Cleaning Up Carabuela:
What to do with Number Two

Team 7
Ian Compton
Adam DeYoung
Josh Scheenstra
Nathan Williams

Engineering 339/340 Senior Design Project
Calvin College

2012-2013
2012 Team 7, Calvin College
Executive Summary

Initial Project Summary
Team 7 collaborated with HCJB (Heralding Christ Jesus’ Blessings), an international missions organization, to design a wastewater treatment facility for a rural community in Ecuador. The community, Carabuela, has a collection system for their wastewater but no treatment method for the waste stream before it is discharged into a nearby river. The task for team 7 is to design a wastewater treatment process that is passive, requires little maintenance, and has low upfront and yearly costs.

Partnering Organization
HCJB is headquartered in Ecuador, although it has extensive branches in other parts of South and Central America as well as parts of Africa and Asia. The organization is divided into two main branches; one that focuses on radio and media ministry and one that focuses on healthcare and community development work. Team 7 worked with the staff engineers in the community development office in Quito, Ecuador on this project. Typically, HCJB works only with clean drinking water infrastructure and has had little experience in large-scale waste treatment.

Map of Ecuador\textsuperscript{1}

\textsuperscript{1} From http://www.ezilon.com/maps/images/southamerica/Ecuador-map.gif. Carabuela highlighted.
**Treatment Facility**
After researching treatment options and traveling to Carabuela, team 7 recommends using a system of septic tanks in series with constructed wetlands in order to treat waste to an acceptable level of discharge. Each septic tank holds about 180 cubic meters of waste. The constructed wetlands are designed in 20m by 30m cells that can be augmented in the future to accommodate community growth and expansion. The effluent stream could be used for local irrigation as an added benefit of treatment.

![Carabuela: Nestled in the shadow of Mt. Cotacachi](image)

**Costs**
An initial constraint given by HCJB was a target cost of $50,000 (US) for the system without considering labor. The primary costs associated with the design are the building and fill material costs. The total of these will be about $75,000 (US). This cost includes piping, estimated land costs, and a 30% contingency.

---

2 Photo courtesy of Nathan Williams
3 Drawn by Joshua Scheenstra
Implementation
Team 7’s design options and recommendations have been sent to their HCJB contact in Quito for evaluation. The recommendations will be presented to the Water Board in Carabuela for discussion and implementation. If the Water Board so chooses, they can build the design in Carabuela in conjunction with HCJB and the Ecuadorian government.
# Table of Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Executive Summary</td>
<td>iii</td>
</tr>
<tr>
<td>Table of Contents</td>
<td>vi</td>
</tr>
<tr>
<td>Table of Figures</td>
<td>ix</td>
</tr>
<tr>
<td>Table of Tables</td>
<td>xi</td>
</tr>
<tr>
<td>Table of Equations</td>
<td>xii</td>
</tr>
<tr>
<td>1. Introduction</td>
<td>1</td>
</tr>
<tr>
<td>1.1 The Organization</td>
<td>1</td>
</tr>
<tr>
<td>1.2 The Team</td>
<td>1</td>
</tr>
<tr>
<td>1.3 Project Background</td>
<td>2</td>
</tr>
<tr>
<td>1.4 Project Motivations</td>
<td>6</td>
</tr>
<tr>
<td>1.5 Carabuela in Context</td>
<td>6</td>
</tr>
<tr>
<td>1.6 Climate</td>
<td>7</td>
</tr>
<tr>
<td>1.7 Village Demographics</td>
<td>9</td>
</tr>
<tr>
<td>2. Existing Conditions</td>
<td>10</td>
</tr>
<tr>
<td>2.1 The Collection System</td>
<td>10</td>
</tr>
<tr>
<td>2.2 The Treatment Plant</td>
<td>11</td>
</tr>
<tr>
<td>3. Design Parameters</td>
<td>13</td>
</tr>
<tr>
<td>3.1 Flows and Loads</td>
<td>13</td>
</tr>
<tr>
<td>3.2 Footprint Constraints</td>
<td>14</td>
</tr>
<tr>
<td>3.3 Effluent Standards</td>
<td>15</td>
</tr>
<tr>
<td>3.4 Costs</td>
<td>16</td>
</tr>
<tr>
<td>4 Preliminary Design</td>
<td>17</td>
</tr>
<tr>
<td>4.1 Collection System</td>
<td>17</td>
</tr>
<tr>
<td>4.2 Treatment Process</td>
<td>17</td>
</tr>
<tr>
<td>4.3 Decision Matrix</td>
<td>17</td>
</tr>
<tr>
<td>5 Final Design</td>
<td>20</td>
</tr>
<tr>
<td>5.1 Bar Screen</td>
<td>22</td>
</tr>
<tr>
<td>5.2 Septic Tanks</td>
<td>23</td>
</tr>
</tbody>
</table>
Appendix C .......................................................................................................................................................... 24
5.3 Distribution Box .......................................................................................................................................... 24
5.4 Constructed Wetlands ............................................................................................................................... 25
5.5 Piping ......................................................................................................................................................... 31
5.6 Irrigation Effluent ...................................................................................................................................... 36
5.7 Infiltration Considerations ....................................................................................................................... 40
5.8 Residuals .................................................................................................................................................... 42
5.9 Construction and Phasing .......................................................................................................................... 42
5.10 Maintenance ............................................................................................................................................ 43
5.11 Costs ....................................................................................................................................................... 44
5.12 Sustainability .......................................................................................................................................... 45
5.13 Implementation ..................................................................................................................................... 46
Appendices .......................................................................................................................................................... I
Appendix A ...................................................................................................................................................... I
Flow Calculation ............................................................................................................................................... I
Appendix B ........................................................................................................................................................ II
Treatment Design Alternatives ........................................................................................................................ II
Bar Screens ....................................................................................................................................................... II
Grit Removal ................................................................................................................................................... III
Waste Stabilization Ponds ............................................................................................................................ III
Septic tanks .................................................................................................................................................... V
Bio-filtration .................................................................................................................................................... VI
Constructed Wetlands ................................................................................................................................ VI
Ground Infiltration ........................................................................................................................................ VII
Preliminary Treatment Decision Matrix .......................................................................................................... VIII
Appendix C ........................................................................................................................................................ X
Preliminary Design ........................................................................................................................................ X
  6.1 Collection System .................................................................................................................................... X
  6.2 Septic Tank and Wetlands ....................................................................................................................... XVIII
  6.3 Waste Stabilization Ponds ..................................................................................................................... XIX
  6.4 Secondary Decision Matrix .................................................................................................................... XXI
Appendix D ........................................................................................................................................................ XXIII
Constructed Wetland Calculations .................................................................................................................. XXIII
Appendix E ........................................................................................................................................XXXIII

Septic Tank Calculations ..................................................................................................................XXXIII

Appendix F ........................................................................................................................................XLVII

Waste Stabilization Pond Calculations ..............................................................................................XLVII

Appendix G ........................................................................................................................................LIII

Bar Screen Sizing .................................................................................................................................LIII

Appendix H ........................................................................................................................................LIV

Residuals Disposal ...............................................................................................................................LIV

Appendix I ........................................................................................................................................LV

Detailed Costs ...................................................................................................................................LV

Works Cited ......................................................................................................................................LVII

Web Sites .........................................................................................................................................LVIII
# Table of Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure 1:</td>
<td>Polluted River in Carabuela</td>
<td>3</td>
</tr>
<tr>
<td>Figure 2:</td>
<td>Carabuela in &quot;Volcano Alley&quot;</td>
<td>4</td>
</tr>
<tr>
<td>Figure 3:</td>
<td>Overview Map of Carabuela</td>
<td>5</td>
</tr>
<tr>
<td>Figure 4:</td>
<td>Physical Map of Ecuador</td>
<td>6</td>
</tr>
<tr>
<td>Figure 5:</td>
<td>National Flag of Ecuador</td>
<td>7</td>
</tr>
<tr>
<td>Figure 6:</td>
<td>Average Yearly Temperatures (°C) in Otavalo, Ecuador</td>
<td>8</td>
</tr>
<tr>
<td>Figure 7:</td>
<td>Average Yearly Rainfall (mm) in Otavalo, Ecuador</td>
<td>8</td>
</tr>
<tr>
<td>Figure 8:</td>
<td>Overview of Carabuela. Looking East</td>
<td>9</td>
</tr>
<tr>
<td>Figure 9:</td>
<td>Current Configuration (Offline)</td>
<td>11</td>
</tr>
<tr>
<td>Figure 10:</td>
<td>Existing Septic Tank and Leach Field</td>
<td>12</td>
</tr>
<tr>
<td>Figure 11:</td>
<td>Eroded Leach Field</td>
<td>12</td>
</tr>
<tr>
<td>Figure 12:</td>
<td>Effluent Bucket Tests. Greywater Clearly Visible</td>
<td>14</td>
</tr>
<tr>
<td>Figure 13:</td>
<td>Wetland Footprint vs. BOD Concentration</td>
<td>16</td>
</tr>
<tr>
<td>Figure 14:</td>
<td>Proposed Waste Treatment Process Flow Diagram</td>
<td>20</td>
</tr>
<tr>
<td>Figure 15:</td>
<td>Plan View Layout</td>
<td>21</td>
</tr>
<tr>
<td>Figure 16:</td>
<td>Treatment Location in Relation to Carabuela</td>
<td>21</td>
</tr>
<tr>
<td>Figure 17:</td>
<td>Bar Screen Structure</td>
<td>22</td>
</tr>
<tr>
<td>Figure 18:</td>
<td>Septic Tank Plan and Profile Views</td>
<td>23</td>
</tr>
<tr>
<td>Figure 19:</td>
<td>Percolation Test near Septic Tank</td>
<td>24</td>
</tr>
<tr>
<td>Figure 20:</td>
<td>Plan and Profile Views of Distribution Box</td>
<td>25</td>
</tr>
<tr>
<td>Figure 21:</td>
<td><em>Phragmites australis</em></td>
<td>26</td>
</tr>
<tr>
<td>Figure 22:</td>
<td><em>Pennisetum purpureum</em> (Elephant Grass)</td>
<td>26</td>
</tr>
<tr>
<td>Figure 23:</td>
<td>Typical Inlet Structure</td>
<td>27</td>
</tr>
<tr>
<td>Figure 24:</td>
<td>Typical Outlet Structure</td>
<td>28</td>
</tr>
<tr>
<td>Figure 25:</td>
<td>Proposed Treatment Site</td>
<td>29</td>
</tr>
<tr>
<td>Figure 26:</td>
<td>Constructed Wetland Terrace Overview</td>
<td>30</td>
</tr>
<tr>
<td>Figure 27:</td>
<td>CW Terrace Detail</td>
<td>30</td>
</tr>
<tr>
<td>Figure 28:</td>
<td>Effluent Pipe Layout of Wetlands With Relative Node Elevations (in feet)</td>
<td>32</td>
</tr>
<tr>
<td>Figure 29:</td>
<td>Effluent Pipe Diameters (in feet)</td>
<td>33</td>
</tr>
<tr>
<td>Figure 30:</td>
<td>Effluent Pipe Velocities</td>
<td>34</td>
</tr>
<tr>
<td>Figure 31:</td>
<td>Effluent Pipe Max Capacities</td>
<td>34</td>
</tr>
<tr>
<td>Figure 32:</td>
<td>Longest Section of Pipe Water Elevation Profile</td>
<td>35</td>
</tr>
<tr>
<td>Figure 33:</td>
<td>Process Flow Diagram with Piping Schedule</td>
<td>36</td>
</tr>
<tr>
<td>Figure 34:</td>
<td>Wastewater Treatment Options and Associated Log Removals</td>
<td>39</td>
</tr>
<tr>
<td>Figure 35:</td>
<td>Manually Cleaned Bar Screen Structure Plan and Profile Views</td>
<td>39</td>
</tr>
<tr>
<td>Figure 36:</td>
<td>Waste Stabilization Pond Layout</td>
<td>40</td>
</tr>
<tr>
<td>Figure 37:</td>
<td>Typical Septic Tank Design</td>
<td>41</td>
</tr>
<tr>
<td>Figure 38:</td>
<td>Typical Bio-Filtration Setup</td>
<td>41</td>
</tr>
</tbody>
</table>
Table of Tables

Table 1: Major Constituents of Typical Domestic Wastewater .......................................................... 13
Table 2: Current and Projected Sanitary Flows at 2% Growth .......................................................... 14
Table 3: Fresh Water Discharge Effluent Standards in Ecuador ......................................................... 15
Table 4: MDEQ Standards for Effluent Discharge into Ground Water .............................................. 15
Table 5: Secondary Design Cost Summary ......................................................................................... 18
Table 6: Secondary Decision Matrix ................................................................................................. 18
Table 7: Final Effluent Pipe Lengths and Diameters ........................................................................ 33
Table 8: Total Piping Schedule .......................................................................................................... 35
Table 9: Some Common Pathogens Carried in Wastewater ............................................................. 37
Table 10: Survival of various Organisms in Selected Environmental Media at 20-30 °C ............... 38
Table 11: Verification Monitoring* (E.Coli numbers per 100ml of treated wastewater) for the various levels of wastewater treatment in Options A-G presented in Figure 35 ........................................ 40
Table 12: Recommended Rates of Wastewater Application for Trench and Bed Bottom Areas* ...... 41
Table 13: Wetland Cells Needed Over Time ...................................................................................... 43
Table 14: Septic Tank Routine Maintenance ...................................................................................... 43
Table 15: Constructed Wetland Routine Maintenance ...................................................................... 43
Table 16: Basic Material Costs ........................................................................................................ 44
Table 17: Total Cost Summary .......................................................................................................... 45
Table 18: Effluent Bucket Tests ....................................................................................................... I
Table 23: Horizontal Flow Grit Chamber Design Criteria ................................................................. III
Table 24: Initial Treatment Options Decision Matrix ........................................................................ IX
Table 25: Preliminary Design 1.1 Pipe Lengths and Diameters ......................................................... X
Table 26: Collection System Flow Calculations ............................................................................ XII
Table 27: Sewer Size and Minimum Slope to Maintain a 2ft/s Flow Velocity ................................... XIV
Table 28: Piping Materials ................................................................................................................. XVIII
Table 29: Waste Stabilization Pond Design Parameters ................................................................. XX
Table 30: Pond Sizing ....................................................................................................................... XXI
Table 31: Secondary Design Summary ............................................................................................ XXI
Table 32: Secondary Decision Matrix ............................................................................................. XXII
Table 33: Total Piping Costs ........................................................................................................... LV
Table 34: Septic Tank Costs ........................................................................................................... LVI
Table 35: Wetland Liner vs. No Liner Comparison ........................................................................ LVI
Table 36: Constructed Wetland Costs ............................................................................................. LVI
# Table of Equations

Equation 1 (Mara, 1997) ........................................................................................................... IV  
Equation 2 (Mara, 1997) ........................................................................................................... IV  
Equation 3 (Mara, 1987) ........................................................................................................... IV  
Equation 4 (Mara, 1976) ........................................................................................................... IV
1. Introduction

1.1 The Organization

Team Carabuela is partnering with the international mission organization HCJB Global (Heralding Christ Jesus’ Blessings). HCJB is divided into two main branches; one that works primarily with healthcare and community development and one that works primarily with radio and media ministry. HCJB is based in Ecuador and has a strong presence in other parts of South and Central America as well as branches in parts in Africa, the Middle East, Southern Asia, and regions in the Pacific. HCJB links development work with the broadcasting of the gospel in order to bring Jesus Christ into all parts of the world.

Team 7 worked with the community development office in Quito, Ecuador (see “Vozandes Community Development” www.hcjb.org). This office is made up of several experienced staff engineers and executives that initiate, plan, and implement engineering projects all over Ecuador. Team 7’s main contact was Bruce Rydbeck, a civil engineer with over 30 years of experience in Ecuador. HCJB’s community development office works primarily with clean drinking water infrastructure and very rarely with large-scale wastewater projects.

1.2 The Team

Team 7: Cleaning up Carabuela is comprised of four civil/environmental engineering students. Each member brings a variety of experiences, passions, interests, skills, and backgrounds to the design project.

Ian Compton
Ian is a senior civil engineering student from Duanesburg, New York. He had the great opportunity to work with HCJB Global, the partnering mission organization of this project, in the summer of 2012. He worked specifically with the community development team in Quito on various clean water projects throughout Ecuador. He is privileged to have the opportunity to work with HCJB again. He is mostly interested in the hydraulic and environmental aspects of the project.

Adam DeYoung
Adam was born and raised in Hudsonville, Michigan. He has a desire to use his skills acquired in Calvin College’s Engineering program to provide clean water and quality water wherever God leads. He has previously been involved with varsity athletics in basketball and track and field at Calvin College. He has been involved in youth ministry for three years through a summer camp in Montana and Young Life in a local high school. This past summer, 2012, Adam worked as an intern at Vriesman & Korhorn Civil Engineers completing a Global Positioning System (GPS) survey, a pipe network survey, and new utility construction supervision. His gained experience using the GPS was put to use when the team
traveled to Ecuador in January. He desires to serve others with his engineering, by providing for their needs, and sharing the Gospel.

**Joshua Scheenstra**

Joshua Scheenstra is a senior at Calvin College who was born and raised in Kenya. His family has served as missionaries to an unreached people group for the past 27 years. He has participated in numerous community development projects in developing countries and many of them had to do with providing clean water. He has interned for Tulare Irrigation District the past three summers in California’s central valley. He is mostly interested in the hydraulics and structural aspects of civil engineering.

**Nathan Williams**

Nathan was born and raised in Howell, Michigan. He is a senior at Calvin College graduating in May 2013 with a Bachelor of Science in Engineering degree. He interned in the summer of 2012 at the City of Kentwood, working with the municipal engineering department where he worked with the city’s asset management programs involving storm water and road networks. He was recently hired as a hydraulic engineer at a local consulting company. Pairing engineering skills with communal needs is a large focus in his vocational search.

### 1.3 Project Background

The village of Carabuela is a small, rural community of approximately 500 homes in the inter-Andean central highlands approximately 100 kilometers north of the capital city, Quito. HCJB has worked with Carabuela a number of times in the past, including a recent project to add a clean drinking water system. Because of their previous relationships, the community of Carabuela has asked HCJB to partner with them in their potential wastewater sanitation project.

About a decade ago, the Ecuadorian government helped build a sewer network in Carabuela along with a septic tank and leach field for treatment. The collection system was adequately sized, however, the septic tank and leach field were severely undersized for the waste loads of the village. This caused the septic tank to overfill and the leach field to erode and cease functioning properly. They were both brought offline and currently the waste stream bypasses both and discharges directly into a nearby stream.
Even though Ecuador has seen tremendous growth in infrastructure, only about 10% of all wastewater nationally is treated\(^5\). This is due mainly to the lack of governmental experience in the field of wastewater treatment. While the Ecuadorian government is willing to sponsor projects and become involved in wastewater treatment, there is little technical experience in the field. The HCJB Quito office has some connections in the national government that previously expressed interest in funding a pilot project in wastewater, but was unable to provide a working model.

A properly functioning wastewater treatment facility in Carabuela could have rippling effects beyond the sanitation of the village itself and could help reform sanitation practices in the region and eventually the country. Therefore, Team Carabuela is devoted to providing the best model that will not only benefit the community, but will also hopefully benefit the country as a whole.

