JOY CONE CO. INDUSTRIAL PRETREATMENT DESIGN REPORT

Final Design Report

For: Joy Cone Co. Flagstaff

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Table of Contents

List of Figures

List of Tables

List of Equations

List of Abbreviations

ABET: Accreditation Board for Engineering and Technology

ADEQ: Arizona Department of Environmental Quality

ADOT: Arizona Department of Transportation

APP: Aquifer Protection Program

AZPDES: Arizona Pollutant Discharge Elimination System

ASTM: American Society of Testing and Materials

CAD: Computer Aided Drafting

COD: Chemical Oxygen Demand

CBOD: Carbonaceous Biological Oxygen Demand

CFS: Cubic Feet per Second

BOD: Biological oxygen demand

EIT: Engineer in Training

ENG: Engineer

GIS: Geographic Information System

GPD: Gallons per day

GPM: Gallons per minute

Inner Basin: Inner Basin Environmental Labs

Lab Tech: Lab Technician

MBBR: Moving bed biofilm reactor

NAU: Northern Arizona University

NOAA: National Oceanic and Atmospheric Administration

NPDES: National Pollutant Discharge Elimination System

POTW: Publicly owned treatment works

RO: Reverse osmosis

SENG: Senior Engineer

S Tech: Survey Technician

TBEL: Technology-based effluent limitation

The City: The City of Flagstaff

TKN: Total Kjeldahl nitrogen

TMDL: Total maximum daily load

TSS: Total suspended solids

USGS: United States Geological Survey

WQBEL: Water quality-based effluent limitation

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1.0 Project Introduction

1.1 Project Objectives

The main objective of this project is to develop a novel design for the Joy Cone Co. factory in Flagstaff, Arizona that will efficiently treat the contaminants of concern in their wastewater on-site to achieve concentrations below those regulated by their City of Flagstaff Pretreatment Permit. The factory produces 585,000,000 cones per year on-site. As of January 2020, the factory has been exceeding the concentration limits of several industrial pollutants regulated by their pretreatment permit with the City of Flagstaff ("the City"). The pretreatment permit regulates the concentration of pollutants that can be discharged to the publicly owned treatment works (POTWs) of Flagstaff and is in place to ensure that influent can be treated efficiently and thoroughly by existing processes. Joy Cone Co. discharges roughly 500,000 gallons of water per year to the City of Flagstaff public wastewater system. Biological oxygen demand (BOD), total Kjeldahl nitrogen (TKN), and total suspended solids (TSS) are the primary pollutants of concern in the discharge, with TKN far exceeding regulatory limits. These pollutants derive from the various ingredients used in the production process of ice cream cones and are found in the discharge stream of the water used to clean their batter transportation lines and other manufacturing processes.

1.2 Project Background

1.2.1 Site Location

The Joy Cone Co. factory is located in southern Flagstaff, Arizona. Figure 1-1 below shows the location of the factory in relation to the rest of the city. [1]

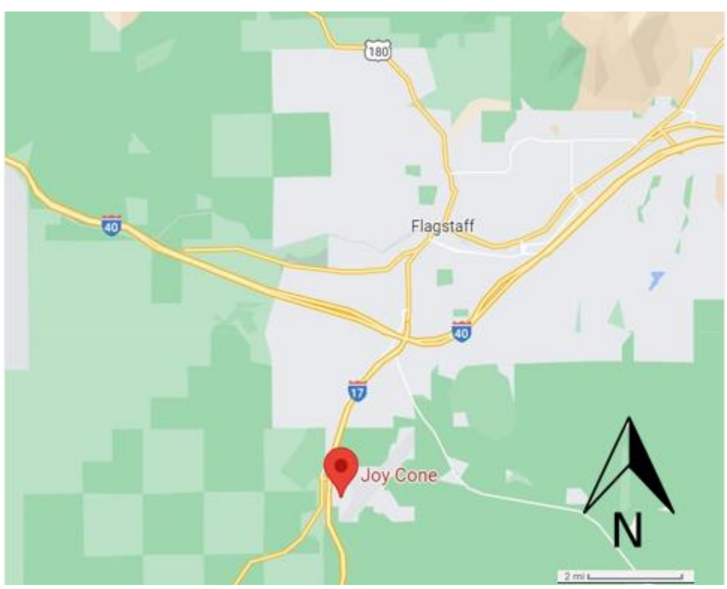

Figure 1- 1: Location of Joy Cone facility in Flagstaff, Arizona

The address of the Joy Cone Co. factory is 2843 W. Shamrell Blvd #9414, Flagstaff, Arizona 86005. The factory owns 30 acres of land bordered by the Flagstaff Pulliam Regional Airport to the east and public Forest Service land to the south. Figure 1-2 shows the aerial view of the factory and the adjacent land. Interstate 17 can be seen west of the building and runways at the Flagstaff Pulliam Airport are to the east. [1]

Most of the land owned by Joy Cone Co. is undeveloped, with the exception of the parking lot and physical building; the rest of the land is a natural forest area. Figure 1-2 shows the property boundaries of the land parcel where the factory is located outlined in red [1]. A City of Flagstaff sanitary sewer is located on the north side of the factory building along Shamrell Blvd. A detention basin is located on the west side of the factory, as seen in Figure 1-2.

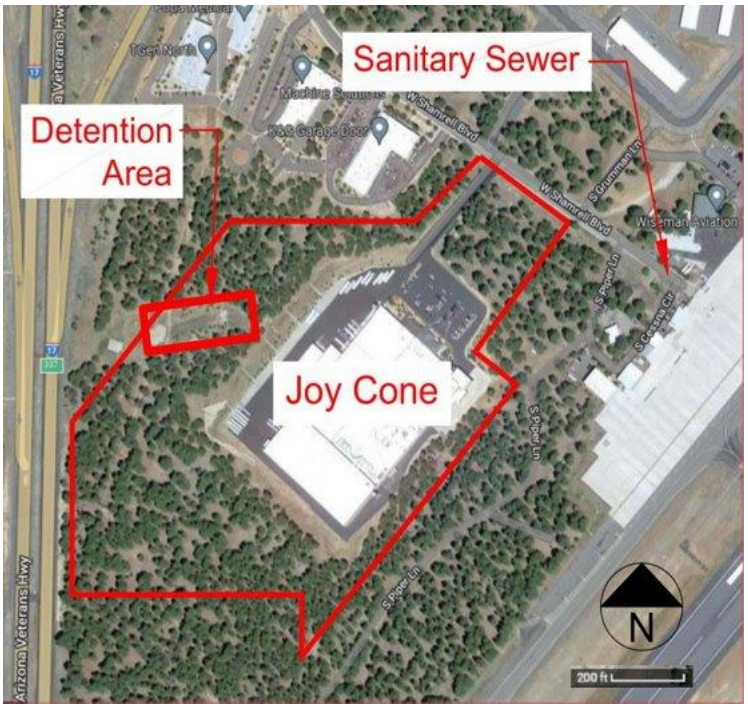

Figure 1- 2: Factory land parcel and detention area aerial view

The 1.46-acre detention basin (see Figure 1-2) is currently used for stormwater runoff that is not connected to the sewer or involved in the current pretreatment process. The basin is located on the west side of the facility and includes a ditch used to transport runoff from the parking lot and surrounding natural areas to the detention basin. There are several culverts directing water southwest within the basin that eventually allow for water flow underneath the I-17. It is unclear whether the basin effluent flows to a specific body of water, but likely flows into the Oak Creek drainage basin.

1.2.2 Current Treatment System

The industrial effluent produced by Joy Cone Co. contains leftover batter that is cleaned from their batter lines and other equipment. Their batter mainly consists of flour, sugar, tapioca, and flavoring and is mixed with water and cleaning chemicals used in the washing process. This wastewater stream is roughly 500,000 gallons per year. Their wastewater treatment system features the use of a rotary drum vacuum filter using an earth media filter to remove suspended solids. There are two 8,000-gallon underground holding tanks located on the north-east side of the building that retain and lightly mix wastewater before pretreatment. Figures 1-3 and 1-4 below are photographs showing the rotary drum vacuum filter located in the water treatment room at the Joy Cone Co. facility.

Figure 1- 3: Current rotary drum vacuum filter

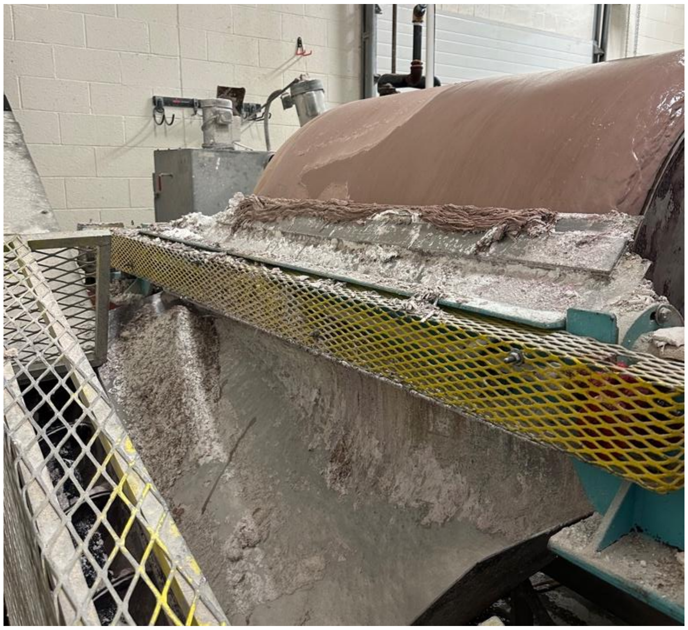

Figure 1- 4: Solids scraping system on drum filter.

Table 1-1 below shows the reported average concentration of contaminants by Joy Cone Co. in the pretreated effluent which were analyzed by Inner Basin Laboratories. Table 1-1 covers data from early 2022. These limits are compared to the permit limits under the City of Flagstaff Industrial Pretreatment Permit currently held by Joy Cone Co.

Table 1- 1: Average contaminant concentrations compared to permit limits

Contaminant	Permit Levels	Average Concentrations (mg/L)	Average Concentrations (lb/day)
BOD	700 lb/day	23440	293
TKN	173 mg/L	191	2
TSS	130 lb/day	307	3

Joy Cone Co. currently discharges their pretreated industrial effluent to the City of Flagstaff sewer system regulated by the City of Flagstaff. The treated effluent is discharged by pump to the sanitary sewer located west of the factory as shown in Figure 1-2. This system has been capable of meeting regulatory requirements in the past, but a change in the regulation of TKN and the age of their current treatment system has caused exceedances in the factory's effluent. The rotary drum system has been effective in removing suspended solids in the past but is no longer functioning well enough to meet the City of Flagstaff standards. The system does not treat TKN or BOD, and the facility has been experiencing exceedances of those contaminants January 2020 and have received fines for TKN exceedances due to the updated TKN standards.

Joy Cone Co. does not plan to increase production at this facility. Since there is no prediction of further growth, the factory does not want a design capable of treating more wastewater than they currently produce in order to reduce costs, maintenance, and land usage.

1.3 Project Constraints/Limitations

There were three major constraints that had to be considered in the design of the new pretreatment system. The first is the transportation of water from the factory to the sewer system. The facility is located at a lower elevation than the surrounding area; it was preferred to minimize the use of pumps to raise/transport wastewater due to cost and maintenance. The elevation of the current detention basin is also lower than that of the City sewer inlets.

Second, because the factory currently owns 30 acres of surrounding land, there was ample area to implement a new treatment system. Although the area was available, the factory is surrounded by trees and Forest Service land. Joy Cone Co. has stressed that removing trees is not ideal due to their aesthetic value, sound reduction qualities, and creation of a natural privacy barrier. The new design was required to minimize disturbance to natural areas as much as possible in order to retain these valuable qualities of the land.

Third, Joy Cone Co. transports batter using a pipe network throughout the facility. These pipes are cleaned at a minimum of every six weeks, but as often as once per week. The pipes are cleaned to maintain the integrity of the batter and to prevent contamination between batter types. The cleaning process includes the use of a chlorinated detergent, Principal®, and a liquid acid sanitizer, Mandate™ Plus, both manufactured by Ecolab®, that are flushed through the system to eliminate buildup on the inside of the pipes. The use of these chemicals has made biological wastewater treatment difficult and could affect the health of microorganisms used in the treatment process. Since the cleaning does not occur daily, the concentration of these chemicals in the effluent is variable. The variability of the concentrations creates a challenge because treatment regimens may need to be adjusted or changed depending on the concentration of the chemicals and the pH of the water. Alternatives for the new treatment system will include changing the cleaning solutions to more ecofriendly alternatives. A biodegradable and ammonia free solution would reduce the harm done to any microorganisms used in the treatment process.

2.0 Regulations Research

Joy Cone Co. currently holds a City of Flagstaff Industrial Waste Discharge Permit, also known as an Industrial Pretreatment Permit. These permits are required for industrial dischargers by the US EPA’s National Pretreatment program, which is in place to ensure that publicly owned treatment works (POTWs) are receiving influent that they can treat with existing infrastructure. Wastewater from industrial sources is monitored for increasing contaminant loads, and standards may be changed by the City according to varying loads coming into the POTWs.

Due to the client's desire to use the existing detention area for treatment, the permit research focused on National Pollutant Discharge Elimination System (NPDES) permits. These permits are obtained by facilities that are discharging any wastewater to Waters of the U.S. A NPDES permit has three different categories of effluent limitations: technology-based, water quality based, and total maximum daily loads (TMDL) based. Technology-based effluent limitations (TBELs) require a facility to treat their wastewater stream to the minimum pollutant level possible with available technology and are the base level limitations used for a NPDES permit. Water quality-based effluent limitations (WQBELs) are directly linked to TMDLs, in that the state is authorized to designate rankings to local bodies of water based on whether TBELs will be sufficient to maintain the quality of their water. If technology-based controls are insufficient, the water body will be assigned a TMDL and a permit to discharge into said water body may be written with more severe water quality-based standards. For each NPDES permit, the quality of the body of water that will be discharged must be analyzed before the applicable effluent limitation category can be determined. Due to the nature of the project, it can safely be assumed that TBELs would likely be used in the development of a permit, since the effluent would be unlikely to cause disruptions to the quality of a receiving water body. In Arizona, the state has jurisdiction over the NPDES program, making it so that any permits are under the Arizona Pollutant Discharge Elimination System (AZPDES), and the effluent limitations are determined by ADEQ. However, there are currently no direct industrial dischargers located in the City, meaning that no industry currently holds an AZPDES permit for their wastewater discharge, and they all operate under Industrial Pretreatment Permits.

