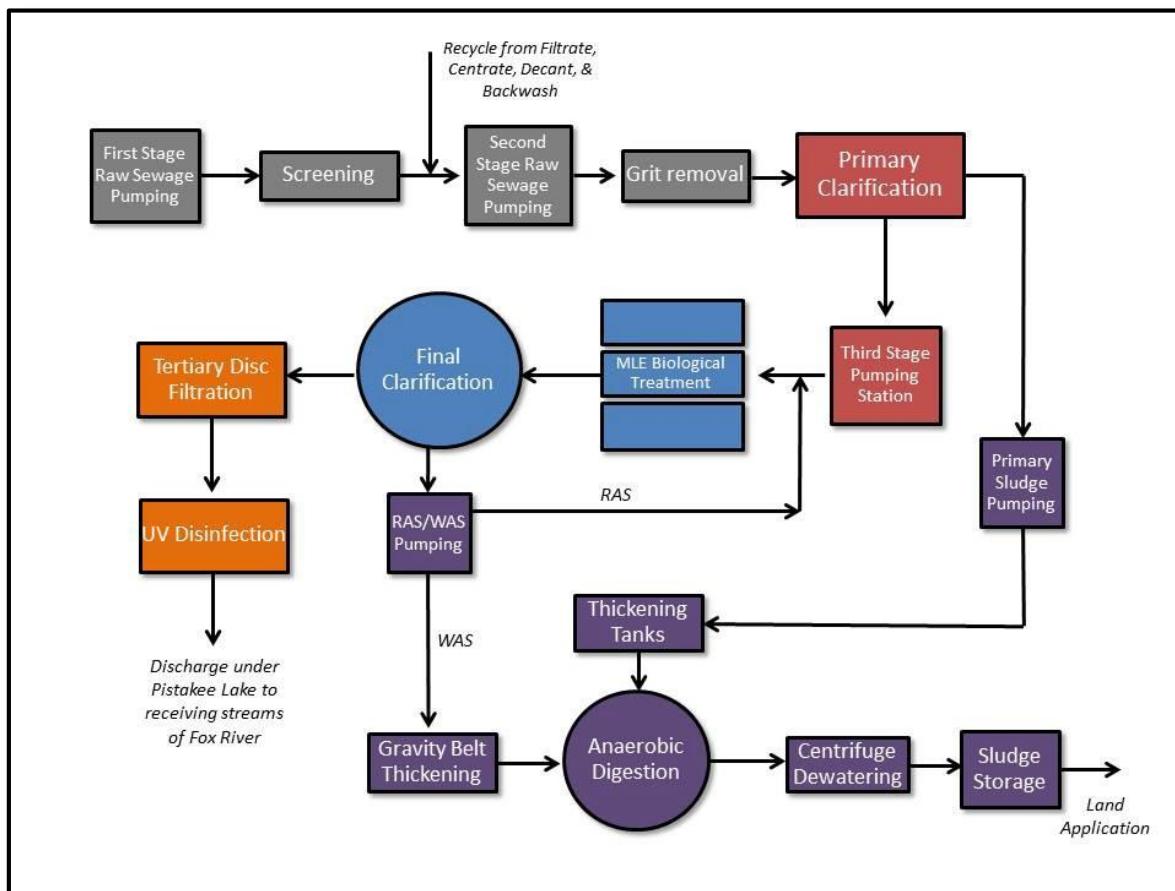
The treatment process at the NWRWRF includes three liquid train pump stations to lift the wastewater so that it may flow by gravity through the preliminary, primary, secondary, and tertiary treatment processes. All influent flow is pumped to the influent screens by the first stage screw pump station. The screens remove inorganic material to protect downstream equipment and processes. The screened wastewater is pumped to the grit removal process by the second stage screw pumps, and flows by gravity from grit removal to primary clarification. The grit removal facility removes dense inorganic material such as sand and gravel, which is hauled to landfill. The primary clarifiers remove settleable solids, which are stabilized and land applied.

Primary clarifier effluent is mixed with return activated sludge (RAS) and then pumped to the MLE process by the third stage screw pump station. The MLE process is designed to metabolize organic material and remove nitrogen compounds. Phosphorus removal is achieved through chemical addition of ferric chloride downstream of the MLE process and upstream of the final clarifiers. MLE process biosolids and phosphorus sludge settle to the bottom of the final clarifiers and are either returned to the MLE process as RAS or removed from the process as waste activated sludge (WAS) for stabilization and land application.

Secondary effluent receives tertiary treatment by rotating disc filtration and is disinfected by ultraviolet (UV) light prior to being discharged to the mouth of the Fox River.

Primary sludge and WAS is thickened prior to digestion (sludge stabilization). Primary sludge is thickened by gravity, while WAS is mechanically thickened using a gravity belt thickener. The anaerobic digestion process reduces volatile solids and includes four primary anaerobic digesters and one secondary anaerobic digester. The stabilized sludge is dewatered by centrifuges. Once dewatered, sludge is stored on-site and land applied by an outside sludge hauling contractor. The existing treatment process diagram is depicted on Exhibit 1.

Exhibit 1: NWRWRF Treatment Process Flow Diagram





2. NPDES PERMIT

The NWRWRF operates under the terms and conditions of its National Pollutant Discharge Elimination System (NPDES) Permit, which regulates the discharge of pollutants of concern in the facility's effluent flow.

The NPDES permit was modified in 2010 to reflect the expansion from 9 to 12 MGD. Effluent limits were modified during that permit cycle to address regulatory changes and anti-degradation requirements. The requirements of that modified permit went into effect upon completion of the plant expansion, which occurred in November 2012. The 2010 modified NPDES Permit expired on August 31, 2013 and a new permit was issued by the IEPA on March 18, 2015. A copy of the current NPDES permit is contained in Appendix A.

The current NPDES Permit has a 5-year term, it became effective on April 2, 2015 and expires on March 31, 2020. Effluent limits contained in the new NPDES Permit are listed in Table 3. These limits are unchanged from the previous permit, with the exception being that a monthly average limit on ammonia is omitted for the months of November through February.

Table 3: 2015 NPDES Permit No. IL0020958 (DAF = 12 MGD)

Parameter	Effluent Limit			
	Monthly Average Loading (lbs/day)	Daily Maximum Loading (lbs/day)	Monthly Average Concentration (mg/L)	Daily Maximum Concentration (mg/L)
CBOD ₅	1,001	2,002	10	20
Suspended Solids	1,201	2,402	12	24
Ammonia Nitrogen				
April-May/Sept-Oct	75	165	1.0	2.2
June-August	75	188	1.0	2.5
Nov-Feb	--	270	--	2.7
March	113	220	1.5	2.2
Total Phosphorus	100	--	1.0	--

The NPDES Permit includes numerous special conditions which further define the terms and conditions for operation of the NWRWRF. Special Condition 16 requires that a Phosphorus Removal Feasibility Report be completed and submitted to the IEPA within 12 months of the effective date of the permit (April 2, 2016). The report must address (specific to the NWRWRF) the method, time frame, and cost for reducing the concentration of phosphorus in the effluent flow to monthly average levels of 0.5 mg/L and 0.1 mg/L. The significance of the proposed, more stringent, phosphorus limits is severe and will carry a high cost in the future.



3. INFLUENT FLOWS AND POLLUTANT LOADINGS

Three years of operating data (2012 to 2014) as reported on NWRWRF Discharge Monitoring Reports (DMRs) were reviewed to analyze flows, loadings, and treatment process performance. The influent data was checked against the facility design parameters as summarized in Table 4. Influent flows and loadings were found to average approximately 60% of the design values.