\(^4\) Photo courtesy of Adam De Young  
\(^5\) From Bruce Rydbeck, January 2013
Figure 2: Carabuela in "Volcano Alley"\textsuperscript{6}

\textsuperscript{6} Image from Google Earth
Figure 3: Overview Map of Carabuela

HCJB
1.4 Project Motivations

Team 7 chose this project for many reasons. Working with a client, especially a missions client, was a large motivation in finding potential projects. The members of team 7 also wanted to work outside of the immediate Calvin College vicinity and take on an international focus. Caring for others was a large focus of the project search along with meeting community needs. This project became an excellent experience for the team as well as a great challenge.

1.5 Carabuela in Context

It is important to be able to understand Carabuela in its geographical and political context. This allows the design to be culturally appropriate and better serve the needs of the people. Ecuador is located in South America and straddles the equator. It is divided into three main geographic regions: La Costa (the Coast), La Sierra (the Highlands), and La Amazonia (the East). La Costa is made up of the coastal western side of Ecuador, La Sierra is comprised of the Andes Mountain range through central Ecuador with a temperate climate, and La Amazonia is made up mostly of rain forest with a tropical climate.

![Physical Map of Ecuador](http://www.ezilon.com/maps/images/southamerica/Ecuador-map.gif)

Figure 4: Physical Map of Ecuador

---

Carabuela is located in La Sierra in the Andes Mountains and has a relatively mild and dry climate because of its altitude over 2500 meters above sea level (about 8200 feet). A simplified overview of Carabuela can be seen in Figure 3. In this figure, a rough contour map of the village is shown along with roads, houses, and the stream running through the village.

Politically, Ecuador gained its independence from Spain in 1820 and has been plagued with governmental instability throughout most of the 19th and 20th centuries. It is currently a republic and has held democratic elections since the late 1970s. Economically, Ecuador is growing and has stabilized considerably over the past few decades. Almost half of its exports are crude oil related and the rest is made up mostly of agricultural products, such as bananas and coca. Ecuador has also grown considerably in health care and infrastructure development over the past few decades; however, some rural areas, such as the area surrounding Carabuela, have seen much less growth than urbanized areas.

Figure 5: National Flag of Ecuador.

1.6 Climate

Carabuela has a temperate and dry climate year round due its high elevation and low latitude. Otavalo is a nearby city with accurate temperature and precipitation records (Figure 6 and Figure 7, respectively). Temperature stays fairly consistent and does not go below the freezing point. This is important when considering the type of outdoor treatment methods used and the depth of pipe burial. Since no records were available detailing evaporation and transpiration rates in the area, a comparison was made to similar climate zones in the United States. By finding a climate zone in the United States with comparable precipitation and temperatures ranges, an extrapolated evapotranspiration rate could be approximated (Appendix F). According to this, the evapotranspiration rate in Carabuela can be

---

estimated to be about 0 – 10 centimeters per year. This rate is low enough to be considered negligible for design purposes.

Figure 6: Average Yearly Temperatures (°C) in Otavalo, Ecuador¹¹

Figure 7: Average Yearly Rainfall (mm) in Otavalo, Ecuador¹²

1.7 Village Demographics

Carabuela is made up of 508 homes. These are spread around a central hill that rises in the middle of the village. Most residents are mainly farmers and craftsmen that own a small plot of land next to their house and raise some crops and animals. On the weekends, many artisans travel to the nearby city of Otavalo to sell craft goods, scarves, hats, blankets, gloves, and belts. Based on information provided by HCJB, an annual growth rate of 2% is assumed for design purposes.

Figure 8: Overview of Carabuela. Looking East.
2. Existing Conditions

To learn more about Carabuela and the current conditions in the community, Team 7 traveled to Ecuador at the end of January 2013 to refine their design approach. Team 7 met with their contact from HCJB, Bruce Rydbeck, and stayed in the village of Carabuela for about five days. During this time, the team mapped out the existing collection system using a Garmin GPSMAP 62sc Handheld Navigator, inspected the septic tank and leach field, scouted out potential treatment plant locations, conducted a soil percolation test, and talked to leading members on the Carabuela Water Board. This allowed the team to better understand the conditions in the village and be better able to fit a design to the village’s needs.

2.1 The Collection System

Very little information was known about the collection system installed in Carabuela before the informational trip. After visiting, the collection system was mapped with the GPS device and its condition was evaluated. The collection system is more extensive than was originally thought by the team and is still in good condition. The sewer system connects about 300 homes to the septic tank with 17 cm diameter concrete pipes. The network is adequately designed for the wastewater flow of the community and does not need to be redesigned. This allows the team to focus solely on the design of the treatment facility.
2.2 The Treatment Plant

The existing treatment plant services about 300 homes in Carabuela. These were once treated in a septic tank and leach field configuration. The septic tank and leach field are currently bypassed and the untreated wastewater is discharged into the stream by the school (Figure 9).

Figure 9: Current Configuration (Offline)\textsuperscript{13}

The septic tank is sized to treat around 73 m\textsuperscript{3} per day of wastewater (Appendix E) and is still in good condition, despite the fact that it is unused. The concrete platform in Figure 10 is the septic tank and the grassy area is what remains of the leach field. These are both bypassed by the white PVC pipe and discharged into the stream on the other side of the leach field.

\textsuperscript{13} Provided by HCJB
Figure 10: Existing Septic Tank and Leach Field

Figure 11: Eroded Leach Field

14 Photo courtesy of Adam De Young
15 Photo courtesy of Ian Compton
3. Design Parameters

The design must satisfy these guidelines discussed below for it to be useable and practical for Carabuela and HCJB. The design must be a passive process with a minimal amount of mechanization and maintenance. The potential use of the treated effluent for irrigation purposes is highly preferred. This design must also be culturally appropriate for the community. It must fit within the parameters of the culture of Carabuela and be designed with their needs in mind.

3.1 Flows and Loads

In order to correctly design a suitable treatment facility for Carabuela, quantifying the concentration and amount of wastewater constituents is crucial. Due to the lack of specific information regarding waste loads in Ecuador, an estimate was made (Table 1). The team assumed a strong waste concentration. This conforms to research relating to wastewater in other developing countries as well as previous design teams that have worked in Ecuador with HCJB.

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Strong</th>
<th>Medium</th>
<th>Weak</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total solids</td>
<td>1200</td>
<td>700</td>
<td>350</td>
</tr>
<tr>
<td>Dissolved solids (TDS)</td>
<td>850</td>
<td>500</td>
<td>250</td>
</tr>
<tr>
<td>Suspended solids</td>
<td>350</td>
<td>200</td>
<td>100</td>
</tr>
<tr>
<td>Nitrogen (as N)</td>
<td>85</td>
<td>40</td>
<td>20</td>
</tr>
<tr>
<td>Phosphorus (as P)</td>
<td>20</td>
<td>10</td>
<td>6</td>
</tr>
<tr>
<td>Chloride</td>
<td>100</td>
<td>50</td>
<td>30</td>
</tr>
<tr>
<td>Alkalinity (as CaCO3)</td>
<td>200</td>
<td>100</td>
<td>50</td>
</tr>
<tr>
<td>Grease</td>
<td>150</td>
<td>100</td>
<td>50</td>
</tr>
<tr>
<td>BOD₅¹⁸</td>
<td>300</td>
<td>200</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 1: Major Constituents of Typical Domestic Wastewater

---

¹⁶ UN Department of Technical Cooperation for Development (1985)
¹⁷ The amounts of TDS and chloride should be increased by the concentrations of these constituents in the carriage water.
¹⁸ BOD₅ is the biochemical oxygen demand at 20°C over 5 days and is a measure of the biodegradable organic matter in the wastewater.
The constituent used most for design was the BOD$_5$ concentration in the influent waste stream. This allows for specific sizing of the various treatment options.

The waste stream flow rate was measured by timing how long it took for 12 liter and 20 liter buckets to fill from the effluent stream (Figure 12). Repeated trials using each bucket were performed. This gave an average flow rate of 2.6 L/s (about 60,000 gpd). To see the bucket test trials, see the Appendix page I. This flow rate was measured around 10 am, a time close to the standard peak flow of wastewater. Based on the measured flow, Table 2 shows current flow rates along with the 5 year design flow and the 20 year design flow.

![Figure 12: Effluent Bucket Tests. Greywater Clearly Visible.](image)

Table 2 : Current and Projected Sanitary Flows at 2% Growth

<table>
<thead>
<tr>
<th>Year</th>
<th>2013</th>
<th>2017 (Projected)</th>
<th>2032 (Projected)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Sanitary Flow (L/s)</td>
<td>2.60</td>
<td>2.83</td>
<td>3.80</td>
</tr>
</tbody>
</table>

3.2 Footprint Constraints

The village location itself is also a constraint that must be considered. Carabuela is in a mountainous and somewhat arid region. Very little land in Carabuela is un-owned and un-used. Because of this, any proposed site for design must be purchased or leased from a local farmer. This limits the amount of land available for the implementation of the design solution as well as the amount of water available to the village for irrigation. This constraint limits the footprint of the design to be as small as possible.
3.3 Effluent Standards

The effluent quality standards are quite vague. The standards for the design were given by HCJB to be comparable to similar projects. In order to find a suitable target, case studies and national and international standards were researched.

Due to comparatively low wastewater effluent standards in Ecuador, finding suitable effluent quality standards for the region is difficult. However, the team did find Ecuadorian standards as listed in Table 3. These standards are considerably weak and are even comparable to some weak waste stream influents.

<table>
<thead>
<tr>
<th>Contaminant</th>
<th>Standard Concentration</th>
</tr>
</thead>
<tbody>
<tr>
<td>BOD₅</td>
<td>100 mg/L</td>
</tr>
<tr>
<td>Total Suspended Solids</td>
<td>100 mg/L</td>
</tr>
<tr>
<td>Nitrogen</td>
<td>10 mg/L</td>
</tr>
<tr>
<td>Phosphorous</td>
<td>10 mg/L</td>
</tr>
<tr>
<td>pH</td>
<td>5 – 9</td>
</tr>
<tr>
<td>Fecal Coliform Bacteria</td>
<td>Removal &gt;99.9% or 0 eggs/L for use in agriculture</td>
</tr>
</tbody>
</table>

After researching effluent standards used in the United States, a set of standards used by the Michigan Department of Environmental Quality (MDEQ) was also considered (Table 4). These are some standards used for discharge into ground water which are used extensively in the state of Michigan.

<table>
<thead>
<tr>
<th>Contaminant</th>
<th>Concentration</th>
</tr>
</thead>
<tbody>
<tr>
<td>CBOD</td>
<td>25 mg/L monthly average; 40 as 7-day average</td>
</tr>
<tr>
<td>TSS</td>
<td>30 mg/L monthly average; 45 as 7-day average</td>
</tr>
<tr>
<td>Total Phosphorous</td>
<td>5 mg/L</td>
</tr>
<tr>
<td>Total Inorganic Nitrogen</td>
<td>10 mg/L</td>
</tr>
<tr>
<td>pH</td>
<td>6.5 - 9</td>
</tr>
<tr>
<td>Sodium</td>
<td>150 mg/L</td>
</tr>
<tr>
<td>Chloride</td>
<td>250 mg/L</td>
</tr>
</tbody>
</table>

---

19 From Ecuadorian Congress: NORMA DE CALIDAD AMBIENTAL Y DE DESCARGA DE EFLUENTES : RECURSO AGUA LIBRO VI ANEXO 1

20 From Derrick Simmons, Senior Environmental Quality Analyst, MDEQDEQ
After considering Ecuadorian standards and Michigan regulatory standards, a compromised system was adopted by the team. This comes in conjunction with the consideration of the potential treatment footprint. The footprint of a constructed wetland was plotted against the BOD\textsubscript{5} effluent standard to provide the sizing curve shown in Figure 13 below. This led to an optimal target of 30 mg/L BOD in the effluent stream. This number is more stringent than the Ecuadorian national standards but requires only about half of the footprint of a conventional standard of 5 mg/L often used in the United States.

![Wetland Footprint vs. BOD in Effluent](image)

This “compromised” standard will have low negative watershed impacts. Currently, the stream already carries a high waste load, even before it passes through Carabuela.; Upstream communities also discharge waste directly to the surface water, causing higher waste concentrations farther downstream. Any restrictions in this area then will be an improvement. The discharged effluent will also be routed through the ground before entering the stream, thus further treating the waste. Downstream river usage is limited, due to the current high levels of waste contamination. Some tributaries are used to water livestock and wash clothes, but surface water is typically avoided for human consumption and bathing.

### 3.4 Costs

It is imperative that design costs remain as low as possible. This includes low upfront costs and low yearly maintenance and operation costs. This is a major constraint for the team, and for HCJB as well as the village itself. The team was given a maximum target cost of $50,000 (US) for the treatment facility and collection system (excluding labor costs). This dollar amount came from the HCJB contact in Ecuador.
4 Preliminary Design

4.1 Collection System

Initially, the scope of the project included the evaluation and possible redesign of Carabuela’s collection system. After closer inspection, however, the existing collection system was shown to be adequately designed for the needs and uses of the community. The existing system is still in good condition and sized appropriately for the flows seen by Carabuela. In some areas of the community, infiltration from the groundwater into the collection system is a problem and increases the flows seen in the pipes. This is currently beyond the scope of the project and the focus will remain on the wastewater treatment process itself. The current design takes into account the existing flow rates, but future improvements in the collection system will help increase the life of the proposed treatment system and increase its efficiency.

4.2 Treatment Process

After researching a variety of passive waste treatment systems, a list of design options was compiled and evaluated. These were divided into preliminary treatments and secondary treatments. For preliminary treatment, the team researched bar screens, grit removal, anaerobic ponds, and septic tanks. For secondary treatment, the team researched facultative ponds, constructed wetlands, biofiltration, and ground infiltration. After evaluating these options, it was decided to look further into treatment ponds in series or septic tanks in series with constructed wetlands. For more information on the design options and preliminary decision process, see page VIII of the Appendix.

4.3 Decision Matrix

With two treatment options it was necessary to implement a design comparison in the form of a decision matrix. The matrix seen in Table 21 is divided into preliminary treatment options and primary treatment options. Seven characteristics are used to evaluate each treatment process.

1) Passive
   - This was a constraint from HCJB and is critical for the design. A passive design considerably lowers maintenance and equipment costs. These processes may still require periodic cleaning and adjusting.

2) Maintenance
   - The facility will be maintained by community members who may not be experienced with wastewater treatment processes. The only maintenance will be in the form of manual labor. Therefore, the decision of the numerical value was based on frequency and amount of labor needed.
3) **Footprint**
   - The treatment facility needs to fit in a particular location in Carabuela. The current size of the land available is unknown, yet the matrix gives preference to a smaller footprint.

4) **Cost**
   - The main cost consideration of the treatment facility will be the materials needed. Manual labor will be provided for construction. In this case, the amount of fill material and liners needed drives the cost characteristic.

5) **Quality**
   - The wastewater needs to be treated effectively. The options presented all provide sufficient treatment. The value of quality is based on the speed and capacity of treated wastewater.

6) **Aesthetics**
   - Since the treatment process will be in close proximity to the community, it is important that it be aesthetically pleasing. The most important factor here being offensive odors associated with treatment.

7) **Irrigation**
   - The amount and quality of the effluent has a large impact on its potential use for irrigation. Some types of effluents are better suited to crop fertilization than others due to their different loads of nutrients, organic matter, and pathogens.

The characteristics were weighted to show a hierarchy of importance. Each of the treatment options is given a rating in a characteristic from worst, 1, to greatest, 10. The two options evaluated cost of the design is a major driver and the most measureable driver. The primary costs are briefly described below with a summary in Table 5 with the decision matrix in

<table>
<thead>
<tr>
<th></th>
<th>Holding Volume</th>
<th>HRT</th>
<th>Area</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Option 1</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Septic Tank</td>
<td>180</td>
<td>1.5</td>
<td>198.0</td>
<td>$19,339</td>
</tr>
<tr>
<td>Wetland</td>
<td>640</td>
<td>2.7</td>
<td>3,499.0</td>
<td>$41,830</td>
</tr>
<tr>
<td>Sub Total</td>
<td>820</td>
<td>4.2</td>
<td>3,697.0</td>
<td>$61,169</td>
</tr>
<tr>
<td>Land cost</td>
<td>$5000/acre</td>
<td></td>
<td></td>
<td>$6,852</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td>$68,021</td>
</tr>
<tr>
<td><strong>Option 2</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Anaerobic</td>
<td>330</td>
<td>1.364</td>
<td>110.9</td>
<td>$288</td>
</tr>
<tr>
<td>Facultative</td>
<td>21790</td>
<td>89.28</td>
<td>14,520.0</td>
<td>$37,760</td>
</tr>
<tr>
<td>Sub Total</td>
<td>22120</td>
<td>90.644</td>
<td>14,630.9</td>
<td>$38,048</td>
</tr>
<tr>
<td>Land cost</td>
<td>$5000/acre</td>
<td></td>
<td></td>
<td>$27,116</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td>$65,164</td>
</tr>
</tbody>
</table>

Table 6.
### Table 5: Secondary Design Cost Summary

<table>
<thead>
<tr>
<th></th>
<th>Holding Volume</th>
<th>HRT</th>
<th>Area</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Option 1</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Septic Tank</td>
<td>180</td>
<td>1.5</td>
<td>198.0</td>
<td>$19,339</td>
</tr>
<tr>
<td>Wetland</td>
<td>640</td>
<td>2.7</td>
<td>3,499.0</td>
<td>$41,830</td>
</tr>
<tr>
<td>Sub Total</td>
<td>820</td>
<td>4.2</td>
<td>3,697.0</td>
<td>$61,169</td>
</tr>
<tr>
<td>Land cost</td>
<td>$5000/acre</td>
<td></td>
<td></td>
<td>$6,852</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td>$68,021</td>
</tr>
<tr>
<td><strong>Option 2</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Anaerobic</td>
<td>330</td>
<td>1.364</td>
<td>110.9</td>
<td>$288</td>
</tr>
<tr>
<td>Facultative</td>
<td>21790</td>
<td>89.28</td>
<td>14,520.0</td>
<td>$37,760</td>
</tr>
<tr>
<td>Sub Total</td>
<td>22120</td>
<td>90.644</td>
<td>14,630.9</td>
<td>$38,048</td>
</tr>
<tr>
<td>Land cost</td>
<td>$5000/acre</td>
<td></td>
<td></td>
<td>$27,116</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td>$65,164</td>
</tr>
</tbody>
</table>

### Table 6: Secondary Decision Matrix

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Passive</th>
<th>Maintenance</th>
<th>Footprint</th>
<th>Cost</th>
<th>Effluent Quality</th>
<th>Aesthetics</th>
<th>Irrigation</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Primary</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Anaerobic Pond</td>
<td>10</td>
<td>9</td>
<td>7</td>
<td>8</td>
<td>5</td>
<td>3</td>
<td>0</td>
<td>320</td>
</tr>
<tr>
<td>Septic Tanks</td>
<td>9</td>
<td>7</td>
<td>9</td>
<td>6</td>
<td>7</td>
<td>7</td>
<td>0</td>
<td>322</td>
</tr>
<tr>
<td><strong>Secondary</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Constructed Wetlands</td>
<td>10</td>
<td>7</td>
<td>8</td>
<td>7</td>
<td>8</td>
<td>8</td>
<td>6</td>
<td>383</td>
</tr>
<tr>
<td>Facultative Pond</td>
<td>10</td>
<td>9</td>
<td>4</td>
<td>6</td>
<td>7</td>
<td>7</td>
<td>8</td>
<td>362</td>
</tr>
<tr>
<td><strong>Summary</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Option 1</strong></td>
<td>19</td>
<td>14</td>
<td>17</td>
<td>13</td>
<td>15</td>
<td>15</td>
<td>6</td>
<td>705</td>
</tr>
<tr>
<td><strong>Option 2</strong></td>
<td>20</td>
<td>18</td>
<td>11</td>
<td>14</td>
<td>12</td>
<td>10</td>
<td>8</td>
<td>682</td>
</tr>
</tbody>
</table>
5 Final Design

The final design decided upon by the team is a system of septic tanks in series with constructed wetland beds. The existing septic tank, leach field, and school will be offline from the proposed waste treatment system and made into a separate system. The existing septic tank and leach field can then be used to treat the wastewater from the school and the school’s waste flow rate can be removed from the proposed system’s calculations. This allows the existing septic tank to be used to cut down on costs and space for the proposed new system. While the school’s waste stream is not very substantial (about 120 cubic meters per month), it will still be helpful to remove a contribution from the total waste flow. Reusing the existing septic tank and leach field also allows the proposed system to be smaller and less costly. The existing tank and leach field are sized appropriately for the school’s waste, even after some erosion (Appendix E).

