Wastewater Treatment Plant Planning Study

Prepared for

City of Mason

October 2018

2180168

Contents

Tables

Figures

Appendices

1 INTRODUCTION

The City of Mason requested a review of the operation of their existing Wastewater Treatment Plant (WWTP) and consideration of options for improving or replacing the existing plant. Primary concerns with the existing WWTP are the age and reliability of most components, the hydraulics of the existing plant when the receiving stream is at flood stage, the processing of significant peak flows, and the inability of the aeration system to provide sufficient oxygen during certain conditions. A specific regulatory concern is that the WWTP is party to a 2011 Administrative Consent Order (ACO) with the State of Michigan that requires taking action to reduce infiltration and inflow and to eliminate sanitary sewer overflows (SSOs) or WWTP bypasses during peak flow rates up to the 25 year/24-hour storm.

This report provides an analysis of the existing treatment plant and cost/benefit analyses of options for the future including: location of new treatment units, treatment methods /technologies, and potential changes in regulatory requirements.

2 HISTORY

In the 1940s, the City of Mason Wastewater Treatment Plant consisted of a bar rack and grit chamber, primary settling tanks, filtration, and digestion of solids. All treatment units were immediately north of the current WWTP on property now used by the Department of Public Works (DPW).

Major improvements were completed in the late 1950s with construction of a new entrance works, new primary clarifiers, new aeration tanks, a pump/blower building, new secondary clarifiers, and two new digesters. All of these treatment units were constructed immediately south of the 1940s treatment plant. In addition, new sludge beds were constructed north of the 1940s plant. These improvements provided full secondary treatment of the wastewater at an apparent design flow rate of approximately 1.2 to 1.5 million gallons per day (MGD). While improvements and upgrades to the plant have been made since, the primary clarifiers, aeration tanks, and digesters constructed in the 1957 project are still in use today.

As part of a 1976 project, the entrance works was again replaced and upgrades were completed in the primary clarifiers, secondary clarifiers, and aeration tanks. Two "Final Settling Tanks" were constructed to supplement the 1950s secondary clarifiers. These tanks, now called the Final Clarifiers, are still in use today. The 1950s secondary clarifiers are no longer utilized.

The 1976 project also added the building that houses the laboratory, tertiary filters, and filter pump room, constructed new sludge drying beds south of the aeration tanks, and added a new chlorine contact tank. The tertiary filter system is still available for use, but the WWTP operators have chosen to not use it due to problems with operation and their ability to meet discharge limits without tertiary filtration.

In the 1990s, a sludge storage tank was added to the north of the plant, west of the DPW building.

In 2008, a waste activated sludge (WAS) and return activated sludge (RAS) splitter box was constructed east of the existing aeration tanks to allow for controlled flow of RAS to the aeration tank using weirs and pumping of WAS back to the entrance works.

In 2016, the entrance works was replaced with a new headworks building containing a rotary drum screen and a vortex grit removal system. The new headworks was also equipped with raw wastewater pumps sized to deliver up to 6 MGD to the primary clarifiers at their current location with flexibility to provide greater lift to a new WWTP with a higher hydraulic grade line in the future.

In summary, many of the tank and building structures are approximately 60 years old. This includes the primary clarifiers, aeration tanks, blower building, and the digesters that are still in use. Much of the equipment in these structures was upgraded in the 1976 project. Therefore, most pumps and the aeration blowers are approximately 40 years old. The final clarifiers and the tertiary filter system are also approximately 40 years old. An aerial photo plan view of the existing treatment plant is provided in Figure 1.

3 REVIEW OF EXISTING SYSTEM

Most of the unit processes in the City of Mason wastewater treatment plant are operating beyond their typical useful lifespan. While plant employees have done a good job meeting NPDES discharge permit requirements, the current system creates significant challenges and under certain conditions does not provide adequate capacity to handle full treatment of the City’s wastewater.

Existing plant flow and water quality data referenced below was compiled from the plant's Monthly Operating Reports (MORs) from 2012 through 2017.

3.1 Influent Flow Rates

The average flow rate entering the City of Mason wastewater treatment plant from 2012 through 2017 was approximately 750 gallons per minute (gpm) or 1.08 million gallons per day (MGD). In the most recent three full years of data (2015-2017), the average influent flow was approximately 785 gpm (1.13 MGD). A 2016 Asset Management Plan prepared by Wolverine Engineers & Surveyors noted that the wastewater collection system served a population of 8,252 with 2,483 residential customers. The system primarily serves the City of Mason with a small number of customers in the neighboring Townships.

Dividing the average flow by an approximation of the currently served population of 8,400 results in an average flow per person of 134 gallons per day. This is significantly higher than the 100 gallons per person per day suggested by the Great Lakes-Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers 2014 "Recommended Standards for Wastewater Facilities" (10 States Standards). This suggests either a significant wastewater flow contribution from industry or a significant amount of infiltration and inflow.

It appears that infiltration and inflow is a significant portion of the average wastewater flow since the influent flow rate typically drops to around 560 gpm (0.8 MGD) during the drier part of the year. At 0.8 MGD, the average flow per person per day is approximately 95 gallons. This suggests that an average of at least 300,000 gallons per day of infiltration/inflow may be entering the City of Mason sanitary sewer.

The average flow rate of 760 gpm (1.1 MGD) with occasional peaks to 1,390 gpm (2 MGD) can typically be handled by the existing treatment equipment without causing any exceedances of permit requirements. However, during and after large rainstorms and/or snow melt events, the plant has had influent flow rates of 2,100 gpm (3 MGD) to over 4,200 gpm (6 MGD) that have lasted for 1 to 4 days with a slow decline over several days or weeks back to average flow rates. There appears to be a significant volume of infiltration and/or inflow entering the sanitary sewer system resulting from precipitation, melting of snow, and elevated groundwater levels.

Figure 2 shows daily flow rates entering the WWTP from 2012 through March 2018. As shown, nearly all flow rates are less than 3 MGD except for peak events in May 2014, April 2017 and February 2018. It is important to note that prior to construction of the Headworks in 2016 and the raw wastewater flow meter installed as part of that project, the influent flow measurements during peak flows were not always accurate. This is because peak flows would occasionally flood the previous influent flow meter, making accuracy during peak events suspect.

However, even without having accurate measurements of the precise peak flow, in general, a WWTP sized for an average flow of 1.5 MGD with max day flow rates of 3 MGD would be able to handle wastewater flow rates in the City of Mason, except during the few days per year when max day flows of 3 to 6 MGD occur.

While the new raw wastewater pumps installed in 2016 allow for moving these peak flows into the primary clarifiers, the downstream treatment system is not able to handle these flows. For example, in February 2018, the raw wastewater pumps delivered a peak hour of up to 7 MGD to the primary clarifiers, but the water level in the primary clarifiers rose to the elevation of a decorative concrete block trim, which failed to hold and allowed wastewater to spill on the ground.

3.2 Existing Treatment Units

3.2.1 Preliminary Treatment

Preliminary treatment is provided by a rotating drum screen with 3 mm (~1/8-inch) openings that removes, washes, and compacts screenings and by a vortex grit removal chamber and grit classifier. Grit and screenings are discharged to a shared waste hopper in a new headworks building constructed in 2016. The headworks building also houses submersible raw wastewater pumps that pump wastewater to the primary clarifiers. The pumps are located downstream of the screening and grit removal systems.

The existing headworks is operating well and provides for much better removal of screenings and grit than the prior systems, significantly reducing accumulation of floating solids and grit in the primary clarifiers and reducing wear on downstream treatment units. The pumps in the headworks have also been proven to handle flow rates in excess of 6 MGD that previously resulted in overflows to surface water.

The pumps in the headworks were sized to deliver the design flow to the current primary clarifiers as well as to future primary clarifiers located adjacent to the headworks with an elevation high enough to allow flow through a future plant by gravity. With some modifications/upgrades, the pumping system could also deliver flow to a new plant located on a separate property.

3.2.2 Primary Clarifiers

The current primary clarifiers were part of the 1950s construction project with upgrades completed in the 1970s. The two clarifiers are each 50 feet long by 16 feet wide and each has a total weir length of 64 feet. The side water depth is approximately 8 feet, which is less than the minimum side water depth of 10 feet recommended by the 10 States Standards. The clarifiers are equipped with mechanical collector flights that deliver sludge to a sump at one end of the clarifier where it is removed by sludge pumps and delivered to the anaerobic digesters. A scum removal system exists, but has not functioned for many years. Operators remove scum manually.

Ferric chloride is added at the head of the primary clarifiers to improve settling. Waste activated sludge from the final clarifiers is returned to the head of the primary clarifiers.

For primary clarifiers receiving waste activated sludge, the 10 States Standards recommend an overflow rate of 700 gpd/ft^2 at design average flow and 1,200 gpd/ft^2 at design peak hourly flow. At the current average day flow rate of 1.1 MGD, the surface overflow rate is 690 gpd/ft^2, which meets the recommended overflow rate for average flow. However, the maximum peak-hour flow recommended by the 10 States Standards for these clarifiers is 1.92 MGD. The average day flow of the maximum month from 2012-2018 was 1.93 MGD. The peak-hour flows often exceed the maximum recommended. The daily flow at the plant exceeds the recommended peak-hour flow approximately 7 days per year with peak daily flows of 2 to 6 MGD. Therefore, during peak flow times of the year, the recommended capacity of the existing primary clarifiers is significantly exceeded.

The 10-States Standards recommendation for maximum weir loading rate is 30,000 gpd/ft at peak hourly flow for plants with an average day flow greater than 1 MGD. With 128 feet of weir length combined, the two clarifiers provide for a peak hourly capacity of 3.8 MGD. As described above, the plant has had daily flows that exceed 3.8 MGD. Again, the clarifiers are undersized for the peak flows observed at the plant during wet weather peaks.

3.2.3 Aeration Tanks

The current aeration tanks were also constructed as part of the 1950s construction project with upgrades completed in the 1970s. The two aeration tanks are each 127 feet long by 18 feet wide with a maximum water depth of 13 feet. The relatively shallow side water depth limits the oxygen transfer efficiency.

Each aeration tank holds approximately 27,000 cubic feet of water. The BOD loadings of approximately 800 to 1,300 lbs/day in the current typical primary effluent result in organic loadings of 15 to 26 pounds per 1,000 cubic feet of aeration tank volume. This means that the 10-States Standards recommended loading of 15 pounds per 10,000 cubic feet for single stage nitrification systems is routinely exceeded.

Centrifugal blowers housed in the existing blower room provide air to the aeration tanks via diffusers. Aeration equipment should be capable of maintaining a minimum of 2.0 mg/L of dissolved oxygen at all times. The existing system often meets this requirement, but during high temperature times in the summer, the blowers are utilized to their maximum capacity and cannot maintain 2 mg/L of oxygen. The existing blowers can be used in various combinations, but the maximum air supplied during high summer temperatures is approximately 1,000 cubic feet per minute. This air capacity routinely falls short of the 10-States Standards recommended 1,500 cubic feet of air per pound of BOD whenever the BOD in the primary clarifier effluent exceeds 960 pounds per day, which currently occurs for months at a time.

3.2.4 Secondary Clarifiers

The existing secondary clarifiers are two 45-foot diameter clarifiers with a side water depth of approximately 12 feet, which meets the 10-States Standards recommended minimum. The 10-States Standards recommended maximum overflow rate is 1,000 gpd/ft^2 at design peak hourly flow rate and the recommended maximum peak solids loading rate is 35 lb/day/ft^2. Based on the overflow rate of 1,000 gpd/ft^2 and a total clarifier area of 3,180 ft^2, the peak-hour flow rate that can be handled by the existing clarifiers is approximately 3.2 MGD. Peak daily flow rates of 4 to 6 MGD are observed for 1 to 3 days during peak wet weather events with peak hour rates even higher. The existing clarifiers are not sized to handle the peak hour flows.

Based on the recommended peak solids loading of 35 lbs/day/ft^2, the existing clarifiers are sized for a peak solids loading of 111,300 pounds/day. This value is not typically exceeded at the Mason WWTP, even at peak flow rates.

The maximum weir loading should be 30,000 gpd/ft. Based on both 45-foot diameter clarifiers in operation, the maximum peak hour flow based on weir loading is 8.5 MGD, which is sufficient for existing flows.

3.2.5 Solids Handling

Solids are removed from the secondary clarifiers by air lift pumps and delivered to a RAS/WAS splitter box. RAS is then delivered to the aeration tanks by gravity and WAS is pumped to the primary clarifier influent channel. Solids removed in the primary clarifiers are pumped to the existing digesters that were constructed in the 1950s. The digesters have been routinely cleaned and upgrades have been completed in the last few years. The digestion system is operating adequately. After digestion, sludge is stored in existing sludge storage tanks and then hauled away for land application.

While the solids handling system functions adequately, the safety equipment is outdated and the structures are beyond their typical useful life. While not directly addressed in this report, it is recommended that the City consider replacing this system as well. It is recommended that further study be completed on this system including a comparison of sludge storage with land application to sludge thickening/drying and disposal in a landfill. The direction for a future solids handling system should be determined before final design of other plant improvements as the type of solids handling system selected could impact design requirements of the other plant components.

3.2.6 Disinfection

The plant utilizes chlorine for disinfection with a chlorine contact tank adjacent to the tertiary filters. This system has been functioning well, but chlorine gas requires special handling and safety procedures. The City would prefer to change to ultraviolet light disinfection.

3.3 Floodplain/Floodway

The Mason WWTP is located on the east side of Sycamore Creek, which is the receiving stream for the plant effluent and flows from south to north. Rayner Creek wraps around the east and north side of the WWTP before joining Sycamore Creek just northwest of the existing sludge storage tank. As shown in Figure 3, the floodplain for these two streams covers most of the WWTP property and the floodway crosses a large portion of the property. The floodplain/floodway elevation varies from approximately 875.5 at the Rayner Creek/Sycamore Creek confluence and is approximately 876.1 at the south end of the treatment plant property.

While floodwater on the ground around the treatment units creates operational challenges, the main problem experienced during flood events is that the water level in Sycamore Creek rises and backs up through the effluent pipe such that treated wastewater will not flow out of the plant by gravity. The effluent pipe must be shut off and temporary pumps used to pump the treated wastewater out of the plant to Sycamore Creek. This situation has contributed to multiple discharges of raw or partially treated wastewater to surface water. While the City has developed a standard procedure to block off the effluent line and use portable pumps to allow discharge, this operation is completely manual resulting in a potential lag between peak flows and implementation of this system as well as susceptibility to problems with the portable pumps.

3.4 Summary

Many of the treatment processes at the Mason WWTP are 40 to 60 years old and have exceeded their typical lifespan. During average daily flow rates, the plant operators are able to use the existing system to treat the water to an excellent quality and meet all discharge permit requirements. However, significant peak flow rates encountered a few times per year exceed the capacity of the treatment units and have resulted in bypass flows to surface water. There are also times during the summer when adequate dissolved oxygen cannot be maintained in the aeration tanks due to limited blower capacity. In addition, the plant was constructed with a hydraulic grade line that is not high enough to allow discharge to the receiving stream when the stream is at or near flood stage.

Graphs of effluent data from 2012 through March 2018 including BOD, total phosphorus, total suspended solids, and ammonia, are included in Appendix A. These graphs confirm that the WWTP typically meets its monthly average Permit limits. However, the graphs also show that the treatment system is not always producing a stable effluent quality. In particular, the graph of ammonia in the effluent shows that while the monthly average typically meets the permit limit, the monthly maximum is often five to ten times greater than the average. This suggests periods on the order of a day or two when the biology in the aeration tanks is not performing as it should.

4 DESIGN PARAMETERS

The following are design parameters that need to be considered for any improvement or replacement of treatment process for the WWTP.

4.1 Wastewater Flows

As noted above, the current average day flow to the WWTP is approximately 1.1 MGD with "normal" max day flows of up to 2 MGD and occasional wet weather peak daily flows of 3 to 6 MGD. Using the 2012 through March 2018 data, the following flow frequency percentiles were observed:

Percentile	Flow (MGD)
10	0.81
25	0.87
50	1.01
75	1.22
90	1.46
99	2.19

The 10th percentile provides an idea of the low end of flow entering the City of Mason WWTP when infiltration and inflow is at a minimum. The 10th percentile flow rate of 810,000 gpd divided by the approximate population of 8,400 people results in an average flow of 96 gallons per capita per day (gpcd), which nearly matches the design value of 100 gpcd recommended in the 10 States Standards. The wastewater flow contributed to the collection system per person is actually likely lower as some of the existing flow is from commercial/industrial customers and there is likely flow from groundwater infiltration and basement sump pumps throughout the year.

The 99th percentile flow of 2.19 MGD shows that while wet weather peak flows of up to 6 MGD are observed, they only occur 1% of the time, or 3 to 4 days per year.

The 10 States Standards suggests the ratio of peak hour flow to average day flow should be approximately 3 to 1 for a population the size of Mason. Therefore, the 3 to 6 MGD occasionally observed over a multiple day period versus the average day of approximately 1.1 MGD suggests a large amount of infiltration and inflow to the Mason system during rain or melt events.

Per terms of the Administrative Consent Order between the MDEQ and the City, the City is required to monitor flow through the plant and investigate ways to reduce infiltration and inflow. In addition, the ACO requires the WWTP to make improvements to be able to provide secondary treatment without bypass of all wastewater flow during events up to the 25-year, 24-hour storm, which has been determined by the MDEQ to be a 24 –hour storm event of 3.9 inches. Based on WWTP staff reports on past events, this is assumed to be a daily flow rate of up to 6 MGD.

Due to the uncertainty of whether a reduction in wet weather flow will be accomplished prior to beginning design of the WWTP, the following review of options is based on an average day flow of the maximum month of 2.2 MGD with consideration given to additional costs and space required for a plant that could handle a 6 MGD peak flow. Note that the 2.2 MGD flow rate encompasses 99% of all current daily flows.