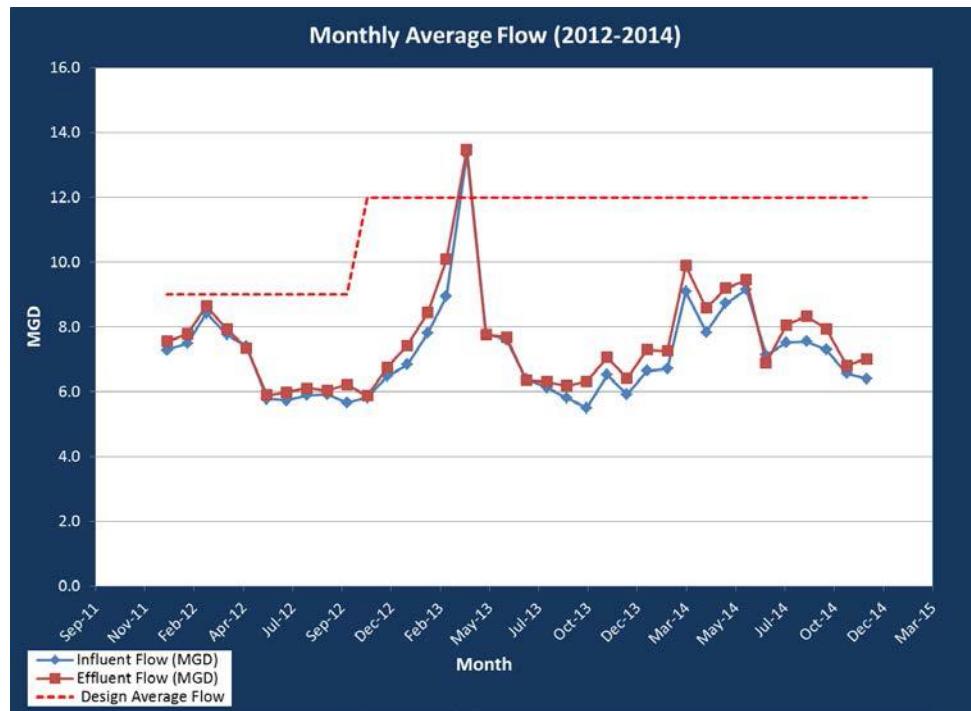
Table 4: Current Influent Flows and Loadings

Parameter	Unit	Current Condition	Design Condition	Percent of Design
Flow	MGD	7.2	12.0	60%
BOD ₅	lbs/day	13,600	21,520	63%
TSS	lbs/day	16,800	28,520	59%
NH ₃ -N ⁽¹⁾	lbs/day	1,300	2,200	59%
Phosphorus ⁽¹⁾	lbs/day	316	600	53%

⁽¹⁾ Ammonia and phosphorus influent loadings limited to two years data (2013 and 2014).

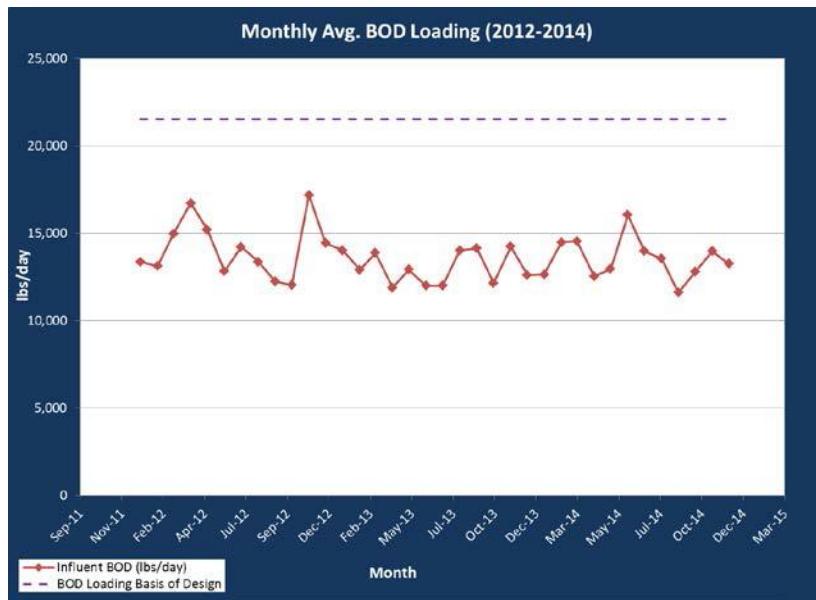
Monthly average influent and effluent flows at the NWRWRF are shown on Figure 1. The daily average influent flow has averaged 7.2 MGD over the past three years, with monthly average flows ranging between 5.5 and 13.3 MGD. Extreme wet weather conditions in the spring of 2013 resulted in a monthly average flow of 13.3 MGD in April due to inflow and infiltration.

Figure 1: Monthly Average Wastewater Flows



Monthly average influent loadings for each pollutant are shown on Figures 2 through 5.

Figure 2: Influent BOD Loading



Monthly average influent BOD mass loadings were well below the plant's design loading of 21,520 lbs/day throughout the three-year period.

The concentration of BOD in the influent flow averaged 226 mg/L, which is indicative of a predominantly domestic wastewater flow.

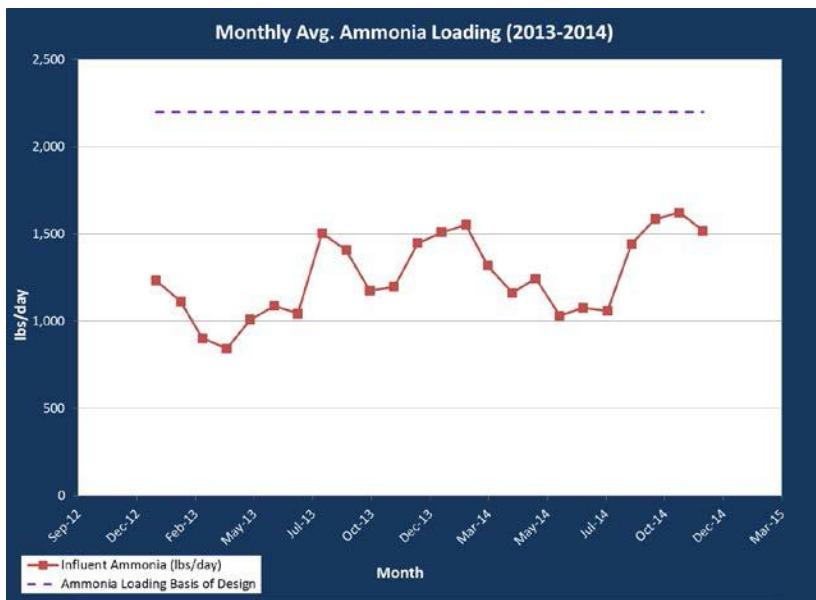
Figure 3: Influent TSS Loading



Monthly average mass loadings of total suspended solids in the influent flow was also comfortably below the plant's design loading of 28,520 lbs/day throughout the period.

The concentration of TSS in the influent flow averaged 280 mg/L, also indicative of a typical domestic wastewater.

Figure 4: Influent Ammonia Loading



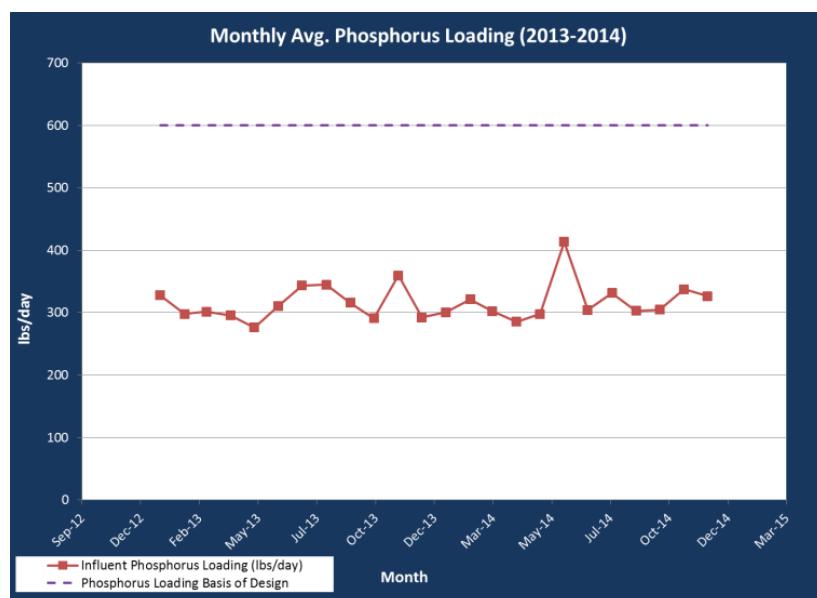
The average monthly mass loadings of total suspended solids was also comfortably and consistently below the plant's design loading of 2,200 lbs/day.

The concentration of TSS in the influent flow averaged 21.6 mg/L, indicative of a typical and normal domestic wastewater flow.

However, the ammonia loading on the biological MLE Process is greatly increased by the high concentration of ammonia

that exists in the sidestream flow that is returned from the sludge dewatering process (centrifuge centrate). Because of this increased sidestream ammonia loading, the biological process becomes stressed to the point that operations staff has needed to place additional aeration tank volume on-line to achieve sufficient nitrification. This deficiency in capacity will become exposed as the influent flow to the plant approaches design. Alternatives to providing additional ammonia removal to the process in order to fully reach the 12 MGD design capacity are presented later in this report.

Figure 5: Influent Phosphorus Loading



The average monthly mass loadings of total phosphorus was only 360 lbs/day in comparison to the plant's design loading of 600 lbs/day.

