AN EVALUATION OF BENCH AND PILOT-SCALE PRIMARY WASTEWATER TREATMENT PROCESSES TO MEET MORE STRINGENT WASTEWATER SYSTEM EFFLUENT REGULATIONS

by

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Submitted in partial fulfillment of the requirements
for the degree of Master of Applied Science

at

Dalhousie University
Halifax, Nova Scotia
April 2022

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ABSTRACT

Under the Fisheries Act, the Wastewater Systems Effluent Regulations (WSER) set Canada's national wastewater effluent quality standards. The discharge effluent of wastewater treatment must not exceed 25 mg/L of total suspended solids (TSS) and 25 mg/L of carbonaceous biochemical oxygen demand ($cBOD_5$). These WSER limits indicate the use of secondary biological treatment, but approximately 50% of wastewater facilities in Nova Scotia are primary treatment facilities. Wastewater treatment facilities not meeting the standards as of 2015 required Transitional Authorization and had until the end of 2020, 2030, or 2040 to make system upgrades.

Dartmouth Wastewater Treatment Facility currently operates a chemically enhanced primary wastewater treatment plant with Transitional Authorization until 2040. While this system does not have secondary treatment, it historically achieved effluent water quality near the WSER standards. Accordingly, the objective of this research was to investigate optimization options for existing infrastructure that could allow it to meet national effluent quality standards consistently. The optimization was achieved through bench-scale testing to determine optimum coagulant and polymer chemical dosing and pilot-scale studies that investigated the use of ballast material to further enhance the primary clarification process.

The bench-scale chemical optimization study found a 2 to 4 mg/L increase in coagulant dose (i.e., Aluminum sulfate 5 to 7 mg/L) and a 0.25 mg/L increase in flocculant dose (i.e., 1.25 mg/L of polymer) could improve water quality, as measured by turbidity. Additionally, it was determined that chemical optimization could not consistently improve the percent turbidity removed when the facility's hydraulic retention time (HRT) was around 24 minutes or less and the influent turbidity was less than 90 NTU.

The pilot-scale ballasted flocculation study identified that the addition of ballast could achieve $cBOD_5 < 40$ mg/L in the treated effluent and outperform the full-scale plant in terms of $cBOD_5$ removal. The average TSS reduction for the pilot-scale ballasted flocculation study was 79-85%. Further, the optimal coagulant dose identified through pilot trials confirmed the optimal dose range determined at the bench-scale.

These findings suggest that optimizing chemicals and adding ballast to the chemically enhanced primary treatment process would improve effluent water quality, but not enough to meet the WSER $cBOD_5$ limit because of the emerging soluble $cBOD_5$**.**

LIST OF ABBREVIATIONS AND SYMBOLS USED

Alum	liquid aluminum sulphate, $Al_2(SO_4)_3 \cdot 14H_2O$
$cBOD_5$	Carbonaceous Biochemical Oxygen Demand – 5-day
CCME	Canadian Council of Ministers of the Environment
CEPT	Chemically Enhanced Primary Treatment
CS	Combined Sewer
CSO	Combined Sewer Overflow
DO	Dissolved Oxygen
DW AFD	Dry Weather Average Daily Flow
DWWTF	Dartmouth Wastewater Treatment Facility
EWT	Evoqua Water Technologies
Floc	Flocculant aggregates micro flocs that settle out of the water
HRT	Hydraulic Retention Time
m^3/hr	Meters cubed per Hour
mg/L	Milligrams per Liter
MLD	Megaliters per Day
NTU	Nephelometric Turbidity Unit

PF	Peak Flow
Q	Flow rate
RPM	Revolutions Per Minute
SA	Surface Area
scBOD$_5$	Soluble Carbonaceous Biochemical Oxygen Demand – 5-day
SOR	Surface Overflow Rate
TSS	Total Suspended Solids
μ**m**	Micrometer
UV	Ultraviolet
UVT	Ultraviolet Transmittance
V	Volume
WSER	Wastewater Systems Effluent Regulations
WW AFD	Wet Weather Average Daily Flow

ACKNOWLEDGMENTS

Firstly, I would like to thank my supervisor, Dr. Amina Stoddart, who has been encouraging and patient throughout this process. Thank you for allowing me to continue my journey in wastewater treatment. Your welcomeness as I explored the option of this program encouraged me. It gave me confidence in my capability as an engineer and person to tackle the difficulties of graduate school. Thank you for your guidance and for introducing me to new topics that were previously beyond my comfort.

Thank you for your insights and advice to keep me focused on “the story” of my research to my committee members, Dr. Margaret Walsh and Dr. Su-Ling Brooks. Your advice improved my comprehension of academic research. Special thanks to Heather Daurie and the Centre for Water Resource Studies staff for your guidance in lab technique and knowledge of equipment. Special thanks to Lindsay Anderson, Gillian Stanton, Maggy McGrath, Maddison Gouthro, and Reilly Pickard. To the rest of the students, thank you for the friendship and encouragement. Graduate school in pandemic conditions wasn’t easy, but you made it enjoyable. Your knowledge and expertise as a collective group are remarkable. Thank you to the students who helped me run the thousands of tests. Without your help, I could not have accomplished the volume of tests ran.

I want to acknowledge the NSERC/Halifax Water Collaborative Research & Development Award and the 2040 Horizon NSERC/ Halifax Water IRC and all its partners for their financial support. Thank you to the ASPIRE CREATE program for their financial support and guidance in professional development.

Thank you to Chris Fahie and Mike Deagle of Halifax Water and the Dartmouth Wastewater Treatment Facility staff. Thank you for your knowledge and guidance of the facility and your passion for cleaning effluent water. I would like to acknowledge Evoqua Water Technologies for their advice and use of their CoMag® pilot system. A special thank you to Cory Brody. You taught me the hands-on application and troubleshooting aspect that wastewater treatment requires. Your help and guidance made me feel confident in my ability to run the pilot and future equipment troubleshooting.

Finally, thank you to my family and friends. Your support did not go unnoticed. Thank you for your patience, babysitting services, food, and encouragement. A special thank you to my husband, Adam Lowe. Your support, reassuring confidence in me, sacrifice, and love made this journey possible. I appreciate everything you do

CHAPTER 1 INTRODUCTION

1.1 Rationale

1.1.1 The Creation of National Effluent Quality Standards for Wastewater Treatment Systems in Canada

In 2013, an estimated 25% of wastewater treatment facilities in Canada required billion-dollar capital upgrades (Dyck and Abrahamson 2013). The Wastewater Systems Effluent Regulations (WSER), issued under the Fisheries Act, created national effluent water quality standards for wastewater treatment systems across Canada(Government of Canada 2012). However, when the WSER came into authorization in 2015, it left 25% of operational wastewater facilities out of compliance (Dyck and Abrahamson 2013). The estimated total cost to have all systems in compliance was expected to be $5.5 billion over 54 years (Dyck and Abrahamson 2013). However, it was expected that the resulting improvement in effluent quality would, in turn, create healthier aquatic systems, lower drinking water system costs, increase recreational use, and increase property values, with an estimated quantifiable value of $16.5 billion (Dyck and Abrahamson 2013).

Beyond protecting receiving bodies of water from the potentially deleterious effects of wastewater effluent, the WSER intended to create a standard approach to wastewater effluent requirements for baseline environmental protection and standard reporting (Environment of Climate Change Canada 2018). This standardization process and environmental baseline protection have been recommended since the mid-1990s to promote equity for all of Canada (Chambers et al. 1997). The new effluent water quality standards, established by WSER, set discharge effluent limits of 25 mg/L of total suspended solids (TSS) and 25 mg/L of carbonaceous biochemical oxygen demand ($cBOD_5$) for applicable systems (Government of Canada 2012). These limits are intended for intermittent and continuous systems where 100 m^3 or more influent enters the system. Exclusions included systems in the Northwest Territories, Nunavut, north of 54th Parallel in Quebec or Newfoundland and Labrador, on-site industrial or institutional facilities, and pulp and paper mills (Government of Canada 2012).

Additionally, the averages for reporting purposes may either be composite or grab samples depending taken three times weekly or once monthly depending on the average daily effluent volume (Government of Canada 2012). These effluent parameters implied using a biological (secondary) wastewater treatment process. Secondary treatment is defined as using a biological process to further degrade the influent contaminants after a primary process and is the most common treatment practice in North America (Morris et al. 2016).

The bulk of the financial burden to upgrade the 850 existing facilities to meet the WSER standards was on the municipalities they operated (Dyck and Abrahamson 2013). Acknowledging the time and financial considerations required to upgrade existing systems or construct new designs to meet the national effluent quality standards, Transitional Authorizations were created to allow systems to continue discharging treated effluent and enough time for capital upgrades to be made. Transitional Authorizations were issued based on a point system of low, medium, and high ecological risk to the receiving water of each facility. The facilities were allowed to operate at their current discharge limits, but the new effluent standards would need to be met by 2020, 2030, or 2040 depending on ecological risk (Government of Canada 2012).

1.1.2 Impact on Nova Scotia and Opportunities for Treatment Optimization

Approximately 50% of wastewater facilities in Nova Scotia are primary treatment facilities (Environment of Climate Change Canada 2018), suggesting the need for significant treatment optimization and potential capital upgrade requirements for wastewater treatment facilities in the province.

The largest water and wastewater utility in the province, Halifax Water, operates three nearly identical (apart from scaling for flow) chemically enhanced primary treatment (CEPT) systems that discharge into the Halifax Harbor. CEPT does not include a biological treatment process and relies only on physical and chemical processes to remove wastewater contaminants. The chemical process uses metal salts with a positive charge (coagulant) to attract negatively charged particulates in water and form micro flocs known as the process coagulation. It is followed by flocculation, where a polymer with large chains (flocculant) aggregates the micro flocs to form larger flocs that become dense enough to settle out of the water (Bratby 2016). The settling stage is a physical process known as clarification. In CEPT systems, clarification occurs in clarifiers that have unique physical traits and/or use the addition of ballast (either organic or inorganic) to shorten settling time (Johnson, Ferguson, and Linda 2005). One of these facilities (Herring Cove) could achieve the WSER standards because the facility still has the capacity for population growth and treats less wastewater than it was designed to treat. The two remaining facilities were not expected to accomplish the WSER standards by 2015 without optimization and/or system upgrades. These facilities (Dartmouth and Halifax) were considered low ecological risk and were granted Transitional Authorization to continue operating at their current discharge limits until 2040. Expressly, the existing

Permit to Operate for the Dartmouth Wastewater Treatment Facility (DWWTF) stipulated effluent requirements of 40 mg/L of TSS and 50 mg/L of cBOD$_5$ in a 24-hour composite sample (Halifax Harbour Solutions 2003).

While the WSER standards for cBOD$_5$ and TSS indicate secondary treatment, CEPT is often used to treat wastewater collected from combined sewers because of the high flows after a storm event (Peters and Zitomer 2021). The DWWTF treats municipal, industrial, and stormwater because it is a combined sewer wastewater collection system. Combined sewer (CS) plants have the unique problem where weather significantly impacts the hydraulic retention time (HRT) of the facility, and the treatment process becomes hydraulically overburdened to the point where untreated water enters the receiving body of water (Peters and Zitomer 2021). CEPT facilities have previously been a management option for the CS collection system. Still, new discharge limits and climbing influent contaminants may make CEPT for CS no longer a feasible option (Shewa and Dagnew 2020).

Despite relying on primary treatment and the challenges associated with treating CS, the Dartmouth WWTF has historically achieved a quarterly average effluent cBOD$_5$, and TSS near the WSER required 25 mg/L. As effluent water quality was already near the new required limits, there is an opportunity to investigate optimization and process changes to existing infrastructure to potentially avoid the cost associated with extensive upgrades for a secondary biological process.

1.2 Objectives

The focus of the study was to develop a framework for the chemical dosing optimization of CEPT plants and analyze the use of ballast to improve existing CEPT systems. Optimization studies were conducted at the bench and pilot-scales. The chemical optimization and ballasted flocculation studies were conducted to better understand the chemically enhanced process and investigate the potential for CEPT facilities to achieve the WSER standards without the capital costs associated with implementing a secondary treatment process. This study may influence other CS CEPT plants' capital upgrade processes as they may also face influent soluble contaminant increases and require lower discharge limits. The following specific objectives were addressed:

1. Determine the optimal combination of coagulant and polymer dose at variable influent turbidities and hydraulic retention times to determine if chemical optimization alone can enable CEPT facilities to achieve the WSER TSS and $cBOD_5$ effluent water quality requirements.

2. Investigate the addition of an external ballast material to improve the settling efficiency of CEPT to assist in achieving the WSER TSS and $cBOD_5$ effluent water quality requirements.

1.3 Organization of Thesis

The project rationale and relevant background information on water quality parameters, limitations of combined sewer overflows (CSO), chemically enhanced primary treatment systems, and ballasted clarification are described in Chapter 2. Chapter 3 provides a detailed description of the materials and methods used throughout the study. Chapter 4 presents the results and discussion of the bench-scale study performed to optimize the coagulant and flocculant chemical dose, followed by the results and discussion of the pilot-scale investigation of the use of magnetite as a ballast material to improve treatment and settling. Finally, Chapter 5 presents the overarching conclusions from the research and suggests recommendations for future study of this topic.

CHAPTER 2 BACKGROUND

2.1 Municipal Wastewater Composition

The composition of municipal wastewater comprises of sanitary sewage from homes, including human waste from toilets and gray water from showers and sinks, and any water that is used and poured down a drain. Beyond wastewater from homes is the industrial waste from businesses and organizations. The wastewater loading can be described as low, medium, and high strength in terms of water quality values used to measure wastewater. Table 2.1 depicts the loadings strength found in typical municipal wastewater, where total suspended solids (TSS) is the total amount of suspended solids, and carbonaceous biochemical oxygen demand (cBOD$_5$) is the amount of oxygen required to degrade carbonaceous organic matter (Tchobanoglous et al. 2003). Other water quality parameters monitored may include turbidity and ultraviolet transmittance (UVT). Turbidity is correlated to TSS but allows the plant to monitor the system in real-time (Ratnaweera and Fettig 2015). UVT may also be used as a real-time monitor but is a more critical value when the disinfection process of treatment is UV radiation (Lepot, Aubin, and Bertrand-Krajewski 2013).