With the possible design of a constructed wetland, the facility may also be required to obtain an Aquifer Protection Program (APP) permit. These permits are typically required for facilities that apply their waste to wetlands for treatment, due to the possibility of contaminant migration to the aquifer or vadose zone. The Arizona Department of Environmental Quality (ADEQ) may decide that a facility is not required to obtain an APP permit if they determine that there is no danger of pollutant migration to an aquifer. However, if a wetland is lined, an APP permit is unnecessary due to the elimination of the potential of migration to an aquifer.

3.0 Treatment Process Research

In order to adequately design a pretreatment or treatment system for the Joy Cone facility, the Honeycomb team researched different possible processes and their advantages and disadvantages.

3.1 Traditional Treatment Methods

For TSS, the first common industrial wastewater effluent treatment method that was considered is chemical treatment paired with the appropriate settling tanks and/or clarifiers. Coagulants are typically aluminum or iron-based chemicals that change the magnetic charge of particles in the wastewater, causing them to attract instead of repelling each other. This is paired with flocculation to induce the coagulation process and a setting tank to collect the layer of solids or sludge at the bottom of the tank to be removed. In general, this combination is widely used due to its efficacy in most wastewater treatment systems. Depending on which chemical coagulant is used, these systems can treat anywhere from 70-90% of TSS, with the removal rates for BOD being typically higher, anywhere from 80-90% [2]. Though this process proves to be very efficient for BOD and TSS removal, it does not typically remove TKN without a joint biological process. Further, a sludge dewatering system would need to be used in order to dispose of the solid waste produced in the settling process. The sludge from the bottom of the tank would likely be pumped to a small-scale sludge dewatering belt. This type of dewatering process would likely also involve the use of a sludge thickener, such as a polymer, which would be added to the sludge before it is dewatered. This process would likely be costly as the polymer would need to be purchased and delivered on a regular basis.

The team also investigated rotary drum vacuum filters, since that is the process that is currently being used by Joy Cone Co. Although this treatment method is not effective on its own as proven by Joy Cone Co.'s current effluent concentrations, it is widely used in food processing when followed up with a form of biological treatment. The Figure 3- 1: Rotary drum vacuum filter schematic [8], shows the basic structure of the rotary vacuum drum filters as well its components [3].This treatment has proven to be effective in the removal of TSS, and thereby the removal of BOD with certain companies' installations having removal rates of up to 99% of TSS and 85% of BOD [4]. In terms of size, they can range anywhere from 93 square feet to 300 square feet determined by how fast the volume flows through the filter, not by water volume [4]. An advantage of rotary drum vacuum filters is that they are low maintenance and easy to operate. These systems work with either Diatomaceous Earth or Perlite, however studies have found that perlite yields up to 20% greater filtering capacity as well as between 20%-50% less dense filter cake after it is removed from the filter [5].

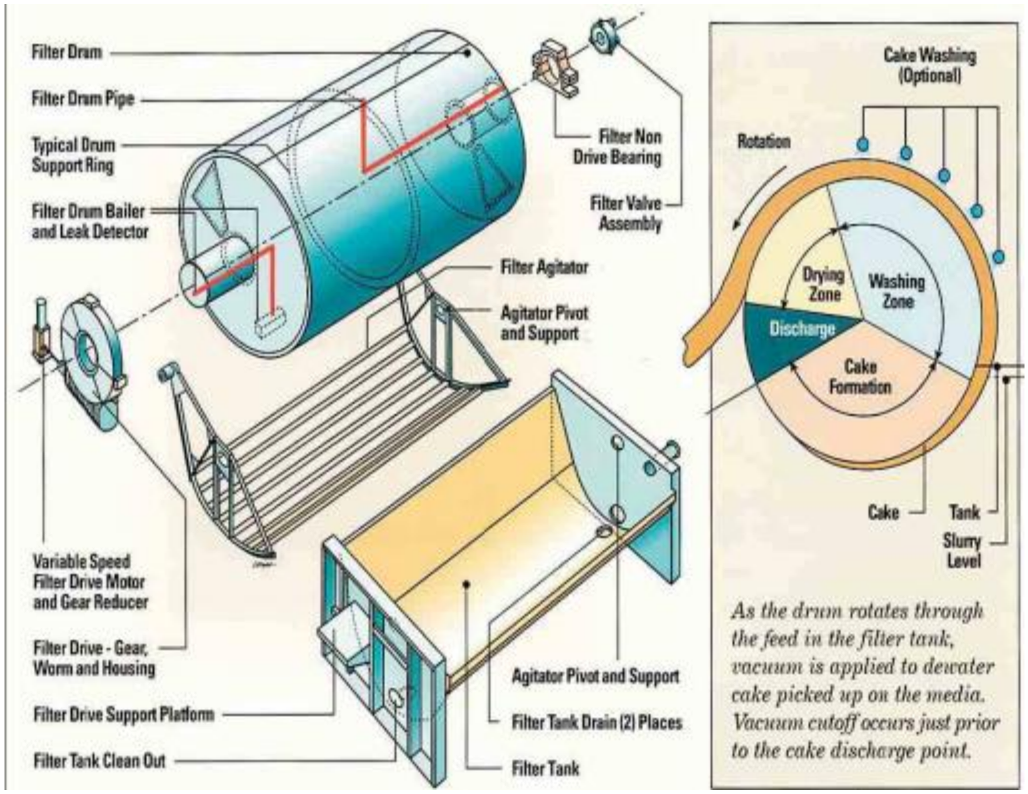

Figure 3- 1: Rotary drum vacuum filter schematic [8]

3.2 Sludge Dewatering

The sole sludge dewatering system that was researched was a plate and frame filter press. While there are other types of sludge dewatering systems available, the plate and frame proved to be the most used across all industrial wastewater treatment applications. Other sludge dewatering systems, such as a belt filter press, are typically used when a sludge with a high-water content is acceptable for disposal. For the purposes of this project, a sludge with similar water content to what is currently being produced is preferable. Upon visual inspection of the sludge from the rotary drum filter, the cake is relatively dry, and does not appear to only have the 18-25% solids that is typical of a belt filter press. High pressure plate and frame filter presses can produce a sludge that is anywhere from 40-70% solids by weight [6].

3.3 Biological Treatments

Due to the high ammonia and BOD levels found in the factory’s wastewater, several biological treatment techniques for wastewater were researched. The first technique was biofiltration, which is a general category of treatment systems that work by using microorganisms that create a biofilm on the growth medium and consume contaminants as a food source. The main advantages of implementing a biofiltration system is that it works year-round, it can remove a variety of contaminants, and the microorganisms are flexible and adaptable. A specific form of biofiltration system that was researched was a trickling filter. Trickling filters treat wastewater by continuously sprinkling the water over a layer of media on which a biofilm is created. The organisms on the biofilm consume the contaminants in the wastewater, thereby removing them. A trickling filter is constructed to produce effluent free BOD, COD, nutrients, and suspended solids. Trickling filters are typically used as a secondary treatment process, as a preliminary filtration to filter out solids is required to not overload the media. A trickling filter is used to degrade a variety of organic wastes. One disadvantage of this type of filter is that there is possibility of clogging and flow rate inflexibility, since trickling filters require continuous flow to prevent the system from going anaerobic. Trickling filters have a BOD removal efficiency of approximately 75 to 90 percent. Nitrification of primary effluent is relatively higher in low-rate filters, whereas high-rate trickling filters produce partially nitrified effluent. The solids removal for this type of filter is approximately 38 to 56 percent. Total nitrogen removal ranges from 15 to 50 percent [7]. A basic overview of a trickling filter can be seen in Figure 3-2 below.

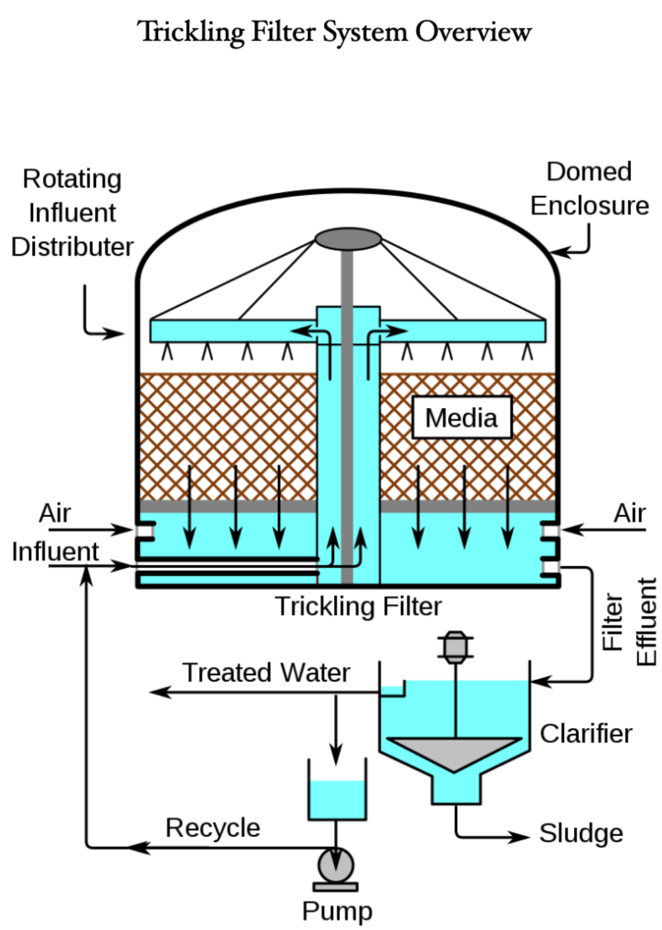

Figure 3- 2: Trickling filter design [8]

The second type of biological treatment that was researched was a moving bed biofilm reactor (MBBR). Unlike a trickling filter, MBBR processes are batch, meaning that they have a hydraulic retention time during which the water is treated, rather than having a continuous flow of wastewater. The process consists of a tank with an aeration grid and suspended media on which biological communities grow, maximizing the treatment surface area. The media used is typically small plastic chips that occupy anywhere from 50%-70% of the tank's volume. The aeration grid ensures that the tank remains thoroughly oxygenated, with the aeration type resembling a fine bubble diffuser. Figure 3-3 below shows a simplified diagram of a commercial MBBR system.

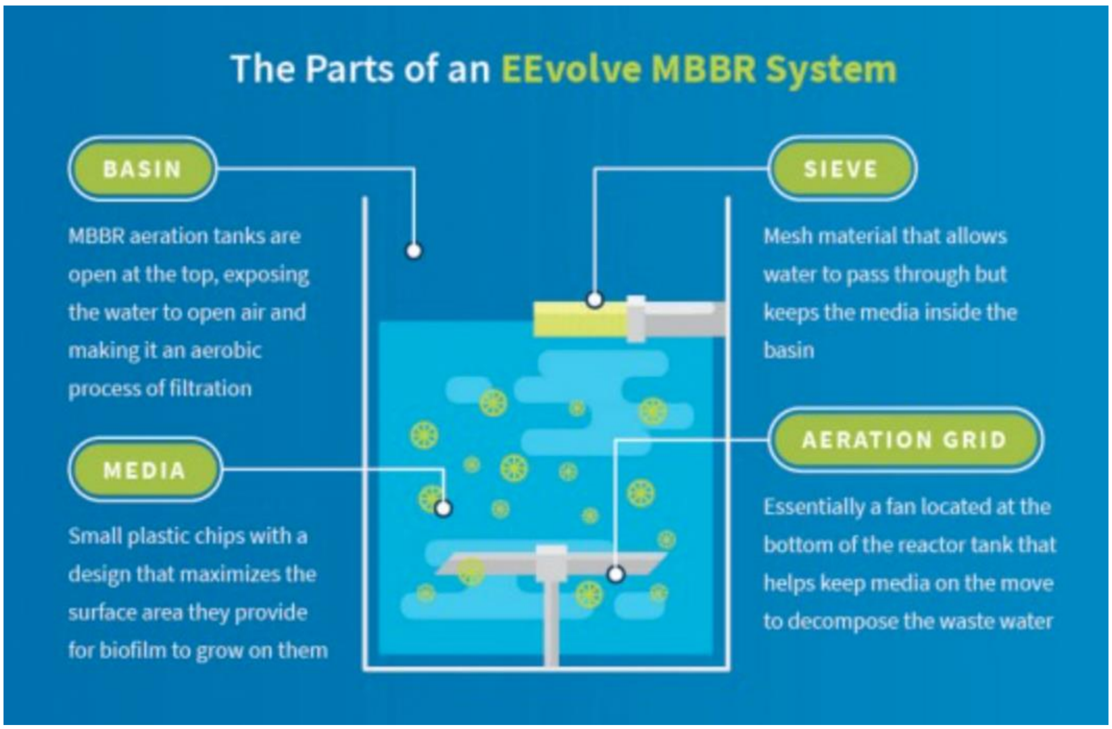

Figure 3- 3: MBBR system diagram [9]

The main advantages of MBBR systems are that they are very low maintenance and easy to operate, and they are able to handle variation in contaminant concentrations easily. Further, they have been found to be highly efficient, with a BOD and nitrogen removal retention time of only 3-4 hours for certain applications [9]. In industrial food and beverage processing applications where there are high BOD loading rates, MBBR has shown BOD removal rates anywhere from 65-80% [10]. The TKN removal efficiency was found to be 99% [10].

Another method evaluated for the pretreatment step was a sequencing batch reactor or a fill-and-draw system. In this system, wastewater is added to a single batch reactor and then treated to remove undesirable components, and then it is discharged. According to multiple case studies involving fill-and-draw installation, the size of the system can range anywhere from 18 x 12 feet to 104 x 90 feet. For example, a system that is 18 by 12 feet holds approximately 0.021 million gallons, whereas a larger system can hold approximately 1.56 MG [11]. Flow rates typically consist of 5 million gallons per day or less; larger flowrates are not recommended. Additionally, fill-and-draw systems are "very cost effective if treatment beyond biological treatment is required such as filtration [11]." It is also most cost effective when clarifiers or other equipment are eliminated from the process train. In relation to flowrates, cost is directly proportional to the design flowrate. Cost in relation to design flowrate is presented in Table 3-4.

