

MEMBRANE BIOREACTOR FOR NUTRIENT REMOVAL IN MUNICIPAL
WASTEWATER

by

Melissa Erin Fraser

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ABSTRACT

The primary objective of this research project was to evaluate membrane bioreactor (MBR) technology in parallel to conventional municipal biological wastewater treatment system design to determine nutrient removal treatment efficacy. A pilot-scale MBR plant was designed, commissioned and operated in parallel to the full-scale sequencing batch reactor (SBR) treatment system at the Aerotech Wastewater Treatment Facility (AWWTF) in Halifax, Nova Scotia. Samples were collected from the effluent of both treatment processes over a 3-month period (June to August 2016) and were analyzed for biochemical oxygen demand (BOD), total suspended solids (TSS), chemical oxygen demand (COD), ammonia (NH_3) and total phosphorus. It was found that the full-scale SBR plant was able to meet discharge regulations for all water quality parameters examined consistently except for total phosphorus. The pilot-scale MBR system was able to meet the discharge regulations for BOD and TSS consistently, and ammonia once stable operating conditions were met, but failed to meet the total phosphorus discharge water quality goals under the experimental set points and during the study period. A bench-scale coagulation study focusing on phosphorus removal was completed to complement the pilot MBR plant. Under the conditions tested, phosphorus removal targets were met, along with TSS regulations. Factors affecting the coagulation process for three coagulants were also determined throughout a factorial design. For phosphorus removal, dose and filter type were statistically significant for alum. No factors were found to be significant for phosphorus removal for PACl or ferric sulphate.

LIST OF ABBREVIATIONS AND SYMBOLS USED

| | |
|---|--|
| Al(OH) ₃ | Aluminum Hydroxide |
| Al(PO ₄) | Aluminum Phosphate |
| Al ₂ (SO ₄) ³ | Aluminum Sulphate (Alum) |
| AOB | Ammonia Oxidizing Bacteria |
| ATP | Adenosine Triphosphate |
| AWWTF | Aerotech Wastewater Treatment Facility |
| BNR | Biological Nutrient Removal |
| BOD | Biological Oxygen Demand |
| CCME | Canadian Council of Ministers of the Environment |
| CIP | Clean In Place |
| COD | Chemical Oxygen Demand |
| DO | Dissolved Oxygen |
| FeCl ₂ | Ferric Chloride |
| Fe(OH) ₃ | Ferric Hydroxide |
| Fe(PO ₄) | Ferric Phosphate |
| Fe ₂ (SO ₄) ³ | Ferric Sulphate |
| HCl | Hypochloride |
| HRT | Hydraulic Retention Time |
| MBR | Membrane Bioreactor |
| MF | Microfiltration |
| µg | Microgram |
| mg | Milligram |
| mL | Milliliter |
| MLSS | Mixed Liquor Suspended Solids |
| N ₂ | Nitrogen Gas |
| NaOCl | Sodium Hypochlorite |
| NaOH | Sodium Hydroxide |
| NF | Nanofiltration |
| NH ₃ | Ammonia |
| NH ₄ ⁺ | Ammonium Ion |

| | |
|------------------------------|-----------------------------------|
| NO ₂ ⁻ | Nitrite |
| NO ₃ ⁻ | Nitrate |
| NOB | Nitrite Oxidizing Bacteria |
| NOM | Natural Organic Matter |
| PACl | Polyaluminum Chloride |
| PAO | Phosphorus Accumulating Organisms |
| PFCl | Polyferric Chloride |
| PFS | Polyferric Sulphate |
| RO | Reverse Osmosis |
| SBR | Sequencing Batch Reactor |
| scfm | Standard Cubic Feet per Minute |
| SRT | Solids Retention Time |
| TMP | Transmembrane Pressure |
| TP | Total Phosphorus |
| TRC | Total Residual Chlorine |
| TSS | Total Suspended Solids |
| UF | Ultrafiltration |
| UV | Ultraviolet |
| VFA | Volatile Fatty Acids |

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Chapter 1 INTRODUCTION

As protection of water supplies and the environment becomes more relevant, more stringent regulations will be put in place to protect the receiving waters of municipal wastewater treatment plants. Municipalities, utilities and government will be looking for ways to upgrade or expand existing facilities to meet these new effluent regulations. One set of parameters of concern in effluent discharge that are regulated by the Canadian Council of Ministers of the Environment (CCME) are total suspended solids (TSS), biological oxygen demand (BOD) and total residual chlorine (TRC). The regulations for TSS and BOD are 25 mg/L while the TRC regulation is 0.02mg/L (CCME, 2009). Wastewater treatment facilities that discharge into sensitive waterways may also be concerned about nutrients such as nitrogen and phosphorus that can cause eutrophication, which can lead to changes in the environment.

Membrane technology has been used in both drinking water and wastewater treatment and is one technology that could be used to improve wastewater effluent qualities as it can provide a physical barrier for solids removal (Wintgen et al, 2004). In the past, membranes have been commonly used in wastewater treatment as a way to improve effluent quality for either non-potable or potable reuse in areas that have water supply issues. (Wintgens et al, 2004). Membrane technology has also been introduced to wastewater treatment trains to replace conventional filtration, to increase plant capacity and to minimize facility footprint. It can be used to treat a variety of wastewater, from municipal to industrial to agricultural. The use of membrane technology in Canada is growing, with established facilities in western and central Canada and interest increasing in Atlantic Canada (Yust, 2007).

Membrane bioreactors (MBR) are one way membranes are integrated into wastewater treatment systems. MBR is a suspended growth activated sludge system with a membrane either immersed in the biological treatment or added after biological treatment. The activated sludge system works to break down the organic matter, and the membrane provides solid separation. One of the benefits of a MBR system is that the process can be operated at higher biomass concentrations compared to conventional activated sludge processes where conventional settling would not be able to handle the increased biomass loadings. MBRs can operate with longer solids retention times compared to conventional

activated sludge systems as the sludge can be wasted at a slower rate than conventional treatment.

With the benefit of longer holding times and solids retention times, MBRs are able to achieve high nutrient removal. Previous studies have shown that MBR plants can reliably achieve effluent total nitrogen (TN) concentrations of 3 mg/L and total phosphorus (TP) of 0.2 mg/L (Cote, 2004; Kubin et al, 2002; Fleischer et al, 2005).

The primary objective of this project was to evaluate membrane bioreactor (MBR) technology at pilot-scale in parallel to a full-scale conventional municipal biological wastewater treatment system design to determine nutrient removal treatment efficacy of the two plant designs. Specific objectives of the research were to:

1. Design, construct and commission a Membrane Bioreactor (MBR) pilot plant in parallel to a full-scale Sequencing Batch Reactor (SBR) wastewater treatment facility.
2. Evaluate the MBR pilot plant in parallel to the full-scale SBR plant with regards to TSS, BOD, COD, ammonia and total phosphorus removal efficacy.
3. Identify operating parameters that impact phosphorus removal efficacy in wastewater treated with coagulation through the use of factorial design of bench-scale jar tests.

Chapter 2 LITERATURE REVIEW

As membrane technology has been used in wastewater treatment in areas with water supply shortages or discharge points with sensitive receiving waters for the past few decades, there is a growing base of knowledge on the use of the technology and the results achievable. This literature review will first discuss a general overview of municipal wastewater treatment. The physical properties of nitrogen and phosphorus will then be discussed, along with nutrient removal in wastewater treatment, including coagulation. An overview of membrane technology is presented, followed by the use of membrane bioreactors for wastewater treatment.

2.1 Municipal Wastewater Treatment

Municipal wastewater is made up of sanitary sewage from households and commercial sources that can also be combined with storm water. Sanitary sewage is made up of human waste from toilets, also known as blackwater and greywater which includes water from sinks, showers etc. The strength of the wastewater depends on what types of businesses or institutions (i.e. hospitals, universities) contributing to the system. Table 2-1 shows typical raw municipal wastewater characteristics for high, medium and low strength wastewater.

Table 2-1 Typical Municipal Wastewater Characteristics (Adapted from Metcalf and Eddy, 2003)

| Parameter | High | Medium | Low |
|-------------------------------|-------------|---------------|------------|
| COD (mg/L) | 1200 | 750 | 500 |
| BOD (mg/L) | 560 | 350 | 230 |
| TKN (mg/L) | 100 | 60 | 30 |
| NH3 (mg/L) | 75 | 45 | 20 |
| TP (mg/L) | 25 | 15 | 6 |
| TSS (mg/L) | 600 | 400 | 250 |
| pH | 8.0 | 7.5 | 7.0 |
| Alkalinity eqv/m ³ | 7 | 4 | 1 |

In Canada, more than 3 trillion litres of wastewater is treated and then discharged into receiving waters (Canadian Council of the Ministers of the Environment, 2009). According to Environment Canada, in 2009, 69% of Canadians had access to secondary wastewater treatment or higher. Primary wastewater treatment consists of screening of gross solids such as objects, grit and rags. Floating and settleable solids are then removed through a process, often sedimentation, that also may involve the addition of chemicals to aid in the removal. Some organic matter may get removed (Metcalf and Eddy, 2003). Secondary treatment involves the removal of most of the biodegradable organic matter in the wastewater following primary treatment through biological and chemical processes. Most often, secondary treatment involves disinfection as well. Tertiary treatment is used to target specific parameters when stringent regulations are needed to be met. These treatments can be physical, biological or chemical and are commonly applied in water reclamation or reuse systems.

2.2 Conventional Secondary Wastewater Treatment

Conventional secondary wastewater treatment facilities are designed for removal of biodegradable organics and solids. This is achieved with the addition of biological treatment to the physical and/or chemical primary treatment. The biological treatments can be divided into two groups; suspended growth processes and attached growth

processes. Attached growth processes have the biomass form a film on a surface which is then used to treat the wastewater. Examples include trickling filters and rotating contactors.

Suspended growth processes have the biomass suspended in an aeration tank where oxygen is added to provide an aerobic environment and to keep the biomass suspended in the wastewater. These conditions help the microorganisms to break down the organic matter. Examples of suspended growth processes are waste activated sludge (WAS) and sequencing batch reactors (SBR) systems.

Following the biological treatment, in a conventional wastewater treatment process, a clarification stage occurs, usually sedimentation. This allows for separation of the biomass and the supernatant, and for the return of the biomass to the aeration tank. Filtration may occur after sedimentation depending on the treatment train of the plant. The wastewater effluent is then disinfected prior to discharge to inactivate any microorganism remaining. Common processes for disinfection include chlorination or UV disinfection.

2.2.1 Secondary Wastewater Treatment Operating Parameters

There are many parameters that are considered when operating a biological treatment system. Biomass in biological wastewater treatment is the mass of microorganisms available for treatment of the wastewater. These microorganisms consume organic matter and nutrients that is found in the wastewater and these constituents are called substrate. The concentration of biomass is used to calculate the solids retention time (SRT) and hydraulic retention time (HRT) which are important to ensure adequate treatment is achieved.

Solids retention time is the amount of time that the activated sludge is in system and in contact with the wastewater/substrate. It is calculated by dividing the mass of solids in the aeration tank by the solids removal rate from the system. This can be seen in

The solids retention time can vary depending on the type of treatment being used, effluent quality and temperature. (Metcalf and Eddy, 2003). The biomass will be separated and recycled through the plant, so it will treat new substrate several times before leaving the treatment train, resulting in longer SRTs (WEF, 2010).

(Metcalf & Eddy 2003).

$$SRT = \Theta_x = \frac{VX}{(Q-Q_w)X_e + Q_w X_r} \quad (\text{Equation 2-1})$$

where SRT= solids retention time (days)

V= volume of tank (m³)

X= concentration of biomass in tank g VSS/m³

Q= influent flowrate (m³/day)

Q_w= waste sludge flowrate (m³/day)

X_e=concentration of biomass in the effluent g VSS/m³

X_r= concentration of biomass in the return activated sludge g VSS/m³

The solids retention time can vary depending on the type of treatment being used, effluent quality and temperature. (Metcalf and Eddy, 2003). The biomass will be separated and recycled through the plant, so it will treat new substrate several times before leaving the treatment train, resulting in longer SRTs (WEF, 2010).

Hydraulic retention time (or also known as the aeration period) is the time that the wastewater is in contact with biomass in the aeration tank. The HRT is less than the SRT since the wastewater will only flow through the aeration tank once (typically measured in hours). It is calculated to ensure that the substrate is in contact with the biomass long enough for the organic matter and nutrients to be consumed. (WEF, 2010) **Error! Reference source not found.** shows the formula for HRT (Metcalf & Eddy 2003).

$$HRT = \Theta = \frac{V}{Q} \quad (\text{Equation 2-2})$$

where HRT = hydraulic retention time (hours or days)

V= volume of tank (m³)

Q= influent flowrate (m³/day)

Mixed liquor suspended solids (MLSS) is another parameter used in biological treatment. It is the concentration of suspended solids in the aeration tank. It is made up of a combination of the biomass in the tank and solids from the influent wastewater. MLSS

is typically reported in mg/L. It is important to have adequate MLSS levels in the aeration tank so that the organic matter in the wastewater is removed.

Biochemical Oxygen Demand (BOD₅) is used to measure organic matter in a water. BOD is one of the most commonly used parameters in both surface waters and wastewaters. BOD testing measures the amount of dissolved oxygen that is used by microorganisms to break down organic matter through biochemical oxidation. The test is conducted over 5 days (BOD₅). The consequence of a high BOD in a water source is that there is less dissolved oxygen for the other organisms in the water and then they become stressed and die (US EPA, 2012)

2.3 Nutrient Removal

Eutrophication and algal blooms can be caused by excess nutrients in a body of water. One source of excess nutrients can be wastewater effluent discharge that did not receive adequate nutrient removal. While all nutrient removal is important for wastewater effluent, limiting the release of phosphorus is important because it is often the limiting nutrient for eutrophication (reducing the phosphorus released limits the amount of eutrophication or algal blooms that can occur) (Metcalf & Eddy, 2003). It is also important to limit the nitrogen that enters the water supply, as it also contributes to eutrophication and can cause other issues on its own. Algal blooms can be harmful to the ecosystem by depleting dissolved oxygen in the water and can cause issues for drinking water plants that receive treat water containing algae.

Nitrogen and phosphorus can be removed biologically during wastewater treatment and this is often referred to as biological nutrient removal (BNR). BNR treatment involves the addition of anoxic or anaerobic tanks to the treatment trains already containing aerated tanks to provide conditions where the nutrients can be transformed or removed. BNR can be achieved in both suspended growth or attached growth treatment trains (Metcalf and Eddy, 2003). Phosphorus can also be removed through chemical coagulation, which can be beneficial as the conditions for biological nitrogen and phosphorus removal are not always achievable concurrently.

2.3.1 Forms of Nitrogen

Nitrogen found naturally in water supplies most often originates from animals and plants as it is a key nutrient for biological growth. Waterways can be polluted with nutrients such as nitrogen through agricultural runoff, stormwater and from wastewater effluent. In municipal wastewater, urea and fecal matter make up the majority of nitrogen found. By the time it gets to the wastewater treatment plant, it is usually converted to ammonia due to the anaerobic conditions of the sewage collection systems. Food waste and other industrial processes can also contribute to the nitrogen content in municipal wastewater influent (US EPA, 2015).

Nitrogen in soil/water can be found in several forms. Organic nitrogen, is made up of amino acids, amino sugars and proteins. It can be in several forms; particulate or soluble form, or biodegradable or non-biodegradable or combinations of the two. The nitrogen that makes up the organic nitrogen is easily converted to ammonium (Metcalf and Eddy, 2003). Ammonia nitrogen can be found as either ammonia gas or as the ammonium ion, depending on the pH of the water. Ammonium ion is favoured at pH below 7 while ammonia gas is more favourable above pH of 7. Nitrites in wastewater are found at lower concentrations as they are not very stable and are easily converted to nitrates. But nitrites can be toxic to fish at higher concentrations and can oxidize chlorine resulting in higher chlorine dosages if in effluent being disinfected by chlorine. The most oxidized form of nitrogen in wastewater is nitrate. It can be harmful to waterways as it can be used for the formation of proteins by animals and plants which could lead to harmful growth in the water system (such as eutrophication) (Metcalf and Eddy, 2003). Nitrogen is also found abundantly in the atmosphere as a gas.

2.3.2 Nitrogen Removal in Wastewater

Nitrogen is removed from wastewater through nitrification and denitrification. Nitrification is the process where ammonia (NH_3) is oxidized and is converted to nitrite (NO_2^-) and then to nitrate (NO_3^-). Autotrophic bacteria, known as nitrifiers are used in nitrification. Nitrosomonas are the ammonia oxidizing bacteria (AOBs) while nitrobacter are the nitrite oxidizing bacteria (NOBs). The conversion of the ammonia is completed through various biological processes in the bacteria and provides energy for the growth of

the bacteria. Aerated conditions are needed for these bacteria to perform nitrification and it often takes place simultaneously with carbonaceous BOD removal. Alkalinity is consumed during nitrification as the autotrophic bacteria can use it as a carbon source. The autotrophic bacteria require longer to mature and become stable in the treatment process, so often it is these bacteria that dictate the systems solids retention time (SRT). The temperature of the wastewater, pH and dissolved oxygen levels (DO) also play important roles in the growth and stabilization of the bacteria (Metcalf and Eddy, 2003).

When ammonia is converted to nitrate, it reduces the amount of ammonia that enters the receiving waters, which is toxic to fish and affects DO levels in the water. But the nitrogen is still in the effluent as nitrate. Nitrates can still contribute to eutrophication and have also been linked to diseases such as Blue Baby Syndrome. Denitrification is the process of converting nitrate to nitrogen gas (N_2). N_2 makes up a large portion of the atmosphere, so it can be released from the wastewater treatment train without major issues.

Denitrification takes place in anoxic conditions and involves heterotrophic bacteria to reduce the nitrate. These heterotrophic bacteria use the oxygen that is attached to the nitrates for respiration as anoxic conditions does not have free dissolved oxygen for the bacteria to use. Pre-anoxic and post-anoxic tanks can be introduced into the treatment process to facilitate denitrification. As the bacteria that reduce the nitrate are heterotrophic, a carbon source is required. During denitrification, 1 equivalent of alkalinity is produced per equivalent of nitrate that is reduced. Pre-anoxic tanks can use the incoming BOD as a carbon source as it has not reached the aeration tank at that point and can increase the alkalinity in the aeration tank for the nitrification processes. Post-anoxic tanks, which would follow an aeration tank, may need a supplementary carbon source to facilitate denitrification. During the conversion of nitrate to nitrogen gas, energy is produced that the bacteria can use for growth, along with a base (OH^-) which can increase the alkalinity in the wastewater.

2.3.3 Forms of Phosphorus

Phosphorus is found in wastewater in the form of phosphates. Phosphates can be found in three different forms; orthophosphates, condensed phosphates and organically

bound phosphates. They can be measured as dissolved or particulate phosphorus. Phosphates can enter a water supply through runoff, domestic and industrial waste, cleaning agents and naturally through erosion, decomposition of rocks and from wildlife. Certain condensed phosphates are used for corrosion control in drinking water distribution systems. Low levels of phosphorus are needed for most plants and animals, but too much in a water supply will result in eutrophication. (APHA, AWWA & WEF, 1998)

Orthophosphates can be found in the form of phosphate (PO_3^-), hydrogen phosphate (HPO_4^{2-}), dihydrogen phosphate (H_2PO_4^-) or phosphoric acid (H_3PO_4) as examples. They occur naturally in uncontaminated water supplies at low levels, but can also be found in runoff from agricultural lands and in fertilizers. Orthophosphates can be used for biological metabolism without any further conversions and is the form condensed phosphates and organic phosphates are converted to during hydrolysis. It is the form that can be readily used for biological metabolism and is used during eutrophication. They are often referred to as phosphates or reactive phosphorus. The structure of orthophosphates is one phosphorus atom bonded to 4 oxygen atoms. Orthophosphates are the form of phosphorus that is targeted for removal through coagulation. (Metcalf & Eddy, 2003).

