CEE:3084
Project Design and Management- Civil Engineering

Fin al Report

Water Reclamation for Terry Trueblood Recreation Area

May 1, 2015

Brandon Willis	**Date**	**Sarah Salomon**	**Date**
Flora Duff	**Date**	**Meghan O'Connor**	**Date**
Zijun Ren	**Date**		

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Executive Summary

Irrigation of the Terry Trueblood Recreation Area (TTRA) and Napoleon Park in Iowa City currently draws from the public water supply. This practice uses vital water resources while expanding the city's footprint. This feasibility study will investigate alternative methods to reclaim wastewater from the Iowa City Wastewater Treatment Plant South (ICWWTPS) and use it to irrigate the TTRA and Napoleon Park. Reclaimed water will be delivered to the areas by a decommissioned sludge force main which runs between ICWWTPS and the former wastewater treatment plant on Capital St. We examined three possible alternatives to accomplish reclamation: (1) water will first be pumped into sand lake, which will act as a reservoir, and then be pumped to each of the two sites; (2) water will first be pumped into holding tanks and then pumped to each of the two sites; and (3) water will be pumped directly to the two irrigation sites. Using a design matrix and input from our client, the final design was determined. The water from the ICWWTPS will be pumped into Sand Lake to maintain water levels and prevent fish kills, drawn through a drilled well, pumped through a UV system, and distributed to both the TTRA and Napoleon Park. The final price of the design is $198,500 + $80,700/ year.

1.0 Introduction

SWORD Engineering was established in January 2015. The SWORD team consists of five engineers of various fields and experiences. The team members are Brandon Willis, Sarah Salomon, Flora Duff, Meghan O'Connor, and Zijun Ren. This firm first presented three (3) design alternatives in the proposal, each of which will satisfy the task presented in the RFP. Upon contraction between the client and this firm, this firm proceeded with detailed specifications of each design. A decision matrix was provided to the client, so that they were able to systematically select the most suitable alternative. This firm worked closely with the client in order to choose the best possible alternative. Upon selection, the chosen alternative was further developed, and detailed plans and specifications were assembled. The scope of the services laid within the bounds of this firm's expertise, and therefore, subcontractors were not utilized. The following report includes the design alternatives presented and a detailed breakdown of the final design chosen. An oral presentation of the design will be made on May 6th to our client and other members of the city of Iowa City.

2.0 Problem Statement

This firm is tasked with re-commissioning the sludge force main that runs from the Iowa City North Wastewater Treatment Plant to the Iowa City South Wastewater Treatment Plant (ICWWTPS). The re-commissioned main will carry reclaimed wastewater from ICWWTPS to the Terry Trueblood Recreation Area and Napoleon Park, south of downtown Iowa City, where it will be used to irrigate the areas and potentially maintain the water level of Sand Lake.

2.1 Design Objectives

Specific tasks of this design will include:

1. Design of pipe and fittings to connect the sludge force main to the wet well at ICWWTPS.
2. Design of pipe and fittings to connect force main to appropriate structures at the irrigation sites.
3. Design pumping station(s) to carry the water to the irrigation sites.
4. Ensure the water will be treated to all required standards.
5. Evaluate the environmental impact of each design.

The main objective of this project is to reduce the cost and environmental footprint associated with irrigating with city water. This is to be accomplished by replacing the current drinking water with reclaimed wastewater. This project will assess the feasibility of alternative means to transport wastewater to the irrigation sites by means of an existing force main.

2.2 Approaches

Limited standards exist for the repurposing of sludge force mains to accommodate reclaimed water. Therefore, the Iowa Department of Natural Resources Wastewater Facilities Design Standards were implemented for force main connections and extensions (Chapter 13).

Irrigated effluent considerations were made with accordance to chapter 567.62 of the Iowa administrative code in addition to the Iowa City South Wastewater Treatment Plant's 2014 NPDES permit.

2.3 Constraints

After a thorough run-through of the project needs with our client, a few constraints were identified. A strict budget was not provided to us by the client, although they did make it clear that they would prefer a ‘reasonable’ cost. They suggested that $1 million would be excessive, but the final cost will not be the major determining factor of the project. Space was not determined to be a major constraint as the piping that we will design will all run underneath the boundaries we are working within. Time, however, is a constraint, as our client wants to implement the wastewater reuse irrigation system as soon as possible. It seems as if this project presents many more soft constraints, ones that should be followed, than hard constraints, ones that must be followed. The cost of the project and the space allowed are both soft constraints. The sole hard constraint, however, is the timeframe of our project.

2.4 Challenges

In regards to challenges that may be faced during the duration of the design of this system, very few were found. A major obstacle that was faced is the allowance by the EPA to refill Sand Lake with wastewater reuse to keep the water levels steady. It was determined that social acceptance of this project will not be an issue as the Terry Trueblood Recreation Center is already using reuse to irrigate their soccer fields. This has been done for the past two decades with appropriate signage placed on the fields, and not one member of the public has contacted our client with any worries or concerns.

2.5 Societal Impacts

Working with the Terry Trueblood Recreation Center has a direct and positive impact on the community’s populace, economy and public revenues within the state of Iowa. Jobs will be created within the surrounding area for construction. The community will benefit as the Terry Trueblood Recreation Center provides an area for weddings, receptions, reunions, and picnics. Irrigating the poor, dry soil will ensure that the land is useable for family activities. This area is Iowa City’s largest park, offering walk-run trail routes, fishing, bird watching, hiking, and concessions and boat rental.

3.0 Preliminary Development of Alternative Solutions

<u>Design 1: Utilize Sand Lake as a Reservoir</u>

The first design will utilize Sand Lake as a reservoir for the irrigation water. Effluent from the ICWWTPS will enter Sand Lake near the south end of the lake. Water in Sand Lake will then be drawn near the north end of the lake and distributed to the irrigation systems. This plan will help control the level of Sand Lake, as it has been experiencing some decline in recent years. It will also promote circulation in the lake and capacitance for the distribution system.

<u>Design 1a: Utilize Sand Lake as a reservoir for both the TTRA and Napoleon Park</u>

One option would be to use the water in Sand lake for the irrigation of both the TTRA and Napoleon Park. In this case, water will be withdrawn from Sand Lake and connected to the main irrigation line at the Lodge. A new pipeline will have to be installed from the lodge back to the force main, where it can be carried to the Napoleon Park Area (Figures 1 and 2).

Figure 1: Terry Trueblood Recreation Area alternative 1a. Effluent from the ICWWTPS will enter Sand Lake at the South end. Water will be withdrawn from the lake south of the lodge, at the north end of the lake. A pipeline will be installed from the lodge back to the force main, where it will be carried to Napoleon Park.

Figure 2: Napoleon Park alternative 1a. Water is directed from the force main to the main irrigation building by a new pipeline, where it will be distributed to the softball fields.

<u>Design 1b: Utilize Sand Lake as a reservoir only for the TTRA, and pump directly to Napoleon Park</u>

Another option would be to use the water in Sand Lake only for the irrigation at the TTRA, and pump directly to Napoleon Park. This would reduce excavation and head loss costs associated with building a pipe from the Lodge to the Napoleon Park (Figures 3 and 4).

Figure 3: Terry Trueblood Recreation Area alternative 1b. Part of the effluent from the ICWWTP will enter Sand Lake at the south end. Water will be withdrawn from the lake south of the lodge, at the north end of the lake. Another portion of the effluent from the ICWWTP will continue through the force main to Napoleon Park.

Figure 4: Napoleon Park alternative 1b. Water is directed from the force main to the main irrigation building by a new pipeline, where it will be distributed to the softball fields.

Design 2: Installation of reservoirs near the irrigation site

The second design will implement reservoirs near the sites to be irrigated. Effluent from the ICWWTPS will enter each of the reservoirs, where it will be stored until irrigation. Several possible reservoir locations are presented. A reservoir for the TTRA will require at least 3,600 gallons, and the reservoir for Napoleon Park will require 12,000 gallons of storage. One option would involve placing a reservoir on the roof of the lodge. In this case, a thorough structural analysis of the lodge is needed. Possible reservoir options include water towers or underground storage tanks. Further consultation with the client is warranted in determining the reservoir type and location. Figures 5 and 6 illustrate potential reservoir locations.

Figure 5: Terry Trueblood Recreation Area alternative 2. Each colored 'X' depicts a possible reservoir location and its associated pipelines to be constructed. The red line marks another potential pipeline which would deliver water to Sand Lake.

Figure 6: Napoleon Park alternative 2. Water is directed from the force main to the reservoir to the main irrigation building by a new pipeline, where it will be distributed to the softball fields.

<u>Design 3: Pump directly to irrigation sites.</u>

The third design will deliver water directly from the force main to the irrigation systems. A new pipeline will be constructed at the TTRA between the force main and the lodge and between the force main and the irrigation building at Napoleon Park. Figures 7 and 8 show the path of the water.

Figure 7: Terry Trueblood Recreation Area alternative 3. Water will travel from the force main directly to the lodge, where the irrigation hook-up is. The red line marks another potential pipeline which would deliver water to Sand Lake.

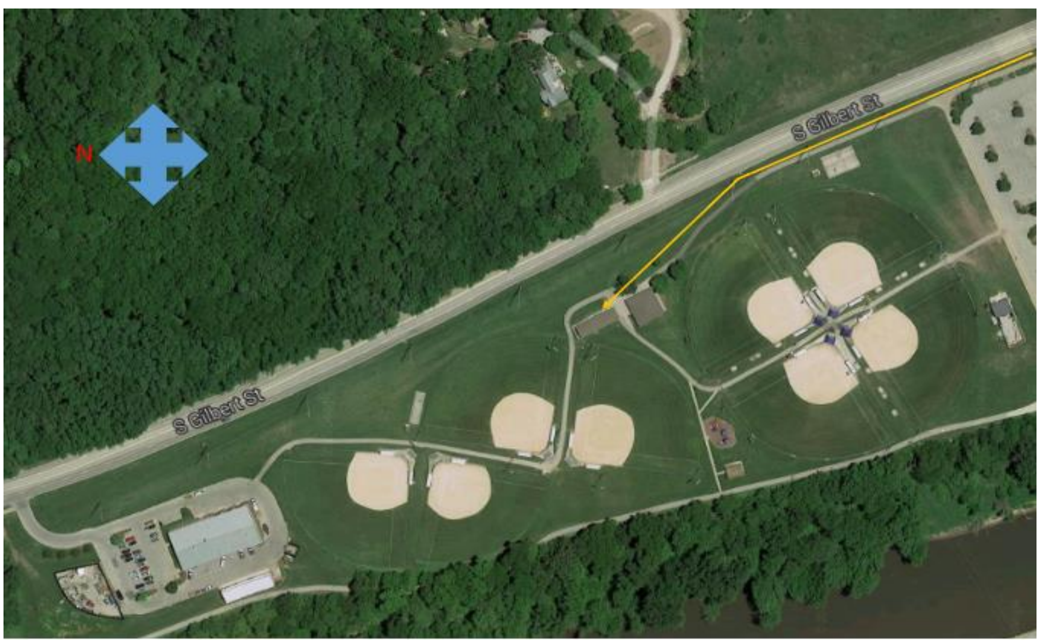

Figure 8: Napoleon Park alternative 3. Water is directed from the force main to the main irrigation building by a new pipeline, where it will be distributed to the softball fields.

4.0 Selection Process

There are multiple parameters that needed to be taken into account when considering the effects of pumping effluent into a lake ecosystem. The first we looked at was pH. Ideally, a natural water system should have a pH between 6 and 8. Any flow into a lake or river system needs to be at the proper pH to prevent the natural waters from leaving this range. The effluent flow from the

Iowa City Wastewater Treatment Plant has an average pH of 7, so this will not cause a pH disturbance in the lake. Should the WWTP experience a dramatic drop or increase in their effluent pH, it needs to be corrected immediately or flow must be stopped, as a pH change within the lake could have dramatic effects on the lake's chemistry and result in death of aquatic life.

There will also be BOD entering the lake via the wastewater effluent. The average BOD, or biological oxygen demand, of the WWTP effluent is 3.0-4.0 ppm, or 3.0-4.0 mg/L. While this could cause an overall drop in dissolved oxygen (DO) levels in the lake and possibly endanger the ecosystem, the effluent also has a 30 day average DO level of 7.5 mg/L. Should this level not drop, the BOD of the effluent will be satisfied and there will be little to no effects on the lake. However, should the effluent DO level drop, the lake may need to supply some of the DO needed to meet the effluent BOD. In order to simulate these effects, the worst case scenario of an effluent DO level of 0 mg/L was assumed. Figure 9 shows how the DO level in the lake drops each day.

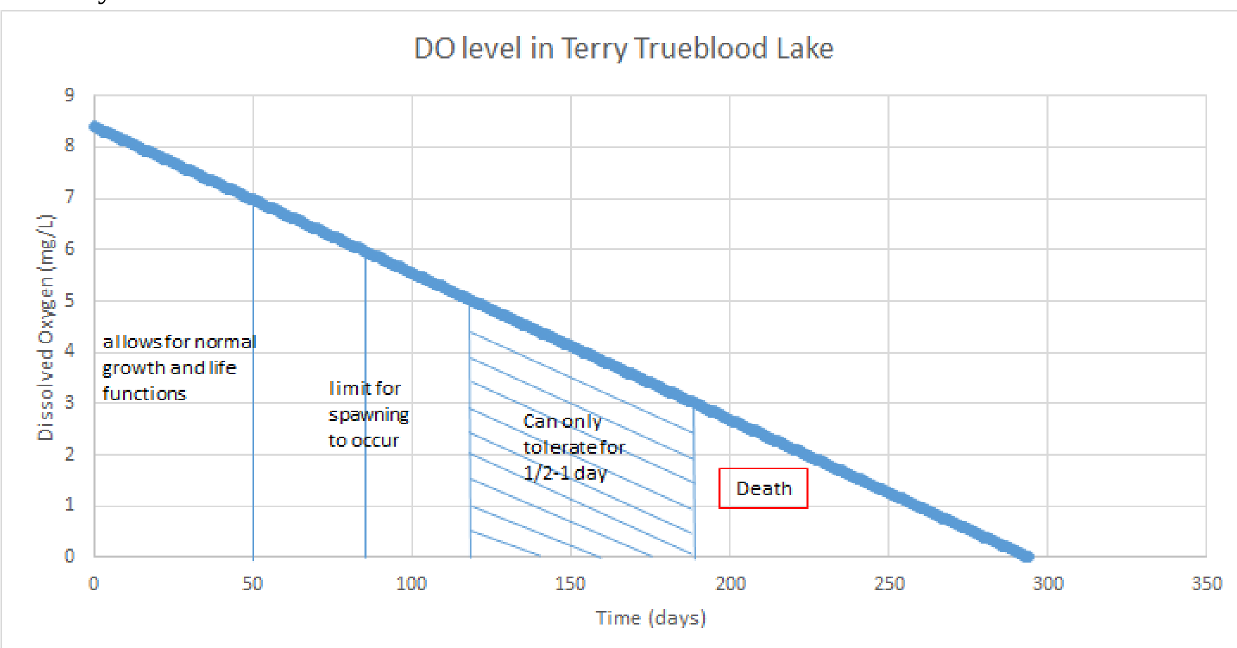

Figure 9: The dissolved oxygen levels in Sand Lake over time, assuming an initial DO level of 8.40mg/L and no influx of oxygen into the lake.

