DESIGN IMPROVEMENTS FOR THE ACTIVATED SLUDGE WASTEWATER TREATMENT SYSTEM AT THE AMERICAN FARM SCHOOL IN THESSALONIKI, GREECE

A Major Qualifying Project Submitted to the Faculty of Worcester Polytechnic Institute in partial fulfillment of the requirements for the Degree of Bachelor of Science in Civil and Environmental Engineering

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Abstract

The on-site activated sludge wastewater treatment system (WWTS) of the American Farm School (AFS) in Thessaloniki, Greece has been inoperative since 2012. The system was designed to treat all agricultural product processing wastewater on-site to acceptable levels of BOD_5, COD, pH, TSS, P, N, and grease/oils for irrigation reuse on the school's crops. As a result of the system being out of commission, this wastewater as well as domestic wastewater was being sent to the municipal wastewater treatment plant. This process could not continue due to municipality regulations. The purpose of this project was to provide information to enable restarting the WWTS as well as suggest economically feasible design improvements that would accommodate additional high organic loads and irregular flowrates anticipated in this system. Samples of wastewater were collected from various sources periodically between the months of October 2015 and December 2015 for chemical analysis and flowrate measurements, both taken simultaneously. It was determined that the irregular influent flowrates and alkaline wastewater sources were the major contributors to system inefficiency. To accommodate the high fluctuation in flowrate, the first aeration and settling tank was evaluated for use as an equalization basin, a modification that would enable a regulation of both flowrate and BOD to the second aeration tank and clarifier and result in acceptable effluent quality. Because of the nature of dairy processing wastewater, which contains concentrations of basic cleaning chemicals, pH adjust was incorporated into the recommended design to decrease the pH from 10.1 to 7.5-8. The effluent quality would be sufficient for irrigation reuse at AFS and the domestic wastewater would again be sent alone to the municipal treatment plant. With these design improvements, AFS will meet discharge requirements and achieve a high quality wastewater effluent for use on site for irrigation.

Collaboration

This project was a collaborative effort completed by four WPI students. Christopher Cerruti, Haley Morgan, and Matheus Pereira completed this project in fulfillment of the Degree of Bachelor of Science in Civil Engineering. Nikos Kalaitzidis completed this project in fulfillment of the Degree of Bachelor of Science in Environmental Engineering, under the guidance of Professor John Bergendahl. Nikos Kalaitzidis completed an additional one-third credit Major Qualifying Project in fulfillment of the Degree of Bachelor of Arts in International Studies, under the guidance of Professor Bland Addison.

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Professional Licensure

Becoming a licensed professional engineer (PE) is an extensive process that ensures competency and maintenance of an engineer's skills and knowledge. The process involves the successful completion of several steps before an engineer is allowed to use the PE seal. These steps include earning an accredited four-year degree in an engineering program, passing the Fundamentals of Engineering (FE) exam, four years of work experience directly under a PE, and successful completion of the Principles and Practice of Engineering exam. A licensed PE differs from an engineer in that solely PE's can prepare, sign, seal, and submit engineering plans and drawings for clients. Licensure is an important responsibility, because the PE is responsible for the work and lives affected by the work prepared under his or her seal. This requires licensed professionals to uphold the highest ethical standards when considering the work they seal. Licensure is becoming increasingly important, especially in government positions, where higher levels of responsibility are desired. Consulting engineers who are in charge of work for a client are required to be licensed (National Society of Professional Engineers, 2016).

MQP Capstone Design Statement

For this project, the team designed improvements for an activated sludge wastewater treatment system (WWTS) at the American Farm School (AFS) in Thessaloniki, Greece. The activated sludge WWTS was intended to treat agricultural wastewater produced on-site from the dairy processing facility in order to make it suitable for irrigation reuse. AFS was most concerned with the high levels of BOD. Samples of the raw wastewater were taken and analyzed in a laboratory to test for pH, BOD, COD, suspended solids, grease/oils, nitrogen, and phosphorus. Flowrate measurements were taken simultaneously with wastewater quality samples. Through analysis of laboratory results and hydraulic data, the group was able to estimate influent conditions that the improved wastewater treatment system would expect to see.

The existing infrastructure of the activated sludge WWTS had been previously decommissioned. The group intended to capitalize on these existing unused tanks in order to minimize the cost of implementation of the proposed design. Using AutoCAD files that detailed the dimensions of the system, along with the predicted influent flow conditions, the group was able to calculate the treatment abilities if using the tanks in sequence. The group designed a system in which the raw wastewater first enters an equalization basin as a means to attenuate the high fluctuation in flowrate, before entering into an aeration basin and clarifier. The attenuated flowrate, calculated to optimally run at 10.10 m^3/hr, and adjusted pH would allow the designed activated sludge treatment system to see consistent influent conditions, and therefore have a consistent effluent quality. This designed system would allow the wastewater to be treated to BOD levels of less than 2 mg/L.

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1. Introduction

The American Farm School (AFS) was founded in 1902 under the notion that man needed to learn through doing:

> "My idea in forming the school was, if possible, to embody in the School a system of education which would train the whole man, the mind and hand as well as the soul."
>
> - Dr. John Henry House (Draper, 1994, p. 9)

The American Farm School is not just for growing crops and raising cattle for teaching purposes; the school also has a history with poultry. In fact, apart from producing chicken eggs, it was also the first commercial producer of turkeys for slaughtering in Thessaloniki, thanks to the American influence of its founding (Nikolaidis, 2015). All three of these forms of hands-on teaching produce wastewater that must be treated before returning to the ground where it joins an aquifer beneath the school. The activated sludge wastewater treatment system at AFS has been inactive for three years. Regulations of the Thessaloniki Water Supply and Sewage Company (EYATH) require AFS to separate non-domestic from domestic wastewater. However, the school has been diverting dairy processing wastewater to EYATH along with all of the campus domestic wastewater. EYATH requires this segregation in order to regulate the amount of biochemical oxygen demand (BOD) that is treated by the city wastewater treatment plant.

Ideally, AFS would continue to have campus domestic wastewater treated by EYATH except for wastewater from three sources: the elementary school, gymnasium and maintenance department. The end goal is that these three domestic sources, along with the dairy processing wastewater, are sent through AFS's existing on-site activated sludge wastewater treatment system (WWTS).

AFS also operates a slaughterhouse for approximately 20 days each year. The wastewater produced currently undergoes a separate solids removal treatment before being discharged into Lagoon

A. All wastewater that passes through Lagoon A undergoes anoxic aerobic treatment and eventually is reused for irrigation. This project analyzed the feasibility of incorporating the treatment of slaughterhouse wastewater in the proposed on-site activated sludge wastewater treatment system along with the aforementioned wastewaters. It also proposed recommendations on how to continue with the treatment of the slaughterhouse wastewater.

In order to satisfy national treated effluent reuse regulations, final effluent from AFS's treatment plant should meet BOD, COD, TSS, nitrogen, and phosphorus levels for the irrigation of corn, which is fed exclusively to the AFS cows. By optimizing the current infrastructure and considering the most cost effective strategies, the team developed an improved design which, pending AFS approval will be implemented after the team's departure.

2. Background

2.1 History of the American Farm School

Originally purchased for $1,000 the 53-acre farm school was designed for 13 Macedonian orphans as a result of Turkish massacres. It has since grown to have a student body of 240 on 320 acres of land (Draper, 1994). Figure 1 shows aerial views of the school's campus in 2015 and 1925.

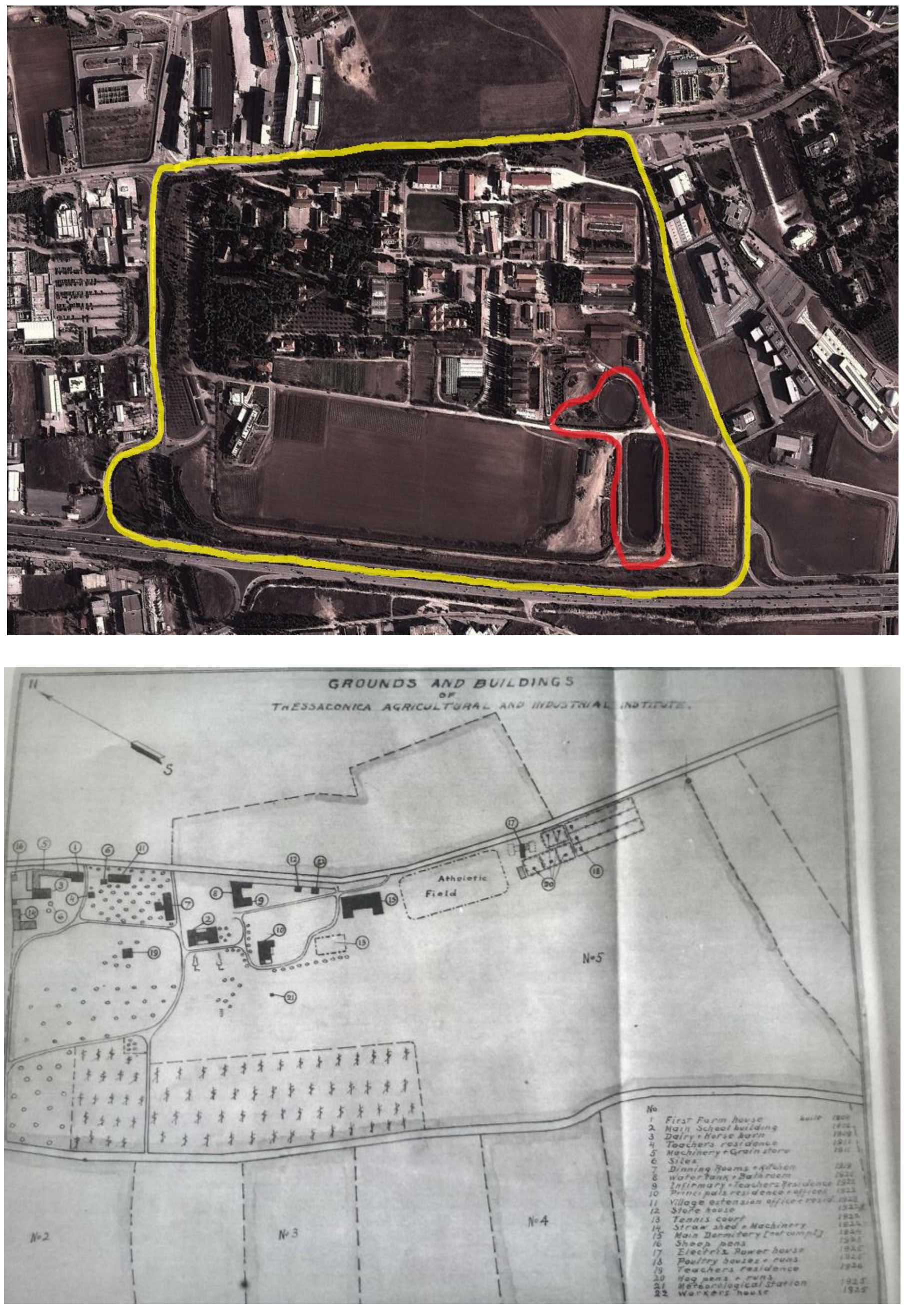

Figure 1: Aerial View of AFS in 2015 (top) and 1925 (bottom) (Draper, 1994)

The original farm was located directly above an aquifer, allowing Dr. House to drill his own wells for irrigation and personal use. In 1902 only two wells were drilled to a depth of approximately 175 feet (Marder, 2004). This amount of water was enough to supply the farm as well as run a water line to the nearby road so that passing travelers could alleviate their thirst in such a dry region. This system was operational until 1911 when the wells stopped producing water and AFS created an agreement with the owner of a neighboring farm, Ali Bey, to purchase water for drinking and watering the crops until another well was drilled and water was once again being supplied from locations on the property (Marder, 2004). The project of drilling new wells was not completed until after WWI when the British sold old and spare parts they were not planning on bringing back to England when the war was over. Once equipment was acquired, the next issue was finding more water; again the British aided AFS in this task by sending an American contractor to the school with modern equipment to drill new wells. This American contractor sank a 280-foot well, capable of yielding an incredible 800 gallons per hour (Marder, 2004).

The American Farm School is not just for growing crops for teaching purposes; the school also has a history with raising poultry. Egg laying chickens were part of the first additions that AFS made as it expanded; in 1955 a Rockefeller Foundation grant supported a new program to raise chickens used for meat. This slowly made chicken a much more affordable commodity instead of previously being a luxury meat (Draper, 1994). Chickens, both for eggs and meat, are not the only bird being raised on campus. Turkey chicks were first introduced by AFS in the early 1960s to raise and sell specifically for Christmas and Thanksgiving (Draper, 1994). Even though Thanksgiving is an American holiday, it is celebrated on the AFS campus due to its American roots (Nikolaidis, Personal Communication, 2015). The use of turkeys on the farm continues today for farm training purposes as well as business training.

Ever since the school was founded water has been a concern, not for the lack of available water but with the treatment of wastewater. Historically, AFS had been collecting wastewater through a sewer network flowing into large storage lagoons on campus, where it would later be used for crop irrigation. In 1970 AFS began treating its wastewater by using a two chamber septic tank, which was upgraded in 1983 to be the first primary treatment using activated sludge. This singular settling tank was the only treatment until the late 1980s when an additional aeration tank and clarifier were added to the process (AFS Archives, 1983). This initial system was designed and built by a local engineering company, Sotiropoulos-Peltekis Engineers, in 1983 based on German waste management standards (AFS Archives, 1983). The second stage of the process was based on the design proposed by an English design company, IMES (AFS Archives, 1983). Operation of this two stage system continued until 2012 when AFS made an agreement to begin pumping wastewater to EYATH.