The team originally designed for a 20-year design period. However, this gave very high upfront costs with little additional benefits over a shorter design period. The team then compromised by designing the system to accommodate a 5-year design period with an additional phasing plan to augment the system with future growth. The proposed new system for the community will consist of two parallel septic tanks in series with six constructed wetland cells. Each wetland cell will be piped in parallel with each other as seen in Figure 14.

![Proposed Waste Treatment Process Flow Diagram](image)
Figure 15: Plan View Layout

Figure 16: Treatment Location in Relation to Carabuela
The location chosen for the site of the septic tanks and CWs is pastureland at the north end of the village (Figure 16). This area is suitable to implement the design because of its fairly flat and even terrain. No trees or extensive flora and fauna grow there, reducing potential costs to clear area. However, the land is currently being used as pastureland for livestock. Acquiring this land would require a prearranged trade or lease agreement with the farmer that owns the land.

5.1 Bar Screen

The bar screen structure is located at the beginning of the treatment system upstream of the septic tanks. The bar screen removes large objects known as “rags and floatables” from the waste stream. According to the 10 States Standards for wastewater facilities, a minimum velocity of 0.4 m/s is needed to prevent settling. Normally, the bar screen channel is sized based on the design flow and target velocity. Since the design flow of around 240 m$^3$ per day is small compared to most municipal wastewater systems in the United States, the channel ends up being unreasonably small (Appendix G). A typical bar screen design is not acceptable in this case. The team’s proposed structure has an influent pipe that discharges directly onto the bar screen. The bar screen is placed at an acute angle of 20° to alleviate settling problems that may occur at a greater angle. A picture of the structure is seen below in Figure 17. The bars are spaced 40 mm apart. The bar screen is designed to be manually cleaned. Since there is only one channel, the screen must be raked while the water is still flowing. This should not be a problem. The effluent pipe from the bar screen channel is connected to the septic tanks.

---

21 Drawn by Ian Compton
5.2 Septic Tanks

The initial two septic tanks are sized for a 5-year design horizon with an additional third septic tank for a 20-year horizon. This phased plan allows for the village to keep the original design cost lower. The tanks are designed using a desired hydraulic retention time of 1.5 days. The tanks have two compartments with the first compartment being 2/3 of the total length for optimal solids removal. The tanks have a length to width ratio of around 2:1. The 2 m height of the tank is small compared to the length and width. This allows for a more consistent water level in the tank. The effluent pipes of the septic tanks are connected with a diversion structure to allow for maintenance on one tank while still utilizing the other for treatment. For detailed specifications of the septic tanks see Appendix E, with Figure 18 providing general information on design.

Figure 18: Septic Tank Plan and Profile Views

---

22 Drawn by Ian Compton
Soil percolation tests were performed on the soil near the existing septic tank to determine the infiltration rate (Figure 19). An infiltration rate of 3.1 minutes per cm (8 minutes per inch) was determined (Appendix A). Using this infiltration rate and the effluent rate of 4 m$^3$ per day from the school, the needed leach field area was calculated$^{23}$. There is about 280 square meters of usable land by the current septic tank. The effluent from the school requires a leaching area of 182 square meters, so the leach area can be used for treatment without threat of being washed out again. This allows for the school to be offline from the rest of the collection system.

![Figure 19: Percolation Test near Septic Tank$^{24}$](image)

### 5.3 Distribution Box

The effluent from the two septic tanks will then flow through a distribution box to be shared equally across the six wetland beds. This box collects the influent from two pipes entering and distributes the waste stream across three effluent pipes (Figure 20). The effluent pipes have the same invert elevation, all being slightly lower than the influent invert elevation to inhibit backflow. A baffle is inserted to prevent short-circuiting of the flow.


$^{24}$ Photo courtesy of Ian Compton
5.4 Constructed Wetlands

Typical constructed wetlands have four main components, including a waterproof basin, specific filter media, wetland-friendly plants, and inlet and outlet structures.

5.4.1 Waterproof Basin
This is used to prevent waste contaminants from leaching into the ground and contaminating groundwater. In this case, however, the wetland will be designed without a waterproof liner. This is because the wetland will sit high enough above the water table to prevent dangerous contamination. The groundwater in the village is also not used as a drinking source and drains into the highly polluted stream.

5.4.2 Filter Media
This is the fill media of the bed. It is recommended to have larger media by the inlet and outlet zones to prevent pipe clogging and maintain an even distribution. This is usually a form of gravel near the inlet/outlet and coarse sand as the main bed media. The sand must be porous enough to allow subsurface flow, and yet must be able to hold the roots of plants. Due to the geologic conditions in South America, volcanic rock is widely available in Ecuador and can be used effectively as a media. A mixture of volcanic sand and crushed rock is used in the final design.

25 Drawn by Nathan Williams
26 From Constructed Wetlands: A Promising Wastewater Treatment System for Small Localities, pg. 10
5.4.3 Wetland-Friendly Plants
The vegetation in a CW must be a native, water-loving plant species. Typically, various forms of reeds are used. These plants must have deep and fibrous roots, considerable biomass for water translocation, and oxygen transportation into root zone\(^{27}\). Many *Phragmites* species reeds fit the qualifications and are found across the globe. Two commonly used wetland reed varieties in South America can be seen in Figure 21 and Figure 22.

![Figure 21: Phragmites australis\(^{28}\)](http://www.leelanaucd.org/wp-content/uploads/2010/11/Phragmites_australis_2-dense-thicket.jpg)

![Figure 22: Pennisetum purpureum (Elephant Grass)\(^{29}\)](http://www.tropicalforages.info/key/Forages/Media/Html/images/Pennisetum_purpureum/Pennisetum_purpureum_7a.jpg)

5.4.4 Inlet and outlet structures
Inlet and outlet structures are important to hydraulically control the flows through the wetland. Typical inlet structures are distribution pipes on the top of the wetland that connect to a perforated pipe, as

\(^{27}\) From Constructed Wetlands – UN pg. 45


\(^{29}\) [http://www.tropicalforages.info/key/Forages/Media/Html/images/Pennisetum_purpureum/Pennisetum_purpureum_7a.jpg](http://www.tropicalforages.info/key/Forages/Media/Html/images/Pennisetum_purpureum/Pennisetum_purpureum_7a.jpg)
shown in Figure 23. This allows an even distribution of wastewater across the entire length of the bed. The gravel zone under the influent pipe also helps distribute the water across the subsurface of the bed. The outlet pipe lies under the media at the opposite end of the bed. This also includes a perforated pipe in gravel that drains the water. The pipe is connected to an adjustable standpipe to control the water level in the wetland (Figure 24).

Figure 23: Typical Inlet Structure

---

30 Drawn by Nathan Williams
The CW's used in the design are cells of 20 m by 30 m long. This is large enough to prevent “short circuiting” inside the wetland and small enough to bring single cells offline for maintenance and repair. Each cell will have its own inlet and outlet structure and can be brought offline if need be. Native reed species are used as the wetland vegetation and native volcanic sand and rocks are used as the wetland media.

The wetlands designed here are to be horizontal flow wetlands. Horizontal flow CWs rely on subsurface flow laterally across the wetland bed. This maximizes media filtration and particle removal. Horizontal flow wetlands remove much lower percentages of phosphorous and nitrogen in the wastewater than vertical flow wetlands, however, this can be beneficial if used in irrigation. Each wetland will be 40 cm deep. This is a standard depth of horizontal flow wetlands and is deep enough to maintain root growth and stability and shallow enough to save on material costs.

Figure 24: Typical Outlet Structure

Drawn by Nathan Williams

From Constructed Wetlands: A Promising Wastewater Treatment System for Small Localities
This treatment process has been designed for a specific plot of land in Carabuela. This land is currently pastureland owned by a local farmer. Purchase or leasing of the land will have to be discussed by the village if the design is going to be implemented by HCJB and the Carabuela Water Board. The land is already cleared of brush and is relatively level, which will save grubbing and excavation costs. However, the land is still at a slope much steeper than the 1% required bed slope. This means that some terracing will have to be built into the design (Figure 27). The wetlands will cover a 5 m drop over a horizontal distance of 70 m.
Figure 26: Constructed Wetland Terrace Overview

Figure 27: CW Terrace Detail

33 Drawn by Nathan Williams
34 Drawn by Nathan Williams
5.5 Piping

The 5-year design calls for six wetland cells and two septic tanks. For redundancy, the system is divided into two parallel systems. Each septic tank discharges to three wetland beds, also in parallel. This allows for individual beds and septic tanks to be brought offline if need be. Future expansion is also taken into consideration with space for an additional parallel line of one septic tank and two more wetland cells as needed.

To size the pipes needed for all the piping in and around the treatment plant, the program EPA SWMM was used with AutoCAD drawings and existing condition data. EPA SWMM (Storm Water management Model) is a computer program created by the US Environmental Protection Agency to model hydraulic conditions. Although SWMM uses only English Standard Units, the results were converted to metric at the end of the simulation. The current collection system uses 20cm concrete pipes everywhere that had been provided by the government for the system. The pipes are adequately sized, but for further additions, PVC pipes are recommended because of their lower costs.

The first section of piping needed is the section from the current outflow from the collection system to the septic tanks located at the start of the treatment process. Because all the waste from the entire system is flowing through this section of pipe, a section of 20cm PVC pipe is recommended. This will allow all the flows and loads to be transported adequately and will add on to the existing 20cm pipes already in place. Based on the scaled AutoCAD drawing, and total pipe length of 136m is needed.

With the piping determined for transporting the waste to the septic tanks, only the piping for the wetland inflows and outflows needed to be determined. The lengths of the pipes could be determined by the AutoCAD drawing of the treatment plant, but the pipes diameters needed further analysis. Because the effluent piping is more complicated and encompasses more flows than the inlet piping, a SWMM model of only the effluent would be sufficient for determining pipe diameters for both sets of pipes. Figure 28 shows the effluent pipe layout in SWMM.
With the pipe layout and conduit lengths to scale, the inflows were inserted into the nodes that connect directly to the wetland beds. The 5-year flow of 240m$^3$/day was divided between the first six beds and then the additional 20-year flow of 83m$^3$/day was divided between the additional two beds. The elevations for the nodes were based on the final design of the wetlands and the current elevation of the field. An arbitrary elevation of 100ft was chosen for the uppermost node and then the elevation changes can be seen in Figure 28.

The velocities in the pipes were required to be between 0.61m/s and 3.05m/s. Because the pipes are on very steep slopes, outlet offsets were used to prevent velocities from getting too high. The final pipe slopes can also be seen in Figure 28. With the flows and pipe slopes in place, the simulation was run at various pipe diameters. Smaller pipe diameters were used first and then upsized until the system could handle all the flows adequately so that costs could be minimized. The final pipe diameters (shows as max depth in feet) can be seen below in Figure 29. Additionally, the pipe diameters shown are summed up in Table 7 below.
The final treatment design with pipe sizes and slopes were analyzed in SWMM to confirm that all flow requirements were met. The velocities were confirmed to be between 0.61m/s and 3.05m/s as shown below in Figure 30. The worst case capacity of the pipes is shown in Figure 31, which shows the ratio of flow depth over the total pipe diameter. Because a capacity of 1 would indicate the pipes being surcharged, Team Carabuela decided to not surpass a capacity of 0.5 as a factor of safety in case of possible obstructions of blockages in the pipes. As seen in Figure 30, the capacity stays below 0.5 in all pipes.
To visualize the pipe diameters, slopes, capacity, outlet offsets, and elevations, the longest path between node 1 (the uppermost node) and the node 19 (the outfall) is shown below in Figure 32 as a water elevation profile.
Similar pipe sizing was evaluated for the inlet pipes but was determined to be too small due to the increased amount of solids. Therefore, the inlet pipes were designed to be 20cm diameter pipes based on the higher risk of obstruction or clogging. The total pipe schedule is shown below in Table 8: Total Piping Schedule.

### Table 8: Total Piping Schedule

<table>
<thead>
<tr>
<th>Design</th>
<th>Diameters</th>
<th>Piping Lengths (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Current Discharge to Bar Screens</td>
</tr>
<tr>
<td>5 Year</td>
<td>12cm</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>15cm</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>20cm</td>
<td>104.7</td>
</tr>
<tr>
<td>20 Year</td>
<td>12cm</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>15cm</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>20cm</td>
<td>-</td>
</tr>
</tbody>
</table>
5.6 Irrigation Effluent

To harness the benefits of wastewater treatment in Carabuela, the team designed for effluent reuse in irrigation. This can be very beneficial for farmers, but also has some risks associated with it. The treated effluent will have much lower BOD and solids concentrations, but will still have relatively high amounts of Phosphorous and Nitrogen. These nutrients are beneficial to plant growth and will cut down on the need for fertilizers. This saves money for the farmer and helps preserve downstream sanitation. On the reuse of wastewater for irrigation, the World Health Organization states that, “Wastewater, excreta and greywater are often reliable year-round sources of water, and they contain the nutrients necessary for plant and fish growth. Irrigation with wastewater can, in most situations, supply all the nutrients required for crop growth” (Vol.1 pg.8). Wastewater is widely used for irrigation in developing countries because it is a consistent source of water and treatment infrastructure is often lacking. The reuse of wastewater for irrigation helps increase crop yields as well as protects the sanitation of areas downstream.

However, wastewater, even if treated, still contains some concentration of pathogenic organisms. The two main ways that humans come in contact with dangerous pathogens from irrigation is through oral consumption and skin contact. Treatment highly decreases pathogens, but protective measures still need to be taken to protect the health of workers and consumers of produce irrigated with wastewater.
Some ways to reduce health risks are through treatment, crop restriction, application techniques, exposure control, and produce handling before consumption. Some crops carry more health risk than others because of the environment they are grown in. For example, salad crops often have higher concentrations of pathogens because they are grown at or below ground level and have rough surfaces that can shelter microorganisms. In order to regulate this, some level of crop restriction is often advised. This means that it is often advised not to irrigate crops intended for direct consumption. High growing crops and crops that are always cooked before consumption are allowed under this restriction. This decreases the risk of consuming pathogens.

Different application techniques and exposure control can drastically decrease the risks associated with skin contact with pathogens. Flood and furrow irrigation leaves the farmer most at risk for contact while

---

**Table 9: Some Common Pathogens Carried in Wastewater**

<table>
<thead>
<tr>
<th>Agent</th>
<th>Disease</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bacteria</strong></td>
<td></td>
</tr>
<tr>
<td><em>Escherichia coli</em></td>
<td>Gastroenteritis</td>
</tr>
<tr>
<td><em>Salmonella</em></td>
<td>Gastroenteritis, Salmonellosis, diarrhea, etc.</td>
</tr>
<tr>
<td><em>Salmonella typhi</em></td>
<td>Typhoid fever</td>
</tr>
<tr>
<td><em>Vibrio cholerae</em></td>
<td>Cholera</td>
</tr>
<tr>
<td><strong>Helminths</strong></td>
<td></td>
</tr>
<tr>
<td><em>Ancylostoma duodenale</em></td>
<td>Hookworm</td>
</tr>
<tr>
<td><em>Necator americanus</em></td>
<td>Hookworm</td>
</tr>
<tr>
<td><strong>Protozoa</strong></td>
<td></td>
</tr>
<tr>
<td><em>Cryptosporidium parvum</em></td>
<td>Cryptosporidiosis, diarrhea, fever</td>
</tr>
<tr>
<td><em>Cyclospora cayetanensis</em></td>
<td>Persistent diarrhea</td>
</tr>
<tr>
<td><em>Giardia intestinalis</em></td>
<td>Giardiasis</td>
</tr>
<tr>
<td><strong>Virus</strong></td>
<td></td>
</tr>
<tr>
<td><em>Adenovirus (many types)</em></td>
<td>Respiratory Disease, Gastroenteritis</td>
</tr>
<tr>
<td><em>Enteroviruses (many types)</em></td>
<td>Gastroenteritis</td>
</tr>
<tr>
<td><em>Norovirus</em></td>
<td>Gastroenteritis</td>
</tr>
<tr>
<td><em>Poliovirus</em></td>
<td>Paralysis, aseptic meningitis</td>
</tr>
</tbody>
</table>

Some crops carry more health risk than others because of the environment they are grown in. For example, salad crops often have higher concentrations of pathogens because they are grown at or below ground level and have rough surfaces that can shelter microorganisms. In order to regulate this, some level of crop restriction is often advised. This means that it is often advised not to irrigate crops intended for direct consumption. High growing crops and crops that are always cooked before consumption are allowed under this restriction. This decreases the risk of consuming pathogens.

---

35 From WHO Guidelines Vol. 1
36 From WHO Guidelines Vol. 2
sprinkler techniques and subsurface flow can be much safer\textsuperscript{37}. Proper covering should also be worn by farmers and field workers. This includes shoes and gloves and avoiding direct contact with the wastewater stream and irrigated soil. Subsurface irrigation can also help treat the wastewater and remove pathogens before the water reaches the crop roots.

Proper produce handling is a very important step in reducing health risks associated with consumption. Rinsing or washing the produce before eating can have a 1 log removal of microorganisms and peeling can have at least a 2 log removal. Cooking is recommended and attains essentially complete pathogen removal at about 5-6 log units\textsuperscript{38}. Ceasing irrigation a period of time before harvest can also reduce pathogens on produce. This pathogen die-off period is typically about two weeks and can have a large effect on sanitation (Table 10). The die-off period typically depends on temperature and sunlight available, with higher temperatures and more sunlight having a higher kill rate.