Table 3- 1: Budget level equipment costs based on different flow rates [11].

Design Flowrate (MGD)	Budget Level Equipment Costs ($)
1	150,000 – 350.000
5	459,000 – 730,000
10	1,089,000 – 1,370,000
15	1,370,000 – 2,000,000
20	2,100,000 – 3,000,000

The costs above do not include the cost for the tanks, sitework, excavation, installation, contractor's overhead and profit, or legal, administrative contingency and engineering services. Sequencing batch reactors achieve good BOD and nutrient removal. The BOD removal efficiency is generally 85 to 95 percent [11]. TSS removal efficiency is approximately 92%. TKN removal efficiency is approximately 80-85%.

Due to the client's expressed interest in using the existing detention basin area on-site, the last biological treatment method researched was a constructed wetland. Constructed wetlands are engineered wetlands that simulate natural wetlands. There are various classifications of wetlands including surface flow, subsurface flow, and hybrid systems. Surface flow wetlands are not appropriate for Flagstaff's climate and will not be further discussed. Vertical subsurface systems are most viable in cold climates and its functionality is further discussed in relation to the case studies in Ontario, Canada and Minnesota.

Subsurface wetlands use elements from both horizontal (Figure 3-4) and vertical subsurface flow (Figure 3-5) systems. The class or design of the wetland should be determined based on the target contaminants, available space, plant selection, and quantity of water that will be treated. Due to Flagstaff's climate and frequent freezing temperatures in the winter, typical design parameters for cold-climate constructed wetlands would need to be employed. In terms of subsurface flow wetlands in the United States, there is a treatment zone and operating water depth of 2 feet (1 foot if there is no risk of freezing) [12]. The shallow operating water depth enhances oxygen transfer potential but also requires a larger surface area. If the bed were constructed at 2 feet due to the freezing potential, special operation to induce desirable root penetration to the bottom of the bed would be needed. Additionally, a subsurface wetland cannot be in use without preliminary treatment to reduce the concentration of easily degraded organic solids that would otherwise accumulate in the entry zone of the wetland system which would result in clogging, possible odors, and adverse impacts on the plants within the entry zone. Typically, these types of wetlands are used in colder environments because water is not exposed during the treatment process. This then minimizes energy loss through evaporation and convection.

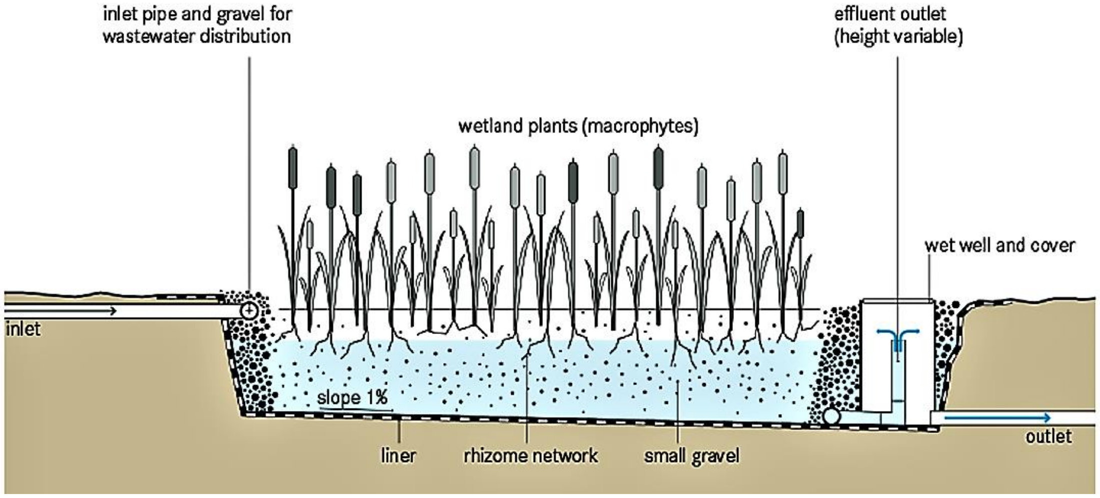

Figure 3- 4: Horizontal subsurface flow constructed wetland [13]

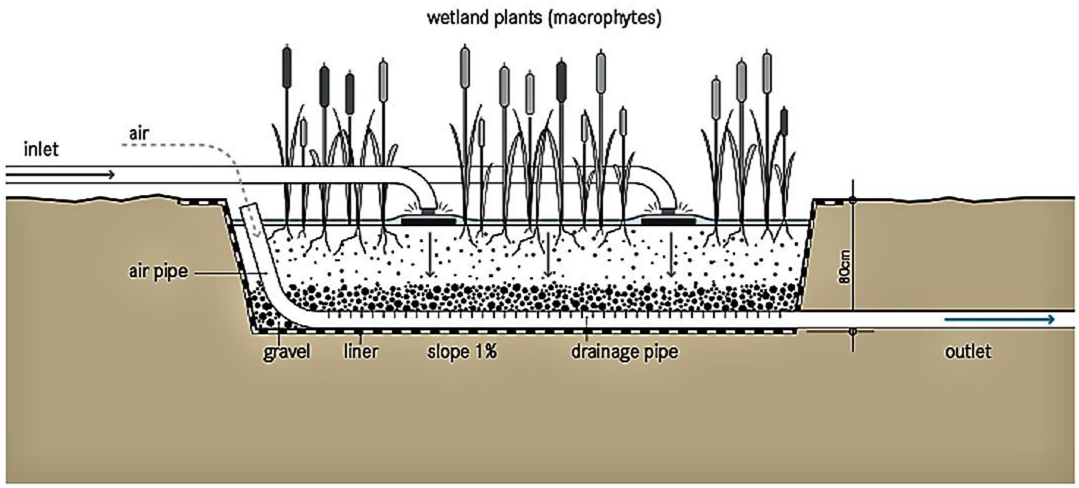

Figure 3- 5: Vertical subsurface flow constructed wetland [13]

Because of the snow, cold climate, and monsoon season in Flagstaff, it is vital to understand how these conditions can affect the functionality of a constructed wetland. In general, it has been found that TSS is not affected by cold weather [13]. This is because TSS removal is a physical process controlled by sedimentation and filtration, which can be done in all climates and isn't heavily impacted by temperature [13]. BOD and TKN treatment, however, are affected by weather and climate. Because these are both biological processes, the bacteria responsible for their removal are less active in cold conditions.

While it is generally accepted that BOD treatment does decrease at temperatures below 15C, the extent to which it decreases is variable. In high BOD loading wetlands, BOD treatment has been shown to decrease as much as 81% and as little as 6% at temperatures below 15C depending on several factors including type of flow, size of the wetland, duration of cold temperatures, and type of digestion (anaerobic or aerobic) [13]. Systems with the best rates of BOD treatment in cold temperatures show that the highest levels of soil bacteria are still active in cold months. Therefore, it would be favorable to design a system with porous media and subsurface flow that would allow for soil bacteria to treat BOD most effectively in cold months [13]. Additionally, plants and plant roots have been shown to not only provide surface area for bacteria but can also be beneficial in raising the temperature of the water during cold months. Maintaining a large and diverse plant population in the wetland can also be beneficial in increasing BOD treatment at cold temperatures [13].

Nitrogen removal is dependent on many biological processes, the most significant of which are plant uptake, nitrification and denitrification which all take place mainly in the root zones of wetlands [13]. Because of this, nitrogen removal, including total nitrogen (TN), ammonia, and TKN, are the most affected by cold temperatures of the three contaminants of concern. During the study of constructed wetlands worldwide, it was found the nitrification and denitrification processes were most effective at removing nitrogen between the temperatures of 25C and 35C [13]. This study reported that the average removal efficiency of TKN of constructed wetlands in temperatures below 15C was 58.5% and increases to 73.9% at temperatures above 15C [13]. Because of this large drop in efficiency between cold and warm temperatures, it is recommended that lower loading rates of TKN be applied to cold-weather wetlands. Additionally, having a variety of vegetation in the wetland will increase removal rates by providing more surface area for bacteria as well as increasing plant root uptake of nitrogen especially in summer months. HRT should also be increased in constructed wetlands designed for cold climates to aid in the removal of BOD, TKN and other nitrogen species [13]. In most cases, an HRT of 6-8 days was sufficient in achieving the maximum treatment for BOD and nitrogen, and no significant increases in treatment efficiencies were seen after 8 days [13].

Several case studies were investigated to determine the efficacy and logistical applications of a constructed wetland in Flagstaff. The first study examined is the vertical subsurface flow constructed wetland located in Ontario, Canada used to treat industrial wastewater from a winery and domestic wastewater from the onsite facilities at the winery. This constructed wetland was designed to treat 16,620 liters (4,390 gallons) of water per day using 4 cells of $101m^2$ each [14]. While the constituents of the wastewater from the winery are slightly different from those that are produced by Joy Cone Co., the untreated industrial wastewater stream is comparable in loading rates of the three contaminants of concern for this project. This study does not examine the efficiency of BOD treatment, but instead uses chemical oxygen demand (COD) to determine the constructed wetland's ability to treat organic wastes. The inlet concentrations of COD range from 500 to 45,000 mg/L produced by the winery [14]. TSS concentrations could reach up to 7,300 mg/L [14]. During this study, the treatment efficiencies of COD, TSS, total phosphorus (TP), TKN, ammonium and nitrate were calculated over a six-year period separated into two seasons each year. The warmest six months of the year were termed the growing season (GS) with average temperatures of 17.1C and the coldest six months of the year were termed the non-growing season (NGS) with an average temperature of 1.4C [14]. Each of the four cells in the wetland were lined with a PVC liner. 5-10mm of gravel and sand mix covered the liner, with 30cm of peat moss and sand mix placed above the gravel and sand layer. The water level in the wetland was maintained between 0.4 and 0.8m in depth and inoculated with various local plant species [14]. Water flow is primarily controlled by gravity except for the pump with moves water from an equalization area to the wetland inlet. Additionally, there is a pretreatment cell not included in the four previously mentioned cells. This cell is filled with gravel which can easily be cleaned and serves to remove excess TSS and organic material before it enters the wetland [14]. Over the six-year period, the team found that there was no significant decrease in the removal efficiency of organic matter in the constructed wetland between GS and NGS months. The average COD influent concentrations were 3043 and 2177 mg/L for the GS and NGS seasons respectively, and the average effluent concentrations were 6.0 and 14.8 mg/L with an average removal efficiency of 98.9% between the two seasons [14]. Additionally, there were no significant differences found between the TSS removal efficiencies between the GS and NGS months. This is an expected result because the physical mechanisms responsible for TSS treatment are not significantly altered by cold temperatures. The average influent TSS concentrations were 332 and 178 mg/L for the GS and NGS months respectively with effluent concentrations of 2.7 and 2.9 mg/L with an average efficiency of 97% [14]. Additionally, there was no clogging observed in the wetland which is attributed to the pretreatment cell reducing the amount of TSS in the influent wastewater stream [14]. The parameter most affected by the temperatures of the NGS was TKN. Average influent concentrations of TKN for the GS and NGSs were 92.2 and 13.9 mg/L respectively. Effluent concentrations were 0.45 and 0.04 mg/L with average removal efficiencies of 88% and 99% respectively. Although the efficiency is higher during the NGS, it should be noted that the influent concentrations of TKN were significantly lower during NGS months, and it is still thought that the cold temperatures do reduce the wetland's ability to remove TKN and other nitrogen species [14]. This wetland case study shows that cold-climate wetlands can adequately treat industrial wastes for organic matter, TSS, and TKN year-round. Emphasis should be placed on the flow regime of the wetland, and the authors of this study attributed much of the success of this wetland to the vertical subsurface flow pattern which helped keep the water from freezing and allowed for diverse microbial growth in the peat, gravel, and sand layers used in the root zone of the wetland. Additionally, pretreatment was very helpful in reducing loading rates and preventing clogging. Given the similarities between the industrial wastewater produced by this winery and the wastewater stream at Joy Cone Co., the design of this wetland provides guidance and assurance in the successful application of a similar constructed wetland design for Joy Cone Co.

Another case study relevant to a cold-climate wetland on Joy Cone's property includes the Lutsen Sea Villas case study. The Sea Villas was a subsurface flow constructed wetland that was built in 1997 to treat domestic wastewater from 27 town home units located on the north shore of Lake Superior in Lutsen, Minnesota [15]. Often the fall and early winter were mild, with no snow cover on the ground. However, on December 19, 1998, temperatures began to drop rapidly, reaching -28 degrees Celsius [15]. The system was insulated with 15 cm of mulch. A 5 cm air gap was present under the mulch insulation. Based on elapsed time meter readings on the pumps, the system was operating at 0.88 cm/day [15]. Mulch insulation is necessary when a constructed wetland must face extreme winter temperatures. In general, good mulch systems performed better than bad mulch systems. Ultimately, nitrogen removal was limited due to a failure to convert ammonia to nitrate [15]. It is apparent that cold climate subsurface flow constructed wetlands are effective in carbonaceous biological oxygen demand (CBOD) removal based on the operational history of full-sized systems in Minnesota. However, these same systems have not reduced Total Nitrogen in a consistent or logical way, even when sized at hydraulic loading rates that should have allowed nitrification to occur. In this specific case study, conclusions can be made about wetlands located in cold climates. Properly designed insulation of the wetland bed is effective in preventing freezing and resulting hydraulic failure. Additionally, relying on snow and ice cover does not provide reliable insulation during cold periods with limited snowpack [15]. Ultimately, the type of mulch insulation used can strongly affect the performance of the system. With this being said, only well decomposed organic materials can be used without degrading treatment efficiency [15]. In addition to this, the presence of a mulch layer will affect the type of vegetation used in the system. Plant species used in the wetland should tolerate the presence of an unsaturated root zone in the mulch layer. Often, properly designed cold climate insulated wetlands can achieve high levels of CBOD removal [15]. Treatment performance will improve after the first growing season. In order to achieve high levels of nitrogen removal, adequate oxygen must be available. Standard horizontal subsurface flow wetlands do not transfer enough oxygen to satisfy both the carbonaceous and nitrogenous oxygen demand in cold climates. Alternative wetland configurations that have higher levels of oxygen transfer are necessary for nitrogen removal. Temperatures below 4 degrees Celsius are not a barrier to nitrification, provided the wetland is designed to prevent freezing.