After the system had been decommissioned, the coarse grain solid removal screen and settleable solids removal were the only treatments, before being pumped to EYATH that continued to operate. Every day the coarse grain screen is mechanically raised and emptied into a waste bin. Liquid waste continues to a pumping chamber where it is pumped ideally into the first stage of the treatment process. The treatment system was designed at a location where runoff would not affect the rest of the campus (Nikolaidis, 2003). Aeration occurs in a central compartment of the first stage in treatment by means of air piping. Vertical flow settling compartments are attached to both sides of the aeration tank. Settling occurs as wastewater flows up the sides of the tank, allowing the denser mixed liquor suspended solids (MLSS) to remain at the bottom. The water gets filtered as it is pushed through the settled MLSS. The two phases in Stage 1 are separated by an asbestos cement sheet. An outflow canal runs the length of the tank in the outer section designed for overflow in the case of abnormal influent flow. After the wastewater has been treated in Stage 1 it is piped into Stage 2 where it is again treated using aeration and settling but in two different chambers. The first phase in Stage 2 is coarse bubble diffusion from the bottom of the chamber. The second phase in Stage 2 is a settling chamber with an air lift designed as a recycle system for settled flocs. The flocs that are recycled are piped back into Stage 1 or can be discharged into a holding lagoon, Lagoon A. The settling chamber in Stage 2 also has overflow canals similar to Stage 1 where excess wastewater will be diverted to the holding lagoon. Following the aeration and settling of Stage 2, treated wastewater is piped into Lagoon A, where it continues to be aerated. Lagoon A is a concrete walled lagoon 60 m in diameter capable of storing approximately 20,000 m^3 of water. The treatment continues in this lagoon with a two stage anoxic-aerobic process with the help of special catalytic $CaCO_3$ powder, two mechanical surface aerators alongside 16 vertical flow reed beds. Sludge from the bottom of Lagoon A is removed every few years when the buildup gets too great. Effluent from Lagoon A is gravity fed into an adjacent holding lagoon, Lagoon B, capable of storing approximately 40,000 m^3, where it undergoes UV disinfection. The final effluent water is used for the following year's irrigation of the corn used for cow feed. Figure 2 shows the treatment system just described.

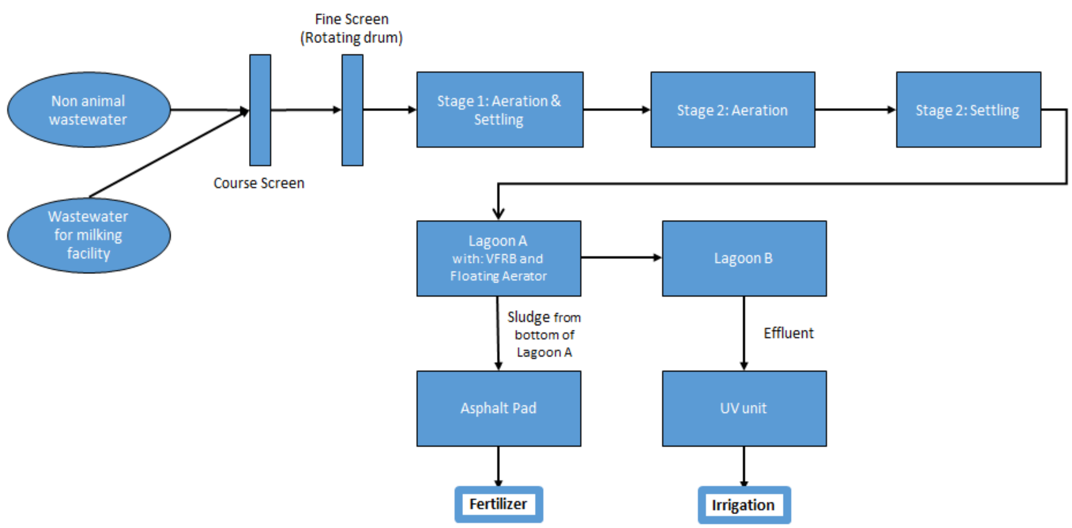

Figure 2: Existing AFS Treatment System

2.2 The Activated Sludge Process:

The activated sludge process is a method of aerobic biological wastewater treatment that was invented in the early 1900s by two Englishmen, Edward Arden and William Lockett (Droste, 2014). Today, it is the most commonly implemented process used to treat municipal wastewater (Metcalf and Eddy, 2003). Since its invention, a number of variations from the original two-step design have been developed. However, the basic idea of the activated sludge wastewater treatment system is a two-stage process that involves both an aeration stage and a settling stage, as shown in *Figure 3: Activated Sludge Process Schematic*. For the purposes of this project, we looked at a system that involved return sludge, or the recycling of sludge from the clarifier back into the aeration basin.

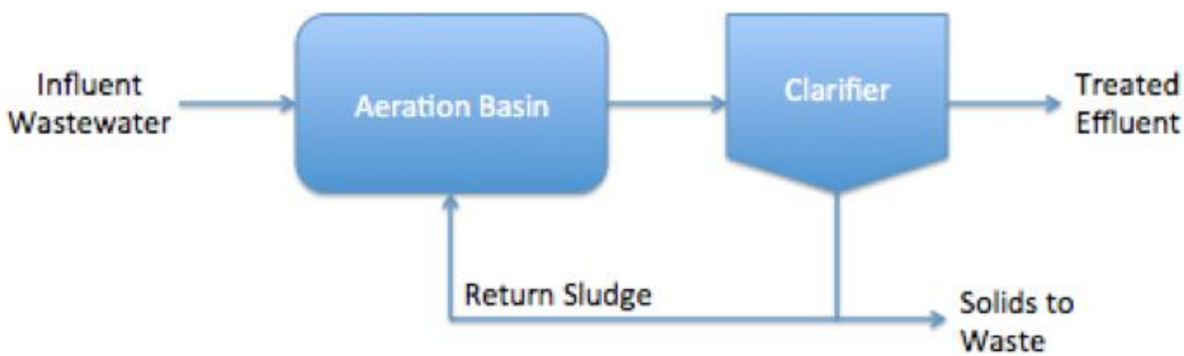

Figure 3: Activated Sludge Process Schematic

In the aeration stage, atmospheric oxygen is mixed into the wastewater typically through mechanical means such as surface aerators or air diffusers. After the wastewater enters the aeration basin, it is usually referred to as mixed liquor (ML). ML is a composition of the influent wastewater, microorganisms, and nutrients (Droste, 2014). The diffusers, and sometimes other mechanical equipment, are used as a means of providing mixing to the ML (Metcalf and Eddy, 2003). The ML is aerated for a specified amount of time, commonly referred to as the hydraulic detention time (HDT), before moving on to the clarifier. This settling stage is where the solids in the mixed liquor are allowed to settle and thicken at the bottom of the clarifier. The clarified wastewater in the top portion of the tank is able to then exit the clarifier to either be discharged or to undergo further treatment. The solids that accumulate at the bottom of the tank exit the clarifier to either be discarded or used as return sludge. These returned solids are the actual "activated sludge" that the treatment process is named after. They are an activated biomass that is used to stabilize waste and continue degradation of the organics in the wastewater in the aeration tank (Metcalf and Eddy, 2003).

The activated sludge process can be implemented for a number of different uses, depending on what type of wastewater is being treated. For all wastewater sources (domestic, industrial, and agricultural), the process is frequently used for the removal of organics. Activated sludge is also used to remove the nutrients nitrogen and phosphorus (Metcalf and Eddy, 2003). It is a very common system used for the treatment of agricultural wastewater that is used for irrigation return, because the removal of nitrogen and phosphorus are known to be detrimental to the growth of aquatic plants (Metcalf and Eddy, 2003).

2.3 Common Wastewater Properties

There are a number of common properties that are used to evaluate the quality of a wastewater. These properties are usually determined in a laboratory using standardized test methods. They are generally used to analyze the wastewater before and after treatment, and the findings can be used to determine the effectiveness of the treatment system in question. Some important properties of a wastewater that are commonly analyzed include pH, biochemical oxygen demand (BOD), chemical oxygen demand (COD), total suspended solids (TSS), lipids, total nitrogen (N) content, and total phosphorus (P) content.

2.3.1 pH

The pH of a water is a measure of the concentration of free hydrogen ions present in a water (Droste, 2014). It is significant to treatment because free hydrogen ions are both directly and indirectly involved in many reactions, and therefore directly affect the stability of a water. The pH of water is also significant because it determines whether the water being tested is acidic or basic. This is something that is monitored closely, because if the treated effluent has too acidic or too basic of a pH, the discharged water could alter the concentration of free hydrogen ions in natural waters (Metcalf and Eddy, 2003.

The pH of a water is also an important factor to monitor because of its influence on the ability of biological life to exist. In an activated sludge system, the wastewater being treated cannot be too acidic or too basic on the pH scale. There is a specific range of pH, 6 to 9, which the wastewater needs to stay within in order to suitably sustain biological life (Metcalf and Eddy, 2003). If the pH is too acidic or too basic, the microorganisms and nutrients in the water will die, and the activated sludge system will become inoperable.

2.3.2 Biochemical Oxygen Demand

One of the most widely used parameters to analyze the quality of a wastewater is biochemical oxygen demand, or BOD. BOD is defined as "the measure of the amount of oxygen required for the biological decomposition of organic matter under aerobic conditions at standardized temperature and time" (Droste, 2014). More simply, it is the measure of the oxygen required to completely break down the organic content in a wastewater. This is measured in a laboratory by measuring the amount of dissolved oxygen (DO) that is used by the microorganisms to break down the organic matter (Metcalf and Eddy, 2003). BOD is the most common means of expressing the amount of organic content in a wastewater.

The BOD of a wastewater is an extremely important factor for a number of reasons. One of the main reasons is that if not monitored and reduced through treatment, it could deplete the oxygen content of the natural water source that the treated wastewater is discharged to, and create undesirable effects on the environment (Metcalf and Eddy, 2003). It is also an important consideration in the design of wastewater treatment systems. Wastewater usually needs to be treated to an acceptable BOD effluent level in order to be discharged into the community wastewater system or to the environment.

2.3.3 Chemical Oxygen Demand

The chemical oxygen demand (COD) is another important analytical parameter used in wastewater treatment. It is similar to BOD, however it is defined as "the amount of oxygen required to stabilize organic matter determined by using a strong oxidant" (Droste, 2014). The COD test is "used to measure the oxygen equivalent of the organic material in the wastewater that can be oxidized chemically using dichromate in an acid solution" (Metcalf and Eddy, 2003). The oxidant chosen is most commonly dichromate, because it is both cost effective and known to be able to oxidize all types of organic matter (Droste, 2014). The COD is different from BOD in that BOD is solely a measure of the oxygen required to break down organic content in a wastewater, whereas COD involves the use of a chemical oxidant to break the organics down. COD is a more inclusive measure of the total organics in a wastewater. However, BOD is more relevant to industrial wastewater treatment systems, because oftentimes it is not desirable to require the use of a chemical oxidant. Most activated sludge wastewater treatment systems, and specifically the one examined in this study, rely solely on oxygen to break down the organic content in the water through microbial activity.

2.3.4 Suspended Solids

There are three types of solids that are typically evaluated in a wastewater sample: dissolved solids, colloidal solids, and suspended solids. Of these three, suspended solids are the main parameter of concern in assessing a wastewater. This is because suspended solids are directly correlated with the turbidity of a wastewater (Droste, 2014). Suspended solids can also be classified as whether they are settleable or not. Solids concentration is an important wastewater quality parameter, in that it determines aesthetic quality of the water. These solids are removed throughout the treatment process.

2.3.5 Lipids

Lipids are typically tested for when analyzing a wastewater because of the significant amount of organic content they contribute. In a typical wastewater, lipids make up about 8% to 12% of the total organic content (Metcalf and Eddy, 2003).

2.3.6 Nitrogen and Phosphorus

Total nitrogen and total phosphorus are two other important parameters that are tested for when examining a wastewater. Nitrogen and phosphorus are nutrients commonly found in wastewaters that are undesirable, and therefore are required to be removed through treatment. Specifically for wastewater that is to be treated and reused for irrigation, these nutrients are required to be removed because of their capability to stimulate the growth of aquatic plants (Metcalf and Eddy, 2003). Microorganisms in the activated sludge process are commonly used to remove nitrogen and phosphorus.

2.4 Design of an Activated Sludge System

There are a series of parameters that need to be calculated in order to evaluate and effectively design any wastewater treatment system. Activated sludge systems in particular have specific parameters that need to be calculated in order to ensure the requirements of the system are met through proper design.

2.4.1 Flowrate Calculations

One of the first and most important parameters that need to be determined for any type of wastewater treatment system is the influent flowrate to the system. Information on flowrates of the various wastewater streams at AFS was not complete, so actual flowrates in individual conduits needed to be calculated. The flowrate is determined by first calculating the velocity of the wastewater through an influent pipe using Manning's Equation, as shown in Equation 1.

$$v = \frac{R_h^{2/3} S^{1/2}}{n} \qquad \textit{Equation 1}$$

Where v is the velocity, R_h is the hydraulic radius, S is the energy slope, and n is the roughness coefficient. The flowrate of the water through the pipe can then be calculated using this velocity as shown in Equation 2.

$$Q = vA \qquad \textit{Equation 2}$$

Where Q is the flowrate, v is the velocity of the water, and A is the cross-sectional area of the pipe that the water is flowing through. In some instances, however, the water does not flow uninterrupted through a pipe. In the case of AFS, the manhole becomes a small retention tank due to headloss from the right angle redirection of flow during slaughterhouse operation. The diversion effluent pipe is smaller than the original effluent pipe, causing it to act as a circular weir. Flow through a circular weir is explained in Equation 3 (Gulliver, 2010).

$$Q = C_d[10.12(\frac{h}{d})^{1.975} - 2.66(\frac{h}{d})^{3.78}] * d^{\frac{5}{2}} \qquad \textit{Equation 3}$$

Where Q is the flowrate, d is the diameter of the circular orifice, h is the height over the weir, and C_d is the coefficient of discharge.