Table 2.1 Loadings of municipality wastewater (adapted from Tchobanoglous et al. 2003)

Parameters	Low	Medium	High
TSS (mg/L)	**120**	**210**	**400**
cBOD$_5$ (mg/L)	**110**	**190**	**350**
Approximate wastewater flow rate (L/capita)	**750**	**460**	**240**

As depicted in Table 2.1, the influent composition is a function of flow rate. Low strength is when there is more water, and the wastewater is diluted compared to high strength. Over the course of a day, the short-term wastewater strength entering a facility will change as anthropological activities occur (Tchobanoglous et al. 2003). In older established cities, sewage, and urban runoff (i.e., stormwater) were combined in one collections system (i.e., combined sewer) (Tibbetts 2005).

2.2 Combined Sewer

In 2017, Municipal Wastewater Systems in Canada reported 164 combined sewers (CS) across Canada (Canada 2017a). Combined sewers are highly affected by the weather and seasonal variance. All urban runoff enters the same sewage collection system during precipitation events and enters the wastewater treatment facility. CS systems reside in urban developments built before 1940 (Chambers et al. 1997). The typical range of influent TSS of combined wastewater during a precipitation event is between 270 to 550 mg/L, and the $cBOD_5$ is between 60 to 220 mg/L (Tchobanoglous et al. 2003). During peak flow, the $cBOD_5$ concentration is lower, indicating that the increase of stormwater dilutes municipal wastewater concentration. Unlike $cBOD_5$, the TSS concentration rises slightly as the collection system undergoes a “flush” and all the surface debris enters the collection system (Tchobanoglous et al. 2003).

Occasionally, if the flow is too great, excess combined wastewater may discharge without treatment into surrounding water bodies at overflow locations, known as a combined sewer overflow (CSO). Although these CSO events only accounted for 4.4% of untreated wastewater entering water bodies in Canada in 2017, between 27% and 56% of the overflow combined wastewater is sanitary sewage (Canada 2017b; Soonthornnonda and Christensen 2007; Phillips et al. 2012; Launay, Dittmer, and Steinmetz 2016). Compared to inland Canada, most CSO events occurred in the coastal regions because the effluent standards were, historically, not as stringent when entering marine bodies compared to freshwater (Canada 2017b; Chambers et al. 1997).

A wastewater treatment facility with a combined sewer collection must account for high peaks and various loadings to properly treat the wastewater. CEPT is a common solution for combined sewers for its capability to treat high flow rates that would otherwise wash out the microorganisms required for a biological treatment process (Shewa and Dagnew 2020; Bratby 2016; Peters and Zitomer 2021). The chemical optimization of coagulant and coagulant aid for CEPT plants at CS WWTF increased the removal efficiency for suspended solids, turbidity, and $cBOD_5$ (Dong et al. 2019; Alameddine et al. 2021). The study done at the Edmonton WWTF expected the alum dose range to be between 5 and 10 mg/L (Alameddine et al. 2021).

2.3 Primary Treatment

The first step of any treatment is screening which removes large debris that enters the treatment facility. The main principle of primary treatment is sedimentation (Mackenzie L. Davis P.E., BCEE 2020). Four types of sedimentation can occur depending on the particulate's concentration and interaction between particles, as seen in Figure 2.1 (Carlsson 1998).

1. Discrete particle settling. Particle settles despite low concentration and no particle interaction.

2. Flocculant settling. Particles begin to settle but flocculate together as particles settle deeper into the basin
3. Hindered settling. The interaction between particles is great enough that settling becomes hindered.
4. Compression Settling. The particle concentration is so great that the higher particles influence the lower particles.

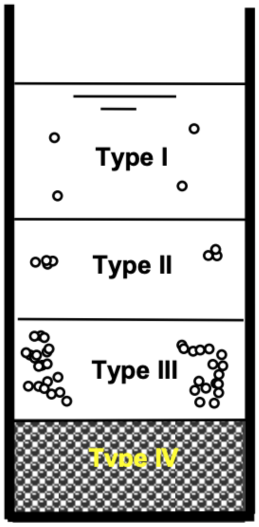


Figure 2.1 Represents the four types of sedimentation as the particulates settle (Ghawi 2008).

2.3.1 Clarification

The method in which sedimentation is optimized in primary treatment is with the design of clarifiers. Clarifiers maximize the sedimentation process and are designed based on the hydraulic retention time of the system, the velocity of water over the surface area (SOR), and the settling velocity of its particulates. The likely dominant type of sedimentation in clarifiers is “type 2” flocculant settling, and the controlling design parameter is the particle's settling velocity (Mackenzie L. Davis P.E., BCEE 2020).

2.3.1.1 SOR and settling velocity

The settling velocity must be greater than the SOR for sedimentation to occur. A settling column is used to determine the settling velocity of the particles in treatment. The theoretical settling velocity is determined using Stokes Law (Equation 1) (Tchobanoglous et al. 2003).

$$V_p = \frac{G(sg_p-1)d_p^2}{18v}$$ Equation 1

V_p= particle velocity (m/s)

G= Gravity (m^2/s)

sg_p= Specific gravity of the particle

d_p= Diameter of the particle (m)

v = Kinematic viscosity of water (m^2/s)

The surface overflow rate (SOR) is calculated by dividing the flow rate by the surface area (Equation 2) (von Sperling, Verbyla, and Oliveira 2020).

$$SOR = \frac{Q}{SA}$$ Equation 2

SOR= Surface Overflow Rate (m^3/hr/m^2)

Q= Volumetric Flow rate (m^3/hr)

SA= Surface area of clarifier (m^2)

If the particle's settling velocity is not greater than the SOR, the particle cannot settle. Clarifier design can promote particle settling by decreasing the SOR by increasing the surface area of the water.

2.3.2 Lamella Clarification

A clarifier design to increase the surface area for sedimentation, but keep a small footprint, is to implement lamella clarification. This innovation places honeycomb membranes, or multiple steel plates, at an incline near the water's surface to (1) increase the amount of surface area the water must cross before entering the subsequent treatment stage, thus lowering the SOR and (2) decrease the settling distance of the particle (Al-Dulaimi and Racoviteanu 2018).

The SOR of an open clarifier is calculated by determining the surface area of the clarifier at the top and dividing the influent flow rate by that surface area Equation 2 (Mackenzie L. Davis P.E., BCEE 2020). In comparison, the lamella plates decrease the SOR by using the projected surface rate of the plates, which is calculated by dividing the flow rate by the number of plates multiplied by the surface area of the plate and the cosine of the angle of the plate (Equation 3) (Ures 2013).

$$SOR_L = \frac{Q}{n * SA_L * cos\alpha} \qquad \text{Equation 3}$$

SOR= Surface Overflow Rate ($m^3/hr/m^2$)

Q= Volumetric Flow rate (m^3/hr)

n= Number of Plates

SA_L= Surface area of lamella plate (m^2)

α = Angle of plates (°)

Additionally, because of the incline of the plates, the settling distance in the lamella plates is shorter than a standard clarifier. The settling velocity of the particle, determined by stokes law, is described in a unit of distance divided by time. Thus, when the distance in which the particle must travel before reaching a solid surface is smaller than the time it takes for that particle to settle would be shorter compared to a particle required to travel a further distance (Al-Dulaimi and Racoviteanu 2018b; 2018a). The implementation of lamella clarifiers can reduce the area of clarifier needed by 95% (Brandt et al. 2017).

2.4 Chemically Enhanced Primary Treatment

Another process to improve sedimentation is to increase the density of particles needed to be settled, thus increasing the settling velocity of the colloidal particles. Chemicals are added before entering the clarifier to increase the density of the particles (Chagnon, Harleman, and Research 2002). This process is known as a chemically enhanced primary treatment (CEPT) and promotes the aggregation of suspended particles via the processes of coagulation and flocculation. CEPT is more efficient than conventional primary treatment and can remove 85% of influent TSS and 57% of $cBOD_5$ compared to conventional primary, removing 55% and 35 %, respectively (Shewa and Dagnew 2020; Dong et al. 2019).

2.4.1 Coagulation and Flocculation

Coagulation destabilizes the colloidal particulates in the water, and flocculation is where the destabilized colloidal particulates form aggregates (Bratby 2016). The four main mechanisms of coagulation using alum are compression of electric double layer, charge neutralization, enmeshment or “sweep flocculation”, and adsorption- interparticle bridging (Duan and Gregory 2003; Mackenzie L. Davis P.E., BCEE 2020). Historically, alum is a preferred coagulant choice because of its low cost and widespread availability (Gebbie 2006). Anionic high molecular weight polymer as a coagulant aid is the most common and effective during flocculation (Crittenden et al. 2012).

2.4.1.1 Alum coagulant

The aluminum species present in the water depends on the influent characteristics of the water and the pH. Figure 2.2 depicts the four aluminum species formed from Alum hydroxide and their presence as the pH changes. The aluminum species are either slightly charged or neutral when the pH is neutral.

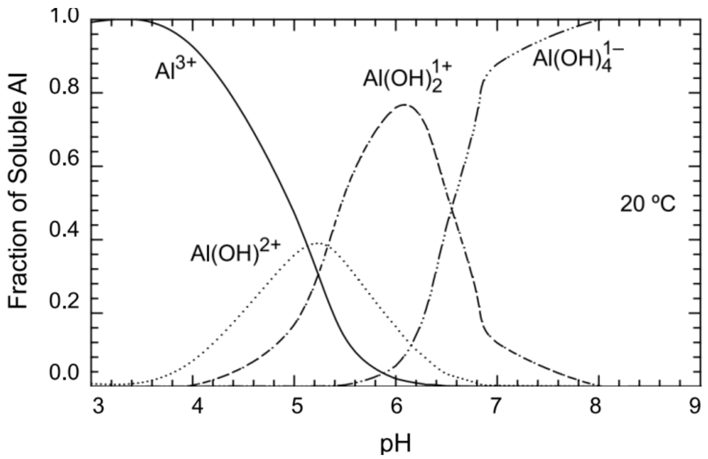
Hydrolytic Reaction	**Aluminum species as a function of pH**
$Al(OH)_{3(am)} \leftrightarrow Al^{3+} + 3OH^-$	
$Al^{3+} + H_2O \leftrightarrow Al(OH)^{2+} + H^+$	
$Al^{3+} + 2H_2O \leftrightarrow Al(OH)_2^{1+} + 2H^+$	
$Al^{3+} + 4H_2O \leftrightarrow Al(OH)_4^{1-} + 4H^+$	

Figure 2.2 Hydrolytic reactions of alum and aluminum species present in water as a function of pH (adapted from Pernitsky and Edzwald 2006)

2.4.1.2 Compression of the Electric Double Layer

Water has naturally occurring positive and negative ions. Colloidal particulates are negatively charged particles that must be removed from the system for water treatment to occur. The negative colloidal particulates attract the positive counterions, attracting the free negative ions, thus forming a double layer around the colloidal particulate (Crittenden et al. 2012). In the double layer, diffusion of ions and counterions occurs and has an overall net charge that can be measured, known as the zeta potential (Mackenzie L. Davis P.E., BCEE 2020). When the absolute value zeta potential of the water is 20 mV or less, rapid flocculation occurs (Crittenden et al. 2012).

The stability of colloidal particulates staying suspended is dependent on the ionic strength of the water and Van der Waal's interactions. When the double layer is compressed due to Brownian motion, the strength of repulsive force decreases, thus decreasing the zeta potential (Mackenzie L. Davis P.E., BCEE 2020; Bratby 2016).

2.4.1.3 Charge Neutralization

With the addition of alum between pH 6.0 to 6.4, the positively charged trivalent Al^{3+} coagulant and the negatively charged colloidal particulates are electrostatically attracted, which lowers the overall charge of the water (Huck and Sozański 2011). This mechanism occurs when the coagulant is not in excess and nearly instantaneously attracts the negative colloidal particulates. Floc formed via charge neutralization are the strongest type of floc (Cruz et al. 2020). When the pH of the wastewater is greater than 6.4, Al^{3+} is not the dominant aluminum species in the water(Pernitsky and Edzwald 2006).

2.4.1.4 Enmeshment (Sweep Flocculation)

At higher coagulant concentrations and out of the pH range where Al^{3+} is the dominating species, just below pH 7, the amorphous aluminum hydroxide precipitate ($Al(OH)_3$ (s)) begins to form, and colloidal particulates are enmeshed in the growing precipitate. The charge of the material is not an essential feature; instead, the size and weight of the precipitate form floc by trapping the colloidal particulates (Duan and Gregory 2003). Although all mechanisms exist in the CEPT process, enmeshment or sweep flocculation is the driving mechanism when pH is greater than Al^{3+} solubility and $Al(OH)_3$ precipitate is the dominant hydrolytic reaction (Gray 2010; Krupińska 2020).

2.4.1.5 Interparticle Bridging

Adsorption-interparticle bridging occurs as aluminum species polymerize during hydrolytic reactions and then adsorb to the destabilized colloidal particulates through electrostatic interaction, dipole moments, hydrogen bonding, or Van der Walls (Bratby 2016). After the adsorption of polymer to the colloidal particulates, the remaining polymer end is available for adsorption to another colloidal particulate, thus creating the bridging moment (Mackenzie L. Davis P.E., BCEE 2020). If too much polymer or coagulant is in the system for the concentration of particulates, then bridging will not occur because there is not a free surface for adsorption (Crittenden et al. 2012).

2.4.1.6 Polymer

To further create denser flocs, a polymer can be added to aggregate the destabilized colloidal particles produced by the coagulant. Coagulant aids are long-chain high molecular weight polymers that use their large chain to aggregate the colloidal particulates and increase the floc density (Brandt et al. 2017). Polymer addition is like interparticle bridging, where an overdose of a polymer may prevent flocculation from occurring (Crittenden et al. 2012). Additionally, a lower mixing speed is required to ensure macro flocs are not broken apart before clarification (Murujew et al. 2019).

2.5 Ballast

A ballasting agent may be added to the system and incorporated into the flocs to improve the settling velocity further after using coagulants and polymer. The ballast is added simultaneously as the flocculant allowing the ballast to be enmeshed before the flocs are fully formed (Kumar, Ghosh, and Kazmi 2016). Many forms of ballast can be used for a CEPT process, but ballast materials currently used in the wastewater treatment industry include microsand, magnetite, and sludge contact (Hun 1998).