As seen in Figure 9, the DO level in the lake would not reach a level below which fish can reproduce until after 88 days. They would not reach their "danger zone", or the DO level which they can only survive for 0.5-1 day, until 124 days. They would not die unless there was no influx of DO for 194 days. Fortunately, this scenario is highly unlikely. This analysis assumed no oxygen would enter the lake, but under normal warm temperature circumstances, oxygen would enter the lake via the atmosphere. The only time this would not occur is if the lake was completely frozen over, such as winter (there still could be oxygen entering via other pathways). It has been noted that the lake does freeze over, however it does not do so for more than 4-5 months, or 120-150 days, straight. Biological activity, and thus BOD, would unfortunately be reduced during cold winter months. This means that even in the worst case scenario, the fish could survive. Considering this, along with the DO level normally in the wastewater effluent, it is believed that the DO level in the lake would not be in danger due to effluent pumping.

Phosphorus is a major nutrient that can lead to algae blooms, and thus endanger the lake ecosystem. The EPA recommends a total phosphorus concentration of 0.1 mg/L to allow moderate diversity in fish populations. The IDNR considered a 0.2 mg/L level of phosphorus to be elevated, and a lake at this level would be classified as impaired. The ICWWTP produces an effluent with a phosphorus concentration of 1.0-2.0 mg/L. An analysis was done observing the increase in phosphorus due to effluent pumping over the course of 1 year. The results of this analysis are shown in Figure 10.

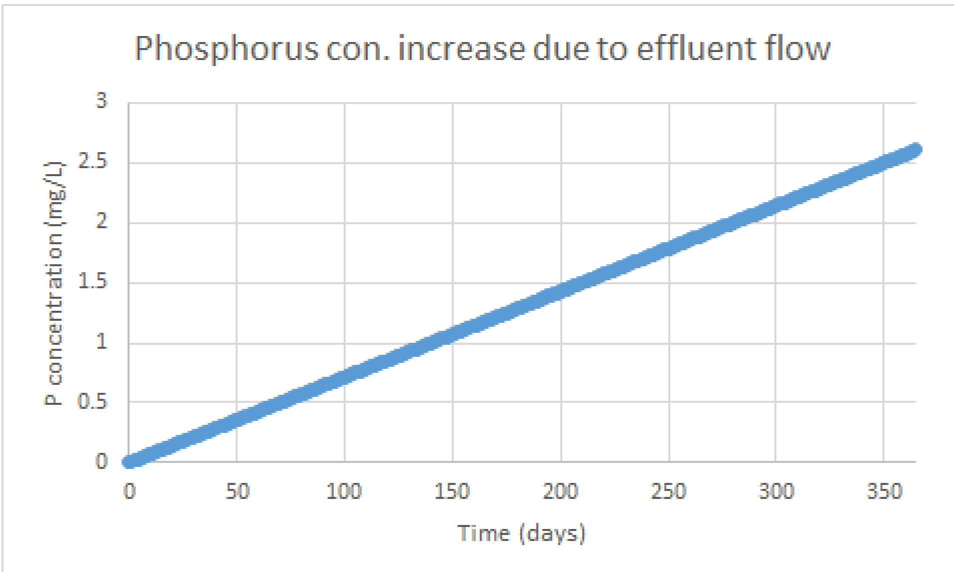

Figure 10: Increase in phosphorus concentration in Sand Lake due to effluent pumping

As seen in Figure 10, the phosphorus concentration would increase rather quickly. The concentration reaches above the 0.2 mg/L level well within the first 50 days. Figure 11 provides a close up of the first 50 days.

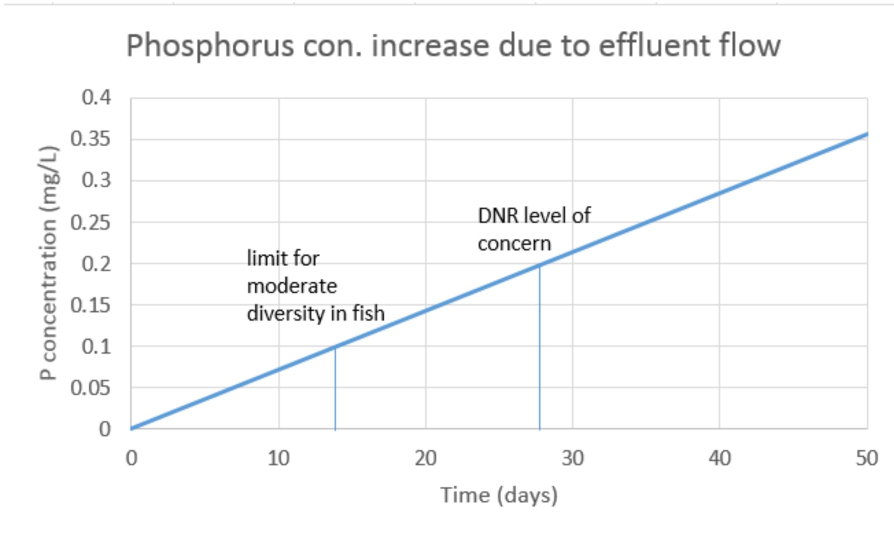

Figure 11: Increase in phosphorus concentration in Sand Lake due to effluent pumping during the first 50 days

The lake would be over the limit which allows moderate fish diversity in 15 days. It would be over the level at which the DNR has become concerned in past cases at 29 days. This analysis only considered phosphorus increase due to effluent entering the lake, so if there was phosphorus already present (which is highly probable) the actual level within the lake would be higher. Due to this analysis, it is recommended that the effluent phosphorus concentration be reduced to prevent algae blooms. Figure 12 shows that the phosphorus level would increase if the effluent phosphorus concentration was reduced to 0.1mg/L.

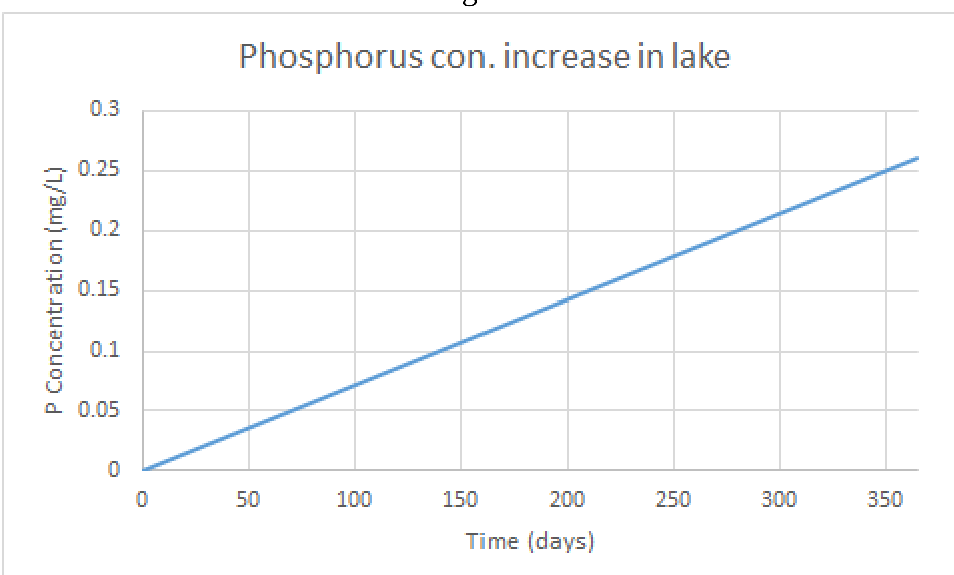

Figure 12: Increase in phosphorus concentration in the lake due to effluent pumping at 0.1mg/L

It would take 144 days to reach the 0.1mg/L limit, and 281 days to reach the 0.2mg/L limit. This still may raise some concern with the Iowa DNR. Should this be used as a possible solution to allow for pumping into the lake, the phosphorus levels within the lake would need to be closely monitored.

This same analysis was completed for nitrogen, another nutrient that can lead to algae growth. The effluent NH_3-N for the ICWWTP is less than 1 mg/L, and the total nitrogen level in the effluent is 13.4 mg/L. Both of these may be important to the DNR, so an analysis of both the ammonia and total nitrogen level increase in the lake due to effluent flow was completed. Figure 13 shows the increase only due to ammonia nitrogen, and Figure 14 shows the increase due to total nitrogen.

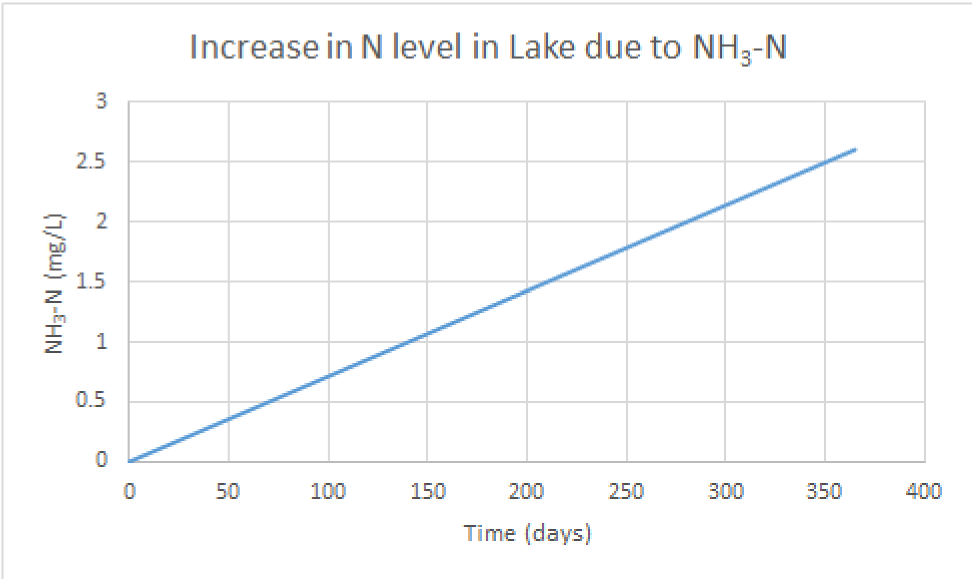

Figure 13: Increase in nitrogen levels in lake due to effluent ammonia levels of 1 mg/L

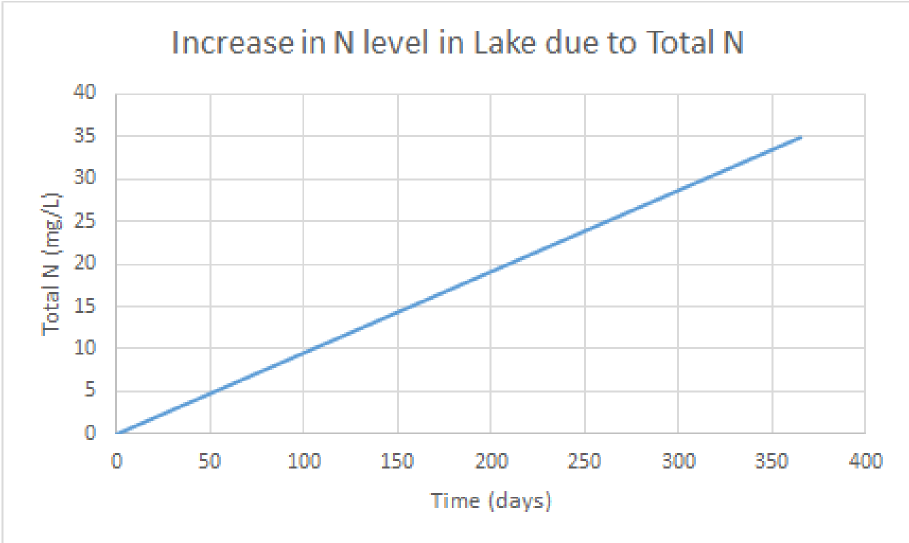

Figure 14: Increase in nitrogen levels in lake due to effluent total nitrogen

The WWTP has also suggested adding chlorine at a level of 1-2 mg/L to the effluent leaving the plant to provide disinfection and cleaning of the pipe during transport. This is not recommended as the aquatic species within the lake are sensitive to chlorine, and the chlorine would still be present when the effluent reaches the lake. A fountain in the lake which would be used to aerate the effluent before it enters the lake could reduce chlorine levels due to its high volatility, but it may not remove all chlorine.

A decision matrix was used to determine which solution should be used for the project. This matrix is shown in Table 1. The alternatives received a ranking of either 1st, 2nd, or 3rd in each category, and the points were totaled at the bottom of the table. The lowest scored option will receive a first place ranking.

Table 1: Decision matrix that was used to determine which alternative will be used.

	Alternative 1*	Alternative 2	Alternative 3
Environmental Impacts	3	2	1
Cost	2	3	1
Footprint	2	3	1
Ease of Operation	1	2	3
Lifetime	3	2	1
Total	*11*	*12*	*7*

*includes options a and b

In the environmental impacts category, Alternative 3 received the highest ranking because it would disturb the environment the least. The only change to the environment that would be made is in the digging and insertion of pipelines to bring the effluent from the force main to the irrigation sites. Alternative 2 ranked second because it would require the additional digging and insertion of holding tanks. Alternative 3 received the lowest ranking due to the changes in the lake that it would cause, as described in the previous section.

Alternative 3 received the highest ranking in the cost category for the same reason it won the environmental impacts category; the only change is the addition of short piping systems. Alternative 1 was second place due to the cost of the sand point filter needed to clean the lake water and the extra pump. Alternative 2 would be the most expensive due to the cost of extra pumps and holding tank purchase and installation.

