Project Plan and Feasibility Study

Team 5: Waste Watchers

Calvin College Engineering Senior Design 2009-2010
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Project Advisor: Professor Leonard De Rooy
Executive Summary

Carabuela is a small village in Ecuador that is currently facing issues with its wastewater treatment system. The system is not working properly for a number of reasons. First, its septic tank and infiltration basin are both being overloaded. Second, the system is not being cleaned routinely. Third, the untreated effluent is being discharged directly into a nearby stream. These issues have been brought to our attention and it is our hope to develop a new wastewater treatment system that would be feasible to the people living in Carabuela. We are working alongside an organization called HCJB, who has been our main contact in Ecuador and has provided us with important information about the project.

Our designed system includes four important components. They are the bar rack/grit chamber, Imhoff Tank, stabilization pond, and sludge drying beds. The waste stream will enter the bar rack/grit chamber where large sized particles or grit will be removed. Following this is the Imhoff Tank, where settling and biological digestion takes place. Next, the wastewater enters the stabilization pond in which an aerobic process takes place and large amounts of BOD are removed. Finally, the treated water is released into farmland for irrigation. The sludge produced during this process is treated using drying beds. The design process is shown in Figure 1 below. Another alternative was to remove the Imhoff Tank and build a larger stabilization pond. This would result in the same effluent quality and reduce sludge handling maintenance, but the problem revolves finding the space for a large lagoon. Carabuela is situated on a hilly terrain, which makes finding flat land for a big lagoon almost impossible.

The total cost for the entire system including labor and materials is estimated to be $15,000. The project is being proposed to the people of Carabuela after the final design proposal has been made. Ultimately, the people of Carabuela have the decision whether or not to approve our design. It is important that we consider cultural appropriateness whenever necessary.
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1. Introduction

1.1. Description of Team

Chris Crock is a 22 year old engineering student at Calvin College and will be attending Graduate School upon finishing his undergraduate studies in Civil/Environmental Engineering. He enjoys traveling the globe and has recently reconnected ties with his distant cousins in southern Germany. His passion for engineering in developing nations has evolved into boundless enthusiasm for water treatment in nations with a severe need for water as a resource. He hopes to continue his endeavors in water when finishing all his schooling with a non-profit organization focused on water treatment around the world. In the distant future, he sees himself giving back to the engineering academic setting through hopes of becoming a professor.

Aaron Lammers is 23 year old civil & environmental engineering student at Calvin College. He is from Villa Park Illinois and enjoys playing sports and exploring new technology in his free time. He is currently looking for full time employment for after graduation in the areas of water resources, water treatment, transportation, construction and field engineering. He has worked as an engineering intern with the City of Elmhurst for the past two summers and during school breaks. He is currently engaged and looks forward to getting married after graduation.

Aaron Raak is from Flagstaff, Arizona. He is currently studying civil engineering at Calvin College, and plans to get a job or attend graduate school after graduation. Last summer he was an intern for an organization in Cambodia, and hopes to do more work overseas. In his spare time he enjoys hiking, rock climbing, and soccer.

Brent Long is an engineering student at Calvin College where he is pursuing his studies in the Civil and Environmental concentration. He is from Rochester, New York and enjoys a variety of activities such as playing the piano, participating in sports and fiddling on his Macintosh computer. After college, Brent hopes to get a job that involves water treatment or wastewater treatment. He has a passion for helping those who do not have necessities such as clean water and is excited about the influence his Senior Design Project will have on small village in Ecuador.
1.2. Project Introduction

Over 80% of Ecuador’s wastewater goes untreated. One example of this is the village of Carabuela, a community of 200 homes near the Pan American highway about two hours’ drive from the capital, Quito. At present wastewater goes mostly untreated into a nearby stream. We are connected to Carabuela through HCJB Global (Herald Christ Jesus’ Blessings), a group that works to bring water sanitation and hygiene to rural communities. As this village is neither wealthy nor technologically sophisticated, we need to choose a program of appropriate cost and complexity.

For this project we will design a water treatment system to reduce the pathogen content of the effluent, use the water for irrigation, and possibly use the sludge for fertilizer. This will involve a bar rack and grit chamber to remove large objects and solids, a primary settling process to remove particulate matter, a lagoon to remove organic matter, aerobic or anaerobic digestion to reduce the pathogen content, and disposal of the sludge. The effluent water will be routed to nearby fields as irrigation.

We chose this project because it has the potential to enhance public safety, to promote a sustainable use of the land, and to improve stewardship of the earth in this locality. Water-borne diseases are a major health concern: according to the WHO, “Globally, improving water, sanitation and hygiene has the potential to prevent at least 9.1% of the disease burden.” One important method is wastewater treatment, which can drastically cut the spread of cholera, dysentery, and many other diseases. Diarrhea, a common symptom of these diseases, ‘is responsible for the deaths of 1.8 million people every year (WHO, 2004). At present a stream is polluted with human waste. Sanitized wastewater can safely irrigate crops, helping agriculture while reducing the amount of water taken from the environment and avoiding contamination of an important local resource.

2. Background & Research

Carabuela is a small village located in the Northern part of Ecuador outside Otavalo. Currently, the village consists of approximately two hundred homes with an average of 5 people living per home. Obviously population growth will continue to expand so our design must meet future demands. Based on a recent analysis of the village, the population of Carabuela is expected to reach 2700 occupants through the year 2029. This is the target population that our design will be based upon.

Some common occupations are weaving, dyeing or farming; weaving is the primary occupation. The people rely on the quality of their hand woven products in order to compete against cheaper woven products that are mass-produced.

The geography of the area consists of a large knoll, which divides the village into two: one side being farmland and the other side being the main village. The effluent wastewater will need to be routed around this knoll to the farmland where it will be released. Since no energy sources such as pumps will be used, gravity will be the only force that will move the wastewater.

As of now, the current system consists of sewage pipes that lead to a main manhole and a septic tank and lagoon, which are both being overloaded. Storm water and wastewater are both entering the system. It is possible to separate the two but for now the original intent is to design with consideration of a combination of both streams.

Some requirements of the system are that it must not be highly sophisticated and it must be proven. Any design that is developed must be simple enough that persons in Carabuela can operate it easily. It should be simple because there will be no one there to guide them in case any sudden problems should arise. Along these lines the system should only have components that have already been tested. A system that is
based on developing theories should not be implemented. The system must be designed with cultural appropriateness in mind. For example, the system should not operate using any energy sources because the people would not be able to finance it.

This project is a proposal to the people of Carabuela. We are working in conjunction with HCJB to develop a safer, sanitary waste water system that can be financed and operated with ease. We hope that our design consideration will be accepted and construction will begin after the final designed has been proposed.

3. Design Norms

Knowledge of the customer’s culture:

The first step in designing a culturally appropriate technology is to understand the culture of the customer and the reason for treating the wastewater. Water used by the community of Carabuela, Ecuador will contain storm water, drinking water, urine, feces, and sullage from textile dyes. This water is to be treated to reduce the transmission of excreta related diseases and to reduce water pollution and the consequent damage to communities downstream of Carabuela. We must remember that we are designing a system that will be built only if the customer is satisfied with the design of the system. The effluent of the treatment facility will be used for irrigation; consequently, the water must be treated so the effluent contains no parasitic eggs and low levels of excreta-related bacteria and viruses.

The financing for the wastewater system will be provided by either the Ecuadorian government or the village of Carabuela, therefore a low cost system should be designed. The main cost of the system will be a result of the construction of the Imhoff tank, the water stabilization pond, and the drying beds. Minimal costs will result from the construction of pre-treatment processes because of the low level of technology and resources needed for their design.

Another factor to be aware of is the level of technology and technically educated laborers required to manage the system. We must remember that the system must provide adequate treatment without the use of electricity and without technically educated personnel (although, according to our contact Bruce Rydebeck, very imaginative and clever people live in Carabuela). A system has been designed with consideration of the above factors, and this includes pre-treatment using bar screens and a grit chamber, primary treatment using an Imhoff tank or large stabilization lagoons, a water stabilization pond for BOD reduction, and sludge treatment using drying beds.

The final design of the system must include construction and operating plans that are easily understood by the community of Carabuela. This involves detailed instructions for construction in Spanish, units in the SI system, and a proper managerial plan translated in Spanish.