In reference to Stoke's Law (Equation 1), the settling velocity calculation uses the specific gravity and diameter of the particle to determine its velocity. The specific gravity of microsand, magnetite, and sludge are 2.65, 5.08, and 2.50, respectively (Lapointe and Barbeau 2015; Lapointe et al. 2017; Andreoli, von Sperling, and Fernandes 2007). Based on the specific gravity of the ballast, magnetite would be expected to have a greater settling velocity than microsand and sludge. Although the density of floc does improve the settling rate, the diameter of the ballasted floc has been shown to have a more significant effect on the velocity than the floc density and shape (Lapoint 2016). Optimization of floc diameter can be accomplished by determining the optimum mixing speed that keeps ballast in suspension but does not shear the mature floc (He et al. 2019).

The addition of ballast does not affect the chemical process. Mixing recycled ballasted floc with a new chemical dosage does not deteriorate the effluent (Young and Edwards 2003). Increased coagulant and polymer doses have been found to increase the ability for floc to hold ballasting agents, thus increasing the settling velocity and decreasing the settling time (Young and Edwards 2003).

2.5.1 Magnetite

Magnetite (Fe_3O_4) is an iron oxide used as a ballast in wastewater but has novel properties that make it unique to use in water and wastewater treatment (Namdeo 2017). Magnetite is beneficial because it is inert and will not rust or change any chemical process within the treatment process ("CoMag® System" n.d.). Additionally, the hydrophobic property allows the fine particles to quickly become enmeshed with the flocs (Carlos et al. 2013). In a CoMag process, the magnetite is added as a ballast before the flocculation stage. The benefit of CoMag and its use of magnetite is the density of magnetite is nearly twice that of other ballasts (Lapointe et al. 2017). Additionally, the magnetic property and proprietary magnetic recovery system have a recovery rate reported to be greater than 95% ("CoMag® System" n.d.). Magnetite has also been used to remove heavy metals, including lead and arsenic (Namdeo 2017; Carlos et al. 2013).

2.6 Densadeg

The Densadeg, a CEPT high-rate clarification system (SUEZ n.d.), uses coagulant, flocculant, sludge ballast, and lamella plates to meet discharge limits with variable flow rates. The design has small square footage but can handle high flow rates. The Densadeg general design is to manage 1,260 to 15,770 m^3/hr of influent flow where the TSS percent removal is expected between 80% to 90%, and the $cBOD_5$ percent removal is 35% to 65% (Qasim and Zhu 2018).

As depicted in Figure 2.3 the treatment occurs in six stages.

1. A coagulant is added and mixed at the greatest intensity of the process.
2. Polymer and recycled sludge ballast are introduced in the flocculation chamber.

3. The flocs are allowed maturation time where only plug flow mixing occurs before entering the settling tank (Suez n.d.).
4. In the settling tank, the flocs settle to the bottom.
5. The clarified water runs over the large surface area of the honeycomb lamella plates before exiting.
6. Settled sludge is either wasted or recycled between 2% and 6% of influent flow depending on the solids content of the sludge and used as the ballast (Qasim and Zhu 2018).

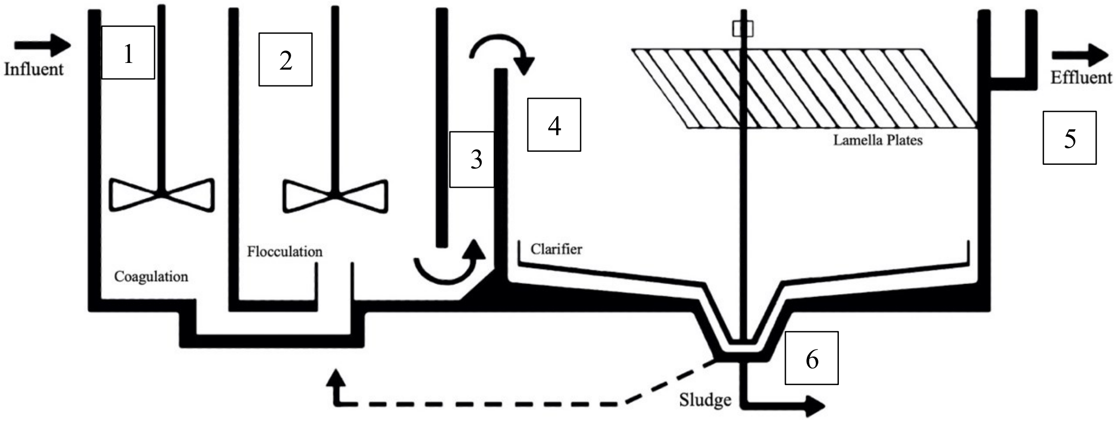


Figure 2.3 Schematic of process of Densadeg XRC

Because the system relies on sludge as a ballast, no additional screens or pumps are required to protect equipment and properly waste sludge (Hun 1998). The process produces thick sludge (7.4%) and could cause sludge handling issues if sized wrong (San Diego State University 2005). This process is dependent on maintaining a sludge blanket of 1 m to properly ballast the system (San Diego State University 2005).

CHAPTER 3 MATERIALS AND METHODS

3.1 Study Site: Dartmouth Wastewater Treatment Facility

The Dartmouth Wastewater Treatment Facility (DWWTF), located in Dartmouth, Nova Scotia, Canada, and operated by Halifax Water, began operation in August 2008 (Halifax n.d.). The plant treats a combined wastewater influent comprised of municipal sewage and stormwater. It is a chemically enhanced primary treatment (CEPT) process designed for an average flow of 83,800 m^3/day. From April 1st, 2018, through March 31st, 2021, 56,922,250m^3 (Halifax Water 2019; Halifax water 2020; Halifax Water 2021) of water was treated, equating to an average of 63% daily plant processing capacity.

3.1.1 Treatment Process

The facility is equipped with two double reaction Densadeg XRC™ trains by Suez. This allows for four reaction tanks of coagulation and flocculation, each with two large lamella clarifiers. The influent enters the facility and goes through a series of processes, as shown in Figure 3.1, and the treated effluent is then discharged into Halifax Harbor.

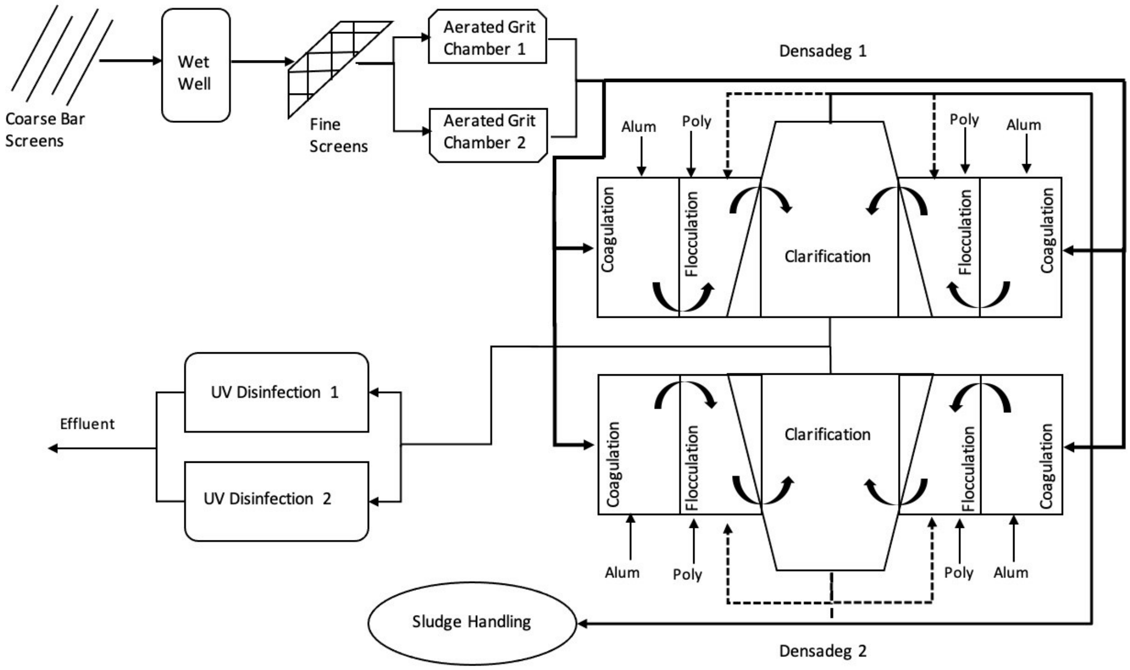


Figure 3.1: Schematic of DWWTF

Wastewater enters the plant and passes through one coarse 25 mm mechanical bar screen. The water enters a wet well where five 250-horsepower submersible pumps lift the water 4.6 m. The rest of the plant's hydraulics is gravitational flow. The water from the five pumps is funneled together and then separated into three channels for fine, 6mm, screen removal. Following the fine screen are two fine bubble aerated grit chambers. The significant step in the physiochemical process is the Densadeg high-rate clarifier and thickener by SUEZ. Once the water exits the aerated chamber, it is known as influent in the case of this project. The water is met with a rapid mix impeller that mixes the liquid aluminum sulfate (alum), ($Al_2(SO_4)_3 * 14(H_2O)$) with the water. After being dosed with alum, the water is then dosed with premixed dry anionic polymer (FLO Polymer AF 4400 SNF Canada) (Table 3.1) and mixed at a lower impeller speed in a process known as flocculation. The large flocs form, then enter the last step, the clarifier, and are separated from the clean water, effluent, that runs over the lamella plates. The collected waste solids are either sent offsite to be disposed of or recycled as ballast. After clarification, the water is disinfected with UV (TrojanUV3000Plus) before being discharged into Halifax Harbor.

Table 3.1: Coagulant and Flocculant (Coagulant Aid) Properties

	Type of Product	Provider	Specific Gravity	Supplied Concentration (w/w%)
Liquid Alum	**Aluminum Sulfate**	**Chemtrade**	**1.33**	**4.4 Al (Neat)**
FLO Polymer AF 4400	**Anionic**	**SNF Canada**	**0.6-09**	**High Molecular Weight**

3.1.2 Water Quality

Over a total study duration between August 2020 and November 2021, three studies were conducted to meet the project objectives. From August 2020 through October 2020, a ballasted pilot study was conducted. From October 2020 through August 2021, bench-scale tests were completed. The following water quality values are from the pilot and bench-scale testing where grab sampling occurred to keep the influent values consistent. Average influent water quality during the bench- and pilot-scale is provided in Table 3.2.

The values in Table 3.2 reflect influent water quality averages from near-daily grab samples between the work hours of 9 am-2 pm August 2020 through April 2021.

Table 3.2 Average full-scale plant influent water quality grab samples taken between 9 am and 2 pm daily from August 2020 through April 2021

	Temperature (°C)	**pH (SU)**	**TSS (mg/L)**	**$cBOD_5$ (mg/L)**	**$scBOD_5$ (mg/L)**	**UVT (%)**	**Turbidity (NTU)**
Average	**23.6**	**6.8**	**196.4**	**116.3**	**42.1**	**25.8**	**82.0**
(± SD)	**10.9**	**0.2**	**206.7**	**56.9**	**27.2**	**10.6**	**42.5**

Because the full-scale plant treated combined sewer wastewater, there were two average daily flows, wet weather, and dry weather.

- Dry Weather Average Daily Flow (DW ADF): 65.47 Megaliters/day (MLD) or 2727.9 m^3/hr
- Wet Weather Average Daily Flow (WW ADF): 98.29 MLD or 4095.4 m^3 /hr

These flows are for the entire full-scale plant (both Densadeg trains running). The influent water quality characteristics and flow rates are highly variable. In general, high influent flowrates corresponded to more dilute water and lowered influent turbidities, while lower flowrates corresponded to more concentrated wastewater (less stormwater influence) and higher influent turbidities (Figure 3.2).

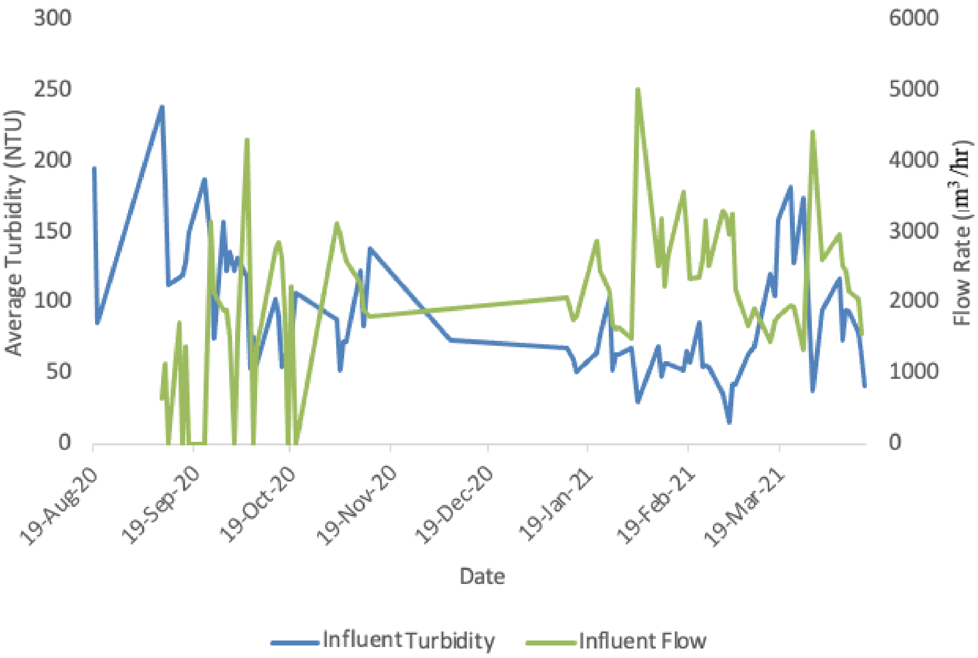


Figure 3.2 Influent turbidity and influent flow depicted over time. Blue is the influent turbidity, and its corresponding y-axis is on the left. Green is the influent flow rate of the full-scale plant, and its corresponding y-axis is on the right.

3.2 Bench-Scale Chemical Dosing Optimization

Bench-scale jar test optimization ran from November 2nd, 2020, through April 14th, 2021. A standard Phipps and Bird Jar Tester was used. Jar tests were completed with variable HRTs, turbidity, and various chemical dosing regimens.