Condensed phosphates are made up of orthophosphate molecules that share covalent bonds. They are formed when the orthophosphate radical is dehydrated. Metaphosphates, pyrophosphates and polyphosphates make up the different chemical structures of the condensed phosphates. Condensed phosphates have at least two phosphorus and oxygen atoms and can also have hydrogen atoms in their structure (Metcalf and Eddy, 2003). The condensed phosphates undergo hydrolysis to convert back to orthophosphates when in aqueous solutions, but this is a slow process.

Organic phosphates, also known as organically bound phosphates, are phosphates that are bonded to organic compounds. They often enter wastewater through body waste or food residue (APHA, AWWA & WEF, 1998) or can be formed during biological treatment processes. Adenosine triphosphate (ATP) is one form of organic phosphates and are used to transfer energy between cells. Organic phosphates do not break down as easily as the condensed phosphates.

When regulated in the wastewater effluent, phosphorus is either measured as orthophosphate or total phosphorus. Due to the nature of polyphosphates and organic phosphates needing to be further broken down to orthophosphates, they are not included in the phosphorus measurement when measuring it based on orthophosphates. Total phosphorus measurements require that the sample is digested with sulfuric acid and heat and has a strong oxidant added before being measured. This ensures that the condensed phosphates and organic phosphates are broken down to orthophosphates and in the case of organic phosphates, are separated from the organic compounds. Total phosphorus measurements then reflect the total amount of phosphorus in the sample (HACH, 2013)

2.3.4 Biological Phosphorus Removal

Phosphorus can be removed biologically from wastewater. Biological removal of phosphorus is achieved by increasing the amount of phosphorus that microorganisms can uptake in their cells. It is often referred to as enhanced biological phosphorus removal (EBPR). In this process, the phosphorus is accumulated in the biomass of the plant and then separated from the effluent through sedimentation, filtration or membrane filtration.

The organisms that are involved in biological phosphorus removal are called phosphorus accumulating organisms (PAOs). Initially the PAOs are in an anaerobic condition where volatile fatty acids (VFAs) are present, which the PAOs can use as a food source. The PAOs require an energy source to be able to transform the VFAs so that the VFAs be stored in the PAOs cells. The energy needed for this transformation is gained by breaking phosphorus bonds in polyphosphates (condensed phosphates) or in organically bound phosphates causing the release of orthophosphates. The release of orthophosphates is necessary to prepare the organisms to uptake phosphorus in the aerobic conditions. When there are more VFAs than energy available for the PAOs to transform the VFAs, the PAOs become stressed and start luxury phosphorus uptake, which includes accumulating the orthophosphates previously released to make up for the lack of energy. The biomass then contains the phosphorus that is to be removed from the wastewater and can be separated from the treated effluent through sedimentation or filtration.

Soluble phosphorus that may pass through the effluent during biological treatment will typically be in the form of orthophosphates, which can then be removed through chemical precipitation if needed to meet discharge regulations.

2.3.5 Chemical Coagulation for Phosphorus Removal

Once the phosphorus is in the orthophosphate form, it can be removed from the wastewater through chemical coagulation. It is the process of adding a chemical to water to destabilize colloidal particles so that flocs can form through flocculation. The flocs are then removed from the water through sedimentation or filtration (Metcalf and Eddy, 2003). In municipal wastewater treatment, coagulation is typically used for reducing biological oxygen demand (BOD) and total suspended solids (TSS), but it can also be used for phosphorus removal.

Coagulant choice may vary between wastewater treatment facilities. Some factors include; wastewater natural alkalinity levels, operating wastewater temperature, operator's preference, compatibility with other treatment processes and cost (Metcalf and Eddy, 2003). The chemicals that are used for coagulation in wastewater are typically aluminum or iron based salts. The most commonly used coagulants are aluminum sulphate (Alum) ($\text{Al}_2(\text{SO}_4)_3$), ferric sulphate ($\text{Fe}_2(\text{SO}_4)_3$) and ferric chloride (FeCl_3). Coagulants that are pre-polymerized have been gaining popularity, such as polyaluminum chloride (PACl) or polyferric chloride (PFCI) (Yang et al, 2010) as they do not have a great effect on the pH and other water quality parameters as compared to the other coagulants. Iron based coagulants tend to perform better at pH around 5.0 while aluminum based coagulants perform better around pH 6.0-6.5 for phosphorus removal (Citulski et al, 2009). Alum is commonly used as a coagulant as many plant operators are familiar and comfortable with working with the chemical. Comparing the cost of the coagulants, Alum is typically more cost-effective compared to other coagulants (Yang et al, 2010). Calcium can also be used as a coagulant for wastewater treatment, but not commonly used as it is more sensitive to pH changes and is more difficult to maintain ideal operating conditions (Yang et al, 2010).

When an aluminum or iron based salt is added to the wastewater, the Al^{3+} or Fe^{3+} ions hydrolyze to produce different hydroxyl species. Wastewater conditions, such as pH,

dictate which species will form. In the range of pH 6 to 8, the hydrolysis occurs rapidly which makes it complicated to control which species are formed. pH control during coagulation is important to help ensure that the species needed for precipitation are formed so that the suspended solids are removed (Jiang and Graham, 1998).

2.3.6 Phosphorus Removal Mechanisms with Chemical Coagulation

There are three main mechanisms involved with contaminant removal in coagulation processes: charge neutralization, sweep floc (enmeshment) or adsorption. The pH of the wastewater being treated will dictate the mechanism that removes the target colloidal particles from the water.

Charge neutralization uses the $\text{Al}^{3+}/\text{Fe}^{3+}$ ions and their respective hydrolyzed species to alter the charge of the negatively charged colloidal particles so that the repulsion forces between the particles are reduced and that flocs can form. Charge neutralization occurs at a $\text{pH} < 6.5$ and at a lower coagulant dose. Because of this, charge neutralization is more sensitive to changes in water conditions and can be effected by overdosing of coagulant (Jiang and Graham, 1998).

Sweep floc or enmeshment occurs when enough coagulant is added so that insoluble precipitates are formed. These precipitates will form larger flocs that can settle more readily and as they sweep through the water the colloidal particles become enmeshed in the floc and are also settled out. Sweep floc is not as greatly affected by changes in water conditions or coagulant dose as charge neutralization is. Sweep floc occurs around a neutral pH and higher coagulant dose (Jiang and Graham, 1998).

Adsorption is another mechanism for removal by coagulation/precipitation and it is effective at removing natural organic matter (NOM) from the water source. Because of this, NOM may increase the coagulant dose required as the humic substances will compete with other constituents for the coagulant and use up some of the coagulant needed for target suspended solids.

Initially, it was believed that phosphorus was removed through coagulation by the formation of $\text{AlPO}_4(\text{s})$ and $\text{FePO}_4(\text{s})$ which are stable precipitates of phosphate in the pH range of 5-7 (Jiang and Graham, 1998). $\text{AlPO}_4(\text{s})$ is stable and most soluble around pH of 6, while $\text{FePO}_4(\text{s})$ is stable and most soluble at a pH of 5 which can be seen in Figure

2-1Error! Reference source not found.. But because the hydrolysis of the metals occurs so rapidly, it is not likely that Al^{3+} or Fe^{3+} will be found in the water to form the precipitates. It is more likely that the coagulation mechanisms that are used are either the formation of Al/Fe-hydroxo-phosphate complexes ($\text{Me}(\text{OH})_{3-x}(\text{PO}_4)_x$) or the adsorption of PO_4 ions onto the hydroxyl species that are formed (predominantly $\text{Al}(\text{OH})_3$ or $\text{Fe}(\text{OH})_3$) (Metcalf and Eddy,2003).

In a study performed by Yang et al (2010), which involved removal of phosphorus by coagulation from secondary treated municipal wastewater, it was shown that phosphorus was removed through both adsorption and precipitation of the hydroxyl-phosphate complexes, for both aluminum and ferric based coagulants. The hydroxyl species that were involved in precipitation of the phosphorus were amorphous $\text{Al}(\text{OH})_3$ and $\text{Fe}(\text{OH})_3$ respectively. For adsorption of phosphorus, they showed that $\text{Fe}(\text{OH})^+$, AlOH^{2+} and $\text{Al}(\text{OH})_2^+$ were involved. (Yang et al, 2010)

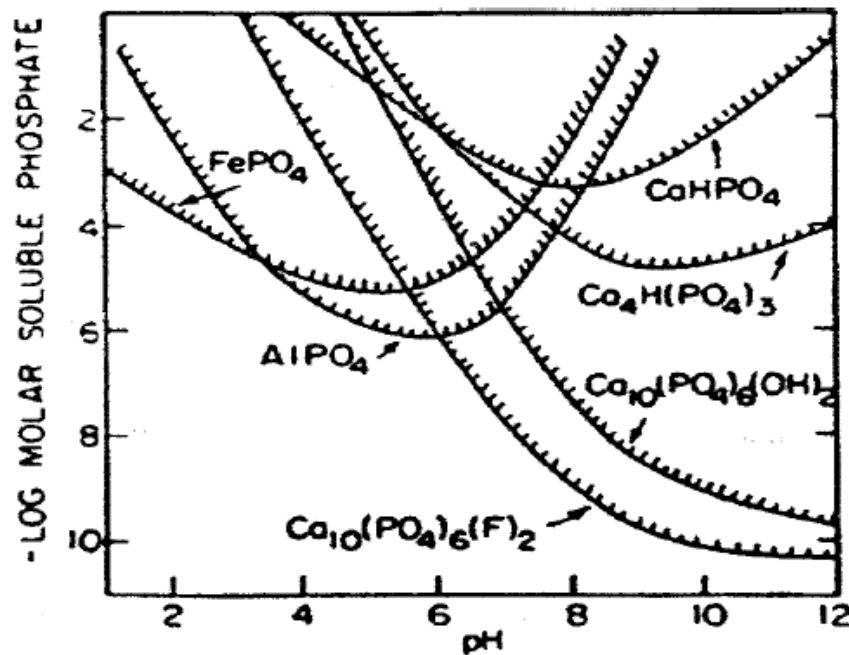


Figure 2-1 Solubility of metal phosphates (from Strumm and Morgan, 1996)

Yang et al, in their 2010 study, also compared four different coagulants for phosphorus removal in municipal wastewater through conducting jar tests. The coagulants that were compared were alum, ferric chloride, PACl and polyferric sulphate

(PFS). The coagulants had phosphorus removal percentages between 82-96%, with Alum providing the highest removal rate.

Ismail et al, (2012) also compared the effectiveness of several coagulants on phosphorus removal from raw municipal wastewater. The coagulants considered were alum, ferrous sulphate and ferrous chloride. Testing was completed at both bench-scale (through jar testing) and at pilot-scale. It was determined that a coagulant dose of 60 mg/L alum was the optimal dose and at pilot-scale it was able to achieve a removal percentage of 76% for total phosphorus.

Raw wastewater to be treated by a membrane bioreactor was studied by Kim et al (2013) in regards to phosphorus removal. They conducted jar tests using PACl and ferric chloride as coagulants and raw wastewater from 5 different municipal wastewater treatment plants. Ferric chloride was more effective at removing total phosphorus than PACl, but when the samples were filtered through 0.45 μ m pore size filter prior to coagulation, PACl was shown to be more effective. This result may show that PACl is more effective when the particles to be removed are smaller than 1 μ m while ferric chloride may be more effective with particles larger than 1 μ m.

Coagulation prior to membrane filtration in wastewater treatment plants has been shown to increase removal of phosphorus by the membranes. Citulski et al (2009) focused on using in-line coagulation as pretreatment for ultrafiltration (UF) membranes for tertiary wastewater treatment at pilot scale. The influent phosphorus levels were 5 mg/L with an effluent target of 0.3 mg/L. The coagulant doses of 40 mg/L and 70 mg/L for ferric chloride and Alum respectively, was able to achieve less than the target 0.3 mg/L phosphorus in the final treated effluent. Both coagulant doses found were lower than the ferrous chloride dose that had been used at full scale.

2.3.7 Chemical Coagulation Addition Points

Chemicals for coagulation can be added to the wastewater treatment train at various locations, depending on the target parameter to be removed. For phosphorus removal, several locations are typically used, either on their own or in combination. When coagulant is added to raw wastewater prior to primary treatment, it is referred to as pre-precipitation and the phosphorus is then removed through primary clarification.

Chemicals for coagulation can also be added to the activated sludge of the biological treatment or to the effluent of the biological treatment prior to secondary clarification and this is referred to as co-precipitation. The phosphorus is removed along with waste biological sludge. Co-precipitation can also help to slightly improve nitrogen levels as the nitrogen may get captured in the flocs that form. Excess chemical that is added to the effluent of the aeration tank will precipitate and be incorporated into the MLSS of the aeration basin or secondary clarifier. This MLSS will be brought back to the front of the treatment train as recycled activated sludge (RAS) and will be available to react with phosphorus in the raw wastewater that could be removed during primary clarification (WEFNET, 2007). The final location where coagulation chemicals can be added is after secondary sedimentation and this is called post-precipitation. When chemical is added here, it is typically to achieve strict phosphorus concentrations less than 0.1 mg/L. Post-precipitation requires further filtration of the effluent to remove the precipitates, and the tertiary treatment usually has a higher chemical and operational cost compared to the other locations. Depending on the raw wastewater being treated and the size of the plant, some of the coagulant dosing locations may be used in combination to achieve the optimal phosphorus removal.

2.4 Sequencing Batch Reactor (SBR) Systems

Sequencing batch reactors are a suspended growth process that can be used for secondary treatment of wastewater. A sequencing batch reactor uses all the same processes as an activated sludge conventional treatment train but they are sequentially completed in one tank. An SBR tank is filled as required by flow into the plant and then is operated as a batch reactor. The frequency that the SBRs are operated can fluctuate based on heavy or light flow into the plant and the timing of each stage of operation can be adjusted easier than conventional activated sludge plants (Irvine et al 1989).

SBRs operate in five stages; fill, react, settle, decant and idle. Figure 2-2 shows a schematic of an SBR plant.

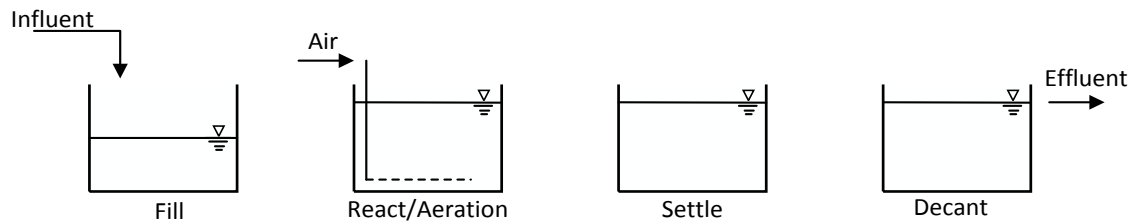


Figure 2-2 Sequencing Batch Reactor Schematic (Adapted from Metcalf and Eddy, 2003)

During the fill phase, wastewater is added to biomass that is already in the SBR tank. The wastewater may be added as a static fill, mixed fill (using mixers but no aeration while the wastewater is added) or as an aerated fill (aerators on as the wastewater is being added). The type of addition is based on the treatment operations that are being used (i.e., if denitrification is required, then a mixed fill may be used to provide an anoxic stage). Once the tank has been filled, the react phase starts. During the react phase, no additional wastewater is added to the tank, and typically aeration and mixing takes place. Most carbonaceous BOD can be removed during the react phase. A longer react phase can also allow for nitrification to continue (New England Interstate Water Pollution Control Commission, 2005). After the react phase, settling occurs, where the activated sludge is settled to the bottom of the tank. Decanting takes place to separate the treated wastewater and the activated sludge so the wastewater can continue along the treatment train. If there is not enough flow to fill the tanks after a decant, the SBR tank may go into an idle phase until a fill phase can occur.

Nitrogen removal can be achieved in SBR during several of the phases. An anoxic state can be created by mixing during the fill stage to help promote nitrate removal. Adjusting the fill time and having sufficient BOD in the influent wastewater can also help to increase nitrate removal from the remaining mixed liquor suspended solids (MLSS) after the settle and decant phases. SBR facilities have been able to achieve effluent $\text{NO}_3\text{-N}$ concentrations of less than 5 mg/L (Metcalf & Eddy, 2003). Fernandes et al (2013) showed that a full scale SBR plant was able to achieve removal rates of 83% for COD, 60% for NH_4^+ and 70% for TSS. Yuan et al (2016) showed that 82.9% total nitrogen removal and 96% phosphorus removal were achievable with an SBR system.

2.5 Membrane Technology

Membrane filtration is a physical process for separating colloidal and particulate solids from a liquid, resulting in permeate (i.e., filtered water) and a concentrate (i.e., waste liquid stream). Membranes are a semi-permeable surface allowing certain particles through while retaining others. Most membranes that are used in wastewater treatment are made of an organic material such as polypropylene, cellulose or thin film composites (Metcalf and Eddy, 2003). Flat sheets, hollow fibers or tubular membranes are typically used in commercial operations and their use can depend on cost, quality of wastewater to be treated and expected membrane fouling.

Membranes are typically characterized by the pore size of the membranes, the size and charge of the retained particles and the pressure that is required to push the permeate through the membrane. Based on these characteristics, there are four main membrane types; microfiltration (MF), ultrafiltration (UF), nanofiltration (NF) and reverse osmosis (RO). Microfiltration and ultrafiltration membranes are considered low-pressure membranes while nanofiltration and reverse osmosis are considered high-pressure membranes. MF and UF membranes are commonly used for MBR plants while NF and RO membranes are used in wastewater systems for tertiary treatment to achieve higher effluent quality.

MF membranes have the largest pore size of all the membranes with pores varying between 0.1 to 10 μm and are typically operated at a pressure between 0.1 to 2 bar (10 to 200kPa) (Van der Bruggen et al, 2003). Because of this larger pore size, MF membranes are the most permeable and require less pressure to pass feed water through the surface of the membrane. MF membranes use a sieving mechanism to separate the permeate from concentrate.

UF membranes have smaller pore size openings than the MF membranes with typical pore size varying between 0.002 and 0.2 μm . UF membranes operate at a pressure range of 0.1 to 5 bars (10 to 500 kPa) (Van der Bruggen et al, 2003). Because the pores are smaller than those of MF, a higher pressure is required to have the same flow as a MF membrane. UF membranes can remove the same materials as a MF membrane along with being able to remove smaller particles.

In wastewater treatment, MF and UF membranes can be used as an alternative to the sedimentation tank and filter in a conventional wastewater treatment facility or as a MBR plant. UF membranes can also be used as a polishing step for water reclamation.

Nanofiltration (NF) membranes have pore sizes that vary between 0.0005 to 0.002 μm . This allows the NF membranes to remove multivalent ions and relatively small organic molecules (Van der Bruggen et al, 2003) along with the particles that are removed by MF and UF membranes. Reverse osmosis membranes are very dense and do not have defined pore sizes ($< 0.0005\mu\text{m}$). This allows for them to be used to remove monovalent ions from the feed stream. RO systems are commonly used for desalination or if ultrapure water is required (for some industries such as electronics). RO require high pressures and energy consumptions to produce permeate from the feed stream since it is not very permeable. The mechanism that is used for removal is a solution-diffusion mechanism. Both NF and RO systems are considered high-pressure membrane systems.

2.5.1 Membrane Operating Parameters

Transmembrane (TMP) pressure and membrane flux are two parameters that are used for the operation of a membrane system. TMP is the pressure that is required to pass water through the surface of a membrane, separating the feed stream into permeate and concentrate. It is also the pressure gradient of the membrane. It can be calculated using the formula shown in equation 2.3, where P_f is the pressure on the feedwater side of the membrane and P_p is the pressure on the filtrate side of the membrane.

$$TMP = P_f - P_p \quad (\text{Equation 2-3})$$

Permeate flux is the ratio of the flowrate of the permeate to the surface area of the membrane. Equation 2.4 shows the formula for permeate flux, where Q_p is the permeate flow rate and A is the membrane surface area. The units typically used for permeate flux are L/hr/m^2 (Lmh) or gal/d/ft^2 (gfd).

$$J = \frac{Q_p}{A} \quad (\text{Equation 2.4})$$

The membrane flux can be affected by several factors in a MBR plant, including Specific flux, also referred to as permeability, is the ratio of the permeate flux to the

TMP. It can be used to monitor the operation of the membrane and to determine if the membrane needs cleaning or maintenance.