The concentration of total phosphorus in the influent flow averaged 5.3 mg/L, slightly less than might typically be anticipated.

4. EFFLUENT WATER QUALITY

Three years of effluent water quality data (2012 to 2014) was obtained from the DMRs and reviewed to analyze treatment process performance. Effluent water quality results were judged based upon compliance with the NPDES Permit effluent limits previously listed in Table 3.

Figure 6: Monthly Effluent BOD Concentrations

Effluent BOD concentrations were in compliance with the NPDES limit of 10 mg/L over the past three years, ranging between 1 mg/L and 5 mg/L with an average of 1.36 mg/L.

These are very low levels of BOD in the treated effluent, and are indicative of an effectively operated and maintained, high-performing wastewater treatment facility.

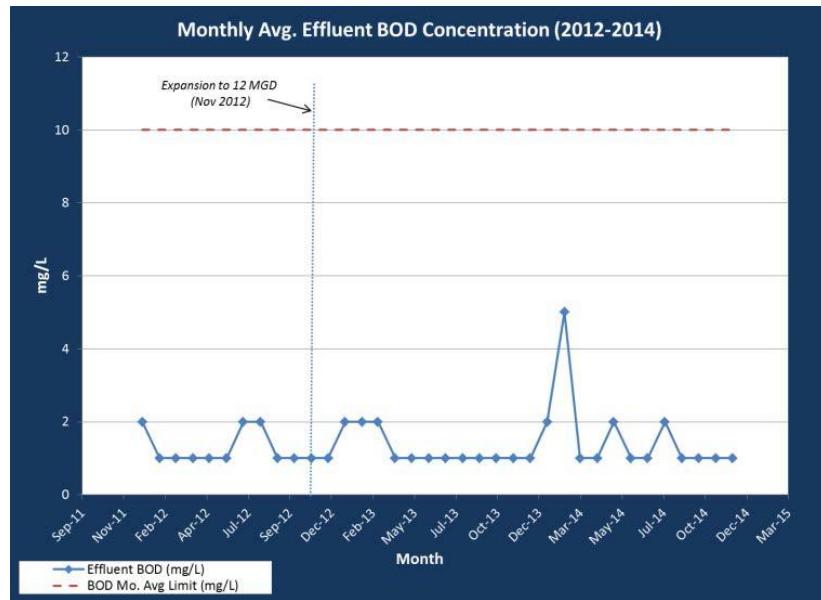
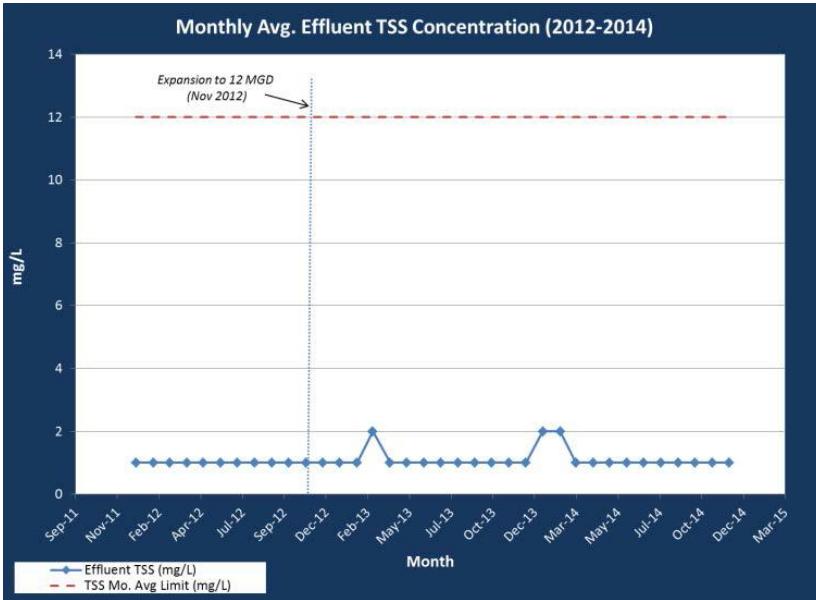


Figure 7: Monthly Effluent TSS Concentrations



Total suspended solids were also in compliance with the NPDES limit of 12 mg/L. The concentration of TSS in the plant effluent ranged from 1 mg/L to 2 mg/L and averaged 1.08 mg/L.

Similarly, these are very low levels of TSS in the effluent flow. The tertiary filters contribute to achieving this high level of solids removal.

Concentrations of ammonia-nitrogen ($\text{NH}_3\text{-N}$) in the effluent has generally remained in compliance with the NPDES permit. However, violations of the effluent limits did occur prior to the most recent plant expansion and also in the months following start-up of the new MLE Process.

The most significant occurrences were between January and April of 2013. These occurred shortly after the biological process was converted to the MLE Process. With the exception of the harsh months of February and March 2014, the process has otherwise done an exceptional job of removing ammonia.

Figure 8: Monthly Average Effluent Ammonia Concentrations

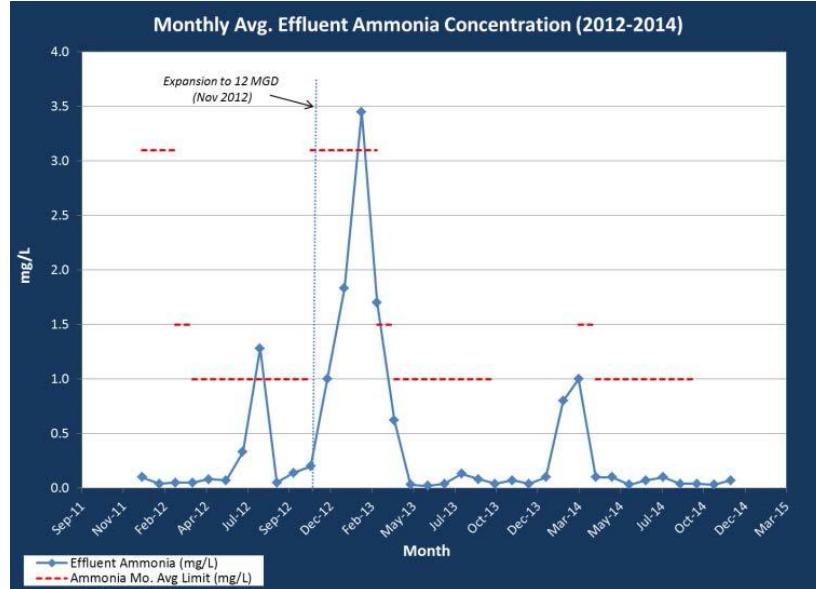
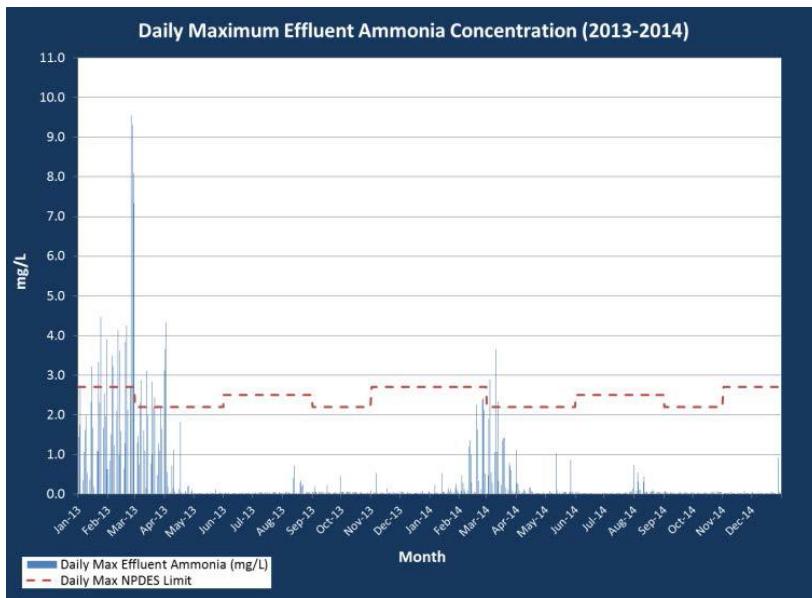


Figure 9: Daily Maximum Effluent Ammonia Concentrations



additional oxygen provided by the third pass diffused air system, has returned the treatment facility to a high level of ammonia removal. This issue is discussed further later in this report.