3.2.1 Bench-Scale Sample Collection

Two five-gallon buckets were used to collect the sample before the Densadeg process and after the aerated grit chamber. Before each sample was collected, the sample port was flushed for 30 seconds. As the sample was collected, the turbidity, pH, and conductivity were recorded as produced by the plant's in-line instrumentation. The influent sample was then tested, and the mixed sample was evenly distributed into the six 2 L jars to be tested.

3.2.2 Jar Test Experimentation Process

To correctly mimic various flows that the full-scale plant may encounter, each reaction stage's hydraulic retention time (HRT) was determined by dividing the tank's volume by the flow rate. Volumes were calculated for all reaction tanks (i.e., coagulation, flocculation, and clarification) using the as-built drawings for the facility. Because the full-scale plant has four coagulation tanks, four flocculation tanks, and only two clarification tanks, the volume of the clarification tank was determined using half the width of the clarifier. Additionally, when calculating the HRT, the flow value had to be divided by 2 to represent the one flow rate entering one Densadeg accurately. The following was used for each hydraulic retention time (Equation 4). Where V is the volume of a reaction tank, and Q is the flow rate (von Sperling, Verbyla, and Oliveira 2020).

$$HRT = \frac{V}{Q}$$ Equation 4

HRT= Hydraulic Retention Time (hr)

V= Volume of Reaction Chamber (m^3)

Q= Volumetric Flow rate (m^3/hr)

Because of the Covid-19 pandemic, extra precautions were made at the facility, and the real-time influent flow rates could not be recorded at the time of each test. To best capture the hydraulic retention time and influent turbidity the full-scale plant may encounter throughout a year, five flow rates were determined to represent the flows of the full-scale plant. These flow rates were 1500 m^3/hr, 2000 m^3/hr, 2400 m^3/hr, 3105 m^3/hr, and 3991 m^3/hr. The influent turbidity value could be captured in real-time; therefore, no artificial turbidity was used. The following Table 3.3 describes each jar's mixing and settling times based on the calculated HRT of each reactor and its corresponding flow rate. However, the influent flow rate was not known at the test time; a “calculated” HRT value would be tested using the values from the table below.

Table 3.3: Mixing and settling time of each stage based on HRT and Flow Rate

Influent Flow	**1500 m^3/hr**	**2000 m^3/hr**	**2400 m^3/hr**	**3105 m^3/hr**	**3991 m^3/hr**
Coagulation (min)	**6**	**5**	**4**	**3**	**2**
Flocculation (min)	**13**	**10**	**8**	**6**	**5**
Settling (min)	**32**	**24**	**20**	**15**	**12**
Total (min)	**51**	**39**	**32**	**24**	**19**

In consultation with Halifax Water, the jar test impellers’ RPM was determined using the velocity gradient curves provided by the manufacturer. The coagulation stage required a rapid mix where the RPM of the impeller was 250. The flocculation stage required mixing at a slower speed (150 RPM) to not break the forming flocs. Both mixing speeds best represented the mixing speeds of the full-scale plant. During the clarification stage, no mixing occurred. The blades were pulled out of the water, and the flocs were allowed the designated settling period before the decanted “effluent water” was tested. About 60 mL of sample was collected and characterized for turbidity, TSS, and UVT.

3.2.3 Jar Test Chemical Dosing

The variable alum dose was tested at 3, 4, 5, 6, 7, and 8 mg/L (Al^{3+} Neat) while the polymer was constant. The polymer dose was then varied between 0.5 mg/L and 2.5 mg/L in increments of 0.25 while the alum was kept constant. Chemical dosing regimens at all the calculated hydraulic retention times (i.e., 51, 39, 32, 24, and 19 minutes) and varied influent turbidity (ranging between 15 NTU and 193 NTU) were completed to investigate all influent water quality conditions the full-scale plant may encounter.

3.3 Ballasted Magnetite Pilot Study

3.3.1 Pilot System

The pilot system was a trailer-mounted CoMag® magnetite ballast system known as “PP2” provided by Evoqua Water Technologies (EWT). The influent water for the pilot was pumped from the DWTF aerated grit chamber supernatant to best mimic the water entering the Densadeg™ process inside the facility. The pilot was controlled by a system that monitored the influent line flow rate, pH, and temperature. Other monitored and controlled rates were the waste and recycle lines and the coagulant and flocculant dosing rate. The chemicals were checked daily by calibration column tests. Because the system monitored the influent flow rate, the coagulant peristaltic pump was flow paced based on an entered dosing set point.

The influent flow rate would be manually set by slightly opening a ball valve to match the testing influent flow rate. Occasionally rags would get caught in the ball valve, and influent flow would drop suddenly. When these clogs occurred in the influent line, the operator had to intervene to clear out the clogs and keep the pilot running smoothly. Right after the ball valve, the water flowed through an inline static mixer where the coagulant would be dosed, acting as a rapid mix like the first reaction tank of the Densadeg™.

After the coagulant was mixed inline, the water entered the first reaction tank, where the magnetite ballast would be introduced, and onto the second reaction tank, where the polymer would be introduced in a two-point location in a similar pumping program as the coagulant. The two reaction tanks were separated by an underflow baffle and mixed with stainless steel mixers controlled by a variable frequency drive to ensure proper mixing energy. The water then entered the lamella clarifier.

The clarifier had four chevron plates, providing 0.195 m^2 of the total clarified area used and 1.663 m^2 of the projected clarifier area. To best compare the pilot and the existing full-scale plant, the nominal SOR is used to determine the flow of the pilot. The nominal SOR is determined by the flow rate (Q) and the used clarifier surface area (SA) (Equation 2) (von Sperling, Verbyla, and Oliveira 2020).

As noted in Table 3.4, the pilot ran at a mean flow rate of 10 m^3/hr, which produced a SOR higher than the WW ADF of the full-scale plant running one clarifier train but with water quality of dry weather. The full-scale plant ran on average 2518 m^3/hr with a SOR similar to wet weather because only one train was running during pilot testing as the facility was undergoing process upgrades. Because only one clarifier train was on at a time, it was possible to compare full-scale plant and pilot directly.

Table 3.4 A comparison of flow rates of one clarifier train online and its corresponding surface overflow rate. (DW- Dry Weather, WW- Wet Weather, ADF- Average Daily Flow)

	Pilot		**Full-Scale Plant**	
	Flow (m^3/hr)	SOR ($m^3/hr/m^2$)	Flow (m^3/hr)	SOR ($m^3/hr/m^2$)
Mean Pilot Flow	**10**	**54**	**2518**	**54**
Mean Plant Flow	**9**	**44**	**2069**	**44**
DW ADF	**6**	**29**	**1364**	**29**
WW ADF	**9**	**44**	**2048**	**44**
Peak Pilot Flow	**17**	**86**	**4013**	**86**

The figure below (Figure 3.3) is a schematic of the EWT pilot. The pilot included two pumps at the bottom of the clarifier for underflow; one was for wasting, and the other was for recycling. During the startup phase of the pilot, it was determined that the recycle would not be used because the full-scale plant was not using it at full-scale because of the high solids loading. The waste line was cycled up through a shear mixer where a magnetic drum recovered the magnetite, and the remaining sludge exited the system. The wasting rate varied between 5% to 55% of the influent during the startup phase.

The pilot ran at an overall higher rate than the full-scale plant because of pump sizing issues in the waste line from the pilot clarifier. For example, without a rake at the bottom of the pilot clarifier, the waste could not be adequately removed and would cause the system to clog the lamella plates and fail. To avoid this problem, the wasting and influent rates had to be higher than the full-scale plant to remove the solids properly. During Phase 2 and Phase 3, the waste rate stabilized at approximately 25% of the influent rate.

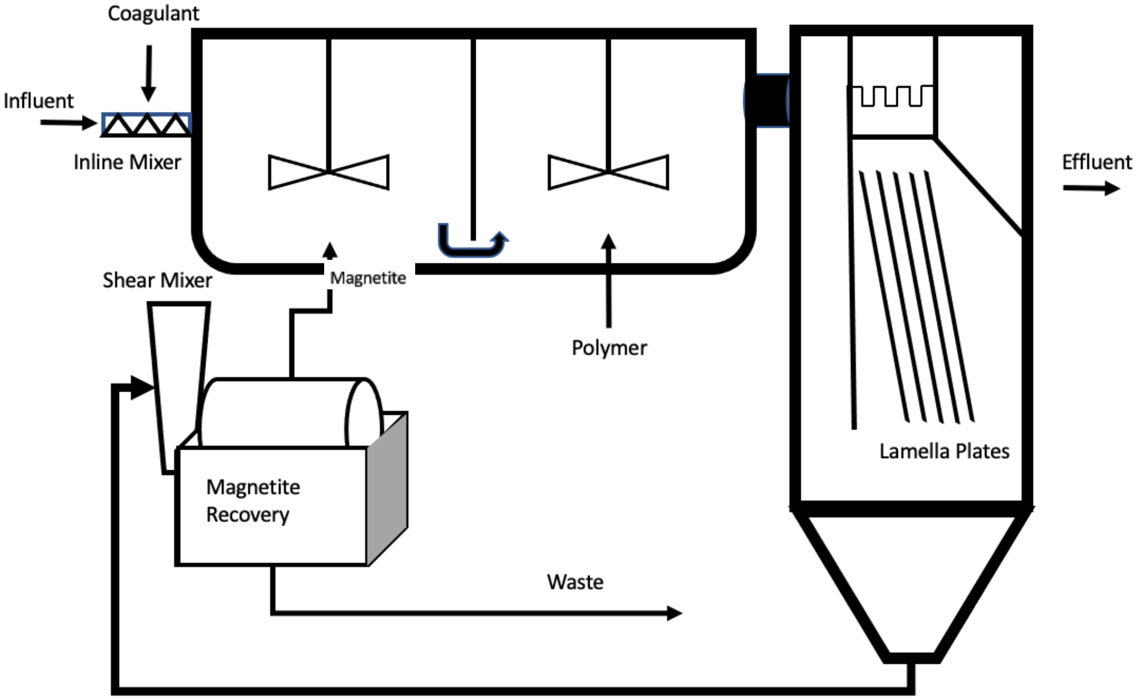


Figure 3.3 Schematic process of the EWT pilot

3.3.2 Pilot Operation

The pilot ran from July 27th, 2020, through October 20th, 2020. To mimic the full-scale DWTF design as much as possible, the same coagulant (ChemTrade Liquid Alum) and flocculant (SNF Flo dry anionic Polymer AF 4400) were used, and no pH adjusting chemicals were used.

Three experimental phases defined the pilot testing period:

- Phase 1 (July 27th – Sept 8th): Startup, operation, and stabilization. Once the pilot stabilized and the pilot operation was running smoothly, Phase 2 was started.
- Phase 2 (Sept 9th – Oct 2nd): Operate pilot at optimized chemical dosing to compare grab samples against the full-scale plant.
- Phase 3 (Oct 3rd – Oct 20th): Operate pilot to consistently meet the new composite regulation limits in the grab samples.

Table 3.5 below describes the testing or work accomplished each day during the pilot period.

Table 3.5 Pilot Study Summary Run

Week		Monday	Tuesday	Wednesday	Thursday	Friday
1	July 27th	Trailer Setup	Trailer Setup	Trailer Setup	Equipment Training	Equipment Training
2	Aug 3rd	Coagulant Dose-response, Polymer constant	Coagulant Dose-response, Polymer constant	Coagulant Dose-response, Polymer constant	Coagulant Dose-response, Polymer constant	Coagulant Dose-response, Polymer constant
3	Aug 10th	Coagulant Dose-response, Polymer constant	Coagulant Dose-response, Polymer constant	Coagulant Dose-response, Polymer constant	Coagulant Dose-response, Polymer constant	Coagulant constant, Polymer dose-response
4	Aug 17th	Coagulant and Polymer no change	Coagulant and Polymer no change	Coagulant dose-response and Polymer constant	Coagulant and Polymer no change	Coagulant and Polymer no change
5	Aug 24th	Coagulant and Polymer no change	Coagulant constant and Polymer dose-response	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change
6	Aug 31st	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change	Maintenance
7	Sept 7th	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change
8	Sept 14th	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change	Maintenance
9	Sept 21st	Maintenance	Optimal Coagulant and Polymer no change	Maintenance	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change
10	Sept 28th	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change	Optimal Coagulant and Polymer no change
11	Oct 5th	WSER Coagulant and Polymer (100%)	WSER Coagulant and Polymer (100%)	WSER Coagulant and Polymer (100%)	WSER Coagulant and Polymer (100%)	Maintenance
12	Oct 12th	Maintenance	Maintenance	WSER Coagulant and Polymer (100%)	WSER Coagulant and Polymer (50%)	WSER Coagulant and Polymer (50%)
13	Oct 19th	WSER Coagulant and Polymer (50%)	WSER Coagulant and Polymer (50%)	Decommission	Decommission	Decommission
	Key:	Dry Weather Average Daily Flow (DWADF)	Wet Weather Average Daily Flow (WWADF)	Peak Flow (PF) Pilot	Maintenance	

Phase 1	**Phase 2**	**Phase 3**	

3.3.3 Sample Collection

Before samples were collected, the pilot underwent two full periods whenever a new dose regiment was introduced to have proper process stabilization. These periods were determined by the hydraulic retention time (HRT), as shown in Table 3.6. The average HRT of the pilot was 32 minutes, which correlated to sample collection every 1 hour and 15 minutes.

Table 3.6 Hydraulic retention time of each process stage at the various flows (DW- Dry Weather, WW-Wet Weather, ADF- Average Daily Flow).