Membranes can be operated with either constant TMP or constant flux. For constant TMP, the permeate flow will decrease as the membrane becomes fouled with the particles being removed from the wastewater. If the membrane treatment is not the final stage of treatment (which is the case in most scenarios) changes in the flow could affect other processes along the treatment train. Constant TMP membrane set ups are can be monitored by the permeate flux. Membranes that are operated with constant flux will see increases in TMP through the treatment process with a constant volume of wastewater being treated and will result in declines in specific flux/permeability.

2.5.2 Membrane Fouling

Membrane fouling is the accumulation of particulates from the feed stream onto the membrane system and it can affect the performance of the membrane. According to Metcalf and Eddy (2003), fouling can occur due to the buildup of material on the membrane surface, chemical precipitates forming or damage to the membranes due to substances reacting with the membrane or biological agents that can colonize the membrane.

There are three main mechanisms that can cause membrane fouling; gel/cake formation, pore plugging and pore narrowing. Gel/cake formation is caused when a buildup of matter accumulates on the membrane surface due to concentration polarization. The matter that makes up the gel/cake is often bigger than the pore size of the membrane and this matter causes resistance for the flowing feed stream that requires a higher TMP. Gel/cake formation is an extreme case of concentration polarization. Pore plugging and pore narrowing is cause by particulates smaller than the pore size of the membrane. In the case of pore plugging the particles attempt to go through the pores and become stuck, causing more particulates to become trapped and eventually plugging the pores. Pore narrowing is similar to pore plugging in that the particles become attached to the side of the membrane pores which causes them to become narrow (Metcalf and Eddy, 2003). The smaller particles tend to be more difficult to remove during backwashing

cycles, which can lead to a greater build up and cause membrane fouling quicker. It is believed that pore narrowing can lead to greater concentration polarization.

As membrane fouling causes resistance to the feed stream, decreasing the flux and resulting in increased TMP, it is important to control the amount of fouling that does occur. There are three main ways to help reduce membrane fouling; preventative measures such as backwashing and air scour, maintenance of the membrane (chemical washes) and membrane recovery (clean in place, CIP). Preventative measures are frequently in the membrane treatment cycle and are a physical method to remove fouling material. During backwashing, the direction of the flow is reversed, forcing permeate back through the membrane and removing the foulants. Unfortunately, not all the material may be removed and this could cause the fouling to build up. Some membranes, such as flat sheet membranes and spiral wound membranes are not designed to be backwashed, so they have to be cleaned by other methods such as air scour, which helps to prevent the foulants from attaching to the membrane surface (Pearce, 2011).

2.6 Membrane Bioreactors (MBR)

Membrane bioreactors (MBR) combine the biological treatment processes that are typically found in secondary wastewater treatment plants with the separation capabilities of membrane technology. The result of combining the treatment processes is a treatment train that is able to produce a high quality effluent to meet discharge regulations or that can be used for water reuse. Advantages of MBR plants over conventional activated sludge plants include smaller plant footprints because they run at higher MLSS, and reduced sludge production. Both of these advantages also contribute to economic benefits as a smaller footprint can result in lower operational costs and sludge disposal can be a high operational cost (Gander et al, 2000). Because of the MF and UF membranes capability to act as a barrier to pathogens (Chaudhry et al, 2015), the addition of membranes to the bioreactor tank can allow for the reduction or removal of disinfection stages further along the treatment train. Log removals of pathogenic viruses by MBR plants have been shown to be up to 5 log removal (Chaudhry et al, 2015) (Melin et al, 2006).

With the ability to control the solids retention time (SRT) and hydraulic retention time (HRT) separately, MBR plants are able to achieve high solids concentrations and are able to retain and develop slower growing microorganisms, which can allow for better removal of nutrients such as nitrogen. Also MBR plants are able to achieve stable effluent quality with variable influent characteristics due to the stability of the biomass and the membrane capabilities (Melin et al, 2006)

MBR plants can be designed with a submersed membrane in the biological tank or with an external membrane. The submersed membrane is found more commonly in industry, as external membranes requires the separated biomass to be returned to the biological tank (Gander et al, 2000).

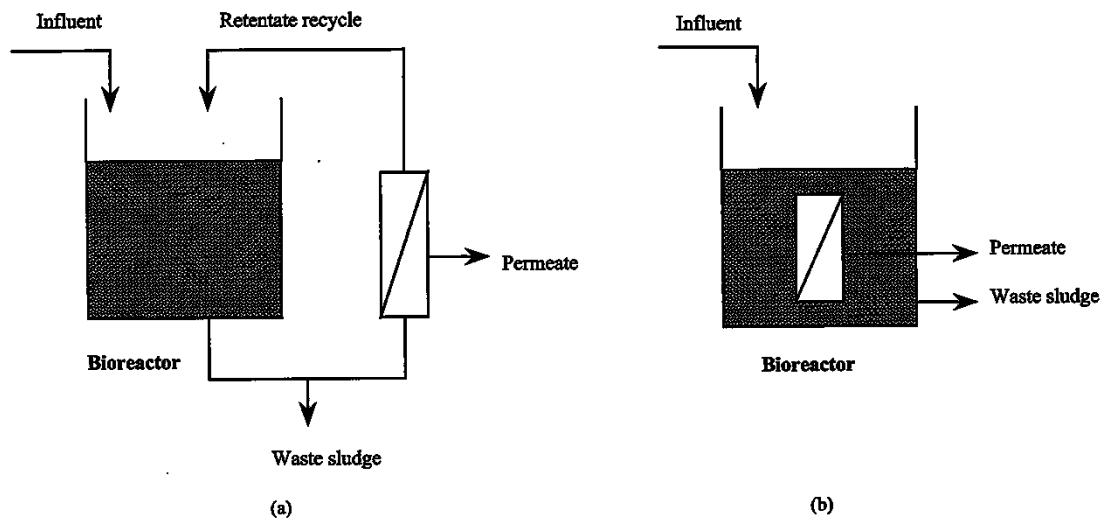


Figure 2-3 (a) Side-stream MBR with separate filtration unit; (b) Submerged MBR. Adapted from Gander et al, 2000

MBR plants first gained traction in the industry in the early 1990's with plants being installed in the United States and Australia, with Europe following shortly after. As the technology improved and the availability of membranes increased, the popularity of MBRs has also increased. The size of the plants varies from small plants in remote communities to larger, full scale MBR plants (Melin et al,2006).

2.6.1 Bardenpho MBR plant

The Bardenpho wastewater treatment process is used for biological nutrient removal (BNR), typically targeting nitrogen removal along with phosphorus and BOD

removal. The Bardenpho process is a modification of the Modified Ludzak-Ettinger (MLE) process, which involves a pre-anoxic tank, followed by an aerobic tank and secondary clarifier. The 4-Stage Bardenpho MBR process has an additional post anoxic tank and MBR tank. (Metcalf & Eddy, 2003). Figure 2-4 shows a schematic of a 4-stage Bardenpho treatment train.

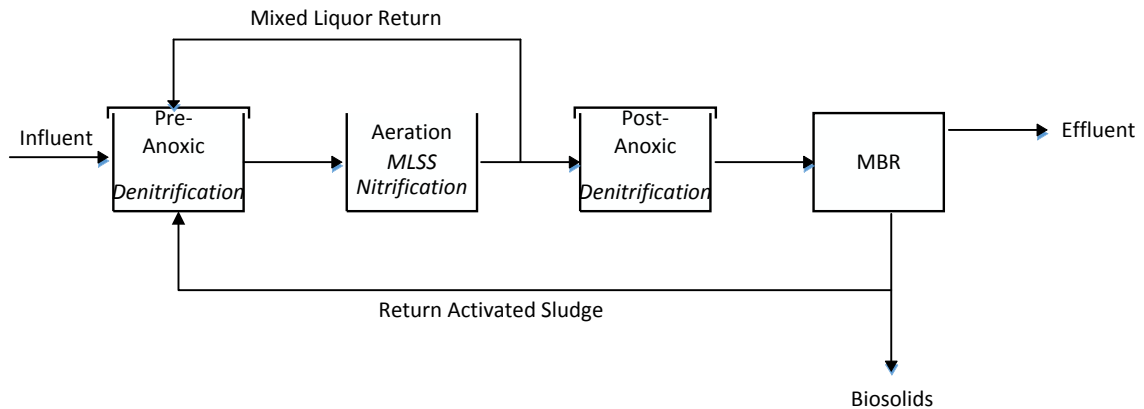


Figure 2-4 Bardenpho 4 Stage MBR

Barnard (1973) is credited with the development of the Bardenpho process, which is named after him and the processes it uses (Bard-den-pho, for Barnard, denitrification and phosphorus) and introducing internal recycling of MLSS to increase nitrates in the pre-anoxic tank.

In the Bardenpho process, the pre-anoxic tank receives influent raw wastewater, and returned activated sludge (RAS). The MLSS of the RAS contains nitrate, which is not commonly found in the influent wastewater. The lack of oxygen in the tank allows for denitrification to occur, and the incoming organic carbon in the influent raw wastewater provides a food source for the heterotrophic bacteria (WEF, 2007). Denitrification in the pre-anoxic tanks also produces alkalinity, which is needed for the nitrification process downstream in the treatment train. (Muirhead, 2013) The pre-anoxic tank helps to mix the influent wastewater with RAS to create conditions that are favorable for bacteria growth in the activated sludge treatment (Metcalf & Eddy, 2003).

Nitrification and carbonaceous organic removal takes place in the aeration tank. Aeration is provided in the tank to provide oxygen to the bacteria that will consume the

CBOD and reduce ammonia (NH_3) to nitrite (NO_2^-) and nitrate (NO_3^-). It also keeps the MLSS well mixed. Aerobic heterotrophic bacteria that consume the organic matter produce biological flocs that can be removed through settling or membrane filtration while the aerobic autotrophic bacteria oxidize ammonia to reduce it down to nitrates and nitrites. (Metcalf & Eddy, 2003).

The post-anoxic zone serves as a location for denitrification to lower the nitrogen concentration in the effluent. As with the pre-anoxic tank, nitrate enters the tank along with any remaining organic carbon that was not consumed in the aerobic tank to be biologically reduced to nitrogen (N_2) gas that can be released. As the atmosphere contains more than 70% nitrogen naturally, the release of N_2 gas from wastewater treatment plants does not affect environmental conditions. (WEF, 2007). Because denitrification is completed by heterotrophic bacteria, an additional carbon source may be required to optimize the process if the amount of readily biodegradable BOD is not adequate. Methanol is commonly used as a carbon source.

Following the post-anoxic tank, a low pressure membrane is used to clarify the effluent and to remove any remaining solids. The effluent is then discharged to the receiving water.

Phosphorus, while not the main parameter targeted for removal by the Bardenpho treatment train, can be removed with the addition of an anaerobic stage. The anaerobic stage allows for the phosphorus accumulating organisms to start the luxury uptake of phosphorus. The phosphorus would then be removed by the UF membrane. The addition of a coagulant prior to the Bardenpho set up would also improve phosphorus removal results.

A study performed in New York showed that a pilot scale 4 stage Bardenpho system was able to achieve an effluent with BOD of 2 mg/L, COD of 21 mg/L, total nitrogen of 6 mg/L, ammonia of 0.8 mg/l and total phosphorus of 0.1 mg/L. (NYSERDA, 2008).

Chapter 3 MATERIALS AND METHODS

3.1 Aerotech Wastewater Treatment Facility

The MBR pilot plant was designed, built and operated at the Aerotech Wastewater Treatment Facility (AWWTF) located in Geoffs, Nova Scotia and operated by Halifax Water. The wastewater treated by the facility at AWWTF is a combination of domestic and industrial waste from the Aerotech Business Park along with domestic wastewater from the Halifax Stanfield International Airport (HSIA). The plant was constructed in 1986 and upgraded to the existing sequencing batch reactor (SBR) treatment train in 2005. The treated effluent from the plant is discharged into the Johnson River System. The facility was designed for average daily and maximum daily flows of 1360 m³ and 2080 m³. Flows in excess of this accumulate in the influent equalization (EQ) tank and adjacent overflow tank. Figure 3-1 shows the process schematic of the AWWTF.

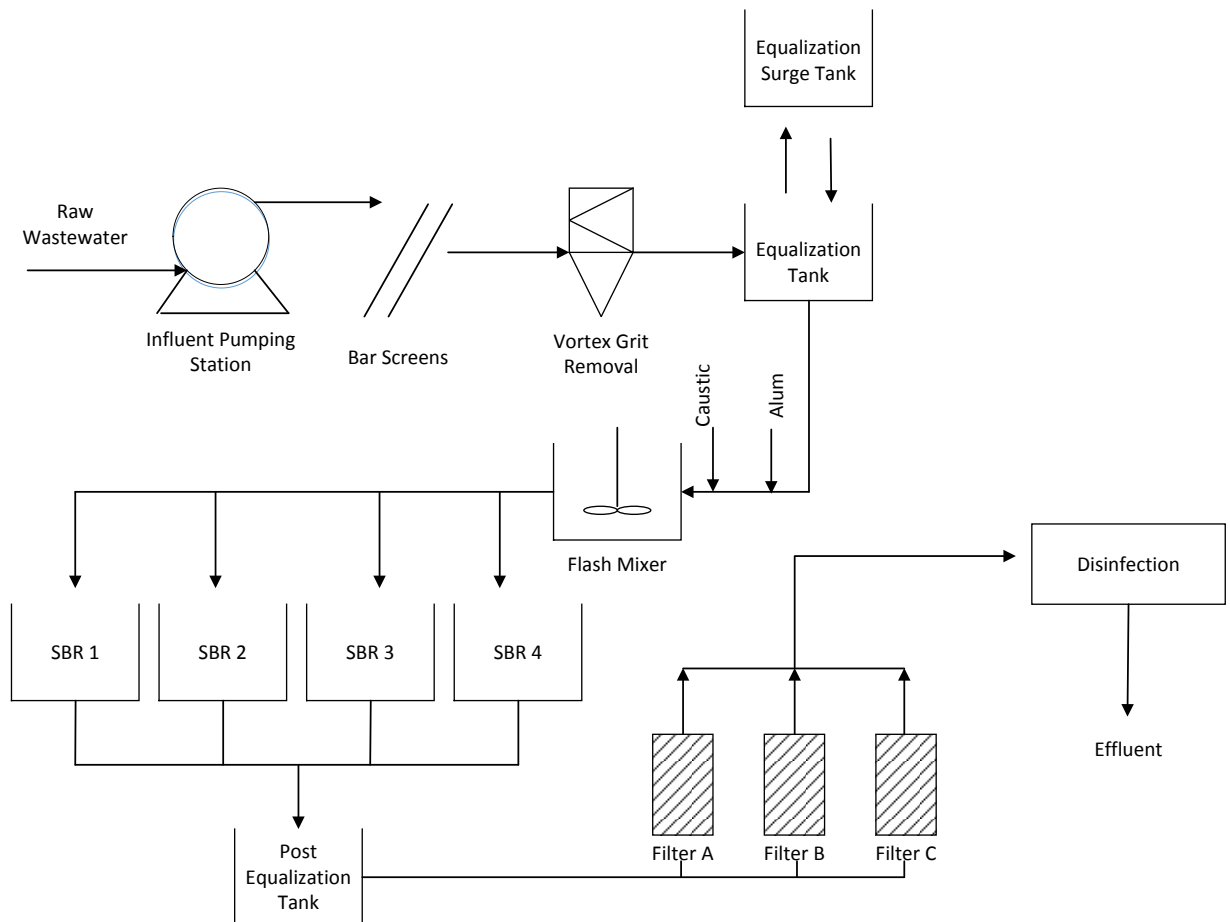


Figure 3-1 Schematic of AWWTF

The raw wastewater characteristics for the plant during the study period (Table 3-1) are similar to those outlined by Metcalf and Eddy (2003) for a medium strength municipal wastewater.

Table 3-1 Average raw wastewater quality for AWWTF

| Parameter | Average |
|------------------|----------------|
| BOD, mg/L | 162.8 ± 111.4 |
| TSS, mg/L | 225.0 ± 94.4 |
| COD, mg/L | 51.0 ± 19.6 |
| Ammonia, mg/L | 36.6 ± 10.6 |
| Phosphorus, mg/L | 16.3 ± 4.5 |

The current operating permit for AWWTF are 5 mg/L for BOD₅ and TSS, total ammonia less than 1.2 mg/L between May 1- October 31 otherwise 5.7 mg/L and total phosphorus less than 0.13 mg/L. The effluent shall be non-acutely lethal as well.

3.2 AWWTF SBR Process

The AWWTF receives wastewater from the Halifax Stanfield International Airport (HSIA) and from residential and industrial users in the Aerotech Business park. A 6 mm bar screen and a vortex grit removal chamber are used as primary screening to remove coarse objects and finer materials before entering the main treatment train. The wastewater then enters the equalization tank to help prevent variation in flow through the treatment process. As the effluent from the equalization tank enters the SBR tanks, it is dosed with aluminum sulfate (alum) for chemical precipitation of phosphorous. Due to the high alkalinity of the influent wastewater due to lagoon influent containing CaCO₃ from an industrial source, pH adjustment is not used for coagulation. In the SBR sequence, there is a 40 minute fill phase, 3 hour react phase followed by a 1 hour settle phase and then decanting. The SBR effluent then enters a post-SBR tank before filtration through upflow Dynasand deep-bed granular media (sand) filters. The treatment final step before discharge into the Johnson River System is UV disinfection.

3.3 MBR Pilot-Plant

A pilot-scale MBR plant was constructed at the Aerotech Wastewater Treatment Facility to model a planned upgrade at this facility. The pilot-plant was built in May 2016 and went through a commissioning phase from May 15 to June 20 2016. Data was collected from the pilot-plant for this study from June 20 – September 4, 2016. A flow diagram of the MBR pilot plant used in this study is presented in Figure 3.2.