Once again, due to the simplicity of Alternative 3, it won in the category for smallest footprint. Alternative 1 would use the lake already in place, so it only needs the extra space for the new piping, sand point filter, and pump. Alternative 2 would require large holding tanks, and therefore took third for the largest footprint.

Rankings for ease of operation were slightly more complicated. Alternative 1 provides simplicity for the WWTP, because it would pump continuously through an invariable system at a constant flow rate to an open atmosphere reservoir. Alternative 2 provides similar simplicities, but isolation valves to control flow to either the tank or the lake would need to be operated in conjunction with a water level sensor in the tank. Alternative 3 would be the most complicated to operate. The pump at the WWTP would need to be operated with a variable frequency, a booster might be needed, and a precise SCADA system would be required to monitor the system and control valves.

This also contributes to why Alternative 1 took third in lifetime of design. As shown by the lake system analysis, the nutrient levels will eventually become too high and action will need to be taken if pumping into the lake is to continue. Alternative 2 received second place because the tanks will eventually degrade and need replacement, although that will not be for a very long time considering they are only holding treated effluent. Alternative 3 will have the longest lifetime, as it is viable as long as the pipes are still useable. However, if the nutrient problem described above is mitigated, Alternative 1 would rank the highest for life time.

Based on the design matrix, Alternative 3 has the lowest score. However, Alternative 3 does not meet the secondary goal of maintaining lake levels and preventing fish kills. Also, Alternative 1 would require far less oversight and operation time for the ICSWWTP than another option and is therefore the only option which meets their needs. The client selected Alternative 1 as it did meet this criteria and provided the bonus of helping to maintain higher water levels in Sand Lake. This would allow for year-round enjoyment by city residents.

5.0 Final Design Details

The following section details the individual systems making up the chosen alternative.

5.1 Transmittal

A Simflo SS12 Vertical Turbine Pump (VTP) will be installed above the effluent holding tanks on top of the concrete slab in the location shown in sheet 01 of the attached plans. The pump will deliver 1400 gpm (2 MGD) to the lake at 472’ TDH, requiring 201 Hp, as seen in Figure 15. An isolation gate valve and swing check valve will be fitted to the pump effluent lines. The pump will run constantly throughout the year. Redundancy is not required in this pump because there is no required discharge into Sand Lake at any time, and in the event of pump failure, effluent can be redirected to the Iowa River via existing pumps. Regarding the required irrigation flow, the lake provides plenty of storage to continually draw water from while the Simflo VTP is being serviced. The daily irrigation requirements are 72,600 gallons during drought periods, roughly 4% of the 2,000,000 gallons we intend on discharging per day.

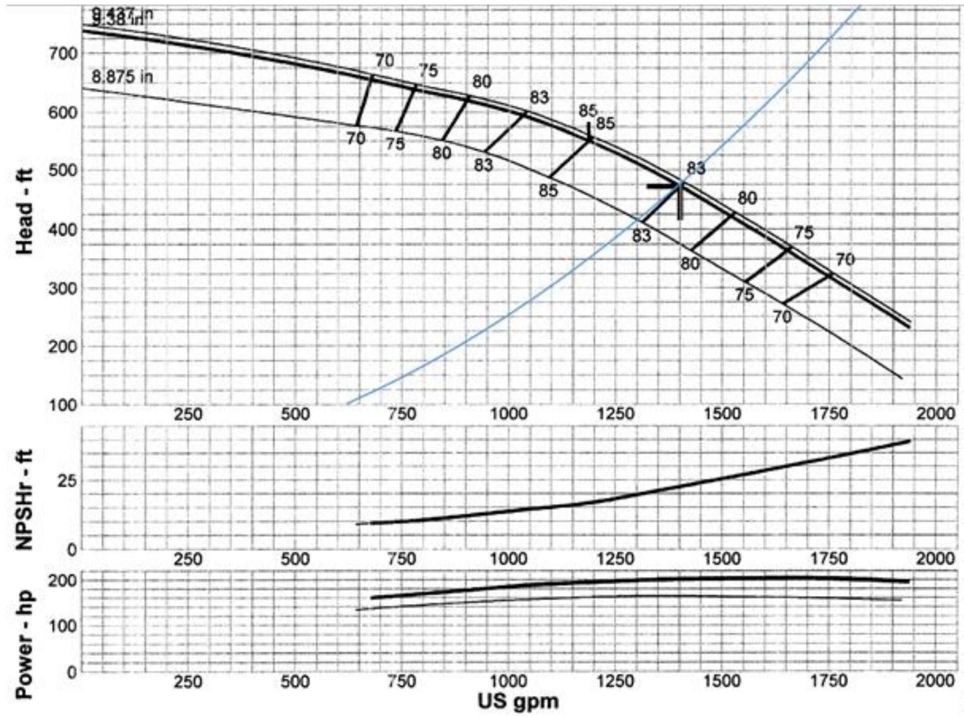

Figure 15: Simflo SS12 Vertical Turbine Pump curves overlaid on the system curve for the path from the effluent holding tanks to Sand Lake

A Goulds 160L03 Submersible Pump will be installed in the well beneath the pump house to transmit water to the surface and into the UV system. A Goulds Close-coupled Ag-Flo Centrifugal Pump will be installed in the pump house and will transmit water from the UV effluent reservoir to Napoleon and TTRA. This pump will provide enough head to overcome the system losses and achieve 90 psi at the sprinkler heads. These two pumps are designed for three situations:

1. The Terry Trueblood Recreation Area is being irrigated, and the pumps will deliver 75 gpm.
2. Napoleon Park is being irrigated, and the pumps will deliver 125 gpm.
3. TTRA and Napoleon Park are being irrigated at the same, and the pumps will deliver 200 gpm.
4.

The pumps will run late April through mid-October, at various times of the day. A diurnal cycle of irrigation requirements during drought conditions is provided in Figure 16. The pump motors will be equipped with a 3-phase motor and variable frequency drive (VFD) in order to conserve energy and eliminate the need for large and expensive storage tanks. The VFD will ensure that the two pumps simultaneously transmit the same flow rate by altering the AC frequency to each motor as necessary. A Supervisory Control and Data Acquisition (SCADA) system will control the VFD.

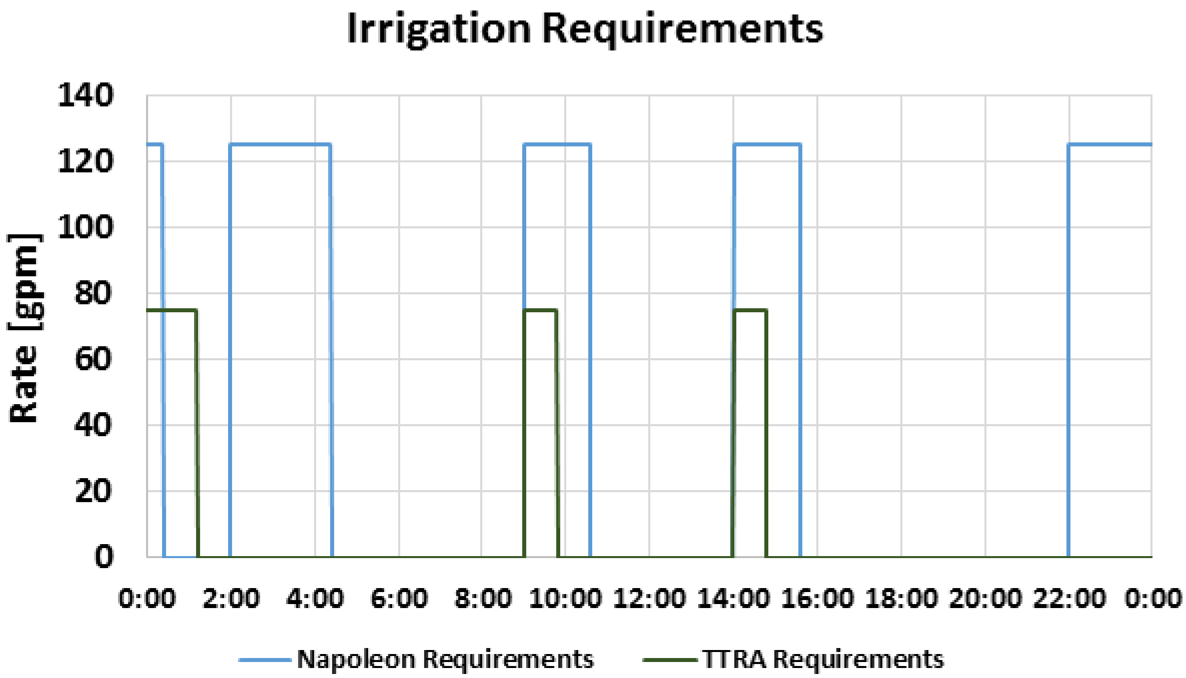

Figure 16: Irrigation requirements during drought conditions

5.2 Piping

In total, 584 linear feet of 8-inch diameter ductile iron pipe, 744 linear feet of 6-inch diameter Class 200 PVC pipe, and 440 linear feet of 4-inch diameter Class 200 PVC pipe is required.

At the wastewater treatment plant, 239 feet of 8” ductile iron pipe will connect the vertical turbine pump at the effluent holding tank to the existing 8” ductile iron force main, as seen in Figure 17. The existing force main, east of the new connection, will be cut and isolated from the new flow, and a concrete thrust block will be installed.

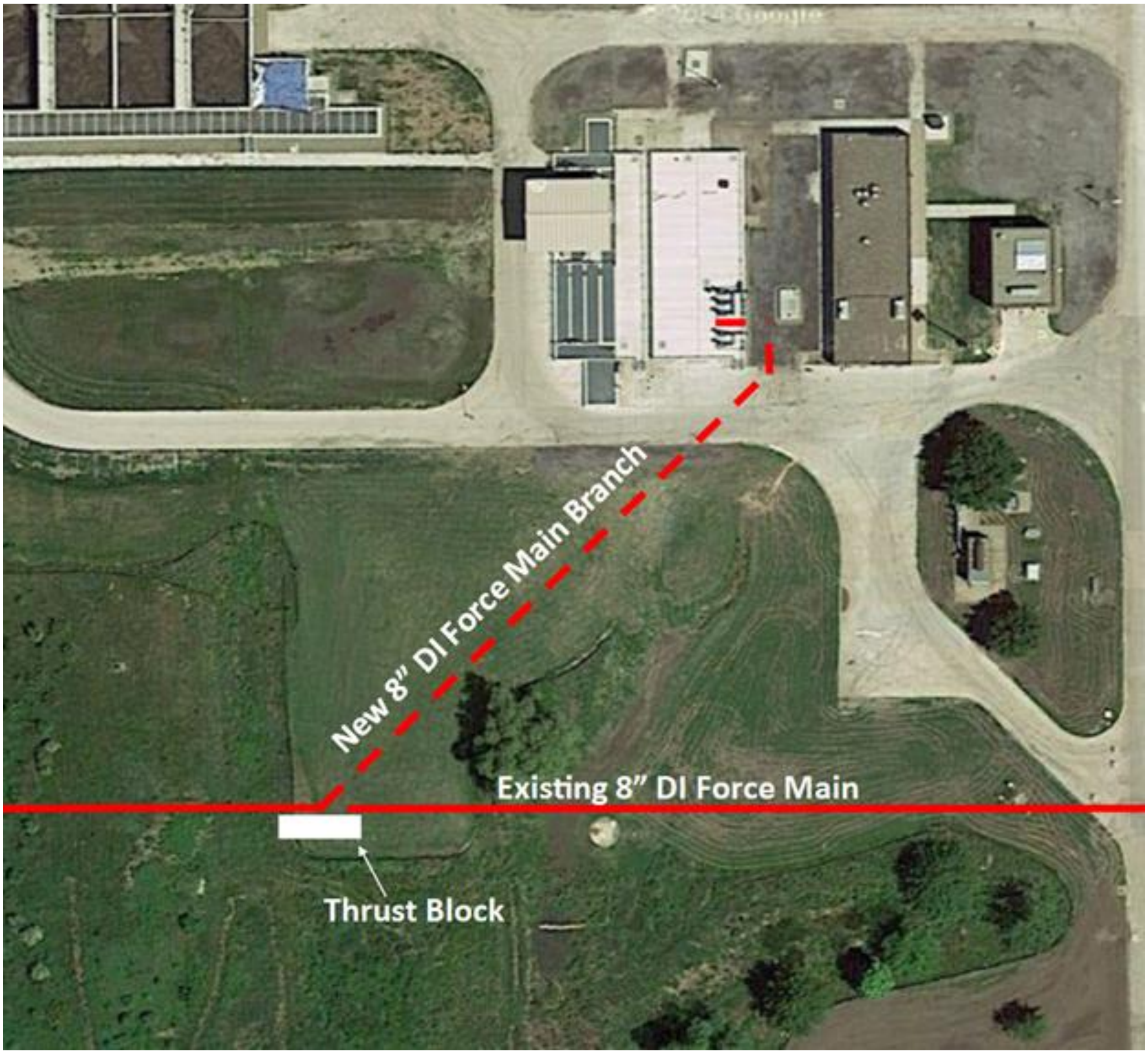

Figure 17: Pipe layout of the piping system at the Iowa City South Wastewater Treatment Plant

A new 8" branch of ductile iron pipe will be added to the existing force main just north of the Sycamore St. & S. Gilbert St. intersection. The location of the new branch requires the least amount of pipe material and avoids the beach at the south end of the lake, shown in Figure 18. The outlet will be submerged. The existing force main, north of the new connection, will be cut and isolated from the new flow, and a concrete thrust block will be installed.

Figure 18: Pipe layout of the piping system at the Sand Lake outlet

Water for irrigation will be drawn from a well, disinfected, and then pumped to either Napoleon Park or the irrigation connection at TTRA. Water will leave the pump house at the Northeast corner and encounter a 6" – 4" reducing tee. Isolation gate valves will control the flow downstream to the branch direction (to TTRA), line direction (to Napoleon), or both. A flow control valve will be installed downstream along the branch direction and set to 75 gpm. This valve is needed in the event that both TTRA and Napoleon areas are irrigated simultaneously. Without this valve, most of the flow would go to TTRA, since the head lost in that branch of the system is less than the Napoleon branch. Higher flows are required at Napoleon, so a flow control valve is crucial to this design. 440 feet of 4" Class 200 PVC will carry water from the pump house to the irrigation connection at TTRA. 584 feet of 6" Class 200 PVC will carry water from the pump house back to the existing 8" force main, where it will then travel to Napoleon Park (Figure 19). 6" PVC was chosen in this situation over 4" in order to avoid significant flow velocity changes when transitioning into the 8" pipe and inducing sedimentation. The existing force main, south of the new connection, will be cut and isolated from the new flow. Detailed drawings can be found in sheet 02 of the attached plans.