		Mean Pilot Flow	**DWADF**	**WWADF**	**PF Pilot**	**Mean Plant Flow**
Plant	**Coagulation (min)**	1.81	3.33	2.22	1.13	2.20
	Flocculation (min)	3.79	6.99	4.65	2.38	4.61
	Clarifier (min)	9.45	17.45	11.62	5.93	11.50
	Total (min)	15.04	27.77	18.50	9.44	18.31
Pilot	**Coagulation (min)**	4.62	8.53	5.68	2.90	5.62
	Flocculation (min)	4.62	8.53	5.68	2.90	5.62
	Clarifier (min)	23.09	42.63	28.40	14.49	28.11
	Total (min)	32.33	59.68	39.75	20.29	39.36

Before collecting water samples, values from the pilot's in-line instrumentation were recorded to ensure proper running of the pilot and to understand the influent properties of the wastewater matrix. The values include the influent flow, influent water temperature, and dosing rate of the polymer and alum. Samples were collected from multiple pilot sample ports: influent, effluent, waste, underflow from the clarifier, and polymer reaction tank. To compare the pilot and the full-scale plant results, additional samples were taken in the DWWTF from sample ports after the grit chamber “full-scale plant influent” and before the UV disinfection “full-scale plant effluent.” Like the pilot, DWWTF values of influent flow, chemical dosing, and turbidity were also recorded. All samples were collected in 1-liter bottles and analyzed, following the Standard Methods for Examination of Water and Wastewater(APHA, AWWA, and WEF 2017).

3.4 Analytical Methods

The water quality analyses were done onsite in the DWWTF lab as quickly as possible after the samples were collected following the Standard Methods for Examination of Water and Wastewater for bench-scale and pilot testing (APHA, AWWA, and WEF 2017). All influent and effluent samples underwent the same analysis to compare the treatment process effects directly. Water quality response parameters for bench-scale testing included turbidity, UVT, and TSS in determining water quality. For pilot-scale testing, water quality response parameters included turbidity, UVT, TSS, $cBOD_5$, and $scBOD_5$.

3.4.1 Turbidity

The turbidity was determined by following the Standard Method (2130) (APHA, AWWA, and WEF 2017) using a spectrophotometer (Hach DR3900). Each test was done in duplicate, and the average value at that data point. If a value was not recorded or recorded as #N/A, it was excluded from the average.

3.4.2 Ultraviolet Transmittance (UVT)

The study's ultraviolet transmittance (UVT) was measured using a field meter (Real Tech UV 254 Field Meter.) Each day, the meter was calibrated by filling the cuvette with DI water, wiping the cuvette with a Kim-wipe to ensure no interference, and set to 100% according to the manufacturer's instructions. After calibration, the samples were placed in the cuvette. The cuvette was wiped with a Kim-wipe, and the transmittance percentage was recorded. Each test was done in duplicate, and the average value at that data point. If a value was not recorded or recorded as #N/A, it was excluded from the average.

3.4.3 Total Suspended Solids (TSS)

The TSS was analyzed following the method described in Standard Methods 2540D (APHA, AWWA, and WEF 2017). If the TSS calculation produced a negative, it was recorded as 0. Each test was duplicated, and the average value a. If a value was not recorded, that data point was excluded from the average.

3.4.4 Five-day carbonaceous biochemical oxygen demand ($cBOD_5$) and five-day soluble carbonaceous biochemical oxygen demand ($scBOD_5$)

Nitrification inhibited carbonaceous biochemical oxygen demand 5-day ($cBOD_5$) test was accomplished following Standard Methods 5210B (APHA, AWWA, and WEF 2017). Additionally, the soluble carbonaceous biochemical oxygen demand 5-day test ($scBOD_5$) was determined by first filtering the sample through a TSS glass fiber filter pore size of

1.5µm and using the filtered water as the sample to be analyzed following the same standards (5210B) (APHA, AWWA, and WEF 2017). All dissolved oxygen values were recorded with a dissolved oxygen probe (Orion RDO). The probe was calibrated each day according to the manufacturer's instructions. If the $cBOD_5$ or SBOD final DO value was less than 1, that test is inconclusive, and the result was recorded as #N/A (von Sperling, Verbyla, and Oliveira 2020). Each test was done in duplicate, and the average value at that data point. If a value was not recorded or recorded as #N/A, it was excluded from the average.

3.5 Data Analysis Bench-Scale Study

It is recommended that full-scale plant performance be evaluated using both removal efficiency and effluent concentrations (von Sperling, Verbyla, and Oliveira 2020). This study's water quality results were analyzed in terms of removal efficiency (Equation 5) for TSS and turbidity.

$$\frac{Influent - Effluent}{Influent} \cdot 100. \qquad \text{Equation 5}$$

The UVT was analyzed in terms of increase (Equation 6).

$$Effluent - Influent \qquad \text{Equation 6}$$

It was determined that percent removal and percent increase better reflected the results than the absolute values during the jar testing Phase. R-Studio was used for calculations and data visualization.

3.6 Data Analysis Ballasted Pilot Study

Paired equal variance t-tests were performed to verify the statistical significance between the pilot performance and full-scale plant performance with a confidence of 95% (α=0.05). Unpaired equal variance t-tests were performed to verify the statistical significance between the pilot's performance in Phase 2 and Phase 3 with a confidence of 95% (α=0.05). Calculated p-values less than 0.05 were deemed statistically significant, while any value greater than 0.05 was not. Data analysis and manipulation were performed using Microsoft Excel.

CHAPTER 4 RESULTS AND DISCUSSION

4.1 Bench-Scale Chemical Dosing Optimization

Extensive jar testing was performed to determine optimal coagulant and flocculant doses for the DWWTF over a range of influent water quality (as measured by turbidity) and system HRTs. Section 4.1.1. provides the expected optimized dose for alum and polymer regardless of influent turbidity and HRT. Section 4.1.2 describes the optimal chemical doses determined as a function of influent turbidity and system HRT.

4.1.1 Expected Alum and Polymer Dose Ranges

After analyzing the dose-response curves and corresponding contour plots, in Figure 4.1, an optimized dose range was determined. Figure 4.1, suggests the optimal dose range for the DWWTF to be between 5mg/L and 8 mg/L for alum and between 0.5 mg/L and 1.5 mg/L polymer.

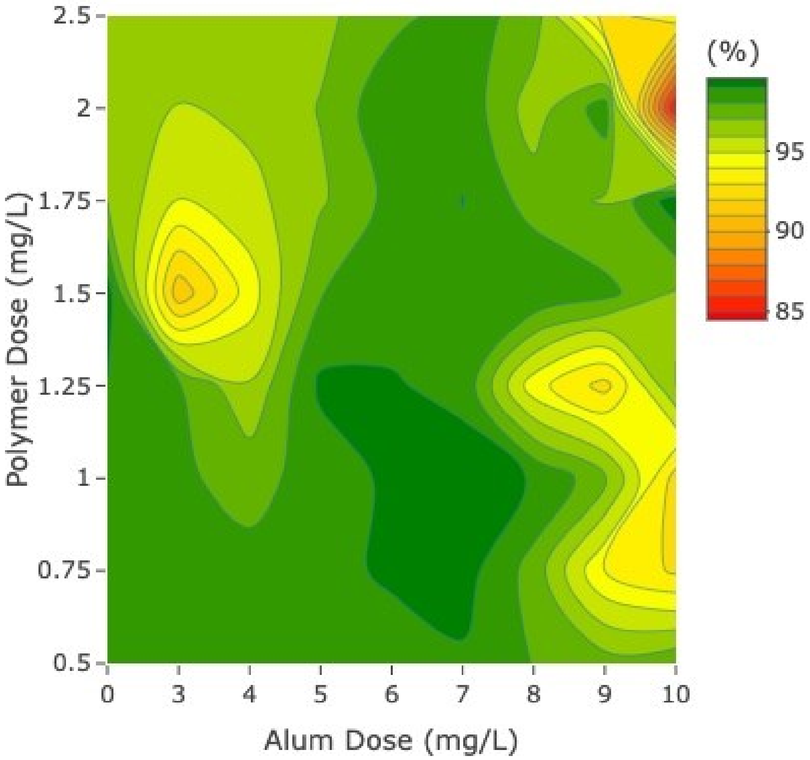


Figure 4.1 Contour plot where the color represents the % turbidity removed. The x-axis is Alum dose (mg/L), and the y- axis is polymer dose (mg/L)

These chemical dosing conditions consistently resulted in turbidity removal efficiency greater than 99%. The alum dose curve in Figure 4.2 depicts the optimal alum dose at 8 mg/L, where the average turbidity efficiency was 99%.

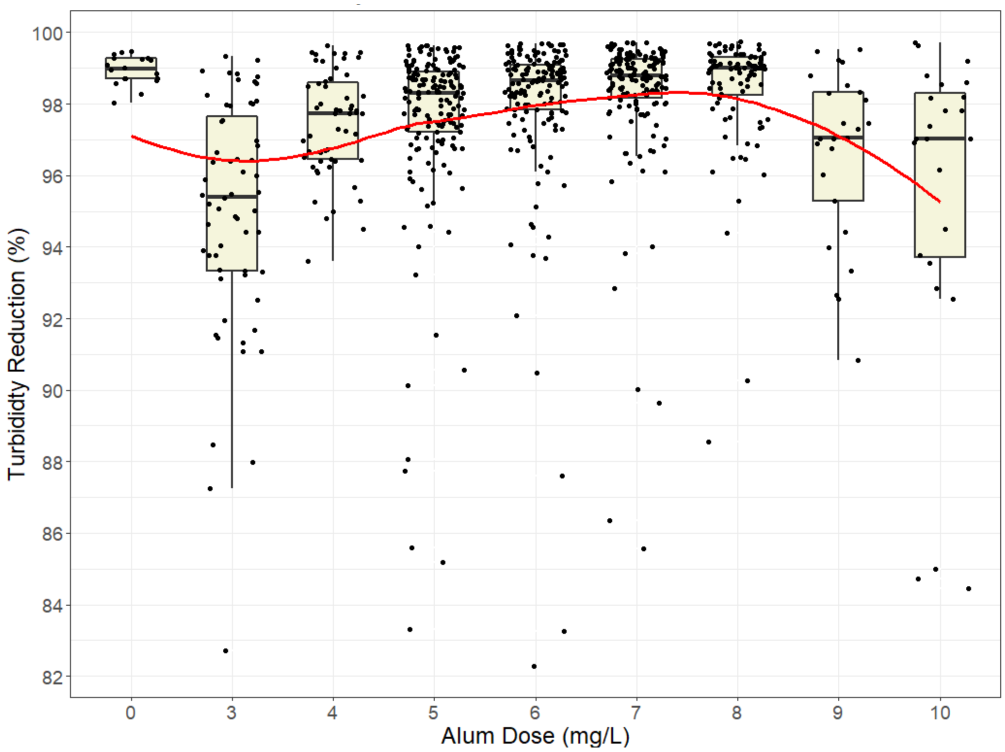


Figure 4.2 Box plots of all tests where the x-axis is the alum dose (mg/L) and the y-axis is % turbidity removed

The polymer dose curve in Figure 4.3 describes the optimal polymer dose between 0.5 and 1.25 mg/L, corresponding with a turbidity removal efficiency of greater than 98%.

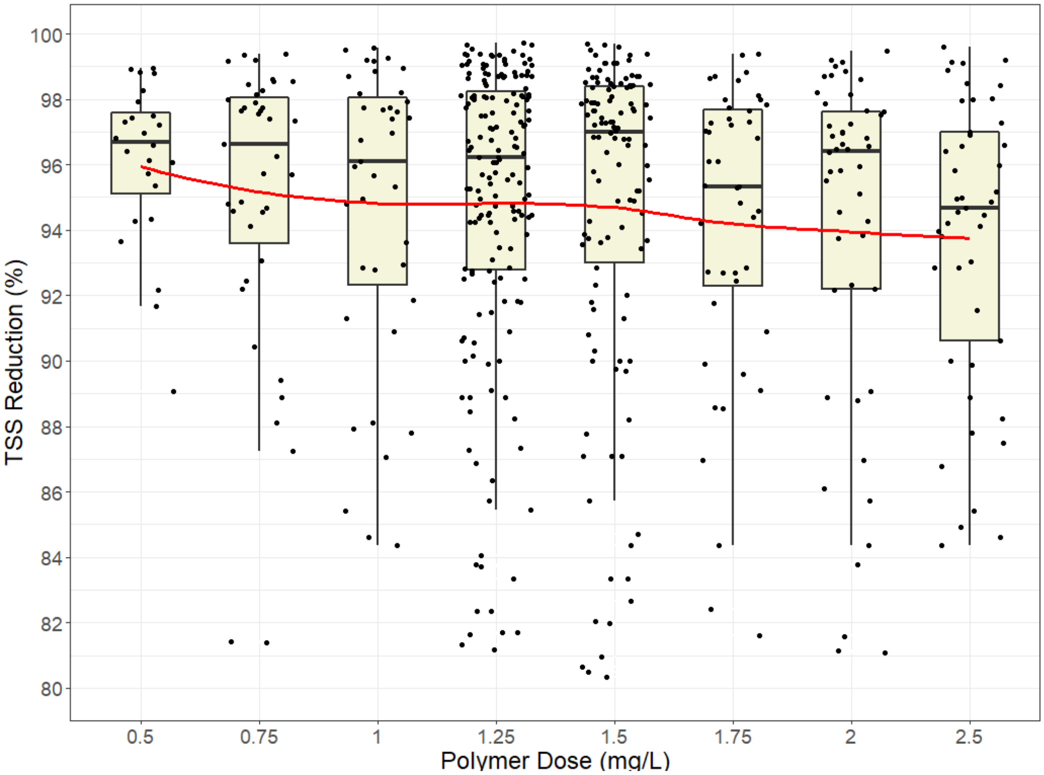


Figure 4.3 Box plots of all tests where the x-axis is the polymer dose (mg/L) and the y-axis is % turbidity removed

TSS removal efficiency and UVT increase had similar trends supporting this finding (Appendix A1). These optimized chemical dosing ranges are recommended for the system when the impact of ballast is not considered. They do not consider the complexity of influent water quality conditions or hydraulic retention time. Further testing is required at the full-scale to know if this optimized dosing regimen improves the effluent water quality of the full-scale plant or if further refinements are needed.

4.1.2 Alum and Polymer Dose as a Function of Influent Water Quality Conditions

It was suspected that as the influent flow of the full-scale plant increases and the hydraulic retention time of the process decreases, more coagulant and flocculant would be needed to maintain adequate treatment. Additionally, it was anticipated that influent water quality conditions (i.e., turbidity) would impact the required chemical doses. Jar tests were conducted and evaluated at five different HRTs (51, 39, 32, 24, and 19 minutes), where the influent turbidity water quality ranged between 15 NTU and 193 NTU.