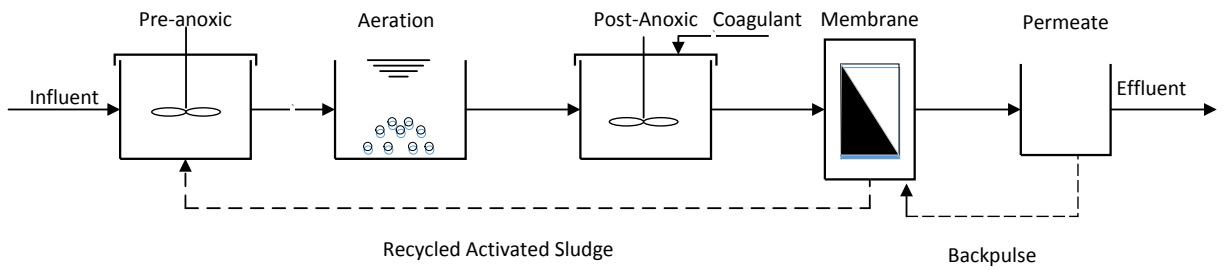
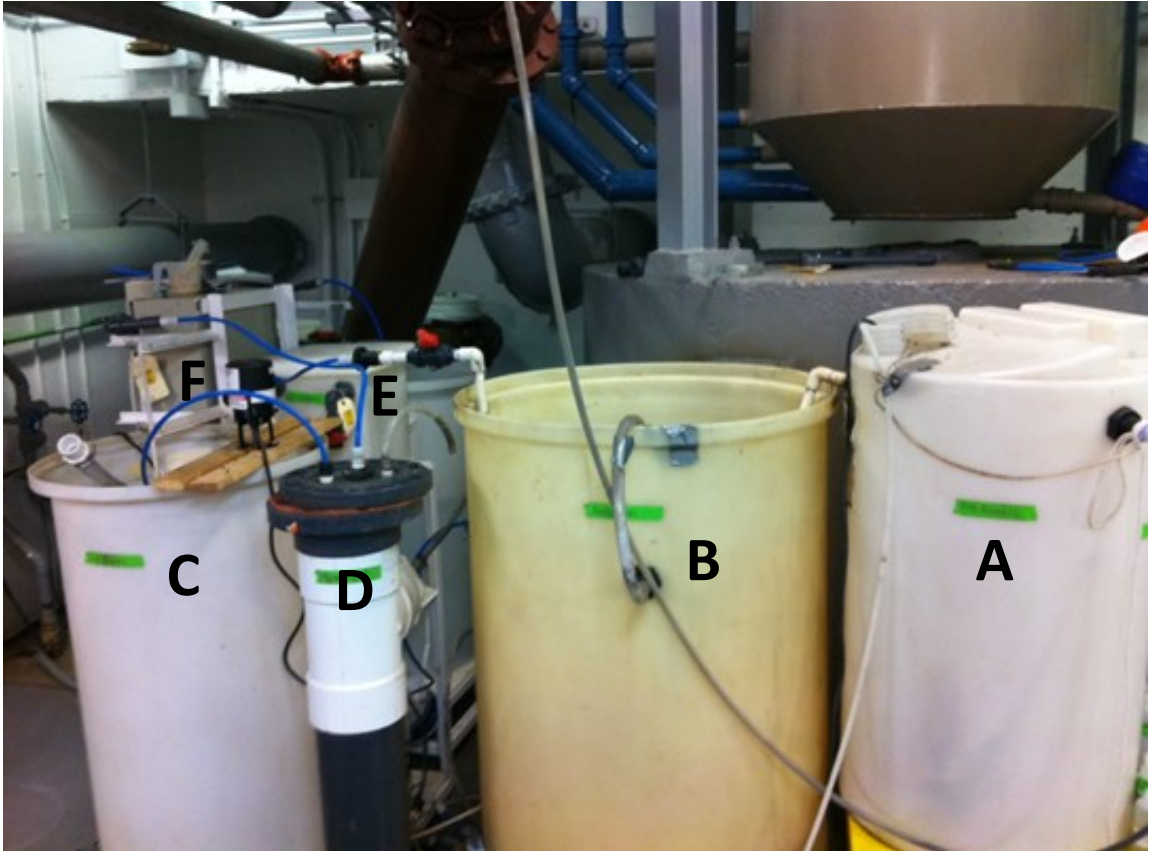


Figure 3-2 Process Diagram of Pilot MBR Plant

The pilot plant had an overall volume of 230 gallons (870 L). The pre-anoxic tank had a volume of 57 gallons (215 L), aeration tank of 107 gallons (405 L), post anoxic tank of 67 gallons (253 L) and the membrane tank of 7.3 gallons (27.6 L) (Figure 3-3).



A: Pre-anoxic tank. B: Aeration tank C: Post-anoxic tank D: Membrane process tank E: Permeate tank F: ZW 10 Skid

Figure 3-3 MBR Pilot Plant

The pre-anoxic, aeration and post anoxic tanks were made of polyethylene. The tanks were staged at varying heights to allow for the wastewater to flow through the pilot plant by gravity. The pump on the ZW 10 membrane skid was used to pump the treated effluent from the membrane tank to the effluent tank. The height differential from the inlet of the pre-anoxic tank to the inlet of the MBR tank was 0.5m. The membrane tank was constructed out of PVC piping and fittings. The flow rate of raw wastewater into the pilot plant was controlled by the influent pump and was 15L/hr. The retention time in each tank calculated based on the flow through the system is as follows: pre-anoxic tank 14.3 hrs., aeration tank 27 hrs., post anoxic tank 16.8 hrs. and membrane tank 1.84 hrs.

Primary screened wastewater from the AWWTF equalization tank was pumped into the pre-anoxic tank, where it was mixed with RAS from the membrane tank. The aeration tank was designed with two aeration diffusers and was connected to a blower (Medo, LA-120-A1108-E1-0511). The air flow into the system was varied between 2 to 3 scfm depending on the dissolved oxygen (DO) concentrations in the tank. The target DO in the aeration tank was 2.0 mg/L to provide adequate aeration for BOD removal.

Alum, at a dose of 300 mg/L, was added to the wastewater in the post-anoxic tank via a chemical feed pump (ProMinent). Alum was selected to match the coagulant being used for the AWWTF SBR system. The membrane that was used for this project was a ZW 10 ultrafiltration (UF) membrane supplied by GE Water and Process Technologies. The ZW10 module is representative of the ZW 500 UF modules which are used in full scale plants. It is a submersible membrane with outside-in flow configuration. The nominal pore size of the membrane is 0.04 microns and has a surface area of 0.93 m². The process pump for the membrane was operated on a 9 minute/15 second cycle. Water was pumped through the membrane for 9 minutes and then the membrane was backwashed for 15 seconds with membrane permeate. Air was supplied to the base of the UF membrane through a blower (Medo, LA-120-A1108-E1-0511) at a flow of 0.5 scfm to achieve air scouring to prevent membrane fouling. Effluent from the membrane was stored in the permeate tank that was also used for backwashing the membrane and for sample collection. Sludge wasting occurred through a valve on the membrane process tank. Due to difficulties maintaining sludge concentrations in the pilot plant system, sludge wasting occurred infrequently. When sludge wasting did occur, it often was the result of a sludge spill and was not controlled.

The pilot-plant was initially designed to have a hydraulic retention time (HRT) of 7 hours and to have a SRT of 10 to 20 days. However, during the commissioning of the pilot-plant, the flow through the plant had to be adjusted to prevent overflow conditions and to stay within the operating parameters of the pumps used in the system. Therefore, the HRT was adjusted to 27 hours which is much longer than the HRT for a typical MBR plant that typically ranges between 3 to 8 hours (Mohammed et al, 2008). Sludge wasting did not occur as frequently as expected, as there were issues maintaining solids concentrations in the system. This resulted in a calculated SRT of 46 days. Typically, the

SRT of an MBR plant will range between 10 to 30 days (Xing et al, 2000) but studies have been completed with SRTs as high as 110 days (Masse et al, 2006) to increase MLSS in the system. The longest that the pilot-plant operated was 24 days before being refilled, so this SRT was never met.

The pilot-plant was seeded with activated sludge from the SBR tanks prior to starting an experimental run. The activated sludge was collected while the SBR tank was either in the settling or decant phase to maximize the amount of solids that would be collected. The SBR tanks at the Aerotech facility typically operate at a MLSS of 3,500 to 4,000 mg/L. The pilot plant was seeded with activated sludge with an average MLSS of $3,630 \pm 220$ mg/L.

The membrane for the pilot plant required regular cleanings to prevent membrane fouling. The membrane was cleaned on a weekly basis or as needed based on the transmembrane pressure (TMP) measurements. A chemical clean-in-place (CIP) procedure was followed when the TMP reached 30 kPa. A 200 ppm sodium hypochlorite (NaOCl) solution was used for the CIPs. The membrane was backwashed with the NaOCl solution for 1 minute and then let sit for 5 minutes in the cleaning solution. This cycle was repeated 8 times. After the CIP was complete, the membrane, lines and membrane tank were rinsed with tap water to prevent any remaining NaOCl from entering the process water of the pilot-plant.

3.4 Bench-Scale Coagulation Study

A series of jar tests were conducted to evaluate alternative coagulants, doses and coagulation pH set points in a factorial design to determine impact of coagulation process on phosphorus removal for the test wastewater.

Three coagulants were used in this study; aluminum sulphate (Alum) ($\text{Al}_2(\text{SO}_4)_3$), polyaluminum chloride (PACl) and ferric sulphate ($\text{Fe}_2(\text{SO}_4)_3$). The coagulants used for the study were supplied by Chemtrade (Ontario, Canada). The coagulant doses that were selected for bench-scale experiments are presented in Table 3-2. A low dose and high dose were selected for each so that a factor analysis could be completed. The doses were selected for this study based on previous jar testing that had taken place at the plant by the operators.

Table 3-2 Bench-Scale Jar Testing Coagulant Dose and pH of Minimum Solubility

| Coagulant | Low Dose (mg/L) | High Dose (mg/L) | pH of minimum solubility (20°C) |
|------------------|------------------------|-------------------------|--|
| Alum | 200 | 400 | 6.0 |
| PACl | 100 | 200 | 6.2 |
| Ferric Sulphate | 200 | 400 | 5.5 |

In one set of trials, the pH was adjusted to the pH of minimum solubility for each coagulant as shown in Table 3-2 using HCl, while the second set of trials had the pH adjusted to 7.0 using NaOH. pH of minimum solubility was selected as this is the pH that has been shown to result in the greatest formation of hydroxide solids that are used for coagulation. The pH of minimum solubility is favorable for TP removal by coagulation, but are not favorable for biological wastewater treatment. A trial of pH 7 was selected to represent the operating condition at the AWWTF, where pH control is not used. The plant typically operates between 6.8 and 7.0 for incoming wastewater. All the experiments were performed in duplicates.

Currently at the wastewater treatment plant, sand filtration is used after the wastewater is treated by the SBRs. The plant is currently undergoing an upgrade to MBR system. To reflect the change in treatment, both sand filtration and membrane filtration were considered. At bench scale, sand filtration was represented by filtering the supernatant from the jar testing through 1.2 µm filter paper. ZW-1 hollow fiber ultrafiltration (UF) membranes were used to perform the membrane filtration.

5-gallon water jugs were used to collect raw wastewater from the equalization tank at the Aerotech Wastewater Treatment Plant. This location was selected for collection as the raw wastewater had received primary screening but had not been dosed with coagulant. The samples were transported to the Clean Water Lab at Dalhousie University, where the jar testing took place.

The experiments were performed using a standard jar test apparatus (Phipps & Bird, Fisher Scientific). The jar test procedure was adapted from Sarparastzadeh et. al.,

(2007) and modified to match operations of the full scale wastewater treatment plant. The modified procedure consisted of a rapid mix at 150 rpm for 1 minute, followed by a slow mix for 15 minutes and then a 30 minute settling period. Once the jar test procedure was complete, the settled water was filtered either through 1.2 µm filter paper or the UF bench scale membrane. Samples were then collected for analysis.

3.5 Analytical Methods

Samples were collected from the pilot-plant influent, pre-anoxic tank, aeration tank, post-anoxic tank, membrane process tank and permeate tank. The SBR treated effluent was also sampled. Samples were stored in a cooler during transportation from the AWWTF to the Clean Water Lab at Dalhousie University. Upon arrival to the laboratory, the samples were stored at 4°C until analysis was conducted. Samples were analyzed following the procedures outlined by the Standard Methods for Examination of Water and Wastewater (APHA, AWWA, & WEF, 1998) and within the recommended storage time for each test parameter.

pH was measured on unfiltered samples and measurements were taken using a Accumet XL50 meter equipped with AccuFlow electrodes (Fisher Scientific). Turbidity was measured using a HACH 2100 AN turbidimeter and was measured on unfiltered samples. TOC measurements were analyzed following the methods described in the *Standard Methods* (5310 B) (APHA, AWWA, & WEF, 1998). The water samples were placed in headspace free 40 mL glass vials, and 85 percent phosphoric acid was added to preserve the samples. The samples were then measured using a TOC-V CHP analyzer (Shimadzu, Japan). Total suspended solids (TSS) testing was performed following the procedure outlined in the *Standard Methods* (2540 D) (APHA, AWWA, & WEF, 1998). Samples for soluble metals were acidified using concentrated nitric acid and were analyzed using ICP-MS (Thermo Scientific, X-Series). BOD was measured according to method 5210B of *Standard Methods* (APHA, AWWA, & WEF, 1998). Nitrate, nitrite and ammonia samples were analyzed using methods 8039 (cadmium reduction method), 8153 (ferrous sulfate method) and 8155 (salicyclate method) with a spectrophotometer (HACH DR5000).

Total Phosphorus (TP) and COD analysis was performed by the AWWTF operators as part of their daily sampling schedule. TP was measured using method 8190 of *Standard Methods* (APHA, AWWA, & WEF, 1998) with a spectrophotometer while COD was performed using method 8000 of *Standard Methods* (APHA, AWWA, & WEF, 1998) with a spectrophotometer (HACH DR2700).

3.6 Data Analysis

The data obtained was compared using a paired t-test in Excel 2016 to determine a p-value. A confidence of 95% ($\alpha=0.05$) was used for all tests. A calculated p-value less than 0.05 was deemed to have a significant result while a p-value greater than 0.05 was considered not significant.

Chapter 4 RESULTS AND DISCUSSION

4.1 MBR Pilot Plant Study

4.1.1 General Pilot-Plant Operations

During the pilot-plant study, there were several instances that caused disruption in the operation of the plant that required that the pilot be taken offline or emptied and reseeded with SBR activated sludge. Some of the disruptions that occurred during the pilot were wash outs or spills of the activated sludge, which resulted in a loss of MLSS from the pilot system. A list of disruptions can be found in Appendix B.

During these shutdowns there was a loss of MLSS from the pilot-plant tanks. On average, the aeration tank achieved a MLSS concentration of $2,114 \pm 696$ mg/L during normal operations. When the plant operation was disrupted, the MLSS would drop to as low as 77 mg/L. This resulted in observed spikes in permeate BOD, COD, nitrogen and phosphorus concentrations after the pilot-plant was put back online. Figure 4-1 shows the MLSS concentrations between August 5 to September 7, 2016 of the study measured in the pilot-plant aeration tank.

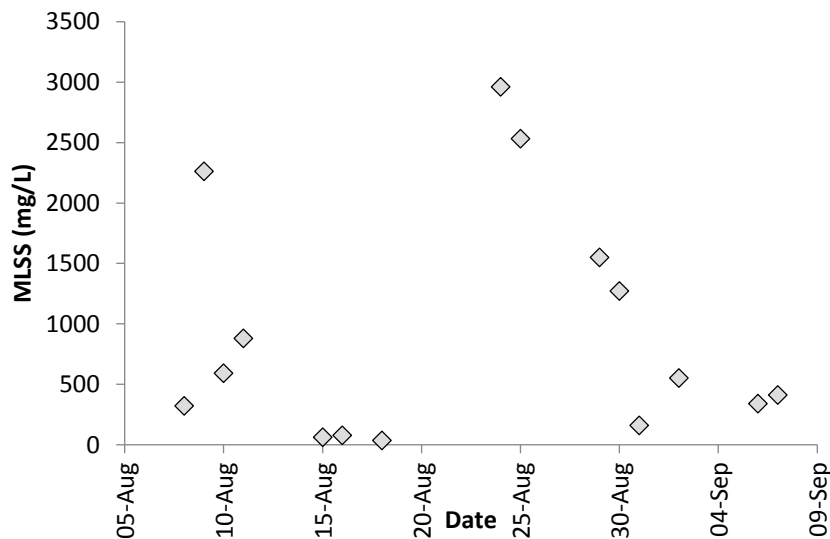


Figure 4-1 MLSS concentration in MBR Aeration Tank

Membrane bioreactors are typically operated at MLSS concentrations of 8,000 mg/L or higher (Metcalf & Eddy, 2003). Previous research has shown MBR plants that have operated with MLSS varying between 9,980 to 26,720 mg/L (Mohammed et al, 2008).

Typically, the AWWTF SBR system operates with MLSS between 3,500 to 4,000 mg/L. Due to the time constraints of the pilot project, the MLSS in the aeration tank of the pilot was not able to be built up to levels reported previously in literature for MBR operations.

Fluctuations in dissolved oxygen (DO) concentrations monitored in the pilot-plant aeration tank also occurred during the study. In biological treatment systems, DO concentrations should be maintained between 1.5 to 2.0 mg/L for optimum BOD removal (Metcalf and Eddy, 2003). The target DO concentration for the pilot plant was 2.0 mg/L. Figure 4-2 shows the DO concentrations measured in the aeration tank during the pilot study.

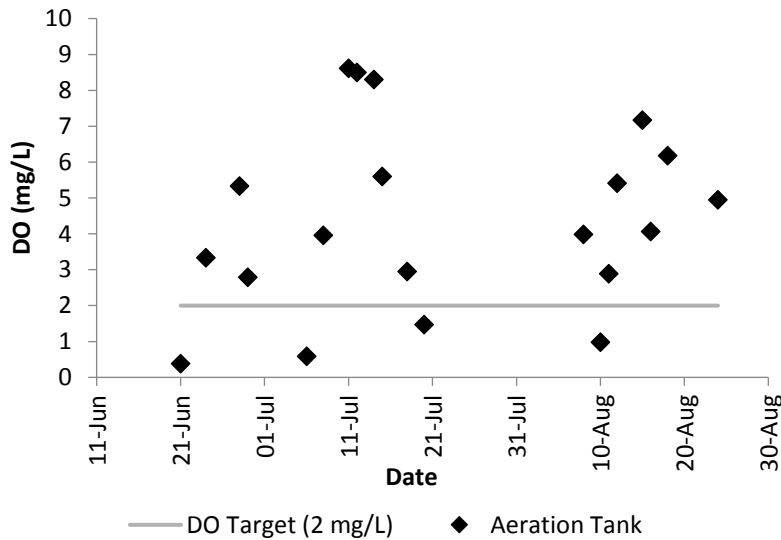


Figure 4-2 DO concentrations in MBR Aeration

The average DO concentration in the aeration tank was 4.37 ± 2.55 mg/L during the pilot study. The increased DO concentration was due to operational issues of the pilot plant. Too much air was pumped into the aeration tank as the air flow was controlled by a valve that did not allow for minor adjustments to be made. Operator inexperience may have also contributed to the higher than expect DO levels. Metcalf & Eddy (2003) states that in some cases DO levels above 2.0 mg/L can have beneficial effects on BOD and nitrogen removal, but that the effects are lost at DO concentrations above 5 mg/L. The periods during the pilot study when the DO concentration varied greatly between 2.0

mg/L and 8.5 mg/L along with pilot-plant effluent BOD concentrations can be seen in Figure 4-3.

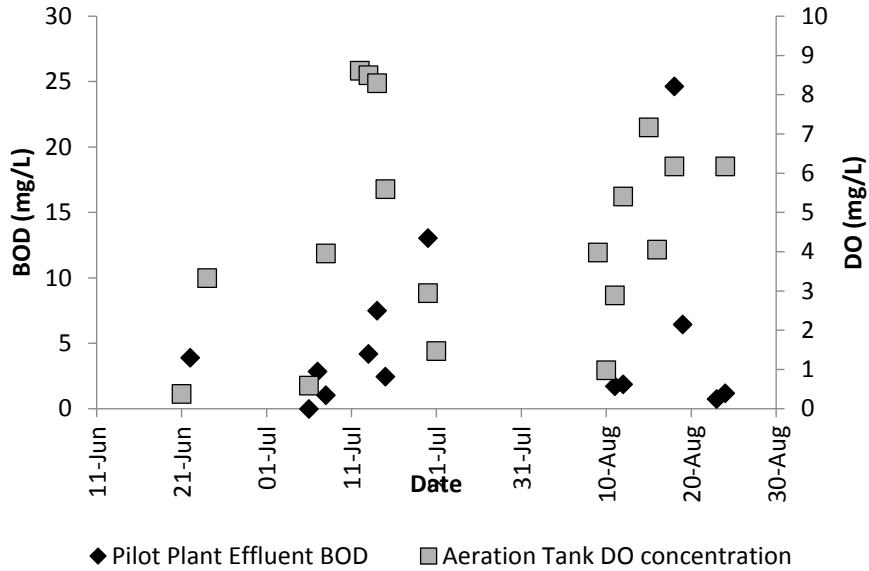


Figure 4-3 Pilot Plant Effluent BOD and MBR Aeration DO concentrations

With the higher DO levels in the aeration tank, DO was able to travel to the post-anoxic tank which was designed to have a DO level below 0.5 mg/L. Increased DO levels could affect the denitrification bacteria and limit nitrogen removal. The DO could also travel to the pre-anoxic tank through the RAS line which would also affect denitrification in that tank.