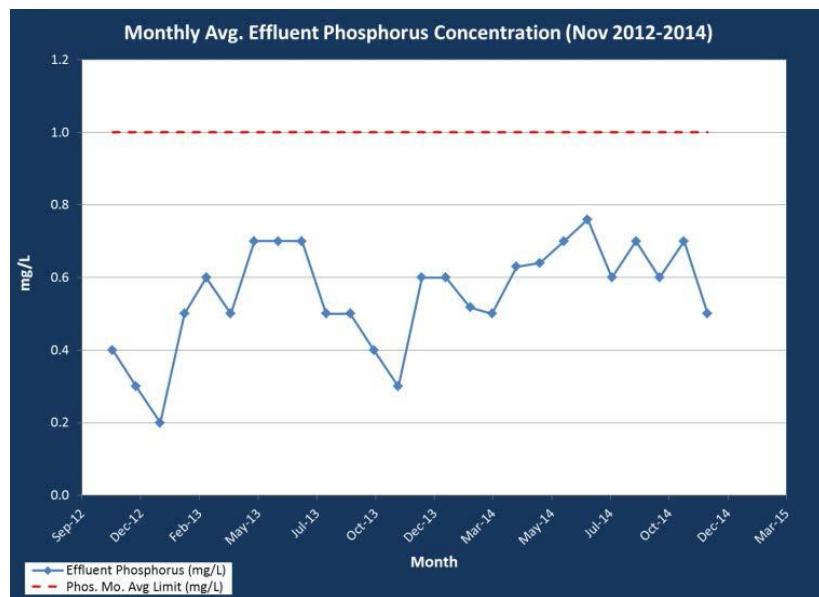
The 2014 excursions are attributed to a combination of factors including power outages during heavy wet weather events, and start-up malfunctions of the automatically actuated air valves in the MLE process. Temperature fluctuations during this period also negatively impacted the effectiveness of the treatment process.

The third pass of the MLE was placed on line to increase the nitrifying bacteria population under aeration, which in combination with the

The effluent limit on total phosphorus of 1.0 mg/L became effective in November 2012 after expansion of the treatment plant had reached substantial completion. During the previous permit cycle, phosphorus levels were only monitored.

Since the chemical phosphorus removal was placed into service, effluent concentrations have consistently been in compliance the NPDES limit, ranging between 0.2 mg/L and 0.8 mg/L with an average of 0.6 mg/L.

Figure 10: Monthly Average Effluent Phosphorus Concentrations



The current NPDES Permit requires submission of a *Phosphorus Removal Feasibility Study* to the IEPA which assesses the method, time frame, and cost of reducing phosphorus levels even further, below the current effluent limit of 1 mg/L down to effluent limits of 0.5 mg/L and 0.1 mg/L. These proposed future effluent limits are significant in that the existing systems alone will not be capable of achieving the most stringent of future limits (0.1 mg/L). The feasibility study must be submitted to the IEPA within 12 months of the effective date of the NPDES Permit, or April 2, 2016.

In summary, the treatment processes that are employed at the NWRWRF have in general consistently complied with NPDES standards. The current wastewater flows and pollutant loadings to the treatment facility do not warrant any immediate expansion. However, the ability of the existing facility to consistently meet ammonia standards will become stressed as flows increase because of the additional loading contributed from the sludge dewatering centrifuge centrate sidestream flow. Improvements and enhancements to the existing systems to address the sidestream flows are discussed later in this report. Finally, when phosphorus limits are lowered to the most stringent levels currently under consideration by the IEPA, additional tertiary treatment (for example, membrane or reactive filters) will likely be required.

Each of the individual wastewater treatment and sludge handling processes at the NWRWRF are discussed in further detail in the following sections of this report.

5. TREATMENT PROCESSES

5.1 SEPTAGE RECEIVING

Private waste haulers have the ability to dump septic and holding tank waste at the treatment facility for a fee. The septic receiving station is equipped with Hycor Helisieve equipment and also flow meters, pH monitors, and sampling capabilities. The equipment removes, dewater and compact heavy solids and grit from the flow prior to introduction to the liquid treatment train. The material that is removed is disposed of to landfill.



Performance and Deficiencies

The station is approximately 15 years old but still in working condition. Septic receiving stations can often lead to increased BOD and TSS loadings in a way that impedes upon a facility's removal processes and increases sludge production resulting in greater sludge disposal costs. However, the amount of sewage received from off-site haulers at the NWRWRF is negligible. In 2014, the total volume of septic waste received for the entire year was only 0.57 million gallons. At a facility designed for 12 million gallons *per day*, this contribution will not affect the treatment process in a meaningful manner.



5.2 FIRST STAGE RAW SEWAGE PUMPING

Wastewater enters the facility through an influent chamber located 20 feet below grade. Influent flow enters the site via the 60-inch Rollins Road Interceptor sewer as well as the Fox Lake 20-inch gravity sewer and the Menard Lift Station 12-inch force main. A manually cleaned bar screen is located in the influent chamber and removes large debris (6-inches and greater) from the flow. The bar screen was replaced with a new stainless steel bar screen in April 2015.

The First Stage Raw Sewage Pumping Station is comprised of five identically sized screw pumps. Under normal operation, three pumps operate to convey flow into the preliminary screening process. Four of the five pumps must run to lift the peak design flow of 30 MGD.

Number of Units	5 each
Screw Diameter.....	60 inches
Horsepower.....	50 HP
Capacity per Unit	7.5 MGD
Installed Capacity.....	37.5 MGD
Firm Capacity (with largest unit out of service).....	30 MGD



Performance and Deficiencies

The only issues prevalent at the first stage raw sewage pumping station were some structural deterioration in the stairwell and corrosion of the influent bar screen. Both of these issues were addressed in April 2015 as part of the 2015 Headworks Rehabilitation Project. When influent flows necessitate expansion of the NWRWRF to treat a peak flow greater than 30 MGD, a sixth screw pump would need to be installed.

5.3 MECHANICAL SCREENING

Inorganic material including rags, plastic, and other debris is removed by means of mechanical screening. Two Andritz automatic perforated plate fine screens are used for normal operations (full redundancy is provided) and one manual screen is used during bypassing. The screens are enclosed within a climate-controlled building. Flow metering takes place directly downstream of the screens by means of an open channel area-velocity meter.

Number of Units	2 each
Screen Width, each	5 feet
Channel Depth	12.8 feet
Opening Diameter.....	6 mm
Rated Capacity, each.....	30 MGD



Performance and Deficiencies

The fine screens have 6 mm circular openings which allow very little inorganic material to pass. As such, the fine screens do an excellent job of processing the raw wastewater, protecting the downstream pumps and processes. The screens have had corrosion issues in the past but were recently rehabilitated in 2014.

The influent flow metering has been operating satisfactorily. Operating staff have reported some minor issues with buildup of solids and grit on the meter elements.

5.4 SECOND STAGE SEWAGE PUMPING

Screened wastewater enters a second stage pumping station where it is combined with recycle flows: gravity thickener supernatant, gravity belt thickener filtrate, centrifuge centrate, tertiary filter backwash, and anaerobic digester supernatant. Flow is pumped via five screw pumps that are configured identically to the first stage pumping station.

Number of Units	5 each
Screw Diameter.....	60 inches
Horsepower.....	50 HP
Capacity per Unit	7.5 MGD
Installed Capacity.....	37.5 MGD
Firm Capacity (with largest unit out of service)	30 MGD



Performance and Deficiencies

The second stage screw pumps are in good condition. It should be noted that this pump station must pump not only the peak influent design flow of 30 MGD, but also the recycle flows, which add up to 1.11 MGD. When influent flows necessitate expansion of the facility, a sixth screw pump is recommended to be installed.

5.5 GRIT REMOVAL

The grit removal system (Building 30) was originally constructed as an aerated grit tank. The system has since been modified to install Headcell grit concentrators in the aerated grit tanks. Building 30 currently contains two Eutek HeadCell concentrator units, a Eutek Slurry Cup and Grit Snail, and grit pumps. In 2009, expansion of the facility added 48 inches of weir length to the original system as well as a second grit system (Building 31) containing one HeadCell, Slurry Cup, and Grit Snail to provide redundancy.