4.0 Facility Investigation

A preliminary site visit was conducted on September 22, 2022, to examine the current production process and water treatment system at the Joy Cone Co. factory. Plant engineer, Lane Fisher, provided documentation detailing the layout of the facility and surrounding area as well as the ice cream cone production process. During the tour of the facility, Honeycomb Engineering documented the flow of city water and wastewater throughout the facility. Figure 4-1 below shows a flow diagram of the industrial processes taking place at Joy Cone Co. and the water and waste streams resulting from these industrial processes. The items outlined in black show how the ingredients and batter flow through the facility eventually creating the final product. City water and cleaning chemicals are also used in the equipment during washing and maintenance periods. These are conducted routinely and between new batter flavors or types. The cleaning solutions are used in combination with city water, and water is flushed through the equipment after the cleaning chemicals to rinse the machines. Solid waste is produced both by the rotary drum vacuum filter from the removal of suspended solids and in the trimming process of the ice cream cones. These solids are stored in an outdoor dumpster and are then transported off site.

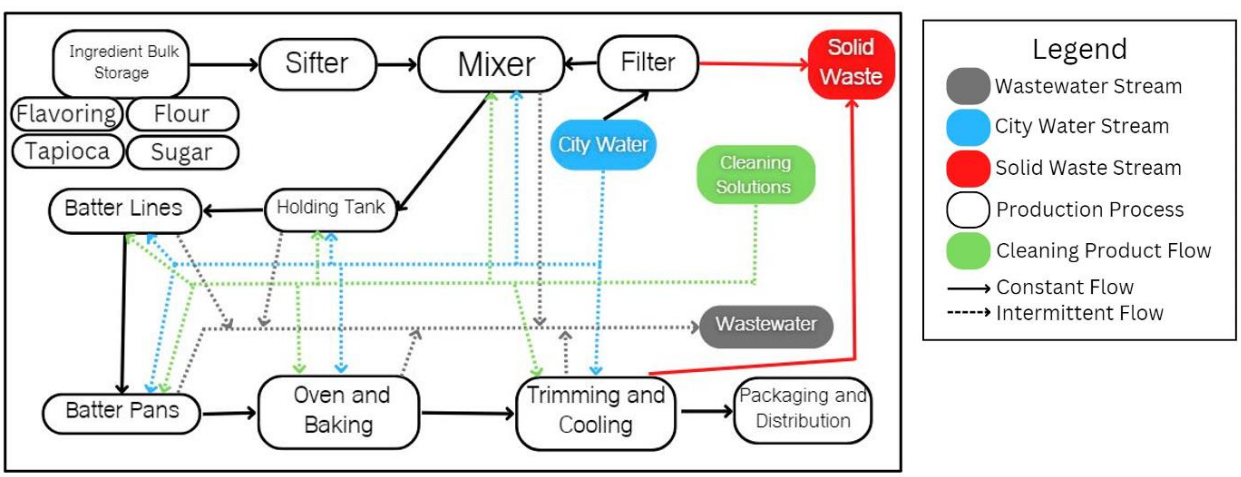

Figure 4- 1: Process flow diagram

It should be noted that the ice cream cone production process is simplified in the above diagram. Dashed arrows represent intermittent flows while solid arrows are constant. Most of the wastewater produced by the Joy Cone Co. factory originates from the cleaning process. After each batch of ice cream cones is produced, the system is flushed to maintain the integrity of the equipment and quality of the product.

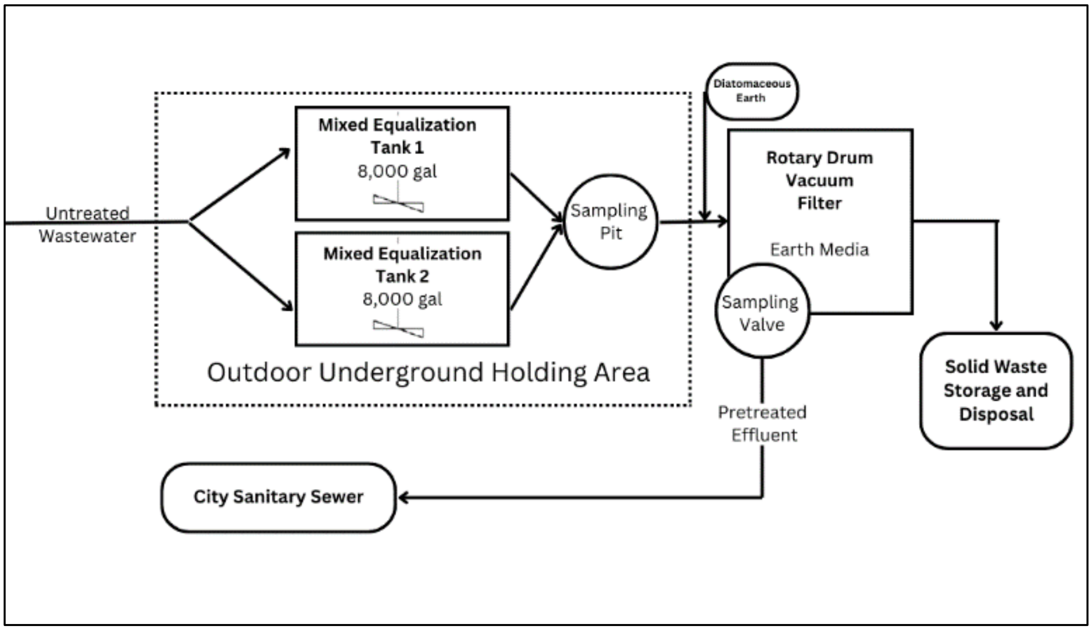

Figure 4- 2: Pretreatment process flow diagram

On February 15th, 2023, Honeycomb Engineering Inc. returned to the facility to learn more about the specifics of the current treatment system. Figure 4-2 below shows a process flow diagram of the pretreatment taking place at the facility at the time of the investigation.

There are two primary mechanisms used to store and treat wastewater at Joy Cone Co.. The first mechanism consists of two 8,000-gallon mixed, outdoor equalization tanks which maintain consistent concentrations of contaminants and store the water throughout the day. These tanks are emptied 1-2 times per day. This equipment is not run on a regular cycle and is utilized only when the holding tanks are reaching capacity. The untreated water is mixed with diatomaceous earth and pumped from the equalization tanks to the rotary drum vacuum filter located inside the water treatment room at the facility. The filter removes TSS and grease which is separated from the filter. A conveyer is used to transport the solid waste to an outdoor dumpster for disposal. An image of the rotary drum vacuum filter is presented in Figure 1-3.

The treated effluent is discharged directly to the City sanitary sewer. A plan view of the treatment room and equalization tanks can be seen in Figure 4-3 below.

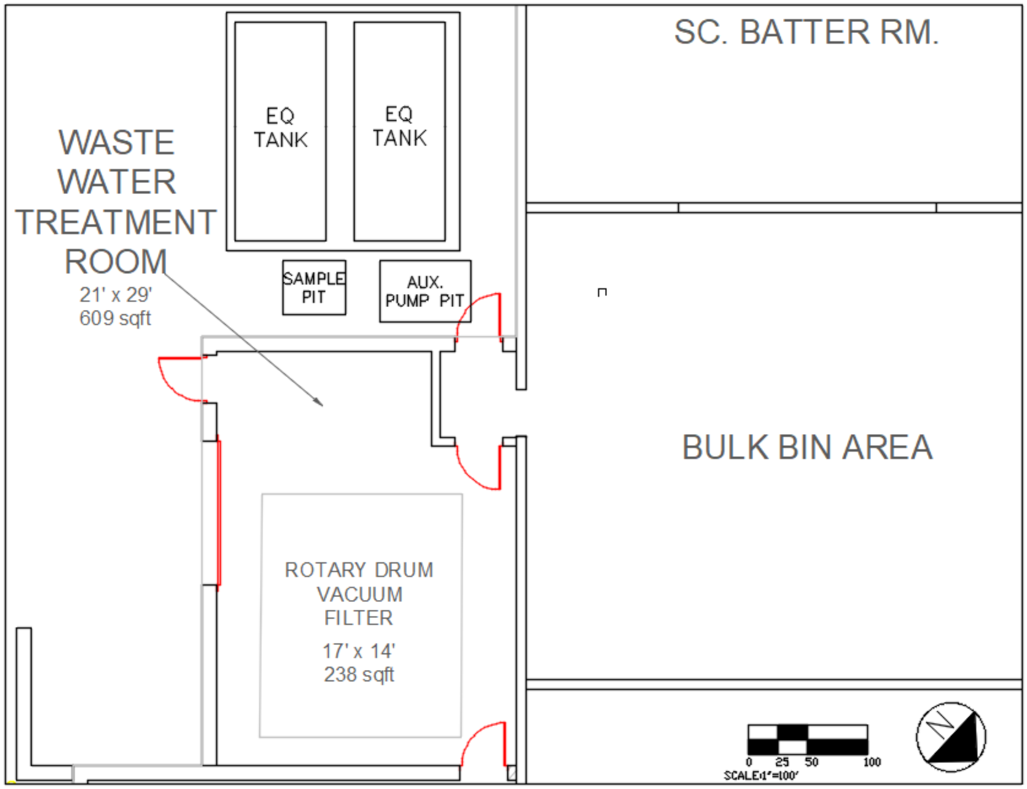

Figure 4- 3: Plan view of current treatment room at Joy Cone

The facility investigation also included a visual examination of the detention area currently used for stormwater runoff. Mr. Fisher noted that it would be desirable to use this 1.46-acre area in the design of a new treatment system because it would free up the underground storage tanks and the indoor treatment area for other purposes and could reduce the operating and maintenance costs of the water treatment system. The area included a shallow drainage ditch leading from the parking lot on the west side of the facility to the detention basin between the Joy Cone Co. factory and Interstate-17. Mr. Fisher indicated that the basin is usually dry. Water is typically only present in the basin during heavy storms and usually empties within hours after a storm event. Figure 4-4 below show the basin photographed from the southwest side.

Figure 4- 4: Detention basin area

5.0 Sampling, Analytical Testing, and Data Analysis

5.1 Sample Collection and Laboratory Testing

Sampling was conducted on February 15, 2023, from 8-11 am at the Joy Cone Co. Factory. Team members Gabrielle LeBlanc, Rachael Haneysmith, and Megan Eisenach collected approximately six half-liters of post-treatment effluent using a grab-sampling method. The team was instructed to collect their grab sample 45 minutes to an hour after the treatment process began to collect effluent that would best represent the average concentrations of the post-treatment effluent. Unfortunately, the team was unable to collect samples from before the pretreatment process due to weather constraints. With the assistance of Mr. Alex Mooneyham, the three total liters of sample were collected following the protocol and safety requirements of the Joy Cone Co. These samples were tested for TSS on February 16, 2023, by Gabriella Sandhu and Gabrielle LeBlanc and for TKN on February 17, 2023, by Rachael Haneysmith and Megan Eisenach. A sample to test for BOD was collected by Joy Cone Co. and sent to Inner Basin Environmental Laboratory (Inner Basin), Joy Cone Co.'s contract laboratory. The Honeycomb Engineering team obtained these BOD results from Inner Basin. The samples were tested using the ASTM D2329 Method of Test for Biochemical

Oxygen Demand of Industrial Water and Industrial Wastewater, ASTM D5907-18 Standard Test Methods for Filterable Matter (Total Dissolved Solids) and Nonfilterable Matter (Total Suspended Solids) in Water, and HACH 10242 Simplified Spectrophotometric Measurement of Total Kjeldahl Nitrogen in Water and Wastewater. Results were obtained for the pretreated effluent concentrations of TSS and TKN using the NAU EnE laboratories and are displayed in comparison to the Inner Basin lab results in Table 5-1.

Table 5- 1: Honeycomb testing results compared to Inner Basin results.

	Honeycomb Testing Results		**Inner Basin Testing Results**
Contaminant	n	Average Concentration (mg/L)	Concentration (mg/L)
TSS	7	306 ± 50	634
TKN	5	340 ± 49	643
BOD	-	-	31380

5.2 Data Analysis

Joy Cone Co. also provided 2022 testing data that the team analyzed to increase the reliability of the data that was used in the design. Figures 5-2 through 5-5 show the concentrations and mass flow rates of TKN, BOD, and TSS over 2022.