2.4.2 Activated Sludge Analysis Calculations

Once the influent flowrate of the system is determined, the hydraulic detention time for the aeration tank can be calculated using Equation 4.

$$\theta_d = \frac{V}{Q} \qquad \textit{Equation 4}$$

Where θ_d is the hydraulic detention time (HRT), V is the volume of the aeration tank, and Q is the flowrate. The HRT is the amount of time that the water will undergo treatment in the aeration tank before it exits to move on to the settling stage. Another parameter in assessing aeration treatment is the rate of oxygen utilization, which can be calculated as shown in Equation 5.

$$L = Q(S_0 - S_e) \qquad \textit{Equation 5}$$

Where L is the rate of oxygen utilization, Q is the flowrate through the system, S_0 is the influent substrate (organic) concentration, and S_e is the effluent substrate (organic) concentration. From the rate of oxygen utilization, the volumetric rate of air supply can be calculated using Equation 6.

$$Q_a = \frac{L}{E} \qquad \textit{Equation 6}$$

Where Q_a is the volumetric rate of air supply and E is the oxygen transfer efficiency. In an activated sludge system, the water travels from the aeration tank to the settling tank. An important parameter for the settling stage is the overflow rate of the clarifier. This is solved for using Equation 7.

$$v_{OR} = \frac{Q}{A} \qquad \textit{Equation 7}$$

Where v_{OR} is the overflow rate of the clarifier, Q is the flowrate from the aeration tank into the clarifier, and A is the surface area of the top of the clarifier. In order to calculate food to microorganism ratio, Equation 8 was used.

$$\frac{F}{M} = \frac{QS_0}{MLSS*V} \qquad \textit{Equation 8}$$

Where $\frac{F}{M}$ is food-to-microorganism ratio, Q is the flowrate through the system, S_0 is the influent substrate concentration, and V is the volume of the aeration tank.

These equations are commonly used to analyze and design all types of activated sludge systems. However, there are different variations of activated sludge systems, as previously mentioned in this section, that have different design requirements. The two configurations of systems relevant to this project include both a system that does not involve return sludge and one that does involve return sludge. Both of these configurations have their own respective calculations, as outlined in the following sections.

2.4.3 Activated Sludge System with No Sludge Recycle

Stage 1 of the system at AFS is an activated sludge system that does not involve the use of sludge recycle. One of the design parameters that needs to be calculated during this stage is the effluent substrate concentration or the concentration of organics in the wastewater as it exits stage 1 after treatment. This can be solved for using Equation 9.

$$S_e = \frac{K(1+k_e\theta_d)}{\theta_d(kY-k_e)-1} \qquad \textit{Equation 9}$$

Where S_e is the effluent substrate concentration, K is the half-velocity constant, k_e is the endogenous decay rate coefficient, θ_d is the HRT, k is the maximum rate constant, and Y is the yield. After solving for the effluent substrate concentration, the volatile suspended solids concentration in the aeration tank can be solved for using Equation 10.

$$X_V = \frac{Y(S_0-S_e)}{(1+\theta_d k_e)} \qquad \textit{Equation 10}$$

Where X_V is the volatile suspended solids concentration, Y is the yield, S_0 is the influent substrate concentration, S_e is the effluent substrate concentration, θ_d is the HRT and k_e is the endogenous decay rate coefficient.

2.4.4 Activated Sludge System with Sludge Recycle

Stage 2 of the system at AFS is an activated sludge system that does incorporate the use of sludge recycle. For this stage, the effluent substrate concentration and the suspended solids concentration in the aeration tank are calculated slightly differently than the previously stated equations. This is because for this stage, the use of return sludge that has to be taken into consideration. The time that the sludge stays in the system before it is wasted or recycled out is known as the mean cell residence time, which can be solved using Equation 11.

$$\theta_X = \frac{V*MLSS}{r_{ES}*Q*S_0} \qquad \textit{Equation 11}$$

Where θ_X is the mean cell residence time (SRT), V is the volume of the aeration tank, MLSS is the mixed liquor suspended solids concentration, r_{ES} is the excess sludge ratio, Q is the flowrate of the wastewater through the system, and S_0 is the influent substrate concentration. To solve for the effluent substrate concentration for this configuration, Equation 12 can be used.

$$S_e = \frac{K(1+\theta_X k_e)}{\theta_X(kY-k_e)-1} \qquad \textit{Equation 12}$$

Where S_e is the effluent substrate concentration, K is the half-velocity constant, θ_X is the SRT, k_e is the endogenous decay rate coefficient, k is the maximum rate constant, and Y is the yield. The volatile suspended solids concentration, taking the return sludge into consideration, can be solved for using Equation 13.

$$X_V = \frac{\theta_X}{\theta_d}\frac{Y(S_0-S_e)}{(1+\theta_X k_e)} \qquad \textit{Equation 13}$$

Where X_V is the volatile suspended solids in the aeration tank, θ_X is the SRT, θ_d is the HRT, Y is the yield, S_0 is the influent substrate concentration, S_e is the effluent substrate concentration, and k_e is the endogenous decay rate coefficient.

There are differing desired ranges for each of these parameters depending on many factors, including the degree of treatment required, the nature of the wastewater, and the desired use for the treated effluent. The wastewater at AFS comes from both agricultural and domestic sources, and the treated effluent is desired to be reused for irrigation purposes.

2.5 Milk Production Facilities

Advances in technology have allowed dairy production plants to monitor parameters such as temperature, pH, and flow rate constantly throughout every process. Legislation has also become stricter with quality control being of the utmost importance, so that public health is never compromised. To ensure this high standard in safety and quality around the world, the equipment must be cleaned daily and thoroughly so that milk residue and bacteria are eliminated completely before the next use. Clean-in-place systems are the most efficient at doing so with the least amount of manual labor (Bruhn, 2015).

The New Zealand Food Safety Authority requires of all dairy facilities in New Zealand four necessary elements for a proper cleaning: thermal, temporal, kinetic, and chemical. Ideal water temperatures should lie between 80-85 °C because water below 55°C begins redepositing milk residue and water which is too hot "...denatures protein, breaks down detergents and damages seals and rubber ware." (DairyNZ, 2015). For an ideal wash, 4-7 minutes should be spent for a hot water rinse along with a hot alkali wash. The kinetic energy element refers to flow rate and volume, which cause necessary turbulence. Chemically, a successful elimination of bacteria growth requires an alkaline and an acid base to work one right after the other respectively. The alkaline detergent removes fats and oils while the acid removes deposits of minerals in the system (DairyNZ, 2015).

2.6 Slaughterhouse Processes

Slaughterhouse waste naturally has much higher levels of organics, inorganics and bacteria because of the large percentage of inedible parts of the animal such as the bones, feathers, and blood.

This waste, if not handled properly, poses the highest potential risk to the environment and the animals and people within it (Franke-Whittle and Heribert, 2012).

Normally wastes may be reused in different industries, but a study done by Alexandria University's faculty of engineering examined a worst-case scenario in which all of the waste was mixed in a laboratory scale reactor. A total of 5 L of sludge and 40 L of slaughterhouse waste were combined. Specifically large amounts of blood, dung, fats and other unusable constituents proved to be problematic for a simple biological process such as aerobic treatment. The result of a poor wastewater treatment system for a meat processing facility is a high suspended solids content, dark color, and extremely unpleasant odors. The Egyptian University's study concluded that an anaerobic treatment system followed by an aerobic process delivers the ideal quality of effluent to discharge back into surface water (Seif and Moursey, 2001).

Slaughterhouses have become much more automated and are nearly standardized depending on the type of animals; because of this, the daily rate of animals slaughtered has increased. The high rate of slaughter requires rigorous cleaning processes to remove proteins, carbohydrates and fats that are nearly impossible to remove with hot water alone. Much like milk processing facilities, a proper mixture of appropriately timed hot water rinses along with alkaline detergents and acids are necessary for a sanitary slaughter environment. However, the convenience of a clean-in-place system at a slaughterhouse is not very feasible because of the large human factor that is necessary at nearly every stage of the slaughter. The Food and Agriculture Organization of the United Nations or FAO describes an effective cleaning operation as a combination of the following cleaning compounds used in timely and appropriate proportional manners depending on the facilities' specifics:

- Alkalis and alkaline salts
- Surface active agents
- Sequestering agents
- Acids
- Inhibitors (anti-corrosive agents)

- Fillers

Alkalis and alkaline salts are used to suspend proteins and convert fats into soap such as sodium hydroxide. To lower surface tension and allow a less strenuous cleaning process, anionic, nonionic, or cationic surface active agents are applied depending on the specific slaughter. Sequestering agents are dependent on the hardness of the water and so the focus of these agents is to prevent the development of insoluble calcium and magnesium deposits. Corrosion comes from the acids, which are necessary to remove natural mineral deposits, which occur with such a high level of organics. To counteract the corrosive effect of the acids, inhibitors such as silicates are sometimes placed in the alkaline detergents. Fillers are simply used to make the detergent become a fluid or to reverse the effect by turning a fluidized detergent into a powder (Skaarup, 1985).

2.7 Activated Sludge Issues Involving Milk and Slaughterhouse Processes

The first activated sludge for a dairy production plant dates back to at least 1935 in New Bremen, Ohio, which has records stating 98.9% BOD removal (Thayer, 1951). Since then, safety, health and environmental requirements of cleanliness at milking plants resulted in large chemical contents combining with the high organic content waste and entering the activated sludge treatment process. To achieve these standards, it is estimated that for every liter of milk produced in the dairy industry the result is 6-10 L of wastewater. The high organic load, such as fats, oils and grease, cause the effluent to degrade quicker and the dissolved oxygen (DO) level to be consumed at a higher rate. An issue that concerns spray irrigation systems is the contamination of the groundwater when improperly treated wastewater contains high levels of nitrogen that may be converted to nitrate (Porwal et al., 2014). Levels of pH are also one of the major concerns of clean-in-place systems because rinse waters produced have pH between 1.0 and 13.0 regularly (Singh et al., 2014). This requires an activated sludge that can handle variety in pH and adjust quickly or an external regulation of the pH before or during the first stage so that the treatment system is never shocked. Keeping pH at a neutral level is necessary so that microorganism growth rate does not slow down. This can be done by either adding an acid or caustic.

Slaughterhouse waste, like milking facilities, produce large amounts of BOD that can be treated properly by an anaerobic, aerobic or combination of both systems. Slaughterhouses specifically produce waste with high levels of nitrogen and phosphorus and so a process of oxidation followed by nitrification is required to convert ammonium into nitrate. Eutrophication of water sources receiving water treated via conventional activated sludge systems may not be prevented. To reduce this possibility; nitrate must be removed through denitrification, which is the conversion of nitrate into nitrogen gas. Irrigation of land provides the least expensive wastewater disposal option, and is relatively easy to perform for wastes which are low in pollution strength. For irrigation purposes, BOD_5 may not exceed 300 mg/L while low levels of nitrate and phosphorus are acceptable as fertilizers (Verheinjen et al, 1996). According to Watson et al. (2007), ammonia is oxidized into nitrites by Nitrosomonas and Nitrosococcus bacteria, which are consecutively oxidized into nitrates by Nitrobacter bacteria. As the Baltimore Ecosystem Study explains, nitrates are essential to plant growth as they help tissue development and seed production which is why fertilizers have such high concentrations of nitrate.

Alone, a milk facility or a slaughterhouse proves to be a small challenge for activated sludge systems, but when both are combined with domestic waste and dining hall waste, the system has much more of a variety to treat. At the American Farm School, this is exactly the case and so the system was constantly running at a much lower efficiency due to high flowrates and high pH along with a mixture of diluted cleaning chemicals and high levels of organics.

One issue that stems from the school owning their own water supply of wells is the lack of water conservation because there is no water restriction or regulations that limit water use (Nikolaidis, Personal Communication, 2015).

3. Methodology

The goal of this project was to analyze the current wastewater system at the American Farm School in Thessaloniki, Greece. Recommendations were made in order to accommodate the changes being made to the wastewater influent to improve effluent to Lagoon A. Wastewater was tested for pH, BOD_5, COD, TSS, fats and oils, total nitrogen, and total phosphorus. The dimensions from the school's AutoCAD files for the wastewater treatment plant were used to calculate flow rates, HRT, F/M ratio, substrate removal, and SRT. This chapter illustrates the methodology used in the field and laboratory to make recommendations.

3.1 Sampling and Field Overview

Samples were taken twice from the manhole where the milking facility wastewater meets the wastewater from the slaughterhouse, gymnasium, elementary school, and maintenance department. These locations are shown in Figure 4. They were taken during a day with normal activity and again during slaughterhouse operation.