Culturally Appropriate Considerations for Design: (Mara, 2004)

One of the most effective processes for wastewater treatment in developing countries is the use of a waste stabilization process. In the proposed treatment process, an Imhoff Tank and waste stabilization pond will be used for stabilization of waste. This design will require designing engineers to be aware of diseases prone to Carabuela, Ecuador, which will fall in the range of excreted infections: non-bacterial faeco-oral diseases (category I), bacterial faeco-oral diseases (category II), geohelminthiases (category III), taeniases (category IV), water based helminthiases (category V), and excreta rodent-vector diseases (category VI).

Category I infections should consider the removal of Rotoviruses, which cause 350,000 to 600,000 deaths per year. Category II infections are non-latent, have a medium-high persistence, have the ability to
multiply, and have a medium-low infectivity. E coli, Salmonella, Shigella and Vibrio Cholera are some of the infections that are found in Category II infections. Category III infections are latent, very persistent, unable to multiply, and have a high infectivity. Common infections can be caused by the Roundworm (200,000 eggs/day), Hookworm (200,000 eggs/day), and Whipworm (5,000-20,000 eggs/day). Category IV infections are latent, persistent, have the ability to multiply, have a very large infectivity, and have either a cow or pig intermediate host. One parasite has the ability to produce $10^6-10^5$ eggs/day. Some of the common examples of parasites in Category IV are the Beef tapeworm and the Pork tapeworm. Category V infections can be caused by the Trematode worms. Category VI infections can either be insect-vector or rodent-vector diseases. Insect-vector diseases result from poorly maintained systems and are transmitted by mosquitoes. Elephantitis is often a result of diseases transmitted by mosquitoes. Rodent-vector diseases are usually spread by brown rats and result from the rat’s contact with urine. A common disease from brown rats is Leptospirosis, which is fatal if not treated. All these emerging diseases need to be considered for the design of a wastewater treatment system in a developing country.

Engineers must also consider essential microbiology that involves the treatment for certain viruses and Archaea. Viruses are parasitic microbes that have a DNA or RNA protein coating and range from 20—200 nm. Archaea are usually a few micrometers and must grow in a 15—40 degree Celsius environment. They thrive in near neutral or slightly alkaline environments. The design of Carabuela’s wastewater system must consider the environment and chemical properties of waste being delivered into the system.

The above considerations are needed for an appropriate design for Carabuela, and effluent qualities are needed to set at an appropriate removal level to have an effective system while reducing the possibility of “over kill” in the system. Qualities of effluent can be taken from either the Ecuadorian government or the World Health Organization (WHO).

4. Scope

This project is to design a complete wastewater process to treat the wastewater from the village of Carabuela, Ecuador. The current wastewater process involves collecting the water from the village and piping it to a septic tank. Currently, the septic tank does not sufficiently treat the wastewater. From the septic tank, the effluent was originally designed to flow into a seepage bed where it would infiltrate through the soil and end up in the groundwater. This system has failed in both of the treatment methods. First, the septic tank has a problem with short-circuiting and the water does not stay in the tank for long enough to be treated adequately. Secondly, the seepage bed has become clogged with sediment and the water no longer infiltrates down to the groundwater at a fast enough rate. This slow infiltration rate leads to the water pooling up near the septic tank exit pipe. The pooling water has an unpleasant odor, and the nearby residents were not very happy. To remedy the situation, the resident disconnected the pipe from the septic tank and rerouted it into a nearby stream. This situation is very detrimental to not only the safety of the local population but also to the surrounding environment and ecosystem. To solve this problem, our team is designing a water treatment process which will treat the water to a safe level for use in local irrigation. The scope of our project is to treat the wastewater from the effluent of the collection system to the effluent to the stream. Our process will receive the water from the collection system and will discharge the water over local farm land for irrigation, resulting in four key unit processes.

To determine parameters of our design, it was necessary to make some educated assumptions. These assumptions are necessary because of the lack of available data about the location. The location is relatively remote and does not have access to the necessary equipment that would be needed to measure the data. Determining the quantity of the flow from the collection system is difficult because the lack of historical data of the flow and no way of obtaining new flow data. It is necessary to have a quantity of the flow for the design of all of the components of our process. To resolve this dilemma we have consulted
both Bruce Rydbeck, who is our contact in Ecuador and also Anne Mikelonis, who has done similar work in developing countries. From their experience we have determined an estimated value which we use for our calculations. The estimated value for the influent flow on average is 8 cubic meters per household per month. We also lack any data on the wastewater quality and are unable to collect any of this data. Because we are unable to measure any of the levels of any contaminants or water properties, we will need to work with those who have experience in the field to make educated assumptions about the quality of the water. A criterion of our project is to remove the necessity for any electricity or pumps to run our process. To meet this criterion, it is necessary to have enough change in elevation throughout each step of the process to have sufficient hydraulic energy to flow through the system. We are able to calculate the hydraulic energy of the system after we know the energy losses associated with our components and elevation data from topographic maps. Our group is working with Mr. Rydbeck to obtain topographic data from the area to determine if this is possible and feasible. Determining the level to treat the water requires knowledge of the Ecuadorian environmental process and standards. Our team will work with Mr. Rydbeck and consult the World Health Organization (WHO) to determine an appropriate level of treatment. A large portion of this project is to make the process culturally appropriate for the community. Our team will work to make sure what we design is able to be built by workers in that area along with using construction practices and materials from that area.

5. Objectives

The main objective of this project is to construct and design a wastewater treatment system to adequately treat for a reduction in the strength of waste (BOD and COD), the microbiological life which cause diseases that result from excreta, and the Nitrogen and Phosphorus levels. The reduction of the above criterion is needed in order to use the effluent for irrigation in Carabuela, Ecuador.

Our secondary objectives are important in the choice of design and result from the cultural standards which involve a treatment system that is low cost, requires little maintenance, and needs no electrical input. The whole system will be gravity controlled, and this is contributed by the extreme hydraulic gradient in Carabuela. This system must be proven in technology and will be culturally appropriate to the people of Carabuela.

As Christians, sustainability is an important factor in the design. The life of the treatment system must be of appropriate length as to handle fluctuations in populations and additional flows due to added sources of wastewater. Reuse of resources already in place must be considered so as to reduce cost of the system and the need for construction of new unit processes.

Some alternative unit processes for treatment are being researched to prove the proposed system is the best alternative for suitable treatment without “over kill” in the design. These current alternatives include the addition of a modified 55 gallon metal barrel used for the removal of grit, large stabilization lagoons as opposed to the Imhoff tank, wetlands as a replacement for the proposed water stabilization pond, and covered drying beds as opposed to the open-air beds.

6. Design Alternatives

6.1. Bar Screen

6.1.1. Introduction:

The first unit process in the treatment of wastewater is screening. In this unit process, larger, coarse solids are removed through a system of bars or screens, and units that use parallel bars or rods are usually called
bar racks or bar screens. Because screening is the first unit process in wastewater treatment, it is important that the system works properly, so processes further downstream are not inhibited by screenings that would otherwise be allowed through the system. Screening helps to prevent the systems downstream from being corrupted by “rags and floatables” (wood, trash, large rocks, plastic, etc.), or screenings, and help to produce the most effective treatment of wastewater. The screening process uses either coarse or fine bar screens that can be mechanically or manually raked. Because mechanically raked bar screens require power, the recommended technology for Carabuela, Ecuador would be a manually raked, coarse bar screen, which would remove “rags and floatables” in the range of 25-50 mm (Vesilind, 2003).

6.1.2. Design and Considerations

The design of bar screens does not involve complex equations, but the understanding of the factors and considerations are important to a design which requires little maintenance, no power, and long life. Listed below are considerations that need to be thoroughly accounted for in the design of bar screens.

- Manually cleaned screens demand frequent cleaning, so as to prevent clogging and the possibility of flow surges when debris is removed.
- A flow surge could cause ineffective removal grit and organics downstream.
- The angle of incline is important as to provide effective area that minimizes headloss, while maximizing the ease of cleaning.
- Maximum approach velocities must be within the range 0.3–0.6 m/s for maximum flows in order to prevent dislodgment or disintegration of larger particles.
- Maximum velocities through the bar screen should be < 1 m/s to prevent dislodgment or disintegration of larger particles.
- Clear openings between bars are the most important factor in the design for removal quantities.
- The purpose of screening is not to remove the organic matter, rather the large inorganics, wood, trash, etc. because processes downstream will perform the organic removal process.
- Accepted practice calls for a minimum headloss through a manually cleaned bar screen of 150 mm (fairly clean) and a maximum of 760 mm (clogged).
- A drainage area for screenings is needed before shoveling and burial or delivery to the drying beds.
  - Non-slip platforms deserve special attention for cleaning and removal of screenings for the workers.
  - The drainage area must provide enough volume to store screenings long enough for dewatering. (Vesilind, 2003)

Manually cleaned screens require frequent cleaning to prevent any clogging and excessive headloss, and the frequency of cleaning will depend on the flows and also the quantity of screenings that are present in the raw wastewater. The clear space of the bar rack is important in determining the amount of screenings that will be removed and the characteristics of the screenings. Table 2 provides that the openings between parallel bars would be 20—50 mm. The screening of excreta may create hygiene issues with workers who must manually rake the rack and in the disposal of the screenings, therefore a proper size that allows excreta to flow through but stops “rags and floatables” should be considered. Quantities and characteristics of coarse screenings can be seen in Table 2, and these values would be used to calculate volume needs for treatment beds of sludge and also for the short-term storage on the drainage area for screening.

