Regional Factors Governing Performance and Sustainability of Wastewater Treatment Plants in Honduras: Lake Yojoa Subwatershed

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Abstract

Lake Yojoa, the largest natural lake in Honduras, is currently experiencing eutrophication from overloading of nutrients, in part due to inadequate wastewater treatment throughout the Lake Yojoa Subwatershed. Some efforts are being made to address the issue of wastewater treatment, but they frequently suffer from lack of oversight and follow-through. As a case study, in 2008, a wastewater treatment plant was built to treat sewage from Las Casetas, a group of restaurants and small homes on the southeastern shore of the lake. However, the piping used to connect the casetas to the treatment plant was not built to specification and subsequently failed, and the project was essentially abandoned.

The aim of this study was to determine whether the treatment plant and piping system at Las Casetas were adequately designed in order to provide recommendations for rehabilitating the system. A full analysis of the system suggests that, with the exception of a few minor changes, both the treatment plant and piping system were well-designed and, in the case of the treatment plant, well-constructed. With funding from PRONADERS (the Program for National Sustainable Rural Development) and involvement from the association of fish restaurant owners at Las Casetas, we propose that rehabilitation of the system move forward.

Rehabilitation of the Las Casetas treatment plant will provide two major benefits to the community. First, it will decrease the nutrient loading to Lake Yojoa, which will be an important first step in solving the eutrophication problem. Residents of the region are well aware of this ecological issue, as local fish farmers are currently being employed to remove water lilies from the lake. These aquatic plants have grown in excess in recent years due to eutrophication.

Secondly, use of a wastewater treatment system will improve public perceptions of the cleanliness of the fish restaurants, whose business has suffered due to negative perceptions of the quality of the lake. These perceptions are also due, in part, to pollution from other sources, including Aquafinca, a tilapia farming cooperation, and agricultural practices in the region. We therefore also propose that a full nutrient budget of the lake be carried out, with input from all sources of pollution, in order to determine where the main causes of the problem lie and how best to proceed to reduce ecological damage.

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Our heartfelt thanks also extend to the Eveline family: Luis, Carolina, and José, for their generous hospitality and their contributions to our learning, during our time in Honduras and beyond.

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Chapter 1: Introduction

1.1 Project Background

1.1.1 Purpose of Study

In 2006, the Massachusetts Institute of Technology (MIT) Master of Engineering (MEng) program in environmental and water quality engineering began sending students to Honduras to make water quality assessments and provide guidance for the nation's leaders in terms of improving the state of sanitation, particularly in the Lake Yojoa region. During the first year, Tia Trate and Mira Chokshi performed a nutrient load and temperature analysis of Lake Yojoa in order to quantify the impact of local practices, including wastewater disposal, agriculture, aquaculture, and mining, on the water quality in the lake (Chokshi and Trate, 2006). The 2008 team, consisting of Anne Mikelonis and Matthew Hodge, investigated the potential for improving wastewater treatment in the municipality of Las Vegas, Honduras, as one means of improving the quality of Lake Yojoa. Las Vegas is a contributor of pollution to the lake, as the municipality's wastewater treatment facilities are largely ineffective and wastewater flows virtually untreated into the lake (Hodge and Mikelonis, 2008). Finally, in 2009, Mahua Bhattacharya, Lisa Kullen, and Robert McLean evaluated a variety of wastewater treatment options for Honduras, in an effort to contribute to the Strategic Plan for Modernization of the Honduran Water and Sanitation Sector (PEMAPS) (Bhattacharya et al., 2009).

While the efforts of the three previous MEng teams have provided a wealth of valuable information, little has actually changed in recent years in terms of the state of wastewater facilities, particularly in the poorer rural communities of Honduras. For three weeks during the month of January 2011, we, the authors, continued the investigation of wastewater treatment in Honduras that began five years ago. The purpose of our investigation was to assess current treatment facilities, identify regional factors governing their performance, and put forth recommendations for improving wastewater treatment. After an initial exploration of wastewater treatment throughout the Lake Yojoa Subwatershed, consisting largely of meetings with various stakeholders as well as site visits to several treatment plants, we decided to focus our work mainly on the rehabilitation of the Las Casetas treatment plant, located in the town of Playas de María on the southeastern shore of the lake.

1.1.2 Team Structure

This project was a result of much collaboration between MIT and *Universidad Politécnica de Ingeniería* (UPI), the Polytechnic Engineering University based in Tegucigalpa, Honduras. During our three weeks in Honduras, an official research partnership between MIT and UPI was established, which will hopefully continue long into the future. The faculty, staff and students of UPI served as an invaluable resource to our research.