### Table 10: Survival of various Organisms in Selected Environmental Media at 20-30 °C\textsuperscript{39}

<table>
<thead>
<tr>
<th>Organism</th>
<th>Fresh water and sewage</th>
<th>Crops</th>
<th>Soil</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Viruses</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Enteroviruses\textsuperscript{a}</td>
<td>&lt;120, usually &lt;50</td>
<td>&lt;60, usually &lt;15</td>
<td>&lt;100, usually &lt;20</td>
</tr>
<tr>
<td><strong>Bacteria</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thermotolerant coliforms</td>
<td>&lt;60, usually &lt;30</td>
<td>&lt;30, usually &lt;15</td>
<td>&lt;70, usually &lt;20</td>
</tr>
<tr>
<td>Salmonella spp.</td>
<td>&lt;60, usually &lt;30</td>
<td>&lt;30, usually &lt;15</td>
<td>&lt;70, usually &lt;20</td>
</tr>
<tr>
<td>Shigella spp.</td>
<td>&lt;30, usually &lt;10</td>
<td>&lt;10, usually &lt;5</td>
<td>ND</td>
</tr>
<tr>
<td>V. cholerae</td>
<td>ND</td>
<td>&lt;5, usually &lt;2</td>
<td>&lt;20, usually &lt;10</td>
</tr>
<tr>
<td><strong>Protozoa</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>E. histolytica cysts</td>
<td>&lt;30, usually &lt;15</td>
<td>&lt;10, usually &lt;2</td>
<td>&lt;20, usually &lt;10</td>
</tr>
<tr>
<td>Cryptosporidium oocysts</td>
<td>&lt;180, usually &lt;70</td>
<td>&lt;3, usually &lt;2</td>
<td>&lt;150, usually &lt;75</td>
</tr>
<tr>
<td><strong>Helminths</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ascariis eggs</td>
<td>Years</td>
<td>&lt;60, usually &lt;30</td>
<td>Years</td>
</tr>
<tr>
<td>Tapeworm eggs</td>
<td>Many months</td>
<td>&lt;60, usually &lt;30</td>
<td>Many months</td>
</tr>
</tbody>
</table>

ND, no data
Sources: Feachem et al. (1983); Strauss (1985); Roberson, Campbell & Smith (1992); Jenkins et al. (2002); Warnes & Keevil (2003).

\textsuperscript{a} Poliovirus, echovirus and coxsackievirus.

Various combinations of treatment and irrigation methods can be used to provide a product safe for consumption. Eight of these combinations from the World Health Organization can be seen in Figure 34.

\textsuperscript{37} From WHO Guidelines Vol. 2

\textsuperscript{38} From WHO Guidelines Vol. 2, pg. 78

\textsuperscript{39} From WHO Guidelines, Vol 2.
and its accompanying Table 11. These resources advocate at least a 6-log removal of pathogens before oral consumption. This is done through a combination of pathogen reduction with treatment processes and safe irrigation techniques.

The proposed design for Carabuela with septic tanks and CWs will yield a minimum of 2-log removal of pathogens. This dictates that if the effluent is to be used for irrigation, it must be combined with safe irrigation methods and proper preparation before consumption. The team recommends using the effluent for the town’s livestock crop such as alfalfa and avoiding the direct application to salad crops and low lying crops such as their strawberries. The effluent can be used to irrigate potatoes since these are commonly washed, peeled, and cooked in soups or other hot dishes. The team also recommends that irrigation using the CW effluent be stopped two weeks before harvest if conditions allow, further reducing the risk of exposure to pathogens.

Figure 34: Wastewater Treatment Options and Associated Log Removals

---

40 From WHO Guidelines Vol. 2
Table 11: Verification Monitoring\(^a\) (\textit{E. Coli} numbers per 100ml of treated wastewater) for the various levels of wastewater treatment in Options A-G presented in Figure 34\(^\text{41}\).

<table>
<thead>
<tr>
<th>Type of irrigation</th>
<th>Option (Figure 2.1)</th>
<th>Required pathogen reduction by treatment (log units)</th>
<th>Verification monitoring level (\textit{E. coli} per 100 ml)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unrestricted</td>
<td>A</td>
<td>4</td>
<td>(\leq 10^{4})</td>
<td>Root crops</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>3</td>
<td>(\leq 10^{6})</td>
<td>Leaf crops</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>2</td>
<td>(\leq 10^{5})</td>
<td>Drip irrigation of high-growing crops</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>4</td>
<td>(\leq 10^{5})</td>
<td>Drip irrigation of low-growing crops</td>
</tr>
<tr>
<td></td>
<td>E</td>
<td>6 or 7</td>
<td>(\leq 10^{4}) or (\leq 10^{6})</td>
<td>Verification level depends on the requirements of the local regulatory agency(^b)</td>
</tr>
<tr>
<td>Restricted</td>
<td>F</td>
<td>3</td>
<td>(\leq 10^{4})</td>
<td>Labour-intensive agriculture (protective of adults and children under 15 years of age)</td>
</tr>
<tr>
<td></td>
<td>G</td>
<td>2</td>
<td>(\leq 10^{5})</td>
<td>Highly mechanized agriculture</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>0.5</td>
<td>(\leq 10^{6})</td>
<td>Pathogen removal in a septic tank</td>
</tr>
</tbody>
</table>

\(^a\) “Verification monitoring” refers to what has previously been referred to as “effluent standards” or “effluent guideline” levels.

\(^b\) For example, for secondary treatment, filtration and disinfection: five-day biochemical oxygen demand (BOD\(_5\)), <10 mg/l; turbidity, <2 nephelometric turbidity units (NTU); chlorine residual, 1 mg/l; pH, 6–9; and faecal coliforms, not detectable in 100 ml (State of California, 2001).

Many health risks are associated with the reuse of wastewater for irrigation; however, if practiced carefully it can yield greater crop harvests, protect sanitation downstream, and save on fertilizer costs. This involves a communal commitment to follow rules regarding wastewater use and, if adopted, crop restrictions. Public perception must also be considered before implementing a wastewater reuse system. Some may find the idea offensive and care must be used when approaching and discussing the topic.

### 5.7 Infiltration Considerations

While most CWs include an impermeable liner, Team 7’s final design will not. If no liner is used, water infiltration into the ground must be taken into consideration. Typically, the largest concern with wastewater infiltration is the contamination of ground water. This depends largely on the permeability and conductivity of the soil and the distance to the groundwater table. In the chosen location, the groundwater table is a sufficient distance from the wastewater source for this not to be an issue. The groundwater is also unused for human consumption, adding a level of redundancy. Carabuela’s drinking water comes from springs in the mountains that is piped into the community and stored in water tanks.

\(^{41}\) From WHO Guidelines Vol. 2
Since groundwater contamination is not a constraint, the option of rejecting a liner is allowed. The main problem with high levels of infiltration associated with this project would be the lack of irrigation water and the possible drying of the wetland beds. If the wetland beds become dry, the hydrophilic plants could suffer and possibly die. This eliminates one of the main biological treatment mechanisms of the CW.

Overtime, infiltration beds and surfaces will clog with filtered media particles and biological growth. Once this occurs in the wetland bed, it will act essentially as an impermeable liner and can be modeled as-is. Until that time, infiltration calculations will have to be a part of the working model of the CWs. A biological mat on the bottom of the wetland can be an added benefit when taken into consideration. The biological organisms and bacteria living in the mat can help add another level of treatment to the waste and anaerobically breakdown waste constituents.

From a measured percolation test, the infiltration rate into the soil is about 3.1 min/cm (8 min/in). The soil in Carabuela (and most of the Sierra region of Ecuador) is made up of volcanic rock particles and sand. With a given percolation rate into the soil, an application rate can be found from Table 12.

<table>
<thead>
<tr>
<th>Soil Texture</th>
<th>Percolation Rate (min/in)</th>
<th>Application Rateb (gpd/ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel, coarse sand</td>
<td>&lt;1</td>
<td>Not suitablec</td>
</tr>
<tr>
<td>Coarse to medium sand</td>
<td>1 – 5</td>
<td>1.2</td>
</tr>
<tr>
<td>Fine sand, loamy sand</td>
<td>6 – 15</td>
<td>0.8</td>
</tr>
<tr>
<td>Sandy loam, loam</td>
<td>16 – 30</td>
<td>0.6</td>
</tr>
<tr>
<td>Loam, porous silt loam</td>
<td>31 – 60</td>
<td>0.45</td>
</tr>
<tr>
<td>Silty clay loam, clay loam</td>
<td>61 - 120</td>
<td>0.2e</td>
</tr>
</tbody>
</table>

a May be suitable for sidewall infiltration rates
b Rates based on septic tank effluent from a domestic waste source. A factor of safety may be desirable for wastes of significantly different character.
c Soils with percolation rates <1 min/in can be used if the soil is replaced with a suitably thick (>2ft) layer of loamy sand or sand.
d Soils with expandable clay

This application rate (0.8 gpd/ft² or 0.033 m/day) over the design area of the wetland gives a total infiltration of about 117 cubic meters per day (Appendix D). This is, or about half of the entire influent flow. For the initial start period then, the effluent flow rate will be about half of what would be expected from a lined wetland system. As the soil begins to clog overtime, the actual effluent flow will approach the expected lined effluent flow.

Depending on the concentrations of specific constituents in the wastewater, the soil will take a variable time to clog. High amounts of grease and fats can clog soils relatively quickly. For the wetland beds

---

42 From Onsite Wastewater Treatment and Disposal Systems, EPA Design Manual
operating at design conditions, it would not be surprising for the soils under the wetland bed to begin to clog after about 1-2 years.43

5.8 Residuals44

Periodically, bio-solids must be removed and disposed of properly in order to keep the treatment process in an optimal condition. The main source of residuals will be from the bar screens and septic tanks. Bar screens will need to be checked daily and the septic tanks will need to have solids removed every six months to a year. The solids removed will have high concentrations of BOD, suspended solids, grease, hair, grit and disease-causing pathogens.45 This requires care in their disposal in order to maintain healthy conditions.

In the United States, the largest volume of residuals comes from septic tanks. This is called septage and is handled in a variety of ways. Septage can be dewatered and spread over land; both on and under the surface, buried in trenches, applied to a landfill, burned, composted, digested (both aerobically and anaerobically), or treated with chemicals. In this design, many of the facilities and infrastructure used in the United States are lacking. This limits options for residual removal in Carabuela.

For this design, composting the removed residuals is recommended. This option does not require much equipment and can be done locally with little of the offensive odors associated with other methods. This is also much safer than some options and contains less risk of contaminating ground water. Composting requires adding a “bulking agent” to the waste in order to help aerate the waste and prevent stagnation. This requires periodic mixing of the waste with an organic agent such as wood chips or shavings. These agents are readily available in the area and are easy to create. After a suitable amount of time, most of the pathogens in the waste will be destroyed and the compost will be acceptable to add to soil.

Solids will accumulate at a predicted rate of about 0.07 cubic meters per person per year (Appendix E, Appendix H), giving a total volume of 210 cubic meters of solids per year for Carabuela. Since composting can be done continuously through the year, about 9m of compost rows would be required at any point in time. This requires a very small footprint and can be done on-site.

5.9 Construction and Phasing

The septic tank and CW designs were made with three targets in mind: current conditions, 5 years in the future, and 20 years in the future. The current conditions allowed the team to plan for future flows and expansion by using a 2% growth rate for the village. The number of needed wetland cells for each timeframe can be seen in Table 13.

43 From Onsite Wastewater Treatment and Disposal Systems, EPA Design Manual, chap. 7
44 From Onsite Wastewater Treatment and Disposal Systems, EPA Design Manual
45 Table 9-1, Onsite Wastewater Treatment and Disposal Systems
Table 13: Wetland Cells Needed Over Time

<table>
<thead>
<tr>
<th>Flow Rate (m$^3$/day)</th>
<th>Area Needed (m$^2$)</th>
<th>Cells (20m x 30m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Current</td>
<td>217</td>
<td>3169</td>
</tr>
<tr>
<td>5-Year</td>
<td>240</td>
<td>3499</td>
</tr>
<tr>
<td>20-Year</td>
<td>323</td>
<td>4710</td>
</tr>
</tbody>
</table>

From the areas needed, the team recommends constructing 6 wetland cells immediately and 2 septic tanks and expanding on an as-needed basis. While the ultimate design life of the entire process is 20 years out, building for 5 years out will allow the system to be up and running much sooner and with lower costs than building for 20 years out. Because the wetlands are designed in cells, it will be much easier to expand as community need dictates.

Each of the 2 septic tanks constructed for the 5-year design will discharge to 3 wetland cells each. This allows for parallel systems and redundancy. As expansion occurs, an additional septic tank can be built to pretreat the wastewater to the additional 3-5 wetland cells.

5.10 Maintenance

Routine maintenance procedures for septic tanks and CWs are fairly simple and small in scope. These procedures are shown in Table 14 in Table 15 and below. Treatment options were all chosen that required low amounts of maintenance. This was part of designing a transparent system. One of the goals in this design was to make a passive process that was easy to understand and operate. This allows for transparency in the design.

Table 14: Septic Tank Routine Maintenance

<table>
<thead>
<tr>
<th>Task</th>
<th>Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Remove Scum from Vent</td>
<td>Every 2 weeks</td>
</tr>
<tr>
<td>Remove Accumulated Sludge</td>
<td>6 months - 1 year</td>
</tr>
</tbody>
</table>

Table 15: Constructed Wetland Routine Maintenance

<table>
<thead>
<tr>
<th>Task</th>
<th>Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inlet Channel</td>
<td>Monthly</td>
</tr>
<tr>
<td>Check/control Water level</td>
<td>Daily</td>
</tr>
<tr>
<td>Harvest Plants</td>
<td>4-9 months</td>
</tr>
<tr>
<td>Change out Clogged Filter Media</td>
<td>15-20 years</td>
</tr>
</tbody>
</table>

46 From Constructed Wetlands: A Promising Wastewater Treatment System for Small Localities
47 From Constructed Wetlands: A Promising Wastewater Treatment System for Small Localities
5.11 Costs

All costs are in US dollars and based on materials available in Ecuador. Ecuador also uses the US dollar as its currency, making costing much simpler because of no exchange rate. The costing is also divided by the 5-year and 20-year design. Because the majority of the labor will be done by volunteers in the community, the team’s client, Bruce Rydbeck, requested that all labor costs be left out of the costing plan. The basic material costs are provided in Table 16. The majority of the costs were given by our client in Ecuador, however the pipe fittings and manholes were found from online listings\(^ {48}\).

<table>
<thead>
<tr>
<th>Material</th>
<th>Cost ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low Grade Concrete</td>
<td>100/m3</td>
</tr>
<tr>
<td>Structural Concrete</td>
<td>120/m3</td>
</tr>
<tr>
<td>Gravel</td>
<td>16.25/m3</td>
</tr>
<tr>
<td>Sand</td>
<td>16.25/m3</td>
</tr>
<tr>
<td>Plastic Liner</td>
<td>2/m2</td>
</tr>
<tr>
<td>Clay Liner</td>
<td>16.25/m3</td>
</tr>
<tr>
<td>Manhole</td>
<td>100/m</td>
</tr>
<tr>
<td>12cm PVC Pipe</td>
<td>6/m</td>
</tr>
<tr>
<td>15cm PVC Pipe</td>
<td>7/m</td>
</tr>
<tr>
<td>20cm PVC Pipe</td>
<td>9.55/m</td>
</tr>
<tr>
<td>12cm PVC 90deg elbow</td>
<td>14/each</td>
</tr>
<tr>
<td>15cm PVC 90deg elbow</td>
<td>25/each</td>
</tr>
<tr>
<td>20cm PVC 90deg elbow</td>
<td>65/each</td>
</tr>
<tr>
<td>12cm PVC T-joint</td>
<td>15/each</td>
</tr>
<tr>
<td>15cm PVC T-joint</td>
<td>25/each</td>
</tr>
<tr>
<td>20cm PVC T-joint</td>
<td>86/each</td>
</tr>
<tr>
<td>12cm PVC Shutoff valve</td>
<td>30/each</td>
</tr>
<tr>
<td>15cm PVC Shutoff valve</td>
<td>60/each</td>
</tr>
<tr>
<td>20cm PVC Shutoff valve</td>
<td>90/each</td>
</tr>
</tbody>
</table>

The piping costs were calculated based on the pipe diameters and their associated lengths, fittings and valves as determined in the Piping Section. The septic tank costs were calculated from the volume of concrete needed to construct the tanks based on a wall thickness of .2m and the septic tank perimeters. The costs for the wetlands were calculated by multiplying the various fill material volume needs by their unit costs. Two calculations were made to compare the wetland costs with or without a plastic liner. Because the plastic liner added $12,000 to the total costs it was deemed unnecessary and not included.


\(^{49}\) Costs Provided by Bruce Rydbeck
in the final design. A 20% contingency was calculated into the costs of the septic tanks and wetlands due to the uncertainty in material costs given by our client. The total cost summary is shown below in Table 17 with the cost calculations and spreadsheets in the Appendix section on Costs. Additional miscellaneous costs were calculated due to the cost of the manhole/distribution box before the septic tanks, and the distribution box before the wetlands.

<table>
<thead>
<tr>
<th>Total Costs</th>
<th>Phasing Plan</th>
<th>Component</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phasing Plan</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5 Year</td>
<td>Piping:</td>
<td>6447.41</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Septic Tanks:</td>
<td>17758.08</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Wetlands:</td>
<td>38610</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Miscellaneous:</td>
<td>300</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>63115.49</td>
<td></td>
</tr>
<tr>
<td>20 Year</td>
<td>Piping:</td>
<td>2583.11</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Septic Tanks:</td>
<td>6654.24</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Wetlands:</td>
<td>12870</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Miscellaneous:</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>22107.35</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td>85222.84</td>
</tr>
</tbody>
</table>

Constructing the 5-year design first and expanding as-needed will save the village approximately $22000 upfront. This is important when considering the financial means of Carabuela in conjunction with those of HCJB.

5.12 Sustainability

The sustainability of the design is of high importance to the team. This means more than simply its carbon footprint or its yearly maintenance costs. Yearly costs for the system must be lower than the utility and health benefits received from Carabuela from the treatment of waste and irrigation benefits. While low costs are a must in order to make this project feasible, there are more aspects that factor into its sustainability.

The Team’s goal is to adequately educate the community members about the use and upkeep of the treatment system. This is essential for the continued maintenance of the septic tank and constructed wetland beds. Often in community development projects, an organization will build or implement something for a community that quickly degrades because of lack of care and maintenance. This often
happens because the community has nothing at stake in the project or is undereducated on its use and upkeep.

Another aspect of the project’s sustainability is the governmental framework in which it is in context. Currently, there is very little regulation regarding wastewater in Ecuador. The team hopes to begin community dialogues regarding wastewater all across the country by providing a working model for Carabuela. The World Health Organization recommends considering projects like this in terms of its context in governmental policy, legislation, institutional framework, and regulations. In terms of policy, is there a clear policy regarding wastewater or wastewater reuse? For legislation and regulation, are there appropriate laws and rules governing the safe use of wastewater? Are they enforceable? And for the institutional framework, are there effective and appropriate legislative bodies to control wastewater? These areas are lacking around Carabuela and in the whole of Ecuador. Hopefully by implementing the design, Carabuela can begin to open up discussion and governmental framework on the issue.