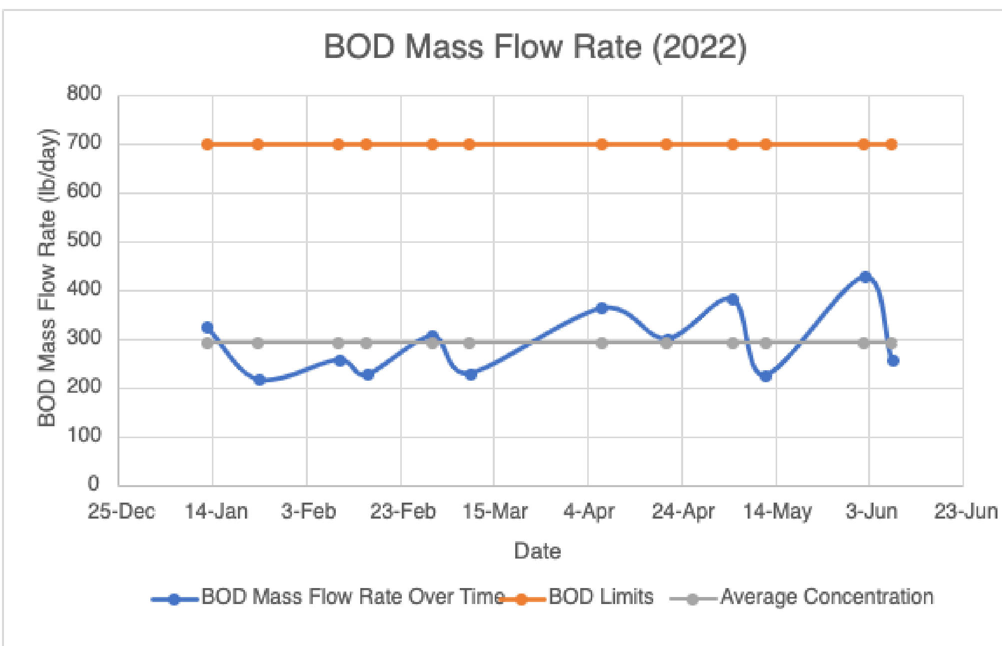

Figure 5- 2: BOD mass flow rate

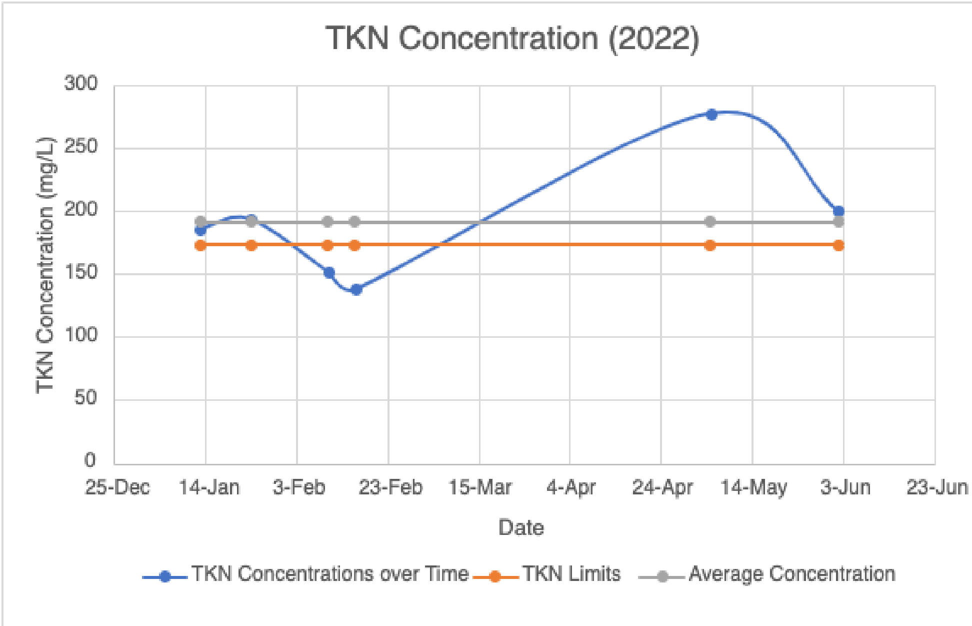

Figure 5- 3: TKN concentration (2022)

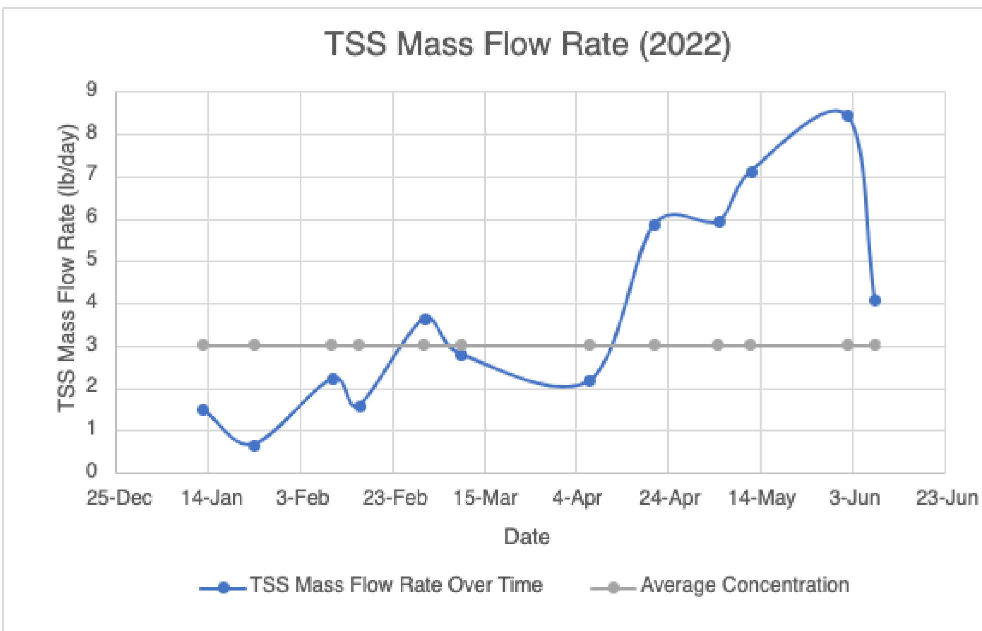

Figure 5- 4: TSS mass flow rate (2022)

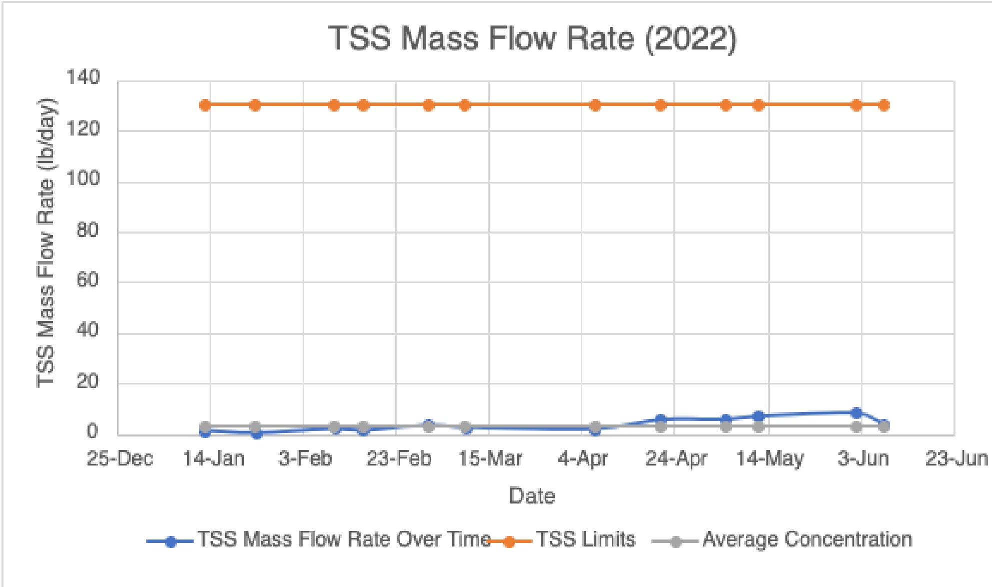

Figure 5- 5: TSS mass flow rate (2022)

These graphs help show how the concentration or mass flow rates of the contaminants change over time and compare to the average and permit levels. These data can be seen in Table 5-2 below.

Table 5- 2: Average concentrations vs. permit levels

Contaminant	n	Average Concentrations		Permit Levels
		(mg/L)	(lb/day)	
BOD	12	23440 ± 5335	293 ± 5	700 lb/day
TKN	6	191 ± 45	2 ± 0.04	173 mg/L
TSS	12	307 ±189	3 ± 0.2	130 lb/day

Because these data are more consistent and reliable, the team decided to use the 2022 data for the design assumptions instead of the data obtained from the laboratory testing performed by the Honeycomb team. This will be discussed in further detail in section 7.1. It is important to note here that the TSS and BOD values are already well below the permit limits. Although the main focus of the design is to treat TKN to the permit levels, the BOD and TSS could pose a problem if a wetland design is implemented. The current wastewater concentrations of TSS and BOD are too high and will overload the wetland if applied directly, so it is likely that these contaminants will also need to be treated prior to introduction to a wetland, although they are already in compliance with the pretreatment permit.

6.0 Hydrological Analysis

6.1 Topographic Maps

Due to severe winter weather, Honeycomb Engineering, Inc. was unable to survey the site. Topographic maps of the factory and the surrounding area were obtained from Mr. Mark Lamer, a professor at Northern Arizona University. The map including data for the detention basin was copied into Autodesk® Civil 3D and used to create a usable surface to complete the rest of the hydrological analysis tasks. Using this data, the volume of the detention basin was approximated as 810,000 cubic feet.

6.2 Watershed Delineation

Using USGS for topographic maps and Geographic Information System (GIS), the team was able to determine the high points of the watershed surrounding the factory to perform a watershed delineation. This delineation was used to determine the peak flow and retention volume of the entire watershed to estimate the amount of water that the existing detention basin experiences purely from rain and snowfall events. Any design using the detention basin will have to account for the runoff from the area along with the influent. Figure 6-1 below shows the watershed of the area surrounding the Joy Cone Co. factory that feeds into the detention basin. Green areas are developed land (21.95 acres) and blue areas are undeveloped land (20.1 acres). This delineation was created in ArcGIS using LiDAR data provided by Coconino County. The longest flow path was determined to be flow path 1 with a total length of 0.302 miles. It should be noted that the area north of Shamrell Boulevard was not included in the watershed due to the storm drains located along the road.

Figure 6- 2: Watershed delineation

6.3 Rainfall Data

Rainfall intensity (inches/hour) is used to determine the peak flow, time of concentration, and the required storage volume for detention and retention facilities. The City of Flagstaff Stormwater Management Design Manual (2009) requires that the 2, 10, and 100-year storm events be used to determine these parameters [16].

Table 6-1: Rainfall data [16]

Duration	Frequency, In Years					
	2	5	10	25	50	100
5-min.	3.96	5.04	5.76	6.84	7.68	8.52
10-min.	3.06	3.90	4.50	5.34	6.00	6.66
15-min.	2.48	3.20	3.48	4.40	4.92	5.48
30-min.	1.58	2.06	2.40	2.86	3.22	3.58
1-hour	0.95	1.25	1.46	1.76	1.98	2.21
2-hour	0.56	0.73	0.85	1.02	1.15	1.28
3-hour	0.41	0.53	0.62	0.74	0.83	0.92
6-hour	0.24	0.31	0.36	0.43	0.48	0.53
12-hour	0.14	0.19	0.21	0.26	0.29	0.32
24-hour	0.08	0.11	0.12	0.15	0.17	0.19

It is assumed that if the volume of the detention facility is large enough for all three storm events, the volume will be satisfactory for all intermediate storms. The manual also indicates that rainfall data can be collected from NOAA Atlas 14, however, the same data is also provided in the manual. The following table (Table 6-1) shows the rainfall data produced by NOAA Atlas 14 and provided by the City of Flagstaff which is adequate for use in the calculation of detention volumes for all locations within the City of Flagstaff [16].

6.4 Time of Concentration (TOC)

The ADOT rational method tool was used to determine the time of concentration for the surrounding watershed. The time of concentration is 10 minutes with regards to the City's minimum standard. The watershed was considered as 20 acres of undeveloped land to account for the forested area and 22 acres of developed land, to account for the factory roof and parking lot. The NOAA Atlas 14 data found previously was imported into the tool, and GIS was used to determine the length of the longest flow path, which was found to be 0.3 miles (indicated on Figure 6-1). GIS was also used to determine the elevation difference between each end of the longest flow path of 64 feet. These two values were then used to find the slope of watershed. The slope was found to be 211.92 ft/mile. This datum was then input into the ADOT Rational Method Tool and parameters including discharge, rational coefficient, rainfall intensity, area, and computed time of concentration as well as applied, were found using the ADOT tool. The equation for time of concentration is displayed below.

Equation 6- 1: Time of concentration [17]

$$Tc = 11.4L^{0.5}{K_b}^{0.52}S^{-0.31}i^{-0.38}$$

Where:

Tc = time of concentration, in hours
S = watercourse slope, in ft/mile
K_b= watershed resistance coefficient
i = average rainfall intensity for a duration of rainfall equal to Tc (in/hr)
L = length of the watercourse to the hydraulically most distant point, in miles

6.5 Peak Flow and Retention Volume

With the ADOT Rational Method tool data, the peak flows for the 2-year, 10-year, and 100-year storms were determined to be 57.7 cubic feet per seconds (cfs), 96 cfs, and 167.8 cfs respectively. The peak flow was found by using the Triangular Hydrograph Method.

Equation 6- 2: Peak flow rate of runoff

$$Q = CiA$$

Where:

Q: estimation of peak rate of runoff (cfs) for a recurrence interval
C: runoff coefficient, surface runoff from contributing drainage area, dimensionless decimal fraction
i: average rainfall intensity (in/hr) for some reoccurrence interval
A: contributing drainage area (acres) to the point of the design that produces the maximum peak rate of runoff

Then, the design storm storage volume was found using the triangular hydrograph volume estimation equation below.

Equation 6- 3: Triangular Hydrograph Volume Estimator

$$V_S = 0.5 \times T_i \times (Q_i - Q_0)$$

Where:

V_s: required storage volume (cubic feet)
T_i: factor of time of concentration, 1.78 times the Tc value (minutes)
Q_i: total post-development flow rate (cfs)
Q_0: total pre-development flow rate (cfs)

T_i can be estimated by multiplying the time of concentration for the watershed (in seconds) by 1.78 the Tc value as shown by research for small, urbanized watersheds. This is because the longevity of the storm will contribute to greater volumes of runoff [18]. The required storage volume was determined to be approximately 172,000 cubic feet.

7.0 Development of Design Alternatives

Although the TSS and BOD levels are already below the permit requirement, in order to be discharged to the wetland, these concentrations must be reduced to avoid overloading in the wetland. Therefore, the design strategy is to first reduce TSS and BOD prior to treatment in a constructed wetland. As to not limit the final design at this stage of the project, the TSS and BOD pretreatment and wetland alternatives were evaluated separately to find the best alternative for each phase of the treatment process. The final design will combine the best of the two phases. Phase I will consist of a treatment which targets BOD and TSS while Phase II (wetland phase) will primarily target TKN although some BOD treatment will also occur.

7.1 Design Assumptions

Several assumptions were made regarding the nature of the industrial wastewater stream produced by Joy Cone Co. These assumptions are described below.