The 10-States Standards recommend consideration of equalization whenever the peak hour flow is greater than 3 times the average daily flow. Since peak day flows at the City of Mason are observed to exceed 5 times the average day flow, equalization is reviewed in this report. Depending on the length of time peak flows are observed and the available volume for equalization, the peaks can be dampened by storing water during peak flow events. The stored wastewater is then released back to the plant when influent flows return to normal.

4.2 Wastewater Influent Quality

The City of Mason has commercial and industrial wastewater customers, but most of the wastewater collected in the City is residential. No significant industrial users have been identified and the WWTP does not receive high-strength wastewater or wastewater with particular constituents of concern relative to the WWTP discharge permit.

The following provides average influent flows and select average wastewater quality parameters as recorded in the MORs. The 5-day biochemical oxygen demand (BOD) values near 200 mg/L are typical of residential wastewater. The total suspended solids (TSS), phosphorus, and ammonia concentrations are also typical of an average residential wastewater.

City of Mason WWTP Influent Flow and Quality

	Flow (MGD)	pH	BOD, mg/L	TSS, mg/L	Phosphorus, mg/L	Ammonia, mg/L
2015 Average	0.98	7.81	210	178	4.8	25
2016 Average	1.23	7.79	166	130	4.1	21
2017 Average	1.19	7.58	199	182	4.7	20
2015-17 Average	1.13	7.73	192	163	4.5	22

The MORs were also used to tabulate values for flow and quality for "Summer Average Day", "Average Day of the Max Month", and "Peak Day". More detail including current values, projected values in year 2038, and preliminary design parameters are tabulated in Table 1.

Graphs of influent data from 2012 through March 2018 including flow, BOD, total phosphorus, total suspended solids, and ammonia, are included in Appendix A. In general, the wastewater quality entering the WWTP is typical of residential strength wastewater.

4.3 Wastewater Effluent Quality

The City of Mason WWTP recently received a renewal of the NPDES permit for discharge to Sycamore Creek. The current NPDES permit includes seasonal limits for 5-day carbonaceous biochemical oxygen demand (BOD), total suspended solids (TSS), and ammonia. The limits are provided in Appendix B. The lowest limits required for BOD, TSS, and ammonia are from May 1 through October 1 as follows:

	Daily (mg/L)	Monthly Average (mg/L)	Monthly Average (pounds/day)
BOD	10	4	50
TSS	NA	20	250
Ammonia	2	0.5	6

As shown in Appendix B, the monthly average limits for phosphorus throughout the year are 1 mg/L and 13 pounds per day. There are additional limits for selected metals, fecal coliform bacteria, and total residual chlorine. All of the limits in the NDPES permit will need to be met by any new treatment system. In addition, consideration needs to be given to possible changes to the permit by the Michigan Department of Environmental Quality (MDEQ) in the future such as addition of new parameters or lowering of any of the existing limits.

5 WWTP TREATMENT UNIT OPTIONS

The City of Mason has requested a review of options to correct the deficiencies observed with the current WWTP and to allow for treatment meeting the NDPES discharge permit limits without bypassing raw or partially treated wastewater during peak flow events. In addition, the City would like the plant to be improved or replaced such that wastewater is pumped once from the Headworks, and then flows by gravity to the receiving stream, even at flood stage. The City has asked for a review of options to construct new plant improvements on the existing WWTP property and/or to complete construction on another property.

As part of the review process, the City also requested that treatment systems be evaluated for their ability to handle higher loadings in the future. The City would like to understand which treatment systems would allow flexibility for modification to handle future industries that move into the City and have higher than average strength wastewater.

The following reviews options for plant improvements or replacement on the current property or a different property. A table comparing the following options and itemized cost estimates for feasible options is included in Appendix C. This summary includes the estimated construction cost as well as a 20-year Present Value of the construction cost with 20 years of aeration equipment operation.

5.1 Reduce Wastewater Flows

It appears that infiltration and inflow contribute 3 to 4MGD during peak flow events at the WWTP. A wastewater plant designed to handle peak flows of up to 7 MGD could cost millions of dollars more than a plant designed to handle current average flows of 1.1 MGD and projected max day flows of 4 MGD for the next 20 years. Therefore, it is important to reduce infiltration and inflow, including basement sump pump discharges, as much as possible. While it may not be cost effective to disconnect all basement sump connections immediately, it is recommended that this be done whenever possible in conjunction with other street or utility construction projects. Any water that does not need to be treated should be removed from the system if at all possible.

The City has already completed investigative activities including smoke testing of portions of the system to look for infiltration and inflow. While large localized flows have not yet been found, a system wide investigation consisting of a combination of video, smoke testing, and flow metering to determine what portion(s) of the system may be contributing to the observed peaking is recommended. It would also be advisable to physically visit all sanitary manholes located in low areas near streams or wetlands to look for any possible direct inflows. Ideally, this would be completed prior to design of wastewater treatment plant improvements so that the sizing and cost of new improvements could be reduced.

5.2 Equalization

Providing equalization volume at the head of the treatment plant can reduce the required size of downstream treatment units by holding a portion of the peak flows and releasing that volume back through the plant at a slower rate over time.

If the decision is made to construct new treatment units adjacent to the existing plant, the existing primary clarifiers, aeration tanks, and abandoned in place secondary clarifiers could be used for equalization storage.

The storage volume available in the existing concrete structures assuming two feet of freeboard is as follows:

1957 Primary Clarifiers – 93,000 gallons

1957 Aeration Tanks – 463,000 gallons

1957 Secondary Clarifiers (not currently in use) – 130,000 gallons

The total volume available is approximately 690,000 gallons. While it is apparent that peak flows would still be significantly greater than average day flows even with this equalization volume, it would allow for some smoothing of the highest peaks as well as smoothing the difference between typical day and night fluctuations. To have the greatest impact during the largest peak flow events, it is recommended that flow only be diverted to the equalization when flows exceed 5 MGD to maximize the storage available during peak infiltration/inflow events.

It is estimated to cost approximately $350,000 to upgrade the existing tanks, add inlets and outlets, add equalization pumps, add aeration to prevent odors, and construct yard piping to return water from the equalization to the Headworks.

5.3 Upgrade Existing Treatment Processes

Consideration could be given to upgrading internals of the existing clarifiers and aeration tanks, replacing pumps and blowers, and repairing structures as necessary. However, increasing the hydraulic capacity of the treatment units would require adding on to the footprint of each of them. In addition, it would be necessary to extend all of the concrete structure walls upward and operate them all at a higher hydraulic grade in order to meet the goal of flowing through the plant by gravity. This would require adding structural load to walls that were not designed for that purpose. It would also result in structures that were partially new and partially more than 60 years old. Therefore, it was determined that reusing and/or expanding the existing systems to meet current flow and treatment requirements is not a feasible alternative.

5.4 Conventional Activated Sludge Process

As evidenced by the successful operation of the existing wastewater treatment system during most conditions, a conventional secondary activated sludge process can provide treatment meeting the current NPDES discharge permit requirements. A new conventional activated sludge process that is adequately designed for peak hydraulic loadings could provide the necessary treatment capacity for the City. A preliminary concept of the footprint for this type of system is provided in Figure 4.

A system consisting of circular primary clarifiers, aeration tanks and circular secondary clarifiers is recommended. The recent discharge permit renewal does not require operation of tertiary treatment. Since the City is meeting permit requirements now without tertiary treatment, it is recommended that space and hydraulic allowance be made for tertiary treatment, but that it not be constructed at this time.

Recommended improvements compared to the existing system include: deeper depth and fine bubble diffusers to improve oxygen transfer, the ability to create anoxic zones within the aeration tanks to promote biological nutrient removal, more efficient blowers, improved sludge handling pumps, an automatic control system that allows for RAS and WAS to be set at a flow rate or percent of plant flow, and an automatic aeration control system that varies the speed of blowers to meet a target dissolved oxygen concentration.

The preliminary concept for this system consists four parallel trains each consisting of one anoxic zone and one aerobic zone. The aerobic zone would provide for removal of BOD and ammonia. The anoxic zones would provide for conversion of nitrate to nitrogen gas and utilization of nitrate to oxidize BOD. This intentional use of nitrates for BOD removal makes the system more efficient by reducing the dissolved oxygen demand.

While the system would be sized for a future average day of the maximum month of 2.2 MGD, it would be capable of peak hydraulic rates of up to 7 MGD. During peak flows, solids would be retained by turning off the air in one or more of the tanks.

Total footprint of the basins with would be approximately 150 by 130 feet with an 18-foot side water depth. For reference, the existing sludge drying beds in an area that could be used for aeration tanks are approximately 110 feet east to west and 130 feet north to south.

Specific advantages of a conventional activated sludge process with design improvements as described above for the City of Mason include:

- Relatively simple construction
- Operators already familiar with process

- Added automation provides appropriate amount of air to efficiently maintain target dissolved oxygen
- Use of nitrates produced during ammonia removal as oxygen source to reduce aeration requirement and restore some alkalinity
- Capability to be upgraded to provide attached growth treatment for higher strength loadings (discussed further in Moving Bed Bioreactor section below)
- Provides flexibility to handle higher loadings with future internal upgrades

Disadvantages include:

- Relatively large footprint will require relocating portions of the existing plant including the existing effluent line during construction.

5.5 Oxidation Ditch Process

The oxidation ditch process is a suspended growth process using multiple concentric wastewater channels operating in series. Each channel would have sets of mechanical aerators to provide oxygen as the wastewater passes by. No aeration blowers would be required. The speed of the mechanical aerators would be automatically varied to efficiently provide the target dissolved oxygen concentration. A preliminary concept showing a footprint for this type of system is provided in Figure 5.

The process is flexible and handles various flows well. It allows for sequential aerobic and anoxic zones, which promotes conversion of ammonia to nitrate via nitrification and conversion of nitrate to nitrogen gas via denitrification. This nitrification-denitrification process is the same concept employed in the aeration tank design described above. It allows for use of the nitrate produced during ammonia removal as a source of oxygen, which allows for a lesser overall demand for aeration.

The footprint for a concentric oxidation ditch for average day flows of 1.7 MGD would be approximately 125 feet by 150 feet with a channel depth of approximately 14 feet. For reference this size would be a little larger than the existing sludge drying beds (110 feet by 130 feet). Another design with two side by side single loop oxidation ditches would have an approximate total footprint of 190 feet by 110 feet. This configuration would not fit in available space on the current WWTP property.

A concentric oxidation ditch reviewed is designed to handle up to 5 times the average flow without solids washout by redirecting the influent flow to skip the outer channels until the peak flow passes.

As is the case with each of the options reviewed, the oxidation ditch process could be operated with or without primary clarifiers. The disadvantage of operating without primary clarifiers is that BOD loading that could be removed and utilized to produce energy in the anaerobic digesters will instead need to be treated in the oxidation ditch, resulting in an increase in overall operational cost. There may not be room for primary clarifiers on the existing WWTP property with an oxidation ditch.

An operational challenge with an oxidation ditch would be greater difficulty in removing a portion of the system for maintenance repair. While the four-train conventional aeration tanks would allow for full treatment with any one of the tanks out of service, the concentric oxidation ditch would require short duration removal of a ring channel from service scheduled during low flow periods.

Specific advantages of the oxidation ditch option for the City of Mason include:

- No blowers or associated building space and controls required
- Added automation provides appropriate amount of aeration to efficiently maintain target dissolved oxygen
- Use of nitrates produced during ammonia removal as oxygen source to reduce aeration requirement and restore some alkalinity

Disadvantages include:

- No room on existing plant property for primary clarifiers, requiring greater load be handled by the oxidation ditch
- Relatively large footprint will require relocating portions of the existing plant during construction
- Use of primary clarifiers with an oxidation ditch may require additional property
- Less flexible than conventional aeration tanks if it was desired to add an attached growth system in existing tankage to handle higher strength wastewater.
- Less flexibility to remove a portion of the treatment system from service for maintenance than a multi-tank conventional aeration tank system. This may require two oxidation ditches be constructed to provide equivalent reliability.

5.6 Continuous Flow Sequencing Batch Reactor

Sequencing batch reactors are a method of filling, treating, and discharging treated water on a batch basis. It is possible to operate a sequencing batch treatment method for municipal wastewater that allows for screened and degritted water to continuously enter the batch reactor during the reaction, settling, and decant phases. This method is promoted as having the ability to treat municipal wastewater without primary or secondary clarifiers, saving considerable space and eliminating two treatment processes. A preliminary idea of the footprint for this type of system based is provided in Figure 6.

Each continuous flow SBR consists of an aerated pre-reaction zone that continuously receives screened influent. Openings in the bottom of a wall of the pre-reaction zone allow flow to continuously pass on to the main reaction zone. While in the fill phase, water is aerated to provide mixing and oxygen for BOD removal and nitrification. The aeration is then turned off and solids settle to the bottom of the main react zone. Decant of treated effluent then proceeds from the top of the tank. Wastewater enters from the influent end throughout the settling and decant phases. Waste solids are removed from the SBR, but no return sludge pumps or piping are required.

Specific advantages of continuous flow sequencing batch reactors for the City of Mason include:

- Overall plant footprint smaller allowing for an easier fit to existing space
- Operation and maintenance of primary clarifiers, secondary clarifiers and return activated sludge pumps not required.
- Automation provides appropriate amount of aeration to efficiently maintain target dissolved oxygen
- Use of nitrates produced during ammonia removal as oxygen source to reduce aeration requirement and restore some alkalinity

Disadvantages include:

- Lack of proven secondary clarification process to assure solids removal
- Operation without primary clarifiers results in significantly higher loading to process
- Discharge from the SBR occurs in large pulses requiring oversizing the disinfection step or significant post SBR equalization. Equalization would require additional cost for tankage and pumps and would also require a larger footprint.
- The large discharge rate would also have a significant impact on any future tertiary treatment system
- If controls malfunction, water does not proceed through process by gravity.
- Mechanically complex with higher repair costs

Because of the lower reliability due to reliance on the control system to handle intermittent discharge and mechanical complexity, this option will not be considered.

5.7 Membrane Bioreactor Process

The membrane bioreactor process provides for excellent treatment of wastewater. The barrier of the membranes allows for operation at higher mixed liquor suspended solids (MLSS) concentration than other processes. The higher MLSS concentration allows for treatment of a given loading in a smaller footprint. This system also provides the solids/liquid separation process, eliminating the need for secondary clarifiers. The membranes produce a better effluent quality than conventional secondary treatment providing an ability to meet extremely low discharge permit limits. When the MDEQ requires very low discharge permit limits based on required protection of the receiving water, membrane bioreactors can be an excellent solution.

However, the membrane bioreactor process does not handle excessive peak flows well without significant oversizing of the system. This is because the flux through the surface area of the membranes cannot vary enough to handle periodic peak flows. Since the Mason system peaks at over 5 times the average day flow, a membrane bioreactor system would need to be oversized substantially.

If equalization could be provided to reduce the max day flow rate to approximately 4 MGD, a membrane bioreactor system sized to fit the existing site could be utilized. One configuration consists of two parallel trains each with an anoxic zone and an aerobic zone followed by treatment by the membranes in separate enclosed tankage for greater ease of maintenance. The dimensions of the two train system would be approximately 135 feet x 70 feet plus a blower/membrane/UV building. This would fit in the available space of the existing sludge drying bed more easily than some of the other options and would not require relocating any existing yard piping. However, the cost for construction would still be substantially higher than the other options due to the equipment costs. A preliminary idea of the footprint for this type of system is provided in Figure 7.

The total estimated cost for this option is greater than $15.5M without including the additional equalization required to reduce the peak flows to 4 MGD. The system also costs more to operate than the other options. Another disadvantage is that if the WWTP experiences loss of power, the system will not operate and water will not move through the plant successfully by gravity.

Unless the MDEQ revises the City's permit limits down substantially or other options, this option would not be financially feasible based on the higher than average capital and operation costs of a membrane bioreactor system.

5.8 Moving Bed Bioreactor/IFAS Process

The moving bed bioreactor process is an advanced treatment process. It is similar to conventional aeration basins, but includes a media inside the aeration basin or in a separate reactor that allows for establishment of attached growth microorganisms. Attached growth microorganisms provide for treatment of higher wastewater loading in a smaller volume than suspended growth organisms by retaining microorganisms and increasing total biomass. This allows for more efficient conversion of ammonia to nitrate, particularly during colder weather. This system would provide greater assurance that the plant would meet its seasonal ammonia permit limits. It would also reduce the footprint of the aeration tanks.

The conceptual layout of this system would provide two treatment trains, each with an anaerobic zone, an aerobic reactor, and an aerobic Integrated Fixed Film Activated Sludge (IFAS) reactor containing polyethylene carrier elements that provide surface area for attached growth microorganisms. The total footprint of this system would be approximately 80 feet by x 70 feet, which is significantly smaller compared to the other options. This would allow for easier construction of the treatment unit without conflicts with existing operation. Primary clarifiers are recommended to be utilized with this system to reduce total loading and to reduce fouling of the media. A preliminary idea of the footprint for this type of system is provided in Figure 8.

Specific advantages of a moving bed bioreactor system for the City of Mason include:

- Less concrete required than other options
- Smallest footprint, allowing for treatment of greater volume and/or loadings than other options in the space available
- Added automation provides appropriate amount of aeration to efficiently maintain target dissolved oxygen
- Use of nitrates produced during ammonia removal as oxygen source to reduce aeration requirement and restore some alkalinity

Disadvantages include:

- More complex with greater maintenance/replacement costs than conventional activated sludge aeration tanks

5.9 Clarifiers

Some of the options above require primary and/or secondary clarifiers as part of the complete process. The following reviews preliminary sizing and costs of clarifiers.

For primary clarifiers receiving waste activated sludge, the 10 States Standards recommend an overflow rate of 700 gpd/ft^2 at design average flow and 1,200 gpd/ft^2 at design peak hourly flow. Assuming a design average flow of 1.7 MGD and a design peak hourly flow of 7 MGD, a clarifier area of 2,430 square feet is required for average flow and an area of 5,830 square feet is required for the peak hourly flow. Two 60-foot diameter clarifiers could provide adequate surface area for the peak hour flow, while one 60-foot diameter clarifier could handle the average day flow. Energy baffles will be considered during design to potentially reduce the overall size of the clarifiers at the design flow.