4.1.2 BOD Removal Results

Samples were collected from the raw wastewater in the SBR equalization tank, the pilot plant effluent and the SBR effluent during the duration of the study. The BOD measurement results are presented in Figure 4-4.

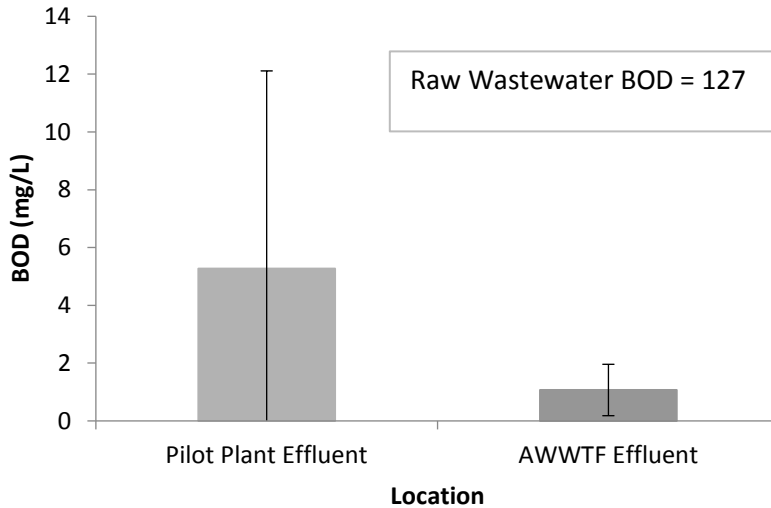


Figure 4-4 Average BOD concentrations during study

The average raw wastewater BOD concentration measured during the study was 127 ± 55 mg/L. The removal rate of BOD achieved by the pilot plant was 95% (permeate BOD = 5.3 ± 6.8 mg/L) while the SBR was able to achieve 99% removal of BOD (SBR effluent = 1.1 ± 0.9 mg/L). This is comparable to removal rates found by other MBR studies, where pilot-plants with pre-anoxic and aeration tanks were able to achieve 98% BOD removal rates with average effluent BOD concentrations of 3.2 mg/L (Santamasas et al, 2013). Comparing the results to the discharge regulations of the plant, the SBR was able to achieve BOD concentrations less than 5 mg/L while the MBR had an average slightly above the regulation at 5.3 ± 6.8 mg/L.

Mixed liquor suspended solids (MLSS) according to Metcalf and Eddy (2003) is “the biomass contained in a treatment reactor used to bring about treatment of the organic material in wastewater”. While it is not a direct count of the microorganisms in the tank available to break down the organic material, it can be used as a surrogate parameter. Losses of MLSS as seen in the pilot plant would result in a loss of biomass available to break down the organic carbon and could have resulted in increases in the BOD concentrations in the MBR system effluent as were seen in the operation of the pilot plant. Appendix B outlines some of the events that occurred that resulted in the loss of MLSS, including tank overflows.

4.1.3 COD Removal Results

The SBR is currently not regulated for COD in the treated effluent, but COD is measured daily by plant operators. Figure 4-5 shows the average COD concentrations measured during the pilot study. The average raw wastewater COD concentration was 364 ± 74 mg/L. The raw wastewater COD is approximately three times larger than the incoming BOD₅. This is likely due to some of the industrial qqqbusinesses in the Aerotech Business Park that contribute wastewater to the system. The pilot plant effluent had an average COD concentration of 49 ± 17.4 mg/L and a removal rate of 86.5%. The SBR effluent had an average COD concentration of 35 ± 4.2 mg/L and removal rate of 90.5%. The COD measurements taking during the pilot study follow a similar trend as the BOD results, with the SBR treatment achieving higher removal rates than the pilot-plant.

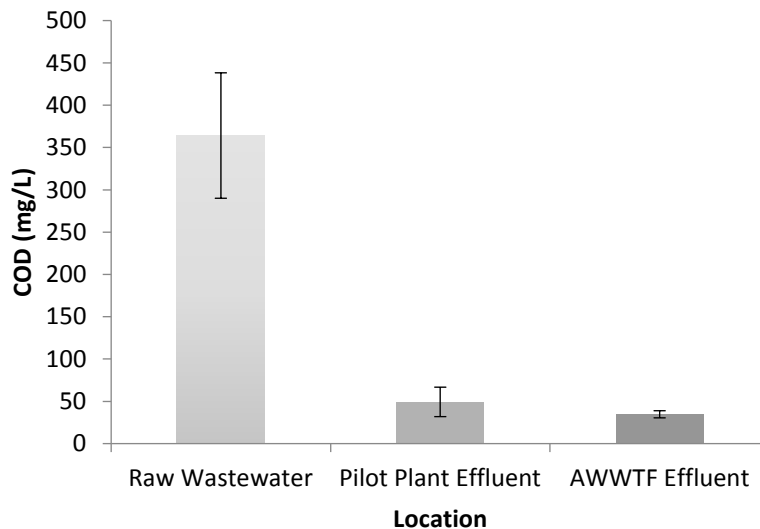


Figure 4-5 Average COD concentrations during study

Operational issues of maintaining MLSS concentrations in the pilot plant may have also affected the COD removal during the study. Other studies show that higher COD removal rates between 97 to 99% are achievable when a MBR treatment system has MLSS concentrations greater than 3,000 mg/L (Al-Malack, 2005; Mohammed et al, 2007).

During the study, sludge wasting was not performed as initially planned. Sludge wasting was reduced to try and increase the MLSS to above 3,000 mg/L. Sludge wasting

should be performed regularly to maintain necessary SRT and to prevent the buildup of inorganic compounds that can become harmful to the active biomass (Mohammad et al, 2007). The calculated SRT for the pilot-plant during the study was 47 days. The calculation can be found in appendix A. However, the sludge was never actually kept in the system that long as the longest trial was 21 days. However, other bench-scale MBR studies have reported SRTs between 5 to 35 days (How et al, 2005; Xing et al, 2000).

Pollice et al (2007) found determined removal rates of COD for SRTs greater than 20 days. That study compared SRTs ranging between 20 days and complete retention (approx 1,200 days) and showed that under all conditions considered, removal rates of 86 to 95% for COD and 99% removal of NH_4^+ for nitrification. Maintaining SRTs longer than 40 days did not hinder the biodegradation activity of the biomass (Pollice et al. 2007). During that study, they showed that the MLSS increased over time and that the biomass concentrations achieved steady state after at least 7 weeks. For the current pilot project, there was no increase in MLSS during the study, which may be why the removal rates of BOD and COD were slightly lower than the other studies mentioned.

Cicek et al (2001) showed that there could be negative impacts of longer SRTs such as limitations for oxygen transfer. Another issue that can arise with complete retention of sludge can be cell lysis (Pollice, 2007), which is the breakdown of a cell's membrane causing integral damage to the cell. The production of extracellular polymeric substances (EPS) could also occur at longer SRTs which could result in the formation of biofilms that can affect the treatment process.

4.1.4 TSS Removal Results

Previously, the TSS regulation for AWWTF effluent was 10 mg/L, which the plant was consistently achieving with their current SBR design. The new regulations will place an effluent discharge limit on TSS to 5 mg/L or less for this particular facility. The average raw wastewater TSS concentration during the pilot study was 245 ± 123 mg/L. The AWWTF, which has sand filters to remove particulate matter at the end of the treatment train was able to achieve an average value of 4.14 ± 1.5 mg/L of TSS in the effluent. The MBR pilot plant was able to achieve 1.5 ± 2.0 mg/L of TSS in the permeate. The removal

rates of TSS for AWWTF and the pilot plant were 98% and 99% respectively. Figure 4-6 shows the TSS results observed during the study.

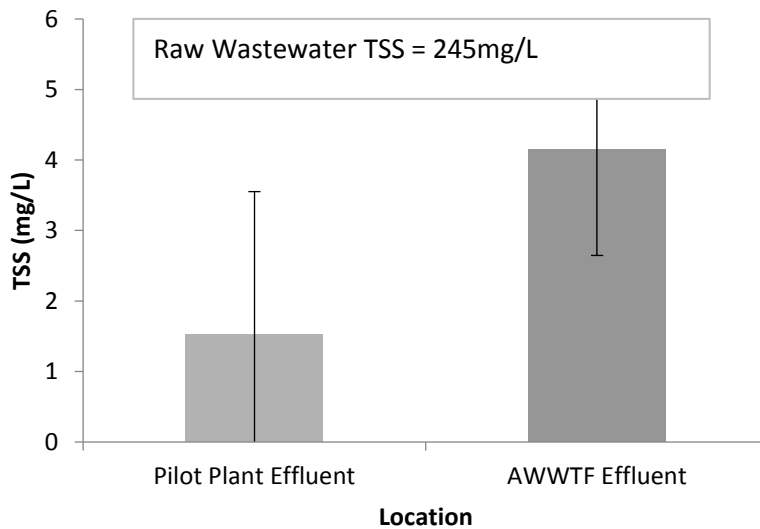


Figure 4-6 Average TSS levels during study

The pilot-plant acted as expected, with the membrane acting as a complete barrier to particulate matter and providing separation between the activated sludge and the treated permeate effluent. The UF membrane, with a rated nominal pore size of 0.04 μm was able to retain the biomass and effectively keep the effluent below the regulated limit for TSS concentrations. The average TSS in the pilot plant effluent was slightly higher than the 1 mg/L that is often stated in literature with membrane filtration (Ferraris et al, 2009) but this could be due to laboratory error.

4.1.5 Nitrogen Removal Results

The AWWTF is regulated for ammonia and total nitrogen (TN) in the plant effluent. The ammonia regulation is determined based on the time of year. Between May 1st- to October 30th, ammonia levels are not to exceed 1.2 mg/L in the treated effluent and 5.7 mg/L during the rest of the year. TN is regulated at 8 mg/L in the treated effluent throughout the year.

The results for ammonia testing during the pilot study are presented in Figure 4-7.

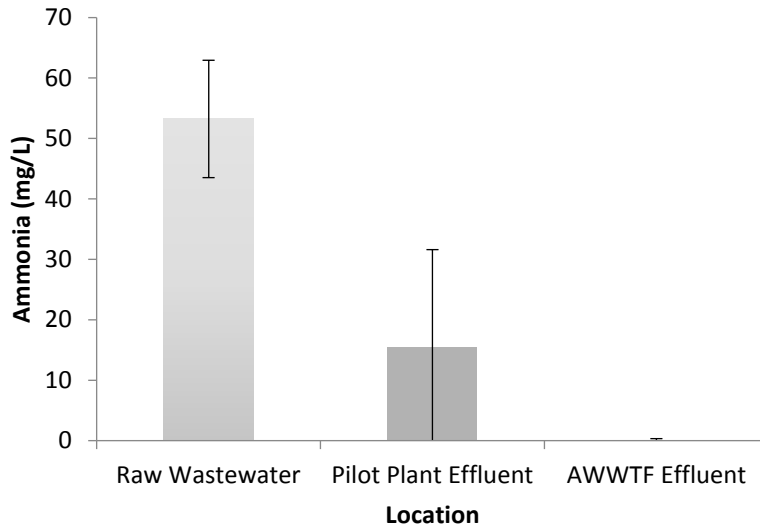


Figure 4-7 Average Ammonia concentrations during study

The average raw wastewater ammonia concentration was 53.2 ± 9.7 mg/L while the pilot plant effluent average was 15.5 ± 16.1 mg/L and the AWWTF effluent average was 0.12 ± 0.21 mg/L. The pilot plant average permeate effluent ammonia quality did not meet the higher 5.7 mg/L winter discharge limit during the pilot study so it did not meet either limit.

Figure 4-8 shows the pilot-plant and AWWTF effluent ammonia concentrations from sampling during the month of August 2016.

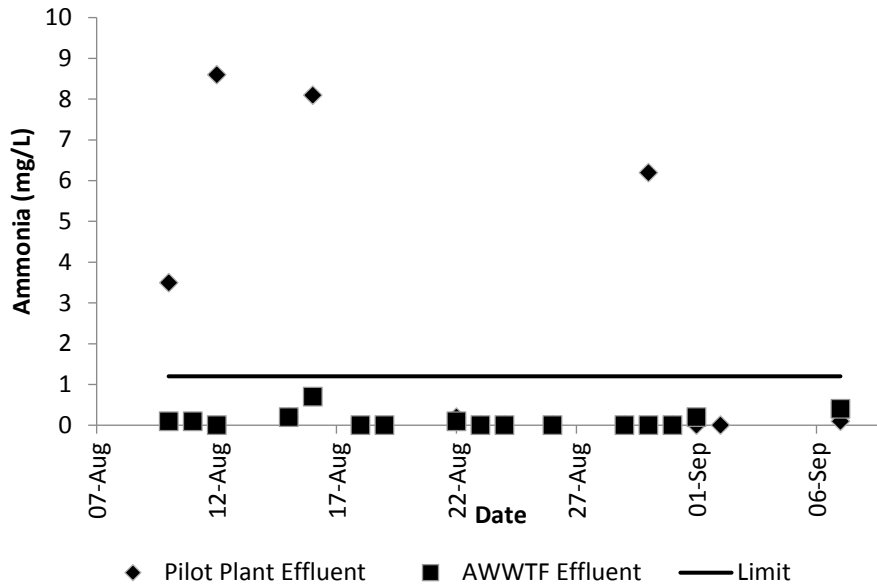


Figure 4-8 Ammonia concentrations during study

During this particular operating period of the study, the MBR pilot-plant was able to achieve the ammonia discharge concentration limit. During the month of July 2016, the average ammonia concentrations in the pilot-plant effluent was 30.5 ± 12.6 mg/L while during August 2016 it was 1.9 ± 3.3 mg/L. The high ammonia concentrations measured on permeate samples during July increased the overall average during the entire pilot study period. The removal rate of ammonia by the pilot plant during July was 43% and improved to 96% during August.

The high ammonia concentrations in permeate samples collected in July suggest that ammonia was not being nitrified to nitrites and nitrates, signaling issues with the biological treatment. According to Ferraris et al (2009), whose study focused on the startup of a pilot-scale MBR treatment system, nitrification can be delayed during the startup of a plant even if seeded with developed activated sludge. In that study, trials took up to 20 days to achieve nitrification after start up. During this study, the pilot plant was at most operated for 24 days, so the nitrifying bacteria may not have had the ability to acclimatize in the system and achieve steady state conditions. This was also found in the study performed by Wang et al (2012) for a pilot scale activated sludge system that did not achieve full nitrification until after 60 days. Once nitrification was achieved, these studies were able to maintain low ammonia concentrations in the plants effluents.

The bacteria involved in ammonia removal, ammonia oxidizing bacteria (AOBs), are sensitive to many environmental factors, such as pH, temperature, dissolved oxygen and nitrogen concentrations in the wastewater (Wang et al, 2012). During the month of July, issues did arise with maintaining the DO concentrations above 2.0 mg/ L, as seen previously in Figure 4-2. Metcalf and Eddy (2003) state that nitrification improves up to a DO concentration of 3 to 4 mg/L. In this study during July the DO varied drastically from 0.5 mg/L to 8.5 mg/L. The change in DO could have impacted AOBs growth, as they are slow growing bacteria.

In August, after three initial ammonia concentrations measurements around 8 mg/L, subsequent sample analysis showed ammonia concentrations below the discharge limit. This corresponds to the startup period after the pilot plant was reseeded with activated sludge from the AWWTF SBR tanks. The operating conditions were more favorable during this startup and the overall operation of the pilot plant was better performed. The acclimation period was shorter, which may have allowed for the nitrification to start and convert ammonia to nitrite. The MLSS concentrations were higher in August in the aeration tank which may have contributed to the higher ammonia removal rate.

Nitrite and nitrate were measured on the pilot plant effluent to determine if the nitrification and denitrification processes were working. It was not measured on the AWWTF effluent. The results for nitrite and nitrate concentrations are presented in Figure 4-9.

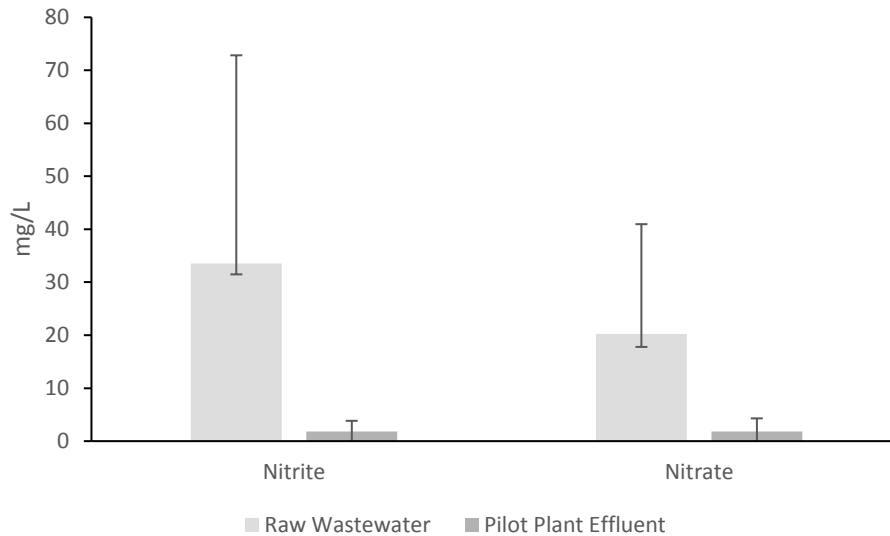


Figure 4-9 Average nitrite and nitrate concentrations for the study from the raw wastewater and pilot-plant effluent

The average nitrite and nitrate concentrations were 20.3 ± 20.7 mg/L and 33.5 ± 1.8 mg/L, respectively, measured in the raw wastewater during the study. In the pilot plant effluent, the average nitrite concentration was 1.8 ± 2.0 mg/L and nitrate concentration was 2.8 ± 4.6 mg/L.

During the initial operating period of the pilot plant, the nitrite levels were on average below 0.9 mg/L. This suggested very low conversion of ammonia to nitrite, which corresponds with the higher ammonia levels in the permeate effluent that were seen at this time. During August 2016, the nitrite concentrations in the permeate effluent increased with the ammonia concentrations measured decreasing, demonstrating that nitrification reactions were occurring during this operating period. This can be seen in Figure 4-10. The nitrite levels during August were higher than the nitrate levels, showing that the nitrite was not being completely converted to nitrate. The nitrite oxidizing bacteria (NOBs), which are typically slower to grow than AOBs (Phillips et al, 2002), may have taken longer to acclimatize to the pilot plant, resulting in the lag in nitrite removal.

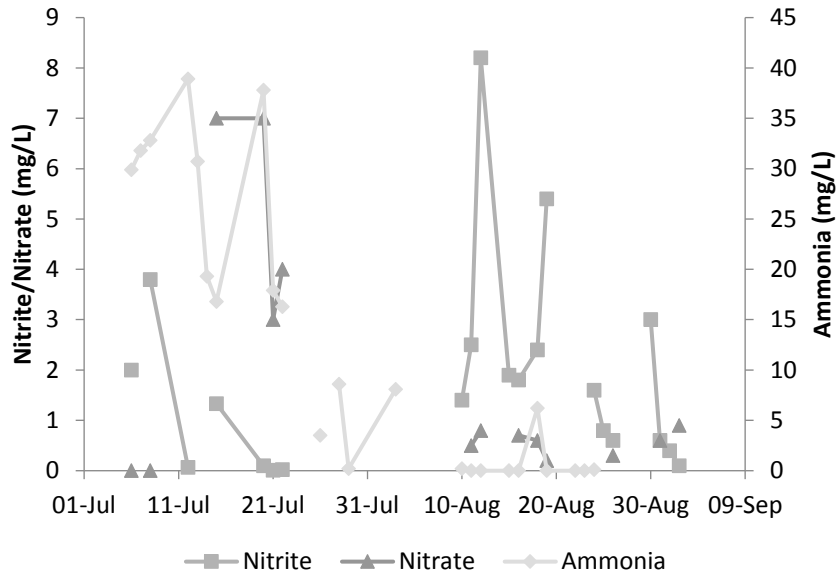


Figure 4-10 Permeate effluent ammonia, nitrite and nitrate concentrations

Nitrate levels peaked during July when the pilot plant was experiencing issues with DO levels and MLSS. High levels of nitrate, that correspond with low nitrite concentrations show that nitrification was taking place but that denitrification was not.