The influent rate of industrial wastewater is 1,500 gallons per day. This assumption was made based on the 500,000 gallons of wastewater produced annually by Joy Cone Co. Although this number will be variable, this is a conservative average flow rate the team will use to move forward with the design. The pretreated effluent concentrations of TSS, BOD, and TKN are the same as the influent concentrations of untreated wastewater. Because of the lack of reliable data collected regarding the untreated wastewater, it is best to use the pretreated effluent concentrations in the creation of the final design. It is likely that the vacuum drum rotary filter does treat some of the TSS, however, the efficiency of this aged unit is unknown. It does not efficiently treat TKN and BOD. To be conservative, the team has assumed that it is likely these concentrations are slightly higher in the untreated wastewater, and larger values will be used in final calculations to ensure that the final design will be able to treat the contaminants produced by Joy Cone Co. Table 7-1 shows the minimum influent contaminant levels in mg/L, mg/gal, and lb/day. These values are based on the 1,500 gallons per day flow and the average reported values of contaminants in the pretreated effluent by Joy Cone Co.

Table 7- 1: Average concentrations vs. design concentrations

Contaminant	Permit Levels	Average Concentrations		Design Concentration		Notes
		(mg/L)	(lb/day)	(mg/L)	(lb/day)	
BOD	700 lb/day	23440	293	30000	376	Design concentration average of highest reported values
TKN	173 mg/L	191	2	210	3	Design concentration 10% increase from average value
TSS	130 lb/day	307	3	337	4	Design concentration 10% increase from average value

The effluent concentration limitations that need to be reached will remain the same as the current Industrial Pretreatment Permit regulations for BOD, TSS, and TKN. Due to the nature of the AZPDES permit, the required effluent concentrations for direct discharge cannot be determined without a review by ADEQ. Therefore, any design will include discharge to the sanitary sewer so the factory can continue operating under their same permit.

7.2 Phase I Design Alternatives

Alternative A for Phase I is a fill and draw system using the two 8000 gal holding tanks that are already on site. This treatment will make use of the agitator arms that are currently in place in the tanks. Wastewater will be pumped to one tank at a time using the same infrastructure that is currently in place at the factory. The fill and draw process maximizes tank use by acting as a sequencing batch reactor; while one tank is filling, the other is actively treating effluent. Once one tank has reached the capacity of 6000 gallons, the agitation process will begin, followed by settling. Each tank will undergo 12 hours agitation, followed by 12 hours settling. This alternative involves no other pretreatment before discharge to the wetland, other than the dewatering of the sludge using a plate and frame filter press. Sludge handling is the same as the current system.

Alternative B involves replacing the current rotary drum filter and pairing it with a moving bed biological reactor. As previously mentioned, the rotary drum filter that is currently in use at the factory was installed in 2002 and has become inefficient. A new filter using perlite as a filter medium is recommended. This filter requires no extra treatment of solids due to the vacuuming properties of the filter, which draws extra water out of the sludge and transforms it to a cake. With this system, Joy Cone Co. can continue to dispose of their sludge using the same system that they currently use. The effluent from the rotary drum filter would be sent to the moving bed biological reactor.

Alternative C is the same fill and draw method as described in the first alternative, paired with a new rotary drum filter.

7.3 Phase II Design Alternatives

Phase I effluent will be discharged to the eastern portion of the detention basin located on site. The western portion of the basin will be used for stormwater runoff. These two areas will not have any mixed or combined flows. All design alternatives for Phase II include the use of a geomembrane liner intended to prevent infiltration to the soil and maintain the integrity of the wetland itself as well as the surrounding natural area. Each alternative will use a subsurface flow regime in which the water will flow through various layers of sand and gravel mixtures where the contaminants will be treated primarily by biological mechanisms. The choice to narrow the design alternatives to subsurface flow and exclude free surface flow was made due to the increased treatment efficiencies in cold climates of subsurface flow wetlands. Subsurface flow wetlands have loading rates based both on concentration and mass of contaminant. These values can be seen in Table 7-2.

Table 7- 2: Subsurface Wetland Loading Rates [19]

Constituent	Typical Influent Concentration mg/L	Target Effluent Concentration mg/L	Mass Loading Rate lb/ac/d*
Hydraulic Load (in./d)	3 to 12**		
BOD	30 to175	10 to 30	60 to 140
TSS	30 to150	10 to 30	40 to 150
NH_3/NH_4 as N	2 to 35	1 to 10	1 to 10
NO_3 as N	2 to 10	1 to 10	3 to 12
TN	2 to 40	1 to 10	3 to 11
TP	1 to 10	0.5 to 3	1 to 4

Due to the similarities in loading rates, both alternatives have similar area requirements. Effluent recirculation will also be practiced in each design to lower the concentration loading rate and increase the retention of microorganisms within the wetland. Wetland effluent will be discharged to the public sanitary sewer after treatment in the wetland. With these parameters in mind, two design alternatives were developed for the constructed wetland.

Alternative A includes the use of a vertical subsurface flow constructed wetland (VSSFCW). This design alternative will include the use of coarse, medium, and fine gravel with a top layer of sand. The design will also include an aeration pipe to allow for increased oxygen to aid in the BOD and TKN treatments. In this flow regime, the wastewater will be applied at the surface of the wetland through a distribution pipe system. As the water exits the pipe along various locations across the wetland, the water will percolate through these layers in a primarily vertical flow pattern. A slope of less than 1% will be applied to encourage the water to flow horizontally at a low rate eventually reaching the outlet. Additional pipe networks can be placed at the bottom of the wetland to aid in the transport of water from the surface application (inlet) to the outlet. Plants will be inoculated throughout the wetland to increase surface area for microbial biological treatment and the uptake of nitrogen from the water. One advantage of this design is that loading can be applied intermittently or continuously. Traditionally, VSSFCWs are operated as a batch process to increase the HRT and ensure that the vertical flow is maintained, however, it is also possible to operate these wetlands using continuous flow provided that the flow rate is compatible with the design. If the flow is continuous, plug flow can be assumed in which concentrations of contaminants are highest at the surface of the wetland and lowest at the bottom. In batch processes, the wetland can be flooded to capacity and drained afterward. In this scenario, concentrations of contaminants can be assumed uniform throughout the wetland but decrease with time. The costs of VSSFCWs are mainly dependent on the size of the wetland, however, compared to other wetland types, VSSFCWs typically have a slightly higher installation and operating cost due to the continuous aeration, deep substrate beds, and additional equipment such as pipes and pumps which are not required, or required in lesser quantity, in other wetland types.

Alternative B includes the design of a horizontal subsurface flow constructed wetland (HSSFCW). The main difference between a HSSFCW and a VSSFCW is the amount of oxygen in the system. While vertical subsurface flow uses aeration and has an oxygen rich environment, horizontal flow systems are typically considered anoxic, meaning the oxygen in the water is low, but not zero. HSSFCW relies entirely on gravity to move the water from the inlet to outlet utilizing a slope of 0.5-2%. The flow is primarily horizontal flowing through a bed of coarse, medium, and fine gravel with a top layer of sand. This is very similar to that of a VSSFCW; however, the depths of each layer are significantly smaller since the treatment will be occurring along the length of the wetland from inlet to outlet versus through the depth of substrate in vertical flow. HSSFCWs are operated as a continuous plug flow reactor in which the concentrations of contaminants are highest at the inlet and lowest at the outlet. Water should be applied continuously to ensure the health of the microorganisms within the wetland.

8.0 Selection of Preferred Alternative

8.1 Creation of a Decision Matrix

The first step in selecting the best design alternative is to evaluate the important criteria and constraints for the design. Table 8-1 shows the decision matrix used to evaluate the treatment options for Phase I. Table 8-2 shows the Phase I scoring system. Table 8-3 shows the decision matrix used to evaluate the treatment options for Phase II. Table 8-3 shows the Phase II scoring system. The scoring criteria were chosen based on the project objectives and client input. The weights given to each criterion were chosen by the team and represent the importance of each criterion. The final score of each design was a weighted average of each individual score given to each criterion. The highest score a design can receive is a 3. This gives the team an efficient way to determine the best design alternative of all possible designs.

Table 8- 1: Phase I Decision Matrix

Phase I Decision Matrix				
Criteria	**Weight of Criteria (%)**	**Option A: Fill and Draw Alone**	**Option B: Rotary Drum Filter with Moving Bed Biological Reactor**	**Option C: Fill and Draw with Rotary Drum Filter**
Treatment Efficiency	40%			
Footprint (area required)	15%			
Total Costs (capital and maintenance)	25%			
Maintenance and Operation	20%			
Total Score	100%			

Table 8- 2: Phase I Scoring System.

Phase I Decision Matrix Scoring System			
Criteria	1	2	3
Treatment Efficiency	At least 80% removal efficiency of only 1 contaminant	At least 80% removal efficiency for 2 contaminants	At least 80% removal efficiency for all 3 contaminants
Footprint (area required)	Requires more than 400sqft of indoor space	Requires between 300 and 399sqft of indoor space	Requires less than 300sqft of indoor space
Total Costs (capital and maintenance)	Of the three alternatives, this is the costliest design	Of the three alternatives, this is the second most costly design	Of the three alternatives, this is the most cost-effective design
Maintenance Required	Requires one or more full time operators AND OR difficult sludge/residual disposal	Requires daily maintenance or operation between 6 and 8 hours AND OR moderately difficult sludge/residual disposal	Requires less than 6 hours of daily maintenance or operation AND easily disposed of sludge/residuals

Table 8- 3: Phase II Decision Matrix

Phase II Decision Matrix			
Criteria	**Weight of Criteria (%)**	**Option A: VSSF**	**Option B: HSSF**
TKN Treatment Efficiency	30%		
BOD Treatment Efficiency	25%		
TSS Treatment Efficiency	20%		
Costs (capital and maintenance)	15%		
Maintenance Required	10%		
Total	100%		

Table 8- 4: Phase II Scoring System.

Phase II Decision Matrix Scoring System			
Criteria	1	2	3
TKN Treatment Efficiency	Below 65%	Between 65-75%	Above 75%
BOD Treatment Efficiency	Below 70%	Between 70-80%	Above 80%
TSS Treatment Efficiency	Below 75%	Between 75-89%	Above 90%
Costs (capital and maintenance)	Most expensive	Both alternatives are similar	Least expensive
Maintenance Required	Most maintenance	Both alternatives are similar	Least maintenance

8.2 Evaluation of Design Alternatives

Using the decision matrices, a preferred design alternative was selected for each phase. Table 8-5 shows the completed decision matrix for Phase 1. The scoring of each criterion was done using the system developed for each phase, and Table 8-6 details the justification for each score. Table 8-7 shows the completed decision matrix for Phase 2, and Table 8-8 shows the justification for each score.

Table 8- 5: Phase I Scoring

Phase I Decision Matrix				
Criteria	**Weight of Criteria (%)**	**Option A_1: Fill and Draw Alone**	**Option B_1: Rotary Drum Filter with Moving Bed Biological Reactor**	**Option C_1: Fill and Draw with Rotary Drum Filter**
Treatment Efficiency	40%	2	3	3
Footprint (area required)	15%	3	1	2
Total Costs (capital and maintenance)	25%	3	1	2
Maintenance and Operation	20%	3	1	2
Total Score	100%	2.6	1.8	2.4

Table 8- 6: Phase I Scoring Justifications

Phase I Scoring				
Alternative	**Treatment Efficiency**	**Footprint (area required)**	**Total Costs**	**Maintenance Required**
A_1: Draw and Fill Alone	BOD: 85-95% TKN: less than 80% TSS: 92%	All treatment occurring outdoors with indoor plate and frame filter press (for sludge) requiring less than 200sqft of indoor area	Least costly	Requires daily operation to mix and settle tanks, pump water from tanks to wetland, plate and frame filter press required for sludge treatment and disposal (weekly)
B_1: Rotary Drum Filter with Moving Bed Biological Reactor	BOD: 85% TSS: 99% TKN: 80%	Rotary Drum: 100-300sqft Moving Bed Bioreactor: 100-300sqft Total: 200-600sqft Average: 400sqft	Most costly	Requires full time operator to manage batch operations, replacing and disposing of used media can be costly and difficult
C_1: Fill and Draw with Rotary Drum Filter	BOD: above 95% TSS: above 95% TKN: 80%	100-300sqft	Medium cost	Same maintenance and operation as alternative A in addition to running rotary drum vacuum filter batch process, disposing of both sludge (from fill and draw) and solids (from rotary drum vacuum filter)

Table 8- 7: Phase II Scoring

Phase II Decision Matrix			
Criteria	**Weight of Criteria (%)**	**Option A_2: VSSF**	**Option B_2: HSSF**
TKN Treatment Efficiency	30%	3	1
BOD Treatment Efficiency	25%	3	2
TSS Treatment Efficiency	20%	2	2
Costs (capital and maintenance)	15%	1	3
Maintenance Required	10%	1	3
Total	100%	2.30	1.95

Table 8- 8: Phase II Scoring Justification

Phase II Scoring					
Alternative	**TKN Treatment Efficiency**	**BOD Treatment Efficiency**	**TSS Treatment Efficiency**	**Costs (capital and maintenance)**	**Maintenance Required**
A_2: VSFF	65-90% (Avg: 77.5%)	84%	80-95% (Avg: 87.5%)	Some operation and maintenance costs for pipe and pump network, aeration, and replacement of substrate media, deeper substrate than HSFF can be more costly	Batch processes require more maintenance than continuous flow systems, but can be reduced using a timer or automated system, same cleaning requirements as HSSF (inlet/outlet structures and plant debris) however, due to the aeration, there is less sludge accumulation than with HSFF
B_2: HSFF	58-70% (Avg: 64%)	77%	80-95% (Avg: 87.5%)	Very low operation and maintenance costs, lower capital costs than VSSF due to less required equipment, no aeration or pumps required	Requires less maintenance than VSSF, continuous flow, no aeration/pumps required, requires removal of plants and cleaning of inlet/outlet structures, sludge may need to be removed every few years

Alternative A_1(fill and draw) was chosen for Phase I and Alternative A_2 (VSSF) was chosen for Phase II.