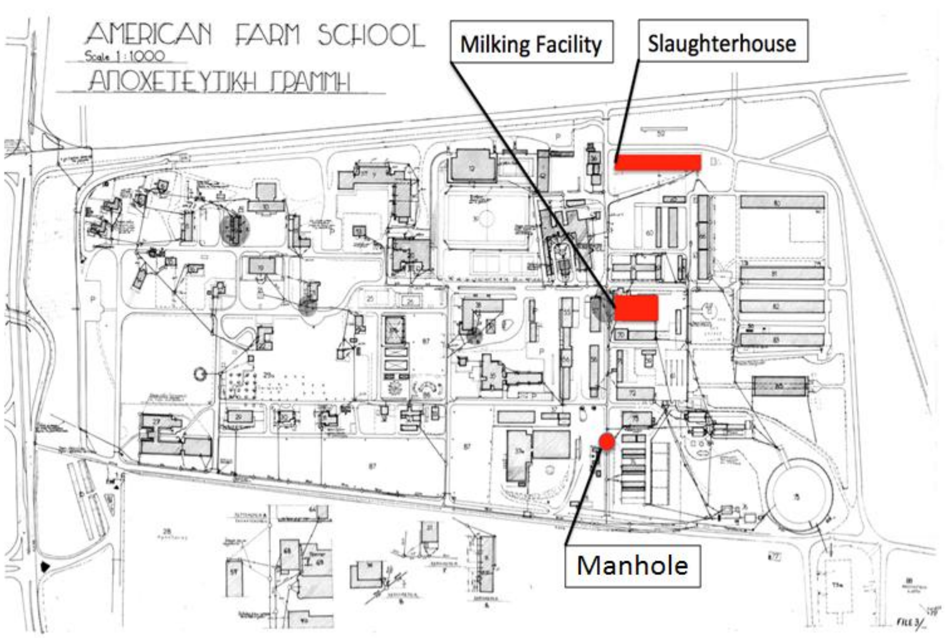

Figure 4: Plan View of the AFS Campus

Sampling was carried out as follows:

1. Seven 0.5 L containers, two 1.0 L containers, and one 1.5 L container were cleaned using dish soap, rinsed thoroughly with tap water and air dried containers overnight.
2. The containers were labeled according to Appendix A.
3. One 0.5 L container was filled from the manhole every hour for seven hours.
4. The sealed samples were stored in a refrigerator at 4°C.
5. The samples were delivered to AGROLAB for analysis.
6. Combined samples were collected in 1.5 L containers.
7. Two 1.0 L containers were filled with combined samples for AGROLAB testing.

Figure 5: Sampling of Milking Facility Effluent

Figure 5 shows the team in the process of acquiring wastewater samples from the manhole during a time when the milking facility was running. Samples were brought to AGROLAB, located approximately 30 km (≈25 minutes driving) from AFS, within 24 hours of the start of collection for analysis. Throughout the sampling process, safety precautions outlined in the Safety Protocol located in Appendix B were adhered to.

3.1.1 The Manhole

Seven samples were taken from the manhole hourly. For each day of sampling, the flowrate was calculated during each of the samplings by measuring pipe diameter of the manhole, the height of water as it exits the pipe, and the slope of the pipe using the topography. A theodolite was used to determine the slope. These values were used to calculate the difference in cross-sectional area of the pipe compared to the cross-sectional area of the water. Thus, actual flowrates were determined.

Flowrate Calculation:

1. Measured the inner diameter of the manhole influent pipe.
2. Determined the slope of the pipe.
3. At each sampling interval, measured the depth of the water before it exited the pipe
    a. Recorded five depth measurements for every interval.
    b. Calculated average depth for each sampling interval.
4. Used pipe and water depth measurements to calculate flowrate.
5. Used data obtained in coordination with the calculations outlined in section 2.3.1 to calculate flowrates.

3.2 Experimental Overview

Analysis was performed by professionals at AGROLAB, a local laboratory. The tests that were able to be performed immediately were overseen by the team for recording purposes. Upon bringing samples to the laboratory, all seven samples were mixed and then divided into two 1.5 L containers. This was done in accordance with AGROLAB procedures in order to have a separate container for the fats and lipids test and for all other parameters. AGROLAB used standard methods according to the Official Methods of Analysis of AOAC INTERNATIONAL (AOAC, 2012) for all testing, including pH, BOD_5, COD, TSS, total nitrogen, total phosphorus, and fats and oils.

4. Results and Discussion

4.1 Introduction

The following results and analysis provide information leading to recommended improvements to the AFS wastewater treatment system. The team gathered data on the typical flowrates that the WWTS would expect to see. Using this information, calculations were made to assess the capacity of the current system to adequately treat the wastewater for reuse as irrigation water. Samples were brought to AGROLAB to be tested for constituents, which allowed for the calculation of HRT, F/M ratio, substrate removal, and SRT. These parameters are necessary in designing an effective WWTS and were used to compare the efficiency of the system as it has been working to its capabilities through design modifications. This section presents suggestions for improvements to the design of the system, considering the lowest maintenance and most cost-effective solution.

4.2 Original Design

The results from the AGROLAB tests in Figure 7 show the respective quantities of influent constituents to the treatment system during normal activity. Due to different conventions in the US and Greece, commas and periods were used interchangeably to signify decimal places. All samples were taken from the manhole. The wastewater BOB concentration of 450 mg/L is higher than the allowable 300 mg/L for spray irrigation as well as above AFS's standard of 100 mg/L (Verheinjen et al., 1996). The pH of the sample was recorded at 10.1. This pH is high compared to that recommended, which is in the range of 6 to 9 typical for a standard wastewater treatment. However, alkaliphilic microorganisms can thrive at pH values between 10 and 12, while growing at slower rates at pH values near neutral (Horikoshi, 1999). Figure 6 compares the growth of alkaliphilic microbes and neutrophilic microbes at pH values between 5 and 12.

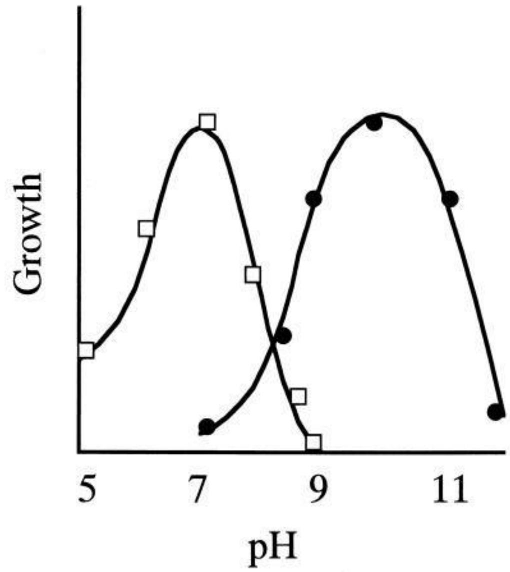

Figure 6: The pH dependence of the growth of neutrophilic and alkaliphilic microbes. Squares represent neutrophilic while circles represent alkaliphilic microorganisms (Verheinjen et al, 1996).

Metcalf and Eddy (2003) suggest a pH level of 6.5 to 8.4 for treated wastewater used for irrigation. Although influent pH levels are above this range, records from past years when the activated sludge WWTS was active indicate that pH reached appropriate levels in Lagoon A, averaging 7.2, 7.9, and 8.0 for Stage 2 aeration, Lagoon A, and Lagoon B respectively during 2011. The pH of Lagoons A and B signify that even with the addition of high pH wastewater influent to the treatment system, its volume is not significant enough to drastically alter the pH of the irrigated water.

Additional parameters are outlined in Figure 7. Not all parameters were vital to the project's design calculations. The parameters were used to acquire a deeper understanding of the quality of the wastewater entering the WWTS and may be used later by AFS for future treatment considerations. Specifically, since wastewater from AFS is used exclusively for campus crops and is not discharged to any external surface water bodies, nitrogen and phosphorus are not extensively treated. Typical levels for strong concentrations in untreated domestic wastewater are 15 mg/L total phosphorus and 85 mg/L total nitrogen (Metcalf & Eddy, 1991). Tested values are within the typical range for domestic wastewater.

Αποτελέσματα Αναλύσεων / Results

Κωδικός δείγματος	*Sample Code*	**2015-43860**
Περίοδος Ανάλυσης	*Period of Analysis*	**06/11/2015 - 17/11/2015**
Χαρακτηρισμός Πελάτη	*Client's Declaration*	**ΑΠΟΒΛΗΤΟ**
Κατάσταση δείγματος κατά την παραλαβή	*Sample condition upon receipt*	**Κανονική / Acceptable**

Παράμετρος Parameter	**Μονάδες** Units	**Τιμή** Result	**Όριο αναφοράς** Reporting limit	**Αβεβαιότητα μεθ. στο νομοθ. όριο** Uncertainty at the accept. level	**Ανώτ. νομοθ. όριο** Max. accept. lev.**	**Μέθοδος** Method
Νιτρικά (NO3)	mg/L	**83,9**	2,0			O.B. 01.018 4500 NO3-B St.Met.
Νιτρώδη (NO2)	mg/L	**0,03**	0,03			O.B. 01.011 4500NO2-B St.Met.
Ολικός φώσφορος / Total P	mg/L	**11.0**	2,5			in house aqua regia ICP
Αμμωνιακά (NH4)	mg/L	**15,5**	1.8			Mod based on 4500 NH3-B,C St.Met.
Χλωριούχα (Cl)	mg/L	**444**	10			O.B. 01.007 4500 Cl Mod. St.Met.*
pH	μονάδες pH 22 oC	**10,1**	1-10			O.B.01.005 4500-H,B St.Met.*
Λίπη & Έλαια (oil&crease)	mg/L	**24**	1			Mod. based on St.Met. 5520 B
Ολικό Άζωτο κατά Kjeldhal (TKN)	mgN/L	**75.5**	1,4			Mod. based on St.Met. 4500-NorgB
BOD	mgO2/L	**450**	20			Mod. based on St. Met. 5210 D
COD	mgO2/L	**10304**	20			HACH-LANGE LCK 314,414,514*
Ολικά Αιωρούμενα Στερεά (TSS)	mg/L	**605**	10			mod. based on 2540D St.Meth.

St. Met.: APHA, Standard Methods 22nd Ed, 2012.
N.D.: Δεν ποσοτικοποιήθηκε στο όριο αναφοράς της μεθόδου/Not determined at the reporting limit of the method.
* **Διαπιστευμένη δοκιμή** κατά ISO 17025, Αρ. 44 ΕΣΥΔ. / Accredited method according to ISO 17025.
** **Τα ανώτατα νομοθετικά όρια** περιγράφονται και επεξηγούνται ως προς την ορθή τους χρήση στο ΦΕΚ Β' 630/26.04.2007 και την οδηγία 98/83/ΕΚ 3-11-1998/Max. acceptable levels described and explained as to their proper use in Greek and European legislation (98/83/EU 3-11-1998).
Η Agrolab δεν αποδέχεται καμία υπευθυνότητα σε σχέση με τα παραπάνω αναγραφόμενα ανώτατα επιτρεπτά όρια τα οποία δίδονται μόνο για λόγους πληροφόρησης. / AGROLAB does not accept any responsibility for the aforementioned max. acceptable levels, which are given only for information reasons.
\# Ο χρόνος τήρησης του αντιδείγματος ορίζεται στον **1 μήνα** από την ημερομηνία έκδοσης του παρόντος πιστοποιητικού (στις κατάλληλες συνθήκες διατήρησης), εκτός και αν ο πελάτης εγγράφως έχει ορίσει διαφορετικά. Εξαιρούνται ευαλλοίωτα δείγματα, τα οποία δεν μπορούν να συντηρηθούν για το προαναφερθέν χρονικό διάστημα.

(a)

Αποτελέσματα Αναλύσεων / Results

Κωδικός δείγματος	*Sample Code*	**2015-45963**
Περίοδος Ανάλυσης	*Period of Analysis*	**20/11/2015 - 02/12/2015**
Χαρακτηρισμός Πελάτη	*Client's Declaration*	
Κατάσταση δείγματος κατά την παραλαβή	*Sample condition upon receipt*	**Κανονική / Acceptable**

Παράμετρος Parameter	**Μονάδες** Units	**Τιμή** Result	**Όριο αναφοράς** Reporting limit	**Αβεβαιότητα μεθ. στο νομοθ. όριο** Uncertainty at the accept. level	**Ανώτ. νομοθ. όριο** Max. accept. lev.**	**Μέθοδος** Method
Νιτρικά (NO3)	mg/L	**13,3**	2,0			O.B. 01.018 4500 NO3-B St.Met.
Νιτρώδη (NO2)	mg/L	**N.D.**	0,03			O.B. 01.011 4500NO2-B St.Met.
Ολικός φώσφορος / Total P	mg/L	**6.5**	2,5			in house aqua regia ICP
Αμμωνιακά (NH4)	mg/L	**15,9**	1.8			Mod based on 4500 NH3-B,C St.Met.
Χλωριούχα (Cl)	mg/L	**426**	10			O.B. 01.007 4500 Cl Mod. St.Met.*
pH	μονάδες pH 22 oC	**6,9**	1-10			O.B.01.005 4500-H,B St.Met.*
Λίπη & Έλαια (oil&crease)	mg/L	**22,1**	1			Mod. based on St.Met. 5520 B
Ολικό Άζωτο κατά Kjeldhal (TKN)	mgN/L	**34,2**	1,4			Mod. based on St.Met. 4500-NorgB
BOD	mgO2/L	**524**	20			Mod. based on St. Met. 5210 D
COD	mgO2/L	**803**	20			HACH-LANGE LCK 314,514*
Ολικά Αιωρούμενα Στερεά (TSS)	mg/L	**282**	10			mod. based on 2540D St.Meth.