For secondary clarifiers, the 10 States Standards recommend a design peak hourly overflow rate of 900 gpd/ft^2 when chemical addition is required to reduce phosphorus to less than 1 mg/L. While the current permit limit and draft permit renewal limit is 1 mg/L, this could be reduced in the future. Two 60-foot diameter clarifiers would provide the necessary surface area for a peak flow of 5.1 MGD. This would be sufficient if equalization could smooth peaks to this rate. Two 70-foot clarifiers would be necessary to handle flows of 7 MGD. Energy baffles will be considered during design to potentially reduce the overall size of the clarifiers at the design flow.

Estimated costs for primary and secondary clarifiers are included in the cost estimates for the overall treatment options in Appendix B.

5.10 Tertiary Treatment

The City of Mason notified the MDEQ of their intent to eliminate tertiary treatment during their recent discharge permit renewal process. The renewed permit does not include any requirement to utilize tertiary treatment and maintains existing discharge permit limits for nutrients, BOD, and TSS.

If future changes to the permit or changes to the wastewater quality indicate a need, tertiary treatment could be provided with sand filtration, disc filters, or other technologies. Since it is not required currently, it is recommended that space and hydraulics of any new design allow for addition of tertiary treatment in the future. The following reviews options that could be considered when and if tertiary treatment is added to the process.

The existing sand filters have not performed well and have caused many operational problems. They have been taken out of service entirely. Based on Mason's experience in the past with sand filtration, this option will not be pursued for the new WWTP. A good alternative tertiary treatment method is the use of cloth or stainless steel disc filters.

Woven stainless steel mesh filters are available with filtration rates of up to 16 gpm per square foot at a head loss of about 1.5 feet. An automatic spray wash system is utilized for cleaning with return water sent back to the headworks. The projected amount of backwash water is approximately 1.6% of the effluent flow rate. This system would consist of an active and a standby unit sized for an average daily flow of 1.7 MGD and peak daily flow of 3.5 MGD each. Flows greater than 3.5 MGD would utilize the standby unit or bypass the filters. The size of each unit would be approximately 11 feet by 7 feet by 7.5 feet tall.

The cost to provide stainless steel disc filters as tertiary treatment including piping, an addition to the Blower building or a separate building, and piping of backwash waste back to the headworks is estimated at approximately $990,000.

Disk filters with a cloth filter media are also available. They are designed with a filtration rate of 3 to 6.5 gpm per square foot. A vacuum backwash system is used to clean the filters. Due to the larger size necessary, it is estimated that the cost for this option will exceed the cost of the stainless steel filters.

Since the existing plant meets the discharge limits without tertiary treatment and tertiary treatment is not required by the permit renewal, it is recommended that tertiary treatment equipment not be installed initially. Space and hydraulic grade can be designed into the proposed treatment plant to allow for easily adding tertiary treatment in the future, if necessary. A preliminary hydraulic profile of a future wastewater treatment plant including provision for future tertiary filtration is provided in Figure 9.

6 WWTP TREATMENT LOCATION OPTIONS

6.1 Existing WWTP Property

As noted above, the existing plant is located in a floodplain/floodway area between Sycamore Creek and Rayner Creek, just upstream of where Rayner Creek flows into Sycamore Creek. Figure 3 shows the FEMA floodplain/floodway map. The location of the plant coupled with the existing hydraulic gradient through the plant has resulted in significant challenges in moving water through the plant when the level of the receiving stream has risen above the level of the plant outfall.

In addition, the location in the floodway/floodplain creates challenges for permitting new construction in a large portion of the property. In general, a permit can be obtained in the floodplain when a compensating cut below the floodplain is made to balance fill placed within the floodplain. Permitting in the floodway is more difficult as it often becomes necessary to complete computer simulations of the flood and to show that the water level is not impacted upstream due to any changes made. If construction is permitted in the floodway, it may require reconstruction of the stream bank and/or removal of existing obstructions to compensate.

During construction of the Headworks in 2015/2016, it was determined that the extent of the floodway as defined in the FEMA maps was not accurate compared to actual ground contours determined in the site survey. As shown in Figure 10, the entire area of the sludge drying beds and additional area to the west, south and east has ground elevations above the floodplain/floodway. In addition, the ground around the existing lab/admin/tertiary treatment building and area south of the 2016 Headworks building is also higher than the floodplain/floodway. Finally, the models used by the MDEQ in this area already show the existing WWTP structures (clarifiers, aeration tanks, digesters) fully blocking the floodway on the WWTP property.

This information was submitted to the MDEQ during design of the 2016 Headworks, and the MDEQ agreed and issued a permit for the Headworks building. This information can also be used to locate additional WWTP improvements in areas outside the floodway. In general, most of the area west of the aeration tanks from Jefferson Street to Sycamore Creek is at an elevation above the 100-year floodplain and out of the floodway. The area of the existing secondary clarifiers and the Headworks are in an area that is within the floodplain but is blocked from the floodway by surrounding ground that is higher than the flood elevation.

In addition, a permit may be available if new construction does not create more of an obstruction to the floodway than existing structures that are demolished. This will take analysis with a computer simulation, but may allow for additional flexibility in where new improvements are placed. Since the DPW may be relocating, it may be possible to gain credit for obstruction removal if any of the existing building(s) can be demolished in conjunction with construction of new WWTP facilities.

6.2 Adjacent City Department of Public Works Property

The DPW property immediately to the north of the WWTP and additional City properties further north have been considered as a potential location for WWTP improvement construction. However, as shown in Figure 3, most of that property is also in the floodway. Since there are fewer existing obstructions in this area, it may be more difficult to permit new construction here.

6.3 Other Property

Other property not currently owned by the City could also be considered for a new wastewater treatment plant. There is a significant amount of farmland 1,000 feet north and farther from the WWTP that is well outside the floodway and floodplain. However, this property is not currently considered to be accessible to the City.

Another potential property that the City may wish to consider is the residential property immediately south of the WWTP. The parcel is 5 acres and a significant portion of it is not in the floodplain/floodway. Acquisition of this parcel would provide for room to construct all of the above options outside the floodway without requiring relocation of existing facilities and without impacting existing treatment processes during construction.

The major advantage of construction on a new property would be the lack of space restrictions and the ability to construct the entire plant without any conflicts with current operation. The major disadvantages would be the need to acquire property, and the cost to install a forcemain from the existing Headworks.

If the property is not adjacent to the existing WWTP, there will be an added requirement to travel to visit the Headworks routinely. In addition, while the digesters at the existing WWTP could be reused with a new plant built on the existing property or the DPW property, new digesters and associated pumps and heat exchangers would be required at the new property if not adjacent to the existing property.

Total cost for utilizing other property would be dependent on the treatment method selected, the property acquisition cost and the distance from the existing Headworks.

7 CONCLUSIONS/RECOMMENDATIONS

This report reviewed the existing performance and sizing of the existing treatment plant. While the existing plant has performed adequately, it is undersized for peak flows and loading. The aeration system cannot provide sufficient oxygen in certain situations, sanitary sewer overflows and/or bypasses occur during peak flow events, and most treatment processes are well beyond their typical useful life span.

In order for the City of Mason to efficiently treat its wastewater to meet all requirements of the MDEQ discharge permit, and meet the terms of the 2011 ACO, it is recommended that a new water treatment portion of the wastewater treatment plant be constructed. It is recommended that inflow and infiltration sources be located and removed from the system so that the wastewater plant is not oversized just to handle infrequent large peak flows.

After considering the space available, construction cost, operation and maintenance costs, treatment effectiveness, and ability to upgrade treatment systems to handle higher strength industries in the future, we recommend the City consider the following ranking of treatment methods starting with the most favorable option. More information on each option is provided in the summary table in Appendix C.

1. Conventional Activated Sludge - Provides cost-effective treatment in a similar manner to the existing plant with better controls for easier operation. Also provides excellent reliability and can be converted to an attached growth system in the event that industry with higher strength wastewater moves into the City.

2. Moving Bed Bioreactor/IFAS – Provides ability to handle larger flows/loadings in smaller footprint than Conventional Activated Sludge and therefore could allow for future higher strength wastewater industries. It requires greater capital expense and significantly greater routine operational costs and maintenance costs.

3. Oxidation Ditch – Does not provide the same reliability as the above options as it is not as easy to take one portion of the system out of service for maintenance or repairs. The large total footprint would make it difficult to find space for primary clarification, creating more loading on the oxidation ditch.

4. Sequencing Batch Reactor – Not recommended as wastewater will not flow through the system by gravity in the event of a loss of power and/or problems with the control system. This system also discharges flow at a rate higher than the influent flow creating a need for post treatment equalization with pumping or significantly oversizing the disinfection step and any future tertiary treatment.

5. Membrane Bioreactor – Not recommended as capital and operation and maintenance costs are significantly higher than other options especially as the system would not handle the peak flows observed without significantly oversizing the membrane units.

In conclusion, we recommend the City consider construction of circular primary clarifiers, conventional activated sludge aeration tanks optimized for nutrient removal, circular secondary clarifiers, and UV disinfection. This system provides reliability as it allows for full design flows that will meet the requirements of the 2011 ACO with one tank out of service and it provides flexibility to handle higher strength waste in the future with the addition of media for fixed film treatment at a cost effective price. Future tertiary treatment should be considered when selecting the layout of the treatment plant as well as when setting the hydraulic grade line of the plant. In addition, consideration and selection of an option for improvements to the solids handling process should be completed prior to design of the wastewater treatment process improvements.

Tables

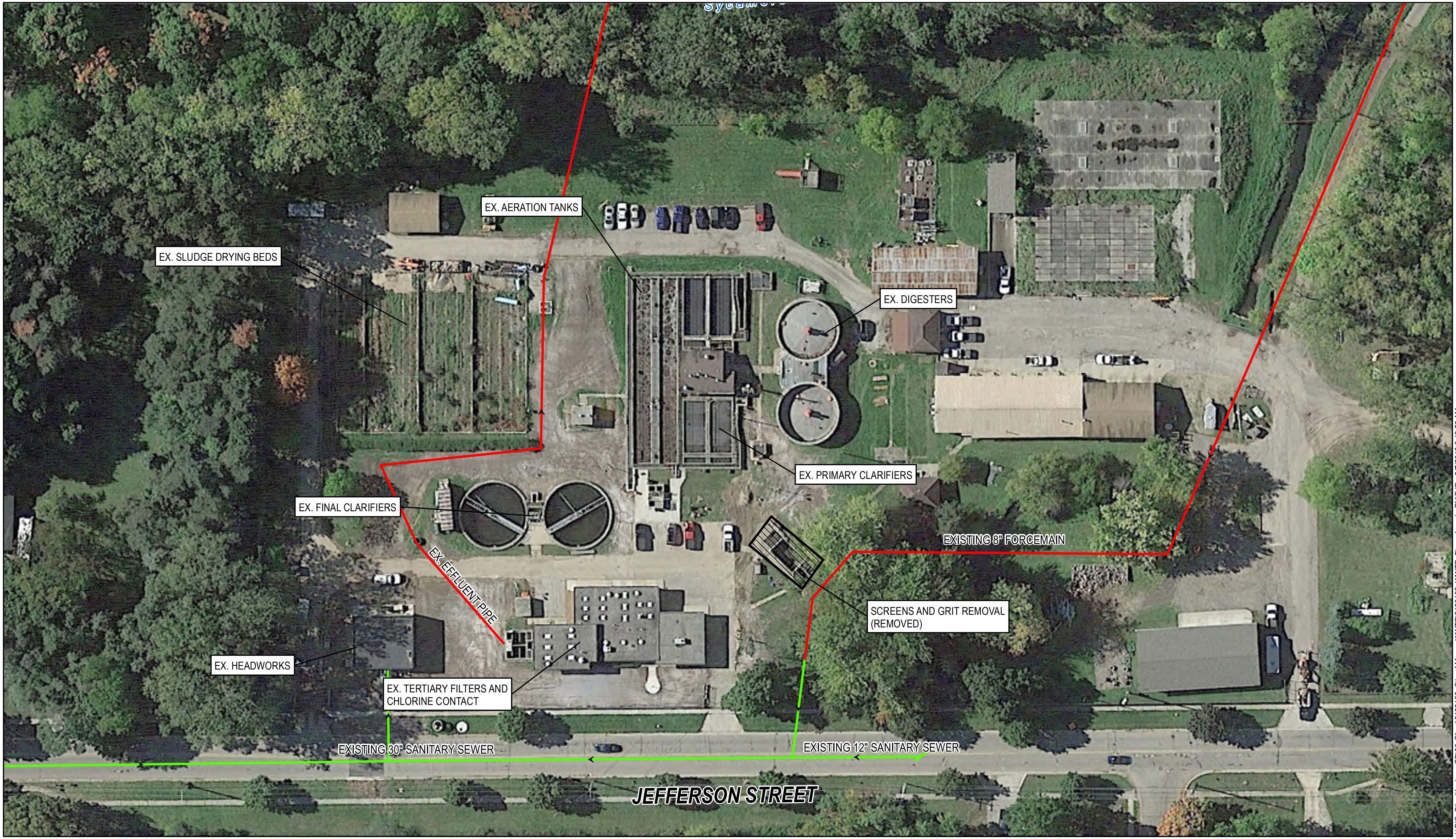
City of Mason
Wastewater Treatment Plant
Table 1
Projected Wastewater Flow and Quality

		Existing, 2012-2017		Projected, 2038		Prelimin. Design Parameters	
1.	**Flow**	**mgd**	**gpm**	**mgd**	**gpm**	**mgd**	**gpm**
	a. Annual Avg Day	1.08	750	1.20	833	1.70	1,181
	b. Summer Avg Day	0.98	681	1.07	743	1.60	1,111
	c. Avg Day of Max Month	1.93	1,340	2.10	1,458	2.20	1,528
	d. Peak Day	6.1	4,236	7.00	4,861	7.00	4,861
2.	**BOD, Influent**	**mg/L**	**lb/day**	**mg/L**	**lb/day**	**mg/L**	**lb/day**
	a. Annual Avg Day	199	1,792	200	2,002 [1]	200	2,836
	b. Summer Avg Day	204	1,667	200	1,785 [2]	200	2,669
	c. Avg Day of Max Month	284	2,141	290	2,391 [3]	290	3,387
	d. Peak Day	498	3,515		3,900 [3]		5,600
	max month loading factor		1.19				
	peak day loading factor		1.96				
5.	**Influent Ammonia**	**mg/L**	**lb/day**	**mg/L**	**lb/day**	**mg/L**	**lb/day**
	a. Annual Avg Day	22.9	196	25	250 [1]	25	354
	b. Summer Avg Day	24.3	196	25	223 [2]	25	334
	c. Avg Day of Max Month	29.3	256	30	330 [3]	30	470
	d. Peak Day	41	348	40	450 [3]	40	630
	max month loading factor		1.31				
	peak day loading factor		1.78				
6.	**Influent Phosphorus**	**mg/L**	**lb/day**	**mg/L**	**lb/day**	**mg/L**	**lb/day**
	a. Annual Avg Day	4.6	40	5.0	50 [1]	5.0	71
	b. Summer Avg Day	4.9	40	5.0	45 [2]	5.0	67
	c. Avg Day of Max Month	6.5	51	6.5	70 [3]	6.5	100
	d. Peak Day	8.3	90	8.3	120 [3]	8.3	160
	max month loading factor		1.28				
	peak day loading factor		2.24				
7.	**Influent TSS**	**mg/L**	**lb/day**	**mg/L**	**lb/day**	**mg/L**	**lb/day**
	a. Annual Avg Day	174	1,499	180	1,801 [1]	180	2,552
	b. Summer Avg Day	193	1,582	200	1,785 [2]	200	2,669
	c. Avg Day of Max Month	276	2,104	280	2,530 [3]	280	3,590
	d. Peak Day	670	8,146 *	670	9,790 [3]	670	13,900
	max month loading factor		1.40				
	peak day loading factor		5.43				
8.	**Influent Temperature**	**°F**					
	a. Minimum	40					
	b. Average	60					
	c. Maximum	72					

*Next highest max day TSS = 4,881

Notes
1. Based on 1.20 mgd
2. Based on 1.07 mgd
3. Based on loading factor

Figures



J:\GIS_Client\Ingham-GD\00-Client\2180168_City of Mason WWTP\2180168_WWTP_Figure1.mxd - PN\EJD - 8/7/2018 9:40:59 AM

SCALE: 1 " = 60 '