Figure 19: Pipe layout of the piping system at the Terry Trueblood Recreation Area

Finally, 160 feet of 6" Class 200 PVC will extend from the existing 8" force main at Napoleon Park to the existing irrigation connection (Figure 20). The existing force main, north of the new connection, will be cut and isolated from the new flow.

Figure 20: Pipe layout of the piping system at Napoleon Park

5.3 Trench Details

Due to the susceptibility to a high water table, the trenches will need to be dug to a depth of 1 foot below the pipe invert and filled with a fine gravel in order to provide pipe stability. This bedding will be haunched to the pipe spring-line in order to increase the pipe supporting strength. Finally, the trench will be filled and compacted above the pipe spring-line to the surface grade. See sheet 06 in the attached plans for trench details. Thrust blocks are to be installed in order to resist the thrust force from pipe pressure and momentum change. Thrust blocks will be placed at new bend locations along the force main to protect the pipes from thrust failure.

Class 200 PVC pipe is recommended for the irrigation extensions, because it is cheap, light-weight, and durable. The irrigation pipes will be buried below the average frost line, 36”. These pipes will not carry water during the winter months, but they should be buried to the specified depth in order to protect them from heaving.

5.4 Well Details

Multiple goals are accomplished by pumping into sand lake. To start, it is keeping the lake at the proper water level. This is important because water will be drawn out of the lake daily for irrigation needs. Water will be drawn through a well, using the concept of a sand filter. The original idea was to use a sand point (driven point) well because it is simple and achieves our system goals. For a sand point well, a pipe with a driven point screen is inserted into the ground. The water will pump through the screen, and the screen will act as a filter, filtering out the sand and other unnecessary particles. However, driven point wells typically have a pipe with a 1-¼” or 2” diameter. Such small well diameters can only accommodate jet pumps, which cannot be sized to deliver 200 gpm. A well which would accommodate a submersible pump was needed. In order for the submersible pump specified to fit, the smallest diameter well required was 6”.

A licensed water well contractor with experience in Iowa will be required for the drilling of the well. The choice of the well site will affect many things such as the safety and performance of the well. The only distance requirement that has to be considered for this design is that it must be at least 11 feet from any existing buildings and 20 feet from any road or public highway. Once the site location is decided, the depth needs to be determined. The water well contractor will complete a formation log with soil and rock samples at the site and find the zone with the best potential for water supply. Generally, a well will be drilled to the bottom of the water table level to ensure the best production from the well.

There are many different types of wells, a board well and a drilled well, and the type used is determined based on the depth of the water table or the diameter of the pipe that will be used. The water table is 15.4 feet deep and the diameter of the well will be 6 inches. Due to the need for a smaller diameter, a drilled well will be used. A drilled well is often better because it is generally less susceptible to pollution from surface sources due to the depth. The well completion option that will be used is a sand screen with continuous slot openings. An example of what this would look like can be seen in Figure 21.

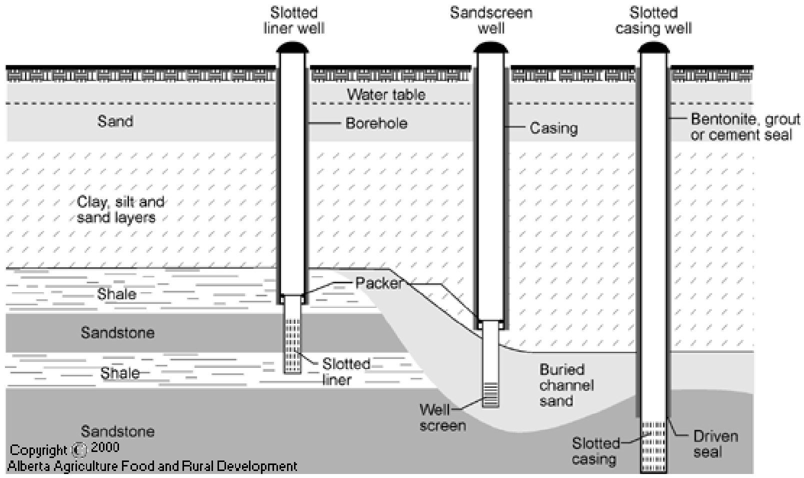

Figure 21: Example of the sandscreen well is shown in the middle of the three well designs

The casing of the well must be large enough to house the pump and allow sufficient clearance for installation and operation. The submersible pump is going to be used, therefore, the casing must have a diameter of at least 4 inches by law. However, it is recommended that the casing be at least one nominal size larger than the outside diameter of the pump. Since the outside diameter of the pump is 6 inches, the well casing will be 8 inches. A lower well casing is recommended to give the formation support and prevent entry of the aquifer material. A steel casing will be used due to its strength but this may create issues in the future due to the casing's susceptibility to corrosion. The length of the casing is required by law to be 40 feet long for a drilled well and should be seated in firm rock.

The well will also need to be sealed in order to protect it from contamination. The diameter of the borehole is usually slightly larger than the casing that is being installed. The borehole will be 13 inches, giving the space between the borehole and the casing, also called the annulus of the well, to be 5 inches. Based off of Iowa standards, the annulus should be 5 inches for a casing made from steel. The annulus is required to be filled in with grout. The grout should go down the full length of the casing, which is 40 feet. The grout will be concrete and based off of Iowa standards, the mixture should meet ASTM Standard C 494-92.

The water will move into the well after passing through a screen, which is manufactured to allow the maximum amount of water to penetrate with a minimal entry of formation sediments. A stainless steel screen will be used due to its strength and ability to withstand corrosive water. The diameter of the screen is determined by the casing of the well, therefore, it will be 8 inches. Due to the large intake of water in the well throughout the day, the slots of the screen should be a lower amount of open area. This allows the water to enter at a higher rate, increasing the pressure as the water moves into the well. However, if the water moves too quickly, there will be an incrustation build-up which will restrict the flow. The length of the screen depends on the thickness of the aquifer. The screen should be 60 feet from the surface and extends to the top of the aquifer. Therefore, the screen should be about 25 feet. See sheet 07 in the attached plans for a schematic of the well.

The final consideration that should be included in the design is a well cap. A well cap is used to keep animals, insects, and contaminants from entering the well. It comes equipped with rubber gaskets and screened steel vents to ensure vermin stay out and air can circulate through.

5.5 UV System

To disinfect the water prior to irrigation, a UV system will be implemented. Not only do UV systems effectively deactivate pathogens, but the system also leaves no disinfection byproducts in the water which are known to do more harm than good. It is important to note that the water leaving the WWTP has been treated to very high standards prior to its discharge into Sand Lake. Turbulence can affect the quality to which water is treated to as floating particles in water can block the UV from deactivating the pathogens. The use of the filter prior to UV treatment is vital to prevent this from occurring.

The chosen UV treatment system is the D3100K from Trojan Technologies. The D3100K is a two-bank system, with the additional bank ensuring redundancy. The system is open channel and is typically gravity fed. The head loss is only several inches from the top of the effluent weir elevation to the maximum upstream water level elevation. With a 200 GPM peak design flow, this system would be able to adequately handle the maximum peak flow required during irrigation times. The system has a minimum 65% UV transmittance and can handle a 30-day average TSS concentration of 30 mg/L . The complete system includes a Type 304 stainless steel channel, a module support rack, a level control weir, transition boxes, a monitoring system, a spare parts packages, an operator’s kit, and a maintenance rack. Each bank contains two stainless steel modules, each containing two UV lamps. Each lamp requires 87.5 Watts, with a total system power requirement of 700 Watts. A minimum UV dose of 31,000 $\mu Ws/cm^2$ will be delivered. The system guarantees an effluent standard of 200 / 100mL fecal coliform. The detailed schematic of the D3100K is provided in Figure 22.

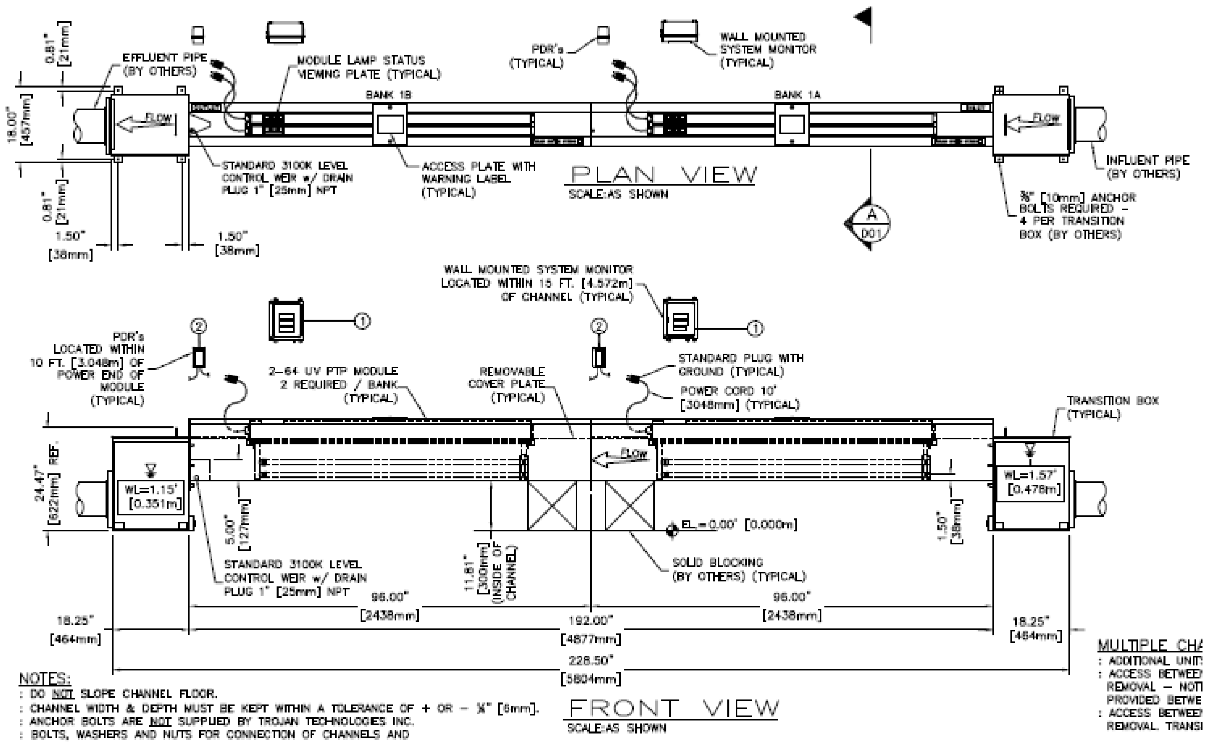

Figure 22: Layout of the UV System with all of its components.

5.6 Solar Panels

In order to power the UV system, solar energy will be harvested from the sun and converted into electricity. Solar power is a renewable resource and does not produce any pollutants, making it one of the cleanest sources of energy. To power the UV system at the recreation center, solar photovoltaics (PV) is used. Solar photovoltaics are typically used for larger installations. A photon strikes a semiconductor and generates electrons to produce a current. There are several things that are needed to go into a solar power system. The actual solar panels (solar cells), inverter, trunk cable, branch terminator, and a battery. The solar panels that were chosen were the Renogy 250W 24V Monocrystalline Solar Panels. At 250 Watts for one solar panel, three will be required for the amount of power required by the UV system. They weigh approximately 40 lbs each, totaling 120 lbs for the solar panel system. These solar panels are guaranteed positive output tolerance (0-3%) and withstand high winds (2400Pa) and snow loads (5400Pa). The monocrystalline is the most efficient kind of panel, proving the most efficiency at approximately 15.4% per space. These solar panels also perform excellently in low light environments.

In addition to the solar panels, inverters are required to invert the direct current (DC) output into alternating current (AC). AC is the standard used by all commercial appliances, therefore it is the gateway between the photovoltaic system and the energy. In addition to the inverter, a battery will be required. The batteries are important because they store the sun's energy throughout the day, and during times when there is less sunlight, the UV system will still be able to function. Rather than drawing from the solar cells, the system will draw from the battery. To hook up the entire system, a branch terminator and a trunk cable are required. Overall, the UV system will be entirely hooked up to the solar panels for electricity. In the case of some sort of problem or failure, the UV system can be hooked up to the grid as a backup. It will not use power from the grid unless absolutely necessary.

The solar panels can be installed next to the housing unit for the pump or on top of the housing unit. The panels would ideally be placed on top of the unit, however, a structural analysis is required to determine if the roof is capable of holding the additional load. A contractor would need to be hired to complete the structural analysis. If they are placed in the grass, a fence would be required to be placed around the solar panel system to ensure that people and animals stay away from the system. Another important consideration for the solar panels is the solar angle. The panels should be placed at the optimum angle, toward the sun, to get the maximum potential of the photovoltaic panels. This angle varies throughout the year depending on the season and location. During the winter, the solar panels should be placed at 24°. In spring and fall, the solar panels should be at a 48° angle. And finally, during summer, the solar panels should be at 72°. When installing the solar panels, it is important to add some sort of device that the panels can lay on that can easily change the angle that the solar panels lay.

5.7 Pipeline Cleaning

The method we choose to clean the pipeline is air scouring. The pipeline is rather old, so we are not willing to choose the method of pigging, as pigs can break the pipeline during the process of cleaning. There is about 40% less water used during air scouring than during swabbing or flushing. The estimated cost for air scouring in the year 2012 was $1.68/m while estimated cost for swabbing was $29.80/m (ESC, 2013), so the preferred method of cleaning the pipeline is through air scouring.

Air scouring is a process in which filtered compressed air (oil free) is injected into an isolated section of the water main to propel a small volume of water at high velocity, which is greater than the minimum velocity required to remove suspended sediments.

Procedure:

1. Before air scouring, numerous tests are performed using the Hazen-Williams “C” value test.
2. A section of the water mains is isolated between the point of entry and exit.
3. The isolated section is cleared of standing water by using high velocity, low pressure and high volume air.
4. The valve present upstream of the entry point is opened. The filtered compressed air mixes with water, creating a vortex that travels through the isolated section removing sediments.

5. Air scouring utilizes a mix of water and air called a slug flow. Given a constant supply of air and water, the slug flow can strip loose deposits and some slime from the pipes.