Jar testing results for alum dose as a function of influent turbidity over the range of HRTs tested (Figure 4.2) revealed two distinct observations:

1. The optimal alum dose was a function of influent turbidity where lower influent turbidity required more coagulant dose compared to higher influent turbidity when the HRT was longer than 32 minutes, and the influent turbidity was less than 90 NTU (Section 4.1.2.1).
2. As the HRT of the system decreased, a specific coagulant dose could not adequately treat the water if the influent turbidity was less than 90 NTU (Section 4.1.2.2).

The polymer dose contour plots followed similar trends as the coagulant and supported these observations (Appendix A. 2).

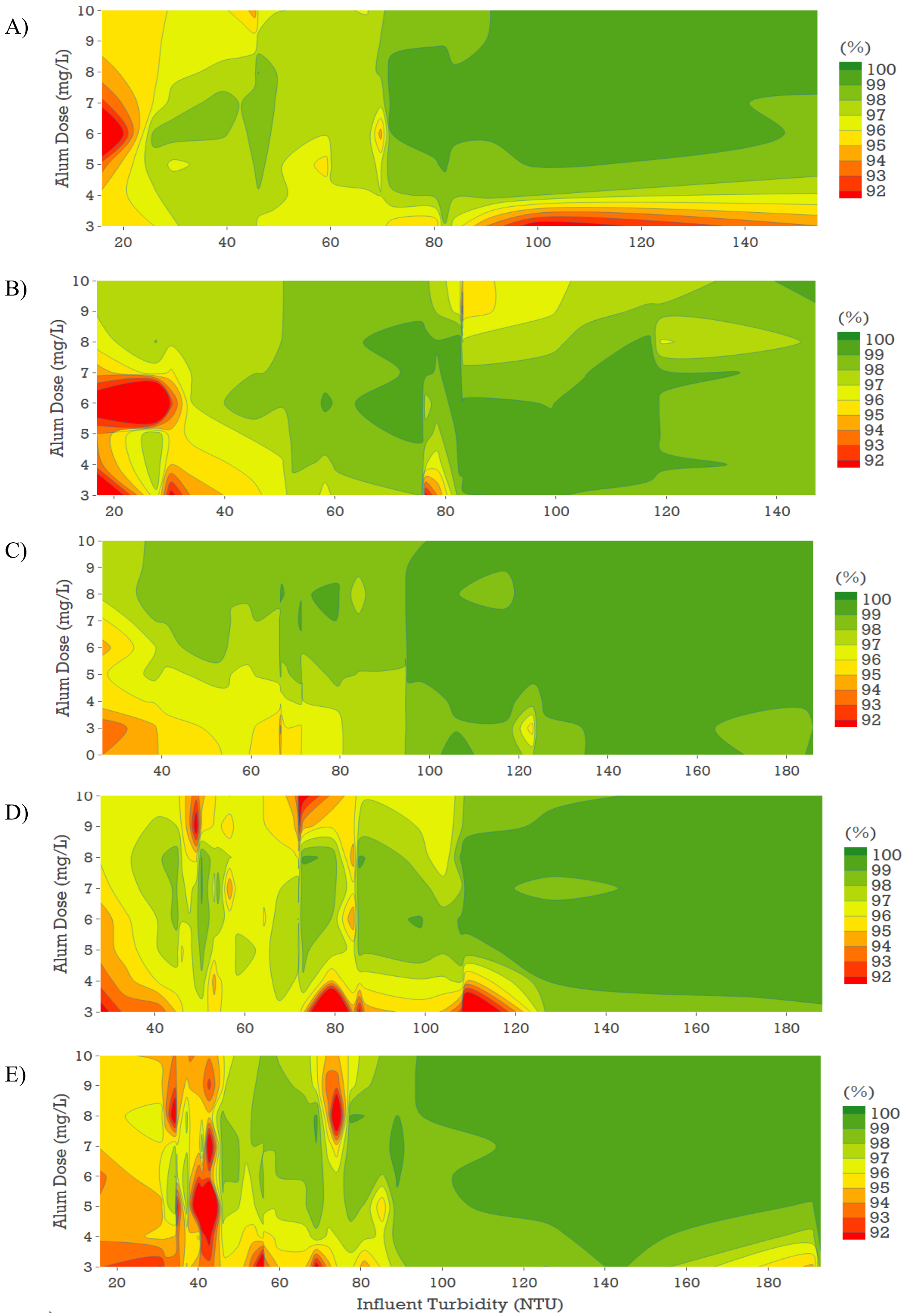


Figure 4.4 Contour plots where the y-axis is alum dose, the x-axis is influent turbidity values, and the color is turbidity removal as a percentage. Plot (A), (B), (C), (D), and (E) is the hydraulic retention time of 51, 39, 32, 24, and 19 minutes respectively.

4.1.2.1 Alum Dose as a Function of Turbidity

As shown in Figure 4.4.A), less coagulant was required (4 mg/L) when the influent turbidity was around 90 NTU compared to the coagulant (6 mg/L) required when the turbidity was less than 40 NTU. Figure 4.4.B) and C) depict a similar trend and reflect findings from the literature that indicate that more coagulant is required when the turbidity is low (Cruz et al. 2020; Jiao et al. 2017). In general, less coagulant was required as the influent turbidity increased. As previously discussed, there are four primary coagulation mechanisms, but for the results obtained in this study, two mechanisms (charge neutralization and sweep flocculation) have a larger impact on wastewater treatment (Duan and Gregory 2003). The more acidic the influent water pH, the greater the effect the charge neutralization mechanism has on turbidity removal because the aluminum species between pH 6.0 and 6.4 is the trivalent aluminum cation (Li et al. 2006). Charge neutralization creates a stronger bond when forming floc and requires a lower coagulant dose than sweep flocculation (Alameddine et al. 2021). But because the full-scale plant's pH is not controlled and typically ranges from 6.5 to 7.2, coagulation is more strongly associated with the sweep flocculation mechanism at the DWWTF.

4.1.2.2 Alum as Function of Hydraulic Retention Time

Unlike in Figure 4.4.A) - C) where the required coagulant dose trended downward as the turbidity increased, Figure 4.4.D) and E) did not depict this trend when the turbidity was less than 90 NTU. Instead, Figure 4.4 illustrated that as the HRT decreased (Figure 4.4. A-E), turbidity removal was impeded when influent turbidity was less than 90 NTU, no matter the coagulant dose (as noted by Figure 4.4 becoming redder between A and E).

Based on this observation, higher floc density was required to compensate for the decreased HRT. Without adding a ballast, the concentration of particulates in the influent must increase (Ghernaout and Ghernaout 2012; Cruz et al. 2020). Once the HRT was less than 24 minutes for one Densadeg train, and the influent turbidity was less than 90 NTU, the chemical dose alone was not adequate for treatment. In this case, a ballast is required to increase the density of the flocs to meet the required settling velocity (Young and Edwards 2003). As shown in Figure 3.2, higher influent flows at the DWWTF typically correspond to lower influent turbidities. Therefore, these results suggest that without the aid of ballast, the HRT is too short during wet weather events or when the flow is greater than 2400 m^3/hr per Densadeg train.

4.2 Results of Ballasted Pilot Study

Jar testing revealed the limitations of chemical treatment at high HRTs and low influent turbidities. A pilot study involving an external ballast was investigated to determine if an external ballast could be used to overcome settling challenges associated with the short HRTs of the system that occur under high influent flows (e.g., during wet weather events). The pilot study ran for 13 weeks, from August 3rd, 2020, through October 20th, 2020. Piloting was separated into three distinct Phases:

1. Phase 1. Startup
2. Phase 2. Use of bench-scale chemical optimization dosage
3. Phase 3. Meet WSER limits

Start-up involved significant operational and system changes to ensure the pilot was operating effectively. As a result, no results are provided for Phase 1. Phase 2 involved operating the pilot at the optimized alum dose and adjusted polymer dose for ballast first identified in the bench-scale study to determine the effect of ballast on treatment performance. Phase 3 involved adjusting the coagulant and polymer doses to determine if meeting the WSER limit was possible. The following sections offer results for Phase 2 and Phase 3. The magnetite ballast was maintained within the ranges prescribed by the plant manufacturer (e.g., 5 to 10 mg/L). The average magnetite concentration in the system was 5.2 mg/L (with a standard deviation of 2.0 mg/L) and 6.9 mg/L (with a standard deviation of 3.7 mg/L) for Phase 2 and Phase 3, respectively.

4.2.1 Ballasted Pilot Study Results - Phase 2

The objective of Phase 2 was to use the optimized chemical dose range determined from the bench-scale study and determine the effects of ballast on TSS, $cBOD_5$, and $scBOD_5$ removal when using optimized chemical dosing.

4.2.1.1 Operation and Water Quality of Phase 2

As shown in Figure 4.5, during Phase 2, the pilot consistently had a higher SOR than the full-scale plant. The pilot handled a SOR 41% greater than the full-scale plant (Table 4.1).

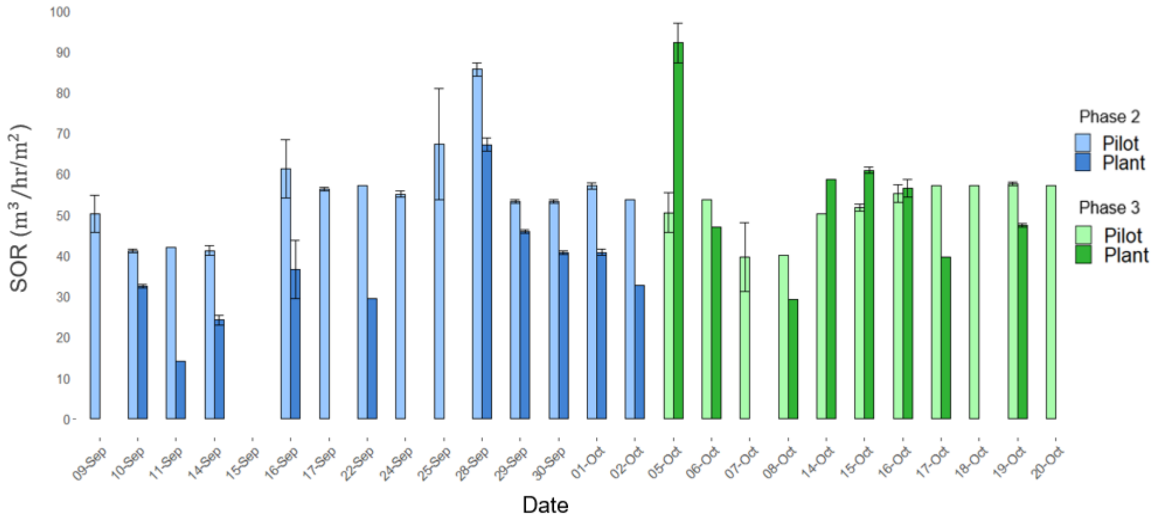


Figure 4.5: Average daily surface overflow rate of the pilot clarifier (light shade) and full-scale plant clarifier (dark shade) overtime where blue is Phase 2 and green is Phase 3

The influent quality results for both the pilot and full-scale plant were similar in value. They are not statistically significant, ensuring that any effluent result difference was from the difference in treatment processes and not a difference in influent water quality (Table 4.1).

Table 4.1 Influent water qualities from the ballasted pilot study Phase 2. Compares each variables' average and standard deviation for pilot and full-scale plant. Depicts the p-value from a paired two-tail test where red indicates not statistically significant, green has a 95% confidence for statistical significance.

	Pilot		**Plant**		**% Difference**	**Paired T-test**
	Average (±SD)		Average (±SD)		$\frac{\lvert Pilot - Plant \rvert}{(Pilot + plant)/2} \cdot 100$	*p*-value (α=0.5)
Average SOR ($m^3/hr/m^2$)	**55.2**	**± 11.5**	**36.3**	**± 14.1**	**41**	***P*<.001**
Average Influent Turbidity (NTU)	**133.7**	**± 39.5**	**146.1**	**± 37.1**	**9**	***P*=.11**

Average Influent UVT (%)	**10.5**	**± 4.0**	**12.0**	**± 3.3**	**14**	***P*=.03**
Average Influent TSS (mg/L)	**504.8**	**± 348.1**	**708.5**	**± 262.2**	**34**	***P*=.06**

4.2.1.2 Results and Discussion of Phase 2

Table 4.2 describes the average chemical dosing of coagulant and polymer during Phase 2 for the pilot and the full-scale plant. On average, the pilot coagulant was dosed at 6.6 mg/L, which was in the optimal range determined during bench-scale testing (5 to 8 mg/L). The pilot polymer was dosed at 2.25 mg/L on average. The polymer dose was outside the optimal range specified during bench-scale testing (0.5 to 1.5 mg/L). The discrepancy between the required polymer dose from the bench-scale to the pilot-scale was likely because more polymer was needed to incorporate the ballast into the chemical floc, similar to findings in the literature (Young and Edwards 2003). The discrepancy could also occur because the pilot was not well equipped to handle the large influent TSS. Based on these results, a higher range of required polymer doses could be expected if a ballasted flocculation system were to be implemented.

As depicted in Figure 4.6, the effluent TSS of both pilot and full-scale plants was never consistently below the 25 mg/L TSS required in the WSER. It was expected that the daily grab samples taken during the day’s working hours would be greater than regulations because samples for regulatory reporting are done in 24-hour composites (Government of Canada 2012). A 24-hour composite sampling is a better indicator for the full-scale plant as the influent quality and flows are variable throughout the 24 hours (APHA, AWWA, and WEF 2017). Because of limitations of the pilot and on-site regulations, composite samples were not feasible.