St. Met.: APHA, Standard Methods 22nd Ed, 2012.
N.D.: Δεν ποσοτικοποιήθηκε στο όριο αναφοράς της μεθόδου/Not determined at the reporting limit of the method.
* **Διαπιστευμένη δοκιμή** κατά ISO 17025, Αρ. 44 ΕΣΥΔ. / Accredited method according to ISO 17025.
** **Τα ανώτατα νομοθετικά όρια** περιγράφονται και επεξηγούνται ως προς την ορθή τους χρήση στο ΦΕΚ Β' 630/26.04.2007 και την οδηγία 98/83/ΕΚ 3-11-1998/Max. acceptable levels described and explained as to their proper use in Greek and European legislation (98/83/EU 3-11-1998).
Η Agrolab δεν αποδέχεται καμία υπευθυνότητα σε σχέση με τα παραπάνω αναγραφόμενα ανώτατα επιτρεπτά όρια τα οποία δίδονται μόνο για λόγους πληροφόρησης. / AGROLAB does not accept any responsibility for the aforementioned max. acceptable levels, which are given only for information reasons.
\# Ο χρόνος τήρησης του αντιδείγματος ορίζεται στον **1 μήνα** από την ημερομηνία έκδοσης του παρόντος πιστοποιητικού (στις κατάλληλες συνθήκες διατήρησης), εκτός και αν ο πελάτης εγγράφως έχει ορίσει διαφορετικά. Εξαιρούνται ευαλλοίωτα δείγματα, τα οποία δεν μπορούν να συντηρηθούν για το προαναφερθέν χρονικό διάστημα.

(b)

Figure 7: (a) Sample results on a day where dairy processing, elementary school, and gymnasium wastewater are combined in the manhole and (b) on a day where dairy processing, elementary school, gymnasium and slaughterhouse wastewater are combined in the manhole.

The tables below represent measurements taken from the manhole over a two day period during normal operation and during the operation of the slaughterhouse. Table 1 provides the heights of the water level through the influent pipe during the seven most active hours of the day, when the dairy facility is expelling wastewater, the elementary school is in session, and the gymnasium is active. The cafeteria wastewater has not been rerouted to go directly to EYATH yet, so the heights also include any wastewater that was produced during the hours of sampling. Table 2 represents all of the above sources of wastewater in addition to that of the slaughterhouse, which was active during the hours of sampling.

Table 1: Normal operation with dairy facility operating

Sample #	unit	6-Nov	6-Nov	5-Nov	5-Nov	5-Nov	5-Nov	5-Nov
	Time:	7:27	8:21	9:32	10:55	12:35	14:01	15:12
h1	cm	1.7	2.1	3.2	7.5	2.6	2.8	1.4
h2	cm	1.2	4.9	4.0	5.2	1.5	2.6	1.9
h3	cm	1.3	5.5	--	7.2	1.4	2.9	1.6
h4	cm	0.8	5.7	--	3.5	1.4	3	1.7
h5	cm	1	5.5	--	7.3	1.7	2.8	1.4

Table 2: Normal operation with dairy facility and slaughterhouse operating

Sample #	unit	16-Dec	16-Dec	15-Dec	15-Dec	15-Dec	15-Dec	15-Dec	15-Dec	15-Dec	15-Dec
	Time:	7:33	8:40	9:05	10:20	11:22	12:14	13:20	14:20	15:23	16:34
h1	cm	9.8	8.9	8.9	11.7	9.5	12.7	7.3	10.2	8.9	9.2
h2	cm	10.2	8.9	9.5	11.1	9.5	12.1	7	10.8	8.3	8.3
h3	cm	9.8	9.8	8.9	12.1	9.2	11.4	7	11.7	8.3	8.9
h4	cm	9.5	9.2	9.2	10.8	8.9	12.1	7.6	10.8	8.3	9.5
h5	cm	8.6	9.5	8.6	11.4	9.5	12.1	6.7	9.8	7.9	10.2

Note: Three data points were not taken on 5-Nov: the following week, wastewater flow in the manhole was diverted directly to Lagoon A and height measurement would have been skewed.

All but two sets of data were taken on 5-Nov in Table 1. The two 6-Nov data sets were taken after observing that there was milk residue during the earliest sampling period and that sampling needed to begin earlier. Since daily flows are consistent throughout the week, this day's height measurements are still a representative sample. Flowrate measurements for days with the slaughterhouse active took place on 15-Dec and 16-Dec. These measurements were taken over a two day period for the same reasons as stated above.

The heights measured from the effluent pipe for the daily wastewater flow varied substantially. Upon calculating the flowrates at each hour for typical dairy flow and typical slaughterhouse flow, it was observed that the peak flowrate for dairy wastewater was 1.8 times higher than that of the slaughterhouse wastewater. This data is inaccurate considering that the slaughterhouse flowrate measurement was a combination of dairy and slaughterhouse wastewater. Because there was a difference in calculation, as described previously, there is reason to believe that one or both methods of flowrate calculations were inaccurate. Flowrate calculations for the original design of the system are outlined in Appendix C.

Three separate flowrates were calculated in order to visualize the extremes that the WWTS could expect along with the typical flow that occurs. The low flowrate was determined by taking a weighted average of the flowrates per hour for the seven hours that the system is most active, during the work/school day, and setting the remaining 17 hours to zero flow. It is unlikely that the system would ever experience a zero flow at any one hour, but flowrates do drop significantly overnight. The maximum flowrate was also calculated by taking a weighted average of the flows during slaughterhouse operation. However, instead of zero flow during non-operational hours, 2.78 m^3/hr was used for the non-operational hours. This flowrate was the lowest of the values obtained from the measurements taken during a typical day with only the dairy facility online. It was assumed that the non-work/school hours would have a flow likened to this. The average flow was calculated in the same manner, as the maximum flow, using 2.78 m^3/hr for the non-operational period but with its own values. Appendix C details this data in more depth.

For the typical flow, when the slaughterhouse is offline, the heights of wastewater flow through the influent pipe were averaged for each day and were used to calculate the velocity of the wastewater using Equation 1. Equation 1 was then incorporated into Equation 2 to calculate the minimum and average flowrates, which were 195 and 242 m^3/d, respectively. The maximum flowrate was calculated using Equation 3, which was 352 m^3/d. Data can be seen in Table 3 and 4 below for Stages 1 and 2.

Sampling methods were changed during slaughterhouse operation because the effluent from the manhole was no longer flowing straight through. The manhole acted as a small retention tank due to headloss from the right angle redirection of flow. The top of the diversion effluent pipe is level with the top of the influent pipe but is 51 millimeters smaller in diameter, causing the effluent pipe to act as a circular weir. Figure 8 shows normal operation and dairy facility operation as well as the combination of normal operation, dairy facility and slaughterhouse.

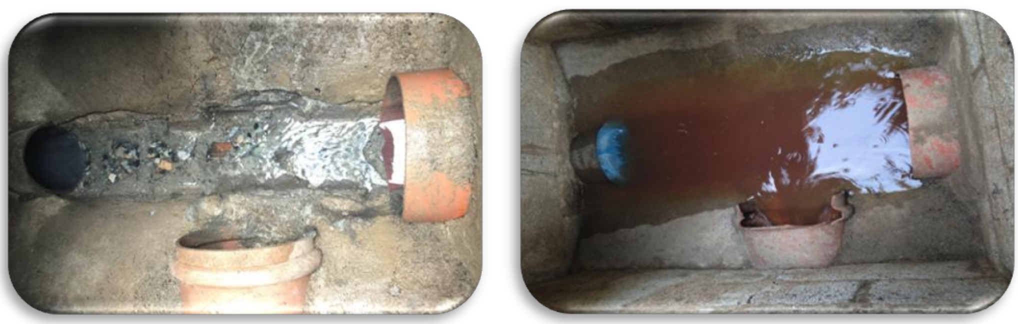

Figure 8: (left) The manhole with normal operation with dairy facility operating and (right) normal operation with dairy facility and slaughterhouse operating.

Table 3: Results of Calculations for Stage 1 (Aeration and Settling) for Existing System with All Inputs

Parameter	Unit	Minimum	Average	Maximum
Q= Volumetric Flowrate	m^3/d	195	242	352
θ_d= HRT	hours	14.0	11.3	7.8
	days	0.58	0.47	0.32
S_e= Effluent Substrate Concentration	mg/L	86.3	160	524
X_v= VSS Concentration	mg/L	211	169	0
L = Rate of Oxygen Utilization	kg/d	71	70	168
Q_a=Volumetric Rate of Air Supply	m^3/d	3504	3464	8281
F/M Ratio	kg/kg/d	3.65	5.66	--

Table 4: Results of Calculations for Stage 2 (Aeration and Settling) for Existing System with All Inputs

Parameter	Unit	Minimum	Average	Maximum
Q_{avg}= Volumetric Flowrate	m^3/d	195	242	352
θ_d= HRT	hours	30.8	24.8	17.1
	days	1.3	1.0	0.71
θ_x= SRT	days	186	81	17.0
S_e= Effluent Substrate Concentration	mg/L	1.34	1.48	2.5
X_v= VSS Concentration	mg/L	607	1273	3698
L = Rate of Oxygen Utilization	kg/d	16.6	38.5	184
Q_a= Volumetric Rate of Air Supply	m^3/d	819	1902	9073
OR = Surface Loading Rate	$m^3{*}m^{-2}{*}d^{-1}$	8.2	82.6	253
F/M Ratio	kg/kg/d	0.11	0.12	0.20

Using the calculated flowrate and the volume of the aeration tanks, Equation 4 was used to calculate the HRT for both Stages 1 and 2. The HRT was incorporated into Equation 9 in order to find the effluent substrate concentration. Since Stage 1 has no sludge recycle, SRT was equal to HRT. As can be seen from the S_e value during maximum flow in Stage 1, there was no decomposition of organics. This is due to the low HRT, which was caused by a high flowrate. With an HRT of 7.8 hours, Stage 1 would theoretically not treat any organics. Nonetheless, Stage 2 makes up for the lack of efficiency of Stage 1, bringing S_e values below 3 mg/L for all three final effluent flowrates.

The concentrations of VSS were calculated for both stages. Stage 1 did not incorporate sludge recycle and so Equation 10 was used. Stage 2 used Equation 13, which did incorporate sludge recycle. Because there is no decomposition of organics during maximum flow in Stage 1, BOD does not change and no additional VSS is produced. A notable difference between Stages 1 and 2 is that, in Stage 1, as the flow rate increases, Se concentrations increase while VSS concentrations decrease. In comparison, the Stage 2 flowrates have a positive relationship with both Se and VSS concentrations. The differentiating factor is the presence of sludge recycle in Stage 2, which recycles VSS through the aeration tank. Typical values, according to Metcalf & Eddy (1991), range between 2500-6500 mg/L for an extended aeration system for a small community. Calculated values can be seen in Table 4 above.

The rate of oxygen utilization was calculated using Equation 5, which took the product of the flowrate and the change in substrate concentration. This value was then incorporated into Equation 6 to calculate the volumetric rate of air supply. This equation took into account the efficiency of the system while running at a high pH.

The surface loading rate (OR) was only calculated for Stage 2 as this wastewater is the final effluent into Lagoon A and it is important to know whether solids are passed through the clarifier without settling. OR was determined to be 8.2, 82.6, and 253 $m^3 \bullet m^{-2} \bullet d^{-1}$ for minimum, average and maximum flows, respectively. The typical range for secondary settling is 8-16 $m^3 \bullet m^{-2} \bullet d^{-1}$ with a peak of 24-32 $m^3 \bullet m^{-2} \bullet d^{-1}$, meaning that the AFS Stage 2 OR is over the maximum standard peak and well over the suggested range for daily use. This peak rate should not exceed a 2 hour sustainment (Metcalf & Eddy, 2003).

The F/M ratio was determined using Equation 8. For each stage, the equation incorporated the respective tank volumes. For Stage 2, F/M was calculated to be 0.11, 0.12 and 0.20 kg substrate/kg biomass•day for the minimum, average and maximum flowrates. Each of these values show that BOD levels are acceptable, with a typical F/M range being 0.04-0.6 for an extended aeration PFR. Also, for a high rate completely mixed activated sludge system (CMAS) this ratio can reach 1.0 (Metcalf & Eddy, 2003).

4.3 Recommended Design Improvements

Based on the previous calculations and the high fluctuation in flowrates that the system expected to see, the team recommended that AFS consider using Stage 1 as an equalization basin. This modification would enable the flowrate to be regulated before entering into the Stage 2 aeration basin.

The team considered Stage 1 as an equalization basin and recalculated the previously mentioned design parameters accordingly. It was determined that Stage 1 would need a flowmeter, pH adjust system, float switches and a series of pumps to regulate the flowrate and adjust the solution pH entering into Stage 2.

With a maximum calculated flowrate of approximately 50 m^3/hour (13,210 gph or 220 gpm), the team recommends the Electromagnetic ENVIROMAG 2000 Flowmeter. Designed specifically for water and wastewater, Krohne manufactures the ENVIROMAG 2000 starting at $4,150.00 and $4,600.00 for a 10 and 12" pipe respectively (in February 2016) (Krohne, 2016). This flowmeter, Figure 9 below and the technical datasheet in Appendix D, is ideal as it is a continuous self-diagnosing system or self-correcting system according to whether or not the pipe is full or not to ensure accuracy, is unaffected by solids, fibers and slurries, is maintenance free, and can be placed at either the school's 10" or 12" pipe depending on where would be the most convenient to AFS.

Figure 9: ENVIROMAG 2000 magmeter from Krohne.

(www.instrumart.com)

The maximum required influent flowrate for pumping from equalization to Stage 1 is 14.69 m^3/h (66 gpm). We recommend 2 pumps at 10 m^3/hour with 0.5 hp per pump and with a maximum head of 3 meters. Zoeller does produce a pump using 115 volts with 89 gpm at a head height of 10 feet for approximately $400.00 each. Figure 10 describes the pump's capacity depending on the head height (Zoeller, 2016).