5.13 Implementation

The design report and calculations are to be translated into Spanish and given to Bruce Rydbeck, the team’s contact in Ecuador for HCJB. HCJB will then present the team’s options and recommended decision to the community of Carabuela and the water board. The water board, in conjunction with HCJB, will then decide if they want to build the project on site. Most of the labor then will be provided by the village. Funding for the project, if implemented, may come from a variety of sources. HCJB will help with some costs and the village itself will also pay for much of it. It is also possible that the Ecuadorian government will fund part of the project as a type of national pilot project.

50 From WHO Guidelines, Vol. 1, pg.43
Appendices

Appendix A

Flow Calculation

Table 18: Effluent Bucket Tests

<table>
<thead>
<tr>
<th>Time to Fill (s)</th>
<th>12L Bucket</th>
<th>20L Bucket</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.8</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td>5.1</td>
<td>7.7</td>
<td></td>
</tr>
<tr>
<td>4.8</td>
<td>7.3</td>
<td></td>
</tr>
<tr>
<td>Average (s):</td>
<td>4.9</td>
<td>7.5</td>
</tr>
<tr>
<td>Flow Rate (L/s):</td>
<td>2.45</td>
<td>2.67</td>
</tr>
<tr>
<td>Total Average (L/s):</td>
<td>2.56</td>
<td></td>
</tr>
</tbody>
</table>

Table 19: Percolation Test

<table>
<thead>
<tr>
<th>Hole Dimensions:</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>17&quot; by 13&quot; with a depth of 21&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Saturated soil for ~4 hours</td>
<td></td>
<td></td>
</tr>
<tr>
<td>no clays observed</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Percolation test Results:</td>
<td>0.125</td>
<td>in/min</td>
</tr>
</tbody>
</table>
Appendix B

Treatment Design Alternatives

Passive treatment can be very cost effective and easier to maintain than more mechanical processes if designed correctly. The passive treatment options considered are discussed below.

Bar Screens

Bar Screens are used for preliminary treatment. The effluent from the collection system flows through a metal screen that filters out large objects such as rags and floatables. This prevents clogging downstream and protects equipment. The closer the bars are together on the screen, the more contaminates are removed, however, this increases the need to rake and remove contaminates from the screen. In most U.S. wastewater treatment plants, bar screens are mechanically raked but this system requires manual raking in order to be passive. Bar screens are very simple and have a very small footprint, which makes it an excellent candidate for the design. A secondary flow path is needed to maintain flow while cleaning the primary flow path as shown in Figure 35.

Figure 35: Manually Cleaned Bar Screen Structure Plan and Profile Views

---

51 Drawn by Ian Compton
Grit Removal

Grit is defined as sand, gravel, food waste, and other heavy solid materials. Removal of grit prevents excess accumulation in pipelines or downstream processes. Grit removal also decreases the amount of manual labor needed to maintain subsequent treatment processes. A passive grit removal technique that could be employed is a horizontal flow grit chamber. This uses weirs and control devices to maintain a constant flow of 0.3 m/s. The length of the chamber depends on the items shown in Table 20.

<table>
<thead>
<tr>
<th>Item</th>
<th>Range Metric (English)</th>
<th>Typical Metric (English)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Detention Time</td>
<td>45-90 s</td>
<td>60 s</td>
</tr>
<tr>
<td>Horizontal velocity</td>
<td>0.24-.0.4 m/s</td>
<td>0.3 m/s</td>
</tr>
<tr>
<td></td>
<td>(0.8-1.3 ft/s)</td>
<td>(1.0 ft/s)</td>
</tr>
<tr>
<td>Settling velocity 50-mesh</td>
<td>2.8-3.1 m/min</td>
<td>2.9 m/min</td>
</tr>
<tr>
<td></td>
<td>(9.2-10.2 ft/min)</td>
<td>(9.6 ft/min)</td>
</tr>
<tr>
<td></td>
<td>0.6-0.9 m/min</td>
<td>0.8 m/min</td>
</tr>
<tr>
<td></td>
<td>(2.0-3.0 ft/min)</td>
<td>(2.5 ft/min)</td>
</tr>
<tr>
<td>Settling velocity 100-mesh</td>
<td></td>
<td>2.9 m/min</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(9.6 ft/min)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.8 m/min</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(2.5 ft/min)</td>
</tr>
<tr>
<td>Head loss (% of channel depth)</td>
<td>30-40%</td>
<td>36%</td>
</tr>
<tr>
<td>Inlet and outlet length allowance</td>
<td>25-50%</td>
<td>30%</td>
</tr>
</tbody>
</table>

Waste Stabilization Ponds

Waste Stabilization Ponds (or WSPs) use the sun and natural processes to treat raw sewage. There are three types of ponds considered for the design; anaerobic, facultative, and maturation as seen in Figure 36. All are open bodies of water that require little to no human supervision or interaction.

---

53 Mara, D. Domestic Wastewater Treatment in Developing Countries. London: Earthscan Publications, 2004. 109 Internet resource
Anaerobic Pond

Anaerobic Ponds treat wastewater primarily through settling of sludge. There is typically no presence of air in the water. The depth is typically 2-5 meter deep. The volume is then calculated based on volumetric loading because the surface is part of the treatment in an anaerobic pond. 

\[ y_v = (20 \times Temperature) - 100 = (\text{gram/day/m}^3) \]  
Equation 1 (Mara, 1997)

Removal of the BOD is similarly modeled according to the temperature as seen in 

\[ \% \text{BOD removal} = 2 \times Temperature + 20 \]  
Equation 2 (Mara, 1997)

Facultative Pond

Facultative Ponds are the largest of the set of ponds. The BOD is removed due to algae growth promoted by sunlight. Because sunlight is vital to the functioning of a facultative pond, it is sized based on a surface loading as seen in 

\[ y_v = (20 \times Temperature) - 100 = (\text{gram/day/m}^3) \]  
Equation 1. The surface area of the pond then is calculated with

\[ \text{Surface Area} = A_f = \frac{10 \times \text{Average Flow}}{y_s} \]  
Equation 4. Having a lower surface loading will avoid odor from being a problem for those living near by.

\[ \text{Surface Loading (kg/ha/day)} = y_s = 350(1.107 - 0.002 \times Temperature)^{Temperature-20} \]  
Equation 3 (Mara, 1987)

---

55 Drawn by Ian Compton
Internet resource
In Facultative ponds the majority of the BOD is removed. When a pond is sized right it can be assumed that 70% of BOD is removed.

**Maturation Pond**
Maturation Ponds are the third treatment process in the normal sequence of wastewater treatment ponds. Their main purpose is to remove fecal coliform. The removal is based on a recommended value retention time. There are many kinetic correlations that model the performance and sizing of Maturation ponds, however they are not as pertinent in the case of Carabuela due to the fact that the effluent will not be used for drinking water near downstream of out fall.

**Septic tanks**
Septic tanks are similar to anaerobic ponds in that they separate the solids from the liquids and biologically degrade the waste\(^{57}\). A septic tank, however, is a watertight tank underground as shown in Figure 37. The tank allows waste to be broken down by bacteria and also relies on a certain residence time in order to optimize effectiveness; however, septic tanks have an average residence time of about 1.5 days, making its volume requirements much less than anaerobic ponds. Septic tanks also require routine removal of accumulated solids.

\(^{57}\) From *Onsite Wastewater Treatment and Disposal Systems*, EPA Design Manual
Bio-filtration

Bio-filtration relies on a gravity feed of the waste stream through a bed of filter media as seen in Figure 38. This is often sand or gravel of various grain sizes, but can be different forms of activated carbon or even man-made material. Filters of this type are used as secondary treatment after much of the solids are removed. The filters then often contain a layer of biofilm which helps further reduce BOD content in waste streams. Some biofilm is flushed out with the water and replenished by bacteria. Scum layers form periodically on the top of bio-filters and need to be routinely backwashed and/or scraped off in order to maintain optimal working conditions. The scum may contain disease-causing pathogens, but can be safely scraped off and buried.59

58 Drawn by Ian Compton
59 From Onsite Wastewater Treatment and Disposal Systems, EPA Design Manual
Constructed Wetlands

Constructed wetlands (CW) are a process to filter out organic waste, pathogens, and nutrients using natural plant processes and media filtration. Constructed wetlands consist of a bed of gravel or sand with water-loving plants. Wastewater then flows through the bed, either vertically or horizontally, and is collected by an under drain. CWs have a high BOD and pathogen removal (80%-90%, and up to 3 log units, respectively) but a lower removal rate for Nitrogen and Phosphorous (20%)\textsuperscript{61}. These are safer than open bodies of water and do not have offensive odors because the flow is all subsurface. CWs can add additional benefits in the form of the plants grown in the wetlands. These can be harvested and used for fuel or livestock fodder.

\textsuperscript{60} Drawn by Ian Compton
\textsuperscript{61} Constructed Wetlands: A Promising Wastewater Treatment System for Small Localities
Ground infiltration is a process in which a treated discharge stream is allowed to percolate through the ground. This effectively uses the soil as a type of filter media. This process relies heavily on the type of soils in the area and the elevation of the water table. At least three feet of dry soil is required to maximize pollutant removal and prevent ground water contamination. Infiltration usually takes place as ground application or as an underground set of perforated pipes.

With ground application, an effluent stream is discharged onto a gravel bed that overlays the intended infiltration area as seen in Figure 40. With underground infiltration, a pipe or pipes are laid along the bottom of an excavated trench or bed and then packed with gravel before backfilling. These methods also require routine maintenance in order to scrape off a biofilm “mat” that forms on top of the filter media and can lower the infiltration capacity. This can be lessened with the use of dosing multiple infiltration beds one at a time. This allows a period of drying for a field and can help prevent a mat from building up.

---

62 From Onsite Wastewater Treatment and Disposal Systems, EPA Design Manual
Preliminary Treatment Decision Matrix

With many treatment options it was necessary to implement a decision matrix. The matrix seen in Table 21 is divided into preliminary treatment options and primary treatment options. Five characteristics are used to evaluate each treatment process.

8) Passive
- This was a constraint from HCJB and is critical for the design. A passive design considerably lowers maintenance and equipment costs. These processes may still require periodic cleaning and adjusting.

9) Maintenance
- The facility will be maintained by villagers who may not be experienced with wastewater treatment processes. The only maintenance will be in the form of manual labor. Therefore, the decision of the numerical value was based on frequency and amount of labor needed.

---

63 Drawn by Ian Compton
10) Footprint
- The treatment facility needs to fit in a particular location in Carabuela. The current size of the land available is unknown, yet the matrix is giving treatment options with a smaller footprint preference.

11) Cost
- The main cost consideration of the treatment facility will be the materials needed. Manual labor will be provided for construction. In this case, the amount of piping drives the cost characteristic.

12) Quality
- The wastewater needs to be treated effectively. The options presented all provide sufficient treatment. The value of quality is based on the speed and capacity of treated wastewater.

The characteristics were weighted to show a hierarchy of importance. Each of the treatment options is given a rating in a characteristic from worst, 1, to greatest, 10. Three treatment options stood out from the rest of the options. These are the most desired treatments to implement. However, the different combinations of the options will dramatically increase the effectiveness of the system. A combination of these three options forms the basis of the team’s preliminary design.

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Passive</th>
<th>Maintenance</th>
<th>Footprint</th>
<th>Cost</th>
<th>Quality</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight</td>
<td>10</td>
<td>6</td>
<td>7</td>
<td>8</td>
<td>6</td>
<td>370</td>
</tr>
<tr>
<td><strong>Preliminary</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bar Screens</td>
<td>10</td>
<td>8</td>
<td>10</td>
<td>8</td>
<td>4</td>
<td>306</td>
</tr>
<tr>
<td>Grit removal</td>
<td>5</td>
<td>8</td>
<td>10</td>
<td>5</td>
<td>3</td>
<td>226</td>
</tr>
<tr>
<td><strong>Primary</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Waste Stabilization Ponds</td>
<td>10</td>
<td>6</td>
<td>3</td>
<td>10</td>
<td>8</td>
<td>285</td>
</tr>
<tr>
<td>Trickling filter</td>
<td>5</td>
<td>4</td>
<td>7</td>
<td>5</td>
<td>9</td>
<td>217</td>
</tr>
<tr>
<td>Septic Tank</td>
<td>8</td>
<td>4</td>
<td>6</td>
<td>6</td>
<td>7</td>
<td>236</td>
</tr>
<tr>
<td>Constructed Wetlands</td>
<td>10</td>
<td>8</td>
<td>4</td>
<td>4</td>
<td>8</td>
<td>256</td>
</tr>
<tr>
<td>Ground Infiltration</td>
<td>10</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>316</td>
</tr>
</tbody>
</table>
Appendix C

Preliminary Design

Based on the treatment design decision matrix seen in Table 21, the two primary treatment options were researched on a more detailed level and fitted to the specific design.

6.1 Collection System

Preliminary Design 1.1
Because the initial information was so limited, the preliminary design for the collection system was based on numerous assumptions. The entire system to collect from all 508 homes was initially modeled with sewage pipes running down each street and laid out in AutoCAD. The design can be seen below in Figure 41. The main assumption made was that no system already exists. The assumption was made to get an assessment of what the total length of pipe would be needed if the entire system needed to be replaced. The initial design yielded a total pipe length of 11500m. The estimated pipe diameters and lengths can be seen below in Table 22. The pipe diameters and locations were chosen to have the smaller pipes on the outskirts and the larger pipes for the main sewers where the largest flows are found. The primary purpose of the preliminary design 1.1 was to find out what total pipe lengths would be to estimate the initial cost. Because the location of the existing system was too vague, the initial cost plan used this design layout of the entire system and did not take into account the possible 200 homes already connected. After further information was collected, a much more accurate assessment was made.

Table 22: Preliminary Design 1.1 Pipe Lengths and Diameters

<table>
<thead>
<tr>
<th>Length (m)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>15cm Concrete Pipe</td>
<td>1014</td>
</tr>
<tr>
<td>20cm Concrete Pipe</td>
<td>953</td>
</tr>
<tr>
<td>25cm Concrete Pipe</td>
<td>1540</td>
</tr>
<tr>
<td>Total</td>
<td>3507</td>
</tr>
</tbody>
</table>
Figure 41: Preliminary Sewer Design 1.1 AutoCAD Approximate Pipe Layout for Entire System
Preliminary Design 1.2
After completing the general layout in Preliminary Design 1.1, a much more accurate and specific model was created in Preliminary Design 1.2. This design’s purpose was to model the area on the east side of the village where the probable location of the existing collection system resides. The computer software used for this design was SWMM (Storm Water Management Model), which is a universally recognized storm water and wastewater simulation program. Although the actual location of the system was unknown, it was assumed that the pipes ran along the major streets and collected from all the homes in the most populated part of the area between the hill and the Pan-American Highway. Figure 41 shows the SWMM system layout. Table 23 shows the flow calculations for the model. (NOTE: Because SWMM only uses US Customary units all units were converted to that, and after the final model was finished all the results were converted back to SI units)

The flow calculations in Table 23 were based on the water consumption and infiltration flows given by HCJB. Assuming that the average household contains 5 members the total population connected to the system was calculated. Additionally, the peak factor as a function of population was calculated using Curve G given in the ASCE Manuals and Reports on Engineering Practice- No. 60.

The peak factor multiplied by the average discharge yielded the peak discharge for the entire system, and when divided by the number of homes, it gave the peak discharge per home. Finally, the peak discharge per home was multiplied by the number of homes contributing to each node in the SWMM system and modeled as inflow in that location.

<table>
<thead>
<tr>
<th>Collection System Flow Calculations</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>US Customary Units</strong></td>
</tr>
<tr>
<td>Discharge</td>
</tr>
<tr>
<td>Infiltration</td>
</tr>
<tr>
<td>Infiltration</td>
</tr>
<tr>
<td># of Homes</td>
</tr>
<tr>
<td>Population</td>
</tr>
<tr>
<td>Total Population</td>
</tr>
<tr>
<td>Peak Factor</td>
</tr>
<tr>
<td>Average Discharge</td>
</tr>
<tr>
<td>Peak Discharge</td>
</tr>
<tr>
<td>Peak Discharge Per Home</td>
</tr>
</tbody>
</table>
Figure 42: Preliminary Sewer Design 1.2 SWMM Map with Manhole Invert Elevations and Pipe Slope
Several key design parameters were assumed for Preliminary Design 1.2. First, to ensure that the pipes conveyed all of the waste adequately, a minimum velocity of .61m/s was assumed (Practice No. 60). To attain the minimum velocity, a minimum slope was required for each pipe. The minimum slopes for various diameters required to ensure a .61m/s velocity are shown below in Table 24 (Practice No. 60) and the slopes of each pipe are seen above in Figure 42. Figure 42 also shows the invert elevations based on the contour map of the village provided by HCJB. To prevent velocities from getting too high on the villages steep slopes, a maximum velocity of 3m/s was also assumed (Practice 60). Figure 43 shows the designs initial velocities in the pipes. This initial design shows some possible problems in the system that needed to be addressed once final pipe locations and slopes were confirmed. Primarily, the main possible problem is found the areas where velocities may not reach the minimum .61m/s or may not exceed 3m/s.

Table 24 : Sewer Size and Minimum Slope to Maintain a 2ft/s Flow Velocity

<table>
<thead>
<tr>
<th>Sewer Size (m)</th>
<th>Minimum Slope (m/100m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>0.40</td>
</tr>
<tr>
<td>25</td>
<td>0.28</td>
</tr>
<tr>
<td>30</td>
<td>0.22</td>
</tr>
<tr>
<td>38</td>
<td>0.15</td>
</tr>
</tbody>
</table>
Second, the minimum cover necessary was assumed to be 1 meter. Because the area is never inflicted with freezing temperatures, no consideration was needed for the possibility of the water freezing in the pipes. Additionally, because none of the houses have basements where bathrooms or other water utilities exist, no consideration was taken to place the pipes below house basement levels. The minimum cover of 1 meter was chosen to ensure that the pipes are always protected from vehicle loads in the roads and possible erosion that would expose the pipes. The pipe and soil profile view can be seen below in Figure 44.
Finally, a pipe diameter of 20 cm was chosen initially based on the smallest available pipe sizes. Sewers in the United States rarely use pipes smaller than 20cm but room was left for further adjustment based on the information gained in Ecuador. The main criteria for the conduit flows were that the pipes should never surcharge and should be able to adequately handle all the flows. Figure 45 below shows the peak flows in each pipe. Figure 45 shows each pipe’s capacity, or the ratio of maximum depth to full depth. The pipe capacity confirms that none of the pipes ever become surcharged because none of the values exceed 1.
Figure 45: Preliminary Sewer Design 1.2 SWMM Model Conduit Peak Flows

Figure 46: Preliminary Sewer Design 1.2 SWMM Model Conduit Capacity (Ratio of Depth to Full Depth)
Materials Standards
When first evaluating the condition of the collection system, materials standards were taken into consideration. The piping materials that are commonly used in the U.S. for sewer design are shown below in Table 25. These materials are also commonly used around the world and will most likely be found in Ecuador.