Table 2 shows that the velocities through the screens should be limited to 0.6 m/s to prevent deposition or displacement of any grit or rags and floatables (Mara, 2004), and due to the low flow system of Carabuela...
(8m³/person/month) this velocity can be met with a minimum cross-section area of the influent channel of only a couple centimeters. Because of the small required cross-sectional area of the channel, it is suggested to use the designed channel height with a freeboard, the distance between the maximum water surface elevation (WSE) and the top edge of the channel, of 0.5 meter, 0.76 m for headloss, 0.037 m for required minimum height of the wetted perimeter. The total height of the channel is suggested to be 1.3 m. The channel width requires a total of 0.3 m, which includes the clear space and the bar widths. The calculations used for determining the width of the channel can be seen in Table 1.

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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.300 m</td>
<td></td>
<td>Channel Height</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.300 m</td>
<td></td>
<td>Channel Height</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.390 m²</td>
<td></td>
<td>Cross Sectional</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The clear space of the bar rack is important in determining the amount of screenings that will be removed and the characteristics of the screenings. Table 2 provides that the openings between parallel bars would be 20—50 mm. In the calculation for channel width, 20 mm was used (the minimum) so as to minimize rags and floatables that flow through, but also to minimize the excreta that would be blocked from passage. The screening of excreta may create hygiene issues with workers who must manually rake the rack and in the disposal of the screenings, therefore a proper size that allows excreta to flow through but stops “rags and floatables” should be considered. Quantities and characteristics of coarse screenings can be seen in Table 3, and these values would be used to calculate volume needs for treatment beds of sludge and also for the short-term storage on the drainage area for screenings.
Table 2: Typical Design Criteria for Coarse Screening Equipment

<table>
<thead>
<tr>
<th>Item</th>
<th>Range*</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trash Rack</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Openings</td>
<td>38—150</td>
<td>Commonly used with a combined sewer system</td>
</tr>
<tr>
<td>Manually Cleaned Screen</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Openings</td>
<td>20-50 mm</td>
<td>Used in small plants and bypass channels</td>
</tr>
<tr>
<td>Approach Velocity</td>
<td>0.3—0.6 m/s</td>
<td></td>
</tr>
<tr>
<td>Mechanically Cleaned</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Openings</td>
<td>25—50 mm</td>
<td></td>
</tr>
<tr>
<td>Approach Velocity, Maximum</td>
<td>0.6—1.2 m/s</td>
<td></td>
</tr>
<tr>
<td>Approach Velocity, Minimum</td>
<td>0.3—0.6 m/s</td>
<td></td>
</tr>
<tr>
<td>Continuous Screen</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Openings</td>
<td>6—38 mm</td>
<td>Ineffective in the 6—18 mm range</td>
</tr>
<tr>
<td>Approach Velocity, Maximum</td>
<td>0.6—1.2 m/s</td>
<td></td>
</tr>
<tr>
<td>Approach Velocity, Minimum</td>
<td>0.3—0.6 m/s</td>
<td></td>
</tr>
<tr>
<td>Allowable Headloss</td>
<td>0.15—0.6 m</td>
<td></td>
</tr>
</tbody>
</table>

*Values from US EPA 1979, 1987; WPCF, 1989

Table 3: Typical Design Properties for coarse Screenings

<table>
<thead>
<tr>
<th>Item</th>
<th>Range*</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quantities</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Separate Sewers</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average screenings per 1000 m³ wastewater</td>
<td>3.5—35L/1000m³</td>
<td>Function of the screen opening space</td>
</tr>
<tr>
<td>Peaking Factor (hourly flow)</td>
<td>1:1—5:1</td>
<td></td>
</tr>
<tr>
<td>Combined Sewers</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average screenings per 1000 m³ wastewater</td>
<td>3.5—84L/1000m³</td>
<td>Function of the screen opening space</td>
</tr>
<tr>
<td>Peaking Factor (hourly flow)</td>
<td>2:1-- &gt; 20:1</td>
<td></td>
</tr>
<tr>
<td>Solids Content</td>
<td>10-20 %</td>
<td></td>
</tr>
<tr>
<td>Bulk Density</td>
<td>640—1100 kg/m³</td>
<td></td>
</tr>
<tr>
<td>Volatile Content</td>
<td>70-95 %</td>
<td></td>
</tr>
<tr>
<td>Fuel Value</td>
<td>12,600 kJ/kg</td>
<td></td>
</tr>
</tbody>
</table>

*Values from US EPA 1979, 1987; WPCF, 1989

The shape and size of the parallel bars are important in calculating the headloss through the bar screen, and Table 4 provides the shape factor required for Kirschmer’s headloss equation below (Equation 1). This headloss is used in finding the headloss for a given bar screen, where \( h \) is the headloss upstream, \( W \) is the width of the bar, \( h_v \) is the headloss through the screen and \( \theta \) is the angle of the bar with respect to the channel.

Equation 1: Kirschmer’s equality for partially clogged bar screens

\[
h = \frac{4W}{3\beta} h_v * \sin \theta
\]
Table 4: Table of bar types and their respective shape factors

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>$\beta^*$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sharp edged rectangle</td>
<td>2.42</td>
</tr>
<tr>
<td>Rectangular with semicircular side upstream</td>
<td>1.83</td>
</tr>
<tr>
<td>Circular</td>
<td>1.79</td>
</tr>
<tr>
<td>Rectangular with semicircular upstream and downstream</td>
<td>1.67</td>
</tr>
</tbody>
</table>

*Kirschmer’s bar shape factors for Kirschmer’s headloss equation

In order to accommodate the manual raking of the screen, it is advised to incline the screen or bars at a maximum angle of 60 degrees from the channel. When higher flows (>1000 m$^3$/day) are common, it is preferred to use mechanically raked screens, so they can be raked every 10-30 minutes (Mara, 2004), but average flow rates of that magnitude will not be expected in Carabuela, so it is fitting to use manually cleaned bar screens with twice daily rakings of the screen. Also, as a precaution for a damaged bar screen, an extra bar screen should be available to quickly replace the damaged screen. A simple bar rack fitted to the incoming channel can be seen in Figure 2.

![Figure 2: Example of Manually Raked Bar Screen](http://www.urbanwater.co.za/)

6.1.3. Cost Feasibility:

The construction of a bar screen requires the cost of labor, materials (stainless steel bars, concrete, drainage rack, and buttresses to hold the bar screen and drainage rack). With respect to other processes, the cost of a bar screen is much less. The importance and low cost of bar screens in the pre-treatment of wastewater makes it highly favorable to construct a bar screen system for the treatment process. Without this unit process, the downstream processes will be affected negatively and can cause ineffective treatment of the wastewater.
6.2. Grit Chamber

6.2.1. Introduction:

Grit Removal follows the unit process of screening and removes grit (heavy metals and sand) as to prevent any unnecessary abrasion of equipment downstream and accumulation of grit in the biological processes downstream. With the high presence of grit in combined sewer, it is necessary to achieve the appropriate levels of removal for our customer (Vesilind, 2003). Grit materials have a greater settling velocity than do organic materials and therefore can be removed without removing organics, which are needed for the digestion process downstream in the Imhoff tank or waste stabilization lagoons.

Grit quantities and attributes are important considerations in design to have minimum negative effects on processes downstream. Because attributes differ among treatment facilities and other requirements such as: headloss, space, removal efficiency, organic content, and economics, a number of processes exist. Some of these are: Aerated grit chambers, Vortex-type, Detritus tank, and Hydroclonic (Vesilind, 2003). The appropriate technology for our customer would be a Detritus tank (short term sedimentation), so as to minimize costs and maintenance and also to eliminate the dependency on electricity. The Detritus tanks acts as a detention tank with a constant level of grit removal. It may be necessary to design a proportional weir to persistently remove a certain level of sediment.