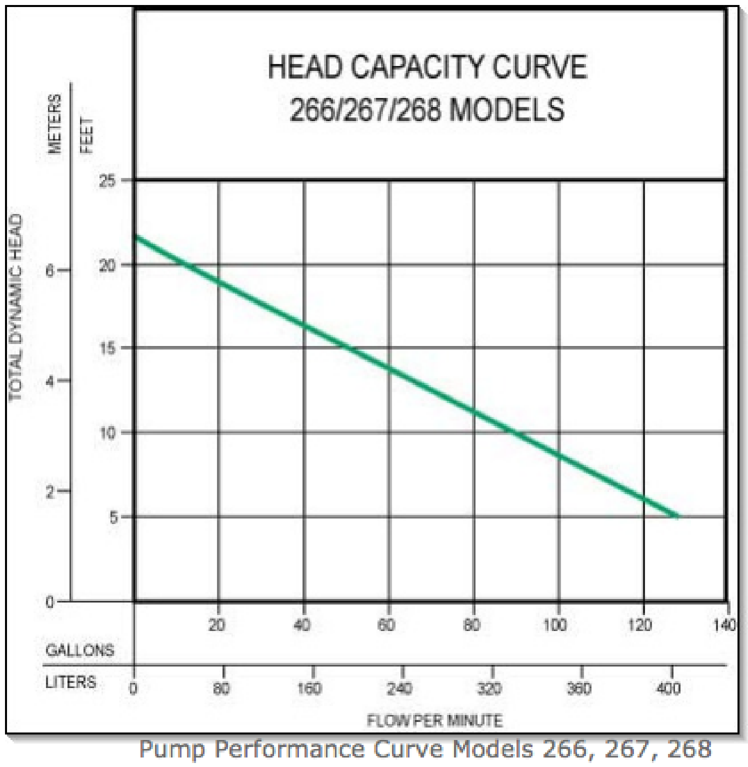

Figure 10: Head Capacity Curve for M267-25

The following, Figure 11, depicts how our planned flowrate compares to the current influent flowrate. Figure 12 describes how the constant accumulated exiting volume compares to the current daily accumulation, and how the current does surpass the straight constant flowrate. This suggests that there would be an excess of at most 83.3 m^3 of wastewater but because the volume of the tank is 160 m^3, this is acceptable and will eventually be equalized by the end of the 24-hour cycle.

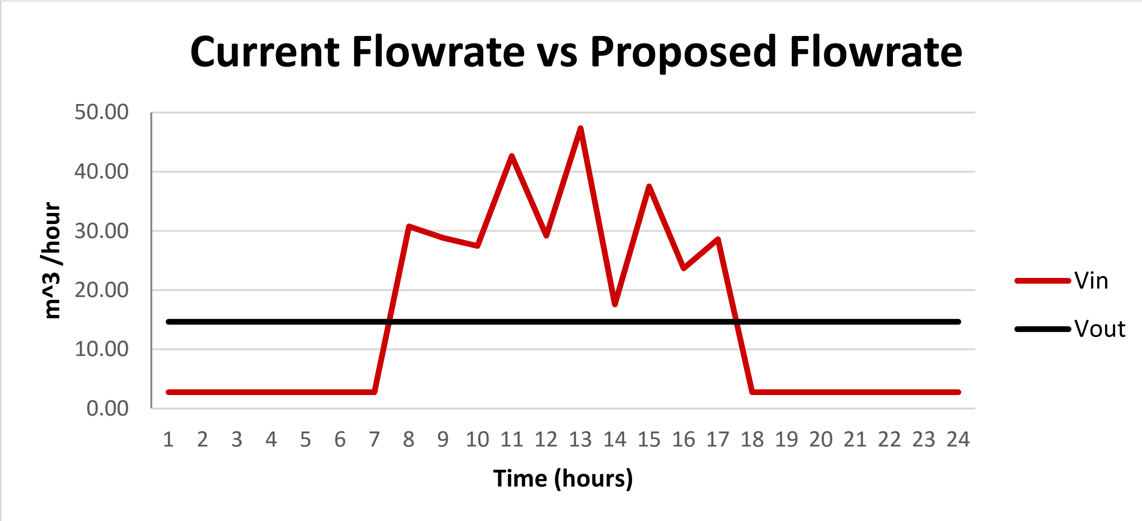

Figure 11: Current influent flowrate and its variability as the slaughterhouse is active compared to steady proposed effluent flowrate with pumps

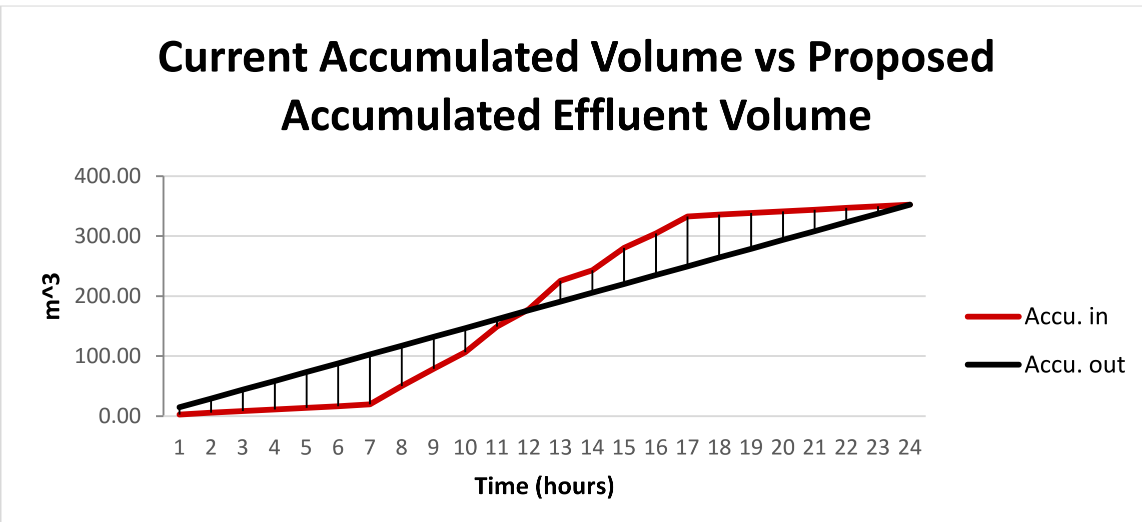

Figure 12: Current accumulated volume, as the slaughterhouse is active compared to steady accumulated volume with pumps

A series of four ENM- 10 Level Regulators or float switches are also recommended to ensure that the pumps are turned on and off at precise water level locations. One is positioned at a stop level, one for each pump and their respective start levels, and a fourth to act an as alarm system for a level that is exceedingly high. Welded and screwed together, this plastic level regulator is ideal as it is very low maintenance. The approximate cost is $300.00 per switch, because the price is also dependent on type of liquid and length of cable required (Technical datasheet in Appendix E). Figure 13 depicts the basic principle behind the ENM- 10 Level Regulator (FLYGT, 2016).

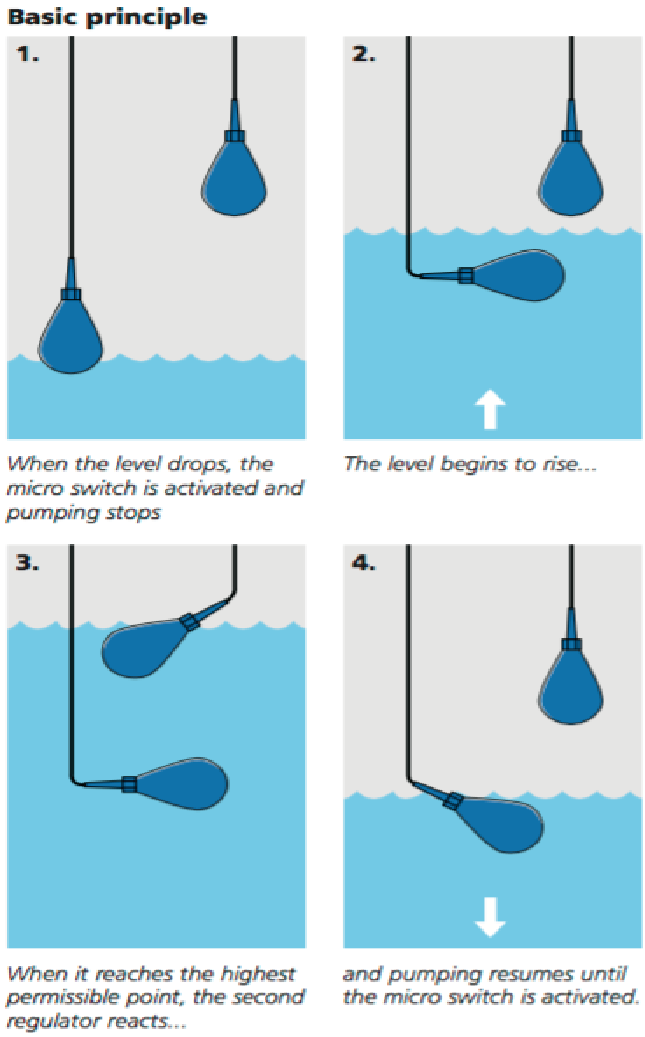

Figure 13: ENM-10 Level Regulator basic principle (FLYGT, 2016)

Regardless of whether or not AFS chooses to implement the pumps, a flowmeter would be the next best step to obtain concrete data on both Stage 1 and Stage 2 influent flowrates. Table 5 shows the newly calculated parameters with consideration to the equalization basin design. In order to operate at both low-flow scenarios, in which only the typical daily operations are contributing to the wastewater flow, as well as the higher-flow scenarios, like when the slaughterhouse is in operation, there should be more than one pump, set on a series of float switches which control when the pumps operate. The following describes the flowrates that the pump would regulate into the new Stage 1; details of the calculations are shown in Appendix F:

Constant flowrate for minimum: 8.13 m^3/h

Constant flowrate for average: 10.10 m^3/h

Constant flowrate for maximum: 14.69 m^3/h

Table 5: Results of Calculations for Stage 2 activated sludge process, Using Stage 1 as an Equalization Basin

Parameter	Unit	Minimum	Average	Maximum
Q=Volumetric Flowrate	m^3/d	195	242	352
θ_d= HRT	hours	30.8	24.8	17.0
	days	1.3	1.0	0.7
θ_x= SRT	days	35.7	28.7	19.8
S_e= Effluent Substrate Concentration	mg/L	1.81	1.96	2.30
X_v= VSS Concentration	mg/L	2377.5	2740.6	3413.2
L = Rate of Oxygen Utilization	kg/d	87.5	108.6	157.8
Q_a= Volumetric Rate of Air Supply	m^3/d	4318.2	5360.8	7789.1
OR = Surface Loading Rate	$m^3*m^{-2}*d^{-1}$	24.1	29.9	43.5
F/M Ratio	kg/kg/d	0.15	0.16	0.19

As shown in Table 5, the flowrates would be exiting the equalization basin at constant rates of 195, 242, and 352 m^3/d for the minimum, average, and maximum flows, respectively. However, the system operates at the most ideal conditions at the lowest flowrate, as well as provides the highest SRT. At the calculated minimum flowrate of 195 m^3/d, the system has an HRT of 1.3 days and an SRT of 35.7 days. In an extended aeration system, a higher SRT is necessary because of the infrequency of sludge wasting, and a large aeration tank is practical because it allows for more MLSS to collect. The low flowrate also provides the most extensive treatment to the wastewater in terms of BOD, decreasing it from 450 mg/L to a mere 1.81 mg/L.

The rate of oxygen utilization at this minimum flow condition was calculated to be 87.5 kg/d using Equation 5. The volumetric rate of air supply needed was calculated to be 4318.2 m^3/d, using Equation 6.

The calculated minimum flow operating scenario also enables a desirable F/M ratio. The F/M ratio, calculated using Equation 8, was determined to be 0.15 kg/kg/d. This falls on the lower end of the range specified for typical extended aeration, which falls between 0.04 and 0.6. Although an equalization basin would require Stage 2 of the current system to treat all of the BOD present without the help of Stage 1, this would not be an issue. This is because the increase in BOD needed to be treated would be balanced by the increase in the VSS concentration, which was calculated to be 2377.5 mg/L. This simultaneous increase between both the BOD and the VSS concentration enable the F/M ratio to stay within the desirable range.

For the low flow condition, the clarifier surface loading rate was calculated to be 24.1 $m^3 \bullet m^{-2} \bullet d^{-1}$. This OR is still outside the typical range specified by Metcalf and Eddy (2003) of 8-16 $m^3 \bullet m^{-2} \bullet d^{-1}$, but it does not fall outside of the peak range specified, which falls between 24-32 $m^3 \bullet m^{-2} \bullet d^{-1}$. As previously mentioned, the peak range is not meant to be sustained for more than 2 hours. If the system were to operate at the constant low flow of 8.13 m^3/h, the clarifier would be operating at an overflow rate in peak range for longer than the specified 2 hours. This means that even with an equalization basin incorporated into the system, the clarifier will still be overloaded. An overloaded clarifier will lead to system failure, because it means that the solids will not fully settle out of the effluent and they will carry over undesirable constituents, such as BOD, nitrogen, and phosphorus. A larger clarifier, with a resultant surface loading rate within the recommended range, is recommended.

4.3.1 Addition of a pH Adjust system

According to laboratory results from AGROLAB, the WWTS pH resides around 10.1, which qualifies it as a standard basic waste stream (Burt, 2016). At this pH, the system is running outside of the recommended range for effective biological growth, resulting in lowered nitrification effectiveness and a reduced COD reduction potential, as seen in Figure 14 below (Lijklema, 1969). The incorporation of a pH adjust system can solve this problem by buffering with a strong acid to reduce the pH to within a typical range of 7.5 to 8.