CITY OF MASON
INGHAM COUNTY, MICHIGAN

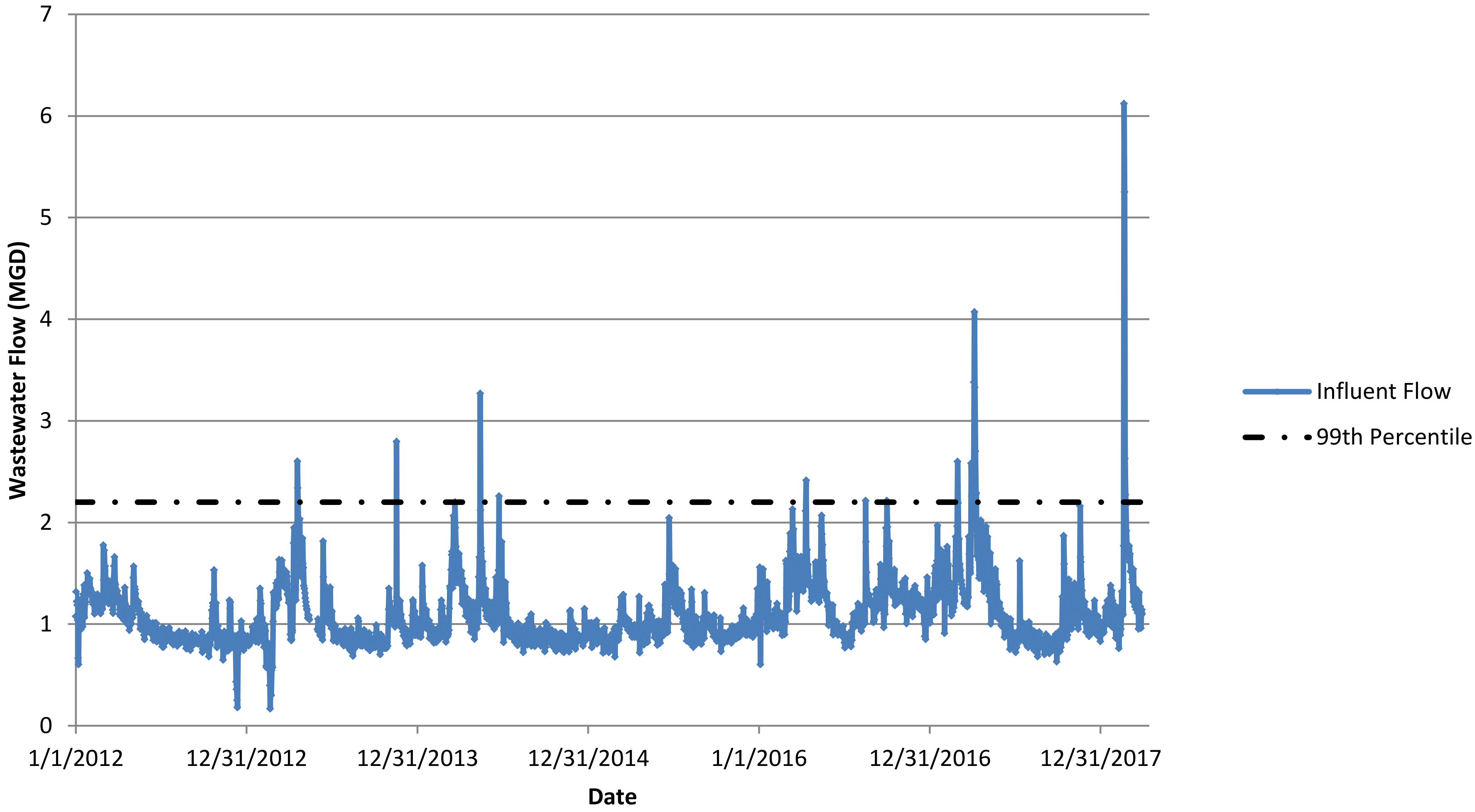
EXISTING SITE PLAN

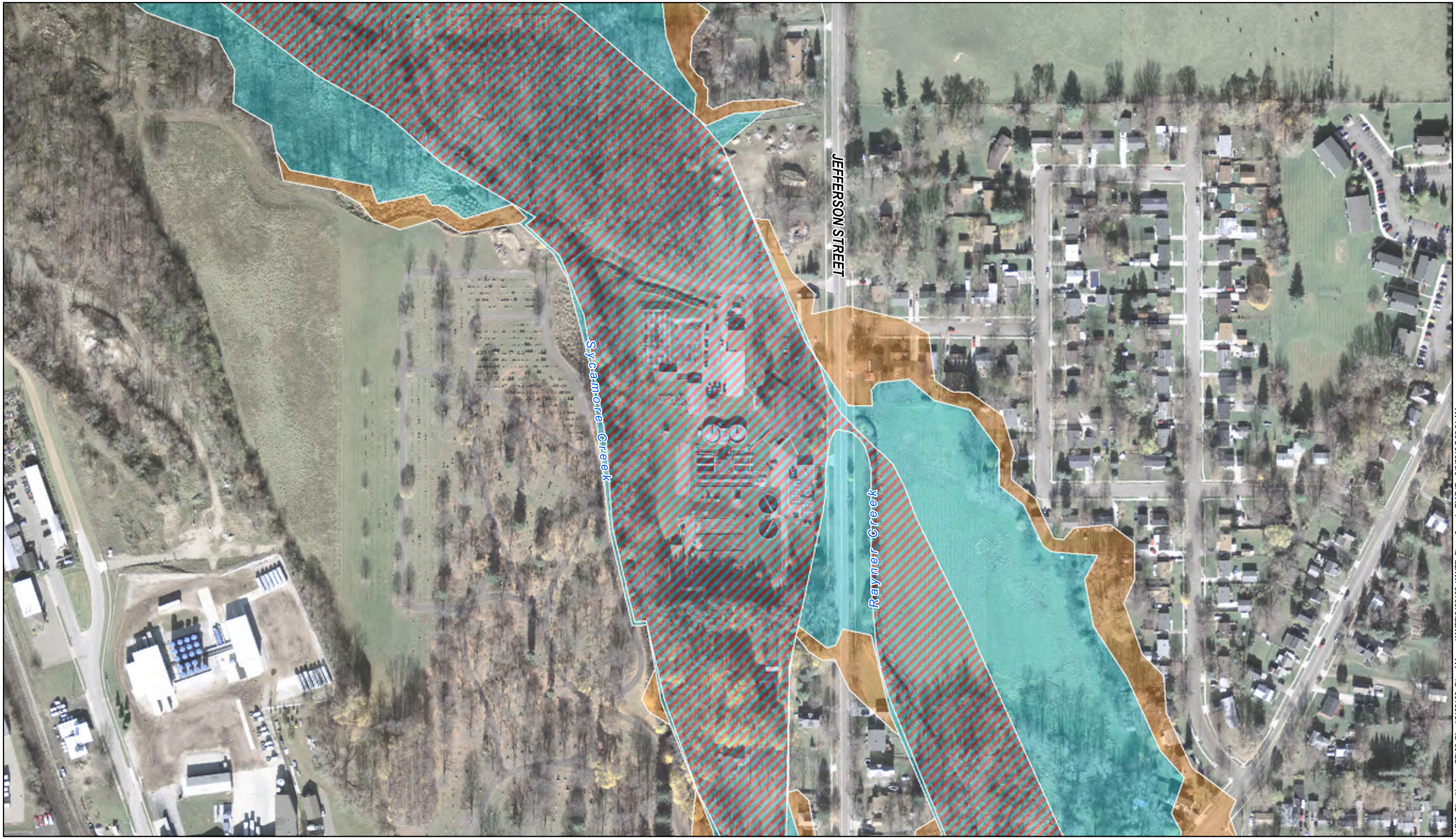
FIGURE 1

Prein&Newhof

2180168

Figure 2 - 2012 through March 2018 Influent Flow





SCALE: 1 " = 200 '

LEGEND

- 1% Annual Chance Flood Hazard
- Regulatory Floodway
- 0.2% Annual Chance Flood Hazard

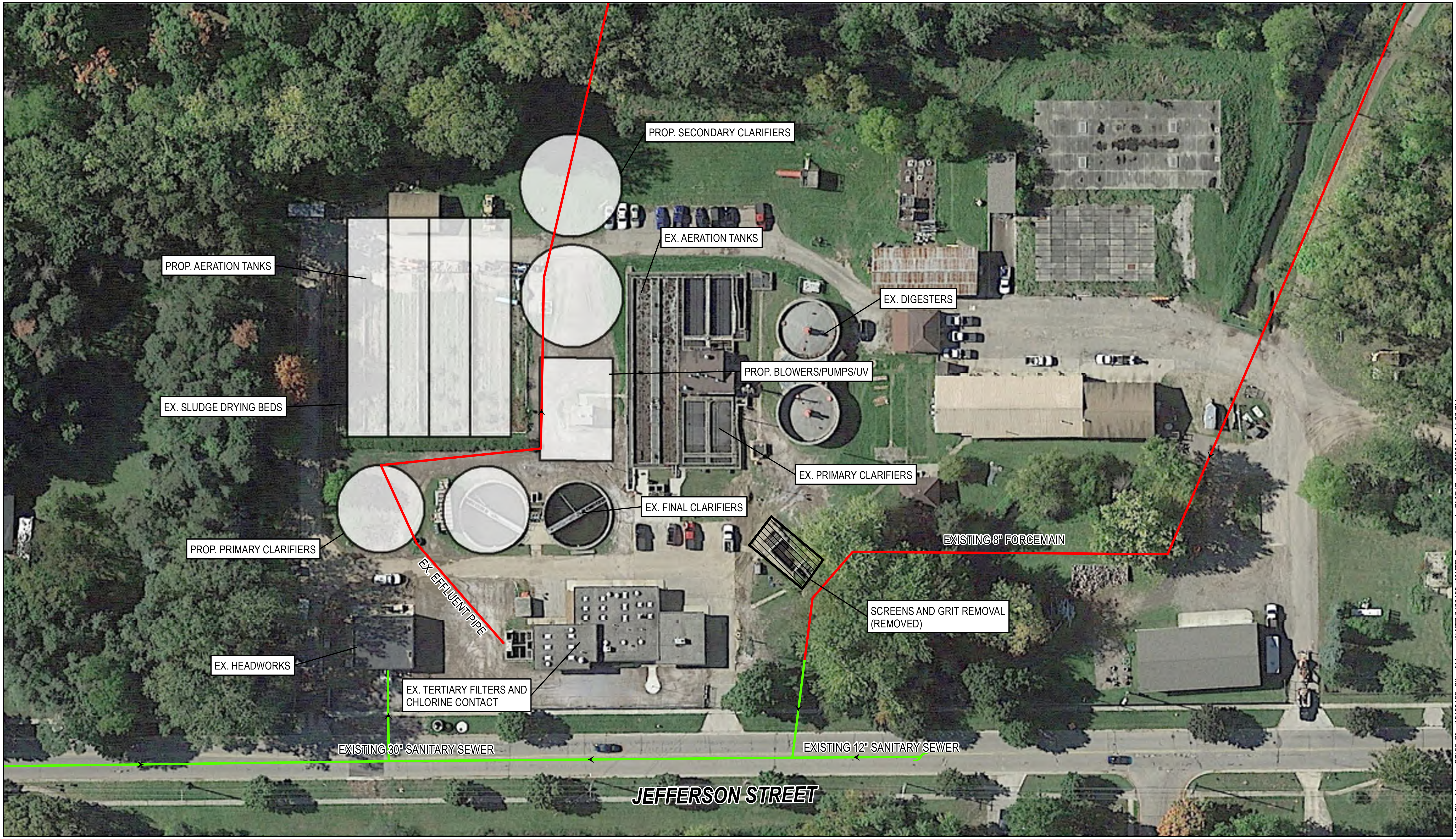
CITY OF MASON
INGHAM COUNTY, MICHIGAN

FLOOD HAZARDS

FIGURE 3

Prein&Newhof

2180168



SCALE: 1 " = 60 '

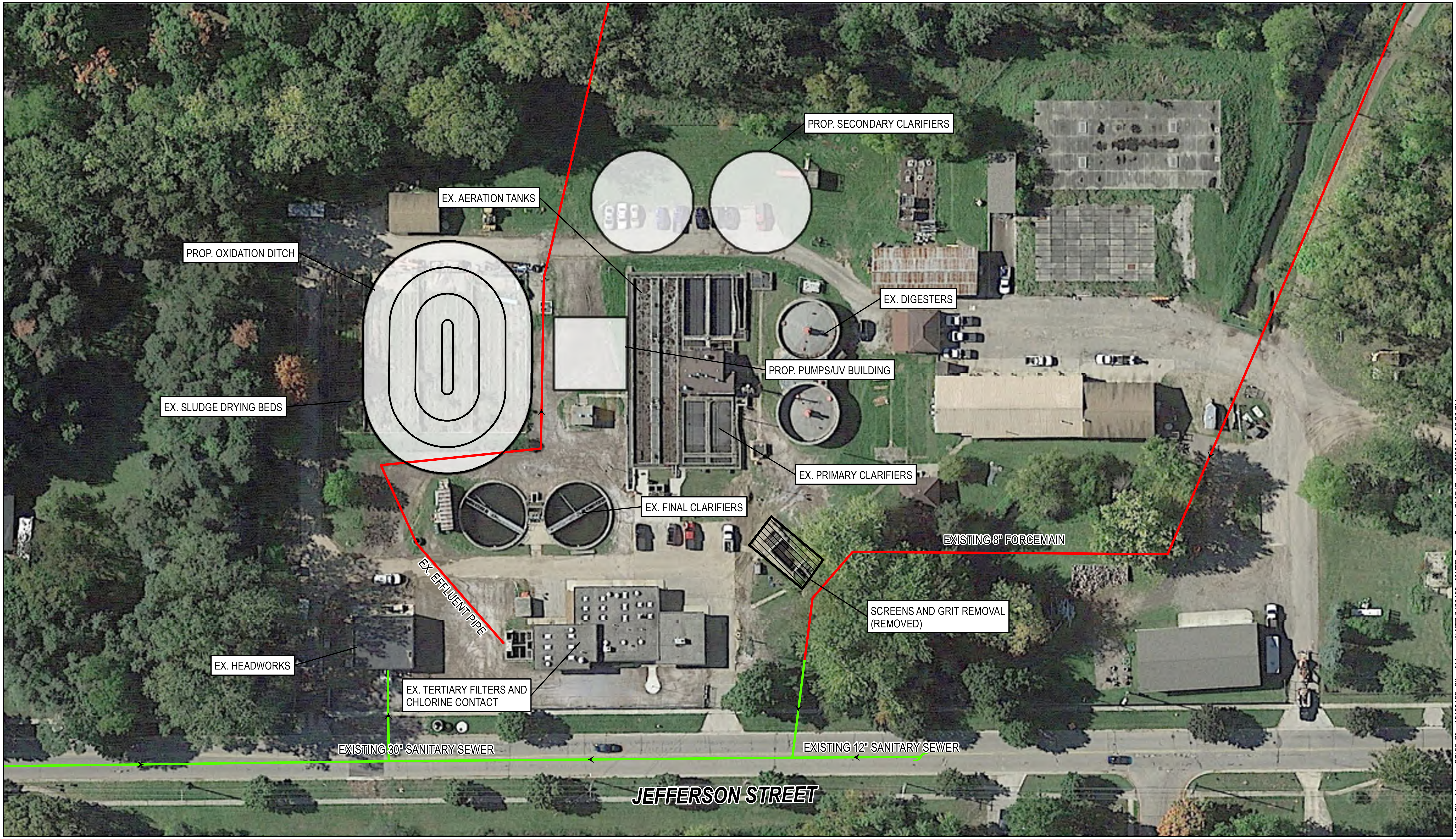
CITY OF MASON
INGHAM COUNTY, MICHIGAN

OPTION A - CONVENTIONAL SECONDARY ACTIVATED SLUDGE

FIGURE 4

Prein&Newhof

2180168



J:\GIS_Client\Ingham-GD\00-Client\2180168_City of Mason WWTP\2180168_WWTP_Figure5.mxd - PN\EJD - 10/15/2018 1:36:36 PM

SCALE: 1 " = 60 '

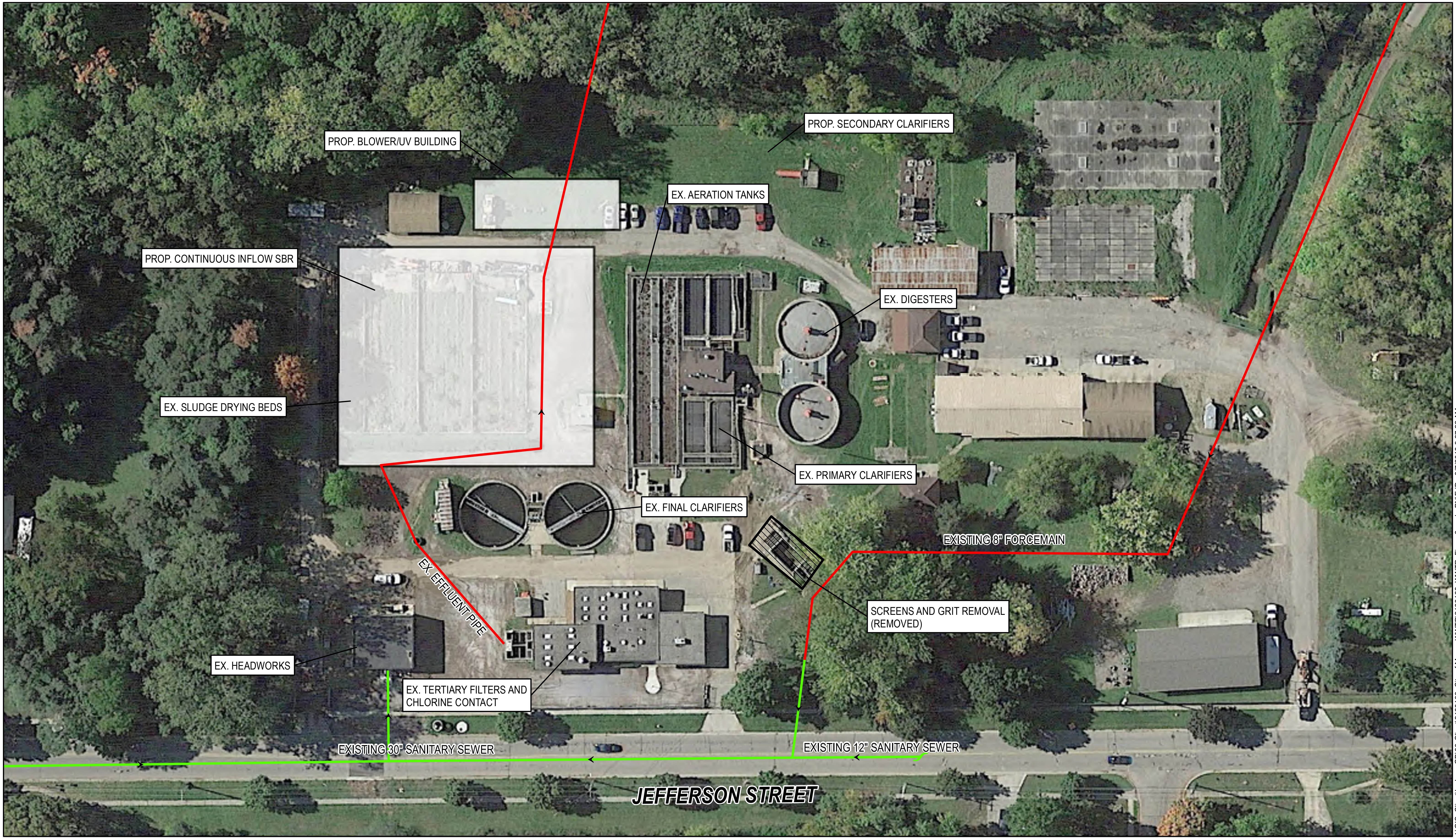
CITY OF MASON
INGHAM COUNTY, MICHIGAN

OPTION B - OXIDATION DITCH

FIGURE 5

Prein&Newhof

2180168



J:\GIS_Client\Ingham-GD\00-Client\2180168_City of Mason WWTP\2180168_WWTP_Figure6.mxd - PN\EJD - 8/7/2018 9:27:22 AM