Denitrification is depended on the recycling of activated sludge to the pre- and post-anoxic tanks to convert nitrate to nitrogen gas (N₂). The pilot plant was initially designed to have a recycle rate of 4 but due to operational issues, the recycle rate was approximately 1. MBRs typically operate at recycle rates of 4-5 to allow for the return of activated sludge and to cycle nitrate to the pre-anoxic tank. This would greatly affect the denitrification process as the majority of the nitrate would not be returned to the pre-anoxic tank to use the incoming organic carbon as a carbon source. A carbon source was not added to the post anoxic tank, which also would have limited denitrification reactions as the aeration tank would remove most of the organic carbon that the microorganisms could use as a food source. Pilot-scale studies such as the one completed by Rosenberger et al (2002), show that with a higher recycle rate of 4, 80% removal of nitrogen can be achieved.

The role of oxygen in the denitrification process is also important. The heterotrophic bacteria require oxygen to consume organic carbon, either BOD or a supplied carbon source. If dissolved oxygen (DO) is available, the heterotrophic bacteria will use it to consume the carbon source. While this can help to remove BOD from the wastewater, it doesn't help with denitrification. When the heterotrophic bacteria used for denitrification

are in an anoxic state, where no or limited oxygen is available while nitrite and nitrate is available, they are able to use the nitrite or nitrate as an oxygen supply. The nitrite/nitrate becomes the electron acceptor in place of oxygen for the reaction. This leads to the reduction of nitrate to N_2 gas. During the operation of the pilot plant, there were fluctuations in the DO in the aeration tank, which led to increases in the DO levels in the pre- and post-anoxic tanks due to the recycling through the plant. The increases in DO during these times would cause the heterotrophic bacteria to use it as the oxygen source rather than the nitrite/nitrate. This would result in higher nitrate levels as denitrification would be limited at that time.

AWWTF is regulated for total nitrogen (TN) in the treated effluent, with a limit of 8 mg/L. Sampling for total nitrogen took place, but due to unforeseen issues with lab analysis equipment, the data was not retrieved. Total nitrogen is the combination of organic nitrogen, ammonia, nitrite and nitrate. According to Metcalf & Eddy (2003), organic nitrogen can account to up to 40% of the nitrogen in wastewater effluent. The addition of the averages for ammonia, nitrite and nitrate collected in this study have the total inorganic nitrogen (TIN) at an average 6.5 mg/l. If using a value of 40% of total nitrogen is organic in nature in the wastewater effluent, this would give the pilot plant an estimated TN concentration of approximately 9.1 mg/L. This is higher than the concentration outlined in the AWWTF regulations. Rosenberger et al (2002) reported a TN concentration of 13 mg/L in the treated effluent of a MBR plant, while Ferraris et al (2009) reported effluent TN concentrations varying between 10 to 12 mg/L in their study of a pilot-scale MBR system. Both studies had a recycle rate of 4.

4.1.6 Phosphorus Removal Results

Total phosphorus is regulated in the plant effluent to be less than 0.13 mg/L. During the pilot study, the average raw water TP was 4.85 ± 2.23 mg/L. The pilot-plant effluent samples showed an average TP of 1.02 ± 0.46 mg/L and the SBR effluent had an average TP of 0.27 ± 0.11 mg/L. Throughout the project, the pilot plant was not able to achieve the 0.13 mg/L discharge limit for TP in the effluent. The results can be seen in Figure 4-11. The removal rate of TP by the pilot-plant was 78% and 94% by the SBR.

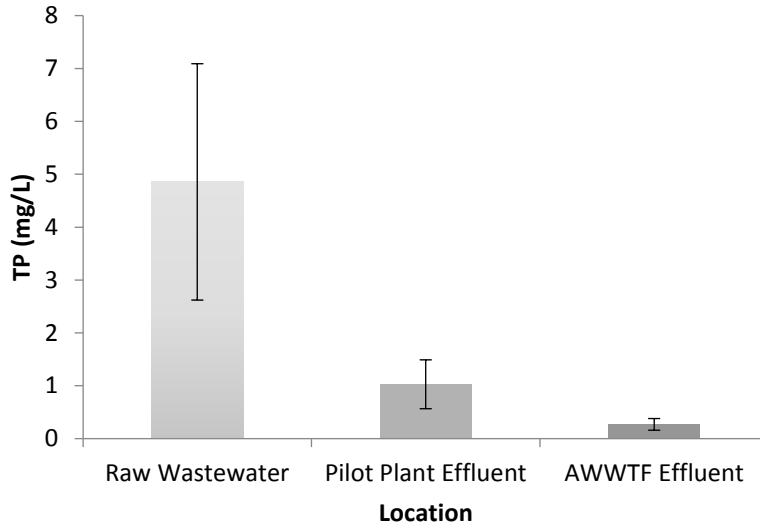


Figure 4-11 Average TP concentration during study

The pilot-plant was designed for phosphorus removal by coagulation rather than by biological treatment. Alum was dosed at 300 mg/L to the post anoxic tank and was mixed using a mixer prior to the membrane process tank. The Alum dose was selected based on the dose being used for the AWWTF SBR plant. No pH control was used to match the operating conditions of the AWWTF. The pH of the raw wastewater during the study was 7.38 ± 0.12 , with the AWWTF SBR tanks operating at a pH of 7.23 ± 0.13 and the pilot-plant effluent having a pH of 7.85 ± 0.37 . The pH of the pilot-plant effluent was higher than the SBR treatment train, which may have resulted in the lower TP removal rate. The pH of the pilot plant was higher than that of the range for optimal coagulation using alum, pH of 6.0, so the precipitates that were formed may not have been what were desired, limiting removal rates.

During the first month of the study, issues arose with dosing of the coagulant in terms of it being added too quickly to the tank and not continuously as designed. The mixer for the tank also was not working properly during this time, which may have also resulted in the higher TP concentrations in the treated effluent that were observed during that period.

For full scale wastewater plants using coagulation, jar tests are regularly completed to determine if the adequate coagulant dose is being used and if not what the coagulant dose should be to achieve optimal phosphorus removal. Due to the length of this study, jar testing was not completed during the pilot plant operation and the coagulant dose was

chosen based on operations of the SBR. Adequate coagulant choice and dosage based on bench-scale jar testing may have improved the phosphorus removal from the effluent.

Song et al (2008) evaluated alum as the coagulant for phosphorus removal at various doses in a bench-scale MBR plant. With a dose of 300 mg/L they were able to achieve a TP value of 1.1 mg/L in the treated effluent, with a removal rate of 90%. Zahid et al (2012) found at bench-scale that an MBR with alum as a coagulant could achieve TP levels of 0.62 and 0.33 mg/L for a dose of 300 mg/L of alum. They also showed in that study that alum doses above 120 mg/L resulted in biomass bacteria attacking the flocs to access phosphorus and released some phosphorus back into the wastewater in the process.

Wang et al (2014) conducted a study on a pilot-scale MBR using ferrous iron as the coagulant. The study found that when the coagulant was added to the membrane tank, 94% of the samples had a TP value less than 0.3 mg/L and that when it was dosed in the pre-anoxic tank, 95% of the samples had TP less than 0.3 mg/L. Collectively, these other published studies show that while the pilot-plant in this study did not achieve the necessary TP levels, that they are attainable by MBR systems.

4.2 Bench-Scale Coagulation Study

The objective of this part of the research project was to determine the effects of coagulant type, dose and pH on phosphorus removal capabilities. Bench-scale testing was completed using raw wastewater collected from the equalization tank of the wastewater treatment plant. Filtration type was also considered during the study as the current treatment train at AWWTF has sand filtration and the upgrades to the full-scale plant will involve membrane filtration in the MBR. It should be noted that the bench-scale testing was completed to determine the effect of different factors on the effluent quality and not for determining optimum coagulant dose.

4.2.1 Aluminum Sulphate

Figure 4.12 **Error! Reference source not found.** presents the settled water TSS concentrations measured in the jar tests with alum. The low alum dose was 200 mg/L while the high dose was 400 mg/L. The pH was adjusted to a pH of 6.0 (low) for the pH of minimum solubility of alum and to a pH of 7.0 (high) to simulate plant operating

conditions. The samples were also filtered through either 1.2 μm filter paper to represent sand filtration or through the ZW-1 UF membrane apparatus with pore size of 0.04 μm to represent membrane filtration.

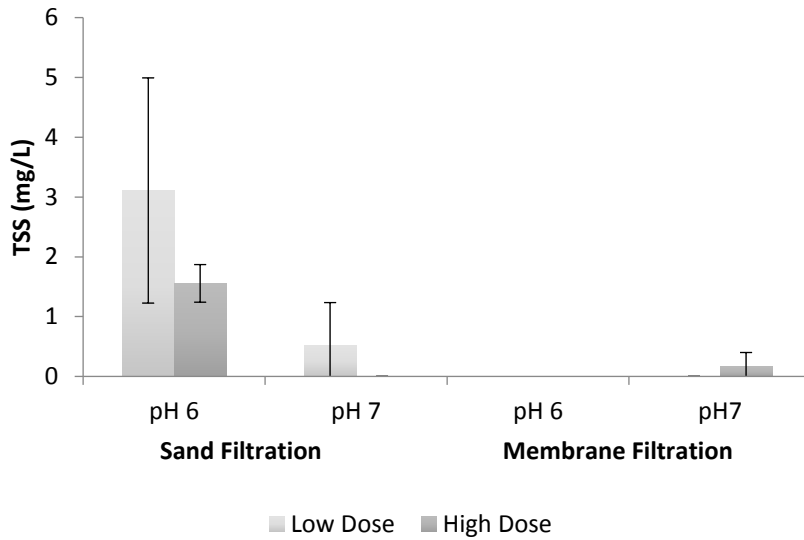


Figure 4-12 TSS concentration for Alum

The samples that were adjusted to pH 7.0 for both the low and high coagulant dose and both filtration types were able to achieve TSS readings below 1 mg/L, with both the 1.5 μm filter paper at the high alum dose and the UF membrane filtration at low alum dose achieving 0 mg/L TSS. Due to laboratory errors, there is no data for the trial at pH 6 with membrane filtration. Under each condition, the settled water TSS concentrations met the regulations for AWWTF.

UF membranes are used in wastewater treatment to act as a barrier and to provide separation between the biomass and the treated effluent when biological treatment is involved. As the UF membrane has nominal pore size of 0.04 μm , it can remove the smaller coagulated particles compared to the 1.2 μm filter paper. The filter paper at a pH of 7.0 and a high coagulant dose was also able to achieve the same result as the membrane. This may mean that better floc formation occurred in these jars which trapped the smaller particles allowing for them to be removed.

Figure 4-13 and Figure 4-14 show the settled water total phosphorus and dissolved phosphorus concentrations measured during the alum jar tests.

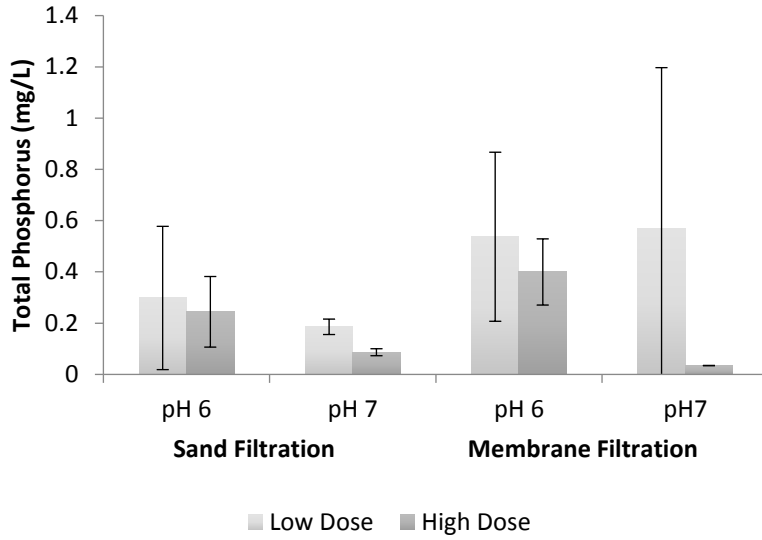


Figure 4-13 Total Phosphorus removal for Alum

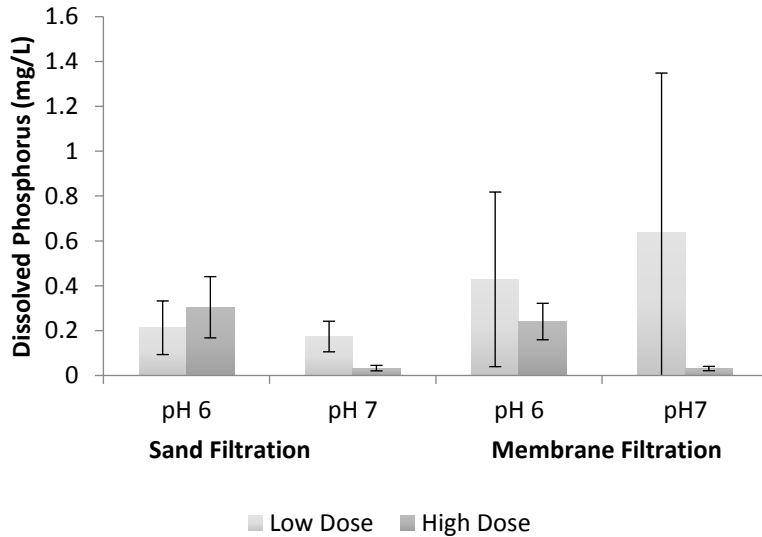


Figure 4-14 Dissolved Phosphorus removal for Alum

Total Phosphorus is regulated at 0.13 mg/L for the plant. Under the conditions that were run during the trials, the high dose of alum at pH 7 was able to achieve this with both filter types. Most of the phosphorus was in the dissolved form as seen when comparing Figure 4-13 and Figure 4-14.

Figure 4-15 shows the total aluminum results for the trials using alum as the coagulant.

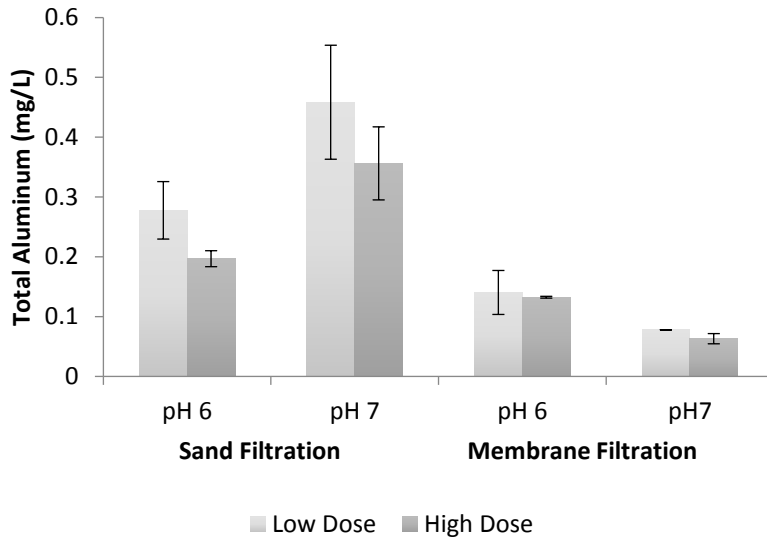


Figure 4-15 Total Aluminum removal for Alum

Aluminum regulations will become more stringent as governments become stricter on effluent discharge, so it was also considered for this study as two of the coagulants are aluminum based. For total aluminum, membrane filtration was able to provide the lowest concentration for the operating conditions considered. Dissolved aluminum ($<0.45 \mu\text{m}$) would be able to pass through the $1.5 \mu\text{m}$ filter paper used to represent sand filtration, but some may be retained by the UF membranes with $0.04 \mu\text{m}$ pore size. This could account for the difference seen by filter type.

4.2.2 PACl Experimental Results

Error! Reference source not found. outlines the TSS concentrations that were achieved at a low PACl dose of 100 mg/L and at a high dose of 200 mg/L, with pH adjustment to 6.2 and 7.0.

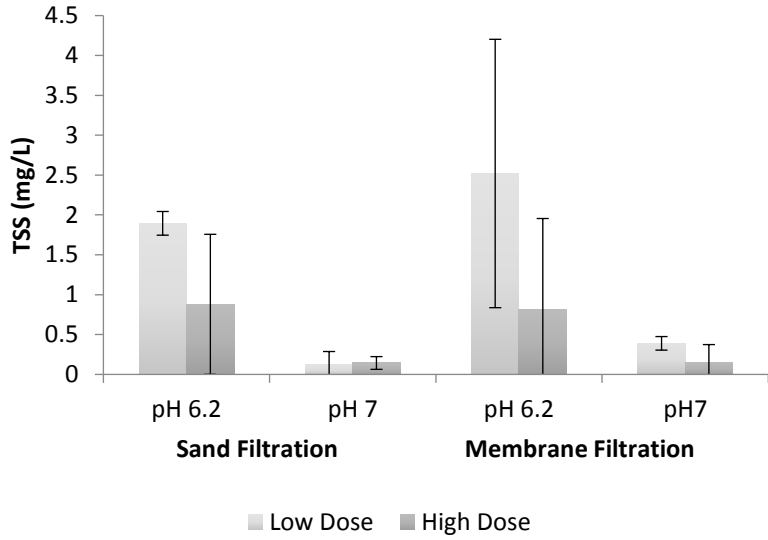


Figure 4-16 TSS removal with PACI

Conditions where the pH was 7.0 provided the greatest TSS removal for both the high and low dose of coagulant and filtration type. With TSS readings below 3 mg/L, laboratory errors can increase, which may have led to the variation in the results, as the membrane filtration values at pH 6.2 were higher than expected. All conditions were able to meet the TSS discharge regulation for AWWTF.

Figure 4-17 shows the removal results for Total Phosphorus for the PACI experiments and Figure 4-18 shows the dissolved phosphorus removal.