6.2.2. Design Considerations: (Vesilind, 2003)

The basis of design for the grit chamber is the settling velocity of grit and the surface loading rate. The velocities of minimum sizes of grit are 0.02 m/s (Vesilind, 2003). With this velocity the cross-sectional area is 0.836 m², where the width is 0.46 m, and the length is 1.8 m. This design will allow for all particles with a settling velocity of 0.02 m/s or grater to be removed from the stream. The calculation for settling velocity uses Equation 2 and Equation 3 where \( V_s \) is the settling velocity, \( g \) is the acceleration due to gravity, \( C_D \) is the drag coefficient, \( \rho_s \) is particle density, \( \rho \) is the water density, \( d \) is diameter of particle settling, and \( Re \) is the Reynolds number. Transitional flow is assumed in the grit chamber because the Reynolds Number is within the transitional range of 1—10⁶.

\[
V_s = \frac{4g \rho_s - \rho}{3C_D \rho} \times d
\]

\[
C_D = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34
\]

- Typical particle sizes include particles > 0.21 mm sp. gravity 2.65 (EPA, 1987)
- Removal of 95% has been traditionally target removal
  - Modern remove 75% of 0.15 mm
- Removal of grit manually requires at least one redundant tank for cleaning purposes
- Velocity or turbulence in grit chamber may be designed to allow the displacement of organic materials but not grit (this could be achieved with the proportional weir)

The Detritus Tank will be a tank with a length to width ratio of 4:1 to meet the minimum cross-sectional areas as can be seen in Table 5. This is a concrete tank that will consist of a baffle to evenly distribute the
flow along the channel and will be manually cleaned with a shovel. In order to allow for cleaning while continuing treatment, a second tank of the same specifications will need to be constructed. This will be constructed in parallel with the channeling to the grit chamber so flow can easily be shut-off from the chamber being cleaned to the tank that will take its place in grit removal. As with the screenings from the bar screen, the disposal of the grit from the grit chamber will be transferred to the drying beds for further treatment. The quantities of grit can be estimated as seen in Table 6. It is important to regularly clean the tank so that it will not be overloaded causing further problems downstream. An example of a simple square Detritus tank can be seen in Figure 3.

Table 5: Grit Chamber Calculations and Dimensions

<table>
<thead>
<tr>
<th></th>
<th>m/s</th>
<th>Minimum settling velocities</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.836</td>
<td>m²</td>
<td>Cross-sectional area</td>
</tr>
<tr>
<td>1.8</td>
<td>m</td>
<td>Length</td>
</tr>
<tr>
<td>0.46</td>
<td>m</td>
<td>Width</td>
</tr>
</tbody>
</table>

Figure 3: Long narrow grit chamber where heavier inorganics are removed

Table 6: Estimated grit quantities for a Detritus tank

<table>
<thead>
<tr>
<th>Type of system</th>
<th>Average Grit Quantity (m³/1000m³ wastewater)</th>
<th>Ratio of max day to average day</th>
</tr>
</thead>
<tbody>
<tr>
<td>Separate</td>
<td>0.004—0.037</td>
<td>1.5—3.0:1</td>
</tr>
<tr>
<td>Combined</td>
<td>0.004—0.18</td>
<td>3.0—15:1</td>
</tr>
</tbody>
</table>

6.2.3. Feasibility and Cost:

The construction of a grit chamber requires the cost of labor and materials (concrete and reinforcement). The cost of the grit chamber is much less than the cost of the Imhoff tank further downstream. The grit chamber is a critical component in the effective treatment of biological matter downstream and would be highly favorable to construct. Without this unit process, the downstream processes will be negatively affected and can cause ineffective treatment of the wastewater.
6.3. **The Imhoff Tank**

6.3.1. **Introduction**

The Imhoff tank was invented and patented by a German engineering named Karl Imhoff in 1906 (Seeger, 1999). The tank combines two wastewater treatment processes, sedimentation and biological digestion, into one physical system. The tank is typically a two-story system in which simple sedimentation and anaerobic digestion are used to treat the influent wastewater. The upper story uses simple Type 1 discrete particle settling as the driving force to remove particulate from the stream. The lower story uses anaerobic digestion to change the physical, chemical and biological properties of the settled sludge. After being treated with an Imhoff tank, the effluent water typically has a 20 to 70 percent reduction in suspended solids and a 10 to 40 percent reduction of BOD$_5$ (Reynolds & Richards, 1996). Figure 4 below provides a cross and longitudinal sections of an Imhoff tank.

![Figure 4: Imhoff tank schematic (Sasse, 1998)](image)

6.3.2. **Sedimentation**

Sedimentation in the upper story of the Imhoff tank uses the principle that particles settle downward when the velocity of the flow drops. Discrete particles accelerate downward until gravitational force pulling the particle downward equals the viscous drag force resisting the motion of the particle and the upward buoyant force. The gravitational force is the force associated with the gravitational attraction of the mass with the mass of the earth. The gravitational force can be determined with the density of the particle, $\rho_p$, the diameter of the particle, $D_p$, and the gravitational constant, $g$, using Equation 4.

\[
F_g = \rho_p \frac{\pi}{6} D_p^3 g
\]

The buoyant force results from the increase of pressure with depth within a fluid (Mihelcic, 1999). The buoyant force can be determined with the density of the fluid, $\rho_f$, the diameter of the particle, $D_p$, and the gravitational constant, $g$, using Equation 5.

\[
F_B = \rho_f \frac{\pi}{6} D_p^3 g
\]
The final force on a particle is the drag force. The drag force is the result of frictional resistance to the flow of fluid past the surface of the particle. This resistance can be correlated to the Reynolds number. Most particle situations involve “creeping flow” conditions where the Reynolds number is less than 1 (Mihelcic, 1999). For this situation, the drag force can be determined with the fluid viscosity, \( \mu \), the diameter of the particle, \( D_p \), and the velocity of the particle relative to the fluid, \( v_s \), using Stokes as seen in Equation 6.

**Equation 6: Force of Drag**

\[
F_D = 3\pi \mu D_p v_s
\]

Assuming that the particle has reached terminal velocity the force of gravity is equal to the drag force plus the buoyant force. From this equality, the settling velocity can be determined with previously defined variables using Equation 7.

**Equation 7: Settling Velocity**

\[
v_s = \frac{g(\rho_p - \rho_f)}{18\mu} D_p^2
\]

If the settling velocity calculated is greater than that of the overflow rate of the settling tank, particles of size \( D_p \) and larger are removed with nearly 100% efficiency. The particle diameter is typically the particle of interest so it is very important to keep the overflow rate lower than the settling velocity for all variations of flow. The equations above it is make the following assumptions and must be verified in the final design on the sedimentation chamber:

1. The settling is type I settling
2. There is an even distribution of the flow entering the basin
3. There is an even distribution of flow leaving the basin
4. There are three zones in the basin: (1) the entrance zone, (2) the outlet zone and (3) the sludge zone
5. There is an even distribution of particles throughout the depth of the entrance zone
6. Particles that enter the sludge zone remain there, and particles that enter the outlet zone are removed

When sizing the sedimentation chamber several considerations must be made. The depth of the sedimentation chamber must be shallow enough as to not inhibit vertical distribution of flow but deep enough so that the slow-motion settling zone is not encroached (Metcalf & Eddy, 1935). Also, the influent flow must be equally distributed across the channel to make it hydraulically ideal. Determination of the slope is critical in the sedimentation chamber as to prevent build up of settled solids; furthermore, to prevent built up sludge from pluming upward.

### 6.3.3. Anaerobic Digestion

Anaerobic digestion in the lower story of the Imhoff tank uses the principle that certain microbes in an environment with no oxygen will use organic matter as the primary food source for new cell growth. The generalized equation for anaerobic digestion is shown in Equation 8.

**Equation 8: Anaerobic digestion**

\[
\text{Organic Matter} + \text{Combined Oxygen} \rightarrow \text{Anaerobic Microbes} \rightarrow \text{New Cells} + \text{Energy for Cells} \rightarrow \text{CH4} + \text{CO2} + \text{Other end products}
\]
The microbial activity during anaerobic digestion consists of three stages: (1) liquefaction of solids, (2) digestion of the soluble solids, and (3) gas production. This process can be seen graphically in Figure 5.