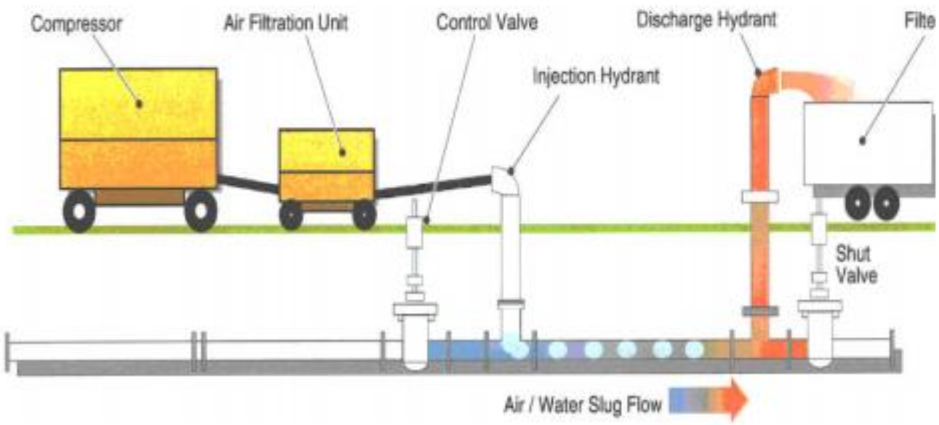

Figure 23: Typical arrangement of air scouring operations

Air scouring equipment includes an air compressor, air cooler, and filtering system. The compressor is sized to the pressure rating of the filtration unit. A 175 cubic feet per minute (cfm) compressor is the minimum requirement to undertake air scouring operation. The volume of compressed air entering the main is controlled by a series of operating valves, including a pressure-regulating valve. The compressed air first passes through an air cooler, which greatly reduces the amount of oil passing through the system in the vapor phase. The temperature generated from the compressor is typically 60°C, which is reduced to 20°C after the air passes through the air cooler. The air passes through a series of three filters: a pre-filter (1 micron- the depth filter removes oil and water vapor), a sub-micron filter (0.1 micron- the depth filter removes any bacteria, dirt and dust particles as well as remaining oil and water vapor), and an activated carbon filter (removes any remaining oil vapor, oil and related taste and odors). Air filtered to breathing standards is then injected into the main to start the cleaning process.

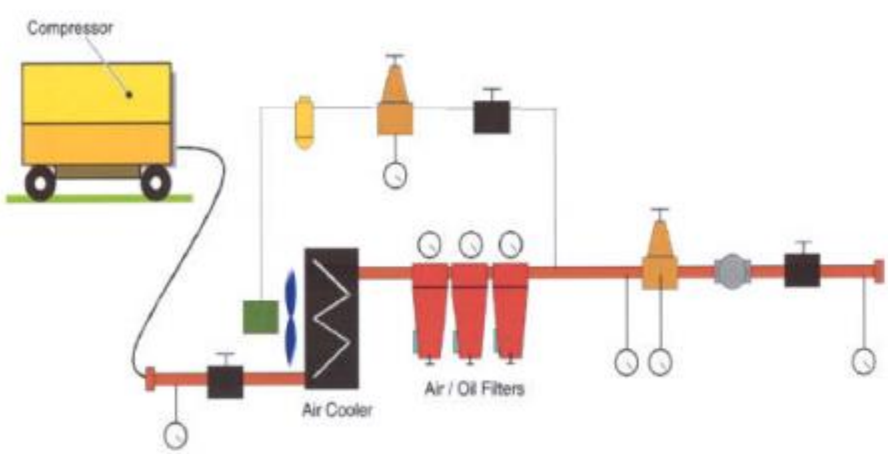

Figure 24: Arrangement of equipment for air scouring

Advantages of Air Scouring:

- Air scouring generally about one third the cost of pigging or swabbing.
- About 40% less water is used during air scouring than during swabbing or flushing
- The likelihood of a pipe break is very low as air pressure is kept below the static operating pressure of the pipe.

5.8 Strength Analysis of Reinforced Concrete Slab

The following breakdown shows that the slab is found to be adequate to support the pump as well as the associated loads from self-weight.

- **Flexural Strength:** Capacity of the slab is 139.008 kip-ft while the requirement is only 70 kip-ft, setting the utilization ratio at 0.504. Because the strength utilization ratio for moment is below 1, the slab is adequate to support the flexural stress imposed.
- **Shear Strength**: Capacity of the slab is 60.263 kip while the requirement is only 13.559 kip, setting the utilization ratio at 0.225. Since the requirement is less than 50% of the capacity, minimum shear reinforcement/ties per code will be sufficient and will not be designed. Because the strength utilization ratio for shear is below 0.5, the slab with minimal shear reinforcement is adequate to support the shear stress imposed.
- **Deflection Check**: Allowable deflection is 0.72 in while the actual deflection at service level is only 0.201 in. The deflection is, thus, within acceptable limits. Because the deflection ratio at service level is below 1.0, the slab passes the serviceability criteria.

5.9 Other Considerations

Chapter 567.62 of the Iowa Administrative Code specifies a few specific criteria are met when reusing treated wastewater. These criteria are aimed at golf course irrigation, but should be applied to the TTRA situation as well.

1. Signs must be posted which warns against consumption of water.
2. All piping and sprinklers should be color coded or tagged to warn against consumptive use.
3. Access must be restricted to areas while they are being irrigated.

6.0 Cost and Construction Estimates

6.1 UV System Cost

The D3100K is budget priced at $32,695.00. This cost includes all the aforementioned parts of the complete system package.
Water disinfection would only occur when irrigation is necessary- which for this site is only seven months (late April through mid-October). Irrigation is also sporadic, and only occurs for 9 of 24 hours each day. The system has a maximum power draw of 700 watts. Assuming the

maximum power draw occurs for the same number of minutes that irrigation occurs, and the price of electricity is $0.06 kWh, the system would only cost about $63 to run each year.

6.2 Well Cost
A complete breakdown of well cost is provided in table 2.

6.3 Solar Panels Cost
With the use of solar panels, the energy cost of the UV system would be negated ($63/year). With a total solar panel cost of $1,913, it would take solar panels about 30 years to produce enough power to justify their installation. This approaches the panels' design life. With possible sustainability tax credits, the solar installation could save the city money.

6.4 Pipeline Cleaning Cost
The length of pipeline that need to be cleaned is 9385 ft, and estimated cost for the mains air scouring was $1.68/m (ESC,2013). The labor cost per hour is $40. The pipeline cleaning can be completed within one day.

6.5 Pump House Cost
The cost estimate of pump building is $11,273.29. See Appendix C for a detailed cost breakdown.

1. Based on pump building drawings and CE Standard Method of Measurement, base quantities are derived describing major work to be done in the project. Consumables cost, which involves perishable raw materials, will be taken as a percentage of the material cost. Total cost of materials is $6,234.
2. The labor hours for each activity will be based on the base quantities using typical productivity from construction data. The labor cost per hour for each trade was taken from national publications on labor rate. Total labor cost is $4,416.
3. Overhead costs will be based on the material cost. These costs will be for supervision and operational costs at site, which is very minimal. Total overhead cost is $ 624.

6.6 Hydraulics Cost
The cost estimation of the Simflo SS12C Vertical Turbine Pump was provided by Quality Flow Iowa, Inc., and includes the pump, motor, and freight. The cost of the submersible pump and centrifugal irrigation supply pump are based on published data in RS Means catalogues (2015), which categorizes pumps by the motor power and type.
Piping, valves, and excavation costs were also obtained from the literature published by RS Means, and include overhead and profit.

Table 2: Complete project cost breakdown

Pumps	Count	$ / Unit	Total $
Simflo SS12C VTP	1	$41,864	$41,864
Goulds 160L03 Submersible Pump	1	$3,025	$3,025
Goulds Close-coupled Ag-Flo Centrifugal Pump	1	$9,650	$9,650
		Pump Total Cost	$54,600
Energy	Power [Hp]	$ / kW-hr	Total $/yr
Simflo SS12C VTP	201	$0.06	$78,840
Goulds 160L03 Submersible Pump	3	$0.06	$201
Goulds Close-coupled Ag-Flo Centrifugal Pump	23	$0.06	$1,541
UV	0.94	$0.06	$63
		Energy Total Cost	$80,700
Solar	Count	$/ Unit	Total $
Renogy 250W 24V Monocrystalline Solar Panel	3	$240	$720
Enphase On-Grid Micro Inverter M215	3	$167	$501
240VAC Portrait Enphase Trunk Cable	3	$23	$68
Enphase Branch Terminator	3	$84	$252
Vmax 12V 125 AH AGM Solar Charge Tank	2	$270	$540
		Total Solar Cost	$2,100
Piping*	LF	$ / LF	Total $
8" DIP, Push-on Joints	584	42.5	$24,820
6" Class 200 PVC, Push-on Joints	744	12.95	$9,635
4" Class 200 PVC, Push-on Joints	440	8.85	$3,894
		Total Piping Cost	$38,400
Valves & Fittings*	Count	$ / Unit	Total $
Swing Check Valve, 8" Diameter	1	2475	$2,475
Gate Valve, 8" Diameter	1	2850	$2,850
Swing Check Valve, 6" Diameter	1	1475	$1,475
Gate Valve, 6" Diameter	1	1800	$1,800
Gate Valve, 4" Diameter	1	1275	$1,275
Flow Control Valve, 6" Diameter	1	4029	$4,029
90° Bend, 8" Diameter	2	705	$1,410
45° Bend, 8" Diameter	3	530	$1,590
45° Bend, 6" Diameter	1	395	$395

45° Bend, 4" Diameter	4	62	$248
8"-6" Reducer	3	530	$1,590
6"-4" Reducing Tee	1	131	$131
4"-3" Reducer	1	75	$75
8"-3" Reducer	1	355	$355
8"-1.5"Reducer	1	385	$385
8" Thrust Block	2	98	$196
		Total Valves & Fitting Cost	$20,300
Excavation*	C.Y.	$/C.Y.	Total $
5'-8" deep x 2'-8" wide Trench (1/2 C.Y. Excavator)	327	6.85	$2,239
Gravel Bedding, compacted	58	57.8	$3,334
Backfill	250	13.6	$3,397
	L.F	$/L.F.	Total $
3' deep x 1'-6" wide Trench + Backfill	1184	1.64	$1,942
		Total Excavation Cost	$11,000
	Count	$ / Unit	Total $
UV	1	$32,695	$32,700
Well*	Count	$ / Unit	Total $
8" Well Screen	1	$360	$360
8" Well Casing	60	$13	$750
Drill Hole	85	$13	$1,020
Well Cap	1	$50	$50
Grouting	1	$110	$110
		Total Well Cost	$2,300
Pipe Cleaning			Total $
Air compressor @ 185 cfm			$7,000
Air filtration unit			$1,500
Model 45 - Electric actuated butterfly valve			$4,112
C903 Flow control valve			$5,805
Cleaning pipeline			$4,788
Labor			$2,560
		Total Pipe Cleaning Cost	$25,800
Pump House**			Total $
Materials			$6,234
Labor			$4,416
Overhead			$623

Total Pump House Cost	$11,300
Total Cost	$198,500
	+ $ 80,700/yr

7.0 Conclusions

SWORD Engineering was provided with the task of designing a water reclamation system to supply irrigation water for the Terry Trueblood Recreation Area from the Iowa City South Waste Water Treatment Plant in Iowa City, IA. The need for treatment of a calculated quantity of water, as well as an investigation into permitting, was evaluated. A force main extension from the decommissioned wastewater treatment plant was designed, and pumping structures and sizing were determined. Looking into environmental, societal, and economic impacts, a design matrix was created and presented to the client. The client's' personal desires in regards to the goals of the project took precedence.

The final design recommended to the Terry Trueblood Recreation Area includes:

1. Air scouring to clear the retired pipe of any bacteria, dirt, dust particles
2. A Simflo SS12 Vertical Turbine Pump to pump water from the effluent holding tanks
3. Various lengths of 8-inch diameter ductile iron pipe, and 4-inch and 6-inch diameter Class 200 PVC pipe

a. Trenches dug to a depth of 1 foot below the pipe invert

1. Installation of a drilled well with a well cap
2. A Goulds 160L03 Submersible Pump to pump water into the UV system
3. Installation of an open-channel, two-bank D3100K UV system
4. Three (3) Renogy 250W 24V Monocrystalline Solar Panels with inverters to convert the DC output into AC, and batteries to store power during low-light hours
5. A Goulds Close-coupled Ag-Flo Centrifugal Pump within the pump house which will transmit water from the UV effluent reservoir to Napoleon Park and TTRA

This design accomplishes all of the client’s goals while also remaining sustainable. For this system to remain in compliance of standards set by the IDNR, it is vital for Sand Lake to be constantly monitored for various nutrient levels, as well as DO levels. If levels of nitrogen and phosphorous can be decreased to a lower concentration prior to leaving the ICSWWTP, less strict monitoring would be required.

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9.0 Appendices

Appendix A: Hydraulic Calculations

The total dynamic head (TDH) that a pump must overcome to deliver a given flow rate through a system is obtained by the sum of three factors.

TDH [ft]= $h_f+h_m+h_s$ Equation A.1

Where h_f is the friction loss induced by interaction with the pipe wall; h_m is the minor losses due to momentum changes and turbulence accrued at valves and fittings, and h_s is the static head, which includes elevation difference and required pressure at the tap.

The friction loss is a function of the flow rate and pipe dimensions and can be calculated using the Hazen-Williams equation.

$$hf\ [ft] = \frac{4.727LQ^{1.852}}{C^{1.852}D^{4.87}}$$ Equation A.2

Where L is the pipe length [ft.], Q is the flow rate [cfs], C is the Hazen-Williams coefficient, and D is the pipe diameter [ft]. The friction head loss calculations through the force main between the Iowa City South WWTP and Sand Lake are tabulated in Table A.1.

Table A.1: Friction head losses through the force main between the Iowa City South WWTP and Sand Lake.

Pipe	**Diameter [in]**	**Length [ft]**	**Material**	**H-W Coeff***	**Q [gpm]**	**h_f [ft]**
WWTP to Existing FM	8	239	DI	100	1400	13.2
FM @ WWTP to FM @ Lake	8	8030	DI	100	1400	444.5
FM @Lake into Lake	8	345	DI	100	1400	19.1

*IDNR, 2015

Minor losses through valves and fittings are calculated by the energy equation.

$$h_m[ft] = \sum K \frac{V^2}{2g}$$ Equation A.3

Where K is the minor loss coefficient, V is the flow velocity [ft/s], and g is the gravitational constant [ft/s/s]. The minor head loss calculations through the force main between the Iowa City South WWTP and Sand Lake are tabulated in Table A.2

Table A.2: Minor head losses through the force main between the Iowa City South WWTP and Sand Lake.