SCALE: 1 " = 60 '

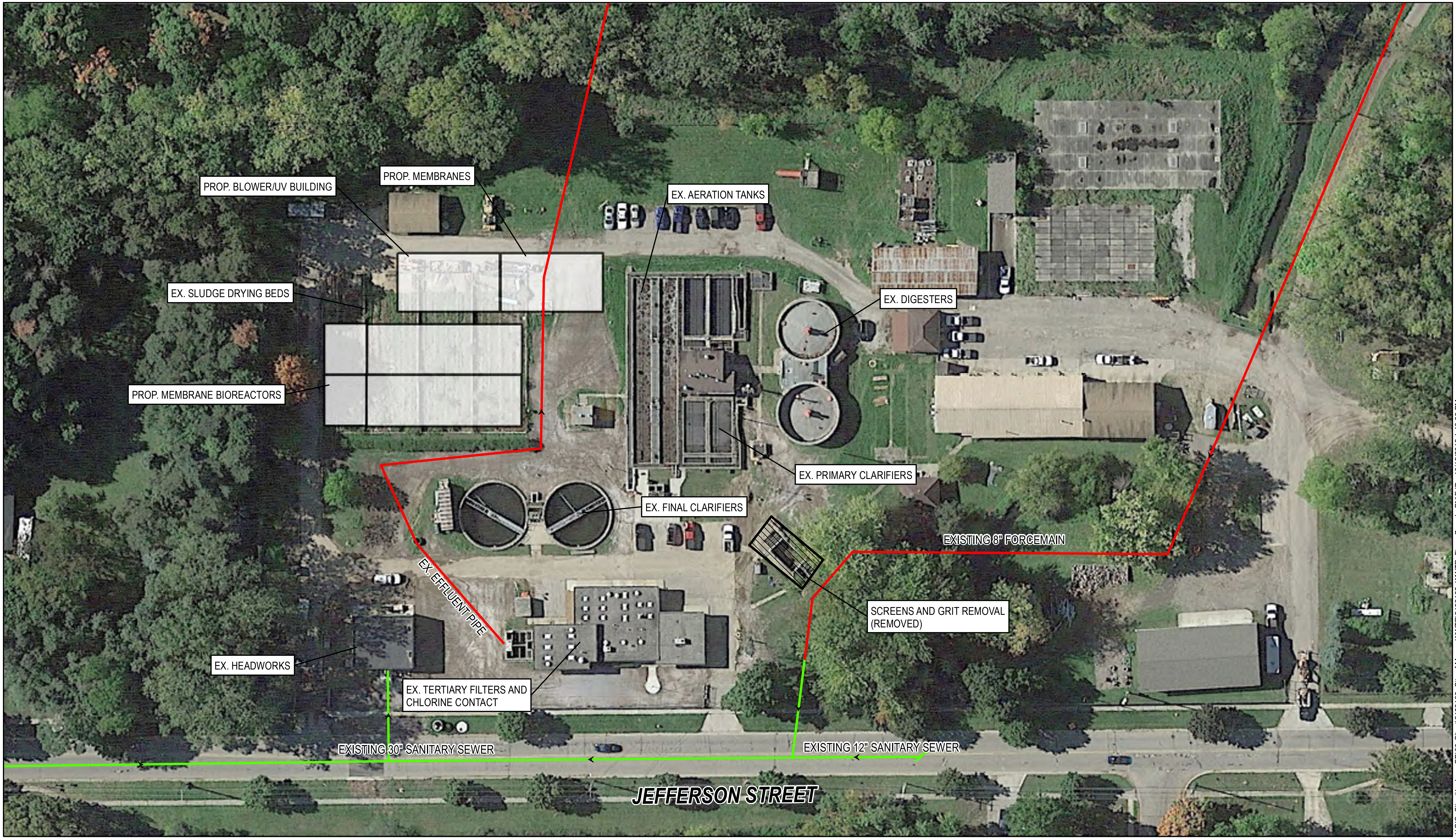
CITY OF MASON
INGHAM COUNTY, MICHIGAN

OPTION C - SEQUENCING BATCH REACTOR (SBR)

FIGURE 6

Prein&Newhof

2180168



J:\GIS_Client\Ingham-GD\00-Client\2180168_City of Mason WWTP\2180168_WWTP_Figure7.mxd - PN\EJD - 10/15/2018 1:48:33 PM

SCALE: 1 " = 60 '

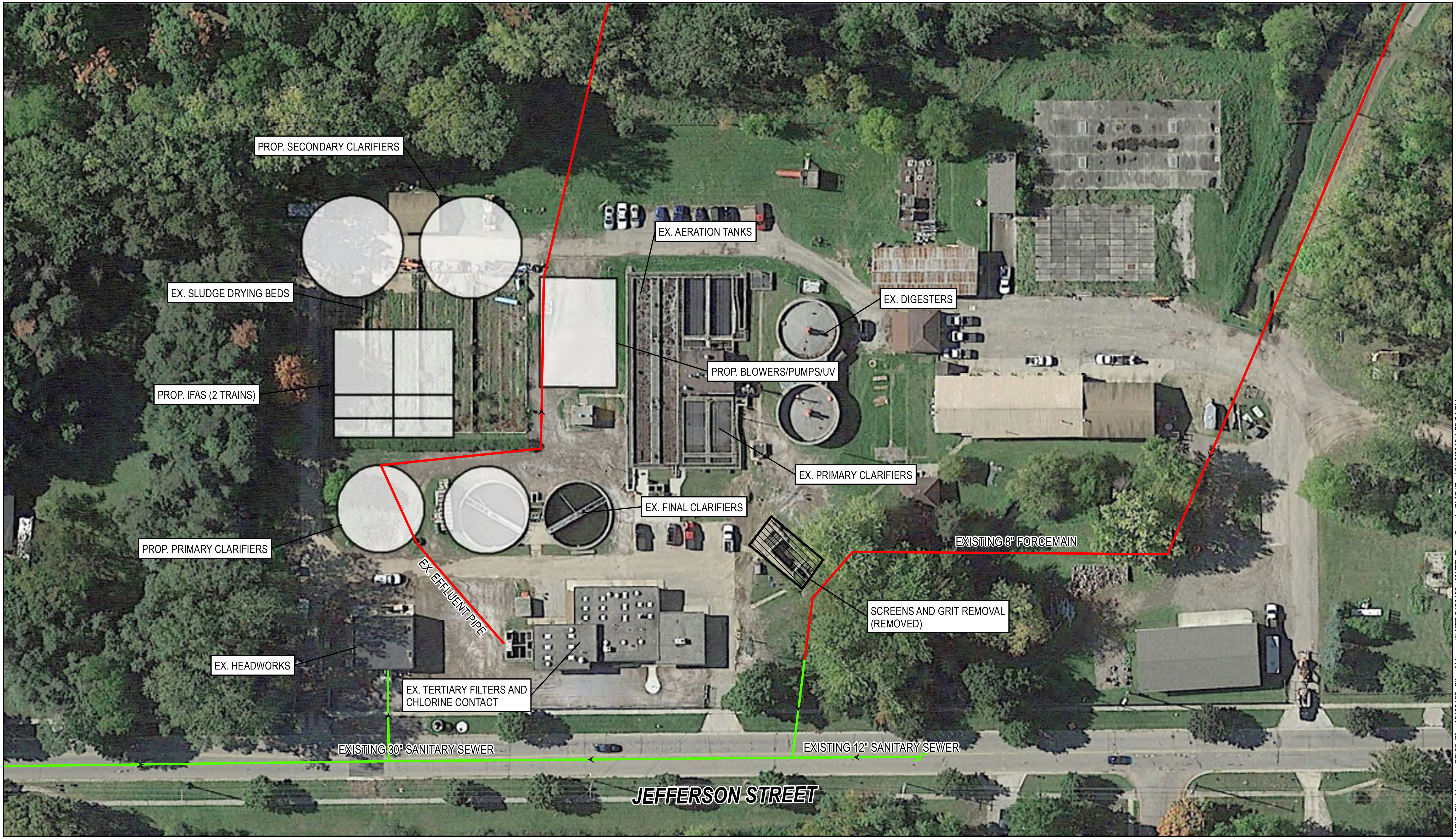
CITY OF MASON
INGHAM COUNTY, MICHIGAN

OPTION D - MEMBRANE BIOREACTORS

FIGURE 7

Prein&Newhof

2180168



J:\GIS_Client\Ingham-GD\00-Client\2180168_City of Mason WWTP\2180168_WWTP_Figure8.mxd - PN\EJD - 10/15/2018 1:53:54 PM

SCALE: 1 " = 60 '

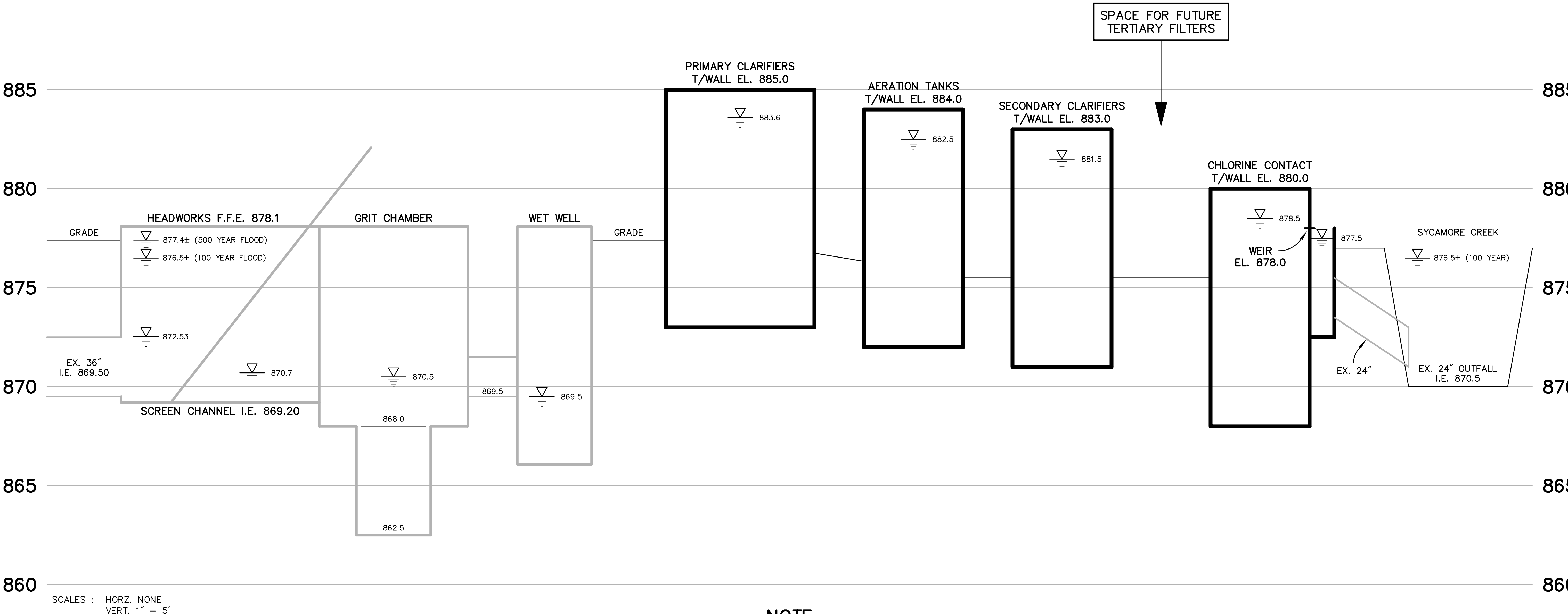
CITY OF MASON
INGHAM COUNTY, MICHIGAN

OPTION E - INTEGRATED FIXED FILM ACTIVATED SLUDGE (IFAS)

FIGURE 8

Prein&Newhof

2180168



NOTE

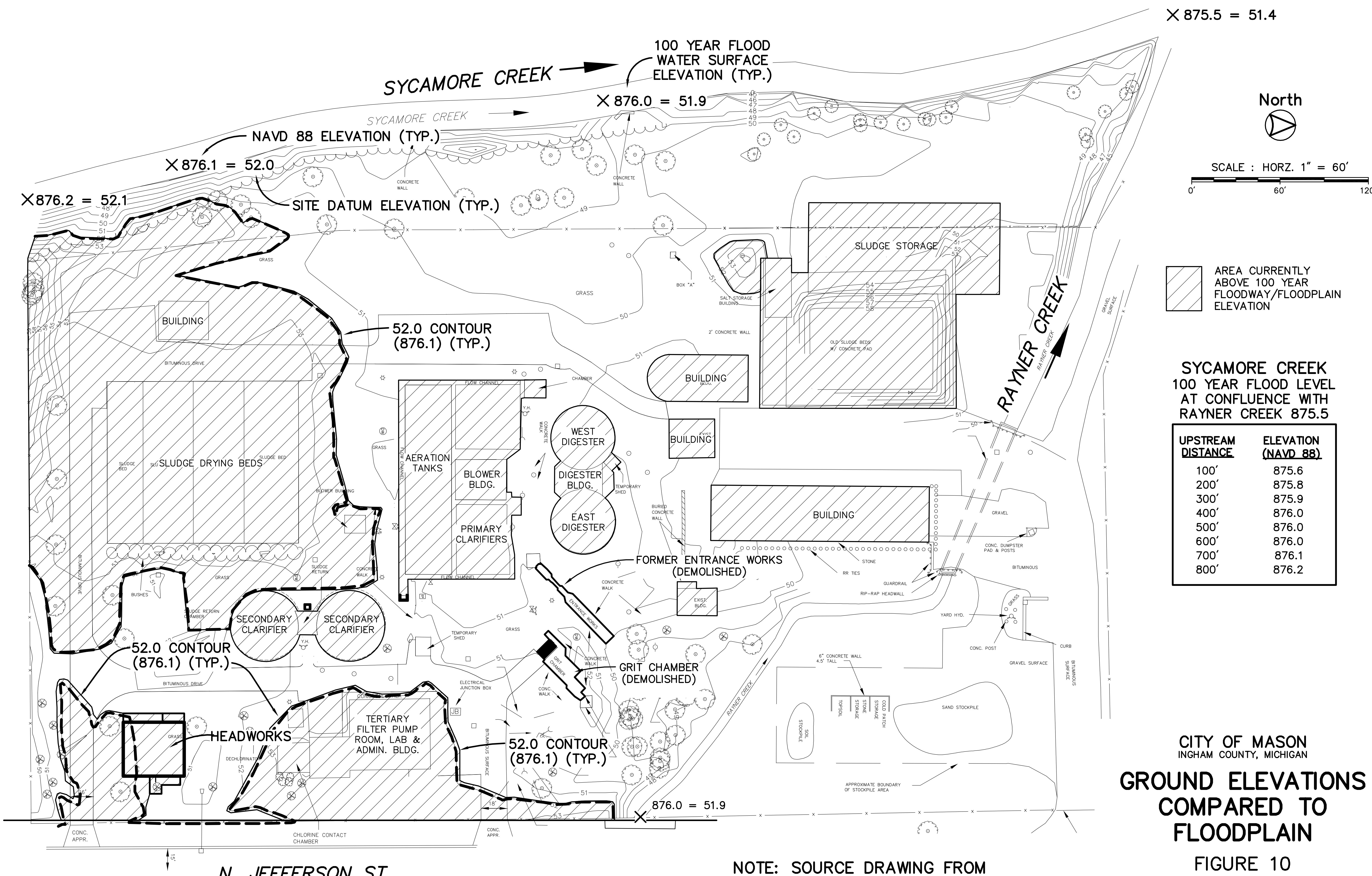
PRELIMINARY HYDRAULIC GRADE LINE INCLUDES CONSIDERATION OF FUTURE TERTIARY FILTRATION.