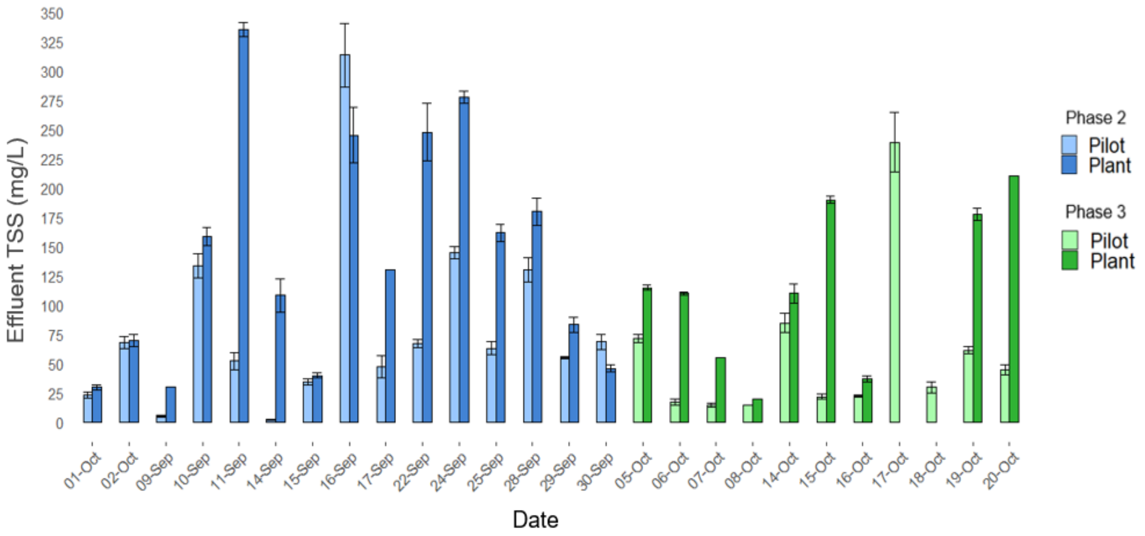


Figure 4.6 Average effluent TSS column plot of the pilot (light shade) and full-scale plant (dark shade) over time, where blue is Phase 2 and green is Phase 3

During Phase 2, the pilot average effluent TSS was 56 % less than the full-scale plant (Table 4.2). The turbidity and UVT followed similar trends as the TSS, where the pilot outperformed the full-scale plant (Table 4.2, Appendix A. 3, Appendix A. 4). Although pilot and full-scale plant averages are not statistically significant, the trend still indicates the potential for improved treatment with ballasted flocculation. Pilots intended to be identical to their full-scale plant cannot consistently achieve statistical significance (Knowles, Mackay, and Gagnon 2012). In addition, as previously discussed, during Phase 2, the SOR rate of the pilot was 41% greater than the full-scale plant. Therefore, the pilot system was able to achieve improved water quality over the full-scale plant, even under higher loading conditions (Table 4.2)

Table 4.2 Effluent water qualities from the ballasted pilot study Phase 2. Compares each variable's average and standard deviation for pilot and full-scale plants. Depicts the p-value from a paired two-tail test where red indicates not statistically significant, green has a 95% confidence for statistical significance.

	Pilot		**Plant**		**% Avg. Difference**	**Paired T-test**
	Average (+SD)		Average (+SD)		$\frac{\lvert Pilot - Plant \rvert}{(Pilot + plant)/2} \cdot 100$	*p*-value (α=0.5)
Average Alum Dose (mg/L)	**6.6**	**± 0.9**	**4.6**	**± 0.9**	**36**	***P*<.001**
Average Polymer Dose (mg/L)	**2.3**	**± 0.4**	**1.3**	**± 0.2**	**50**	***P*=.02**
Average Effluent TSS (mg/L)	**80.7**	**± 77.5**	**142.9**	**± 97.8**	**56**	***P*=.02**
Average Effluent TSS Removed (%)	**86.8**	**± 6.5**	**83.4**	**± 10.8**	**4**	***P*=.42**

The pilot was able to outperform the full-scale plant for two main reasons. First, the optimized chemical dose (for the ballasted system), which was near the range of the bench-scale, was applied. The pilot dosed at 6.6 mg/L of alum within the optimized range of 5-8 mg/L determined from the bench-scale. At the time, the full-scale plant was dosing at a lower coagulant dose of 4.6 mg/L. The pilot outperformed the full-scale plant based on the effluent quality TSS (Table 4.2). The caveat of this objective was that the polymer dose (2.3 mg/L) had to be double the bench-scale dose (0.5-1.25mg/L) to compensate for the ballast. Secondly, the ballast supplied at the pilot-scale increased the settling velocity of the chemically formed flocs enough to overcome the low hydraulic retention times, similar to results within the literature (Young and Edwards 2003).

Based on both the bench-scale and the pilot-scale results, an effective way for a chemically enhanced primary treatment system to compensate for high flows and their subsequent low hydraulic retention time in the clarifier is to add ballast. This is a similar conclusion determined in other studies (Imasuen, Judd, and Sauvignet 2004; Hun 1998). The current Densadeg system of the full-scale plant is a high-rate clarification system designed to recycle the sludge to act as a ballast (Lapointe et al. 2017). Historically, this facility does not consistently operate the sludge ballast recycle within the manufacturer's prescribed recycle rates. Results from this study indicate that the use of magnetite ballast could be beneficial.

4.2.2 Ballasted Pilot Study Results - Phase 3

The purpose of Phase 3 was to investigate whether the ballasted system could achieve the WSER regulation requirements of 25 mg/L TSS and 25 mg/L $cBOD_5$.

4.2.2.1 Operation and Water Quality of Phase 3

In Phase 3, corrections were made to have the pilot and full-scale plant SOR be more equivalent (Table 4.3). Additionally, all influent water qualities were statistically equivalent, and therefore any conclusions drawn are not because of influent variability (Table 4.3).

Table 4.3 Influent water qualities from the ballasted pilot study Phase 3. Compares each variable's average and standard deviation for pilot and full-scale plants. Depicts the p-value from a paired two-tail test where red indicates not statistically significant, green has a 95% confidence for statistical significance.

	Pilot	**Plant**	**% Difference**	**Paired T-test**

	Average (±SD)		Average (±SD)		$\frac{\lvert Pilot - Plant \rvert}{(Pilot + Plant)/2} \cdot 100$	p-value (α=0.5)
Average SOR ($m^3/hr/m^2$)	**51.7**	**± 6.5**	**53.9**	**± 18.7**	**4**	***P*=.78**
Average Influent Turbidity (NTU)	**70.8**	**± 19.5**	**75.7**	**± 21.8**	**14**	***P*=.23**
Average Influent UVT (%)	**18.8**	**± 3.5**	**20.9**	**± 5.7**	**10**	***P*=.17**
Average Influent TSS (mg/L)	**248.6**	**± 149.2**	**268.7**	**± 126.4**	**8**	***P*=.28**

4.2.2.2 Results and Discussion of Phase 3

In Phase 3, as seen in Figure 4.6, the pilot outperformed the full-scale plant by 67% of effluent TSS (Table 4.4). The pilot had a 79.1% TSS removal compared to the full-scale plant's 58.0%, as referred to in Table 4.4. Both turbidity and UVT followed similar trends and were significantly different from the full-scale plant's effluent (Appendix A. 3, A. 4). As shown in Table 4.4, the pilot plant was operated at an alum dose of 12mg/L, which was approximately double. The belief was that by doubling the chemical dose, the TSS removal rate would increase, and the pilot would meet the WSER TSS requirements (Zhu et al. 2007). Despite the increased alum and polymer doses for Phase 3, the pilot was unable to produce effluent water with a TSS less than 25 mg/L with twice the amount of chemicals dosed.

Table 4.4 Effluent water qualities from the ballasted pilot study Phase 3. Compares each variable's average and standard deviation for pilot and full-scale plants. Depicts the p-value from a paired two-tail test where red indicates not statistically significant, green has a 95% confidence for statistical significance.

	Pilot		**Plant**		**% Difference**	**Paired T-test**
	Average (±SD)		Average (±SD)		$\frac{\lvert Pilot - Plant\rvert}{(Pilot + plant)/2} \cdot 100$	*p*-value (α=0.5)
Average Alum Dose (mg/L)	**12.0**	**± 2.4**	**4.1**	**± 0.9**	**98**	***P*<.001**
Average Polymer Dose (mg/L)	**4.0**	**± 1.0**	**1.4**	**± 0.3**	**94**	***P*<.001**
Average Effluent TSS (mg/L)	**56.8**	**± 65.2**	**113.9**	**± 68.2**	**67**	***P*=.008**
Average Effluent TSS Removal (%)	**79.1**	**± 12.5**	**58.0**	**± 20.1**	**31**	***P*=.008**

4.2.3 Ballasted Pilot cBOD5 and scBOD5

Figure 4.7 depicts the differences between pilot and full-scale plants during Phase 2 and Phase 3 of cBOD$_5$, but it also illustrates what fraction of the remaining cBOD$_5$ is scBOD$_5$.

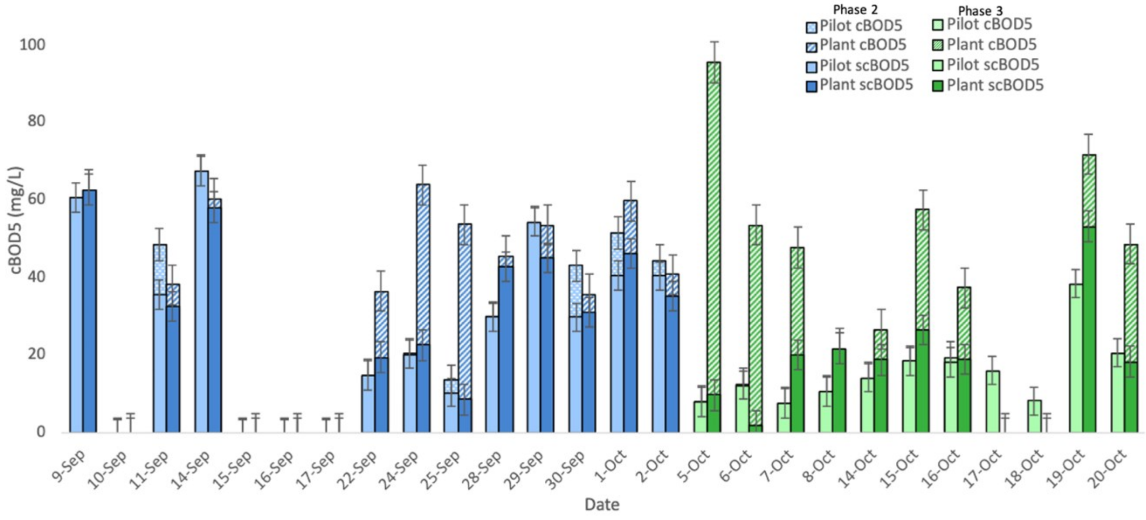


Figure 4.7 Average effluent cBOD5 column plot of the pilot (dot pattern), scBOD5 of the pilot (light shade), cBOD5 of the full-scale plant (stripe pattern), and scBOD5 of the full-scale plant (dark shade) over time where blue is Phase 2 and green is Phase 3

The critical result gained from the pilot study was evaluating the $scBOD_5$ as a fraction of $cBOD_5$. From Figure 4.7, the remaining effluent amount of $cBOD_5$ is likely $scBOD_5$. Results presented in Table 4.5 show that the effluent $cBOD_5$ and $scBOD_5$ of the pilot are statistically insignificant, further proving that the remaining $cBOD_5$ in the system is $scBOD_5$. The greatest $scBOD_5$ percent removed from the pilot was 30%. Assuming the system is chemically optimized, and the magnetite ballast accomplishes similar results as sludge ballast, this 30% removal matches values found in the literature that a Densadeg enhanced primary treatment system would be expected only to remove 30% $scBOD_5$ (Qasim and Zhu 2018; Leng et al. 2004). Therefore, if the influent $scBOD_5$ is near the WSER $cBOD_5$ limit of 25 mg/L, a magnetite ballasted chemically optimized enhanced primary treatment process that does not control influent flow, temperature, or pH will not be able to meet the $cBOD_5$ WSER regulation without the use of secondary biological treatment after primary wastewater treatment secondary biological treatment is implemented. In comparison to primary treatment, biological treatment could remove soluble pollutants but requires longer detention times for the biological activity to occur (Morris et al. 2016).

Table 4.5 Effluent water qualities from ballasted pilot study Phase 2 and 3. Compares the average and standard deviation of $cBOD_5$ and $scBOD_5$ for the pilot. Presents the percent difference between the variables $cBOD_5$ and $scBOD_5$ and depicts the p-value from a paired two-tail test where red indicates not statistically significant, and green has a 95% confidence for statistical significance.

		cBOD5		scBOD5		% Difference	Paired T-test
		Average (+SD)		Average (+SD)		$\frac{\lvert Pilot - Plant\rvert}{(Pilot + plant)/2} \cdot 100$	p-value (α =0.5)
Phase 2	**Average Influent Pilot (mg/L)**	152.6	53.4	58.7	+ 33.1	89	P<.001
	Average Effluent Pilot (mg/L)	39.0	+ 18.3	39.9	+ 18.9	2	P=.70
	Average Effluent Removed Pilot (%)	73.5	+ 8.3	30	+ 17.2	84	P<.001
Phase 3	**Average Influent Pilot (mg/L)**	71.3	± 36.9	21.0	± 19.5	109	P=.10
	Average Effluent Pilot (mg/L)	13.1	± 4.7	17.9	± 8.7	31	P=.20
	Average Effluent Removed Pilot (%)	77.5	± 11.7	24.4	± 35.0	104	P=.04

CHAPTER 5 RECOMMENDATIONS AND CONCLUSION

5.1 Objectives

The objectives of this research project were to:

1. Determine the optimal combination of coagulant and polymer dose (neat) at variable influent turbidities and hydraulic retention times to determine if chemical optimization alone can enable CEPT facilities to achieve the WSER TSS and $cBOD_5$ requirements.
2. Investigate the ability of addition of external ballast material to improve settling efficiency CEPT to assist in achieving the WSER TSS and $cBOD_5$ requirements

5.2 Key Findings

The key findings of the project were:

- The jar tests determined that DWWTF should dose their alum between 5 and 8mg/L and that the polymer dosing range should be between 0.5 and 1.25 mg/L.
- From the bench-scale, coagulant dosing is a function of influent turbidity. Where higher doses of coagulant and polymer are required at lower influent turbidity values to compensate for the lack of concentrated particulates in the system when influent turbidity is less than 90 NTU.
- The bench-scale testing depicted that when the HRT is less than 24 minutes, optimized chemical dosing alone cannot remove the turbidity efficiently when the influent turbidity is less than 90 NTU

- From the pilot study, a chemically optimized dosage of coagulant and polymer with the addition of ballast had 56% better water quality based on TSS than the full-scale plant, even though the SOR of the pilot-scale was 41% higher.