Fitting	**Quantity**	**K**	**K_{eq}**	**Q [gpm]**	**V [ft/s]**
45° Bend	4	0.2	0.8	1400	8.9
11°15' Bend	1	0.1	0.05	1400	8.9
Tee, Line Flow	1	0.9	0.9	1400	8.9
22°30' Bend	1	0.1	0.1	1400	8.9
Gate Valve	3	0.2	0.45	1400	8.9
Swing Check Valve	1	2.0	2	1400	8.9

Exit	1	1.0	1	1400	8.9
h_m = 6.6 ft					

The static head is -8 feet. The lake is actually lower in elevation than the effluent holding tank, and zero pressure is required at the terminus of the force main.

Therefore, the total dynamic head, at 1400 gpm, is 13.2+ 444.5 + 19.1 + 6.6 - 8 = 471 feet.

A similar analysis was performed on the well pump and irrigation supply pump (Tables A.3 –A.7)

Table A.3: Submersible well pump hydraulic calculations.

∆El. [ft]	Diameter [in]	Material	H-W Coeff	Q [gpm]	h_f [ft]	TDH [ft]
30	6	PVC 200	150	200	0.1	30.1

Table A.4: Minor Loss coefficients associated with the irrigation supply system.

To Napoleon	Fittings	Quantity	K	K_{eq}
	45° Bend	2	0.2	0.4
	Swing Check Valve	1	2.0	2.0
	Isolation Valve	1	0.2	0.2
	6-8 Expander (10°)	1	0.1	0.1
	8-6 Reducer (10°)	1	0.1	0.1
	Flow control valve	1	10.0	10.0
	Tee, line flow	1	0.2	0.2
To TTRA	Swing Check Valve	1	2.0	2.0
	Isolation Valve	1	0.2	0.2
	45° Bend	4	0.2	0.8
	Tee, branch flow	1	1.0	1.0
	4-3 Reducer (10°)	1	0.1	0.1

Table A.5: Friction head losses through the system between the well and Napoleon Park.

Pipe	Diameter [in]	Length [ft]	Material	H-W Coeff	Q [gpm]	h_f [ft]
Well to Existing FM	6	584	PVC 200	150	125	0.7
FM @ Bend to Napoleon	8	2560	DI	100	125	1.6
FM @ Napoleon to Napoleon	6	160	PVC 200	150	125	0.2

Table A.6: Friction head losses through the system between the well and the irrigation connection at the lodge in the Terry Trueblood Recreation Area.

Pipe	Diameter [in]	Length [ft]	Material	H-W Coeff	Q [gpm]	h_f [ft]
Well to TTRA	4	440	PVC 200	150	75	1.5

A.7: Irrigation supply pump hydraulic calculations.

Destination	Σh_f	ΔEl. [ft]	ΣK	h_m [ft]	Pressure [ft]	Irrigation Loss [ft]	TDH [ft.]
To Napoleon	2.5	12.6	12.86	0.1	208	50	286.1
To TTRA	3.8	-5	10.31	0.0	208	50	267.1
To Both	2.5	12.6	12.9	0.1	208	50	286.1

Appendix B: Lake Study Calculations

To consider the worst case scenario in regards to biological oxygen demand (BOD) for the lake, a DO concentration of 0 mg/L for the wastewater effluent was assumed. This leaves 4 mg/L (taking high value) of BOD in the effluent stream entering the lake. To see how this affects the overall dissolved oxygen level in the lake, the initial lake DO concentration needs to be determined. The average high and low temperature for each month of the year was found, and used to calculate the average temperature using the following relationship:

Average Temperature = (High Temperature + Low Temperature) / 2

From this temperature, the saturated dissolved oxygen level, or the maximum concentration of dissolved oxygen possible, in the lake can be found. These values are summarized in Table B.1.

Table B.1: Summary of DO_{sat} determination

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
High Temp (^{0}F)	32	37	50	65	75	83	87	85	78	66	50	35
Low Temp (^{0}F)	14	18	29	40	51	61	65	63	54	42	31	18
Avg. Temp (^{0}F)	23	27.5 (28)	39.5 (40)	52.5 (53)	63	72	76	74	66	54	40.5 (41)	26.5 (27)
Avg. Temp (^{0}C)	-5	-2.22	4.44	11.7	17.2	22.2	24.4	23.3	18.9	12.2	5	-2.78
DO_{sat} (mg/L)	14.19*	14.19*	13.09	10.76	9.65	8.72	8.40	8.56	9.26	10.76	12.75	14.19*

As can be seen from Table B1, the lowest DO concentration occurs in July, 8.40 mg/L. This is assuming that the dissolved oxygen level in the lake reaches steady state at the saturated condition. Once this has been found, the total mg of DO in the lake can be calculated. The total area of the lake was 95.5 acres and the depth was 9 feet. The lake was assumed to be a cylinder in order to simplify calculations. With this assumption, the total volume of the lake was found:

$$\text{Volume of lake (ft}^3\text{)} = 95.5 \text{ acres} * (43{,}560\text{ft}^2/1 \text{ acre}) * 9\text{ft} = 37{,}439{,}820\text{ft}^3$$

This, along with the DO level of the lake, was used to find the mg of DO in the lake:

$$\text{DO (mg)} = 8.40 \text{ mg/L} * 37{,}439{,}820\text{ft}^3 * (28.3168\text{L}/1\text{ft}^3) = 8{,}905{,}477{,}518 \text{ mg}$$

The effluent would have a BOD concentration of 4.0 mg/L, and a flow of 2 MGD. Using this information, an equation modeling the BOD in mg entering the lake per day was calculated:

$$\text{BOD (mg/d)} = 4 \text{ mg/L} * 2 \text{ MGD} * (106\text{gal/MG}) * (3.785\text{L}/1 \text{ gal}) = 30{,}280{,}000 \text{ mg/d}$$

Subtracting this BOD from the total dissolved oxygen in the lake, the dissolved oxygen remaining in the lake was found for each day. This was assuming that no source of oxygen was entering the lake during this time. This remaining level can be converted to mg/L by dividing by the total volume of the lake. A summary of this level per day is shown in Table B.2:

Table B.2: Summary of the levels per day of Dissolved Oxygen

Time (days)	DO Level in lake (mg/L)	Time (days)	DO Level in lake (mg/L)	Time (days)	DO Level in lake (mg/L)
0	8.40	110	5.26	220	2.12
10	8.11	120	4.97	230	1.83
20	7.83	130	4.69	240	1.55
30	7.54	140	4.40	250	1.26
40	7.26	150	4.12	260	0.97
50	6.97	160	3.83	270	0.69
60	6.69	170	3.54	280	0.40
70	6.40	180	3.26	290	0.12
80	6.12	190	2.97	295	0.00
90	5.83	200	2.69		

100	5.54	210	2.40		

A plot of the decrease overtime is shown in Figure B.3.

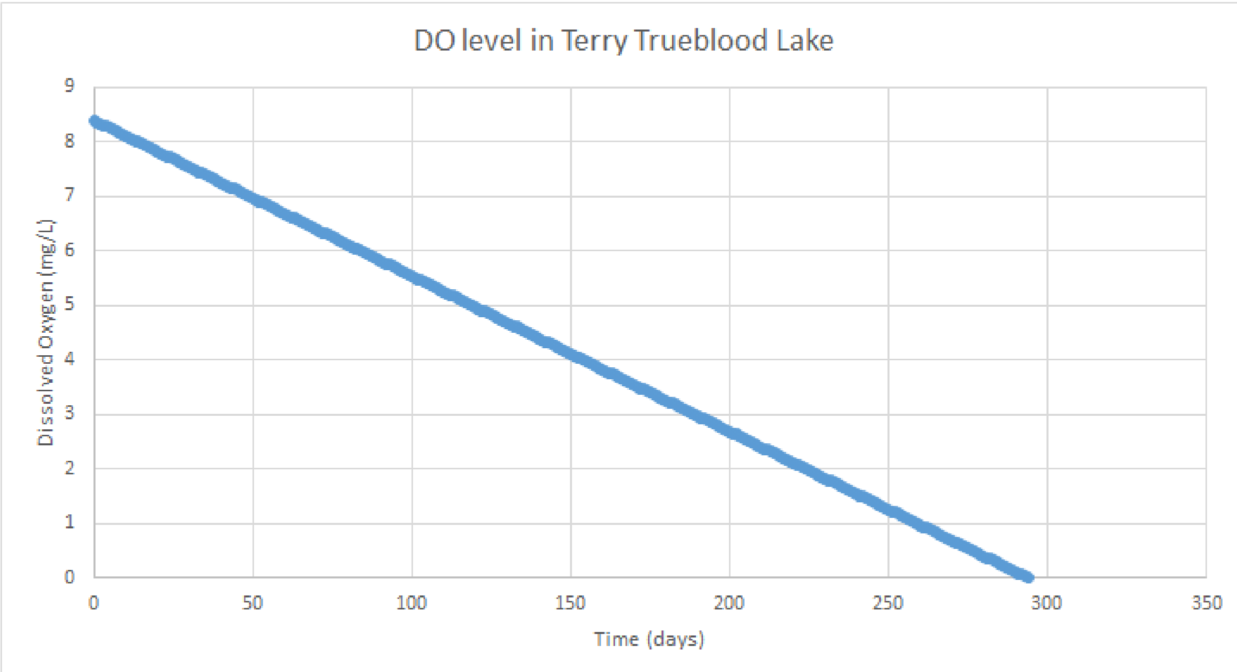


Figure B.3: A plot of the decrease of dissolved oxygen over time

The phosphorus level in the lake over time was also calculated. The concentration of phosphorus in the wastewater effluent for the ICWWTP is 1-2 mg/L. 1 mg/L was assumed for this study so that the minimum effect could be shown. The total phosphorus entering the lake in mg/d was found using the relation:

Phosphorus in lake (mg/d)=1 mg/L *2 MGD *(106gal/1 MG)*(3.785 L/1 gal) = 7,570,000mg/d

This represents the increase in phosphorus in the lake per day. This can be converted to mg/L the same as the DO level, by dividing by the total lake volume. A summary of this data is shown in Table B.4:

Table B.4: Summary of the Phosphorous data in the lake

Time (days)	Inc. in P (mg/L)	Time (days)	Inc. in P (mg/L)	Time (days)	Inc. in P (mg/L)	Time (days)	Inc. in P (mg/L)
0	0.00	100	0.71	200	1.43	300	2.14
10	0.07	110	0.79	210	1.50	310	2.21
20	0.14	120	0.86	220	1.57	320	2.28
30	0.21	130	0.93	230	1.64	330	2.36
40	0.29	140	1.00	240	1.71	340	2.43
50	0.36	150	1.07	250	1.79	350	2.50
60	0.43	160	1.14	260	1.86	360	2.57
70	0.50	170	1.21	270	1.93	365	2.61
80	0.57	180	1.29	280	2.00		
90	0.64	190	1.36	290	2.07		

A plot of this data is shown in Figure B.5.

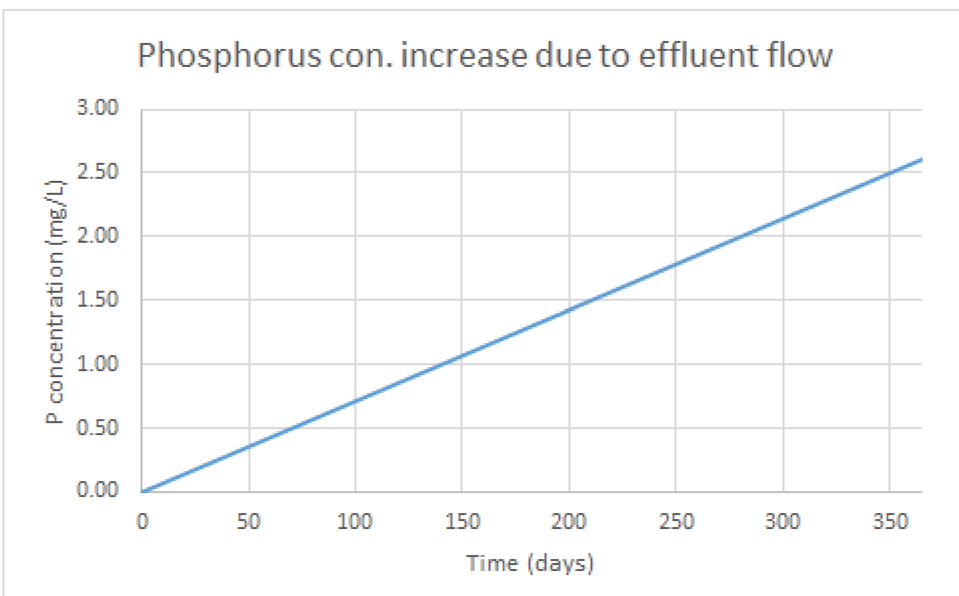


Figure B.5: Plot of the phosphorus increase due to effluent flow in the lake.

This analysis was repeated assuming an effluent concentration of 0.1 mg/L. A summary of this data is shown in Table B.6.

Table B.6: Phosphorus data in the lake assuming effluent concentration of 0.1 mg/L

Time (days)	Inc. in P (mg/L)	Time (days)	Inc. in P (mg/L)	Time (days)	Inc. in P (mg/L)	Time (days)	Inc. in P (mg/L)
0	0.00	100	0.07	200	0.14	300	0.21
10	0.01	110	0.08	210	0.15	310	0.22
20	0.01	120	0.09	220	0.16	320	0.23
30	0.02	130	0.09	230	0.16	330	0.24
40	0.03	140	0.10	240	0.17	340	0.24
50	0.04	150	0.11	250	0.18	350	0.25
60	0.04	160	0.11	260	0.19	360	0.26
70	0.05	170	0.12	270	0.19	365	0.26
80	0.06	180	0.13	280	0.20		
90	0.06	190	0.14	290	0.21		

A plot of this data is shown in Figure B.7.