![Figure 5: Anaerobic Digestion Schematic of Decomposition (Lesson 4: Aerobic and Anaerobic Digestion and Types of Decomposition, 2009)](image)

The digestion of soluble solids is divided into the fermentation and acetogenesis processes in this graphic to show an added level of detail. Digestion is accomplished by two groups of microorganisms: (1) the organic-acid-forming heterotrophs and (2) the methane-producing heterotrophs. The organic-acid-forming heterotrophs use complex organic substrates, such as carbohydrates, proteins, fats, oils and produce organic fatty acids, primarily acetic and propionic with some butyric and valeric acids. These microbes thrive in a relatively wide pH range. The methane-producing heterotrophs use organic acids created by the acid formers as substrates and produce methane and carbon dioxide. The methane producers grow more slowly and require a narrow range of pH of about 6.7 to 7.4.(Reynolds & Richards, 1996). Each of these microbial processes reduces the volume of sludge, volatile solids and organic content of the wastewater stream. After digestion, the volatile solids are usually reduced from 65%-70% down to 32% - 48%. Dry solids are usually increased from 4% to 6% up to 8% to 13%. Approximately 99.8% of coliforms are destroyed during digestion (Reynolds & Richards, 1996).

The digestion chamber of the tank has four sections: the sludge storage area, the clearance area, the scum/gas vent area and the sedimentation slot area. Each section plays a vital role in the digestion of the sludge. The sludge storage area holds the sludge for the necessary time to go through proper digestion. If this area is improperly sized or cleaned, sludge can plume into the effluent water and create a higher strength waste than when it entered. The clearance area allows the water and solids entering the sludge chamber to slow down enough so that the settled sludge does not get swept back up into the sedimentation chamber. The scum/gas vent area allows for the gases produced in the digestion to escape without having to exit through the sedimentation chamber. In this area the scum accumulates on the surface of the water which should be periodically cleaned to insure clean effluent water. The sedimentation slot area is designed such that there is an overlap of bottom of the sedimentation slot. This
overlap is crucial in keeping the gases and sludge from rising vertically into the sedimentation chamber, minimizing contamination of the sedimentation chamber water. Typical sizing of the digestion chamber accounts for storage of 6 months of sludge accumulation (Tchobanoglous, 1991).

6.3.4. Design Criteria

The Decentralized Wastewater Treatment Systems (DEWATS) organization has worked extensively with wastewater treatment systems in the developing regions. In 1998 the organization, in conjunction with the Bremen Overseas Research and Development Association (BORDA), published a report written by Ludwig Sasse titled “Decentralized Treatment Systems in Developing Countries”. In this report, many aspects of wastewater treatment are discussed as they pertain to developing countries, including the design of Imhoff tanks. Our system would not technically be a decentralized system because of the collection system which brings all of the waste from the village to a central location for treatment. However, because of the small scale of the project and the remote location, many of the designs and alternatives that are discussed can be directly related to our design. DEWATS has created a set of spreadsheet calculations that can be used for initial designs of the decentralized wastewater treatment systems. The calculations are based off typical rules of thumb for developing regions and also from years of experience in design of these systems. The standards and calculations from DEWATS were appropriate for the preliminary design because they would give an accurate approximation for the size tank that would be typically designed for our location.

The spreadsheet was used to determine design parameters such as tank dimensions, COD and BOD removal rates, COD and BOD outflow, and the volume of sludge. The spreadsheet requires inputs for the daily flow, the peak hour of flow, the COD and BOD inflow, the hydraulic residence time, the de-sludging interval, and several dimensions of the tank. Previously, we have received an estimate of 8 m³/month/household for the flow (Bruce Rydbeck). This corresponds to a flow of about 53 m³/day. Because of the lack of available data from the location, approximations of the influent COD and BOD were assumed, being similar to designs by DEWATS. A hydraulic residence time of 1.5 hours was used because retention times in excess of 2 hours jeopardize the separation of the active sludge and the effluent water (Sasse, 1998). A settleable solids ratio was assumed to be in the domestic range recommended. The de-sludging interval was taken to be 6 months to ensure that sludge would not harden and create difficulty when cleaning the tank. An inner width of the tank was taken to be 1.5 meters to minimize the total width of the tank. The space between the sedimentation chamber and the side of the tank was chosen to be 0.5 meter to allow for the passage of persons if necessary for maintenance. The spreadsheet design can be seen in Table 7.
## Table 7: DEWATS spreadsheet design

<table>
<thead>
<tr>
<th>General spreadsheet for Imhoff tank, input and treatment data</th>
</tr>
</thead>
<tbody>
<tr>
<td>daily waste water flow</td>
</tr>
<tr>
<td>given</td>
</tr>
<tr>
<td>m³/day</td>
</tr>
<tr>
<td>144</td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

### Dimensions of Imhoff tanks

<table>
<thead>
<tr>
<th>de-sludging interval</th>
<th>flow tank volume</th>
<th>sludge volume</th>
<th>inner width of flow tank</th>
<th>space beside flow tank</th>
<th>total inner width of Imhoff tank</th>
<th>inner length of Imhoff tank</th>
<th>sludge height</th>
<th>total depth at outlet</th>
<th>biogas 70%CH₄; 50% dissolved</th>
</tr>
</thead>
<tbody>
<tr>
<td>chosen</td>
<td>calc.</td>
<td>calc.</td>
<td>chosen</td>
<td>chosen</td>
<td>calc.</td>
<td>calc.</td>
<td>calc.</td>
<td>calc.</td>
<td>calc.</td>
</tr>
<tr>
<td>months</td>
<td>m³</td>
<td>m³</td>
<td>m</td>
<td>m</td>
<td>m</td>
<td>m</td>
<td>m</td>
<td>m</td>
<td>m³/d</td>
</tr>
<tr>
<td>6</td>
<td>6.63</td>
<td>4.21</td>
<td>1.50</td>
<td>0.50</td>
<td>2.80</td>
<td>12.80</td>
<td>0.32</td>
<td>3.37</td>
<td>6.22</td>
</tr>
</tbody>
</table>

| sludge l/g BODrem | 0.0046 |

### COD removal 27% BOD removal 29%

Imhoff or Emscher tanks are typically used for domestic or mixed wastewater flows above 3 m³/day (Sasse, 1998). Our system would have a flow of approximately 144 m³/day, which is significantly higher than the minimum recommended flow. Because the flow is much higher than the 3 m³/d further research and designs would be needed to insure viability of the DEWATS standards are accurate for our final design.

To verify the components of the DEWATS design, alternative design criteria were consulted. The design criteria used for the design of unheated Imhoff tanks is taken from the third edition of “Wastewater engineering: Treatment, disposal, and reuse.” by Metcalf and Eddy, Inc.(Metcalf & Eddy, 1935). The design criteria can be seen in Table 8.
Table 8: Typical design criteria for unheated Imhoff tanks (Tchobanoglous, 1991)

<table>
<thead>
<tr>
<th>Design Parameter</th>
<th>Unit</th>
<th>Range</th>
<th>Typical</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Settling Compartment</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Overflow rate peak hour</td>
<td>gal/ft² · d</td>
<td>600 - 1,000</td>
<td>800</td>
</tr>
<tr>
<td>Detention time</td>
<td>h</td>
<td>2 - 4</td>
<td>3</td>
</tr>
<tr>
<td>Length to Width</td>
<td>ratio</td>
<td>2:1 - 5:1</td>
<td>3:1</td>
</tr>
<tr>
<td>Slope of settling compartment</td>
<td>ratio</td>
<td>1.25:1 to 1.75:1</td>
<td>1.5:1</td>
</tr>
<tr>
<td>Slot opening</td>
<td>in</td>
<td>6 - 12</td>
<td>10</td>
</tr>
<tr>
<td>Slot overhang</td>
<td>in</td>
<td>6 - 12</td>
<td>10</td>
</tr>
<tr>
<td>Scum Baffle</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Below surface</td>
<td>in</td>
<td>10 - 16</td>
<td>12</td>
</tr>
<tr>
<td>Above surface</td>
<td>in</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>Freeboard</td>
<td>in</td>
<td>18 - 24</td>
<td>24</td>
</tr>
<tr>
<td><strong>Gas vent area</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Surface area % of total surface area</td>
<td></td>
<td>15 - 30</td>
<td>20</td>
</tr>
<tr>
<td>Width of opening&lt;sup&gt;a&lt;/sup&gt;</td>
<td>in</td>
<td>18 - 30</td>
<td>24</td>
</tr>
<tr>
<td><strong>Digestion Section</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Volume (unheated)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storage capacity</td>
<td>ft³/capita</td>
<td>2 - 3.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Sludge withdrawal pipe</td>
<td>in</td>
<td>8 - 12</td>
<td>10</td>
</tr>
<tr>
<td>Depth below slot to top of sludge</td>
<td>ft</td>
<td>1 - 3</td>
<td>2</td>
</tr>
<tr>
<td><strong>Tank Depth</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water surface to tank bottom</td>
<td>ft</td>
<td>24 - 32</td>
<td>30</td>
</tr>
</tbody>
</table>

<sup>a</sup> Minimum width of opening must be 18 in to allow for a person to enter for cleaning.