Table 25: Piping Materials

<table>
<thead>
<tr>
<th>Pipe Material</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asbestos cement</td>
<td>Rigid yet light-weight; moderate resistance to corrosion</td>
</tr>
<tr>
<td>Ductile iron</td>
<td>Very leak-proof; susceptible to acid corrosion</td>
</tr>
<tr>
<td>Reinforced concrete</td>
<td>High availability; vulnerable to corrosion if waste stream contains hydrogen sulfide or in high-sulfate environment</td>
</tr>
<tr>
<td>Pre-stressed concrete</td>
<td>Well-suited to long transmission mains; vulnerable to corrosion</td>
</tr>
<tr>
<td>Polyvinyl chloride</td>
<td>Lightweight and strong plastic material; resistant to corrosion</td>
</tr>
<tr>
<td>Vitrified clay</td>
<td>Commonly used in past; resistant to corrosion; quite brittle and susceptible to leakage</td>
</tr>
</tbody>
</table>

Preliminary Collection System Conclusion
On the tip to Ecuador the initial assumptions were proven false because there was found to be an adequate collection system already in place. Although the collection system was found to be adequate and in no need of replacement or repair at this time, the previous preliminary design process for the collection system can still be used in the future as a model for any additions or repairs needed. Because there was no need for a final design of a collection system, this allowed the team to focus solely on the design of the treatment process.

6.2 Septic Tank and Wetlands
In the sizing of the constructed wetlands, the effluent concentration of BOD was crucial. By using the equation:

\[
A_h = \frac{Q \ast (\ln(C_i) - \ln(C_e))}{K_{BOD}}
\]

64 Source: From Metcalf & Eddy, Inc. [6-8]
65 From “Constructed Wetlands Manual”, UN

XIX
\[ A_h = \frac{Q \cdot (\ln(C_i) - \ln(C_e))}{K_{BOD}} \]  

Equation 5

In this equation, \( A_h \) is the footprint area, \( Q \) is the flow rate, \( C_i \) and \( C_e \) are the BOD influent and effluent concentrations, respectively, and \( K_{BOD} \) is a kinetics constant. It was assumed that the septic tanks would reduce BOD in the waste stream by 30\%\(^6\). According to the expected influent concentrations in Table 23, this gave an estimated BOD influent of 200mg/L into the wetlands. By using the 30mg/L effluent standard adopted by the team, the wetlands footprint was found to be between 3,300m\(^2\) and 4,800m\(^2\) depending on the design life.

The costs associated with CWs are the costs of excavation, piping, and the fill material. Since labor costs are left out of the calculations by HCJB’s request, the fill material makes up the bulk of the wetland cost. The total volume of the wetland options then are between 2,000m\(^3\) and 2,640m\(^3\). This volume is made up of mainly sand and gravel, both of which cost $16.25 per cubic meter (See Table 16). The total cost for the various sizing options is between $41,830 and $55,770 in US dollars.

6.3 Waste Stabilization Ponds

As seen previously in Error! Reference source not found. typical waste stabilization ponds are done with three main components.

Waste Stabilization Ponds

The most important design parameters for waste stabilization pond design are temperature, net evaporation, flow, and BOD inflow. Table 26 shows the parameters used to size the ponds. The flow used is the 20 year projected flow. We used an estimate of 30 gcd\(^6\) for BOD concentration, which results in a wastewater BOD of 287 mg/L. Net evaporation rate data was hard to find, so a conservative estimate was used. A pan test needs to be conducted to more accurately define the net evaporation rate. The target reduction of the fecal coliform per 100 ml of wastewater is <10\(^4\).\(^6\) This will allow the treated effluent to be used for restricted irrigation based on WHO standards. Figure 47 shows the potential layout for the waste stabilization ponds. There are two sets of anaerobic, facultative, and maturation ponds connected in parallel. The ponds connected in parallel provide redundancy so that a pond can be shut down for desludging while the system remains operational. The additional ponds also provide the desired removal of fecal coliform to provide an effluent suitable for irrigation purposes. The pond sizing required to produce an effluent of 1767 fecal coliform per 100 ml of wastewater is given in Table 27. This effluent would be clean enough for restricted irrigation. In order to achieve a cleaner effluent the ponds would need to be larger or have the influent pretreated. The area of land available

\(^6\) From “Constructed Wetlands Manual”, UN
will be determined upon visiting the site. The preferred placement of the ponds would be at a higher
elevation so that the treated effluent can be conveyed to irrigation fields without installing pumps.

![Diagram of Waste Stabilization Pond Layout](image)

**Figure 47: Waste Stabilization Pond Layout**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>temperature (°C)</td>
<td>16</td>
</tr>
<tr>
<td>net evaporation rate (mm/day)</td>
<td>4.2</td>
</tr>
<tr>
<td>flow (L/day)</td>
<td>479700</td>
</tr>
<tr>
<td>BOD₅ (mg/L)</td>
<td>287</td>
</tr>
<tr>
<td>Volumetric Loading (gm/day/m³)</td>
<td>200</td>
</tr>
<tr>
<td>Anaerobic Pond Depth (m)</td>
<td>3</td>
</tr>
<tr>
<td>Facultative Pond Depth (m)</td>
<td>1.5</td>
</tr>
<tr>
<td>Maturation Pond Depth (m)</td>
<td>1</td>
</tr>
<tr>
<td>Surface Loading (kg/hectare-day)</td>
<td>262</td>
</tr>
<tr>
<td>Fecal Coliform/100 ml of Wastewater</td>
<td>10,000,000</td>
</tr>
</tbody>
</table>

---

69 [www.weather.com](http://www.weather.com)

### Table 27: Pond Sizing

<table>
<thead>
<tr>
<th>Pond</th>
<th>Quantity</th>
<th>Retention Time (days)</th>
<th>Size (m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anaerobic</td>
<td>2</td>
<td>1.43</td>
<td>127</td>
</tr>
<tr>
<td>Facultative</td>
<td>2</td>
<td>8.54</td>
<td>1512</td>
</tr>
<tr>
<td>Maturation</td>
<td>2</td>
<td>4.47</td>
<td>1186</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>6</strong></td>
<td><strong>14.44</strong></td>
<td><strong>5650</strong></td>
</tr>
</tbody>
</table>

### 6.4 Secondary Decision Matrix

#### Table 28: Secondary Design Summary

<table>
<thead>
<tr>
<th>Option 1</th>
<th>Holding Volume</th>
<th>HRT</th>
<th>Area</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>m$^3$</td>
<td>days</td>
<td>m$^2$</td>
<td>US $</td>
</tr>
<tr>
<td>Septic Tank</td>
<td>180</td>
<td>1.5</td>
<td>198.0</td>
<td>$19,339</td>
</tr>
<tr>
<td>Wetland</td>
<td>640</td>
<td>2.7</td>
<td>3,499.0</td>
<td>$41,830</td>
</tr>
<tr>
<td><strong>Sub Total</strong></td>
<td><strong>820</strong></td>
<td><strong>4.2</strong></td>
<td><strong>3,697.0</strong></td>
<td><strong>$61,169</strong></td>
</tr>
<tr>
<td>Land cost</td>
<td>$5000/acre</td>
<td></td>
<td></td>
<td>$6,852</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>$68,021</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Option 2</th>
<th>Holding Volume</th>
<th>HRT</th>
<th>Area</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>m$^3$</td>
<td>days</td>
<td>m$^2$</td>
<td>US $</td>
</tr>
<tr>
<td>Anaerobic</td>
<td>330</td>
<td>1.364</td>
<td>110.9</td>
<td>$288</td>
</tr>
<tr>
<td>Facultative</td>
<td>21790</td>
<td>89.28</td>
<td>14,520.0</td>
<td>$37,760</td>
</tr>
<tr>
<td><strong>Sub Total</strong></td>
<td><strong>22120</strong></td>
<td><strong>90.644</strong></td>
<td><strong>14,630.9</strong></td>
<td><strong>$38,048</strong></td>
</tr>
<tr>
<td>Land cost</td>
<td>$5000/acre</td>
<td></td>
<td></td>
<td>$27,116</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>$65,164</strong></td>
</tr>
</tbody>
</table>
### Table 29: Secondary Decision Matrix

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Passive</th>
<th>Maintenance</th>
<th>Footprint</th>
<th>Cost</th>
<th>Effluent Quality</th>
<th>Aesthetics</th>
<th>Irrigation</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight</td>
<td>10</td>
<td>7</td>
<td>8</td>
<td>8</td>
<td>5</td>
<td>4</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td><strong>Primary</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Anaerobic Pond</td>
<td>10</td>
<td>9</td>
<td>7</td>
<td>8</td>
<td>5</td>
<td>3</td>
<td>0</td>
<td>320</td>
</tr>
<tr>
<td>Septic Tanks</td>
<td>9</td>
<td>7</td>
<td>9</td>
<td>6</td>
<td>7</td>
<td>7</td>
<td>0</td>
<td>322</td>
</tr>
<tr>
<td><strong>Secondary</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0</td>
</tr>
<tr>
<td>Constructed Wetlands</td>
<td>10</td>
<td>7</td>
<td>8</td>
<td>7</td>
<td>8</td>
<td>8</td>
<td>6</td>
<td>383</td>
</tr>
<tr>
<td>Facultative Pond</td>
<td>10</td>
<td>9</td>
<td>4</td>
<td>6</td>
<td>7</td>
<td>7</td>
<td>8</td>
<td>362</td>
</tr>
<tr>
<td><strong>Summary</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Option 1</strong></td>
<td>19</td>
<td>14</td>
<td>17</td>
<td>13</td>
<td>15</td>
<td>15</td>
<td>6</td>
<td>705</td>
</tr>
<tr>
<td><strong>Option 2</strong></td>
<td>20</td>
<td>18</td>
<td>11</td>
<td>14</td>
<td>12</td>
<td>10</td>
<td>8</td>
<td>682</td>
</tr>
</tbody>
</table>
Appendix D

Constructed Wetland Calculations

**Constructed Wetland - Horizontal Flow**

Calculated by: Nathan Williams

Checked by: Adam

**Design Parameters**

**Flow Rates**

\[ Q_{measured} = 2.56 \, \text{L/s} \]

\[ Q_{present} := Q_{measured} \]

\[ Q_{future20} := Q_{present} \cdot (1.02)^{20} \]

\[ Q_{future5} := Q_{present} \cdot (1.02)^{5} \]

\[ Q_{school} = \frac{120}{30} \, \text{m}^3 \text{/ day} \]

Flow rate for school is 120 cubic meters per month!

\[ Q_{noschool} = Q_{present} - Q_{school} \]

\[ Q_{noschool5} := Q_{noschool} \cdot (1.02)^{5} \]

\[ Q_{noschool20} := Q_{noschool} \cdot (1.02)^{20} \]

**Design Flow Rates**

<table>
<thead>
<tr>
<th>School On-Line</th>
<th>School Off-Line</th>
</tr>
</thead>
<tbody>
<tr>
<td>Present</td>
<td>Q_{present} = 2.56 , \text{L/s}</td>
</tr>
<tr>
<td>Present 5 Year</td>
<td>Q_{future5} = 2.826 , \text{L/s}</td>
</tr>
<tr>
<td>Present 20 Year</td>
<td>Q_{future20} = 3.804 , \text{L/s}</td>
</tr>
</tbody>
</table>
Q := Q_{quoschool5}  

Established design Q for rest of calculations

---

**Loading Rates**

\[ K_{BOD} = 0.11 \frac{m}{day} \]

Rate constant from UN Wetland doc

\[ C_e := 30 \]

Effluent BOD concentration (mg/L)

---

**Wetland Footprint**

Assumption:
Influent BOD is considered "strong waste" at 300 mg/L.
Primary Treatment would remove 90% of initial BOD.

\[ C_{primary} := 200 \]

\[ A_{primary} = \frac{Q (\ln(C_{primary}) - \ln(C_e))}{K_{BOD}} \]

\[ A_{primary} = 3.499 \times 10^3 \text{ m}^2 \]
\[ \begin{align*}
A_{B0} &= \frac{Q_{\text{present}} (\ln(C_{\text{primary}}) - \ln(C_0))}{K_{\text{BOD}}} \\
A_{B5} &= \frac{Q_{\text{future5}} (\ln(C_{\text{primary}}) - \ln(C_0))}{K_{\text{BOD}}} \\
A_{B20} &= \frac{Q_{\text{future20}} (\ln(C_{\text{primary}}) - \ln(C_0))}{K_{\text{BOD}}} \\
A_{\text{noschool}} &= \frac{Q_{\text{noschool}} (\ln(C_{\text{primary}}) - \ln(C_0))}{K_{\text{BOD}}} \\
A_{\text{noschool5}} &= \frac{Q_{\text{noschool5}} (\ln(C_{\text{primary}}) - \ln(C_0))}{K_{\text{BOD}}} \\
A_{\text{noschool20}} &= \frac{Q_{\text{noschool20}} (\ln(C_{\text{primary}}) - \ln(C_0))}{K_{\text{BOD}}} 
\end{align*} \]
Depth = 0.4 m

Media grain diameter varies from 0.2mm to 30mm (UN). Inlet and outlet zones should be bigger, between 40mm and 80mm in diameter. Volcanic rock is often used and is a suitable material (WSP Doc). Substrate usually arranged in Figure 14 in UN Doc. Gravel, sand, gravel, coarse gravel.

Liners typically used to prevent wastewater from leaching into groundwater. However, this could be useful in eliminating wastewater and creating a smaller footprint.

Wetland cells should be no longer than 30m in length and 20m in width.
-From Constructed Wetlands: A promising Wastewater Treatment System for Small Localities

CellArea := 30 m\(^2\) 20 m

CellArea = 600 m\(^2\)

\[
\begin{align*}
\text{Cells}_0 := & \frac{A_{h0}}{\text{CellArea}} \\
\text{Cells}_5 := & \frac{A_{h5}}{\text{CellArea}} \\
\text{Cells}_{20} := & \frac{A_{h20}}{\text{CellArea}} \\
\text{CellNS0} := & \frac{Ahnschool}{\text{CellArea}} \\
\text{Cells}_{NS5} := & \frac{Ahnschool5}{\text{CellArea}} \\
\text{Cells}_{NS20} := & \frac{Ahnschool20}{\text{CellArea}} \\
\text{CellNSR} := & \frac{(Ahnschool^{1.25})}{\text{CellArea}} \\
\text{Cells}_{NS5R} := & \frac{(Ahnschool5^{1.25})}{\text{CellArea}} \\
\text{Cells}_{NS20R} := & \frac{(Ahnschool20^{1.25})}{\text{CellArea}}
\end{align*}
\]

Total Cells to account for 25% redundancy

\[
\begin{align*}
\text{Cells}_0 &= 5.38 \\
\text{Cells}_5 &= 5.49 \\
\text{Cells}_{NS0} &= 5.282 \\
\text{Cells}_{NS5} &= 5.832 \\
\text{Cells}_{NSR} &= 7.29 \\
\text{Cells}_{NS20} &= 7.849 \\
\text{Cells}_{NS20R} &= 9.812
\end{align*}
\]
Volume

Due to 1% Bedslope in cells, there is an additional 90 cubic meters of material needed per cell above CellArea Depth Calculation. However, this number is reliant upon a single bed. If terraced, the beds will require less extra volume.

Terraces := 0

AdditionalVolume := \left[ \frac{30 \text{ m}}{(\text{Terraces} + 1)} \right] \times 0.01 \times \left[ \frac{30 \text{ m}}{(\text{Terraces} + 1)} \right] \times 5.26 \text{ m} = 90 \text{ m}^3

TotalVolume_0 := \text{ceil}(\text{Cells}_0) \times \text{CellArea Depth} + (\text{ceil}(\text{Cells}_0) \times \text{AdditionalVolume})

TotalVolume_5 := \text{ceil}(\text{Cells}_5) \times \text{CellArea Depth} + (\text{ceil}(\text{Cells}_5) \times \text{AdditionalVolume})

TotalVolume_{20} := \text{ceil}(\text{Cells}_{20}) \times \text{CellArea Depth} + (\text{ceil}(\text{Cells}_{20}) \times \text{AdditionalVolume})

TotalVolume_{NS_0} := \text{ceil}(\text{Cells}_{NS_0}) \times \text{CellArea Depth} + (\text{ceil}(\text{Cells}_{NS_0}) \times \text{AdditionalVolume})

TotalVolume_{NS_5} := \text{ceil}(\text{Cells}_{NS_5}) \times \text{CellArea Depth} + (\text{ceil}(\text{Cells}_{NS_5}) \times \text{AdditionalVolume})

TotalVolume_{NS_{20}} := \text{ceil}(\text{Cells}_{NS_{20}}) \times \text{CellArea Depth} + (\text{ceil}(\text{Cells}_{NS_{20}}) \times \text{AdditionalVolume})

TotalVolume_0 = 1.98 \times 10^3 \text{ m}^3

TotalVolume_5 = 1.98 \times 10^3 \text{ m}^3

TotalVolume_{20} = 2.64 \times 10^3 \text{ m}^3

TotalVolume_{NS_0} = 1.98 \times 10^3 \text{ m}^3

TotalVolume_{NS_5} = 1.98 \times 10^3 \text{ m}^3

TotalVolume_{NS_{20}} = 2.64 \times 10^3 \text{ m}^3
Cost Considerations

Total Costs include 30% contingency.

\[
\text{Price}_{\text{gravel}} = 16.25 \frac{n}{m^3} \\
\text{Price}_{\text{sand}} = 16.25 \frac{n}{m^3} \\
\text{Price}_{\text{liner}} = 2 \frac{n}{m^2} \\
\text{Price}_{\text{clay}} = 16.25 \frac{n}{m^3} \\
\text{Price}_{\text{LGConcrete}} = 100 \frac{n}{m^3} \\
\text{Price}_{\text{StructConcrete}} = 120 \frac{n}{m^3}
\]

\[
\text{Cost}_{\text{fill0}} = \text{TotalVolume}_{0} \cdot \text{Price}_{\text{gravel}} \\
\text{Cost}_{\text{liner0}} = \text{Price}_{\text{liner}} \cdot \text{ceil} \left( \text{Cell}_{0} \right) \cdot \text{CellArea} \\
\text{Cost}_{\text{fill20}} = \text{TotalVolume}_{20} \cdot \text{Price}_{\text{gravel}} \\
\text{Cost}_{\text{liner20}} = \text{Price}_{\text{liner}} \cdot \text{ceil} \left( \text{Cell}_{20} \right) \cdot \text{CellArea}
\]

\[
\text{TotalCost}_{\text{liner0}} = \left( \text{Cost}_{\text{fill0}} + \text{Cost}_{\text{liner0}} \right) \cdot 1.3 \\
\text{TotalCost}_{\text{noliner0}} = \text{Cost}_{\text{fill0}} \cdot 1.3 \\
\text{TotalCost}_{\text{liner5}} = \left( \text{Cost}_{\text{fill5}} + \text{Cost}_{\text{liner5}} \right) \cdot 1.3 \\
\text{TotalCost}_{\text{noliner5}} = \text{Cost}_{\text{fill5}} \cdot 1.3 = 4.183 \times 10^6 \text{ft}^3
\]
\[
\text{CellCost}_{\text{liner}} := \text{CellArea Price}_{\text{liner}} + \text{CellArea Depth Price}_{\text{gravel}} = 5.1 \times 10^3 \pi
\]

\[
\text{CellCost}_{\text{noliner}} := \text{CellArea Depth Price}_{\text{gravel}} = 3.9 \times 10^3 \pi
\]

\[
\text{TotalCost}_{\text{liner}0} = 5.119 \times 10^4 \pi
\]

\[
\text{TotalCost}_{\text{noliner}0} = 4.183 \times 10^4 \pi
\]

\[
\text{TotalCost}_{\text{liner}5} = 5.119 \times 10^5 \pi
\]

\[
\text{TotalCost}_{\text{noliner}5} = 4.183 \times 10^4 \pi
\]

\[
\text{TotalCost}_{\text{liner}20} = 6.825 \times 10^4 \pi
\]

\[
\text{TotalCost}_{\text{noliner}20} = 5.577 \times 10^4 \pi
\]
Adams' Soils calculations. For Holding Volume and HRT checked by Nato