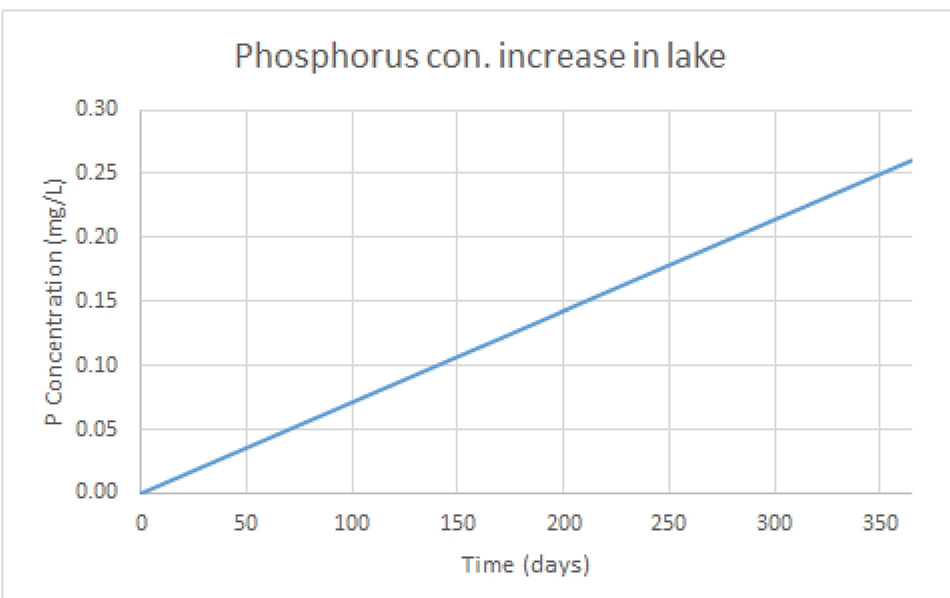


Figure B.7: Plot of the phosphorus increase due to effluent flow in the lake assuming effluent concentration of 0.1 mg/L

The nitrogen increase due to both ammonia nitrogen and total nitrogen was analyzed over time due to effluent supply to the lake. The ammonia nitrogen level in the ICWWTP effluent was less than 1 mg/L. For the analysis, a conservative value of 1 mg/L was assumed. The increase of ammonia nitrogen per day can then be found using the following equation:

NH3-N (mg/d) = 1 mg/L * 2 MGD * (106gal/1 MG)*(3.785 L/1 gal) = 7,570,000 mg/d

This can then be converted to mg/L by dividing by the total lake volume. A summary of the mg/L increase over the course of a year is summarized in Table B.8.

Table B.8: Summary of nitrogen due to ammonia data in lake.

Time (days)	Inc. in NH_3-N (mg/L)	Time (days)	Inc. in NH_3-N (mg/L)	Time (days)	Inc. in NH_3-N (mg/L)	Time (days)	Inc. in NH_3-N (mg/L)
0	0	100	0.71	200	1.43	300	2.14
10	0.07	110	0.79	210	1.50	310	2.21
20	0.14	120	0.86	220	1.57	320	2.28
30	0.21	130	0.93	230	1.64	330	2.36
40	0.29	140	1.00	240	1.71	340	2.43
50	0.36	150	1.07	250	1.79	350	2.50
60	0.43	160	1.14	260	1.86	360	2.57
70	0.50	170	1.21	270	1.93	365	2.61
80	0.57	180	1.29	280	2.00		
90	0.64	190	1.36	290	2.07		

A plot of the data can be seen in Figure B.9.

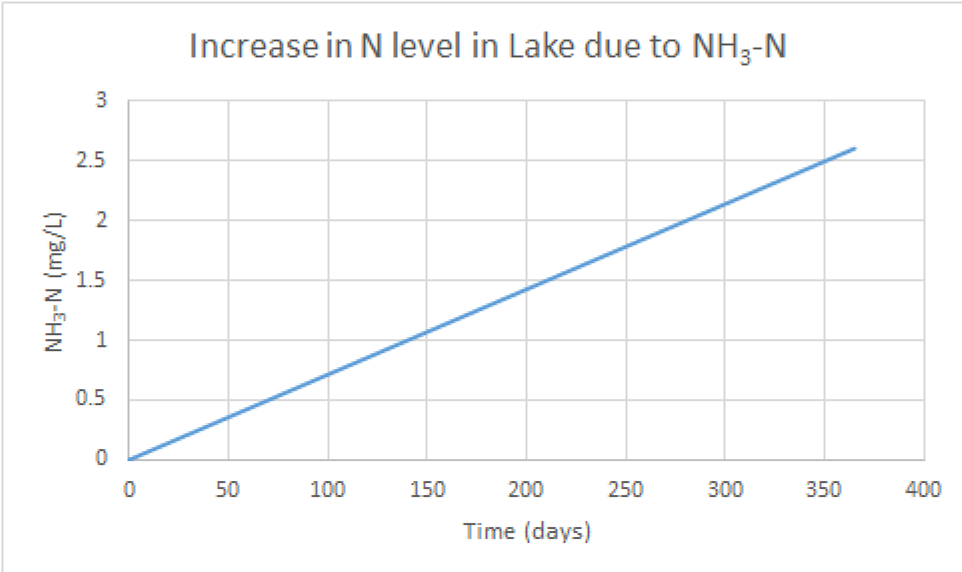


Figure B.9: Plot of increase of nitrogen level in lake due to effluent ammonia concentration

This same analysis was completed for the total nitrogen increase in the lake. The effluent total nitrogen for the ICWWTP is 13.4 mg/L. Using the same calculation done for the NH_3-N analysis, the inflow of total nitrogen into the lake is 101,438,000 mg/d. A summary of the daily increase in mg/L total N for the lake is shown in Table B.10.

Table B.10: Summary of nitrogen due to total nitrogen data for lake

Time (days)	Inc. in Total N (mg/L)	Time (days)	Inc. in Total N (mg/L)	Time (days)	Inc. in Total N (mg/L)	Time (days)	Inc. in Total N (mg/L)
0	0.0	100	9.57	200	19.14	300	28.70
10	0.96	110	10.52	210	20.09	310	29.66
20	1.91	120	11.48	220	21.05	320	30.62
30	2.87	130	12.44	230	22.01	330	31.57
40	3.83	140	13.40	240	22.96	340	32.53
50	4.78	150	14.35	250	23.92	350	33.49
60	5.74	160	15.31	260	24.88	360	34.44
70	6.70	170	16.27	270	25.83	365	34.92
80	7.65	180	17.22	280	26.80		
90	8.61	190	18.18	290	27.75		

A plot of this data is shown in Figure B.11.

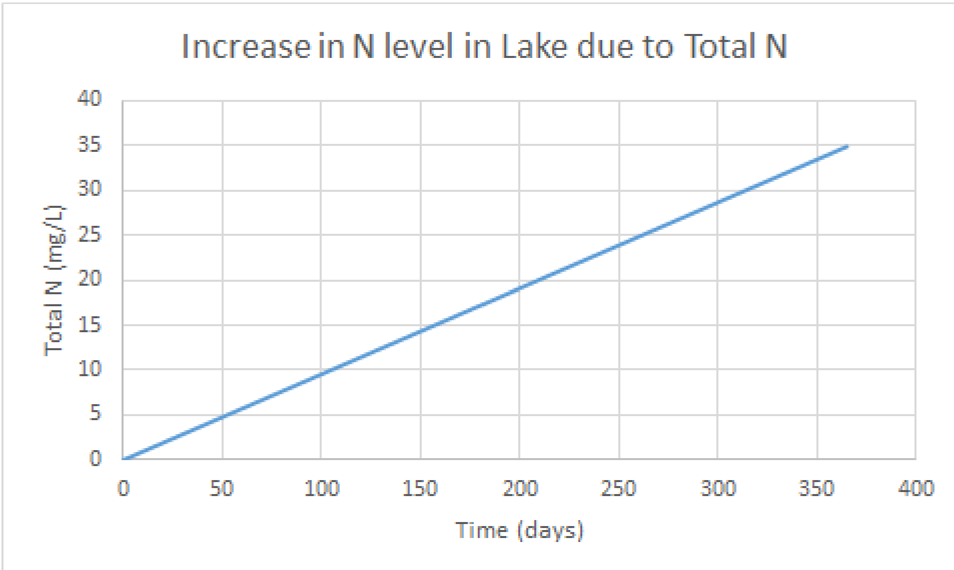


Figure B.11: Plot of increase in nitrogen levels in lake due to effluent total nitrogen concentration

Appendix C: Cost estimate of pump building

Table C.1. Quantity Takeoff

MATERIAL	QTY	UNIT	REMARKS	WASTAGE	GROSS QTY
Roof					
5/8" Asphalt Shingles	16.33	ft	length		
	5.67	ft	width		
	2.00	sides			
	185.18	sq.ft	area	10%	203.70
3/8" plywood sheathing	185.18	sq.ft	area	17%	216.66
1" fiberglass insulation	15.00	ft	length		
	10.00	ft	width		
	150.00	sq.ft	total insulation area	17%	175.50
5/8" gypsum ceiling	150.00	sq.ft	total ceiling area	17%	175.50
2x3 Douglas Fir-Larch #3	5.81	ft	webs length		

- Web					
	9.00	trusses			
	52.33	ft	total timber length	18%	61.74
2x4 Douglas Fir-Larch #3 - Chord	21.34	ft	top and bottom chord		
	9.00	trusses			
	192.06	ft	total timber length	18%	226.63
Walls					
6" Concrete	15.00	ft	building length		
	10.00	ft	building width		
	10.00	ft	building height		
	6.00	in	wall thickness		
	9.26	cu.yd	total concrete volume	10%	10.19
Formworks	1,000.00	sq.ft	total formworks area	25%	1,250.00
Reinforcing Bars	540.00	ft	vert bars length		
	500.00	ft	horiz bars length		
	1,040.00	ft	total bar length	5%	1,092.00
	1,085.72	lbs	total bar weight	5%	1,140.00
Brick Wall	500.00	sq.ft	exterior wall area		
	8.00	ft	interior wall length		
	10.00	ft	interior wall height		
	80.00	sq.ft	interior wall area		
	580.00	sq.ft	total brick wall area	0%	580.00
6x6 Douglas Fir-Larch #1 - Post	10.00	ft	height		
	4.00	ea	number		
	40.00	ft	total	18%	47.20
Doors	2.00	ea	number of doors	0%	2.00

Table C.2: Labor Estimate

DESCRIPTION	ACTIVITY	QTY	UNIT	PRICE/HR	LABOR COST
CARPENTRY	Roofing	8.00	hours	$ 39.00	$ 312.00
	Wooden Truss	24.00	hours	$ 39.00	$ 936.00
	Ceiling	8.00	hours	$ 39.00	$ 312.00
	Formworks	16.00	hours	$ 39.00	$ 624.00
	Post	8.00	hours	$ 39.00	$ 312.00
MASON/BRICKLAYERS	Steelworks	16.00	hours	$ 40.00	$ 640.00
	Concreting	8.00	hours	$ 40.00	$ 320.00
	Bricklaying	16.00	hours	$ 40.00	$ 640.00
	Door Installation	8.00	hours	$ 40.00	$ 320.00
TOTAL					$ 4,416.00

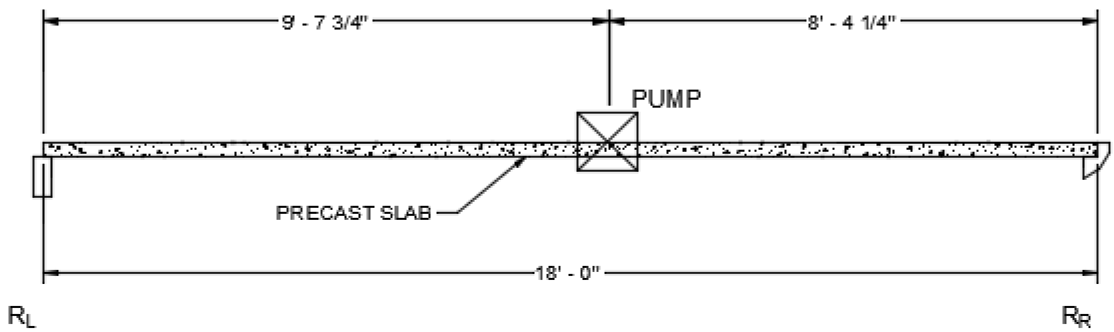
Table C.3. Bill of Materials and Total Project Cost

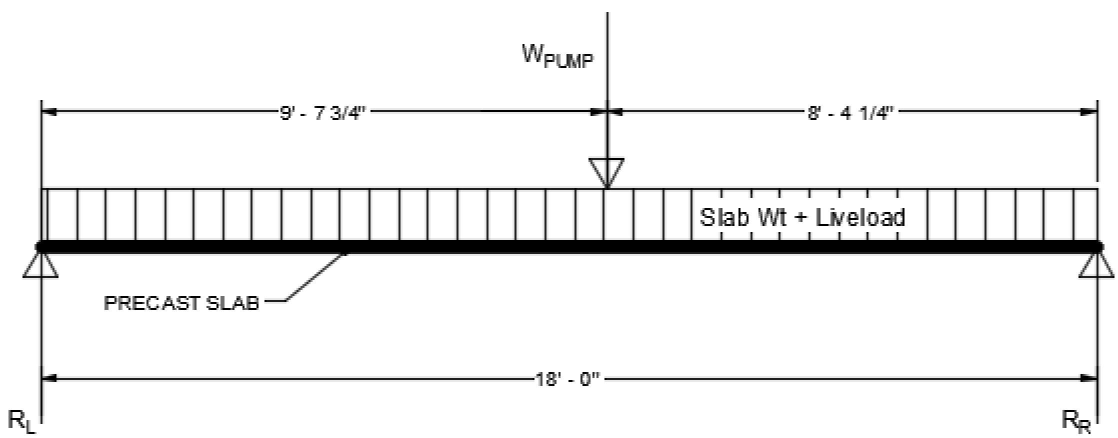
NO.	DESCRIPTION	QUANTITY	UNIT	PRICE PER UNIT	PRICE
1	Ready Mix Concrete (4000psi)	11.00	cu. yd	$ 95.00	$ 1,045.00
2	#5 Reinforcing Bar Grade 60 - 20ft	55.00	lengths	$ 16.52	$ 908.60
3	3/8" Plywood - 4'x8'	27.00	sheets	$ 17.35	$ 468.45
4	5/8" Gypsum Board - 4'x8'	6.00	sheets	$ 7.25	$ 43.50
5	Fiberglass Insulation	6.00	rolls	$ 14.65	$ 87.90
6	Roof Shingles - 32.8 sq ft Bundle	7.00	bundles	$ 25.00	$ 175.00
7	2x3 Douglas Fir-Larch #3 - 10'	7.00	lengths	$ 4.00	$ 28.00
8	2x4 Douglas Fir-Larch #3 - 12'	19.00	lengths	$ 5.00	$ 95.00
9	6x6 Wood Post - 10'	4.00	lengths	$ 30.00	$ 120.00
10	Modular Brick - 3-5/8"x2-1/4"x7-5/8"	3,979.00	each	$ 0.55	$ 2,188.45
11	Heavy Duty Door	2.00	each	$ 150.00	$ 300.00
12	Consumables (Nails, Wires, etc)	1.00	lot	$ 774.00	$ 774.00
13					
13	MATERIALS TOTAL				$ 6,233.90

14	LABOR TOTAL				$ 4,416.00
15	OVERHEAD AND CONTINGENCIES				$ 623.39
16					
17	TOTAL PROJECT COST				$11,273.29

Appendix D. Strength Analysis of Reinforced Concrete Slab

1. Sketch: The precast slab supported on wall of storage tank is idealized as a simply supported beam. The pump shall be a concentrated load on this beam.