	Building 30	Building 31
<u>Grit Concentrators</u>		
Number of Concentrators.....	2 each.....	1 each
Concentrator Average Capacity, each.....	4 MGD.....	4 MGD
Peak Capacity, each	12.5 MGD.....	12.5 MGD
Concentrator Total Capacity	8 MGD.....	4 MGD
Total Peak Capacity	25 MGD.....	12.5 MGD
Max Surface Loading Rate	118 gpm/ft ²	118 gpm/ft ²
<u>Grit Pumps</u>		
Number of Pumps	2 each.....	2 each
Capacity, each.....	300 gpm.....	200 gpm
Total Capacity.....	600 gpm.....	400 gpm
TDH, each	32 feet.....	30 feet
Horsepower	10 HP.....	7.5 HP
<u>Grit Classifiers/Escalators</u>		
Number of Units.....	1 each.....	1 each
Capacity, each.....	185-295 gpm.....	185-295 gpm
Escalator Capacity.....	2.0 yd ³ /hr	1.0 yd ³ /hr

Grit Concentrators

Number of Concentrators.....	2 each.....	1 each
Concentrator Average Capacity, each.....	4 MGD.....	4 MGD
Peak Capacity, each	12.5 MGD.....	12.5 MGD
Concentrator Total Capacity	8 MGD.....	4 MGD
Total Peak Capacity	25 MGD.....	12.5 MGD
Max Surface Loading Rate	118 gpm/ft ²	118 gpm/ft ²

Grit Pumps

Number of Pumps	2 each.....	2 each
Capacity, each.....	300 gpm.....	200 gpm
Total Capacity.....	600 gpm.....	400 gpm
TDH, each	32 feet.....	30 feet
Horsepower	10 HP.....	7.5 HP

Grit Classifiers/Escalators

Number of Units.....	1 each.....	1 each
Capacity, each.....	185-295 gpm.....	185-295 gpm
Escalator Capacity.....	2.0 yd ³ /hr	1.0 yd ³ /hr

Performance and Deficiencies

It has been reported that the slurry cup in Building 30 needs to be replaced approximately every five years due to regular wear-and-tear from the centrifugal force. There have also been operational issues related to the operation of the two submersible grit pumps for this system due to their piping configuration. The influent ducts for the two HeadCells in Building 30 are currently being replaced with new stainless steel units, scheduled for completion in May 2015.



5.6 PRIMARY CLARIFICATION

After removal of inorganic materials and grit, flow is conveyed to six primary clarifiers for removal of suspended solids and BOD. Solids are allowed to settle and clarified effluent continues to secondary treatment. Primary settled sludge and scum skimmed from the surface of the tanks are pumped to the gravity thickeners prior to anaerobic digestion. Improvements to the clarifiers including new valves and piping occurred during the most recent expansion.

Number of Tanks	6 each
Length, each.....	85 feet
Width, each	30 feet
Sidewall Depth, Units 1-4.....	8 feet
Sidewall Depth, Units 5-6.....	10 feet
Total Volume	991,848 gallons
Total Surface Area	15,300 sf
Surface Loading at DAF including recycle flow	809 gpd/sf
Surface Loading at Peak Flow including recycle flow	1,986 gpd/sf
Total Weir Length.....	900 lf
Weir Overflow Rate at Peak Flow including recycle flow	33,760 gpd/lf



Performance and Deficiencies

The primary clarification process has been reported to be working effectively. Surface overflow rate at peak design flow is slightly less than that recommended in the Ten State Standards (2,000 gpd/sf), while weir overflow rates slightly exceed the maximum recommended by the Ten State Standards (30,000 gpd/lf). When influent loadings necessitate expansion of the primary treatment process, two additional clarifiers are recommended to be constructed to the east of Tanks 5 and 6. Primary sludge pumping is discussed further in Section 5.12.



5.7 PRIMARY EFFLUENT PUMP STATION (THIRD STAGE)

Effluent from the primary clarifiers is pumped to secondary treatment via five screw pumps that are configured identically to the first and second stage pump stations. These pumps were refurbished during the most recent expansion of the facility.

Number of Units	5 each
Screw Diameter.....	60 inches
Horsepower.....	50 HP
Capacity per Unit	7.5 MGD
Installed Capacity.....	37.5 MGD
Firm Capacity (with largest unit out of service)	30 MGD



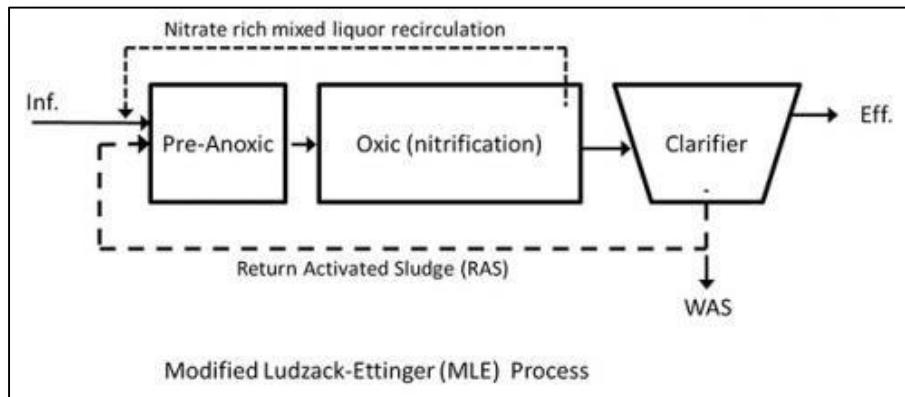
Performance and Deficiencies

Operations staff reported that the original drive boxes at this pumping station were different from those of the stage one and stage two pumping stations. Several operational issues occurred prior to the pumps being re-equipped with drives identical to those installed at the other two stations. Staff has not experienced any notable operational issues since then. When influent flows necessitate expansion of the facility, a sixth screw pump is recommended to be installed.

5.8 SECONDARY TREATMENT (MLE PROCESS)

Secondary biological treatment of wastewater consists of an MLE process made up of three parallel trains with three passes per train. The mixed liquor flows through an anoxic zone before continuing through two aerobic zones where nitrification occurs. Internal recycle flow carries nitrate created in the aerobic zone to the head of the process for denitrification.

Figure 11: MLE Process



Aeration is provided to the tanks via four blower units, two of which are backup units and two of which are new variable-speed turbo blowers. A fifth blower exists within the blower building that is diesel engine-driven. The engine-driven blower is used only during power outages.

In addition to the aeration process, chemical addition of ferric chloride facilitates the removal of phosphorus in the waste stream. Chemical addition occurs at the mixed liquor diversion structure prior to final clarification. The system includes two 6,000 gallon tanks which store the chemical in an enclosed building as well as chemical feed pumps. The average daily ferric dosage from 2013-2014 was 23 mg/L. On average, 286 gallons of ferric chloride solution was used per day for phosphorus removal during this time period.

Aeration Tanks

Number of Basins	3 each
Passes per Basin.....	3 each
Length, each	112 feet
Width, each	40 feet
Sidewall Depth, each	20 feet
Total Volume	806,400 cu ft
BOD ₅ Loading at 30% primary removal	15,923 lbs/day
Maximum BOD ₅ Loading.....	40,965 lbs/day
Average Oxygen Requirement.....	26,045 lbs/day
Max Oxygen Requirement.....	66,265 lbs/day
Average NH ₃ Loading.....	2,200 lbs/day
Maximum NH ₃ Loading	5,500 lbs/day

Multi-Stage Centrifugal Blowers

Number of Units	3 each
Blower Capacity, each	5,800 SCFM
Blower Horsepower	400 HP
Installed Capacity.....	17,400 SCFM
Firm Capacity.....	11,600 SCFM

Variable Speed Turbo Blowers

Number of Units	2 each
Turbo Blower Capacity, each	3,500 SCFM
Turbo Blower Horsepower	200 HP
Installed Capacity.....	7,000 SCFM
Firm Capacity.....	3,500 SCFM

Performance and Deficiencies

When influent loadings necessitate expansion of the biological process, a fourth train is recommended to be added to the MLE process that will provide an additional anoxic zone and two aerobic zones. These tanks should be installed directly west of the existing tanks. Additionally, the existing multi-stage centrifugal blowers (excluding the engine-driven blower) are scheduled to be replaced with positive displacement units in 2015.

During startup of the MLE process in 2013 as well as the spring of 2014, the plant experienced operational issues which exceeded NPDES permit limits for ammonia. In the summer of 2014, TAI performed an analysis of the existing MLE process and identified limitations related to the overall detention time and mixed liquor concentration required for conversion of ammonia to nitrite and ultimately nitrate.

The existing MLE process includes three process trains. The plant during this period was only operating 66% (two trains) based on influent load in comparison with design load values. The inability to adequately convert ammonia was directly related to the temperature, mixed liquor volatile suspended solids concentration, and available detention time in the aerobic zones. It may have also been effected by limitations in oxygen transfer efficiency of the existing system.

TAI worked with NWRWRF staff to evaluate the MLE process and developed a tool to help predict the appropriate amount of mass to provide complete conversion of ammonia based on influent loadings. Staff has utilized this in the spring of 2015 and it appears that the operational issues have ceased. However, it should be noted that in order to accomplish this objective staff has needed to operate all three trains even though influent flow is only 60% of the design flow.