Total Volume = 5 \times 10^3 \times \text{m}^3

\text{Volume_{bed}} = 30 \times 20 \times 4 = 240 \text{ m}^3

\text{Total_{bed_volume}} = \text{Volume}_{bed} = 1.44 \times 10^3 \text{ m}^3

From my soils book, Page 59

\epsilon_{\text{loose_sand}} = 0.8

\epsilon_{\text{dense_sand}} = 0.45

V_{\text{solids}} = \frac{\text{Total_{bed_volume}}}{1 + \epsilon_{\text{loose_sand}}} = 8 \times 10^3 \text{ L}

V_{\text{voids}} = \text{Total_{bed_volume}} - V_{\text{solids}} = 640 \text{ m}^3

HRT := \frac{V_{\text{voids}}}{Q_{\text{noshool5}}} = 2.669 \text{ day}

V_{\text{solids dense}} = \frac{\text{Total_{bed_volume}}}{1 + \epsilon_{\text{dense_sand}}} = 9.93 \times 10^5 \text{ L}

V_{\text{voids dense}} = \text{Total_{bed_volume}} - V_{\text{solids dense}} = 446.897 \text{ m}^3

HRT_{\text{dense}} = \frac{V_{\text{voids dense}}}{Q_{\text{noshool5}}} = 1.864 \text{ day}
Results Summary

\[ Q_{\text{present}} = 221.184 \text{ m}^3 \text{ day}^{-1} \]
\[ Q_{\text{future 5}} = 2.826 \text{ L s}^{-1} \]
\[ Q_{\text{future 20}} = 3.804 \text{ L s}^{-1} \]

\[ Q_{\text{hoschool 1}} = 217.184 \text{ m}^3 \text{ day}^{-1} \]
\[ Q_{\text{hoschool 5}} = 239.789 \text{ m}^3 \text{ day}^{-1} \]
\[ Q_{\text{hoschool 20}} = 322.724 \text{ m}^3 \text{ day}^{-1} \]

\[ A_{h0} = 3.228 \times 10^3 \text{ m}^2 \]
\[ A_{h5} = 3.564 \times 10^3 \text{ m}^2 \]
\[ A_{h20} = 4.796 \times 10^3 \text{ m}^2 \]

\[ A_{\text{hoschool 0}} = 3.169 \times 10^3 \text{ m}^2 \]
\[ A_{\text{hoschool 5}} = 3.499 \times 10^3 \text{ m}^2 \]
\[ A_{\text{hoschool 20}} = 4.71 \times 10^3 \text{ m}^2 \]

\[ C_{\text{cell 0}} = 5.38 \]
\[ C_{\text{cell 5}} = 5.94 \]
\[ C_{\text{cell 20}} = 7.994 \]

\[ C_{\text{cell NS 0}} = 5.282 \]
\[ C_{\text{cell NS 5}} = 5.832 \]
\[ C_{\text{cell NS 20}} = 7.849 \]
Infiltration Considerations

IR = 0.125 \text{ in} \text{ min}^{-1}

Infiltration rate measured from percolation test by school

From the Onsite book, table 7-2

\[
\frac{1}{IR} = 8 \text{ min} \text{ in}^{-1}
\]

This gives it a qualification of Fine sand, loam sand

\[
\text{Application Rate} = 0.8 \text{ gal day}^{-1} \text{ ft}^2
\]

\[
\text{Application Rate} = 0.013 \text{ m day}^{-1}
\]

\[
A_{\text{nonschool5}} = 3.499 \times 10^3 \text{ m}^2
\]

\[
C_{\text{cellNS5}} = 5.832
\]

\[
\text{Cell Area} = 600 \text{ m}^2
\]

\[
\text{Infiltration} := \text{Application Rate} \times \text{cell}(C_{\text{cellNS5}}) \times \text{Cell Area}
\]

\[
\text{Infiltration} = 117.348 \text{ m}^3 \text{ day}^{-1}
\]

\[
Q_{\text{nonschool5}} = 239.789 \text{ m}^3 \text{ day}^{-1}
\]

\[
\frac{\text{Infiltration}}{Q_{\text{nonschool5}}} = 48.938\%
\]

Percent of flow infiltrating

\[
\text{Flow Per Bed} := \frac{Q_{\text{nonschool5}}}{6} = 39.965 \text{ m}^3 \text{ day}^{-1}
\]

\[
\text{Actual Application Rate} := \frac{\text{Flow Per Bed}}{\text{Cell Area}} = 1.635 \frac{\text{gal}}{\text{m}^2 \cdot \text{ day}}
\]
Appendix E

Septic Tank Calculations

Calculated by: Ian Compton

Septic Tank Sizing

Present Wastewater Flow:

\[ Q_p = \frac{1}{s} = 2.56 \text{ m}^3/\text{day} \]

Future Wastewater Flow (2% growth rate):

5 year outlook

\[ Q_{5y} = Q_p \left(1 + 0.02\right)^5 \]

\[ Q_{5y} = 244.205 \text{ m}^3/\text{day} \]

Hydraulic Retention Time:

\[ t = 1.5 \text{ day} \]

Septic Tank Volume:

\[ V_{total} = t \cdot Q_{5y} \]

\[ V_{total} = 366.308 \text{ m}^3 \]

number of tanks:

\[ n = 2 \]

\[ V_{tank} = \frac{V_{total}}{n} \]

\[ V_{tank} = 183.154 \text{ m}^3 \]
Two Compartment Septic Tank: guidelines for septic tank design: UN pdf

1st Compartment Volume: \[ V_1 = \frac{2}{3} V_{\text{tank}} \]
\[ V_1 = 122.103\text{-m}^3 \]

2nd Compartment Volume: \[ V_2 = \frac{1}{3} V_{\text{tank}} \]
\[ V_2 = 61.051\text{-m}^3 \]

Depth of Septic Tank:
\[ h := 2\text{m} \]

Width of Septic Tank:
\[ w := 6.7\text{m} \]

Length of Septic Tank:
\[ L_1 := \frac{V_1}{(h \cdot w)} \]
\[ L_1 = 9.112\text{m} \]
\[ L_2 := \frac{V_2}{(h \cdot w)} \]
\[ L_2 = 4.556\text{m} \]

Length-Width Ratio:
\[ LW > 2\text{ is OK} \]
\[ LW = 2.04 \]
HRT Check:

Sludge Accumulation Rate:

\[ \text{SAR} := 70 \frac{\text{L}}{\text{yr}} \text{ per person} \]

Desludging Interval:

\[ I = 1 \text{yr} \]

Sludge Volume:

\[ V_{\text{sludge}} := \frac{\text{SAR} \times 3000}{n} \]

\[ V_{\text{sludge}} = 105 \text{ m}^3 \]

Available Volume for wastewater in Septic Tank:

\[ V_{\text{available}} := V_{\text{tank}} - V_{\text{sludge}} \]

\[ V_{\text{available}} = 78.154 \text{ m}^3 \]

HRT after sludge accumulation:

\[ \theta_{\text{sludge}} := \frac{V_{\text{available}}}{Q_{\text{5}}/n} \]

\[ \theta_{\text{sludge}} = 0.64 \text{ day} \]

*If \( \theta_{\text{sludge}} > 0.5 \text{ days} \) the design is OK*
Septic Tank Sizing (Using Current Tank)

Present Wastewater Flow:
\[ Q_{\text{new}} := Q_p - Q_{\text{pschool}} \]

Future Wastewater Flow (2% growth rate):
\[ Q_{\text{new}} := Q_5 - Q_{\text{school}} \]
\[ Q_{\text{new}} = 238.261 \text{ m}^3/\text{day} \]

Hydraulic Retention Time:
\[ \theta_{\text{new}} := 1.5 \text{ day} \]

Septic Tank Volume:
\[ V_{\text{total new}} := \theta_{\text{new}} \cdot Q_{\text{new}} \]
\[ V_{\text{total new}} = 357.392 \text{ m}^3 \]

number of tanks:
\[ n_{\text{new}} := 2 \]
\[ V_{\text{tank new}} := \frac{V_{\text{total new}}}{n_{\text{new}}} \]
\[ V_{\text{tank new}} = 178.696 \text{ m}^3 \]
Two Compartment Septic Tank:

1st Compartment Volume: \( V_{1\text{new}} = V_{\text{tanknew}} \frac{2}{3} \)
\[ V_{1\text{new}} = 119.131 \text{ m}^3 \]

2nd Compartment Volume: \( V_{2\text{new}} = V_{\text{tanknew}} \frac{1}{3} \)
\[ V_{2\text{new}} = 59.565 \text{ m}^3 \]

Depth of Septic Tank: \( h_{\text{new}} := 2 \text{m} \)

Width of Septic Tank: \( w_{\text{new}} := 6.6 \text{m} \)

Length of Septic Tank:
\[ L_{1\text{new}} = \frac{V_{1\text{new}}}{(h_{\text{new}} - w_{\text{new}})} \]
\[ L_{1\text{new}} = 9.025 \text{ m} \]
\[ L_{2\text{new}} = \frac{V_{2\text{new}}}{(h_{\text{new}} - w_{\text{new}})} \]
\[ L_{2\text{new}} = 4.513 \text{ m} \]

Length-Width Ratio:
\[ LW_{\text{new}} := \frac{L_{1\text{new}} + L_{2\text{new}}}{w_{\text{new}}} \]
\[ LW_{\text{new}} = 2.051 \]
HRT Check:

Sludge Accumulation Rate:
\[ \text{SAR}_{\text{new}} := \frac{70}{\text{yr}} \]

Desludging Interval:
\[ t_{\text{new}} := \text{yr} \]

Sludge Volume:
\[ V_{\text{sludge}_{\text{new}}} := \frac{\text{SAR} \cdot L \cdot 3000}{n} \]
\[ V_{\text{sludge}_{\text{new}}} = 105 \text{ m}^3 \]

Available Volume for Wastewater in Septic Tank:
\[ V_{\text{available}_{\text{new}}} := V_{\text{tank}_{\text{new}}} - V_{\text{sludge}_{\text{new}}} \]
\[ V_{\text{available}_{\text{new}}} = 73.696 \text{ m}^3 \]

HRT after sludge accumulation:
\[ \theta_{\text{sludge}_{\text{new}}} := \frac{V_{\text{available}_{\text{new}}}}{Q_{\text{new}}} \]
\[ \theta_{\text{sludge}_{\text{new}}} = 0.619 \text{ day} \]

*if \( \theta_{\text{sludge}} > 0.5 \text{ days} \) the design is OK*
Leach Field For School

pg 214

Soil Texture: Fine sand, Loamy sand

Percolation Rate: 6-15 min/in

Application Rate:

\[ R_{\text{leach}} = 0.5 \frac{\text{gal}}{\text{day} \cdot \text{ft}^2} \]

School Flow:

\[ Q_{\text{school}} = 1.57 \times 10^3 \frac{\text{gal}}{\text{day}} \]

Require Leach Area:

\[ A_{\text{leach}} = \frac{Q_{\text{school}}}{R_{\text{leach}}} = 182.343 \text{ m}^2 \]

Available Area:

\[ A_{\text{available}} = 280 \text{ m}^2 \]
Third Septic Tank Sizing for Phase Plan

Remaining Wastewater Flow:

\[ Q_{20} = Q_p (1 + 0.02)^{20} = 3.804 \text{ L/s} \]

\[ Q_r = Q_{20} - Q_{15} = 0.978 \text{ L/s} \]

Third Septic Tank Volume:

\[ V_{tank_r} = Q_r \cdot 9 = 126.694 \text{ m}^3 \]

Two Compartment Septic Tank:

1st Compartment Volume:

\[ V_{1r} = \frac{V_{tank_r}}{3} \]

\[ V_{1r} = 84.463 \text{ m}^3 \]

2nd Compartment Volume:

\[ V_{2r} = \frac{V_{tank_r}}{3} \]

\[ V_{2r} = 42.231 \text{ m}^3 \]
Depth of Septic Tank:

Width of Septic Tank: \( w_r = 5.6 \text{m} \)

Length of Septic Tank:
\( L_{1t} = \frac{V_{1r}}{b_w} \)
\( L_{1t} = 7.541 \text{ m} \)
\( L_{2r} = \frac{V_{2r}}{b_w} \)
\( L_{2r} = 3.771 \text{ m} \)

Length-Width Ratio:
\( L/W_r = \frac{L_{1t} + L_{2r}}{w_r} \)
\( L/W_r = 2.02 \)
HRT Check:

Sludge Accumulation Rate:
\[ \text{SAR}_f = 70 \frac{L}{yr} \text{ per person} \]

Desludging Interval:
\[ \text{I}_r = 1 \text{yr} \]

Sludge Volume:
\[ V_{\text{sludger}} = \frac{\text{SAR}_f \cdot I_r \cdot 3000}{3} \]
\[ V_{\text{sludger}} = 70 \text{ m}^3 \]

Available Volume for wastewater in Septic Tank:
\[ V_{\text{available}} = V_{\text{tank}} - V_{\text{sludger}} \]
\[ V_{\text{available}} = 56.694 \text{ m}^3 \]

HRT after sludge accumulation:
\[ \theta_{\text{sludger}} = \frac{V_{\text{available}}}{Q_f} \]
\[ \theta_{\text{sludger}} = 0.671 \text{ day} \]

If \( \theta_{\text{sludger}} \) is \( > 0.5 \text{ days} \), the design is OK
Volume of Concrete

\[ V_{\text{wall1}} = (w + 2 \cdot h_{\text{wall}}) \cdot (h + 2 \cdot h_{\text{wall}}) = 2.88 \cdot \text{m}^3 \]

\[ V_{\text{wall2}} = (l_{1r} + L_{2r}) \cdot h_{\text{wall}} \cdot (h + 2 \cdot h_{\text{wall}}) = 5.43 \cdot \text{m}^3 \]

\[ V_{\text{wall3}} = t_{\text{wall}} \cdot w_r \cdot (l_{1r} + L_{2r}) = 12.669 \cdot \text{m}^3 \]

\[ V_{\text{wall4}} = t_{\text{wall}} \cdot h \cdot w_r = 2.24 \cdot \text{m}^3 \]

\[ V_{\text{baffler1}} = t_{\text{wall}} \cdot 0.8 \cdot w_r = 0.896 \cdot \text{m}^3 \]

\[ V_{\text{baffler2}} = t_{\text{wall}} \cdot 1 \cdot w_r = 1.12 \cdot \text{m}^3 \]

\[ V_{\text{concrete}} = 2 \cdot V_{\text{wall1}} + 2 \cdot V_{\text{wall2}} + 2 \cdot V_{\text{wall3}} + V_{\text{wall4}} + V_{\text{baffler1}} + V_{\text{baffler2}} = 46.214 \cdot \text{m}^3 \]

\[ V_{\text{concretetotal}} = V_{\text{concrete}} = 46.214 \cdot \text{m}^3 \]
Sizing With Current Septic Tank

Septic Tank Volume: \( V_{\text{current}} = 105 \text{m}^3 \) 

Based on measurements taken of tank while in Ecuador.

Treatable Flow:

\[
Q_{\text{pschool}} = 4 \frac{\text{m}^3}{\text{day}} = 0.046 \frac{\text{L}}{\text{s}}
\]

\[
Q_{\text{pschool}} = Q_{\text{pschool}} (1 + 0.02)^{30} = 0.069 \frac{\text{L}}{\text{s}}
\]

% Of Wastewater Flow that is Treatable:

\[
\%_{\text{total}} = \frac{V_{\text{current}}}{V_{\text{total}}} \times 100 = 29.756
\]

Sized to Treat:

\[
Q_{\text{current}} = \frac{V_{\text{current}}}{6} = 72.667 \frac{\text{m}^3}{\text{day}}
\]

This is the amount of flow.
Volume of Concrete

Wall Thickness: \( t_{\text{wall}} \geq 0.2 \text{m} \)

\[
V_{\text{wall1}} = (w + 2t_{\text{wall}}) (h + 2t_{\text{wall}}) = 3.408 \text{ m}^3
\]

\[
V_{\text{wall2}} = (L_1 + L_2) t_{\text{wall}} (h + 2t_{\text{wall}}) = 6.561 \text{ m}^3
\]

\[
V_{\text{wall3}} = t_{\text{wall}} w (L_1 + L_2) = 18.315 \text{ m}^3
\]

\[
V_{\text{wall4}} = t_{\text{wall}} b w = 2.68 \text{ m}^3
\]

\[
V_{\text{buffer1}} = t_{\text{wall}} 0.8m w = 1.072 \text{ m}^3
\]

\[
V_{\text{buffer2}} = t_{\text{wall}} 1m w = 1.34 \text{ m}^3
\]

\[
V_{\text{concrete}} = 2V_{\text{wall1}} + 2V_{\text{wall2}} + 2V_{\text{wall3}} + V_{\text{wall4}} + V_{\text{buffer1}} + V_{\text{buffer2}} = 61.66 \text{ m}^3
\]

\[
V_{\text{concretetotal}} := V_{\text{concrete}} = 123.32 \text{ m}^3
\]
Volume of Concrete

\[ V_{\text{wall new 1}} := (w_{\text{new}} + 2 \cdot t_{\text{wall}}) \cdot t_{\text{wall}} \cdot (h_{\text{new}} + 2 \cdot t_{\text{wall}}) = 3.36 \text{ m}^3 \]

\[ V_{\text{wall new 2}} := (1 \cdot t_{\text{wall}} + 1 \cdot t_{\text{wall}}) \cdot t_{\text{wall}} \cdot (h_{\text{new}} + 2 \cdot t_{\text{wall}}) = 6.498 \text{ m}^3 \]

\[ V_{\text{wall new 3}} := t_{\text{wall}} \cdot h_{\text{new}} \cdot (1 \cdot t_{\text{wall}} + 1 \cdot t_{\text{wall}}) = 17.87 \text{ m}^3 \]

\[ V_{\text{wall new 4}} := t_{\text{wall}} \cdot h_{\text{new}} \cdot w_{\text{new}} = 2.64 \text{ m}^3 \]

\[ V_{\text{baffle new 1}} := t_{\text{wall}} \cdot 0.8 \cdot w_{\text{new}} = 1.056 \text{ m}^3 \]

\[ V_{\text{baffle new 2}} := t_{\text{wall}} \cdot 1 \cdot w_{\text{new}} = 1.32 \text{ m}^3 \]

\[ V_{\text{concrete new}} := 2 \cdot V_{\text{wall new 1}} + 2 \cdot V_{\text{wall new 2}} + 2 \cdot V_{\text{wall new 3}} + V_{\text{wall new 4}} + V_{\text{baffle new 1}} + V_{\text{baffle new 2}} = 60.471 \text{ m}^3 \]

\[ V_{\text{concrete total new}} = V_{\text{concrete new}} + n_{\text{new}} = 120.942 \text{ m}^3 \]
Appendix F