CITY OF MASON
INGHAM COUNTY, MICHIGAN
WWTP PLANNING STUDY

PRELIMINARY HYDRAULIC PROFILE EXISTING HEADWORKS TO FUTURE WWTP

FIGURE 9

Prein&Newhof
2180168



SYCAMORE CREEK
100 YEAR FLOOD LEVEL
AT CONFLUENCE WITH
RAYNER CREEK 875.5

UPSTREAM DISTANCE	ELEVATION (NAVD 88)
100'	875.6
200'	875.8
300'	875.9
400'	876.0
500'	876.0
600'	876.0
700'	876.1
800'	876.2

NOTE: SOURCE DRAWING FROM
WOLVERINE ENGINEERING

CITY OF MASON
INGHAM COUNTY, MICHIGAN

GROUND ELEVATIONS COMPARED TO FLOODPLAIN

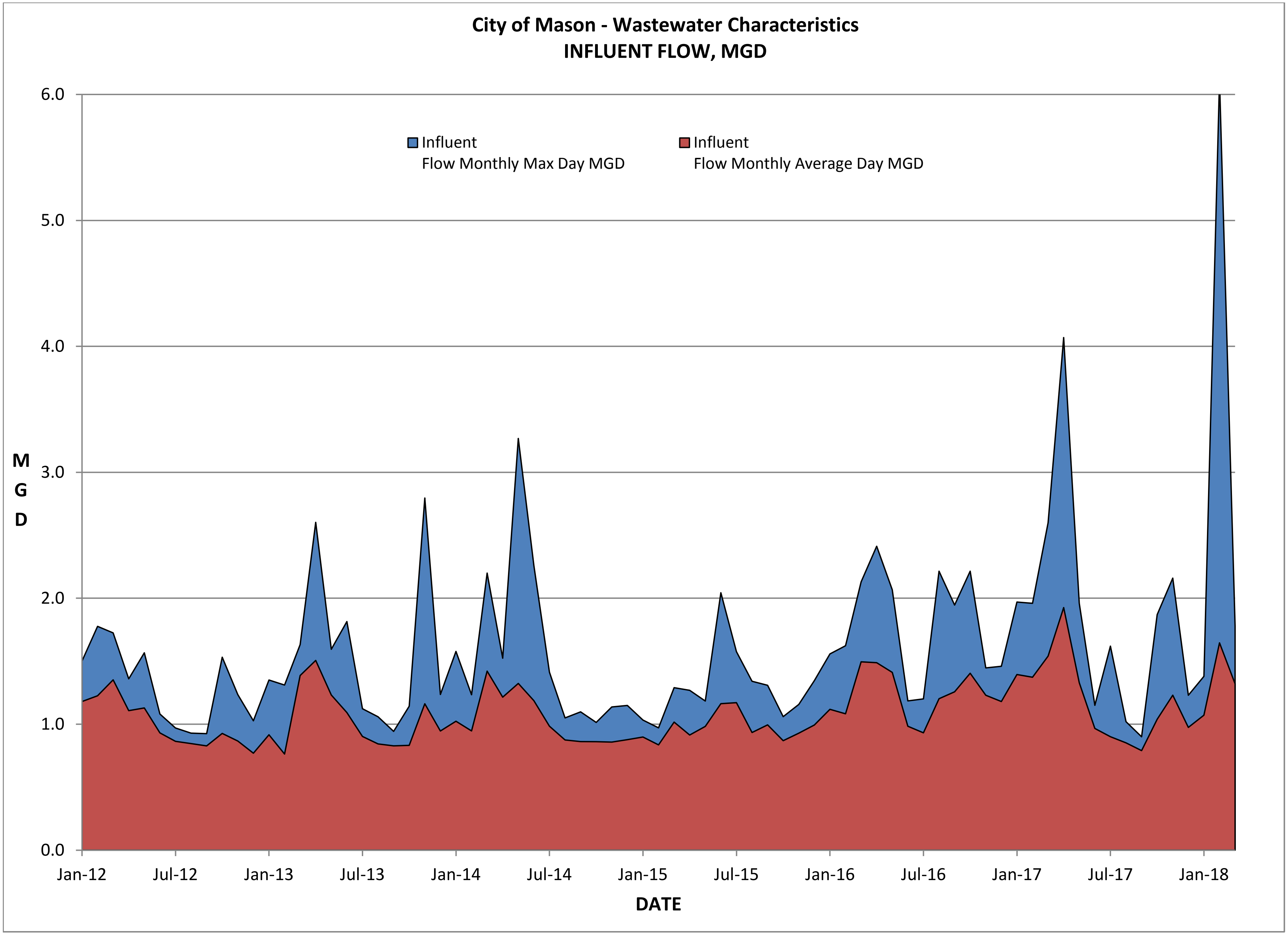
FIGURE 10

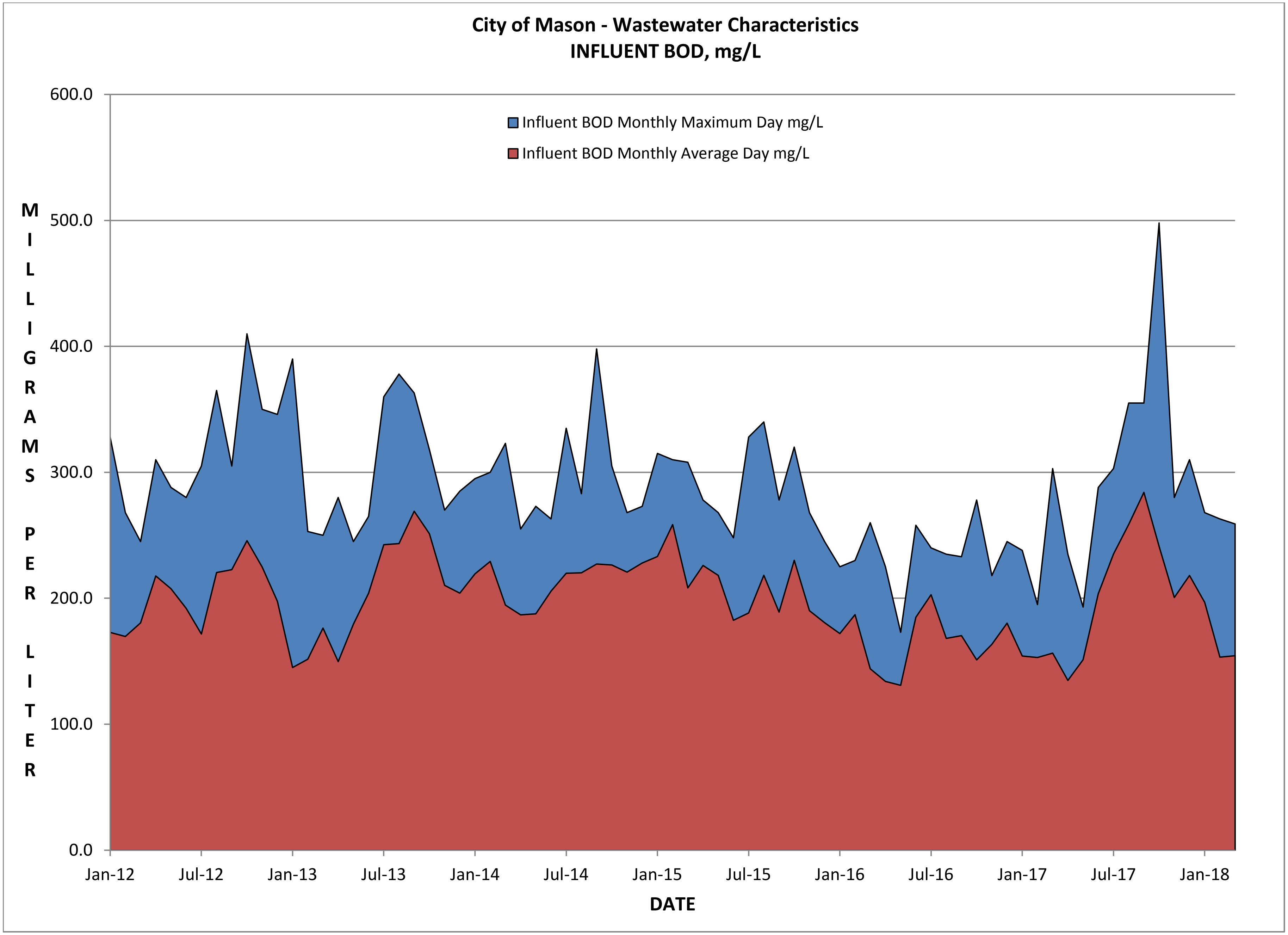
Prein&Newhof

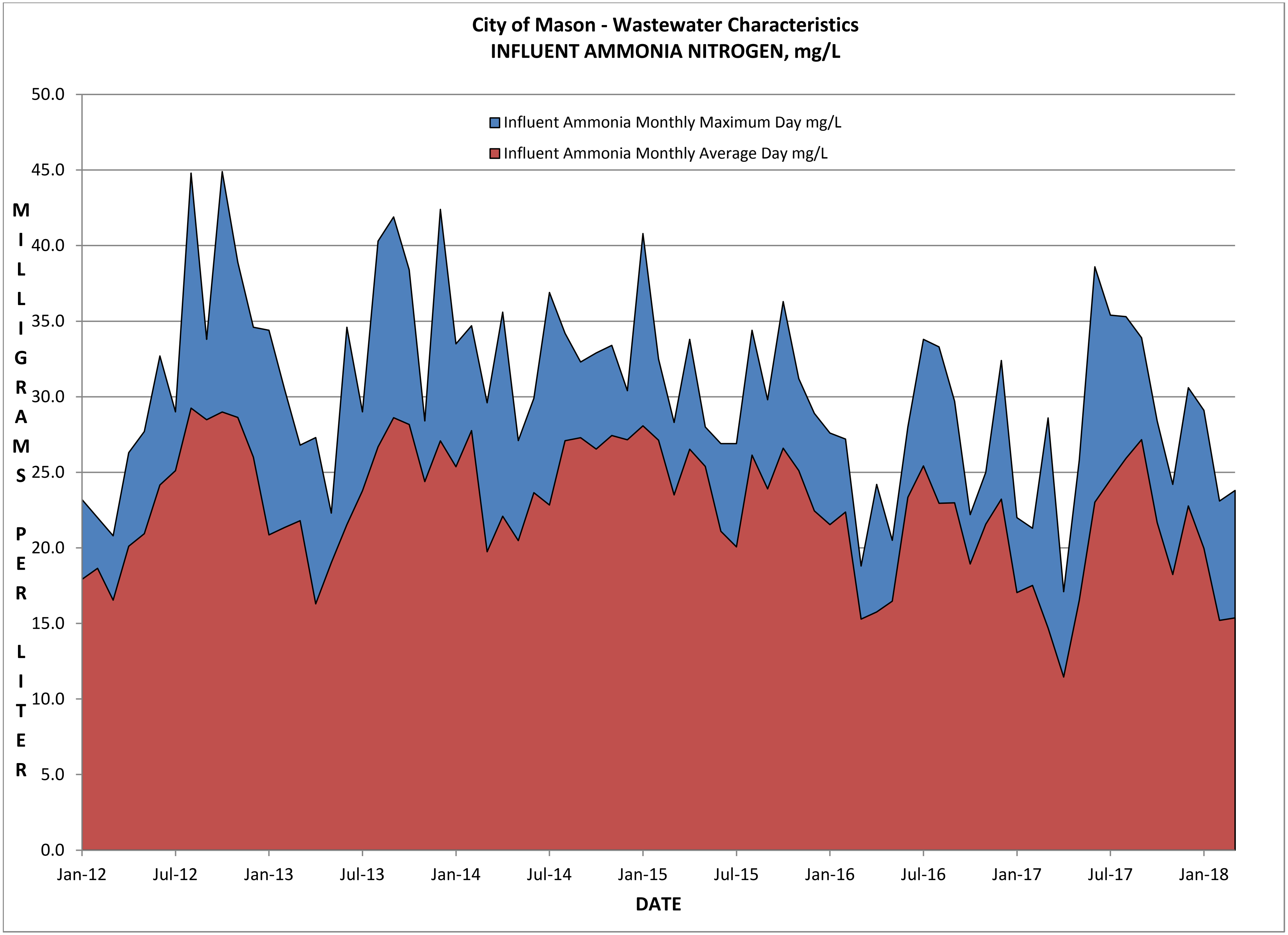
2180168

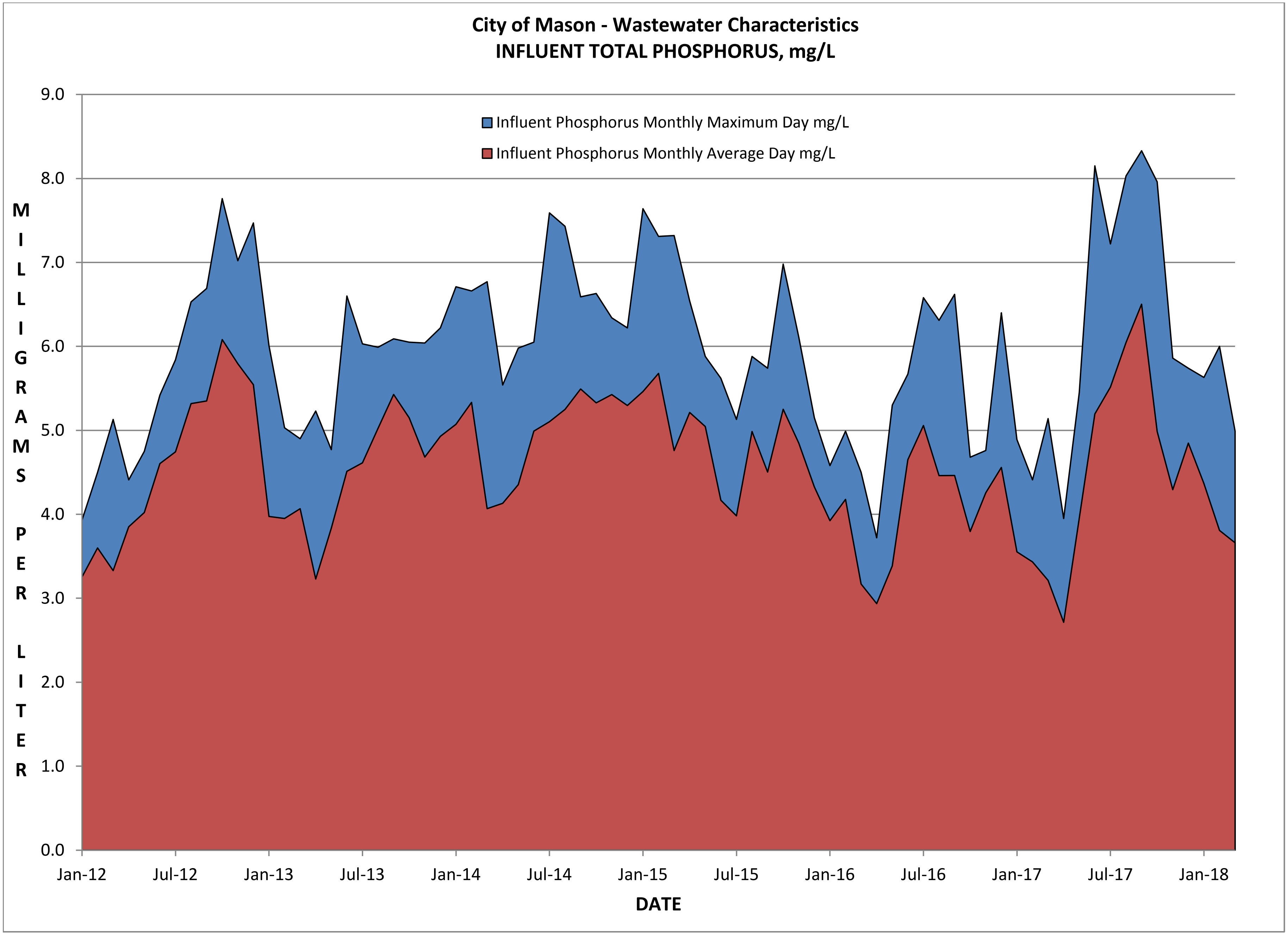
Appendix A

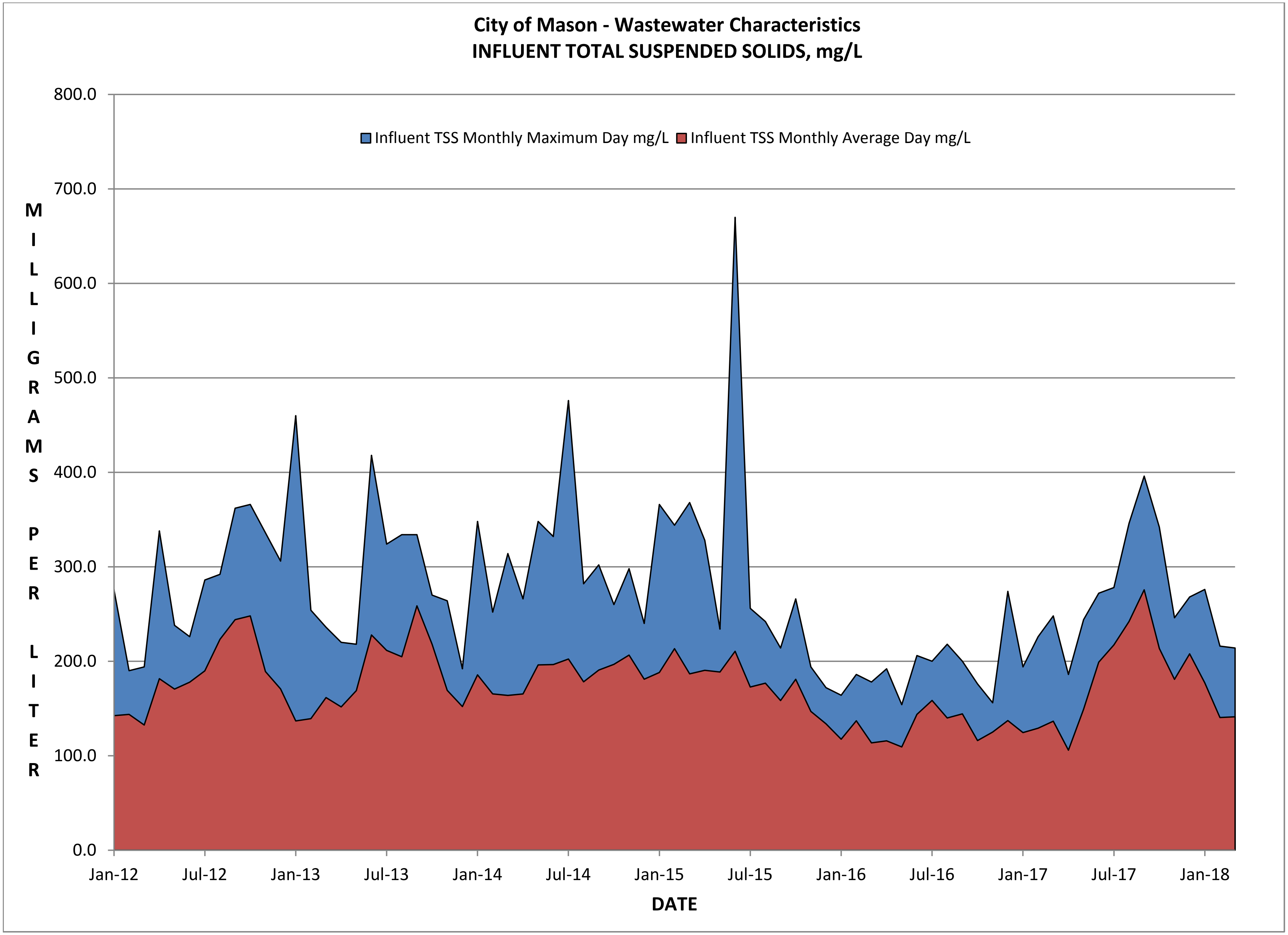
Existing WWTP Wastewater Quality Graphs

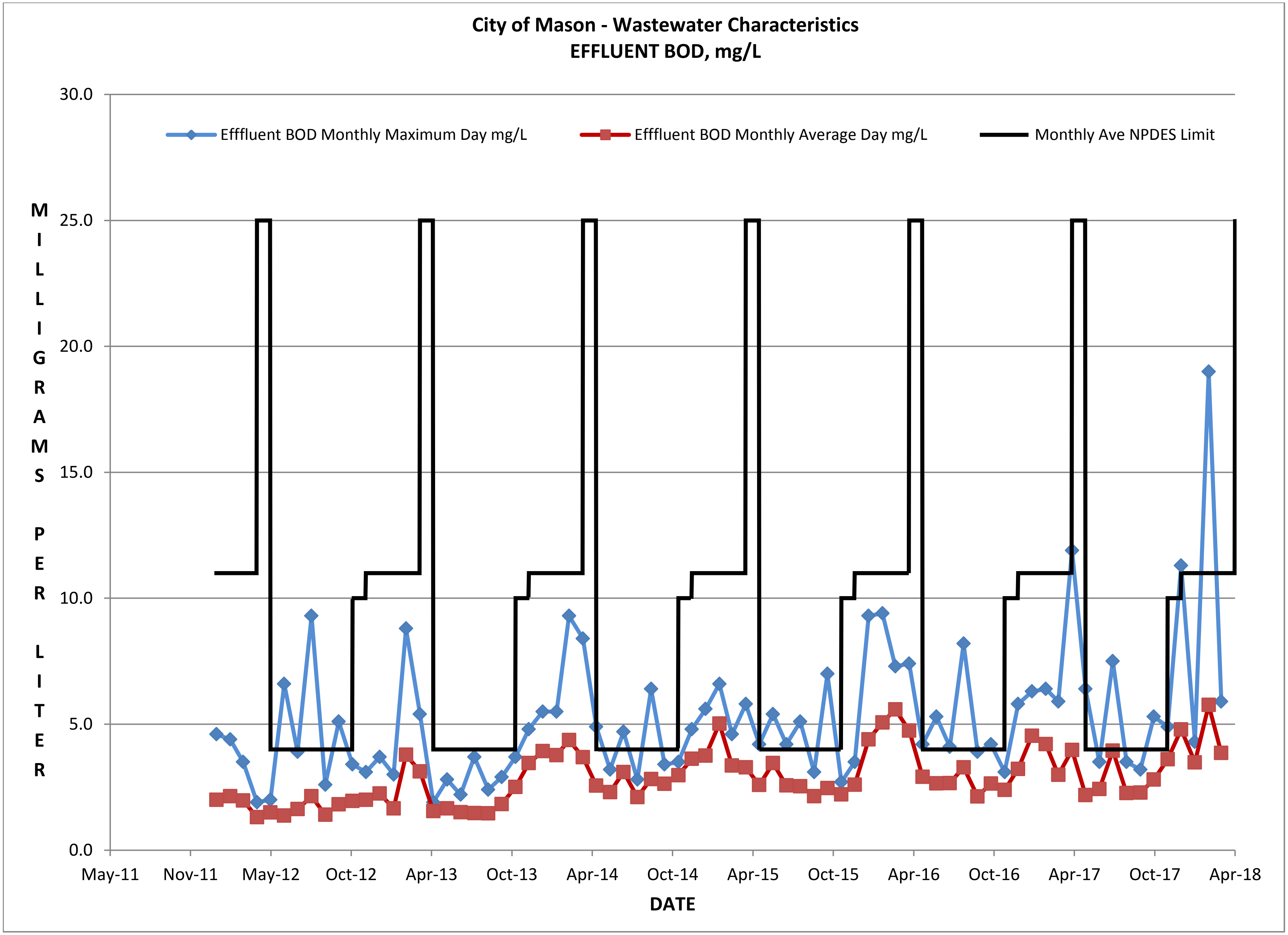

City of Mason - Wastewater Characteristics
INFLUENT FLOW, MGD
Influent
Flow Monthly Max Day MGD
Influent
Flow Monthly Average Day MGD
M
G
D
6.0
5.0
4.0
3.0
2.0
1.0
0.0
Jan-12
Jul-12
Jan-13
Jul-13
Jan-14
Jul-14
Jan-15
Jul-15
Jan-16
Jul-16
Jan-17
Jul-17
Jan-18
DATE

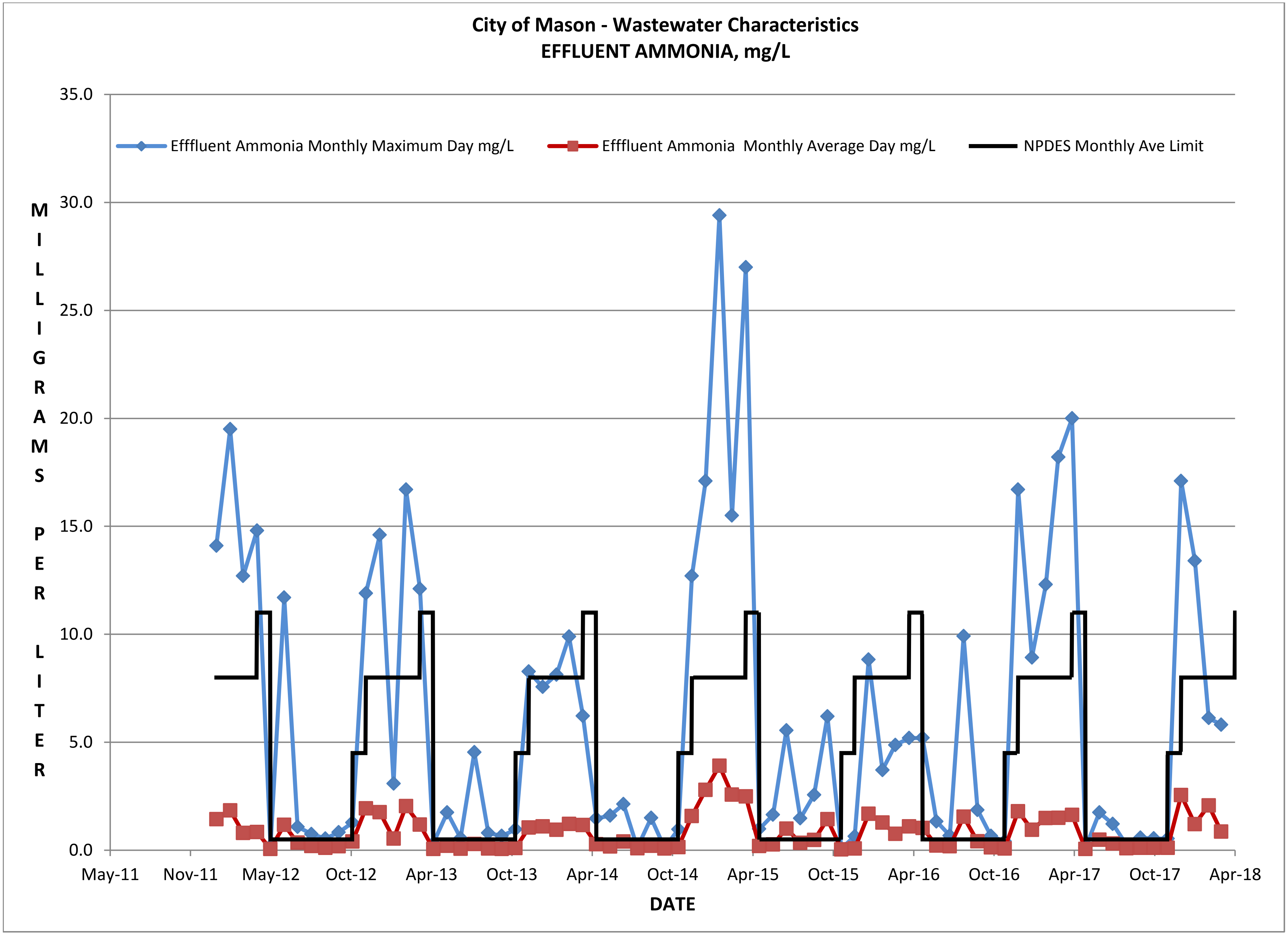

City of Mason - Wastewater Characteristics
INFLUENT BOD, mg/L
Influent BOD Monthly Maximum Day mg/L
Influent BOD Monthly Average Day mg/L