- A magnetite ballasted and chemically optimized enhanced primary treatment pilot-scale system without sludge recycle and influent characteristics manipulation did not remove more than 30% of the influent $scBOD_5$. It is likely that DWWTF will not be able to meet the WSER limits using the investigated chemical dosing and ballast conditions without the addition of a secondary biological process when $scBOD_5$ is greater than or near 25mg/L.

Chemically enhanced primary systems cannot be optimized by the chemical dose alone. Although increased coagulant decreases the remaining particulates in the solution and polymer forms heavier flocs, the proper quantity alone is insufficient to properly settle when lower hydraulic retention times occur in the clarification process. During these high flows, the full-scale plant could increase the settling velocity of the forming flocs to overcome the short HRT by either adding an artificial ballast, such as CoMag® , or using a contact sludge recycle system (Hun 1998). The ballasted flocculation pilot work proved that the ballast does increase the settleability of the flocs at the optimized chemical dosage. Although the ballast and chemical optimization improve the water quality, further upgrades to the full-scale plant are required to meet the demands of the cBOD5. The $scBOD_5$ left in the effluent water quality is too great for a chemically enhanced primary system to treat without using a biological process. Plants that are chemically enhanced primary treatments or have soluble particulates that have increased in their systems would benefit from similar optimizations but will likely require secondary treatment to meet the WSER discharge limits.

5.3 Future Recommendations

Implementing the chemical coagulant and polymer dose ranges determined from jar tests at the full-scale is recommended to optimize chemical treatment at the full-scale. After optimization of coagulant and polymer at full-scale, it is recommended to optimize the sludge ballast recycle process and sludge handling system to ensure a proper sludge blanket is always present and to determine what the recycle rate of the system should be.

Additional jar tests could also investigate the use of pH control and determine if less chemical is required by shifting the primary coagulation method from sweep flocculation to charge neutralization. The zeta potential of the system could be studied to ensure charge neutralization.

Further research is recommended to review the historical trend of soluble $cBOD_5$ to determine if a biological process is required for the $cBOD_5$ to meet the WSER limit. An investigation of secondary biological processes at the bench-scale or pilot-scale is recommended to ensure that the additional unit process is the best fit.

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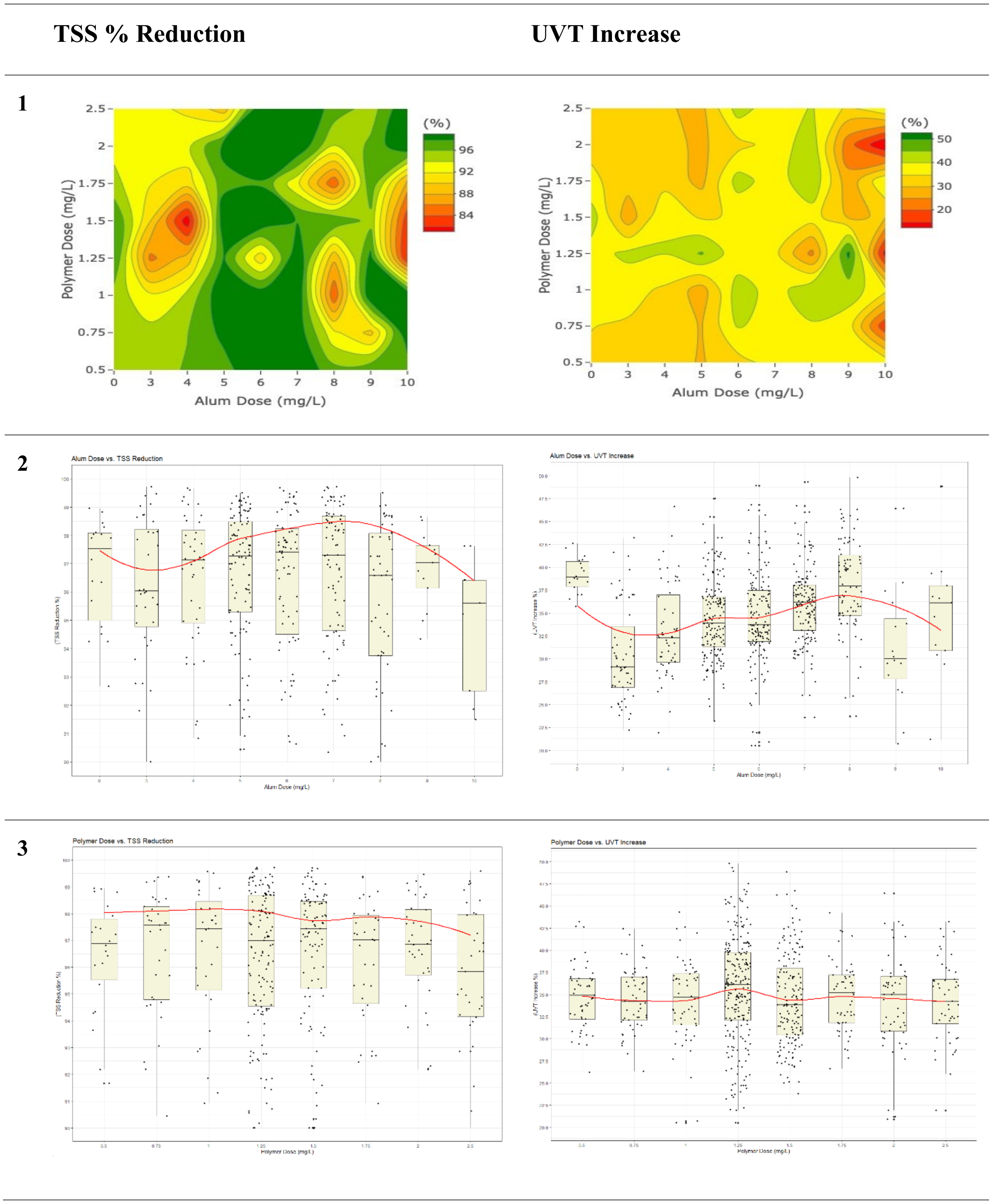
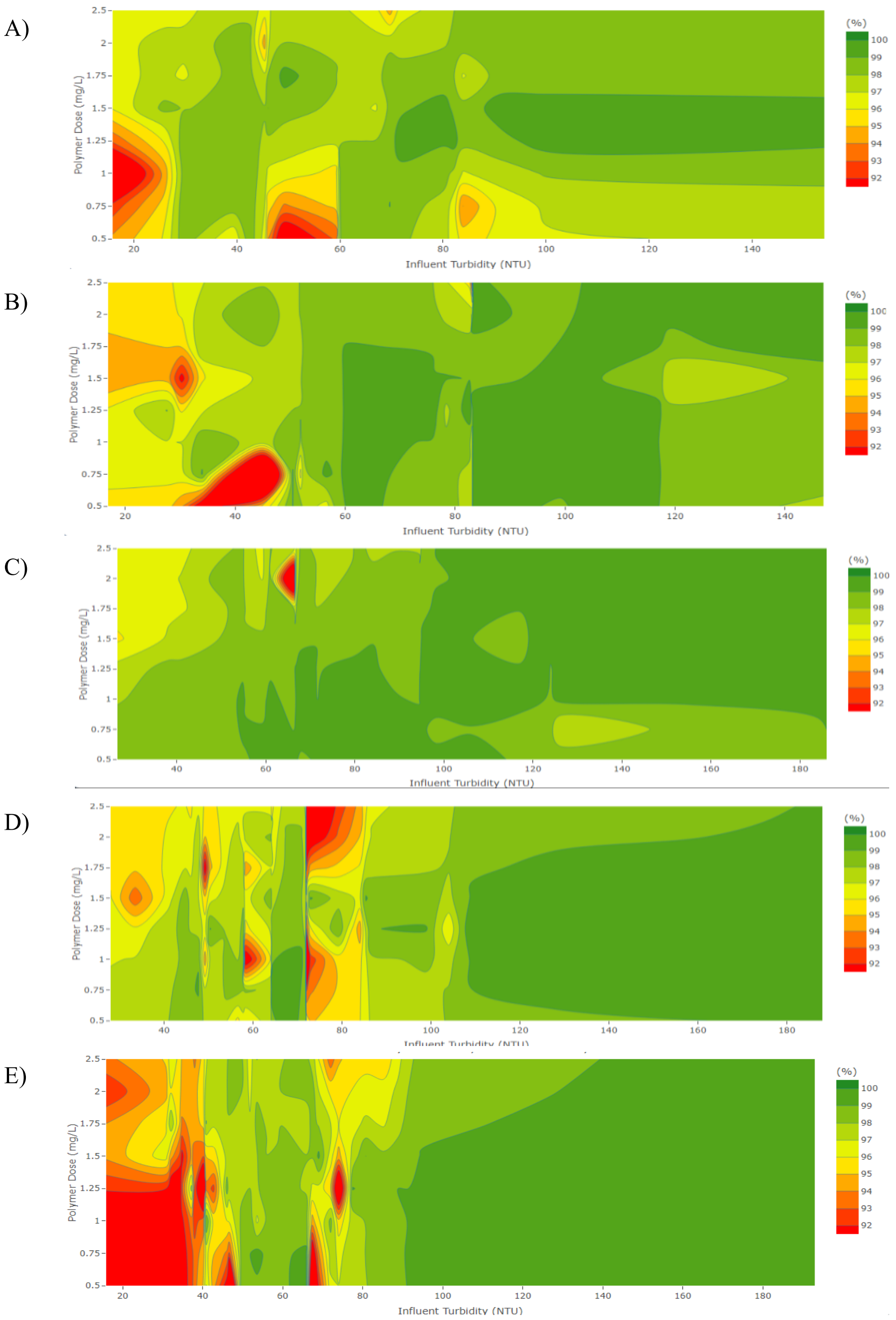
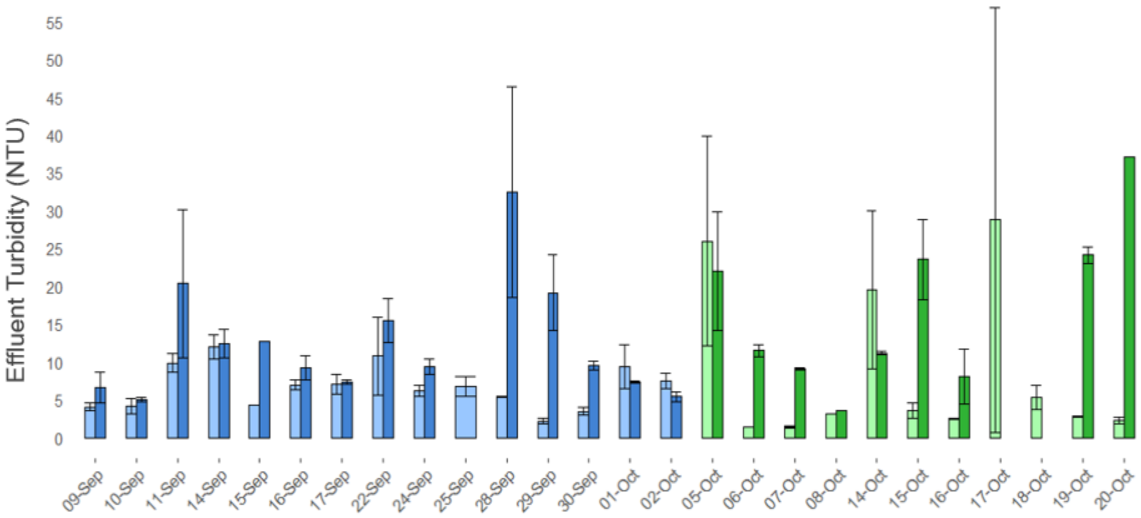
APPENDIX A



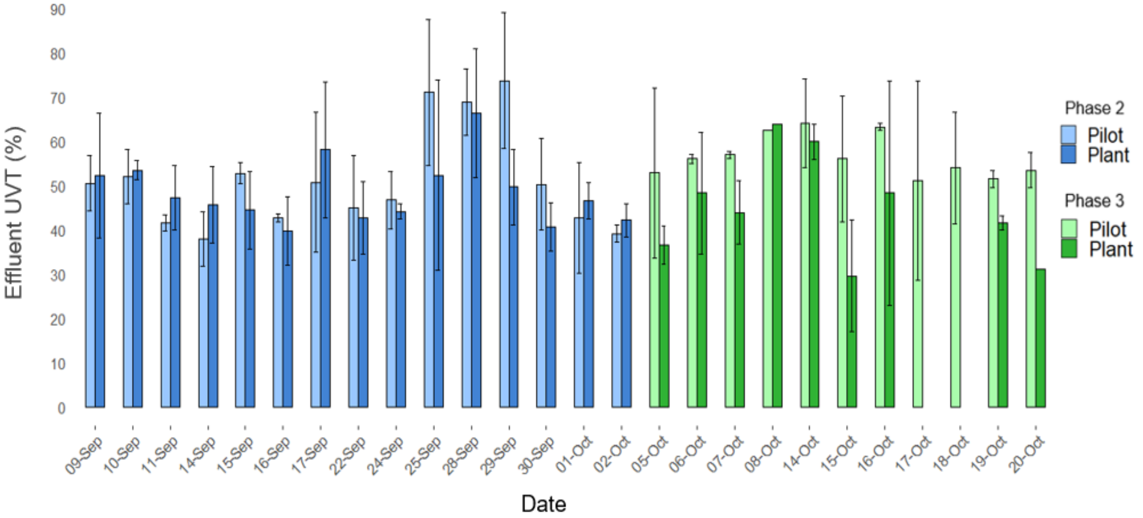
A. 1 Comparison of water quality results from all Jar Tests conducted where (A) is the TSS percent removal is the UVT increase. Row (1) is the contour plot where the color represents the water quality parameter. Row (2) and Row (3) are boxplots for alum and polymer dose of each water quality parameter, respectively.



A. 2 Contour plots where the y axis is polymer dose, the x-axis is Influent turbidity values, and the color is turbidity removal as a percentage. Plot (A), (B), (C), (D), and (E) is the hydraulic retention times of 51, 39, 32, 24, and 19 minutes, respectively.



A. 3 Average effluent turbidity column plot of the pilot (light shade) and full-scale plant (dark shade) over time where blue is Phase 2 and green is Phase 3 *Exclusion of an outlier value from the full-scale plant effluent on September 22nd



A. 4 Average effluent UVT column plot of the pilot (light shade) and full-scale plant (dark shade) over time where blue is Phase 2 and green is Phase 3