Waste Stabilization Pond Calculations

Calculated by: Adam DeYoung

Pond Calculations

Primary source
WHO pond design manual. Main guidelines from Myra

Secondary source
Modern Design of Waste Stabilization Ponds in Warm Climates: Comparison with Traditional Design Methods by Chimwemwe Banda Garaaari

Population

\[ P_{\text{present}} := 2540 \]

\[ P_5 := P_{\text{present}} (1 + .02)^5 = 2.804 \times 10^3 \]

\[ P_{20} := P_{\text{present}} (1 + .02)^{20} = 3.774 \times 10^3 \]

\[ Q_{\text{in}} := 2.557 \text{ L/s} \]

Flows

\[ Q_{\text{w,c}} := \frac{Q_{\text{in}}}{P_{\text{present}}} = \frac{2.558}{2540} = 0.001 \text{ L/s} \]

\[ Q_{\text{w,s}} := \frac{Q_{\text{in}}}{P_5} = \frac{2.824}{2540} = 0.001 \text{ L/s} \]

\[ Q_{\text{w,20}} := \frac{Q_{\text{in}}}{P_{20}} = \frac{3.801}{2540} = 0.001 \text{ L/s} \]

Assumed Peak flow because measured at peak time.

\[ \text{BOD entering} := 300 \text{ mg/L} \]

\[ L_{\text{in}} = 300 \text{ mg/L} \]

BOD
Average Temperature \( T_{\text{ave}} = 16 \) based on a neighboring city

**Anaerobic Pond**

Volumetric Loading: \( \lambda = \frac{L_{\text{g}} - T_{\text{ave}}}{m^3} = 220 \frac{\text{gm}}{\text{day}} \frac{\text{day}}{m^3} \) and from table 3.1

\[
V_{\text{anaerobic, c}} = \frac{L_{\text{g}, c} Q_{\text{w, c}}}{\lambda} = 301.355 \text{ m}^3
\]

\[
V_{\text{anaerobic, 5}} = \frac{L_{\text{g}, 5} Q_{\text{w, 5}}}{\lambda} = 332.721 \text{ m}^3
\]

\[
V_{\text{anaerobic, 20}} = \frac{L_{\text{g}, 20} Q_{\text{w, 20}}}{\lambda} = 447.798 \text{ m}^3
\]

**Hydraulic Retention Time:**

\[
\theta_{\text{c, c}} = \frac{V_{\text{anaerobic, c}}}{Q_{\text{w, c}}} = 1.364 \text{ day}
\]

\[
\theta_{\text{c, 5}} = \frac{V_{\text{anaerobic, 5}}}{Q_{\text{w, 5}}} = 1.364 \text{ day}
\]

\[
\theta_{\text{c, 20}} = \frac{V_{\text{anaerobic, 20}}}{Q_{\text{w, 20}}} = 1.364 \text{ day}
\]

**Depth of pond** \( h_a = 3 \text{ m} \)
Surface Area

\[ A_{\text{anaerobic c}} = \frac{V_{\text{anaerobic c}}}{h_a} = 100.452 \text{ m}^2 \]

\[ A_{\text{anaerobic 5}} = \frac{V_{\text{anaerobic 5}}}{h_a} = 110.907 \text{ m}^2 \]

\[ A_{\text{anaerobic 20}} = \frac{V_{\text{anaerobic 20}}}{h_a} = 149.266 \text{ m}^2 \]

Percent BOD removal

\[ \text{BOD}_{\text{anaerobic removal}} = \frac{2T_{\text{ave}} + 20}{100} - 0.52 \]

\[ L_{\text{L-f}} = L_e \cdot \text{BOD}_{\text{anaerobic removal}} = 156 \text{ mg/L} \]

Facultative Pond

Surface Loading:

\[ \lambda_g = 350 \left( 1.107 - 0.002T_{\text{ave}}\right)^{T_{\text{ave}} - 20} = 262.08 \]

Surface Area

\[ A_{\text{facultative c}} = \frac{10L_f Q_{W, c}}{\lambda_s \text{ kg/hectare-day}} = 1.315 \times 10^4 \text{ m}^2 \quad A_{\text{facultative c}} = 3.251 \text{ acre} \]

\[ A_{\text{facultative 5}} = \frac{10L_f Q_{W, 5}}{\lambda_s \text{ kg/hectare-day}} = 1.452 \times 10^4 \text{ m}^2 \quad A_{\text{facultative 5}} = 3.589 \text{ acre} \]

\[ A_{\text{facultative 20}} = \frac{10L_f Q_{W, 20}}{\lambda_s \text{ kg/hectare-day}} = 1.955 \times 10^4 \text{ m}^2 \quad A_{\text{facultative 20}} = 4.83 \text{ acre} \]
Depth of Pond

\( h_f := 1.5 \text{m} \)

Hydraulic Residence Time

\[
\theta_C := \frac{A_{\text{facultative}, c} h_f}{Q_{w, c}} = 89.286 \text{-day}
\]

\[
\theta_{5} := \frac{A_{\text{facultative}, 5} h_f}{Q_{w, 5}} = 89.286 \text{-day}
\]

\[
\theta_{20} := \frac{A_{\text{facultative}, 20} h_f}{Q_{w, 20}} = 89.286 \text{-day}
\]

Method One for BOD removal

\[
L_{2c, f, c} := 3 \cdot L_{f, f} = 46.8 \frac{\text{mg}}{\text{L}}
\]

\[
L_{2c, f, 5} := 3 \cdot L_{f, f} = 46.8 \frac{\text{mg}}{\text{L}}
\]

\[
L_{2c, f, 20} := 3 \cdot L_{f, f} = 46.8 \frac{\text{mg}}{\text{L}}
\]

Method Two for BOD removal

\[
\lambda_f = \left(10.75 + 0.725 \lambda_g\right) \cdot \frac{\text{kg}}{\text{hectare-day}} = 2.324 \times 10^{-7} \frac{\text{kg}}{\text{m}^2 \text{s}}
\]

BOD inflow

\[
M_{L, \text{BOD}, f, c} := L_{f, f} Q_{w, c} = 399.017 \frac{\text{mg}}{\text{s}}
\]

\[
M_{L, \text{BOD}, f, 5} := L_{f, f} Q_{w, 5} = 440.547 \frac{\text{mg}}{\text{s}}
\]

\[
M_{L, \text{BOD}, f, 20} := L_{f, f} Q_{w, 20} = 392.918 \frac{\text{mg}}{\text{s}}
\]
BOD removal rate

\[ M_{\text{remove}_c} := (A_{\text{facultative}_c} \lambda_q) = 3.057 \times 10^3 \text{ mg} \text{/s} \]

\[ M_{\text{remove}_5} := (A_{\text{facultative}_5} \lambda_q) = 3.375 \times 10^3 \text{ mg} \text{/s} \]

\[ M_{\text{remove}_20} := (A_{\text{facultative}_20} \lambda_q) = 4.542 \times 10^3 \text{ mg} \text{/s} \]

BOD left

\[ I_{w,f,c} := \frac{M_{i,BOD,f,c} - M_{\text{remove}_c}}{Q_{w,c}} = -1.039 \times 10^3 \text{ mg} \text{/L} \]

\[ I_{w,f,5} := \frac{M_{i,BOD,f,5} - M_{\text{remove}_5}}{Q_{w,5}} = -1.039 \times 10^3 \text{ mg} \text{/L} \]

\[ I_{w,f,20} := \frac{M_{i,BOD,f,20} - M_{\text{remove}_20}}{Q_{w,20}} = -1.039 \times 10^3 \text{ mg} \text{/L} \]

Costing

\[ A_{\text{total}_c} := A_{\text{anaerobic}_c} + A_{\text{facultative}_c} = 1.329 \times 10^4 \text{ m}^2 \]

\[ A_{\text{total}_5} := A_{\text{anaerobic}_5} + A_{\text{facultative}_5} = 1.463 \times 10^4 \text{ m}^2 \]

\[ A_{\text{total}_20} := A_{\text{anaerobic}_20} + A_{\text{facultative}_20} = 1.97 \times 10^4 \text{ m}^2 \]

\[ A_{\text{total}_c} = 3.775 \text{ acre} \]

\[ A_{\text{total}_5} = 3.616 \text{ acre} \]

\[ A_{\text{total}_20} = 4.867 \text{ acre} \]

\[ \text{Cost}_{\text{liner}} := \frac{2}{m^2} \]

\[ h_{\text{clay}} = 1 \text{ ft} = 0.305 \text{ m} \]

\[ \text{Cost}_{\text{clay}} := \frac{16.25}{m^3} \]
LIII

Checked by L.C

\[ \text{Cost}_{\text{anaerobic L, c}} := \text{Cost}_{\text{liner A, anaerobic, c}} 1.3 = 261.175 \]

\[ \text{Cost}_{\text{anaerobic L, s}} := \text{Cost}_{\text{liner A, anaerobic, s}} 1.3 = 288.358 \]

\[ \text{Cost}_{\text{anaerobic L, 20}} := \text{Cost}_{\text{liner A, anaerobic, 20}} 1.3 = 388.092 \]

\[ \text{Cost}_{\text{facultative L, c}} := \text{Cost}_{\text{liner A, facultative, c}} 1.3 = 3.42 \times 10^4 \]

\[ \text{Cost}_{\text{facultative L, s}} := \text{Cost}_{\text{liner A, facultative, s}} 1.3 = 3.776 \times 10^4 \]

\[ \text{Cost}_{\text{facultative L, 20}} := \text{Cost}_{\text{liner A, facultative, 20}} 1.3 = 5.082 \times 10^4 \]

Total cost using plastic liner

\[ \text{Cost}_{\text{total L, c}} := \text{Cost}_{\text{liner A, total, c}} 1.3 = 3.446 \times 10^4 \]

\[ \text{Cost}_{\text{total L, s}} := \text{Cost}_{\text{liner A, total, s}} 1.3 = 3.805 \times 10^4 \]

\[ \text{Cost}_{\text{total L, 20}} := \text{Cost}_{\text{liner A, total, 20}} 1.3 = 5.121 \times 10^4 \]

\[ V_{\text{clay, c}} := A_{\text{total, c}} h_{\text{clay}} \approx 4.04 \times 10^3 \text{ m}^3 \]

\[ V_{\text{clay, s}} := A_{\text{total, s}} h_{\text{clay}} \approx 4.461 \times 10^3 \text{ m}^3 \]

\[ V_{\text{clay, 20}} := A_{\text{total, 20}} h_{\text{clay}} \approx 6.003 \times 10^3 \text{ m}^3 \]

Total cost using clay liner

\[ \text{Cost}_{\text{total L, c}} := \text{Cost}_{\text{clay A, total, c}} \approx 8.535 \times 10^4 \]

\[ \text{Cost}_{\text{total L, s}} := \text{Cost}_{\text{clay A, total, s}} \approx 9.423 \times 10^4 \]

\[ \text{Cost}_{\text{total L, 20}} := \text{Cost}_{\text{clay A, total, 20}} \approx 1.268 \times 10^5 \]
Appendix G

Bar Screen Sizing

Calculations by: Ian Compton

Design Flow:

\[ Q_d = 238.3 \text{ m}^3\text{ day}^{-1} \]

Target Velocity:

\[ V_t = 0.6 \text{ m sec}^{-1} \]

Channel Cross Section:

\[ A_c \equiv \frac{Q_d}{V_t} = 4.597 \times 10^{-3} \text{ m}^2 \]

Depth:Width Ratio \( \rightarrow 1.5 \)

\[ W_c = \frac{A_c}{1.5} = 0.055 \text{ m} \]

\[ D_c = 1.5 W_c = 0.083 \text{ m} \]

*Flow too small for typical channel design.
Appendix H
Residuals Disposal

Residuals Disposal

Calculated by: Nathan Williams
Checked by: Ian Compton

**Sludge Accumulation Rate:**

\[ \text{SAR}_{\text{new}} := 70 \frac{\text{L}}{\text{yr}} \]

\[ \text{SAR}_{\text{new}} = 0.07 \frac{\text{m}^3}{\text{yr}} \]

**Desludging Interval:**

\[ I_{\text{new}} := \text{byr} \]

**Sludge Volume:**

\[ V_{\text{sludge}_{\text{new}}} := \text{SAR}_{\text{new}} I_{\text{new}} \times 3000 \]

\[ V_{\text{sludge}_{\text{new}}} = 210 \text{ m}^3 \]

105 cubic meters of sludge accumulate per year. Composting can be done in month batches.

\[ V_{\text{sludge}_{\text{month}}} = \frac{V_{\text{sludge}_{\text{new}}}}{12} = 17.5 \text{ m}^3 \]

Compost piles have an approx cross section of 1m x 1m. This would require about 9m in length of compost rows.
### Appendix I

#### Detailed Costs

<table>
<thead>
<tr>
<th>Phasing Plan</th>
<th>Diameter</th>
<th>Component</th>
<th>Length (m)/Quantity</th>
<th>Unit Cost ($)</th>
<th>Total Cost ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>5 Year</strong></td>
<td>12cm</td>
<td>Pipe</td>
<td>91.5</td>
<td>6.00</td>
<td>549.00</td>
</tr>
<tr>
<td></td>
<td>12cm</td>
<td>90deg elbow</td>
<td>2</td>
<td>25.00</td>
<td>50.00</td>
</tr>
<tr>
<td></td>
<td>12cm</td>
<td>T-joint</td>
<td>4</td>
<td>15.00</td>
<td>60.00</td>
</tr>
<tr>
<td></td>
<td>12cm</td>
<td>Shutoff valve</td>
<td>0</td>
<td>30.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>15cm</td>
<td>Pipe</td>
<td>55.5</td>
<td>7.00</td>
<td>388.50</td>
</tr>
<tr>
<td></td>
<td>15cm</td>
<td>90deg elbow</td>
<td>1</td>
<td>25.00</td>
<td>25.00</td>
</tr>
<tr>
<td></td>
<td>15cm</td>
<td>T-joint</td>
<td>1</td>
<td>25.00</td>
<td>25.00</td>
</tr>
<tr>
<td></td>
<td>15cm</td>
<td>Shutoff valve</td>
<td>0</td>
<td>60.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>20cm</td>
<td>Pipe</td>
<td>389.1</td>
<td>9.55</td>
<td>3715.91</td>
</tr>
<tr>
<td></td>
<td>20cm</td>
<td>90deg elbow</td>
<td>10</td>
<td>65.00</td>
<td>650.00</td>
</tr>
<tr>
<td></td>
<td>20cm</td>
<td>T-joint</td>
<td>4</td>
<td>86.00</td>
<td>344.00</td>
</tr>
<tr>
<td></td>
<td>20cm</td>
<td>Shutoff valve</td>
<td>8</td>
<td>80.00</td>
<td>640.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Total: 6447.41</td>
</tr>
<tr>
<td><strong>20 Year</strong></td>
<td>12cm</td>
<td>Pipe</td>
<td>56.6</td>
<td>6.00</td>
<td>339.60</td>
</tr>
<tr>
<td></td>
<td>12cm</td>
<td>90deg elbow</td>
<td>1</td>
<td>25.00</td>
<td>25.00</td>
</tr>
<tr>
<td></td>
<td>12cm</td>
<td>T-joint</td>
<td>2</td>
<td>15.00</td>
<td>30.00</td>
</tr>
<tr>
<td></td>
<td>12cm</td>
<td>Shutoff valve</td>
<td>0</td>
<td>30.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>15cm</td>
<td>Pipe</td>
<td>0</td>
<td>7.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>15cm</td>
<td>90deg elbow</td>
<td>0</td>
<td>25.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>15cm</td>
<td>T-joint</td>
<td>0</td>
<td>25.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>15cm</td>
<td>Shutoff valve</td>
<td>0</td>
<td>60.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>20cm</td>
<td>Pipe</td>
<td>132.2</td>
<td>9.55</td>
<td>1262.51</td>
</tr>
<tr>
<td></td>
<td>20cm</td>
<td>90deg elbow</td>
<td>8</td>
<td>65.00</td>
<td>520.00</td>
</tr>
<tr>
<td></td>
<td>20cm</td>
<td>T-joint</td>
<td>1</td>
<td>86.00</td>
<td>86.00</td>
</tr>
<tr>
<td></td>
<td>20cm</td>
<td>Shutoff valve</td>
<td>4</td>
<td>80.00</td>
<td>320.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Total: 2583.11</td>
</tr>
</tbody>
</table>

**Total Combined:** 9030.52
Table 31: Septic Tank Costs

<table>
<thead>
<tr>
<th>Phasing Plan</th>
<th>Volume of Concrete (m³)</th>
<th>Cost of Concrete: ($/m³)</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 Year</td>
<td>147.984</td>
<td>$120</td>
<td>$17,758.08</td>
</tr>
<tr>
<td>20 Year</td>
<td>55.452</td>
<td>$120</td>
<td>$6,654.24</td>
</tr>
<tr>
<td>Total</td>
<td>203.436</td>
<td></td>
<td>$24,412.32</td>
</tr>
</tbody>
</table>

Table 32: Wetland Liner vs. No Liner Comparison

<table>
<thead>
<tr>
<th>Wetland Cells</th>
<th>Cost (with Liner) $</th>
<th>Cost (Without Liner) $</th>
</tr>
</thead>
<tbody>
<tr>
<td>5-Year</td>
<td>6</td>
<td>42630</td>
</tr>
<tr>
<td>20-Year</td>
<td>8</td>
<td>63560</td>
</tr>
</tbody>
</table>

Table 33: Constructed Wetland Costs

<table>
<thead>
<tr>
<th>Phasing Plan</th>
<th>Number of Cells</th>
<th>Volume of Fill per cell (m³)</th>
<th>Total Volume of Fill (m³)</th>
<th>Cost of Material ($/m³)</th>
<th>Total Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 Year</td>
<td>6</td>
<td>330</td>
<td>1980</td>
<td>16.25</td>
<td>$38,610.00</td>
</tr>
<tr>
<td>20 Year</td>
<td>2</td>
<td>330</td>
<td>660</td>
<td>16.25</td>
<td>$12,870.00</td>
</tr>
<tr>
<td>Total</td>
<td>8</td>
<td>330</td>
<td>2640</td>
<td>16.25</td>
<td>$51,480</td>
</tr>
</tbody>
</table>
Works Cited


“Gravity Sanitary Sewer Design and Construction”. ASCE Manuals and Reports on Engineering Practice-No. 60.


Mara, D. Domestic Wastewater Treatment in Developing Countries. London: Earthscan Publications, 2004. Internet resource


Web Sites


http://weatherspark.com/history/33550/2012/Quito-Pichincha-Ecuador

www.platictrends.com