MILLIGRAMS PER LITER
600.0
500.0
400.0
300.0
200.0
100.0
0.0
Jan-12
Jul-12
Jan-13
Jul-13
Jan-14
Jul-14
Jan-15
Jul-15
Jan-16
Jul-16
Jan-17
Jul-17
Jan-18
DATE

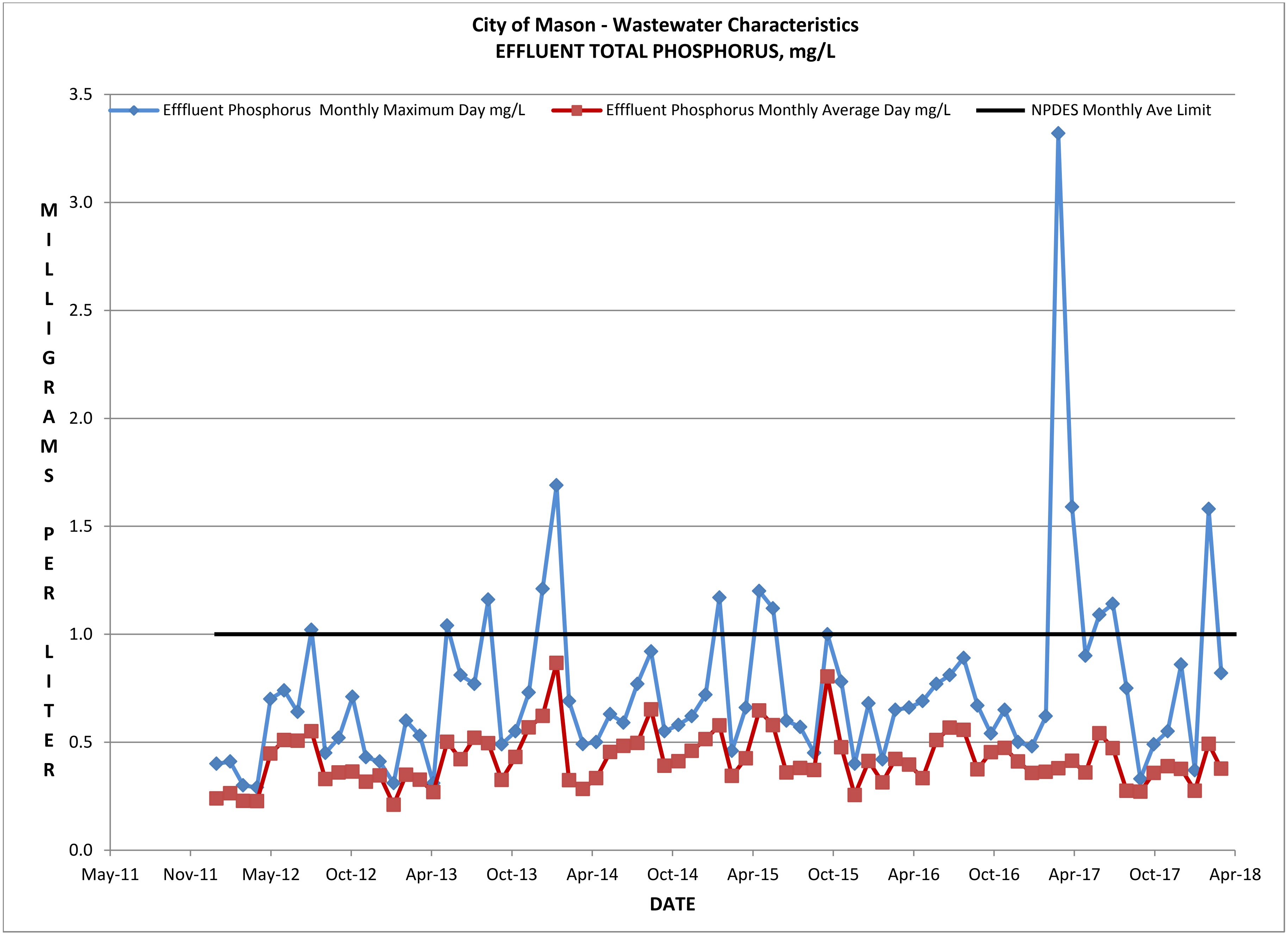

City of Mason - Wastewater Characteristics
INFLUENT AMMONIA NITROGEN, mg/L
Influent Ammonia Monthly Maximum Day mg/L
Influent Ammonia Monthly Average Day mg/L
MILLIGRAMS PER LITER
50.0
45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
Jan-12
Jul-12
Jan-13
Jul-13
Jan-14
Jul-14
Jan-15
Jul-15
Jan-16
Jul-16
Jan-17
Jul-17
Jan-18
DATE

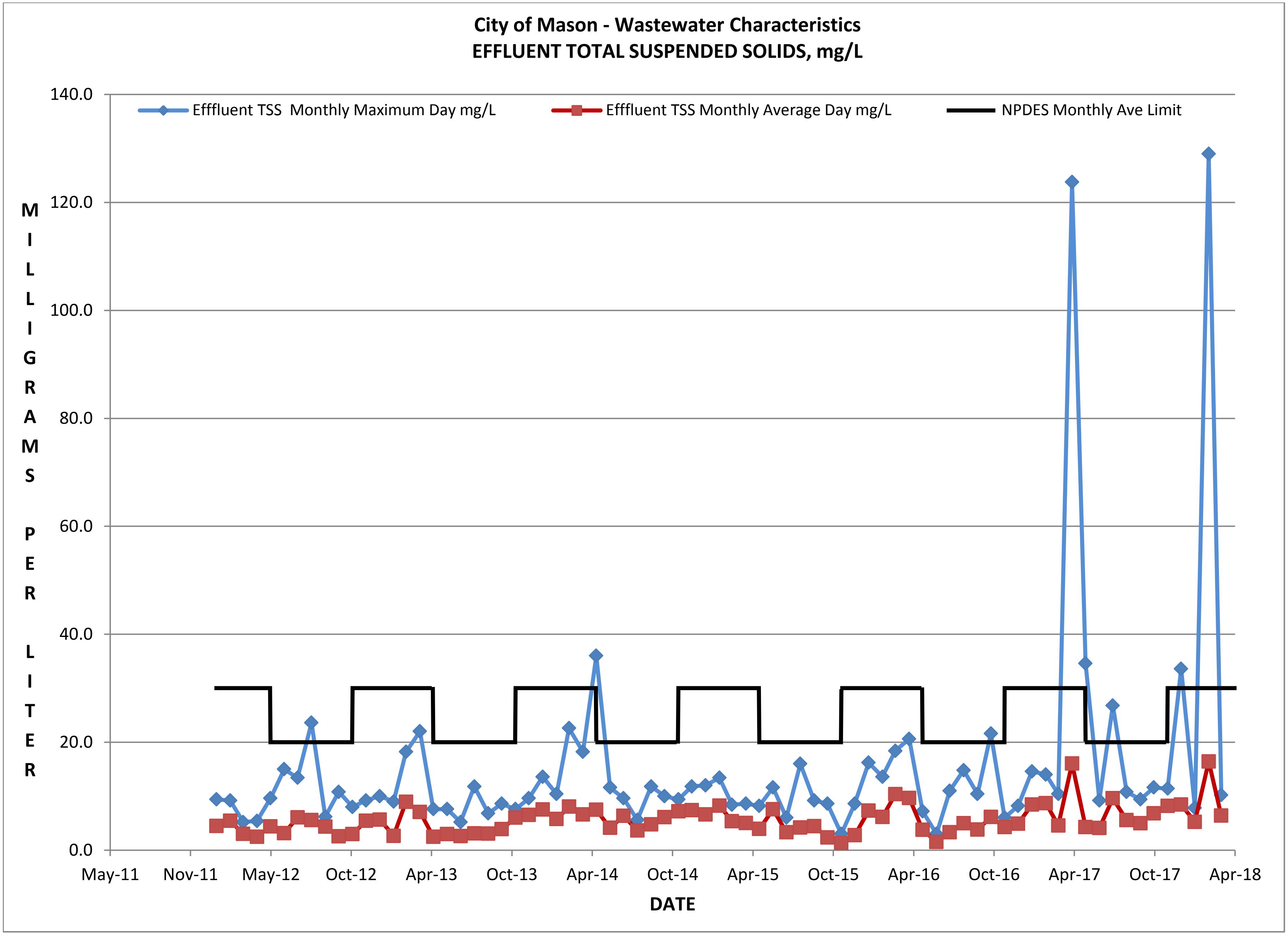

City of Mason - Wastewater Characteristics
INFLUENT TOTAL PHOSPHORUS, mg/L
Influent Phosphorus Monthly Maximum Day mg/L
Influent Phosphorus Monthly Average Day mg/L
MILLIGRAMS PER LITER
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
Jan-12
Jul-12
Jan-13
Jul-13
Jan-14
Jul-14
Jan-15
Jul-15
Jan-16
Jul-16
Jan-17
Jul-17
Jan-18
DATE


City of Mason - Wastewater Characteristics
INFLUENT TOTAL SUSPENDED SOLIDS, mg/L
Influent TSS Monthly Maximum Day mg/L
Influent TSS Monthly Average Day mg/L
MILLIGRAMS PER LITER
800.0
700.0
600.0
500.0
400.0
300.0
200.0
100.0
0.0
Jan-12
Jul-12
Jan-13
Jul-13
Jan-14
Jul-14
Jan-15
Jul-15
Jan-16
Jul-16
Jan-17
Jul-17
Jan-18
DATE