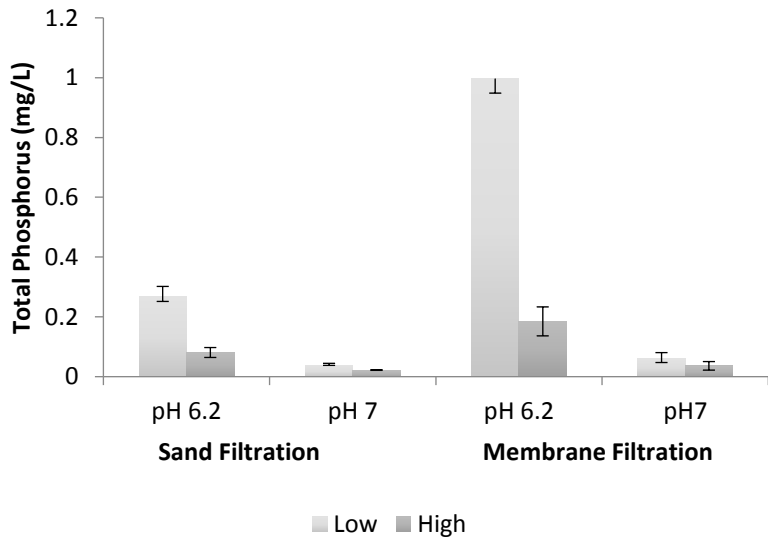


Figure 4-17 Total phosphorus removal for PACI

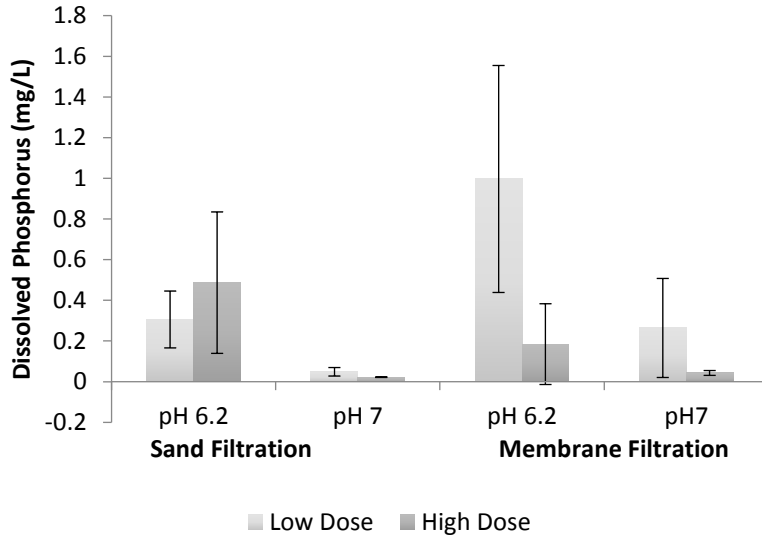


Figure 4-18 Dissolved phosphorus removal for PACl

Filtering through the 1.2 μm filter with the high coagulant dose at pH 7.0 could achieve a total phosphorus reading of 0.022 mg/L which is below the discharge limit for the wastewater plant. The membrane filtration at the high dose and pH of 7.0 also met the discharge regulation. Figure 4-17 and Figure 4-18 show that most the phosphorus was in the dissolved form ($<0.45\mu\text{m}$).

Figure 4-19 shows the effluent total aluminum concentrations for the PACl experiments.

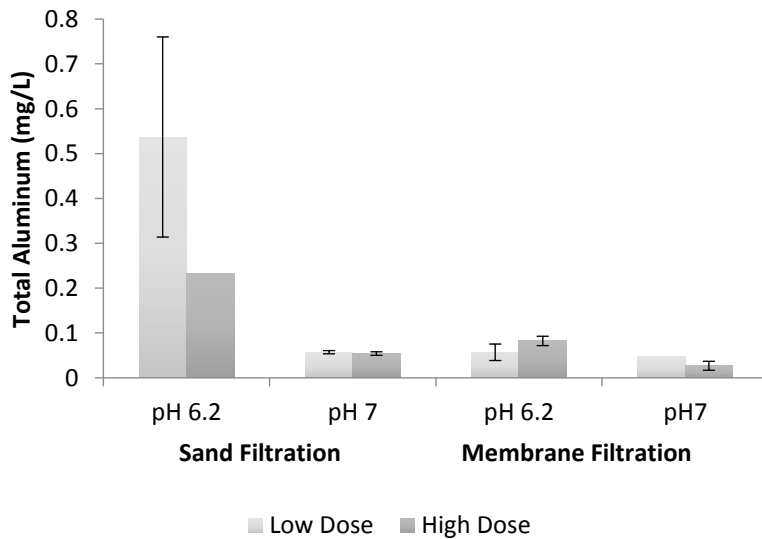


Figure 4-19 Total aluminum removal for PACl

Once again, the coagulant performed better at pH 7.0 for both coagulant doses and filtration types.

PACl is known to have a pH of minimum solubility of 6.2, which was the target pH for this study. The pH of minimum solubility is the point where the most coagulant will form the solid floc particles that will coagulate with the particles in the water. Above or below this, the species that form from the coagulant are not as useful in coagulation or are in the dissolved form and contribute to the TSS of the wastewater rather than improving it. One of the benefits to using PACl as a coagulant is that it can perform over a larger range of pH (typically between pH 5.0-8.0).

The pH of minimum solubility for PACl depends on the basicity of the coagulant, which effects the amount of alkalinity that it consumes. PACl with higher basicity will have a higher pH of minimum solubility and will be able to perform in a higher pH range. Initially in this study, the pH chosen for pH adjustment was 6.2 for PACl based on previous literature review (Metcalf and Eddy, 2003, Pernitsky & Edzwald, 2006). This pH is typical for a medium basicity PACl, at 20°C. During the study, a higher basicity PACl coagulant was used that had an optimal pH range between 6.0-7.8. High basicity PACl typically have a pH of minimum solubility of 6.5-6.7. This could be one reason that the samples adjusted to pH of 7.0 had better removal results than those at pH 6.2 as it was closer to the pH of minimum solubility, and in the middle of the optimal pH range for the coagulant.

Temperature can also affect the pH of minimum solubility for a coagulant. According to Pernitsky and Edzwald (2006), lower temperatures increases the pH of minimum solubility for a coagulant. The raw wastewater samples for this study were stored in a 4°C refrigerator prior to the experiment. As the raw wastewater was not at 20°C when the initial pH adjustment calculations were completed, this may have affected the pH the experiment was conducted at, if the temperature of the wastewater increased during the testing (as it was not temperature controlled).

4.2.3 Ferric Sulphate

Figure 4-20 shows the TSS results for the trials where ferric sulphate was used as the coagulant. The low ferric sulphate dose was 200 mg/L and the high dose was 400 mg/L. The pH was adjusted to 5.5 for the pH of minimum solubility and to pH 7.0

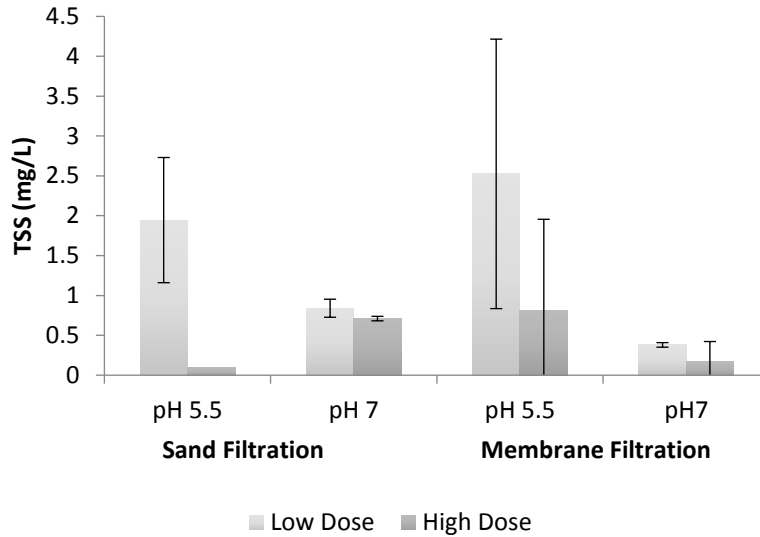


Figure 4-20 TSS removal for ferric sulphate

The ferric sulphate performed similarly to the other two coagulants in regards to TSS removal, with membrane filtration at pH 7.0 achieving TSS concentrations below 0.5 mg/L. The 1.2 μm filter paper at a high dose and pH 5.5 had TSS removals with an average TSS of 0.1 mg/L.

Figure 4-21 and Figure 4-22 show the total phosphorus and dissolved phosphorus results for ferric sulphate.

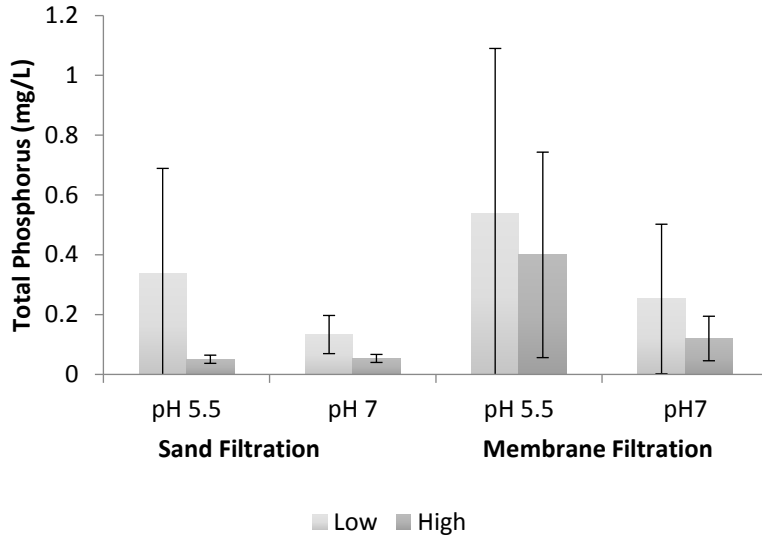


Figure 4-21 Total Phosphorus removal for ferric sulphate

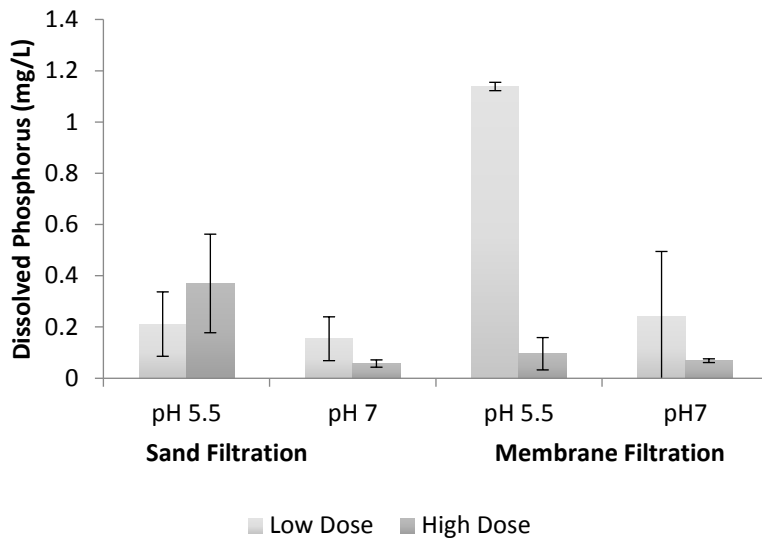


Figure 4-22 Dissolved phosphorus removal for ferric sulphate

The TP results for ferric sulphate followed a similar trend to that of the results for alum. The high dose at both pH 5.5 and pH 7 were able to meet the discharge regulations. As with the other coagulant trials, most of the phosphorus was in the dissolved form. Figure 4-23 shows the total aluminum results for the ferric sulphate trials.

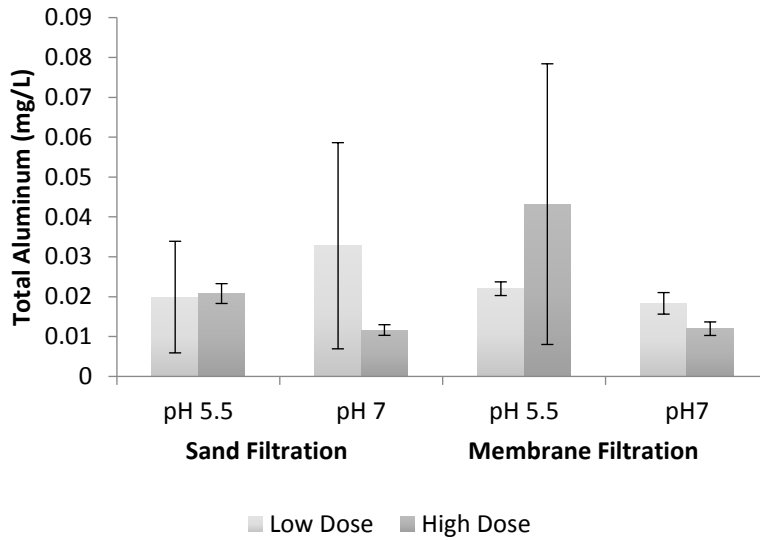


Figure 4-23 Total Aluminum results for ferric sulphate

The total aluminum concentrations in the effluent for ferric sulphate are a magnitude lower than those found for alum and PACl. This is expected as the previous two coagulants are aluminum based while ferric sulphate is iron based. If the plant is regulated in the future for total aluminum, ferric sulphate may be an alternative coagulant that could be used to limit it as it does not introduce more aluminum into the system.

While this jar testing experiment was not designed to optimize the removal of TSS and TP, it did show that there are operating conditions that would meet the discharge regulations. All trials were able to meet the TSS discharge regulation of 5 mg/L. Alum at pH 7 and a dose of 400 mg/L, PACl at the high dose and both pH except for membrane filtration at pH 6.2, and ferric sulphate at pH 7 for both doses was able to achieve TP below 0.13 mg/L. It was expected that the coagulants would perform better at the pH of minimum solubility, but this study showed that the regulation could also be met at pH 7, which is close to what the plant typically operates at, without pH control.

4.2.4 Factorial Design

The analysis in the previous section showed that under certain conditions, total phosphorus and TSS removal that met the AWWTF discharge regulations could be achieved. The following section will take into consideration that the experiment was completed as a factorial design and the analysis that goes with it. A factorial design was

selected as it allowed for the comparison of several factors (pH, coagulant dose and filtration type) for each coagulant. The factors are varied together in factorial design, unlike other strategies that look at the factors individually. This allows for the effect of the factor to be considered individually and in combination with the other factors. Each factor was evaluated at a low and high value, eliminating the need to perform the test at a range of factor values.

For this experiment, a 2^3 factorial design was completed for each coagulant. pH, coagulant dose and filtration were the factors considered individually and in combination. The results from the experiment were then analyzed with analysis of variance (ANOVA) to determine if the results were statistically different with varying the factors. A p-value of 0.05 was used to determine statistical significance. Minitab17 was used for the analysis. A preliminary model was then developed using Minitab, with factors or interactions that were determined to be the least statistically significant were removed. This procedure is outlined in Figure 4-24. Some of the trials had several factors or interactions that were significant while other trials had none.

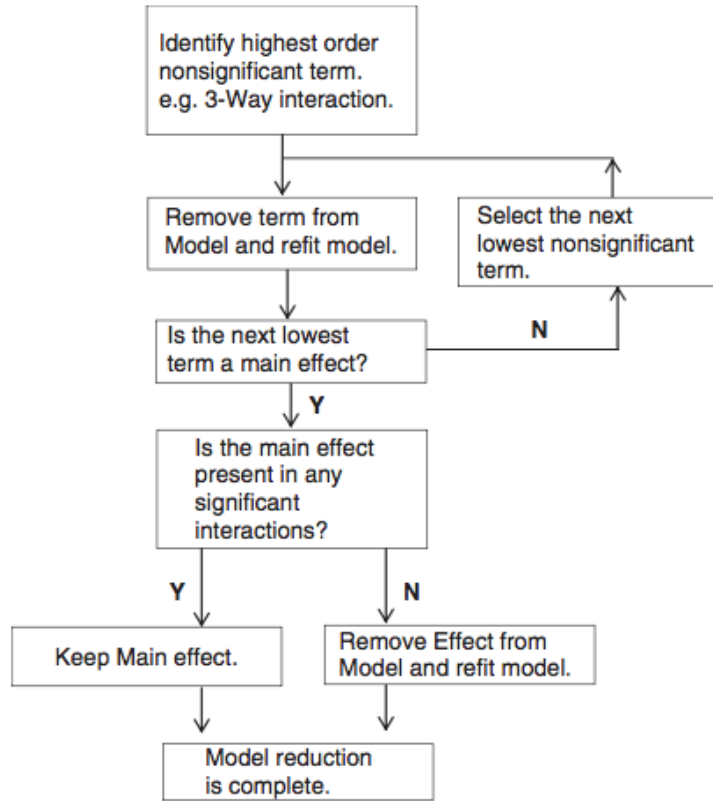


Figure 4-24 Flow chart for determining model for Minitab17 (adapted from Khan, R.M., 2003)

Looking at the alum results, a sample print out of the Minitab is included for the TSS analysis as seen in Figure 4-25. Initially, with all factors and interactions considered, none were statistically significant based on a p-value of 0.05. When the model was re-analyzed based on the procedure outlined in Figure 4-24, it was determined that statistically the interaction between dose and filtration type along with the interaction between pH and filter type were significant. Figure 4-26 shows the final Minitab printout for TSS. This can be seen from the results shown in section 4.2.1 as the results for membrane filtration were lower than sand filtration and that at pH 7, the higher coagulant dose achieved lower TSS concentrations. It would be expected that membrane filtration would play a significant role as it can act as a barrier up to 0.04 μm , while the filter that was used to represent sand filtration was 1.5 μm .

Factorial Regression: TSS versus dose, pH, Filtration

Analysis of Variance

| Source | DF | Adj SS | Adj MS | F-Value | P-Value |
|--------------------|----|---------|--------|---------|---------|
| Model | 6 | 12.6479 | 2.1080 | 3.59 | 0.384 |
| Linear | 3 | 0.7596 | 0.2532 | 0.43 | 0.775 |
| dose | 1 | 0.1116 | 0.1116 | 0.19 | 0.738 |
| pH | 1 | 0.1639 | 0.1639 | 0.28 | 0.691 |
| Filtration | 1 | 0.4841 | 0.4841 | 0.82 | 0.531 |
| 2-Way Interactions | 3 | 11.8883 | 3.9628 | 6.74 | 0.274 |
| dose*pH | 1 | 2.2578 | 2.2578 | 3.84 | 0.300 |
| dose*Filtration | 1 | 3.2169 | 3.2169 | 5.48 | 0.257 |
| pH*Filtration | 1 | 6.4136 | 6.4136 | 10.92 | 0.187 |
| Error | 1 | 0.5875 | 0.5875 | | |
| Total | 7 | 13.2355 | | | |

Model Summary

| S | R-sq | R-sq(adj) | R-sq(pred) |
|----------|--------|-----------|------------|
| 0.766504 | 95.56% | 68.93% | 0.00% |

Figure 4-25 Minitab17 Initial Factorial Regression for TSS results using Alum

Factorial Regression: TSS versus dose, pH, Filtration

Analysis of Variance

| Source | DF | Adj SS | Adj MS | F-Value | P-Value |
|--------------------|----|---------|--------|---------|---------|
| Model | 4 | 12.3724 | 3.0931 | 10.75 | 0.040 |
| Linear | 1 | 0.4841 | 0.4841 | 1.68 | 0.285 |
| Filtration | 1 | 0.4841 | 0.4841 | 1.68 | 0.285 |
| 2-Way Interactions | 3 | 11.8883 | 3.9628 | 13.78 | 0.029 |
| dose*pH | 1 | 2.2578 | 2.2578 | 7.85 | 0.068 |
| dose*Filtration | 1 | 3.2169 | 3.2169 | 11.18 | 0.044 |
| pH*Filtration | 1 | 6.4136 | 6.4136 | 22.29 | 0.018 |
| Error | 3 | 0.8630 | 0.2877 | | |
| Total | 7 | 13.2355 | | | |

Model Summary

| S | R-sq | R-sq(adj) | R-sq(pred) |
|----------|--------|-----------|------------|
| 0.536356 | 93.48% | 84.79% | 53.63% |

Figure 4-26 Minitab17 Final Factorial Regression for TSS results using Alum

For total phosphorus, it was determined that the effects of the factors dose and filtration were significant. Dissolved phosphorus showed that all the main factors and

interactions had p-values less than 0.05. Looking at the results shown in Figure 4-13 and Figure 4-14, it is shown that most of the phosphorus was in the dissolved form. Filtration type would be important as stated before due to the pore size of the membrane. The coagulation dose would be important as studies have shown that increase in coagulant dose results in increased phosphorus removal (Wang et al, 2014). Total aluminium concentrations saw that filter type was the significant factor in the trials, while for dissolved aluminum, pH was significant.