<sup>b</sup> Based on a six-month digestion period.

Note: 
\[
\text{gal/ft}^2 \cdot \text{d} \times 0.0404 = \text{m}^3/\text{m}^2 \cdot \text{d}
\]
\[
\text{in} \times 25.4 = \text{mm}
\]
\[
\text{ft}^3 \times 2.8317 \times 10^{-2} = \text{m}^3
\]
\[
\text{ft} \times 0.3048 = \text{m}
\]

These standards give a range of values for many of the designed components that allow for a preliminary design to be conducted. These standards are from years of operating experience and are not all from theoretical and mathematical modeling of the system. When designing the system specifically for Carabuela, it is not possible to achieve the typical and sometimes even stay within the range of these criteria. For example because of the variable flows entering the system, it would not be possible to maintain the overflow rate and detention time within the specified range.

The two sources of design criteria lead to a comprehensive preliminary design. The main changes that were made to the DEWATS system were the sedimentation chamber overhang and the angle of the sedimentation chamber walls. The sedimentation chamber overhang was not accounted for in the DEWATS design, but the Metcalf and Eddy typical design criteria require an 18 inch overhang of the
lower sedimentation wall. This overhang increases the depth of the tank. It is also recommended that there is an 18 inch clearance from the top of the sludge to the sedimentation chamber. For both of these 18 inch additions, a metric equivalent was necessary, and they were taken to be 0.5 meters, approximately 19.7 inches. The second change that was made to the DEWATS system was the angle of the sedimentation chamber which was increased to about a 2:1 ratio. This change was made because of the recommendations from Anne Mikelonis who has worked on Imhoff tanks in developing countries. The increased slope decreases the potential for the settled solids to plume back upwards in the sedimentation chamber. The slope was only increased to 2:1 because there would be a significant addition of height to the tank if higher slopes were used.

A schematic drawing of the preliminary design for the Imhoff tank can be seen in Figure 6.

![Figure 6: Imhoff Tank Final Design](image)

6.3.5. Maintenance

As with any working system maintenance is a vital part of the continuing operation of the design. The maintenance can be divided into two categories: daily and monthly operation maintenance and long term maintenance.

6.3.5.1. Daily and monthly operation

The daily and monthly operation maintenance can be sub-divided into five sections. First, the sedimentation chamber must be skimmed. All floating solids should be skimmed from the surface, and the material removed should be placed in the gas vent or buried (Texas Water Commision, 1991). Second, the total submerged interior surfaces of the chamber sides, ends, and sloping walls should be squeegeed to remove solids adhering to them (Texas Water Commision, 1991). Third, to be assured that all solids slide into the digestion compartment and that no obstructions exist along the slot, one lowers a chain through the slots and then proceeds from one end to the other end of the tank in a sawing type motion (Texas Water Commision, 1991). After these processes are complete, the skimming process should be repeated to remove any additional floating solids. Fourth, one should break apart the scum in the gas vents with a scum hoe to ensure proper escape of gases resulting from digestion of sludge and to aid in settling of the solids trapped in the scum (Texas Water Commision, 1991). Any scum which will not settle should be removed from the scum chamber and be buried to prevent odors and fly and mosquito breeding (Texas Water Commision, 1991). Finally, at least once a week the elevation of the sludge blanket should be measured. The sludge level should be 18 inches or more below the slots of the sedimentation chamber (Texas Water Commision, 1991).
6.3.5.2. Long term maintenance

When the level of the sludge in the digestion compartment reaches within 18 inches of the sedimentation slot or about every 6 months, the entire tank must be drained. The water and sludge can be removed by pumps or by hand. The wet sludge would then be sent to the drying bed. This maintenance would require significant time and work but would only be needed infrequently.

In order to keep the tanks operating correctly and optimally, it would be necessary to educate the residents the proper way to maintain the system. To accomplish this, a manual would need to be created, translated and sent to the residents.

6.3.6. Construction Alternatives

The Imhoff tank could be built and designed several ways. Two construction alternatives are possible and vary in construction materials used. A new tank made of typical concrete could be built or a new tank made of ferrocement could be used. For each method it would be necessary to design for redundancy and so the final design of the tank would involve a tank with two independent cells. Each cell could be opened or closed independent of the status of the other cell. This is to account for the condition that one cell is closed for maintenance and sludge removal. This allows for continued operation even during the required maintenance.

Building a typical concrete tank could pose some problems when it comes to construction. Forming the tank could be a problem because the walls of the tank would be high and would require concrete to be pumped up to the top of the forms. Concrete pumps and cranes which would typically be used in construction in the United States are not available in the remote regions of Ecuador. Construction of the tank could be difficult depending on the availability of skilled concrete workers. Also the tank would require a significant number of reinforcing steel bars which would add to the construction cost. The main advantage of using typical concrete for the construction is that it is a very proven and well known technology that is durable and dependable. The alternative to typical concrete construction is using ferrocement.

Ferrocement is a type of thin wall reinforced concrete commonly constructed of hydraulic cement mortar reinforced with closely spaced layers of continuous and relatively small size wire mesh (Naaman, 2000). The main advantages of using ferrocement as compared to typical reinforced concrete are as follows:

- Thinner material
- Ferrocement has a high reinforcement ratio in both tension and compression
- Smaller crack widths which provide excellent leakage characteristics for water tanks
- Good durability under various environmental exposure
- Can accommodate lower levels of technology because it requires less mechanization and less heavy equipment
- Easy to repair and maintain

Using ferrocement would provide many advantages for the design of any concrete construction for our project. It is a technology that would be very appropriate for construction in developing areas.

6.3.6.1. Cost Estimate

Ferrocement and typical concrete construction were both analyzed for a preliminary cost estimate. First, the cost of designing a new tank made of typical concrete construction was analyzed. For this analysis, the cost of the concrete, the cost of steel, the cost of construction, and the cost of piping are needed. To calculate the cost of concrete, the volume was calculated from schematic drawings. Values for the amount
of steel and the length of pipe were estimated. For construction work, the time of construction was estimated to be three workers working for five days, eight hours each day.

Second, the cost of designing a tank made of ferrocement was analyzed. For this analysis, chapter 8, Cost Estimates, of “Ferrocement and Laminated Cementitious Composites” by Antoine Naaman was used to estimate the cost of the construction. To approximate the cost for Ecuador, the costs of the construction from a similar case study in Mexico were used. The cost data was taken in 1980 so it was necessary to translate the cost to the current market cost. The cost of construction using ferrocement is estimated to be $51 per plan view square meter, which includes the cost for reinforcement, the cost of mortar, and the cost of construction.

The complete cost estimate can be seen in Table 9.

### Table 9: Imhoff Tank Cost Estimate

<table>
<thead>
<tr>
<th></th>
<th>New Typical Concrete Tank</th>
<th>New Ferrocement Tank</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Cost of Concrete</strong></td>
<td><strong>Cost of Ferrocement Construction</strong></td>
<td></td>
</tr>
<tr>
<td>Concrete Price</td>
<td>$77.00</td>
<td>Ferrocement Price</td>
</tr>
<tr>
<td>Total Volume</td>
<td>78.88 m³</td>
<td>Plan view area</td>
</tr>
<tr>
<td>Total Concrete Cost</td>
<td>$6,073.45</td>
<td>Total Concrete Cost</td>
</tr>
<tr>
<td>Cost of Steel rebar</td>
<td></td>
<td>Cost of Piping</td>
</tr>
<tr>
<td>Rebar Cost</td>
<td>1400</td>
<td>Length</td>
</tr>
<tr>
<td>Total Rebar Cost</td>
<td>$1,400.00</td>
<td>Cost of Pipe</td>
</tr>
<tr>
<td><strong>Cost of Construction</strong></td>
<td></td>
<td>Total Cost of Pipe</td>
</tr>
<tr>
<td>Hours</td>
<td>120</td>
<td></td>
</tr>
<tr>
<td>Price Skilled Labor</td>
<td>$3.00</td>
<td></td>
</tr>
<tr>
<td>Total Construction Cost</td>
<td>$360.00</td>
<td></td>
</tr>
<tr>
<td><strong>Cost of Piping</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length</td>
<td>50 ft</td>
<td></td>
</tr>
<tr>
<td>Cost of Pipe</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Total Cost of Pipe</td>
<td>$250.00</td>
<td></td>
</tr>
<tr>
<td>Total Cost</td>
<td>$8,083.45</td>
<td>Total Cost</td>
</tr>
</tbody>
</table>

The final cost for building a redundant Imhoff tank system would depend on the choice of the construction method. For building two tanks out of typical concrete it would cost approximately $8,083.45. For building two tanks out of ferrocement it would cost approximately $6,415.90.