9.0 Final Design

9.1 Completed Final Design

The final design for Phase I uses the two existing 8,000 gallon tanks on site to create a fill and draw system, as well as one new above ground tank with the same capacity of 8,000 gallons as longer hydraulic retention times were required to adequately reduce contaminant concentrations. The new above ground tank will be 13.5 feet long, 10 feet wide, and 8 feet deep.Wastewater will enter one of the existing EQ tanks and will continue to fill the tank to an approximate wastewater volume of 6,250 gallons. The treatment period will occur over 7 days, with 6.8 days of agitation and around 5 hours of settling. As soon as the 7 day treatment period begins in one tank, the second tank will be filling with the next volume of wastwater to be treated. Similarly, the third tank will begin to fill as soon as the second tank begins treating. Each tank will be equipped with a centrifugal pump with at least a 25 gpm pump capacity in order to reach a four hour pump out time goal. The design parameters for the three tanks are shown in Table 9-1 below.

Table 9- 1: Design parameters for fill and draw tanks.

Design Parameters for Tanks	
Flow (gpd)	1,500
Influent BOD (mg/L)	30,000
Influent TSS (mg/L)	307
% BOD Removal	90
% TSS Removal	80
Effluent BOD (mg/L)	140
Effluent TSS (mg/L)	27
Hydraulic Retention Time (days)	7

The removal efficiencies stated in Table 9-1 are averages of BOD and TSS removal efficiencies that are typical for sequencing batch reactor (SBR) systems. Due to the lack of data required for modeling the system, typical values were used for many of the unknown parameters required to calculate for hydraulic retention time, and were manipulated accordingly and reasonably.

Equation 9-1 below was used to calculate for the hydraulic retention time [20].

Equation 9- 1: Hydraulic retention time

$$HRT = \frac{\theta_c Y(Influent\ BOD - Effluent\ BOD)}{X\ MLVSS(1 + k_d\theta_c)}$$

Where:

> θ_c: mean cell residence time (days) (see Equation 9-2)
> Y: yield coefficient (mg VSS/mg BOD_5)
> X MLVSS: volatile suspended solids concentration (mg VSS/L)
> k_d: growth constant ($days^{-1}$)

Typical parameters for the growth constant and yield coefficient were used as 0.05 day^{-1} and 0.1 mg VSS/mg BOD_5 respectively, and a volatile suspended solids (VSS) to TSS fraction of one was used, meaning the X MLVSS was modeled as equal to the total suspended solids concentration [20]. A fraction higher than one is not possible, while a fraction any lower than one would produce a lower hydraulic retention time. In order to be conservative, a fraction of one was chosen to produce the highest possible retention time value. Influent BOD is the design BOD value, where the effluent BOD was chosen to be the highest concentration that can typically be discharged to a wetland (140 mg/L) in order to minimize hydraulic retention time while still being as conservative as possible.

Equation 9-2 below is used to calculate mean cell residence time, which is an important factor in the hydraulic retention time equation.

Equation 9- 2: Mean cell residence time

$$\theta_c = \frac{K_s + Effluent\ BOD}{Effluent\ BOD \cdot \mu_M - Effluent\ BOD \cdot k_d - K_s \cdot k_d}$$

Where:

K_s: growth constant (mg/L BOD_5)
μ_M: growth constant ($days^{-1}$)
All other variables consistent with what has been decribed previously.
A typical K_s value of 100 mg/L BOD_5 was chosen along with a typical μ_M value of 2.2 $days^{-1}$.

These two equations provide a hydraulic retention time of 6.8 days. Typical settling time for settling and sedimentation tanks can range from 1-5 hours. To be conservative, a settling time of 5 hours was chosen to promote maximum settling, thus providing a total hydraulic retention time of 7 days. This value could not be modeled due to the lack of data pertaining to the nature of the solids particles in the effluent. Further, flow rates out of each tank were assumed to have no flow loss due to settling of TSS, though it is likely that an average of 10% of the flow would be lost to TSS. This assumption was made in order to have the most conservative contaminant concentrations for the flow into the wetland. A plan view of the Phase I treatment system showing the placement of the new tank and the plate and frame filter is shown in Figure 9-1.

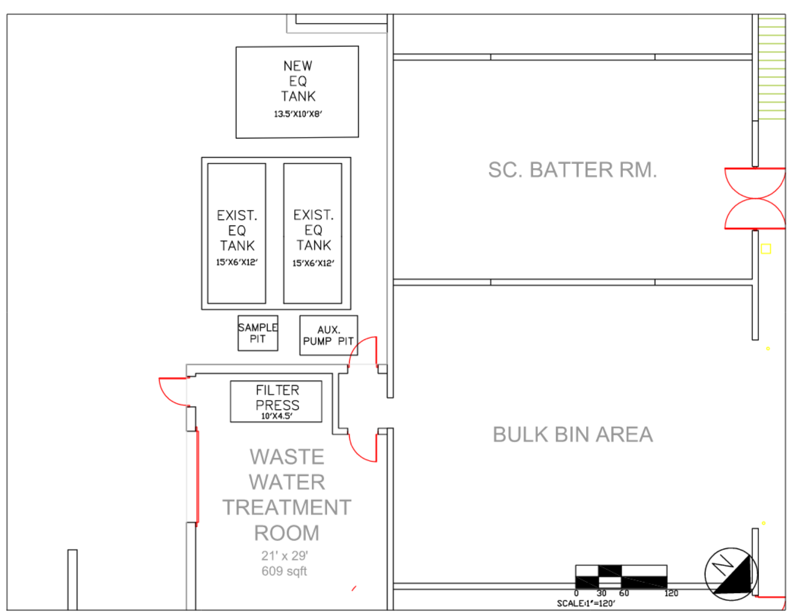

Figure 9- 2: Plan view of new treatment system

After Phase I treatment, the wastewater will be discharged to Phase II of the design which is the vertical subsurface flow wetland. The wetland has a capacity of 31,700 gallons. When the volume of media is excluded from the total capacity, the wetland will be able to hold roughly 11,200 gallons of water. Since Phase I of the design operates as a batch process, approximately 6,000 gallons of water will be discharged to the wetland every four days, averaging approximately 1500 gpd. Based on an average winter temperature of 4C and an average summer temperature of 27C, the evaporation rates were estimated to be 15% in the winter and 40% in the summer [20]. The cumulative flow rate (average of inlet and outlet flows) was calculated to be 1088 gpd. The dimensions of the wetland were based on the space constraints within the detention basin and the flow rate out of the Phase I design that would provide an ideal hydraulic retention time. Table 9-2 shows the known values based on the Phase I design that were used in the following design process.

Table 9- 2: Known values for Phase II design

Parameter	Variable	Value	Units
Flow In	Q_{in}	1500	gal/day
Flow Out	Q_{out}	1088	gal/day
Cumulative Flow (average Q_{in}/Q_{out})	Q	1294	gal/day
Length	L	130	ft
Width	W	17	ft
Length to width ratio	L:W	8:01	-
Total Volume	V_t	31700	gal
Winter Temperature	T_w	39.2	°F
Summer Temperature	T_s	80.6	°F

Using these values, the hydraulic retention time (HRT) and depth (d) of the wetland were developed using Equation 9-3 below.

Equation 9- 3: Hydraulic retention time for wetland

$$HRT = \frac{nLWd}{Q}$$

Where:

HRT: hydraulic retention time (days)
n: average porosity of media
L: length (ft)
W: width (ft)
d: depth (ft)
Q: average flow rate (ft^3/day)

Using typical design values, the team determined that the depth of the wetland would be approximately 2 ft. The depth and porosity of the media are given values determined by the media type [19]. The depth and porosity values are shown in Table 9-3. Figure 9-3 shows the layered design of the wetland.

Table 9- 3: Depth and porosity values for wetland

Media Type	Depth (ft)	Porosity
Sand	0.755	0.300
Fine Gravel	0.230	0.365
Medium Gravel	0.492	0.380
Coarse Gravel	0.492	0.415
Total/Average	1.969	**0.357**

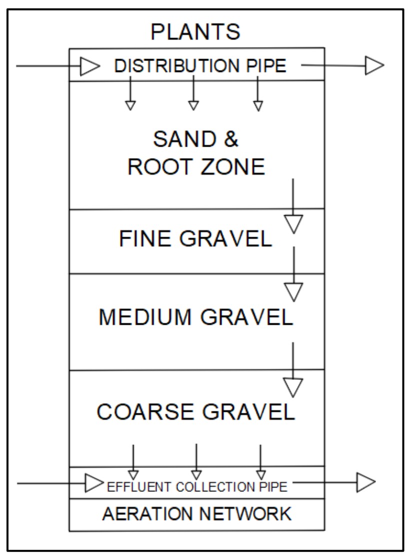

Figure 9- 3: Vertical subsurface flow constructed wetland media layers.

The total porosity (n) is a weighted average of the individual porosities of the media according to depth. Using n = 0.357, the HRT was determined to be 8.7 days according to Equation 9-3.

The next step in the process was to determine the effluent concentration rates to ensure that the wetland will be able to effectively treat the contaminant loads to below permit levels. Since BOD and TSS will already be below these levels, the main contaminant of concern for the wetland is TKN. Both TKN and BOD were modeled as first order decomposition reactions using Equation 9-4 [21].

Equation 9- 4: Effluent concentration rates

$$\frac{C_e}{C_i} = e^{-K_T * HRT}$$

Where:

C_e: Effluent concentration (mg/L)
C_i: Initial concentration (mg/L)
K_T: First order reaction rate constant (day^{-1})

When determining the reaction rate constants for TKN and BOD, the team used the average summer and winter temperatures in Flagstaff and assumed that the temperatures for fall and spring would fall between these high and low temperatures. For BOD, the reaction rate constants were determined using Equation 9-5 [21].

Equation 9- 5: Reaction rate constants

$$K_T = K_{20}\theta^{(T-20)}$$

Where:

K_{20} = BOD reaction rate constant at 20℃, 0.23day^{-1} [21]
θ = 1.056 when $0 < T < 20$℃
= 1.047 when $T > 20$℃
T = Temperature (℃), 27℃ for summer and 4℃ for winter conditions

The 1st order reaction rate constants were determined for TKN using Equation 9-6 and Equation 9-7 below [21].

Equation 9- 6: 1st order reaction rate constant (0-10℃)

$$K_T = 0.1376(1.15)^{(T-10)}$$

Equation 9- 7: 1st order reaction rate constant (>10℃)

$$K_T = 0.2187(1.048)^{(T-20)}$$

Equation 9-6 applies when the temperature is between 0 and 10℃ while Equation 9-7 is applicable when the temperature is above 10℃. Using these equations and the initial concentrations (the final concentrations from Phase I), Table 9-4 shows the results of the analysis.

Table 9- 4: Results of analysis

Parameter	Variable	Value	Units
BOD Inlet Concentration	C_i (BOD)	140	mg/L
TKN Inlet Concentration	C_i (TKN)	210	mg/L
BOD Winter Reaction Constant	K_T	0.0962	day^{-1}
BOD Summer Reaction Constant	K_T	0.3172	day^{-1}
TKN Winter Reaction Constant	K_T	0.0591	day^{-1}
TKN Summer Reaction Constant	K_T	0.3037	day^{-1}
BOD Winter Outlet Concentration	C_e (BOD_W)	61	mg/L
BOD Summer Outlet Concentration	C_e (BOD_S)	9	mg/L
TKN Winter Outlet Concentration	C_e (TKN_W)	125	mg/L
TKN Summer Outlet Concentration	C_e (TKN_S)	15	mg/L

The final exit concentrations of the design compared to the permit levels can be seen in the following table.

Table 9- 5: Outlet levels vs. permit levels

Contaminant	Outlet Levels		Permit Limits
	Winter	**Summer**	
BOD	0.134 lb/day	0.020 lb/day	700 lb/day
TKN	125 mg/L	15 mg/L	173 mg/L
TSS	0.001 lb/day	0.001 lb/day	130lb/day

After exiting Phase I, the wastewater will travel through an underground pipe to the wetland area. This water will then flow through perforated distribution pipes on the surface of the wetland which will evenly disperse the flow over the surface area of wetland media. The water will travel through the media for the HRT of 8.7 days where it will pool at the bottom of the wetland. Collection pipes will move the water using a 1% slope to a wet well located at the outlet. The wet well will contain an automated pumping system which will pump the water when the capacity of 132 gallons is reached to the sanitary sewer located along Shamrell Blvd. With a pump capacity of 52 gpm, pumping will be initiated every 10 minutes. This design does not require any changes in Joy Cone Co's current wastewater permit. The wetland will be lined with a 33mm think PVC liner which serves to protect the soil and surrounding environment from any infiltration or contamination. To aid in the degradation of TKN and BOD, an aeration system will also be distributed along the bottom of the wetland. The aeration will consist of a series of perforated pipes in a lattice pattern situated between the collection pipes. Native plant species will be included on the surface of the design which will help with TKN uptake and provide surface area for microbial growth in the root zone. Figure 9-3 shows a profile view of the entire wetland design.

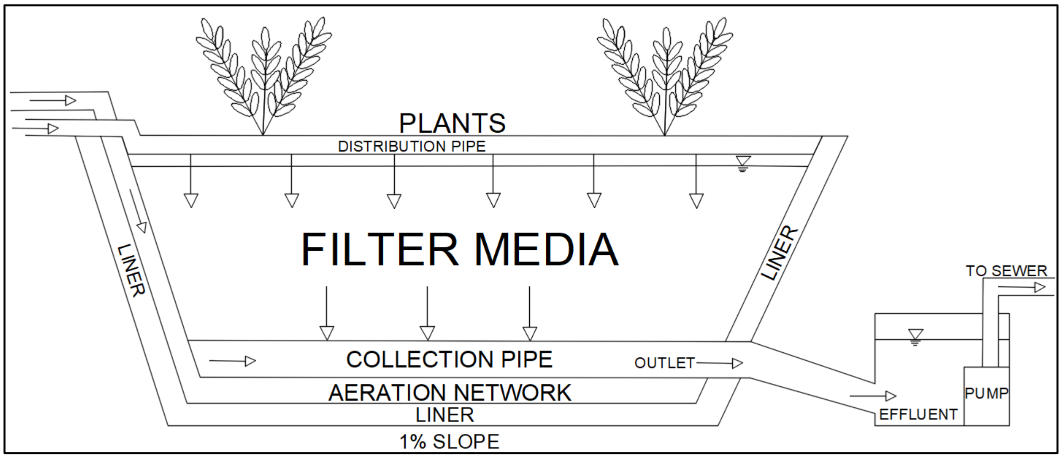

Figure 9- 4: Profile view of constructed wetland

A profile view of the pipe networks located at the bottom of the wetland can be seen in Figure 9-4.