The design of experiment programming in Minitab also allows for a response optimization of the models developed for the factorial design. The goal of the optimization was to find the conditions that would provide the lowest concentrations of the target parameters in the effluent. For alum, it was determined by the model, that the high dose of 400 mg/L, at pH 7 with sand filtration would provide optimized effluent quality.

The analysis that was completed for Alum was completed also for both PACl and ferric chloride. The following section discusses the PACl results. The statistical results for TSS showed that dose, pH and the interaction between dose and pH were significant. This varies from the results that were found in the alum trials, where the interaction between dose and filter type along with pH and filter type. For both total and dissolved phosphorus, there were no factors or interactions that were determined to be significant. Total aluminum also did not have any factors or interactions that were significantly significant. pH and filter type were both significant factors for dissolved aluminum.

When the models for PACl were optimized using Minitab17, it was found that, similar to the alum, the higher dose and pH of 7 would provide the minimum concentrations for the parameters. For PACl it was found that membrane filtration rather than sand filtration would provide the better effluent quality.

The analysis of the ferric sulphate data showed that for TSS removal, both coagulant dose and coagulation pH were statistically significant. The interactions of dose and pH along with the interaction between pH and filter type were also significant. These interactions are highlighted in the results with the high dose and high pH having lower TSS concentrations than the other conditions and with the high dose and membrane filtration out performing the other conditions for those factors. There were no factors that

were statistically significant for both total and dissolved phosphorus for ferric sulphate. The main factor pH along with the interactions between dose and pH, pH and filter type and dose and filter type were significant for total aluminum concentrations while no factors were significant for dissolved aluminum. The concentrations of aluminum for the ferric sulphate trials was a magnitude less than those for Alum or PACl as it is not an aluminum based coagulant.

Optimization of the Minitab17 model for ferric sulphate determined that the high ferric sulphate dose, pH of 7.0 and membrane filtration would optimize the model. Table 4-1 summarizes the results of the factorial design statistical analysis. It can be seen that the factors or interactions that are significant to a parameter vary depending on the coagulant used.

Table 4-1 Factorial Design results

| Coagulant | Parameter | Significant factors or interactions |
|------------------------|--------------------|---|
| Alum | TSS | pH/filter type, dose/filter type |
| | TP | dose, filter type |
| | Dissolved P | N/A |
| | Total aluminum | filter type |
| | Dissolved aluminum | PH |
| PACl | TSS | dose, pH, dose/pH |
| | TP | N/A |
| | Dissolved P | N/A |
| | Total aluminum | N/A |
| | Dissolved aluminum | pH, filter type |
| Ferric Sulphate | TSS | dose, pH, dose/pH, pH/filter type |
| | TP | N/A |
| | Dissolved P | N/A |
| | Total aluminum | pH, dose/pH, pH/filter type, dose/filter type |
| | Dissolved aluminum | N/A |

Coagulant dose was a factor that was statistically significant for several of coagulants and parameters considered or was significant in an interaction of the factors. This would be expected as it not only has an effect on the removal of the target parameter, but also on the pH of the wastewater. Adding a higher dose of coagulant will consume more alkalinity in the wastewater and will then cause a drop in the wastewater pH. Depending on the coagulant used, the basicity of the coagulant varies and the change in pH after addition to the wastewater will vary. The species that are formed during

precipitation are based on the pH of the wastewater, so the pH will factor into if the species necessary for precipitation are formed.

Increasing the coagulant dose has been shown in literature to increase the removal of phosphorus from wastewater (Wang et al. 2014, Song et al, 2008, Szabo et al, 2008), which was seen in this study as well. Increasing coagulant dose increases the metal to phosphorus ratio which would increase the surface area available for removal by adsorption. But, according to Rittmann et al (2011), it is the orthophosphate portion of phosphorus that can be removed by adsorption and precipitation and would see increased removal with an increased coagulant dose. Polyphosphates and organic phosphorus can be removed by adsorption, but the removal is limited. Polyphosphates and organic phosphorus is typically removed through biological uptake or is converted to and removed as orthophosphates. Having the optimal coagulant dose for phosphorus removal in wastewater is important as adding too much coagulant could affect the biological phosphorus removal, which would result in residual phosphorus concentrations that would not be removed by coagulation and would be in the treated effluent (Wang et al, 2014).

Increasing the coagulant dose can lead to increases in the TSS of the wastewater, as more precipitates would be formed. As the coagulant dose increases, it will reach a point where the addition of coagulant will not significantly increase the removal of a target parameter, and the efficacy of the coagulant will go down (Song et al, 2008). The excess coagulant will contribute to the TSS concentration and would have to be removed through filtration. This could lead to more frequent cleanings to be required for both the sand filtration or the membrane filtration. Depending on the precipitate species that are formed, they could be in the dissolved form and would pass through the filters. If the coagulant was aluminum based, this could lead to an increase in aluminum concentrations in the effluent and would make it more difficult to meet discharge regulations if they were put in place.

It was expected that the coagulants would perform better at the pH of minimum solubility based on literature (Metcalf and Eddy, 2003). But in most cases, the trials at pH 7 out performed those at the pH of minimum solubility for the respective coagulant. Szabo et al (2008) also found that they were able to get adequate phosphorus removal

between the range of pH 5.5-7 and not just at the pH of minimum solubility in wastewater. They concluded that factors such as mixing and hydraulics of the system would play a larger role than pH. At a pH in the range of 5.5-7 the species necessary for precipitation are formed but just not in the concentration as at pH of minimum solubility and the process still will take place.

For the coagulant doses that were used in this study, it is likely that sweep coagulation was the mechanism for removal. Sweep coagulation can occur when high doses of coagulant are added. It can be less depended on pH as there is excess coagulant that can be used to sweep up the particles. This could be one reason why pH was not found to be significant for the trials.

Filter type was considered in this study to align with the rest of the project comparing the SBR with the pilot scale MBR as they had different filtration systems. 1.5 μ m filter paper was used to represent the sand filtration of the SBR while a bench-scale UF membrane was used to represent the MBR, with a pore size of 0.04 μ m. This difference in size would allow smaller particles between the size of 0.04 μ m and 1.5 μ m after coagulation to pass through the filter paper, but could be removed from the effluent by the membrane. While there would not be much difference between the filters if the particles are bigger than 1.5 μ m as both would remove the particles, if the majority of particles were between 0.04-1.5 μ m there could be a difference in the final effluent quality. Filter type was a statistically significant factor for total aluminum in the Alum trials. For the trials, the dissolved aluminum concentration was between 75-99% of the total aluminum concentration for all the conditions except for pH 7 with sand filtration. This means that the majority of the aluminum was dissolved and less than 0.45 μ m. Membrane filtration was able to provide the lower aluminum concentration in the effluent as it was able to remove more of the dissolved aluminum.

Each coagulant for this study had a variation of factors or interactions that were statistically significant to the effluent concentration of TSS, phosphorus and aluminum. This means that the optimization process is specific to the coagulant and that jar testing should be completed regularly to ensure that the necessary factors are considered when coagulation is used at the full scale.

Based on the results found, all three coagulants were able to meet the discharge regulations at least one of the operating conditions considered. Decisions for coagulant choice for application at a full scale should consider the cost of the coagulant, operator familiarity of the coagulant and sludge production along with the factors determined in this study.

Chapter 5 CONCLUSIONS AND RECOMMENDATIONS

The objectives of this research project were to:

1. Design, construct and commission a Membrane Bioreactor (MBR) pilot plant in parallel to a full-scale Sequencing Batch Reactor (SBR) wastewater treatment facility.
2. Evaluate the MBR pilot plant in parallel to the full-scale SBR plant with regards to TSS, BOD, COD, ammonia and total phosphorus removal efficacy.
3. Identify operating parameters that impact phosphorus removal efficacy in wastewater treated with coagulation through the use of factorial design of bench-scale jar tests.

The key findings of the project are as followed:

- The importance of stable operating conditions was evident from the commissioning phase of the pilot plant. Loss of mixed liquor suspended solids (MLSS) in the system due to spills and not being able to seed the MBR with activated sludge with a MLSS typical of that found in research, aligned with increases in BOD and COD that were seen in the pilot plant effluent.
- Adequate time is required by microorganisms to acclimatize and reach stable operating conditions following seeding the pilot plant for biological nitrogen and phosphorus removal.
- The pilot plant was able to meet the discharge regulations for BOD, TSS and ammonia. The full scale SBR was able to meet the effluent regulations for BOD, TSS and ammonia and was able to out perform the pilot plant MBR for TP and COD. Neither the full scale SBR or the pilot plant MBR was able to achieve the 0.13 mg/L TP limit.
- Jar testing showed that under the conditions considered, all three coagulants that were tested were able to achieve TP values below 0.13 mg/L and that all trials met the TSS guideline.

- Each target parameter considered had different factors that were significant depending on the coagulant considered. For phosphorus removal, dose and filter type were statistically significant for alum. No factors were found to be significant for phosphorus removal for PACl or ferric sulphate.

5.1 Recommendations for Further Study

Research of MBRs for municipal wastewater treatment is important as the popularity of the treatment process increases and as effluent regulations increase. Research has been in the past, typically performed at bench-scale and more needs to be completed at pilot and full scale.

For this study, the pilot plant was operated for less than 90 days and during that time period, there were several pilot-plant upsets which took the system offline. One recommendation would be to run the pilot plant for a longer continuous period of time. This would allow for better acclimatizing of the activated sludge when the pilot plant is seeded. It would also allow for the pilot plant to be operated during different seasons and influent wastewater qualities. Other bench- and pilot-scale studies in literature typically reported at least 162 days (Xing et al, 2000; Santasmasas et al 2013; Pollice et al, 2007) or longer for testing.

The MBR pilot plant was operated at a constant coagulant dose and without pH control or adjustment. Further research could include varying the coagulant dose and type used in the pilot plant based on preliminary jar testing. The location of coagulant addition along the treatment train was held constant. Moving the coagulant addition to the pre-anoxic tank or just prior to the membrane filtration or having multiple addition points to assist with phosphorus removal could be considered.

AWWTF is regulated with discharge regulations for total nitrogen of 8 mg/L. MBR plants have been shown to be able to achieve less than 5 mg/L TN in the effluent. Further operation of the pilot plant should include TN measurements to see if this could be achieved at the site.

Membrane fouling was not considered during this study. Further research on coagulant use in a MBR could look at in combination the effect on membrane fouling and

phosphorus removal. In addition, the effect of adding a coagulant on biological removal of phosphorus and nitrogen could be studied.

Studies have shown that the mixing speed and contact time for coagulation can also be important factors for total phosphorus removal during jar testing. Varying the jar testing procedure to determine optimal operating conditions could be further studied. Jar testing could also be completed to determine an optimal coagulant dose for phosphorus removal.

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APPENDIX A: SOLIDS RETENTION TIME CALCULATION

The equation for Solids Retention Time (SRT) is:

$$SRT = \theta_x = \frac{VX}{(Q-Q_w)X_e + Q_w X_r}$$

where SRT= solids retention time (days)

V= volume of tank (m³)

X= concentration of biomass in tank g VSS/m³

Q= influent flowrate (m³/day)

Q_w= waste sludge flowrate (m³/day)

X_e=concentration of biomass in the effluent g VSS/m³

X_r= concentration of biomass in the return activated sludge g VSS/m³

The average SRT for the pilot plant MBR was calculated as followed:

Volume of tanks :

Pre-anoxic – 0.22 m³

Aeration – 0.405 m³

Post-anoxic – 0.25 m³

Membrane tank – 0.027 m³

Total volume – 0.902 m³

Typical solids concentrations in the tanks:

Pre-anoxic – 3263 mg/L

Aeration – 1227 mg/L

Post- Anoxic – 1076 mg/L

Membrane tank – 1710 mg/L

Q= 0.355 m³/day

Q_w= 0.02m³/day *wasting did not occur every day, but this is an average including the loss of solids due to upsets of the pilot plant

X_e= 1mg/L

X_r = 1710 mg/L

$$SRT = \Theta_x = \frac{VX}{(Q-Q_w)X_e + Q_w X_r}$$

$$SRT = \frac{[(0.22m^3 \cdot 3262 \frac{mg}{L}) + (0.405m^3 \cdot 1227 \frac{mg}{L}) + (0.25m^3 \cdot 1076 \frac{mg}{L}) + (0.027m^3 \cdot 1710 \frac{mg}{L})]}{(0.355m^3 - 0.02m^3) \cdot 1 \frac{mg}{L} + 0.02 \frac{mg}{L} \cdot 1710 \frac{mg}{L}}$$

$$SRT = \frac{1529.7 m^3 \frac{mg}{L}}{(34.5 \frac{m^3 mg}{day L})} = 44.3 \text{ days}$$

APPENDIX B: PILOT PLANT DISRUPTIONS

A list of disruptions that occurred during the operation of the pilot plant. It includes tank overflows, unexpected shut downs and equipment malfunctions.

| Date | Notes |
|------------------|---|
| June 11-12, 2016 | Pre-anoxic tank overflowed Sump pump supplying influent wastewater from AWWTF EQ tank shut down over weekend |
| June 13, 2016 | Pipe from pre-anoxic tank to aeration tank was clogged, causing the pre-anoxic tank to overflow Aeration tank was not being mixed as the blower had been turned down to attempt to lower DO levels |
| June 14, 2016 | Membrane pump speed was set too low, resulting in the levels in the membrane tank and post anoxic tank to equal out and the post anoxic tank overflowed |
| June 15, 2016 | Post-anoxic tank was overflowing, so AWWTF operator shut down flow to the pilot plant. Tubing between the post-anoxic tank and membrane tank was inspected for clogs and cleaned. |
| June 16, 2016 | Recirculation pump for RAS was not turned on after membrane clean |
| June 17, 2016 | Post-anoxic tank overflowed, so AWWTF operator turned off flow to the pilot plant. Ammonia spiked in pilot plant effluent. Pilot plant was taken offline for the weekend |
| June 20, 2016 | Pilot plant was drained and re-seeded with AWWTF SBR activated sludge. Membrane and pumps were cleaned. |
| June 21, 2016 | Membrane tank overflowing due to air scour and flow in to tank |
| June 22, 2016 | Membrane tank overflowing from top. Hole was drilled in top of membrane tank to allow air from air scour to be released |
| June 24, 2016 | Post-anoxic tank has layer of foam on the top, issues with mixing in the tank |

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| June 27, 2016 | <p>Recirculation pump tubing was punctured over the weekend, resulting in loss of WW (pre-anoxic tank was ½ full). Influent WW pump was clogged, reducing flow into the plant.</p> <p>Thick foam on post-anoxic tank</p> <p>Pilot plant was shut down over night by AWWTF operator to prevent overflowing of tanks</p> |
| June 28, 2016 | <p>Recirculation pump tubing was punctured; pump head appears to be destroying tubing quickly. Replaced tubing with different tubing</p> |
| June 29, 2016 | <p>New tubing for recirculation pump was restricting flow so it was replaced with the previous type of tubing</p> <p>Membrane tank was slightly overflowing</p> |
| June 30, 2016 | <p>Pilot plant was turned off by AWWTF to prevent overflowing. It remained off for the long weekend to prevent overflows</p> |
| July 5, 2016 | <p>The pilot plant tanks were drained as the WW had turned septic. Re-seeded with AWWTF SBR activated sludge</p> |
| July 7, 2016 | <p>Recirculation pump making squeaking noise, tubing needed to be replaced</p> |
| July 8, 2016 | <p>Alum pump lost prime so it was not added to post-anoxic tank</p> <p>Recirculation pump tubing needed to be replaced</p> |
| July 11, 2016 | <p>Membrane tank and permeate tank were empty upon arrival. Clog in tubing between the aeration tank and post-anoxic tank</p> <p>When membrane was being cleaned, one valve was open slightly and may have let the sodium hypochlorite solution enter the pilot plant</p> <p>Recirculation pump tubing needed to be replaced</p> |
| July 12, 2016 | <p>Transmembrane pressure (TMP) was showing 0-5kPa and no WW in the permeate tank. Waste valve for permeate tank was open, resulting in the empty tank</p> |
| July 14, 2016 | <p>Alum tank was still full, pump shut down over night so Alum was not added. Turned the pump alum pump on</p> |
| July 18, 2016 | <p>Some MLSS was lost from membrane tank and pre-anoxic tank when adjusting tubing</p> |

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| | <p>MicroC (carbon source for denitrification in post-anoxic tank) was added</p> <p>Influent WW pump was shut off due to power outage, preventing flow into the plant</p> |
| July 19, 2016 | Sump pump in the AWWTF EQ tank was not working and had to be cleaned, resulting in low levels in the tanks |
| July 20, 2016 | <p>Post-anoxic tank overflowed, due to the influent pump for the pilot plant being left on max after previous day's attempt to refill the tanks</p> <p>Had to drain the post-anoxic and membrane tanks a little bit to prevent overflowing</p> <p>Bubbles appeared in the aeration tank after the membrane clean</p> <p>High TMP (> 50 kPa) so swapped out the membrane with an older one and soaked it over night to clean it</p> |
| July 22, 2016 | <p>Membrane tank was foamy so it was rinsed out</p> <p>Pumps in pre and post- anoxic tanks for mixing were cleaned</p> <p>MicroC was removed from the pilot plant trials (as it increased the effluent COD to 1700 mg/L)</p> |
| July 25, 2016 | <p>Influent WW pump was clogged over the weekend so no flow into the pilot plant</p> <p>Membrane was severely caked with sludge (wood like), had to be removed and cleaned</p> |
| July 27, 2016 | <p>Membrane tank overflowed (loss of MLSS)</p> <p>Sump pump in the AWWTF EQ tank was clogged, so no flow to the pilot plant</p> <p>Post-anoxic tank looks dark in color. The consistency of the sludge in the membrane tank is runny</p> |
| July 28, 2016 | Membrane tank overflowing with thick sludge and post-anoxic levels were high, tubing between post anoxic tank and membrane tank may be clogged |

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| August 2, 2016 | Sump pump in EQ tank was tripped on the weekend (shut off) so no flow into the pilot plant. Line from aeration tank was clogged |
| August 8, 2016 | Foam formed on the surface of the pre-anoxic tank Influent WW pump was clogged and was cleaned Post-anoxic tank had dark, slimy sludge on the sides of the tank, so the post-anoxic tank was drained to prevent it from going septic. This resulted in a loss of MLSS |
| August 9, 2016 | No permeate in the permeate tank, membrane tank was empty Sump pump in the EQ tank shut off |
| August 10, 2016 | Membrane pump was not priming Membrane waste valve was open and led to loss of MLSS in membrane tank as it was empty |
| August 11, 2016 | Post-anoxic tank was overflowing and membrane tank was leaking (recirculation line was bumped at base of membrane tank) Fuse tripped for air blower so no air going to the aeration tank |
| August 12, 2016 | Line from aeration tank was clogged so it was overflowing Activated sludge in aeration tank looks weak (lack of MLSS) |
| August 15, 2016 | Membrane tank was low Aeration tank and post-anoxic tank look weak (lack of MLSS) |
| August 18, 2016 | Membrane doesn't appear to be pulling the permeate through |
| August 23, 2016 | Tanks appear to have no solids (appear clear when sampling) So the tanks were drained and re-seeded. There was a build up of solids on the sides of the tanks. The hydraulic jack that supports the pre-anoxic tank was moved, effecting the flow through the tanks. |
| August 24, 2016 | Line from aeration to post-anoxic tank was clogged, causing the aeration tank to fill up Influent WW pump was clogged so it had to be cleaned |
| August 25, 2016 | Sump pump in EQ was disconnected so there was no flow to the pilot plant |

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| August 29, 2016 | Loss of MLSS over the weekend Influent WW pump clogged and needed to be cleaned |
| August 30, 2016 | Loss of MLSS from post-anoxic and membrane tank Membrane was caked with sludge and had to be cleaned Added some solids from the SBR plant (did not drain the tanks) |
| August 31, 2016 | Recirculation pump line was clogged |
| Sept. 1, 2016 | Influent WW pump was clogged Membrane tank was low |