6.3.6.2. Feasibility

Changing from the current system with a septic tank to a system with an Imhoff tank would not be a large difference in physical size or method of treatment. The main effects of implementing an Imhoff tank would be seen in the effluent quality, the maintenance of the tank and the initial construction. Since the positive effects of implementing the Imhoff tank are purely environmental, the cost feasibility is difficult to compute. The amount of increase in quality versus cost must be evaluated without the addition of any money being recovered. The possibility of using the old septic tank in the design of the new tank is an option that would increase the appropriateness of the project and decrease the associated cost of the project. It was initially theorized that the current septic tank could be modified to be used as an Imhoff tank. Unfortunately, after preliminary designs of the necessary dimensions of the tank it was determined
that this would not be possible. The current septic tank has dimensions of 3.5 x 2 x 11.5m which is not compatible with the Imhoff tank dimensions of 6.2 x 5.8 x 12.8m. The land required for the Imhoff tank is not significantly larger than that of the current septic tank so little land would need to be acquired to build a new tank.

6.4. Stabilization Pond

6.4.1. Background

The third process of the wastewater system involves a stabilization pond, which will reduce large amounts of BOD, COD and TSS in the waste stream. Stabilization ponds can either be aerated, anaerobic or aerobic. The information presented here will discuss the advantages and disadvantages of each type of stabilization pond, but first let’s take a look at the constraints that may affect our design choice.

The stabilization pond of our choice will be placed at a location 500 meters away from the original septic tank. The vertical drop in this distance is about 1%, which means that the waste stream entering the lagoon will be 5 meters below the point at which waste stream leaves the Imhoff Tank. Because of the landscape in this region, an inverted siphon is required. An inverted siphon is the makeup of a sewage pipe that goes under an obstruction. The wastewater will flow under pressure in this particular section.

6.4.2. Design Alternatives

6.4.2.1. Aerated Ponds

Aerated ponds differ from aerobic or anaerobic ponds by the fact they are equipped with mechanical aerators or submerged pipes that supply oxygen. This is an advantage over aerobic ponds, which uses oxygen from photosynthesis and surface reaeration. More BOD is removed (60 to 90%) as well as COD (70 to 90%) and TSS (70 to 90%) (Martin & Martin, 1991). Another advantage of this system is that it requires less land because it has a higher oxygen content to degrade organic matter. In the case of our wastewater system, constructing an aerated lagoon would be troublesome because of its complexity and cost. The people of Carabuela do not have the necessary resources or funds to operate such a design. They are more interested in a design that is simpler and easier to operate and maintain.

6.4.2.2. Anaerobic Ponds

Anaerobic ponds are deeper than aerobic ponds, are heavily loaded with strong organic waste, and contain large amounts of anaerobic microorganisms that quickly deplete any oxygen that might be available in the influent (Okun, 1975). These ponds are often used to treat strong organic industrial wastes. We would not be interested in this type of stabilization pond because it is not as efficient as the other two. You would see this types of pond used to convert methane gas into energy, which is a highly sophisticated process.

6.4.2.3. Aerobic Ponds

Aerobic ponds operate off microbial reactions. Organic materials are bio-oxidized, giving off CO2, NH3 and inorganic radicals. Algae use CO2, inorganic radicals and sunlight to produce dissolved oxygen in a cyclic-symbiotic relationship (Reynolds & Richards, 1996). The principle advantage of this pond is that it removes pathogens at a much lower cost than any other forms of treatment (Martin & Martin, 1991). Table X lists some of the parameters of an aerobic lagoon provided by Unit Operations on page X. The deeper the lagoon, the less oxygen there will be at the bottom; therefore, designing a lagoon with a shallow depth is optimal because more sunlight can penetrate throughout the pond creating a stronger photosynthesis reaction. Carabuela is located high up in the mountains and achieves warm climate year-
round (50 – 72 degrees Fahrenheit). This is a great place to use an aerobic lagoon, which thrives off warm temperature climates. There are two types of aerobic ponds. They differ by the total depth with one being approximately 15 to 46 cm and the other being 1.5 meters deep. The shallow depth pond contains high populations of algal growth and the other contains a high population of bacteria (Martin & Martin, 1991).

Aerobic ponds should be cleaned periodically. It is important to remove grass and other plant growth from the surrounding area and in the pond itself. Floating scum on top of the lagoons should be removed, or oxygen transfer is impaired. If large amounts of scum are either black or brown, this is an indication that the lagoon is being overloaded (Martin & Martin, 1991).

Table 10: Typical Design Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Aerobic Pond</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lagoon Size, ha</td>
<td>&lt;4</td>
</tr>
<tr>
<td>Detention time, d</td>
<td>10 - 40</td>
</tr>
<tr>
<td>Depth</td>
<td>1 - 1.5</td>
</tr>
<tr>
<td>Opt. temp, degrees Celsius</td>
<td>20</td>
</tr>
<tr>
<td>BOD5 loading, kg/ha/d</td>
<td>40 - 120</td>
</tr>
<tr>
<td>BOD5 conversion, %</td>
<td>80 - 95</td>
</tr>
<tr>
<td>Principal conversion products</td>
<td>Algae, CO2, cells</td>
</tr>
<tr>
<td>Algal concentration, mg/L</td>
<td>40 - 100</td>
</tr>
</tbody>
</table>

6.4.3. Larger Lagoon

During one of our team’s meeting with our industrial consultant it was suggested that we build a larger stabilization pond. In that way the Imhoff tank in our system would be unnecessary, resulting in less maintenance. The effluent quality would be just the same with or without the Imhoff tank. The principle disadvantage of a larger lagoon is space. The amount of space that was suggested for a lagoon originally was 1 acre. In order to produce the same effluent quality without the Imhoff tank there would need to be 27 acres allotted for the stabilization pond. To add to the difficulty, most of Carabuela is situated on a hill, which would make the location of 27 available acres that are flat quite troublesome. If we were to implement a 27 acre lagoon the costs for land alone would be around $200,000.

6.4.4. Feasibility and Costs

The best design alternative would be to use an aerobic stabilization pond with an Imhoff tank. These ponds are by far the most common in developing countries (Okun, 1975). A condition that was given to us is that the stabilization pond must not exceed 1 acre, which is plenty of land availability. Therefore, the pond can be shallow to allow oxygen transfer from the surrounding air to mix throughout the entire depth of water. Constructing an aerobic lagoon should not be a problem. Excavated earth could be used for the dikes surrounding the pond (Okun, 1975). One problem that we might face is creating enough pressure to ensure that the waste stream can enter the aerobic pond due to an inverted siphon positioned just before the entrance. Most of the costs will occur during the construction phase. Land costs average to be $2 per square meter. If the stabilization pond is in fact 5,000 square meters, the total cost of land for the lagoon will be $10,000. This does not include labor for digging or maintaining the pond.
6.5. Sludge Handling

6.5.1. Introduction

After each treatment process, the sludge produced will need to be treated. Often this involves aerobic or anaerobic digestion, but with an Imhoff tank this is done during the settling stage. The sludge will be piped to the dewatering stage. This decreases the volume by up to three-quarters and makes the sludge handle as a solid. There are several types of dewatering beds: open-air sand beds, covered sand beds, and several more recent types such as vacuum-assisted beds, paved drying beds, and wedgewire or plastic-bottomed beds. Vacuum-assisted drying, wedgewire-bottomed beds, and plastic-bottomed beds all require substantial energy inputs, chemical inputs, sophisticated equipment, and trained operators, and are therefore not appropriate to the situation. If the large lagoon option is adopted, drying beds will be unnecessary. The sludge will have to be removed approximately every twenty years and incinerated, brought to a landfill, or used as fertilizer.