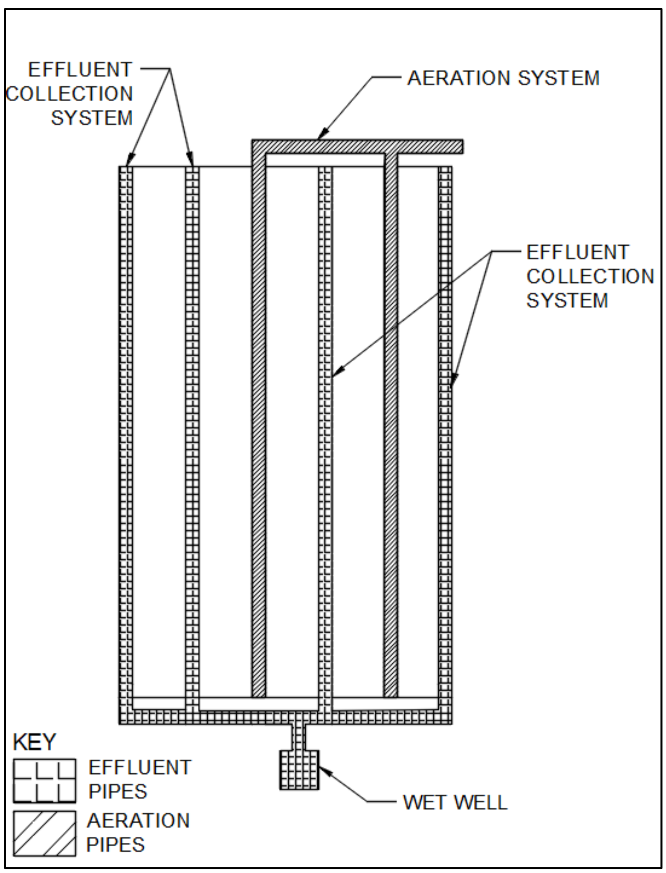

Figure 9- 5: Plan View of Aeration and Collection Pipe Networks

The wetland will be placed in the southeast portion of the current detention basin. There is a natural divide between the two halves of the basin including a culvert connecting them. To avoid mixing stormwater with the wastewater, the design includes the extension of the natural channels that currently transport runoff from the parking lot and surrounding forest to the culvert dividing the northwest and southeast portions of the detention basin. Figure 9-5 below shows a plan view of the detention basin design.

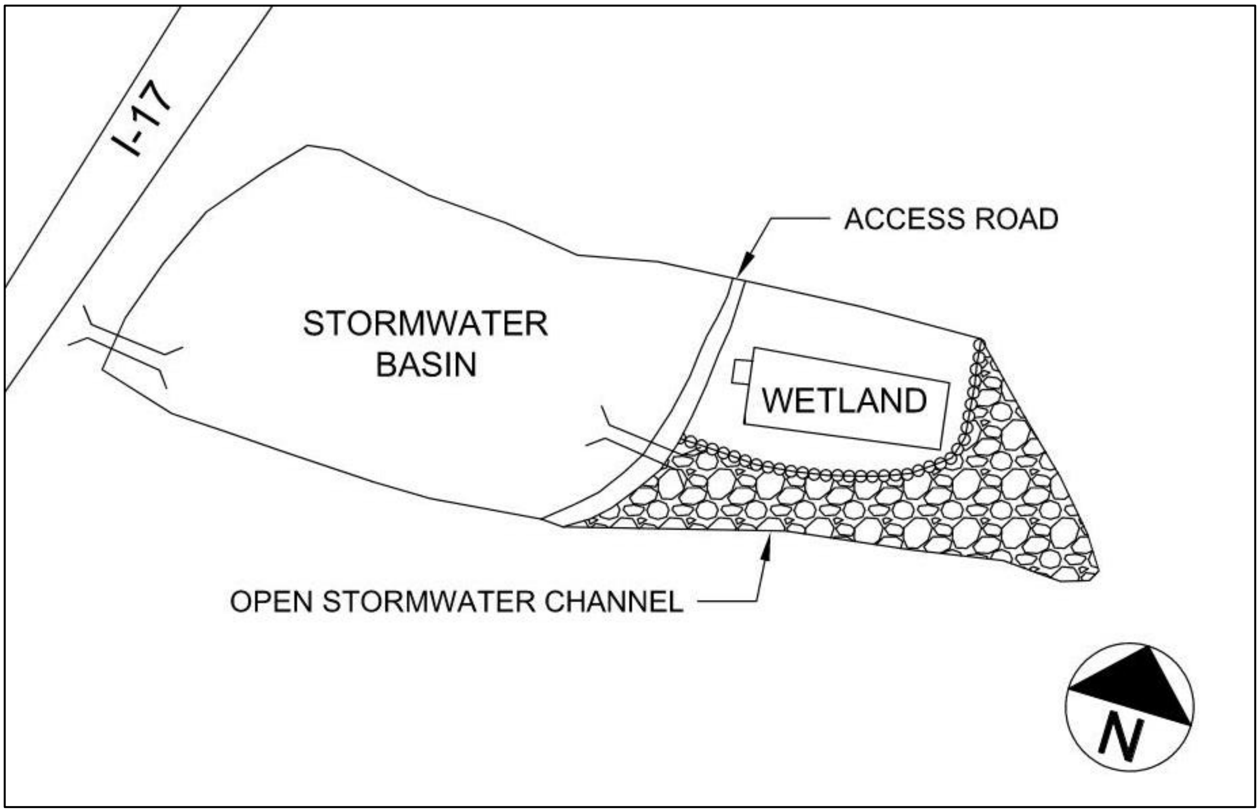

Figure 9- 6: Plan view of stormwater and wetland basin.

The culvert located in the most western portion of the detention basin will serve as an outlet for stormwater. The water will flow from the detention basin to an existing channel under the I-17.

It should be noted here that the wetland design does allow for expansion. Because of the large size of the detention basin, there is space for a second wetland of equal dimensions and parameters to be included in the design. Joy Cone Co. may elect to add a second wetland basin during construction which will double the amount of wastewater that can be treated in the wetland and provide extra space to continue treatment during maintenance periods. Unlike traditional treatment methods, wetlands must be operated continuously to maintain the integrity of the biological treatment provided by living microorganisms. Because of this, Joy

Cone Co. should be aware that the choice to add a second basin means that it will need to be operated in conjunction with the first basin. This can be done by alternating batch discharges between basins which will not affect the HRT of the basin but could cause lower treatment in summer months if the media becomes too dry between discharges.

9.2 Cost of Construction

Cost of construction for this design was determined through costs of equipment, installation, operations, and maintenance. The costs in table 9-6 were estimated using averages of typical commercial equipment costs. The costs of the pipe network transporting water from the facility to the wetland and from the wetland to the sewer are included in the Phase II design costs. The piping costs are estimated costs based on approximate pipe lengths and do not account for utility relocation and inspection. Costs are also only estimated for a single constructed wetland basin.

Table 9- 6: Cost analysis

Cost Analysis	
Item	**Cost**
New Aeration Tank [22]	$7,500
25 gpm Centrifugal Pump (3) [23]	$2,400
Raidan Standard 2 HP Plate Mount Top Entry Mixer (6) [24]	$40,000
6" Eccentric Plug Valve, Full Port, Mechanical Joint, Resilient Seated (3) [25]	$2,000
Advanced Wireless Fully Automatic Water Level Controller with Indicator [26]	$19,500
Plate and Frame Filter Press [27]	$30,000
Maintenance and Operation (per year)	$5,000
Phase I Total	**$106,400**
Earthwork [19]	$15,000
PVC Liner [28]	$1,200
Filter Media [19]	$12,000
Plants [19]	$1,000
52 gpm Centrifugal Pump [29]	$1,100
PVC Piping [30]	$21,000
Construction	$12,000
Pipe Installation [30]	$48,000
Maintenance and Operation (per year) [19]	$6,000
Phase II Total	**$117,300**
Total Cost	**$223,700**

As shown in Table 9-6, the final cost of both Phase I and Phase II resulted in an approximate cost of $223,700

9.3 Evaluation of Impacts

As part of the design, the environmental, economic, and societal impacts were evaluated.

9.3.1 Environmental Impacts

The positive impacts associated with the proposed design include creating habitat for toads, insects, animals, and plants. On the other hand, more solids may be sent to the landfill for disposal if the fill-and-draw system is successful. Additionally, some trees will need to be cut down within the detention basin to adequately construct the wetland.

9.3.2 Economic Impacts

The primary economic impact is that City of Flagstaff fees imposed on Joy Cone Co. for TKN exceedances will be eliminated. Additional economic benefits include job security for employees, and economic stimulation for the greater community. Although this project will be a large investment for Joy Cone Co., it is within the planned budget and will eliminate exceedance fees from the City.

9.3.3 Societal Impacts

Societal impacts include the addition of aesthetic value for the property. This design will also encourage the upkeep of paths in the area as well as the extension of the Flagstaff Urban Trails System. The public image of Joy Cone Co. will improve overall. Workers will also have a new large green space to enjoy their time during breaks outside of the workplace. In addition, production at the Joy Cone facility has the opportunity to increase due to the treatment systems' ability to accept increased volumes of wastewater. This project will overall reduce the contaminant load within the wastewater that is discharged to the City’s POTWs, though exceedances of TKN will continue for the duration of the wetland’s lag period (1-2 years).

10.0 Summary of Engineering Work

Engineering work was completed by the four members of the Honeycomb Engineering Capstone team. This section compares the work hours estimated in the Fall 2022 proposal to the actual hours completed by the team. Five roles were developed including senior engineer (SENG), engineer (ENG), engineer in training (EIT), survey technician (S

Tech), and lab technician (Lab Tech). Table 10-1 below shows the estimated working hours as stated in the design proposal.

Table 10- 1: Fall 2022 Estimated engineering staffing hours

Staffing Hours Summary	
Position	**Hours**
Senior Engineer (SENG)	108
Engineer (ENG)	199
Survey Technician (STECH)	30
Lab Technician (LAB TECH)	92
Engineer in Training (EIT)	196
Total Hours	**625**

At the completion of the project, the team logged a total of 564.5 hours of engineering work. The actual engineering hours completed by the team are provided in table 10-2 below according to the roles developed in the proposal. At the start of the design process, the team elected to eliminate the EIT position as there was little to no work that would be required for that role. Additionally, due to the procurement of survey data instead of collection by the team, the survey technician hours are much lower than estimated in the proposal. A majority of the work was completed by the engineer.

Table 10- 2: Logged spring 2023 project hours

Staffing Hours Summary	
Position	**Hours**
Senior Engineer (SENG)	117.5
Engineer (ENG)	370
Survey Technician (STECH)	8.5
Lab Technician (LAB TECH)	68.5
Engineer in Training (EIT)	0
Total Hours	**564.5**

11.0 Summary of Engineering Cost

This section will compare the proposed engineering costs to the estimated costs based on the completed engineering work. Table 11-1 shows the costs quoted in the Fall 2022 proposal. The total quoted cost was $52,608. Table 11-2 shows an itemized list of laboratory supplies.

Table 11- 1: Fall 2022 proposed cost of engineering services.

Cost of Engineering Services				
Item	Description	Quantity	Rate	Cost
1.0 Personnel Cost		hours	$/hr	$
Personnel	SENG	108	155	16,740.00
	ENG	199	120	23,880.00
	STECH	30	50	1,500.00
	LAB TECH	92	50	4,600.00
	EIT	196	25	4,900.00
			Total	$ 51,620.00
2.0 Laboratory Facilities		days	$/day	
Lab Rental	NAU ENE Laboratory	5	100	500.00
			Total	$ 500.00
3.0 Supplies		-	-	
Lab Supplies	See Table 11-2	-	-	488.00
			Total	$ 488.00
Total Cost of Engineering Services:				**$ 52,608.00**

Table 11- 2: Itemized list of laboratory supply costs

Item	Description	Quantity	Unit Cost ($/ea.)	Cost ($)
Glass Fiber Filters	100 per pack	1	57.00	57.00
Gloves	100 per box Disposable	1	12.00	12.00
Goggles	Laboratory goggles	4	1.50	6.00
ASTM D5907-18	TSS test document	1	57.00	57.00
ASTM D2329	BOD5 test document	1	69.00	69.00
HACH TKN Test Kit	25 samples per kit	1	220.00	220.00
Pipettes	Disposable 250 per pack	1	67.00	67.00
			Total:	$ 488.00

The actual costs of engineering services are provided in table 11-3. The total cost of the project based on personnel hours, laboratory facilities, and laboratory supplies is $67,381.50. The cost for laboratory usage and laboratory supplies remains the same with the exception of the BOD test document, however, the main difference in cost comes from the personnel hours.

Table 11- 3: Actual cost of engineering services

Cost of Engineering Services				
Item	Description	Quantity	Rate	Cost
1.0 Personnel Cost		hours	$/hr	$
Personnel	SENG	117.5	155	18,212.50
	ENG	370	120	44,400.00
	STECH	8.5	50	425.00
	LAB TECH	68.5	50	3,425.00
	EIT	0	25	-
			Total	$ 66,462.50
2.0 Laboratory Facilities		days	$/day	
Lab Rental	NAU ENE Laboratory	5	100	500.00
			Total	$ 500.00
3.0 Supplies		-	-	
Lab Supplies	See Table 5-2	-	-	419.00
			Total	$ 419.00
Total Cost of Engineering Services:				**$ 67,381.50**

Although the actual cost of engineering services is higher than the quoted cost, the client will only be billed the costs stated in the proposal.

12.0 Conclusion

Joy Cone Co. produces 500,000 gallons of industrial wastewater annually which is treated by a rotary drum vacuum filter. After experiencing fines for TKN exceedances, Lane Fisher (plant engineer) has requested a new industrial wastewater pretreatment design to target TKN, BOD, and TSS. The design created by Honeycomb Engineering uses a two-phase system to treat TSS and BOD using a fill and draw sequencing batch reactor and a vertical subsurface flow wetland before discharging to the public sanitary sewer. This system will treat all three contaminants of concern to below the current permit levels and eliminate the fines imposed by the City.

12.0 Bibliography

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