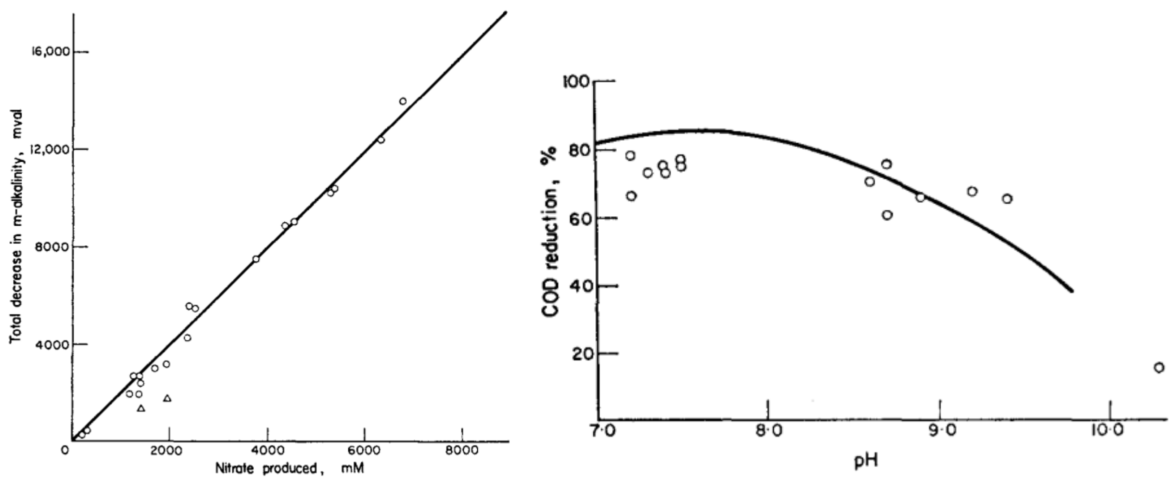

Figure 14: Nitrate produced vs. Alkalinity (left) and pH vs. Percent COD reduction (right) (Lijklema, 1969)

With the addition of an equalization tank, the flowrate and influent wastewater quality of the improved system will be entering the activated sludge system at a steady rate. As such, manual addition of sulfuric acid can be supplied at the influent of the equalization tank. Sulfuric acid is the most commonly used strong acid for buffering in wastewater treatment due to its low cost, accessibility, and safety (Digital Analysis, 2016). As the strong acid travels the length of the tank and is pumped into the aeration tank, it will be naturally mixed, providing a stable pH entering the system. The pH would be measured at the influent of the aeration system ensuring the correct amount of sulfuric acid is being added.

The cost of pH meters range from \$100 to \$4,000. According to www.Grainger.com, some of the top sellers are around \$200-\$400 depending on the brand, but any one from this website or others like it would be sufficient for the system at hand. Along with a pH meter, a metering pump would need to be incorporated to constantly add acid to the system. From www.Grainger.com, metering pump costs range between \$500 and \$1400 depending on capacity. For the school's purposes, a lower capacity pump would be suitable such as the *Diaphragm Metering Pump*, costing \$535.00 on www.Grainger.com and pumping 5 L/hr (1.25 gph). As is, AFS faculty regularly checks the ORP of the WWT Lagoons. During these checks, the same faculty can measure the pH of the activated sludge WWTS to ensure that the metering pump is adding sufficient amounts of acid.

4.3.2 Aeration Design

Based on the currently installed system, the aeration of both stage one and stage two is easily accomplished. The two blowers installed are capable of producing 7200 m^3/d of air each which is over the required amount of air at 4318.2 m^3/d, 5360.8 m^3/d, and 9073.4 m^3/d for minimum, average and maximum flow respectively. The distribution of the air is where the system could run into problems; currently the distribution is metal pipes with sporadic holes drilled to produce coarse bubbles in the tanks.

The advantages that come with using fine bubble diffusers allows for high aeration and oxygen transfer efficiency, requires less energy to operate, and most importantly is easy to adapt to existing systems. Fine bubble diffusers have pores up to 5 mm in diameter. There are some disadvantages to using such small pored diffusers; the smaller pores are susceptible to chemical and biological fouling, they can become more expensive (requiring more routine maintenance to prevent fouling), energy input could increase if fine pores becomes clogged, and fine bubbles do not mix wastewater as well as coarse bubbles. Switching to a membrane diffuser, either coarse or fine bubble, reduces the risk of biological flocs entering the air supply pipes causing blockages. It is recommended that the metal pipes be switched out for PVC and install either coarse or fine bubble diffusers. Having an older system installed, the operating air volume may not be what the manufacturer states. By switching the aeration delivery system it ensures that the blowers will be able to supply ample air to the system even if they do not run at 100% capacity.

4.3.3 Clarifier Design

The existing Stage 2 clarifier that is on-site at AFS would not be sufficient to handle the incoming flow of aerated wastewater. The consistently fluctuating flowrate makes the clarifier's expected efficiency even lower.

As previously stated, the typical range for surface loading rate is 8 – 16 $m^3•m^{-2}•d^{-1}$. The only time the existing clarifier would see an OR within this range would be when the system is seeing the absolute minimum flow it is expected to see. This flow only occurs during the hours of the day when no facilities, like the dairy processing facility and slaughterhouse, are running. Therefore, if the existing system is kept as is, the clarifier would be overloaded whenever the system is running at both average and maximum flows. The absolute peak surface loading rate range that a clarifier should operate at is between 24 – 32 $m^3•m^{-2}•d^{-1}$, also previously mentioned. However, it is not recommended that the system operate at this range for a longer duration than 2 hours. Even considering this high range, the clarifier would still be overloaded during the average and maximum flowrates.

With the addition of an equalization basin, the surface loading rates the existing clarifier would expect to see are more reasonably close the desired range. Table 6 shows the expected surface loading rate for the existing clarifier with the equalization basin-controlled inflows.

Table 6: Existing Clarifier OR with Equalization Basin Flowrates

Parameter	Unit	Minimum	Average	Maximum
Q = Volumetric Flowrate	m^3/d	195	242	352
OR = Surface Loading Rate	$m^3•m^{-2}•d^{-1}$	24.1	29.24	43.5

As can be seen in the table above, the surface loading rates see less of a drastic change when comparing the values from the minimum, average, and maximum flowrates. Although the addition of the equalization basin would control the high fluctuation of surface loading rate the clarifier would expect to see, it would still not be operating within the desirable range of 8 – 16 $m^3 \bullet m^{-2} \bullet d^{-1}$. The OR for the minimum and average flowrates would fall within the acceptable range for peak loading, but this operating duration would last more than 2 hours, which would be undesirable and ineffective in the long run.

Using the desired range of 8 – 16 $m^3 \bullet m^{-2} \bullet d^{-1}$ for the OR, and the minimum, average, and maximum flowrates expected to be seen from the equalization basin, the surface area of the optimum clarifier was calculated. In order to be conservative, an OR of 12 $m^3 \bullet m^{-2} \bullet d^{-1}$ was used to design the clarifier. This is the average of the typical clarifier OR range. Table 7 shows the optimum surface areas of the clarifier for the respective flowrates.

Table 7: Ideal Clarifier Design Parameters

Parameter	Unit	Minimum	Average	Maximum
Q = Volumetric Flowrate	m^3/d	195	242	352
A = Surface Area of Clarifier	m^2	16.3	20.2	29.4

As shown in Table 7, in order for the clarifier to operate at an OR within the typical range, the surface area of the tank would need to be somewhere between 16.3 and 29.4 m^2. This was calculated using Equation 14, which is a variation of Equation 7.

$$A = \frac{Q}{OR} \qquad \textit{Equation 14}$$

Where A is the surface area of the clarifier, Q is the influent flowrate, and OR is the surface loading rate. The existing clarifier has a surface area of 8.1 m^2, which is a bit too small to be able to handle the expected incoming flowrates. If the size of the clarifier was increased to the sizes outlined in Table 7, according to the operational flowrate the system would be running at, it would be more efficiently able to handle and clarify the incoming aerated wastewater.

Another consideration for the improvement of the existing clarifier would be the addition of Lamella plates, which is a clarification method that incorporates plates that run length-wise across the clarifier (Metcalf and Eddy, 2003). These plates allow for additional settling surface area, so the particles in the incoming aerated wastewater have more material surface area to settle on. The addition of these plates to the existing clarifier would be a possibility, in order to compensate for the lack of surface area of the tank by itself.

5. Conclusions

The American Farm School has been using a wastewater treatment system designed and constructed in 1983 that was decommissioned in 2012. After shutting down the system, the school attempted to minimize storm runoff from entering the system; which was part of the reason for the system's failure. This project was used as a preliminary phase in the American Farm School's goal of reducing agriculture waste and improving the quality of their reclaimed wastewater for irrigation. AFS must adhere to effluent quality standards for discharging to the municipality's system. The domestic wastewater, which was being combined with dairy processing wastewater, was not acceptable from a water quality standpoint for delivery to EYATH's system. For this reason, EYATH was threatening to fine AFS if the effluent conditions did not improve. Upon arrival, the team realized the potential in improving the amount of treated wastewater effluent quality by incorporating slaughterhouse wastewater into the activated sludge treatment process in addition to treating dairy facility wastewater. The greatest challenge was that wastewater flowrates into the system fluctuated due to the fact that the majority of wastewater flowed into the system in a span of 7 hours. With the inclusion of the slaughterhouse waste, this exaggerated the variation in flow. High influent pH was determined to be insignificant in changing the final effluent pH, which has not exceeded 8.6 throughout the system's use.

In order to mitigate problems, the team suggested the use of an equalization tank using the same tank as the current Stage 1. This design modification would allow for a regulated flowrate and consistent BOD concentrations. As is, the current system is over designed to treat the organic load that enters per day. However, the majority of the organic load enters the system in a short timeframe. By converting Stage 1 Aeration and Settling into an aeration tank, the organic load is dispersed throughout the day.

With this design modification, no new infrastructure is required. The equalization tank design would require at least two pumps: one to pump wastewater during daily flow conditions and a second when the flowrates increase because the slaughterhouse is online. A third pump is recommended as a backup in the event of pump failure. The team also recommends the installation of a flowmeter to measure flowrates entering the equalization tank continuously and a series of 4 float switches to control the pumps. The minor changes needed to the system in order to accommodate all desired wastewater make the cost of installation and maintenance minimal.

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Appendix A: Sample Labeling Guidelines

On each sample bottle, place a piece of plain white tape and mark the following details:

- Letter designating whether a milk processing wastewater sample, or a slaughterhouse wastewater sample
    - *M* designates milk processing waste
    - *S* designates slaughterhouse waste
- Date of sample
- Time of sample

Appendix B: On-Site Safety Manual

Laboratory Safety:

- Wear gloves of a material compatible with the solutions/chemicals to be used when working with samples or dangerous chemicals
- Wash hands and forearms before and after in a designated bathroom that isn't used for food preparation
- Wear safety glasses or goggles, mask and lab coat
- Secure dangling jewelry
- No food/drink
- Keep work area clean
- Never leave an experiment or open flame unattended
- Always work in groups and, as a minimum, in pairs
- Know what you are working with
    - Be sure what you are mixing can be mixed
    - Review Material Safety Data Sheets (MSDS) of all materials to be in contact with
    - Dispose of samples/chemicals properly
    - Label all containers when working with waste/chemicals/etc.
- Wear proper footwear (close toed, sturdy, non-slip) and pants
- Hair should be pulled back and secured
- Have appropriate spill cleanup supplies available
- Get help from professional staff or a qualified employee if you are unsure about how to carry out a specific task. Procedures for laboratory experiments and analyses should be approved by the project advisors.
- Be aware of surroundings, your own abilities, and use common sense
    - Take an extra measure of caution because verbal warnings will not be understood if they are said in Greek
    - If you have any physical disabilities or restrictions, make sure to take them into consideration and make others aware of them
    - Eating, drinking and sleeping all affect physical and cognitive mobility. Be aware of how you are taking care of yourself outside of work hours
    - Phones may be necessary in the lab but caution should be used
        - Do not try to multitask. Step away from activity and return when you can give your full attention.

Field Safety:

- Wear gloves of appropriate material when working with any equipment or taking samples
- Wear a mask when working with samples
- Wear proper footwear (close toed, sturdy, non-slip) and pants

- Hardhats shall be worn in all construction areas. The on-site supervisors should be consulted to verify locations where hard hats are needed.
- Wash hands and forearms before and after in a designated bathroom that isn't used for food preparation
- Hair should be pulled back and secured, and beards trimmed
- Never enter a confined space or tank
    - This includes enclosed areas with low oxygen concentration, strong odors or high concentrations of carbon dioxide, methane, or other gases that won't sustain aerobic life. The on-site supervisors should be consulted to identify confined spaces.
- Always work in groups and, as a minimum, in pairs
- No driving any equipment including cars, vans, trucks, motor bikes, tractors, loaders, or excavators
- Get help from professional staff or a qualified employee if you are unsure about how to carry out a specific task
- Be aware of surroundings, your own abilities, and use common sense
    - Take an extra measure of caution because verbal warnings will not be understood if they are said in Greek
    - If you have any physical disabilities or restrictions, make sure to take them into consideration and make others aware of them
    - Eating, drinking and sleeping all affect physical and cognitive mobility. Be aware of how you are taking care of yourself outside of work hours
- Phones may be necessary in the field but caution should be used
    - Do not try to multitask. Step away from activity and return when you can give your full attention.