City of Mason - Wastewater Characteristics
EFFLUENT BOD, mg/L
Effluent BOD Monthly Maximum Day mg/L
Effluent BOD Monthly Average Day mg/L
Monthly Ave NPDES Limit
MILLIGRAMS PER LITER
30.0
25.0
20.0
15.0
10.0
5.0
0.0
May-11
Nov-11
May-12
Oct-12
Apr-13
Oct-13
Apr-14
Oct-14
Apr-15
Oct-15
Apr-16
Oct-16
Apr-17
Oct-17
Apr-18
DATE


City of Mason - Wastewater Characteristics
EFFLUENT AMMONIA, mg/L
Efffluent Ammonia Monthly Maximum Day mg/L
Efffluent Ammonia Monthly Average Day mg/L
NPDES Monthly Ave Limit
MILLIGRAMS PER LITER
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
May-11
Nov-11
May-12
Oct-12
Apr-13
Oct-13
Apr-14
Oct-14
Apr-15
Oct-15
Apr-16
Oct-16
Apr-17
Oct-17
Apr-18
DATE


City of Mason - Wastewater Characteristics
EFFLUENT TOTAL PHOSPHORUS, mg/L
Efffluent Phosphorus Monthly Maximum Day mg/L
Efffluent Phosphorus Monthly Average Day mg/L
NPDES Monthly Ave Limit
MILLIGRAMS PER LITER
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
May-11
Nov-11
May-12
Oct-12
Apr-13
Oct-13
Apr-14
Oct-14
Apr-15
Oct-15
Apr-16
Oct-16
Apr-17
Oct-17
Apr-18
DATE


City of Mason - Wastewater Characteristics
EFFLUENT TOTAL SUSPENDED SOLIDS, mg/L
Efffluent TSS Monthly Maximum Day mg/L
Efffluent TSS Monthly Average Day mg/L
NPDES Monthly Ave Limit
MILLIGRAMS PER LITER
140.0
120.0
100.0
80.0
60.0
40.0
20.0
0.0
May-11
Nov-11
May-12
Oct-12
Apr-13
Oct-13
Apr-14
Oct-14
Apr-15
Oct-15
Apr-16
Oct-16
Apr-17
Oct-17
Apr-18
DATE

Appendix B

Existing NPDES Permit Limits

PART I

Section A. Limitations and Monitoring Requirements

1. Final Effluent Limitations, Monitoring Point 001A

During the period beginning on the effective date of this permit and lasting until the expiration date of this permit, the permittee is authorized to discharge treated municipal wastewater from Monitoring Point 001A through Outfall 001. Outfall 001 discharges to Sycamore Creek. Such discharge shall be limited and monitored by the permittee as specified below.

	Maximum Limits for Quantity or Loading				Maximum Limits for Quality or Concentration					
Parameter	**Monthly**	**7-Day**	**Daily**	**Units**	**Monthly**	**7-Day**	**Daily**	**Units**	**Monitoring Frequency**	**Sample Type**
Flow	(report)	---	(report)	MGD	---	---	---	---	Daily	Report Total Daily Flow
Carbonaceous Biochemical Oxygen Demand ($CBOD_5$)										
May 1 – Oct. 31	50	130	---	lbs/day	4	---	10	mg/l	5×Weekly	24-Hr Composite
Nov. 1 – Nov. 30	130	190	---	lbs/day	10	---	15	mg/l	5×Weekly	24-Hr Composite
Dec. 1 – Mar. 31	140	210	---	lbs/day	11	---	17	mg/l	5×Weekly	24-Hr Composite
Apr. 1 – Apr. 30	310	500	---	lbs/day	25	---	40	mg/l	5×Weekly	24-Hr Composite
Total Suspended Solids										
May 1 – Oct. 31	250	380	---	lbs/day	20	30	---	mg/l	5×Weekly	24-Hr Composite
Nov. 1 – Apr. 30	380	560	---	lbs/day	30	45	---	mg/l	5×Weekly	24-Hr Composite
Ammonia Nitrogen (as N)										
May 1 – Oct. 31	6	25	---	lbs/day	0.5	---	2	mg/l	5×Weekly	24-Hr Composite
Nov. 1 – Nov. 30	---	56	---	lbs/day	---	---	4.5	mg/l	5×Weekly	24-Hr Composite
Dec. 1 – Mar. 31	100	---	---	lbs/day	8	---	---	mg/l	5×Weekly	24-Hr Composite
Apr. 1 – Apr. 30	140	---	---	lbs/day	11	---	---	mg/l	5×Weekly	24-Hr Composite
Total Phosphorus (as P)										
	13	---	---	lbs/day	1.0	---	---	mg/l	5×Weekly	24-Hr Composite
Total Silver	0.02	---	0.28	lbs/day	1.4	---	22	µg/l	Monthly	24-Hr Composite
Total Copper	0.6	---	1.8	lbs/day	47	---	140	µg/l	Monthly	24-Hr Composite
Total Nickel	2.5	---	40	lbs/day	200	---	3200	µg/l	Monthly	24-Hr Composite
Total Selenium	0.07	---	1.5	lbs/day	5.8	---	120	µg/l	Monthly	24-Hr Composite
Fecal Coliform Bacteria	---	---	---	---	200	400	---	cts/100 ml	5×Weekly	Grab
Total Residual Chlorine	---	---	---	---	---	---	0.038	mg/l	5×Weekly	Grab
Total Mercury	(report)	---	---	lbs/day	(report)	---	---	ng/l	Monthly	Grab
	12-Month Rolling Average				**12-Month Rolling Average**					
Total Mercury	0.0001	---	---	lbs/day	8.0	---	---	ng/l	Monthly	Calculation

Appendix C

Summary of Options

CITY OF MASON
WASTEWATER TREATMENT PLANT UPGRADES
APPENDIX C - SUMMARY OF WASTEWATER TREATMENT SYSTEM OPTIONS

Rank	Option	Process	Manufacturers	Approximate Footprint for Suspended and/or Attached Growth Process	Installed Aeration Power (hp)	Approx. Average Operating Power (hp)	Approx. Aeration Operation Cost per Year	Aeration Method	Flow (MGD)	Controls	Total Estimated Construction Costs	Total Estimated 20-yr Present Value	Advantages	Disadvantages
1	A	Conventional Secondary Treatment	Sanitaire, EDI	160' x 130', (1) anaerobic reactor, (2) aerated-anoxic, (2) aerated -aerobic	180 hp	61	$35,863	Blowers/ Diffusers	Ave Day - 1.7 Max Mo. - 2.2 Peak - 7.0	Automatic Aeration Control, DO probes	$13,000,000	$13,600,000	Matches existing process with nutrient removal enhancements, lowest operation cost, provides full capacity with any one tank out of service	Larger footprint than some options
2	E	Moving Bed Bioreactor/IFAS	Kruger	80'x75' (2) Anaerobic (2) "C-Stage" (2) IFAS	300	150	$88,186	Blowers/ Diffusers	Ave Day - 1.7 Max Mo. - 2.2 Peak - 7.0	Automatic Aeration Control, DO probes	$13,200,000	$14,600,000	Smaller footprint than conventional activated sludge, can provide better treatment of higher loadings	Operation is more complex than Conventional Secondary
3	B	Oxidation Ditch	Evoqua	Evoqua - 148' X 124' (3) Concentric Channels	120 hp (2) 30 hp (2) 60 hp	82	$48,209	Rotating Disc Aerators	Ave Day - 1.7 Max Mo. - 2.2 Peak - 7.0	Automatic Aeration Control, DO probes	$12,300,000	$13,100,000	No blowers or diffusers to replace/maintain, lower cost	Does not provide equivalent redundancy with any one channel out of service as Conventional Secondary treatment unless two ditches constructed at additional cost of $1.9M, primary clarifiers recommended, but space not available
		Oxidation Ditch	Westech	Westech - 193'X110', (2) Single Channel Loops	Westech 240 hp (3) 60 hp	180	$105,824	Vertical Shaft Impellers	Ave Day - 1.7 Max Mo. - 2.2 Peak - 7.0	Automatic Aeration Control, DO probes	NA, Does not fit	NA, Does not fit	No blowers or diffusers to replace/maintain, provides redundancy with two separate loops	Does not fit in area of existing sludge drying beds, would likely need separate property to utlize this system
4	C	SBR	Sanitaire	Sanitaire 180'x155'	100 hp (2 blowers) and 25 hp (1 mixer)	102	$59,967	Blowers/ Diffusers	Ave Day - 1.7 Max Mo. - 2.2 Peak - 7.0	Automatic Aeration, Settling, Decant	$12,400,000	$13,400,000	No primary or secondary clarifiers needed. No RAS pumps needed.	All treatment depends on one treatment unit and automatic cycling of stages of batch, high flow discharge
		SBR	Aqua Aerobics	Aqua Aerobics 225'x75'		135	$79,368	Blowers/ Diffusers	Ave Day - 1.7 Max Mo. - 2.2 Peak - 7.0	Automatic Aeration, Settling, Decant	$13,000,000	$14,300,000	No primary or secondary clarifiers needed. No RAS pumps needed.	Requires 125,000 gallon post-SBR EQ basin with aeration and 5 submersible pumps
5	D	Membrane Bioreactor	Memcor GE Suez	132'x78' (2) Anoxic (2) Aerobic	530	235	$138,159	Blowers/ Diffusers	Ave Day - 1.7 Max Mo. - 2.2 Peak - 4.0 (requires EQ)	Automatic Aeration, Doand ORP Probes, Auto Cleaning	$15,700,000	$18,000,000	No secondary clarifiers required	Finer screening required prior to membranes, energy intensive and does not handle ex. peak flows, Highest Capital Cost

Notes:
Operation costs do not include labor, expected to be nearly equal for all options
20-Year Present Value includes construction cost and Present Worth of aeration equipment operation costs for 20 years at a 2% rate.

Appendix D

Cost Estimates

City of Mason
Wastewater Treatment Plant

Option A: Conventional Activated Sludge

Plant Unit	**Cost Estimate**
1. Primary Clarifiers (60')	
a. Concrete	$264,000
b Equipment	$250,000
c Equipment Installation	$175,000
2. Aeration Tank System	
a. Concrete	$1,600,000
b. Equipment	$395,000
c. Equipment Installation	$276,500
3. Blower, RAS/WAS and UV Building	
a. Structure	$1,050,000
b. Blowers	- Incl. Item 2
c. RAS/WAS Pumps	$100,000
d. UV	$340,000
4. Secondary Clarifiers (70')	
a. Concrete	$440,000
a. Equipment	$295,000
a. Equipment Installation	$206,500
5. Site Piping	
a. Total site pipng about 2,200 feet	$550,000
6. SCADA System	$650,000
7. Site Electrical Including Generator	$1,100,000
8. Site Excavation incl. small diameter buried piping	$500,000
9. Conversion of Existing Tankage to Equalization with Pumps	$350,000
10. Temporary Reroute of Effluent Pipe during Construction	$250,000

Option A: Conventional Activated Sludge (continued)

11.	Demolition of Structures, Pipe Abandonment and Connections	$300,000
12.	Contractor General Conditions	$910,000
	Construction Total	$10,002,000
	Contingency, Legal, Engineering, and Fiscal	$3,000,000
	Project Total	$13,002,000

City of Mason
Wastewater Treatment Plant

Option B: Oxidation Ditch (Concentric Loops)

Plant Unit	**Cost Estimate**
1. Primary Clarifiers (60')	
a. Concrete	$264,000
b Equipment	$250,000
c Equipment Installation	$175,000
d Property Acquisition	$400,000
2. Oxidation Ditch	
a. Concrete	$1,240,000
b. Equipment	$390,000
c. Equipment Installation	$273,000
3. RAS/WAS and UV Building	
a. Structure	$630,000
b. RAS/WAS Pumps	$100,000
c. UV	$340,000
4. Secondary Clarifiers	
a. Concrete	$440,000
b. Equipment	$295,000
c. Equipment Installation	$206,500
5. Site Piping	
a. Total site pipng about 1,800 feet	$450,000
6. SCADA System	$650,000
7. Site Electrical Including Generator	$1,100,000
8. Site Excavation incl. small diameter buried piping	$500,000
9. Conversion of Existing Tankage to Equalization with Pumps	$350,000

City of Mason
Wastewater Treatment Plant

Option B: Oxidation Ditch (Concentric Loops) (continued)

10.	Temporary Reroute of Effluent Pipe during Construction	$250,000
11.	Demolition of Structures, Pipe Abandonment and Connections	$300,000
12.	Contractor General Conditions	$860,000
	Construction Total	$9,463,500
	Contingency, Legal, Engineering, and Fiscal	$2,800,000
	Project Total	$12,263,500

City of Mason
Wastewater Treatment Plant

Option C: Continuous Influent SBR

	Plant Unit	**Cost Estimate**
1.	Primary Clarifiers - Not Required	
2.	Continuous Influent SBR	
	a. Concrete	$1,808,000
	b. Equipment	$830,000
	c. Equipment Installation	$581,000
3.	Blower, RAS/WAS and UV Building	
	a. Structure	$1,050,000
	b. Blowers	- Incl. Item 2
	c. WAS Pumps	$50,000
	d. UV	$1,000,000
4.	Secondary Clarifiers - Not Required	
5.	Site Piping	
	a. Total site pipng about 1,000 feet	$250,000
6.	SCADA System	$600,000
7.	Site Electrical Including Generator	$1,100,000
8.	Site Excavation incl. small diameter buried piping	$500,000
9.	Conversion of Existing Tankage to Equalization with Pumps	$350,000
10.	Temporary Reroute of Effluent Pipe during Construction	$250,000
11.	Demolition of Structures, Pipe Abandonment and Connections	$300,000
12.	Contractor General Conditions	$870,000
	Construction Total	$9,539,000
	Contingency, Legal, Engineering, and Fiscal	$2,900,000
	Project Total	$12,439,000

City of Mason
Wastewater Treatment Plant

Option C: Alternate Continuous Influent SBR

	Plant Unit	Cost Estimate
1.	Primary Clarifiers - Not Required	
2.	AQUA Aerobics SBR	
	a. Concrete	$1,440,000
	b. Equipment	$1,000,000
	c. Equipment Installation	$700,000
	b. Post SBR EQ Basin with Submersible Pumping System	$750,000
3.	Blower, RAS/WAS and UV Building	
	a. Structure	$1,050,000
	b. Blowers	-
	c. WAS Pumps	$50,000
	d. UV	$700,000
4.	Secondary Clarifiers - Not Required	
5.	Site Piping	
	a. Total site pipng about 950 feet	$237,500
6.	SCADA System	$650,000
7.	Site Electrical Including Generator	$1,100,000
8.	Site Excavation incl. small diameter buried piping	$500,000
9.	Conversion of Existing Tankage to Equalization with Pumps	$350,000
10.	Temporary Reroute of Effluent Pipe during Construction	$250,000
11.	Demolition of Structures, Pipe Abandonment and Connections	$300,000
12.	Contractor General Conditions	$910,000
	Construction Total	$9,987,500
	Contingency, Legal, Engineering, and Fiscal	$3,000,000
	Project Total	$12,987,500

City of Mason
Wastewater Treatment Plant

Option D: Membrane Bioreactor

Plant Unit	Cost Estimate
1. Fine Screen incl. Building	$1,000,000
2. MBR	
a. Concrete	$880,000
b. Equipment	$2,500,000
c. Equipment Installation	$1,750,000
3. Blower, RAS/WAS and UV Building	
a. Structure	$1,260,000
b. Blowers	-
c. RAS/WAS Pumps	$50,000
d. UV	$340,000
4. Secondary Clarifiers - Not Required	
5. Site Piping	
a. Total site pipng about 1,100 feet	$275,000
6. SCADA System	$650,000
7. Site Electrical Including Generator	$1,100,000
8. Site Excavation incl. small diameter buried piping	$500,000
9. Conversion of Existing Tankage to Equalization with Pumps	$350,000
10. Demolition of Structures, Pipe Abandonment and Connections	$300,000
11. Contractor General Conditions	$1,100,000
Construction Total	$12,055,000
Contingency, Legal, Engineering, and Fiscal	$3,600,000
Project Total	$15,655,000

City of Mason
Wastewater Treatment Plant

Option E: Moving Bed Bioreactor/IFAS

Plant Unit	**Cost Estimate**
1. Primary Clarifiers	
a. Concrete	$264,000
a. Equipment	$250,000
a. Equipment Installation	$175,000
2. MBBR/IFAS	
a. Concrete	$720,000
b. Equipment	$1,250,000
c. Equipment Installation	$875,000
3. Blower, RAS/WAS and UV Building	
a. Structure	$1,050,000
b. Blowers	- Incl Item 2
c. RAS/WAS Pumps	$100,000
d. UV	$340,000
4. Secondary Clarifiers	
a. Concrete	$440,000
b. Equipment	$295,000
c. Equipment Installation	$206,500
5. Site Piping	
a. Total site pipng about 1500 feet	$375,000
6. SCADA System	$650,000
7. Site Electrical Including Generator	$1,100,000
8. Site Excavation incl. small diameter buried piping	$500,000
9. Conversion of Existing Tankage to Equalization with Pumps	$340,000
10. Demolition of Structures, Pipe Abandonment and Connections	$300,000
11. Contractor General Conditions	$920,000
Construction Total	$10,150,500
Contingency, Legal, Engineering, and Fiscal	$3,000,000
Project Total	$13,150,500

City of Mason
Wastewater Treatment Plant

Tertiary Disc Filters

Plant Unit	**Cost Estimate**
1. Tertiary Filter	
a. Equipment	$285,000
b. Equipment Installation	$199,500
2. Added Area in Blower Building for Tertiary	
a. Structure	$160,000
3. Site Piping	
a. Waste return to Headworks	$36,000
4. SCADA System Integration	$75,000
Construction Total	$755,500
Contingency, Legal, Engineering, and Fiscal	$230,000
Project Total	$985,500