Based on this previous analysis, the NWRWRF will need to either revert back to single-stage nitrification, or provide expanded aeration basin capacity to utilize the MLE process at the full average day design flow of 12 MGD. Alternatively, sidestream treatment to remove the ammonia loading created in the digestion process may be sufficient.

TAI performed an analysis to determine the recommended alternative for improvement of the MLE process. The design loading of ammonia coming into the process was calculated to be the sum of the design primary effluent ammonia loading plus all ammonia loads from the recycle sidestream flows. Assuming winter conditions (9-deg C), the pounds of MLVSS required to nitrify the ammonia loading was calculated. This was converted to a concentration and then used to determine the effective MLSS concentration assuming 75% volatile solids (existing condition). The MLSS concentration computed in this manner was compared to a recommended operational maximum MLSS concentration of 3,500 mg/L.

It was concluded that the existing MLE process is not capable of complete nitrification at design flow (12 MGD) and loadings without running the process at a much higher MLSS concentration than recommended. Higher MLSS concentrations also lead to other operational issues such as the limitations posed by the return activated sludge pumps.

If the anoxic zones were converted to aeration creating a single-stage nitrification process, nitrification could be accomplished under existing conditions with two trains running and under design conditions with three trains running. However, IEPA anti-degradation requirements entail the use of both nitrification/denitrification, so this option is not feasible.

If a fourth MLE train was constructed and put online, nitrification could be accomplished at full design flows and loadings. Alternatively, the same result could be reached by removing the sidestream ammonia prior to mixing with the primary effluent. In this case, the system could continue to be effective using only three trains at design. A further analysis into sidestream ammonia removal is described in Section 6.1.

A quantitative summary of this analysis is included in Table 5 below.

Table 5: Ammonia Removal

Parameter	Unit	Existing Conditions (2 Trains)	Existing Conditions (3 Trains)	Design Conditions	Design minus Sidestream	Design + 4th Train
Flow	MGD	7.5	7.5	12	12	12
Number of Trains Running	trains	2	3	3	3	4
Primary Effluent Ammonia Loading	lbs/day	1,375	1,375	2,200	2,200	2,200
Recycle Flow Ammonia Loading	lbs/day	428	428	685	0	685
Total Ammonia Loading to Process	lbs/day	1,803	1,803	2,885	2,200	2,885
Ammonia applied/MLVSS Required	ratio	0.0090	0.0135	0.0084	0.0085	0.0113
MLVSS required for Nitrification	lbs/day	199,947	133,269	341,259	259,434	255,921
MLVSS required for Nitrification	mg/L	3,197	2,131	3,410	2,592	2,557
Recommended MLSS Concentration	mg/L	3,500	3,500	3,500	3,500	3,500
MLSS Required @75% Volatile	mg/L	4,262	2,841	4,546	3,456	3,410



5.9 SECONDARY CLARIFICATION

Following biological treatment of wastewater, flow is conveyed to four circular secondary clarifiers of which three are utilized during normal operation. One secondary clarifier is typically kept offline because current flows do not warrant placing it into service under normal conditions. This unit is used for reserve clarification in high-flow events, and will be placed into normal service when influent flows warrant. Return activated sludge (RAS) from secondary treatment is recycled to the beginning of the MLE process. This is done by means of five variable-speed centrifugal RAS pumps. The piping arrangement allows for each clarifier to be served by either of two of the five available pumps.



Secondary Clarifiers

Number of Tanks.....	4 each
Diameter	100 feet
Sidewall Depth	14 feet
Total Surface Area.....	31,416 sf
Total Capacity	3,290,000 gallons
Total Weir Length	1,256 feet
Detention Time at DAF	6.4 hour
Surface Settling Rate at DAF	382 gpd/sf
Surface Settling Rate at DMF	955 gpd/sf
Weir Overflow Rate at DAF	9,550 gpd/lf

RAS Pumps

Number of Units.....	5 each
Pump Rated Capacity, each.....	2,100 gpm
Pump Minimum Flow, each	450 gpm
Pump Horsepower, each.....	40 HP
Installed Pumping Capacity.....	15 MGD
Firm Pumping Capacity.....	12 MGD

Following secondary clarification, flow enters a valve vault where it is either conveyed to tertiary filtration or diverted around the filters. Bypassed flow is measured at a Parshall flume and chlorinated at a downstream sampling manhole.

Performance and Deficiencies

Operating staff have noted that the MLE process at times requires a long MCRT to achieve nitrification. Long MCRTs promote the growth of filamentous bacteria. Staff have expressed the desire to chlorinate the return activated sludge to remove filamentous bacteria, as excess filamentous negatively impacts sludge settleability. This would entail the installation of a chlorine feed in the lower level of the RAS pumping station. This is a common operational control strategy for improving sludge settleability, however it should not be relied upon on a continuous basis. It is preferred to control sludge age and thereby filamentous.

Additionally, staff have noted that the capacity of the RAS pumps are not able to sufficiently return activated sludge under high flow events if the MLSS concentration is maintained at a high level, and as such MLSS is maintained at around 2,800 mg/L. Given a maximum design flow of 30 MGD, a primary effluent TSS concentration of 158 mg/L, and a RAS concentration of 7,500 mg/L, a mass balance was performed to determine the required RAS pumping rate. This was calculated to be approximately 16.9 MGD under these conditions, which exceeds the current firm RAS pumping capacity of 12 MGD.

In order to run the MLE process at a MLSS concentration of 4,220 mg/L per the design, the RAS pumping rate would need to be increased to almost 29 MGD during peak flow events. This would greatly exceed the current firm RAS pumping capacity of 12 MGD, and returning sludge at that rate would overwhelm the hydraulics of the existing processes causing tank overflows, and is therefore not recommended.

A summary of the RAS pumping requirements is provided in Table 6 below. In conclusion, the RAS pumps will need to be upsized for additional capacity. The motor controls for these pumps have also had technical issues and need to be addressed. When necessitated, expansion of the facility will include the construction of a fifth clarifier.

Table 6: RAS Requirements

Parameter	Existing RAS Conditions			Design RAS Conditions	
	Existing Conditions	Design Average Flow	Design Maximum Flow	Design Average Flow*	Design Maximum Flow*
Flow (MGD)	7.5	12	30	12	30
Primary Effluent TSS Concentration (mg/L)	158	158	158	158	158
RAS Concentration (mg/L)	7,500	7,500	7,500	8,440	8,440
MLSS Concentration (mg/L)	2,800	2,800	2,800	4,220	4,220
RAS Flow Required (MGD)	4.22	6.75	16.86	11.55	28.88
Total RAS Pumping Capacity (MGD)	12	12	12	12	12

*Based on Clark-Dietz design MLSS Concentration and assuming 1% RAS

5.10 TERTIARY FILTRATION

Tertiary filtration provides further BOD and TSS removal utilizing seven rotating disc filters manufactured by Kruger. All seven units are used for filtration. The disc filters were installed to replace the previously existing sand filters in 2009 and provide improved effluent quality.

Number of Filters	7 each
Length, each	22.3 feet
Width, each	7.5 feet
Height, each8 feet
Surface Area, each	1,203 sf
Total Submerged Area, each.....	760 sf
Pore Size	10 microns
Drive Horsepower.....	1.5 HP
Total Submerged Area, one offline.....	4,560 sf
Hydraulic Loading at DMF, one offline	4.57 gpm/sf



Performance and Deficiencies

At this time there is an open trough for an eighth disc filter to be installed when necessitated. Operations staff have expressed that this space may alternatively be used to implement a proposed W3 water filter system. Additionally, there exists a room within the filter building that has unused equipment and could be repurposed as a storage facility. Improvements to the ventilation system in this building should also be addressed.



5.11 DISINFECTION

Tertiary effluent is conveyed to a high intensity, low pressure ultraviolet disinfection system, installed in 2004 to replace the chlorination system. Field tests have shown the existing UV system to be effective at the DMF of 30 MGD. The system is likely over-designed as a result of the filtered tertiary effluent having a much higher UV transmittance than the 65% transmittance upon which the system was designed. Final effluent flow metering is performed downstream of the UV disinfection system using an ultrasonic level sensor mounted upstream of a sharp-crested weir. Effluent flows are recorded on the facility's SCADA system.

Channel Length.....	30 feet
Channel Width	56 inches
Channel Depth	62 inches
Number of Lamps	224 each
Number of Banks2 each
Modules per Bank.....	14 each
Lamps per Module8 each
Design Peak Flow25 MGD



Performance and Deficiencies

Operating staff have experienced issues with the UV System Control Panel, namely extremely slow response times when in use. It is recommended that this panel be replaced with updated technology. Additionally, the effluent automatic level control (ALC) gate at the UV channel is in need of calibration by the manufacturer to ensure proper level control.