2. Loads

$L_{pc} := 14\,ft - 3in + 0.5\cdot(8ft + 6in)$

$L_{pc} = 18\cdot ft$ length of precast slab

a. Dead loads:

Pump

$W_{pu} := 3730\ lbf$ weight of pump as provided

$b_{pu} := 8\,ft + 4.25\ in$ distance of pump from nearest support

$a_{pu} := L_{pc} - b_{pu}$

$a_{pu} = 9.646\cdot ft$ distance of pump from farthest support

Precast Slab - Self weight

$w_{pc} := 150\ pcf$ unit weight of normal weight concrete

$thk_{pc} := 10\ in$ thickness of slab

$width_{pc} := 4ft$ width of slab

$W_{pc} := w_{pc}\cdot thk_{pc}\cdot width_{pc}$

$W_{pc} = 0.5\cdot klf$ uniformly distributed load along slab length

b. Live load (LL)

$w_{LL} := 100\ psf$ live load for operating areas

$W_{LL} := w_{LL}\cdot width_{pc}$

$W_{LL} = 400\cdot plf$ live load uniformly distributed along beam length

3. Load Combinations

a. Load Summary

Dead loads (DL)/Fluid Loads (FL)

$$R_{DL.L} := \frac{W_{pc}\cdot L_{pc}}{2} + \frac{W_{pu}\cdot b_{pu}}{L_{pc}}$$

$R_{DL.L} = 6.231\cdot kip$ reaction at left support

$$R_{DL.R} := \frac{W_{pc}\cdot L_{pc}}{2} + \frac{W_{pu}\cdot a_{pu}}{L_{pc}}$$

$R_{DL.R} = 6.499 \cdot kip$ reaction at right support

$$V_{DL.MP} := \frac{W_{pc} \cdot L_{pc}}{2} + \frac{W_{pu} \cdot b_{pu}}{L_{pc}}$$

$V_{DL.MP} = 6.231 \cdot kip$ shear at midpoint

$x := \frac{b_{pu}}{L_{pc}} = 0.464$ ratio of pump distance to precast length

$$V_{DL.PU} := \left(\frac{1}{2} - x\right) \cdot W_{pc} \cdot L_{pc} + \frac{W_{pu} \cdot a_{pu}}{L_{pc}}$$

$V_{DL.PU} = 2.322 \cdot kip$ shear at pump location

$$M_{DL.MP} := \frac{W_{pc} \cdot L_{pc}^{\ 2}}{8} + \frac{W_{pu} \cdot b_{pu}}{2}$$

$M_{DL.MP} = 35.831 \cdot kip \cdot ft$ moment at midpoint

$$M_{DL.PU} := \frac{1}{2} x \cdot (1 - x) \cdot W_{pc} \cdot L_{pc}^{\ 2} + \frac{W_{pu} \cdot a_{pu} \cdot b_{pu}}{L_{pc}}$$

$M_{DL.PU} = 36.844 \cdot kip \cdot ft$ moment at pump location

Live loads (LL)

$$R_{LL.L} := \frac{W_{LL} \cdot L_{pc}}{2}$$

$R_{LL.L} = 3.6 \cdot kip$ reaction at left support

$$R_{LL.R} := \frac{W_{LL} \cdot L_{pc}}{2}$$

$R_{LL.R} = 3.6 \cdot kip$ reaction at right support

$V_{LL.MP} := 0 kip$

$V_{LL.MP} = 0 \cdot kip$ shear at midpoint

$$V_{LL.PU} := \left(\frac{1}{2} - x\right) \cdot W_{LL} \cdot L_{pc}$$

$V_{LL.PU} = 0.258 \cdot kip$ shear at pump location

$$M_{LL.MP} := \frac{W_{LL} \cdot L_{pc}^{2}}{8}$$

$M_{LL.MP} = 16.2 \cdot kip \cdot ft$ moment at midpoint

$$M_{LL.PU} := \frac{1}{2} x \cdot (1 - x) \cdot W_{LL} \cdot L_{pc}^{2}$$

$M_{LL.PU} = 16.117 \cdot kip \cdot ft$ moment at pump location

b. Load Combination per Code

By inspection, 2 load combinations (per [1]) at 4 points could be critical for strength analysis:

LC1: 1.4DL (at left support, right support, midpoint and pump location)
LC2: 1.2DL + 1.6LL (at left support, right support, midpoint and pump location)

For LC1:

$V_{LC1.L} := 1.4 \left(R_{DL.L}\right) = 8.724 \cdot kip$ ultimate shear at left support

$V_{LC1.R} := 1.4 \left(R_{DL.R}\right) = 9.098 \cdot kip$ ultimate shear at right support

$V_{LC1.MP} := 1.4 \left(V_{DL.MP}\right) = 8.724 \cdot kip$ ultimate shear at midpoint

$V_{LC1.PU} := 1.4 \left(V_{DL.PU}\right) = 3.25 \cdot kip$ ultimate shear at pump location

$M_{LC1.MP} := 1.4 \left(M_{DL.MP}\right) = 50.163 \cdot kip \cdot ft$ ultimate moment at midpoint

$M_{LC1.PU} := 1.4 \left(M_{DL.PU}\right) = 51.582 \cdot kip \cdot ft$ ultimate moment at pump location

For LC2 :

$F(DL, LL) := 1.2\, DL + 1.6\, LL$

$V_{LC2.L} := F\left(R_{DL.L}, R_{LL.L}\right) = 13.237 \cdot kip$ ultimate shear at left support

$V_{LC2.R} := F\left(R_{DL.R}, R_{LL.R}\right) = 13.559 \cdot kip$ ultimate shear at right support

$V_{LC2.MP} := F\left(V_{DL.MP}, V_{LL.MP}\right) = 7.477 \cdot kip$ ultimate shear at midpoint

$V_{LC2.PU} := F\left(V_{DL.PU}, V_{LL.PU}\right) = 3.199 \cdot kip$ ultimate shear at pump location

$M_{LC2.MP} := F\left(M_{DL.MP}, M_{LL.MP}\right) = 68.917 \cdot kip \cdot ft$ ultimate moment at midpoint

$M_{LC2.PU} := F\left(M_{DL.PU}, M_{LL.PU}\right) = 70 \cdot kip \cdot ft$ ultimate moment at pump location

c. Design Strength Requirements

$V_u := \max(V_{LC1.L}, V_{LC1.R}, V_{LC1.MP}, V_{LC1.PU}, V_{LC2.L}, V_{LC2.R}, V_{LC2.MP}, V_{LC2.PU})$

$V_u = 13.559 \cdot kip$ — shear strength requirement

$M_u := \max(M_{LC1.MP}, M_{LC1.PU}, M_{LC2.MP}, M_{LC2.PU})$

$M_u = 70 \cdot kip \cdot ft$ — flexural strength requirement

4. Strength Analysis

a. Concrete Data

$f_{cp} := 4000\ psi$ — 28-day compressive strength of concrete (assumed)

$E_c := 57000 \sqrt{f_{cp} \cdot psi}$

$E_c = 3.605 \times 10^6 \cdot psi$ — modulus of elasticity of concrete

$F_y := 60\ ksi$ — yield strength of reinforcing bars

b. Flexural Strength Check

$\beta_1 := 0.85$

$Ø_{mb} := 0.75\ in$ — diameter of main bars

$Ø_{ti} := 0.5\ in$ — diameter tie bars

$d_t := thk_{pc}$ — thickness/depth of precast slab

$b_t := width_{pc} = 48 \cdot in$ — width of precast slab

$cc_{bot} := 1.5\ in$ — concrete cover of slab at bottom

$d_f := thk_{pc} - cc_{bot}$ — furnished depth of reinforcement from top

$n_{mb} := 9$ — number of main bars (bottom)

$As_{mb} := n_{mb} \cdot \frac{\pi \cdot Ø_{mb}^2}{4} = 3.976 \cdot in^2$ — area of main steel reinforcement

$\rho_b := \frac{0.85 \beta_1 \cdot f_{cp}}{F_y} \cdot \frac{87000}{87000 + \frac{F_y}{psi}}$ — balanced steel ratio

$\rho_{max} := 0.75 \cdot \rho_b = 0.021$ — maximum allowable steel ratio

$$\rho_w := \frac{As_{mb}}{b_t \cdot d_f} = 0.0097$$

actual steel ratio

since ρ < ρ max, beam is under-reinforced (tension-controlled), ϕ = 0.90

$$\phi M_n := 0.90\left[As_{mb} \cdot F_y \cdot \left(d_f - \frac{As_{mb} \cdot F_y}{1.7\, f_{cp} \cdot b_t}\right)\right]$$

$\phi M_n = 139.008 \cdot kip \cdot ft$

ultimate flexural strength capacity of precast slab

$M_u = 70 \cdot kip \cdot ft$

flexural requirement at ultimate condition

$$UR_m := \frac{M_u}{\phi M_n}$$

$UR_m = 0.504$

strength utilization ratio

c. Shear Strength Check

$V_u = 13.559 \cdot kip$

design shear force at ultimate condition

$$M_{u.v} := 1.2\left[\frac{W_{pu} \cdot a_{pu} \cdot d_f}{L_{pc}} + \frac{1}{2} \cdot \frac{d_t}{L_{pc}} \cdot \left(1 - \frac{d_t}{L_{pc}}\right) \cdot W_{pc} \cdot L_{pc}^2\right] + 1.4 \cdot \left[\frac{1}{2} \cdot \frac{d_t}{L_{pc}} \cdot \left(1 - \frac{d_t}{L_{pc}}\right) \cdot W_{LL} \cdot L_{pc}^2\right]$$

$M_{u.v} = 9.996 \cdot kip \cdot ft$

moment at design shear location

$\lambda := 1$

concrete unit weight modification factor

$$\phi V_{c1} := 0.85\left(1.9 \cdot \lambda \cdot \sqrt{f_{cp} \cdot psi} + 2500psi \cdot \rho_w \cdot \frac{V_u \cdot d_t}{M_{u.v}}\right) \cdot b_t \cdot d_t = 60.263 \cdot kip$$

$$\phi V_{c2} := 0.85 \cdot 3.5\sqrt{f_{cp} \cdot psi} \cdot b_t \cdot d_f = 76.767 kip$$

$$\phi V_c := \min(\phi V_{c1}, \phi V_{c2})$$

$\phi V_c = 60.263 \cdot kip$

shear strength provided by concrete

$$UR_{vc} := \frac{V_u}{\phi V_c} = 0.225$$

shear strength utilization ratio

5. Deflection Check

$E_c = 3.605 \times 10^3 \cdot ksi$

$I_c := \frac{b_t \cdot d_t^{\,3}}{12} = 4 \times 10^3 \cdot in^4$

a. Deflection by Load and Location

$\delta_{PC.MP} := \frac{5 \cdot W_{pc} \cdot L_{pc}^{\,4}}{384 \cdot E_c \cdot I_c} = 0.082 \cdot in$ deflection at midpoint due to self weight

$\delta_{PC.PU} := \frac{x \cdot \left(1 - 2x^2 + x^3\right) \cdot W_{pc} \cdot L_{pc}^{\,4}}{24 \cdot E_c \cdot I_c} = 0.0$ deflection at pump location due to self weight

$$\delta_{PU.MP} := \frac{W_{pu} \cdot b_{pu}}{12 \cdot E_c \cdot I_c} \cdot \left[2 \cdot L_{pc} \cdot \left(L_{pc} - \frac{L_{pc}}{2}\right) - b_{pu}^{\,2} - \left(L_{pc} - \frac{L_{pc}}{2}\right)^2\right]$$

$\delta_{PU.MP} = 0.054 \cdot in$ deflection at midpoint due to pump

$$\delta_{PU.PU} := \frac{W_{pu} \cdot a_{pu} \cdot b_{pu} \cdot \left(a_{pu} + 2 \cdot b_{pu}\right) \cdot \sqrt{3 \cdot a_{pu} \cdot \left(a_{pu} + 2 \cdot b_{pu}\right)}}{27 \cdot E_c \cdot I_c \cdot L_{pc}}$$

$\delta_{PU.PU} = 0.054 \cdot in$ deflection at pump location due to pump

$$\delta_{PU.PU} := \frac{W_{pu} \cdot a_{pu} \cdot b_{pu} \cdot \left(a_{pu} + 2 \cdot b_{pu}\right) \cdot \sqrt{3 \cdot a_{pu} \cdot \left(a_{pu} + 2 \cdot b_{pu}\right)}}{27 \cdot E_c \cdot I_c \cdot L_{pc}}$$

$\delta_{PU.PU} = 0.054 \cdot in$ deflection at pump location due to pump

$\delta_{LL.MP} := \frac{5 \cdot W_{LL} \cdot L_{pc}^{\,4}}{384 \cdot E_c \cdot I_c} = 0.066 \cdot in$ deflection at midpoint due to self weight

$\delta_{LL.PU} := \frac{x \cdot \left(1 - 2x^2 + x^3\right) \cdot W_{LL} \cdot L_{pc}^{\,4}}{24 \cdot E_c \cdot I_c} = 0.065 \cdot in$ deflection at pump location due to self weight

b. Deflection Limits

Standard practice limits deflection of structural members supporting piping/equipment to L/300.

$\delta_{all} := \frac{L_{pc}}{300} = 0.72 \cdot in$ allowable deflection at service level

$\delta_{sct} := \max\left(\delta_{PC.MP} + \delta_{PU.MP} + \delta_{LL.MP}, \delta_{PC.PU} + \delta_{PU.PU} + \delta_{LL.PU}\right)$

$\delta_{sct} = 0.201 \cdot in$ deflection at service level

$DR := \frac{\delta_{sct}}{\delta_{all}} = 0.28$ deflection ratio