Appendix C: Original Design Calculations

			Minimum	Average	Maximum
	Q_{avg} = volumetric flowrate per day (including off work hours)	m^3/s	0.0022590	0.002805	0.004079
		m^3/h	8.1325068	10.099	14.685
		m^3/d	195.18016	242.386	352.444
Stage 1 (Aeration and Settling)	θ_d = HRT	sec	50527.89	40687.38	27981.89
		min	842.13	678.12	466.36
		hours	14.04	11.30	7.77
		days	0.58	0.47	0.32
	S_0 = Influent BOD	mg/L	450.00	450.00	524.00
	S_e = Typical effluent substrate concentration	mg/L	86.34	160.46	524.00
	X_v = VSS Concentration (no sludge recycle)	mg/L	210.80	168.95	0.00
	L = rate of oxygen utilization	mg/s	821.53	812.29	1941.70
		kg/day	70.98	70.18	167.76
	Q_a = volumetric rate of air supply	m^3/d	3503.818	3464.392	8281.353
	F/M	mg/mg/d	3.650	5.656	
Stage 2 (Aeration)	$S_0 = S_e$ (stage 1)	mg/L	86.34	160.46	524.00
		kg/m^3	0.0863351 3	0.16045541	0.524
	θ_d = HRT	sec	111032.18	89408.23	61488.61
		min	1850.54	1490.14	1024.81
		hours	30.84	24.84	17.08
		days	1.29	1.03	0.71
	θ_x = Assumed SRT	days	186.06	80.62	16.98
	S_e = Effluent substrate concentration	mg/L	1.34	1.48	2.48
	X_v = VSS Concentration (with sludge recycle)	mg/L	607.042	1273.037	3697.857

	L = rate of oxygen utilization	mg/s	192.014	445.977	2127.405
		kg/day	16.590	38.532	183.808
	Q_a = volumetric rate of air supply	m^3/d	818.939	1902.089	9073.367
Stage 2 (Settling)	OR = surface loading rate	m/s	9.52263E-05	0.00095620	0.00292741
		m/d	8.228	82.616	252.928435
	F/M	mg/mg/d	0.111	0.122	0.199
Air Requirements	Q_a Capacity of Pump	m^3/d	14400	14400	14400
		m^3/h	600	600	600
	Q_a use in aeration tank	m^3/d	4322.757	5366.481	17354.720
		m^3/h	180.115	223.603	723.113
	Remaining Capacity (Air Lift)	m^3/d	10077.243	9033.519	-2954.720
		m^3/h	419.885	376.397	-123.113

Appendix D: ENVIROMAG 2000 Flowmeter Datasheet

ENVIROMAG 2000

Technical Data

Nominal diameter	VB14											VB15								VB16											
ASME [inch]	3/8"	1/2"	1"	1 1/4"	1 1/2"	2"	2 1/2"	3"	4"	5"	6"	8"	10"	12"	14"	16"	18"	20"	24"	28"	32"	36"	40"	48"	54"	56"	60"	64"	72"	80"
DN [mm]	10	15	25	32	40	50	65	80	100	125	150	200	250	300	350	400	450	500	600	700	800	900	1000	1200	1350	1400	1500	1600	1800	2000

Nominal Flange Pressure

- ASME B16.5 - 150 lbs RF
- ASME B16.5 - 300 lbs RF
- AWWA - class B FF
- AWWA - class D FF

> 80"/2000 mm on request (OPTIFLUX Series)

AWWA Class D flanges $\leq$ 150PSI

AWWA Class B flanges $\leq$ 50 PSI

Liner

- Polypropylene
- Hardrubber
- Polyurethane

See pressure and temperature limits for various liners

Electrodes (Replaceable)

- Hastelloy C4
- Stainless steel(AISI 316 L)
- Hastelloy B2

(Titanium, Tantalum, Platinum available on request)

Grounding rings

- Virtual Reference [1]
- Hastelloy C4
- Stainless steel 1.4571 (AISI 316 Ti)

[1]*Only with IFC300 Converter. Must specify option at time of order.*

Materials

- Measuring tube - austenitic stainless steel
- Housing
- Sheet steel (polyurethane coated)
- Stainless steel
- Flanges
- Carbon Steel
- Stainless steel(AISI 316 L)
- Stainless steel (AISI 304)
- Connection box
- Die-cast aluminium (polyurethane coated)
- Stainless steel connection box

Protection category

- IP 66 / 67 (NEMA 4/4X / 6)
- IP 68 (NEMA 6P [1])

Approvals

- General Purpose
- CSA - Ordinary Locations

Please note the approvals are for flow sensors only.

Versions

- Compact + IFC 300 C
- Separate + IFC 300 F, R, W
- Compact + IFC 010 C
- Separate + IFC 010 W

Conductivity

Min. conductivity	min. 20 µS/cm

[1]*Separate Only with Stainless Steel Junction Box*

Standard Feature — Optional Feature

ENVIROMAG 2000

Dimensions and Weights

Refer to diagrams on page 7

Nominal size	Flange Type/	Dimensions [mm]						Dimensions [inch]						Approximate weight	
					T						T				
[inch-mm]	Rating	L[1]	H	W	box	010	300	L[1]	H	W	box	010	300	[kg]	[lb]
3/8"- 10	ANSI 150	150	179	89	257	285	339	5.9	7.1	3.5	10.1	11.2	13.3	7	16.0
1/2"- 15	ANSI 150	150	179	89	257	285	339	5.9	7.1	3.5	10.1	11.2	13.3	7	16.0
1" - 25	ANSI 150	150	179	108	257	285	339	5.9	7.1	4.3	10.1	11.2	13.3	8	18.0
1½" -40	ANSI 150	150	203	127	281	309	363	5.9	8.0	5.0	11.1	12.2	14.3	10	22.0
2" -50	ANSI 150	200	191	152	269	297	351	7.9	7.5	6.0	10.6	11.7	13.8	13	29.0
3" - 80	ANSI 150	200	210	191	288	316	370	7.9	8.3	7.5	11.3	12.4	14.6	17	37.0
4" - 100	ANSI 150	250	256	229	334	362	416	9.8	10.1	9.0	13.2	14.3	16.4	23	51.0
5" - 125	ANSI 150	250	280	254	358	386	440	9.8	11.0	10.0	14.1	15.2	17.3	27	60.0
6" - 150	ANSI 150	300	304	279	382	410	464	11.8	12.0	11.0	15.0	16.1	18.3	34	75.0
8" - 200	ANSI 150	350	355	343	433	461	515	13.8	14.0	13.5	17.0	18.1	20.3	50	110.0
10" - 250	ANSI 150	400	433	406	511	539	593	15.8	17.1	16.0	20.1	21.2	23.3	73	160.0
12" - 300	ANSI 150	500	499	483	577	605	659	19.7	19.7	19.0	22.7	23.8	25.9	100	220.0
14" - 350	ANSI 150	500	552	533	630	658	712	19.7	21.7	21.0	24.8	25.9	28.0	114	250.0
16" - 400	ANSI 150	600	608	597	686	714	768	23.6	23.9	23.5	27.0	28.1	30.2	155	340.0
18" - 450	ANSI 150	600	672	635	750	778	832	23.6	26.5	25.0	29.5	30.6	32.8	170	375.0
20" - 500	AWWA Cl. D	600	739	699	817	845	899	23.6	29.1	27.5	32.2	33.3	35.4	191	420.0
24" - 600	AWWA Cl. D	600	852	813	930	958	1012	23.6	33.5	32.0	36.6	37.7	39.8	250	550.0
28" - 700	AWWA Cl. D	700	918	927	996	1024	1078	27.6	36.1	36.5	39.2	40.3	42.4	320	704.0
30" - 750	AWWA Cl. D	750	974	984	1052	1080	1134	29.5	38.3	38.8	41.4	42.5	44.6	358	787.6
32" - 800	AWWA Cl. D	800	1038	1060	1116	1144	1198	31.5	40.9	41.8	43.9	45.0	47.2	395	869.0
36" - 900	AWWA Cl. D	900	1144	1168	1222	1250	1304	35.4	45.0	46.0	48.1	49.2	51.3	450	990.0
40" - 1000	AWWA Cl. D	1000	1258	1289	1336	1364	1418	39.4	49.5	50.8	52.6	53.7	55.8	665	1463.0
42" - 1050	AWWA Cl. D	1300	1313	1350	1391	Not Applicable	1473	51.2	51.7	53.1	54.8	Not Applicable	58.0	683	1502.6
48" - 1200	AWWA Cl. D	1300	1483	1511	1561	Not Applicable	1643	51.2	58.4	59.5	61.5	Not Applicable	64.7	970	2134.0
54" - 1350	AWWA Cl. D	1600	1635	1682	1713	Not Applicable	1795	63.0	64.4	66.2	67.4	Not Applicable	70.7	TBA	TBA
60" - 1500	AWWA Cl. D	1700	1782	1860	1860	Not Applicable	1942	66.9	70.2	73.2	73.2	Not Applicable	76.5	TBA	TBA
70" - 1750	AWWA Cl. D	1800	2139	2197	2217	Not Applicable	2299	70.9	84.2	86.5	87.3	Not Applicable	90.5	TBA	TBA

Nominal size	Flange Type/	Dimensions [mm]						Dimensions [inch]						Approximate weight	
					T						T				
[inch-mm]	Rating	L[1]	H	W	box	010	300	L[1]	H	W	box	010	300	[kg]	[lb]
1" - 25	ANSI 300	150	145	124	223	251	305	5.9	5.7	4.9	8.8	9.9	12.0	8	18.0
1½" -40	ANSI 300	200	169	156	247	275	329	7.9	6.7	6.1	9.7	10.8	13.0	9	20.0
2" -50	ANSI 300	250	186	165	264	292	346	9.8	7.3	6.5	10.4	11.5	13.6	13	29.0
3" - 80	ANSI 300	250	214	210	292	320	374	9.8	8.4	8.3	11.5	12.6	14.7	17	37.0
4" - 100	ANSI 300	300	275	254	353	381	435	11.8	10.8	10.0	13.9	15.0	17.1	23	51.0
6" - 150	ANSI 300	350	316	318	394	422	476	13.8	12.4	12.5	15.5	16.6	18.7	36	79.0
8" - 200	ANSI 300	400	382	381	460	488	542	15.8	15.0	15.0	18.1	19.2	21.3	71	157.0
10" - 250	ANSI 300	500	448	445	526	554	608	19.7	17.6	17.5	20.7	21.8	23.9	112	247.0
12" - 300	ANSI 300	600	519	521	597	625	679	23.6	20.4	20.5	23.5	24.6	26.7	170	375.0
14" - 350	ANSI 300	700	595	584	673	701	755	27.6	23.4	23.0	26.5	27.6	29.7	215	474.0
16" - 400	ANSI 300	800	646	648	724	752	806	31.5	25.4	25.5	28.5	29.6	31.7	290	639.0
18" - 450	ANSI 300	800	709	711	787	815	869	31.5	27.9	28.0	31.0	32.1	34.2	359	789.0
20" - 500	ANSI 300	800	777	775	855	883	937	31.5	30.6	30.5	33.7	34.8	36.9	426	937.0
24" - 600	ANSI 300	800	903	914	981	1009	1063	31.5	35.5	36.0	38.6	39.7	41.8	611	1345.0

Notes:

[1] If flowmeter is supplied with separate grounding rings, the totals fitting length "L" is computed as follows: "L" + 2 x 3 mm (1/8")+ 2 x gasket thickness

Appendix E: ENM-10 Level Regulator Datasheet

This level regulator is available in different versions.

Dimensions

For density g/cm3	Regulator Length mm	Diameter mm
0.65 - 0.80	194	100
0.80 - 0.95	177	100
0.95 - 1.10	162	100
1.05 - 1.20	142	100
1.20 - 1.30	133	100
1.30 - 1.40	130	100
1.40 - 1.50	126	100

Materials

Body:	polypropylene
Bending relief:	EPDM rubber
Cable:	special compound PVC or CPE (chlorinate polye-thylene rubber)

Technical data

Liquid tempera-ture:	min. 0°C max. 60°C
Liquid density:	min. 0.65 g/cm3 max. 1.5 g/cm3
Degree of pro-tection:	IP68, 20 m

Micro switch data

IC*, AC:	250 V/ 10 A resistive load 250 V/ 3 A inductive load at cosφ= 0,5
IC*, DC:	30 V/ 5 A 250 V/ 0,05 A

** IC = Interrupting Capacity*

Appendix F: Equalization Basin Calculations

			Minimum	Average	Maximum
	Q_{avg} = volumetric flowrate per day (including off work hours)	m^3/s	0.002259	0.002805	0.004079
		m^3/h	8.133	10.099	14.685
		m^3/d	195.180	242.386	352.444
Stage 1eq (Aeration)	S_0	mg/L	450	450	524
		kg/m^3	0.45	0.45	0.524
	θ_d = HRT	sec	111032.18	89408.23	61488.61
		min	1850.54	1490.14	1024.81
		hours	30.84	24.84	17.08
		days	1.29	1.03	0.71
	θ_x = Assumed SRT	days	35.70	28.74	16.98
	S_e = Effluent substrate concentration	mg/L	1.81	1.96	2.48
	X_v = VSS Concentration (with sludge recycle)	mg/L	2377.528	2740.626	3697.857
	L = rate of oxygen utilization	mg/s	1012.467	1256.934	2127.405
		kg/day	87.477	108.599	183.808
	Q_a = volumetric rate of air supply	m^3/d	4318.162	5360.813	9073.367
Stage 1eq (Settling)	OR = surface loading rate	m/s	0.0003	0.0003	0.0005
		m/d	24.096	29.924	43.512
	F/M Ratio	mg/mg/d	0.1473	0.1587	0.1991
Air Requirements	Q_a Capacity of Pump	m^3/d	14400	14400	14400
		m^3/h	600	600	600
	Q_a use in aeration tank	m^3/d	4318.162	5360.813	9073.367
		m^3/h	179.923433	223.367201	378.056958
	Remaining Capacity (Air Lift)	m^3/d	10081.838	9039.187	5326.633
		m^3/h	420.077	376.633	221.943