5.12 PRIMARY SLUDGE PUMPING AND THICKENING

Sludge from the primary clarification process is pumped by four pneumatically-operated diaphragm pumps, which pump primary sludge to two circular gravity thickener tanks. Solids settle and the clear supernatant is decanted and returned to the second stage pumping station. Thickened sludge is pumped to the anaerobic digesters via two rotary lobe pumps.

Number of Primary Sludge Pumps	4 each
Rated Capacity, each.....	90 gpm
Installed Pumping Capacity	360 gpm
Firm Pumping Capacity (with largest unit out of service).....	270 gpm
Number of Thickener Tanks	2 each
Tank Diameter	40 feet
Sidewall Depth.....	8 feet
Surface Area, each	1,256 sf
Capacity, each	94,000 gallons
Total Capacity.....	188,000 gallons
Rotary Lobe Pump Rated Capacity, each	125 gpm



Performance and Deficiencies

The primary sludge pumps for Primary Clarifiers 1-4 are housed in the lower level of the clarifier structure in a narrow space. Due to space constraints, it is difficult to maintain the equipment, and it would be difficult to facilitate replacement of pumps or install additional equipment in the future. It is recommended that the compressor that is present in this location be relocated. This would free up some space for ease of maintenance and to facilitate expansion.



5.13 WASTE ACTIVATED SLUDGE PUMPING AND THICKENING

Waste activated sludge (WAS) from the secondary clarifiers is thickened by means of two gravity belt thickeners (GBT). The sludge is supplied to the belt thickeners by two variable speed progressing-cavity feed pumps. At low heads, scum is not pumped but is rather drained to the head of the treatment facility. Polymer is added to the WAS prior to thickening to provide chemical conditioning and enhance thickening. There is always at least one GBT operating on a 24-hour basis. Thickened sludge is pumped to a sludge well by two variable-speed progressing-cavity pumps.

Number of Gravity Belt Thickeners	2 each
Belt Thickener Horsepower	5 HP
Width.....	2 meters
Weekly Scheduled Operation	168 hours
Sludge Processed per Day.....	14,190 lbs
Design Solids Loading.....	505 lbs/hr/meter
Number of WAS Pumps	2 each
Feed Pump Horsepower.....	15 HP
Feed Pump Total Rated Capacity	600 gpm



Performance and Deficiencies

A liquid polymer is added to the WAS prior to belt thickening. The polymer system currently in use has been reported to have excellent results. However, it is outdated and no longer being manufactured, so spare parts are not readily available for repairs when necessitated. It is recommended that the polymer system be replaced with a newer, serviceable system.



5.14 ANAEROBIC DIGESTION

All sludge produced by primary and secondary clarification is processed by means of anaerobic digestion. The treatment facility includes five digesters, of which four are considered primary and one is considered secondary. Several modifications have occurred over the years including rehabilitation of mixers and covers. The secondary digester received a spray-wall liner in 2014 to help prevent against digester gas leaks.

Number of Primary Digesters	4 each
Primary Digester Diameter, each.....	60 feet
Primary Digester Sidewall Depth, three units.....	20 feet
Primary Digester Sidewall Depth, one unit	22 feet
Number of Secondary Digesters	1 each
Secondary Digester Diameter	45 feet
Secondary Digester Sidewall Depth	22 feet
Total Digestion Capacity	261,404 cu ft
Total Design Solids Load, 6% solids, 70% VSS	27,010 lbs/day
Retention Time per Unit	37 days



Performance and Deficiencies

The facility was designed to allow for installation of a fifth primary digester. When solid loadings necessitate expansion of the solid stabilization process, the digester should be constructed in its designated location.

5.15 DEWATERING

Sludge is dewatered and concentrated using two centrifuge units. Under normal conditions only one centrifuge is utilized and run at roughly 100 GPM. Typical operation provides a 95.5% solids recapture, producing a dewatered sludge solids concentration of approximately 22%.

Number of Units	2 each
Rated Capacity, Unit 1	150 gpm
Rated Capacity, Unit 2	200 gpm
Daily Run Time.....	16 hours



Performance and Deficiencies

The dewatering building is has little to no room for equipment replacement or expansion of dewatering process capabilities. Therefore it is recommended that a new sludge dewatering building be constructed to allow for expansion of dewatering capacity. Additionally, power system constraints only allow for one centrifuge to be run at a time. A new dewatering building could provide sufficient power supply and open floor space for installation of any future process equipment. Implementation of this building is detailed in Section 6.2.



5.16 SLUDGE STORAGE AND DISPOSAL

After dewatering, sludge is stored in the sludge storage barn. The treatment facility operates under a sludge permit for the land application of 1,600 dry tons of digested sludge per year (combined for both the NWRWRF and the Tall Oaks treatment facility). The NWRWRF is currently in compliance with all conditions under the sludge permit. Sludge is hauled and land applied by a private sludge hauling contractor, Dahm Trucking.

Storage Barn Length	120 feet
Storage Barn Width.....	90 feet
Storage Barn Area.....	10,800 sq ft



Performance and Deficiencies

Existing sludge storage at the facility has proven to be adequate. However, future loadings will increase the volume of sludge produced and therefore increase the required storage capacity. Consideration of this and recommendations for addressing these needs is detailed in Section 6.2.



5.17 ODOR CONTROL

Three odor control buildings contain systems to strip odorous compounds from the air ventilated from odorous treatment processes prior to discharge to atmosphere. The odor control facilities are located at the headworks, the thickening tanks, and the sludge storage barn. The facility at the storage barn is designated solely for odor control of dewatered sludge being stored.



5.18 DIGESTER GAS

Digester gas is created as a by-product of anaerobic digestion. This gas is fed through a gas conditioning skid system and burned in an engine. The energy created from the engine is returned to the grid to supply additional power to the facility. Insulation of the system has been necessary due to cold outdoor temperatures.



5.19 ELECTRICAL SYSTEMS

The electrical system at the NWRWRF consists of primary distribution at 13.2 KV (three circuits). The 13.2 KV circuits feed transformers at the Control Building (Distribution Center 1), and two transformers at the Blower Building (T1 and T2) which feed 45-MCC-1 Sections 1 & 2. These transformers step-down the 13.2 KV primary power to 3-phase 480 volt secondary power.

As-built drawings (B&W 1975) indicate that the three phase 480 V secondary power at the main Control Building (and Bar Screen and Generator Building) is configured as 3-phase, 3-wire, and is ungrounded. In addition, later as-built drawings (Donohue 1990) indicate that 3-phase 480 V secondary power at the Blower Building is also 3-wire, and is ungrounded. It is believed that these systems are ungrounded wye secondary systems, rather than ungrounded delta.

Ungrounded in this context means there is not a transformer neutral (or in some cases B phase) that is bonded to ground, and therefore the system has no earth reference. Since there is no earth reference, while the line to line potential is 480 volts, line to ground potential is not well constrained.

Under certain conditions, the line to ground potential in ungrounded delta systems can exceed 3800 volts, and can exceed 7600 volts in ungrounded wye systems. ComEd often will try to control these voltages by installing a shunt MOV (metal oxide varistor) on the B phase of ungrounded systems, and bond the MOV to ground. The MOV allows voltages in excess of 600 volts to pass through to ground, and can help to control the system.

In modern plants which have variable frequency drives and other equipment which produces electrical noise, these shunt MOVs become overheated and fail, resulting in a completely unprotected system, and extreme over-voltages. These extreme voltages can damage motors, control power transformers, UPS systems, and Variable Frequency Drives, which typically have an internal withstand rating of 3000 volts.

These are extreme cases which require specific conditions, however less extreme over-voltages are common, and will damage electrical equipment over time. There are effective mitigation strategies that can be used to reduce the hazard associated with ungrounded systems, including harmonic filters and High Resistance Grounding systems (HRG).

HRG systems provide a controlled path to ground, establish an earth reference, limit overvoltage to two and a half times line to line (1200 volts), limit line to ground fault current to 5 amps, eliminate line to ground arc flash hazard, mitigate harmonic noise, control surge, and prevent resonance in the system. An HRG system would retain the advantages of the ungrounded system (fault tolerance, ability to run with one phase shorted) but would eliminate the negative aspects of the ungrounded system.

Certainly, an ungrounded wye electrical system is much more concerning than an ungrounded delta system, and TAI recommends that the exact grounding configuration of the system be determined, and if in fact the system is ungrounded wye, we would likely recommend that the system be converted to HRG.