6.5.2. Design Criteria

Sand drying beds typically consist of a layer of gravel with underdrains, a layer of sand, and vertical partitions. The gravel layer is typically 25-35 cm thick. The sand layer on top of it is typically 15-25 cm thick. The pipes are no less than 10 cm in diameter and no more than six meters apart. The walls should be watertight and extend at least 40 cm above and 15 cm below the sludge. Drying occurs by way of two processes: percolation and evaporation. Percolation is complete after one to three days, while evaporation takes from weeks to months, depending on the climate. The sludge dries until it is approximately thirty-five percent solids, at which point it handles like a solid; it is then manually removed and transported to land disposal or incineration. It can be useful for fertilizer if the level of pathogens is acceptable. Alternatively, the waste could be sent to a landfill or incinerator, if one is feasibly close. The final alternative for disposal of dried sludge is to simply move it to a lagoon, fill it up, and abandon it. This periodically requires a new lagoon, but leaves fertile land in its wake.

**Table 11: Drying Bed Design Criteria**

<table>
<thead>
<tr>
<th></th>
<th>Open bed</th>
<th></th>
<th>Closed bed</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Avg</td>
<td>Min</td>
<td>Max</td>
<td>Avg</td>
</tr>
<tr>
<td>Area (m²)</td>
<td>567</td>
<td>378</td>
<td>756</td>
<td>405</td>
</tr>
<tr>
<td>Solids Handled (kg/yr)</td>
<td>48195</td>
<td>32130</td>
<td>64260</td>
<td>52650</td>
</tr>
<tr>
<td>Sand required (m³)</td>
<td>113.4</td>
<td>75.6</td>
<td>151.2</td>
<td>81</td>
</tr>
<tr>
<td>Gravel required (m³)</td>
<td>187.11</td>
<td>124.74</td>
<td>249.48</td>
<td>133.65</td>
</tr>
</tbody>
</table>

6.5.3. Feasibility

Considering the level of sophistication appropriate to Carabuela, drying beds are the best choice; they only require land for disposal, a minimal amount of construction and materials, and manual labor to remove the sludge afterwards. They have no moving parts or electrical requirements; some drying beds use chemical coagulants to quicken the process, but this is not required. Drying beds can be open to the air or covered by a greenhouse. Bed area is determined by empirical formulas based on the number of people to be served. The required area will depend on climate, so the results are given as a range. Problems with drying beds can include odors and flies associated with large areas of waste. The other problem is Carabuela’s cool and rainy climate. Its average temperatures hover between 50° F and 60° F year-round. Carabuela also receives about 99 inches of rainfall a year. In the rainy season, about 14 inches can fall in a month; by comparison, in an average year Grand Rapids gets 4.2 inches in its rainiest
month. On the other hand, this might lessen the problems with odors and flies. Using a greenhouse can treat both problems, since it sequesters the odors and controls the temperature; it would require ventilation to control humidity. Construction costs for a greenhouse are partly offset by the reduced land area required. With either covered or open drying beds, obtaining the required land area (a square 20-23 m to a side) would not be difficult.

6.5.4. Cost Estimate

Labor and land costs dominate the total cost. Assuming average conditions, the drying beds would probably cost slightly over $3000.

Table 12: Drying Bed Costs

<table>
<thead>
<tr>
<th></th>
<th>Average</th>
<th>Minimum</th>
<th>Maximum</th>
<th>Average</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$1,134.00</td>
<td>$756.00</td>
<td>$1,512.00</td>
<td>810.00</td>
<td>$540.00</td>
<td>$1,080.00</td>
</tr>
<tr>
<td>Land Cost</td>
<td>$170.10</td>
<td>$113.40</td>
<td>$226.80</td>
<td>121.50</td>
<td>$81.00</td>
<td>162.00</td>
</tr>
<tr>
<td>Sand Cost</td>
<td>$280.67</td>
<td>$187.11</td>
<td>$374.22</td>
<td>200.48</td>
<td>$133.65</td>
<td>267.30</td>
</tr>
<tr>
<td>gravel Cost</td>
<td>$1,701.00</td>
<td>$1,134.00</td>
<td>$2,268.00</td>
<td>2,025.00</td>
<td>$1,350.00</td>
<td>2,700.00</td>
</tr>
<tr>
<td>Labor Cost</td>
<td>$3,285.77</td>
<td>$2,190.51</td>
<td>$4,381.02</td>
<td>3,156.98</td>
<td>$2,104.65</td>
<td>4,209.30</td>
</tr>
</tbody>
</table>

7. Additional Considerations

In addition to the primary design of the waste water system for the village of Carabuela our team is also interested in new technology in the field of water treatment. We are looking possibly doing some research into some of the newer water treatment processes. This research would not be feasible for use in our primary design because our primary design requires the use of proven technology. Since this technology has not been shown to be consistent in developing regions we will not be implementing any of our research. In particular we are interested into creating bench scale tests for a vortex grit chamber. We are interested in using a 55 gallon drum and modifying it to be used as a vortex chamber in high flow or be used as a traditional chamber in lower flows. We would modify the drum by welding partitions into the center of the drum to direct the flow in a tangential direction. Extensive research into the technology along with any patents would need to be done. Along with the grit chamber design we are also interested in constructed wetlands. This technology is a proven technology but would not be a cost effective way to treat the waste for our primary design. We would like to consider the possibility of creating a small model that would demonstrate how wetlands work and increase the public awareness of these environmentally friendly treatment systems. These research projects would not be associated with our primary design and so they would be added to our project only if there is adequate time for the primary design to be completed. These secondary projects would not be included in the submission of the designs to the people of Carabuela.
8. Budget

The available budget for our project is $300. This money will be spent on office supplies for our primary project. Costs for construction and management of the process will be funded by grants from the Ecuadorian government or the people of Carabuela. Our secondary project would require us to acquire materials such as 55 gallon drums, metal plates, pumps, piping and measurement equipment. We do not currently know if these secondary projects will be completed so the costs associated with them have been left out of the budget.

9. Schedule

Our team has created a preliminary schedule to plan out the major milestone dates. Our milestones include but are not limited to the following: project plan and feasibility study first draft, the final draft of the project plan and feasibility study, the preliminary design memo, bench scale design, analysis of bench scale data, and senior design night. Additional milestones will be added as necessary. To see the schedule in further detail, see the Appendix.

10. Conclusion

In summary, we will design a wastewater treatment program for the village of Carabuela in the highlands of Ecuador. We were connected with this project by HCJB Global, an organization that works to bring water services to rural areas. Currently waste from about 200 homes goes through an ineffective septic tank and leaching field into a stream. Carabuela has limited funds and technological expertise, so complicated or expensive systems are inappropriate for this project. We will design a bar rack and grit chamber to remove solids, a settling system to remove particulates, digestion to reduce pathogens, a lagoon to remove organic matter, and drying beds for disposal of the sludge. Another alternative is to construct a larger lagoon and eliminate the Imhoff Tank, but a cost analysis suggests that this would be very unfeasible, and the amount of land required would be unattainable. The preliminary budget estimate is $15,000.

Another possible project for this community is a storm water management system. Presently drains from rooftops run into the sanitary sewers, overloading the waste treatment system. The village already has a satisfactory drinking water system.

An effective treatment system will prevent contamination of the stream and provide water for irrigation; the sludge produced can be used as fertilizer. Water-borne diseases, many of them fecal in origin, are some of the greatest health hazards in developing countries, and are especially dangerous to young children; our system will greatly reduce the risk.
11. Acknowledgements

We would like to thank the following contributors for their invaluable assistance in our preliminary design work:

Professor Leonard De Rooy [Senior Design Advisor]

Professor De Rooy advised and mentored our team throughout the project. He provided value information about project management and construction practices. He also connected us with our industrial consultant.

Bruce Rydbeck [HCJB contact]

Bruce brought this project to our attention. He continued to be our contact throughout the project. He provided data about the location along with local expertise.

Professor David Wunder [Environmental Engineer]

Professor Wunder helped us find our project contact along with providing knowledge pertaining to the field of wastewater treatment.

Anne Mikelonis [CDM Engineer]

Anne Mikelonis provided knowledge and experience to the design of Imhoff tanks and construction practices in developing nations.

Janice Skadsen [CDM Engineer]

Janice Skadsen provided knowledge to the treatment and disposal of the sludge byproduct.

Tom Newhof [Prein & Newhof]

Tom Newhof is our industrial consultant. He has been very helpful with information about designing wastewater treatment systems. He has also provided us with several very helpful contacts.
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http://water.me.vccs.edu/courses/ENV149/changes/Feat11_picII-1.jpg


13. Appendix