5.20 BASIS OF DESIGN

The basis of design calculations prepared by others for the 12 MGD facility was reviewed to analyze intermediate flows and loadings throughout the treatment process. During this review several alterations were made to reflect more accurately the current and future conditions. Other deficiencies were identified (for example, oxygen requirements for aeration were under-estimated). Additional oxygen calculations were performed to determine current and future air requirements for the aeration blowers. This updated basis of design can be found in Appendix B.

5.21 TREATMENT FACILITY HYDRAULICS

The hydraulic calculations prepared by others (partial-plant computations) for the 12 MGD facility were also reviewed and checked for correctness and completeness. Complete hydraulics were computed during the preparation of this report. The calculations indicate that the existing treatment facility has no existing hydraulic areas of concern, it can pass the design peak flow of 30 MGD without concern, although there are limitations on the degree to which RAS pumping rates can be increased. These plant hydraulic calculations can be found in Appendix C.

6. RECOMMENDATIONS FOR IMPROVEMENTS

6.1 SIDESTREAM TREATMENT

Since incorporating an MLE process, the facility has continued to have issues with spikes in effluent ammonia-nitrogen concentrations. This is likely due to the additional nutrient loading contributed in the sidestream flow that is recycled to the biological process. Sidestream flow includes centrate from dewatering, gravity thickening supernatant, GBT filtrate, digester supernatant, and filter backwash. The sidestream flow can contain between 20 and 40% of the TN load to the biological process while only comprising 1% of the total flow. This significant contribution to the nutrient loading of the MLE process does not appear to have been considered in the MLE process design. It is recommended that the implementation of sidestream treatment for nutrient removal be investigated to relieve the ammonia loading to the MLE process.

Alternative 1 for sidestream treatment would entail mixing the centrate with a percentage of the RAS flow in a reaeration tank. Nitrifiers in the RAS convert ammonia to nitrite to nitrate, which is returned to the anoxic zone of the MLE process to promote denitrification of nitrate to nitrogen gas. This method of sidestream treatment is referred to as Centrate and RAS Reaeration Basins, or CaRRB. It is recommended that centrate be isolated from other recycle flow because the highest ammonia loadings come from dewatering, and diluting the sidestream will make it more difficult to remove this ammonia.

Alternative 2 would be a similar configuration except instead of RAS, the WAS would be diverted to be mixed with centrate. Effluent from this process would be drained to the GBTs. This would remove the need to provide additional pumping back to the MLE process since filtrate from the GBTs already drains to the main recycle line.

Both Alternatives 1 and 2 require the construction of a sidestream reaeration tank with adequate diffusers for oxygenation of ammonia. The repurposing of the old chlorine contact tank for sidestream treatment was investigated, but it was determined that the tank is not large enough to permit the installation of the required diffusers based on air requirements. These factors, along with the associated piping and pumping costs, make both alternatives undesirable.

Alternative 3 entails converting the abandoned chlorine contact tank to an equalization tank. Centrate from dewatering would be drained to this tank and stored throughout the day. The centrate would then be pumped to the recycle line at a reduced, controlled rate over a 24-hour period, thereby mitigating large spikes of ammonia to the biological process. This alternative is the least cost alternative because it does not require any additional tankage. Capital costs would include structural alterations, associated piping from the centrifuges, and pumping and piping from the equalization tank to the recycle line.

However, the total daily ammonia loading to the process will be unchanged under Alternative 3, as the centrate is not being treated. Therefore, though it may provide mitigation to issues with peak ammonia flows, the process will still require the same MLSS as described in the ammonia analysis in Section 5.8.

Emerging technologies such as the Anammox process are being developed that might provide a unique solution to nutrient removal and a future opportunity for the NWRWRF to be at the forefront of the application of these processes. However, these treatment methods are currently not fully established or proven to be cost effective, and have slow startup requiring intensive process control. Should the NWRWRF be interested in exploring these options further, additional analysis could be performed.

6.2 SLUDGE DEWATERING BUILDING / SLUDGE STORAGE BARN

Future design loadings for the facility are estimated to result in approximately 36 cubic yards of dewatered sludge generated per day for disposal. Given the Illinois Title 35 standard for sludge storage of 150 days, this amounts to a total dewatered sludge storage requirement of 23,000 cubic yards or 145,800 cubic feet. Assuming a sludge cake pile height of 6 feet, the total square footage required for dewatered sludge storage amounts to 24,300 square feet. The existing sludge storage barn provides 10,800 square feet. Therefore, an additional 13,500 square feet of dewatered sludge storage is necessary to plan for future solid loadings.

To account for this additional storage requirement, it is proposed that additional sludge storage be constructed in tandem with a new sludge dewatering building for centrifugal dewatering. It is recommended that these structures be located in close proximity to each other to allow for convenient conveyance of dewatered sludge into the storage barn.

Based on conversations with Village operating staff, the implementation of future storage requirements could consist of rebuilding the existing barn to ease access for sludge haulers, demolition of Building 66 and construction of an additional barn in its place, and the construction of the remaining storage requirement in the junkyard south of the tertiary filters. This would provide 150 days of storage for future dewatered sludge.

The proposed sludge dewatering building would include two dewatering centrifuges with room for installation of a third. Also included would be thickened sludge pumping and all conveyance equipment. The dewatering building would be located directly west of the existing sludge storage barn to allow for easy conveyance of dewatered sludge into storage. The first floor of the building could provide garage space to replace the shop in Building 66.

6.3 EXCESS FLOW TANKS

The Prairie Property located east of the treatment facility is currently being acquired by the Village to be used for the construction of three 6.0 MGD excess flow tanks. Influent flow beyond the DAF of 12 MGD could then be stored until capacity is freed up in the treatment process. Upon the implementation of the excess flow tanks, expansion of the plant would not be required until well beyond that time originally anticipated.

6.4 OUTFALL PIPE REHABILITATION

Repairs to the outfall structure along Pistakee Lake occurred in 2010 which included installation of a standpipe and rehabilitation of several hundred feet of sewer between the effluent box and



the outfall structure. Further rehabilitation or replacement of the outfall sewer beneath Pistakee Lake, from the structure to the discharge point at the mouth of the Fox River, is needed due to this again sewer which has been inspected and is failing.

6.5 SUMMARY

Operating staff have expressed a desire to implement sidestream treatment, excess flow storage, and the new sludge dewatering building. In addition to these larger projects, there are a number of items detailed in Section 5 as potential facility improvements. The projects to be considered at the NWRWRF over the next several years are summarized in Table 7.

Table 7: Summary of NWRWRF Improvement Projects

Project	Year
Headworks Rehabilitation	2015
Aeration Blower Improvements	2015
Phosphorus Removal Feasibility Study	2015-2016
UV Control Panel Replacement	2016
Sidestream Treatment	2017
Prairie Property Excess Flow Tanks	TBD
RAS Chlorination / RAS Pump Upsizing	2018
W3 Water Filtration	2018
Sludge Dewatering / Storage Barn	2018
Outfall Pipe Rehabilitation/Replacement	2020
Facility Expansion	2030

Discussions with Village operations staff determined that preliminary opinions of probable costs should be created for several projects including the aeration blower improvements, the phosphorus removal feasibility study, the RAS pump upsizing, and the outfall pipe rehabilitation. These can be found in the tables below.

Table 8: Blower Improvements

Fox Lake NWRWRF Aeration Blower Improvements	
Description	Probable Cost
Construction Sub-Total	\$ 528,200
General Conditions	\$ 66,025
Contingency @ 15%	\$ 79,230
Estimated Construction Cost	\$ 673,455
Engineering @ 15%	\$ 101,020
PROJECT TOTAL	\$774,475

Table 9: Phosphorus Removal Feasibility Study

Fox Lake NWRWRF Phosphorus Removal Feasibility Study	
Description	Probable Cost
Engineering and Lab Work	\$ 60,000

Table 10: RAS Pump Upsizing

Fox Lake NWRWRF RAS Pump Upsizing	
Description	Probable Cost
Construction Sub-Total	\$ 417,500
General Conditions	\$ 52,188
Contingency @ 15%	\$ 62,625
Estimated Construction Cost	\$ 532,313
Engineering @ 15%	\$ 79,847
PROJECT TOTAL	\$ 612,200

Table 11: Outfall Rehabilitation – 63-inch HDPE from Outlet Structure to Fox River

Description	2010 Probable Costs	2015 Probable Costs
Raw Construction Costs	\$ 14,157,403	\$ 15,326,822
General Conditions	\$ 1,769,675	\$ 1,915,853
Contingency @ 15%	\$ 2,123,610	\$ 2,299,023
Construction Cost	\$ 18,050,688	\$ 19,541,698
Engineering @ 15%	\$ 2,707,603	\$ 2,931,255
TOTAL CAPITAL COSTS	\$ 20,758,300	\$ 22,473,000