E. Eric Adams, PhD., of MIT, served as the program manager for this project, responsible for the overall execution of the project and ensuring that the required work was done right and on time. Aridai Herrera, MSc., an MIT affiliate, filled the position of project manager; his duties including monitoring progress on all tasks associated with the project, as well as serving as a point of contact between RAS-HON (the Honduran Network of Water and Sanitation, described in Section 1.3.2.7) and the research team. We, the authors, rounded out the MIT portion of the team.

Luis Eveline, MSc., the project coordinator and Rector of UPI, provided a great deal of assistance to the research team, coordinating research efforts and periodic meetings. His contributions also included coordinating site visits with UPI students. Several UPI students assisted us in collecting data, taking measurements, and interacting with the local people.

1.2 Background: Honduras

Honduras is the second largest country in Central America, with a population of about 7.5 million. It is also one of the poorest, with 65 percent of its population living on less than two dollars a day and a per capita GDP of only $1,635 (Bhattacharya et al, 2009). Given the level of poverty in the country, wastewater sanitation is inadequate in most places. Based on a study conducted by Water For People in 2006, less than half of the population living in rural areas had access to sanitation in 2001; see Table 1.1. Furthermore, many of the wastewater treatment facilities in Honduras are in a state of disrepair. Because of the lack of sanitation, waterborne diseases are a major threat to the nation's population, primarily children, who are most susceptible.

Table 1.1: Sanitation Coverage in Honduras as of 2001
(Water for People, 2006)

Population Group	2001 Population	Population with sewerage service	Population with latrines	Total population served	Coverage %
Rural	3,113,304	-	1,541,085	1,541,085	49.5
Urban	2,895,776	1,538,440	1,006,947	2,545,387	87.9
Global	6,009,080	1,538,440	2,548,032	4,086,472	68.0

These statistics have improved since 2001. According to RAS-HON, more recent assessments indicate that about 92% of urban communities, 67% of concentrated rural communities, and 52% of dispersed rural communities now have access to improved sanitation; see Table 1.2. Overall, 88% of Honduras currently has sanitation coverage, although it is important to note that coverage does not imply treatment (Garcia, 2011).

Table 1.2: Percentage of Honduran Population with Access to Improved Sanitation
(Garcia, 2011)

Type of community	Number of residents per community	Number of communities in Honduras	Percentage with access to improved sanitation
Urban	> 2,000	~100	92
Concentrated rural	200-2,000	~5,700	67
Dispersed rural	< 200	~22,000	52

According to Water For People, of the 26,341 settlements (ranging from metropolises to families) throughout the country, only 4,917 communities have a conventional water system with household taps, and even these systems do not necessarily provide water to every family in the community. A study conducted by the Pan American Health Association (PAHO) Collaborative Council for Water and Sanitation found that in 2001, 76.9% of the population of Honduras had some form of water coverage, which comprised 70% of the rural and 90% of the urban population. However, 90% of this water supply was found to be intermittent, only 44% of the water supplied was effectively disinfected, and water quality control and monitoring was severely lacking (Water For People, 2006).

In September 2000, Honduras was one of 189 countries that convened and signed the Millennium Declaration, committing to numerous goals that would contribute to creating a more just and equitable world for all, with measureable targets in place for 2015. One of the Millennium Development Goals (MDGs) established, Target 7C, was to halve the proportion of the population without sustainable access to safe drinking water and basic sanitation. Honduras has already made great strides towards reaching this goal, with the proportion of the national population with access to improved water sources having increased from 76.9% in 1990 to 86.1% in 2009. The target for 2015 is 88.5% (United Nations System in Honduras, 2010). Improving wastewater treatment throughout Honduras will help the country to reach this goal, and to make sure that access to safe water is sustainable.

1.3 Study Site: Lake Yojoa and the Subwatershed

The project described herein took place in and around Lake Yojoa and its subwatershed, a region of vital importance to local communities due to its impact on the fishing industry, agricultural practices, and day to day life of local populations. Lake Yojoa is the largest natural lake in Honduras and is located in the western region of the country, as indicated in Figure 1.1.

The Lake Yojoa Subwatershed covers an area of approximately 44,000 hectares, which includes nine municipalities, 53 communities, and a population of approximately 75,000. Itself a part of the Ulua River Watershed, the Subwatershed is further divided into 12 microwatersheds, as shown in Figure 1.2. Of these, the Quebrada El Cianuro microwatershed is the largest, as it contains the cities of El Mochito and Las Vegas and approximately 29% of the total population of the Subwatershed. The Ulua River Watershed was recently identified by RAS-HON (the Honduran Water and Sanitation Network, described in Section 1.3.2.7) as one of four watersheds in Honduras of primary concern due to increased loading of nutrients, partially due to the influx of untreated wastewater to the watershed (Garcia, 2011).

Figure 1.1: Map of Honduras, with Location of Study Site, Lake Yojoa (Lago de Yojoa)
(Central Intelligence Agency, 2011)

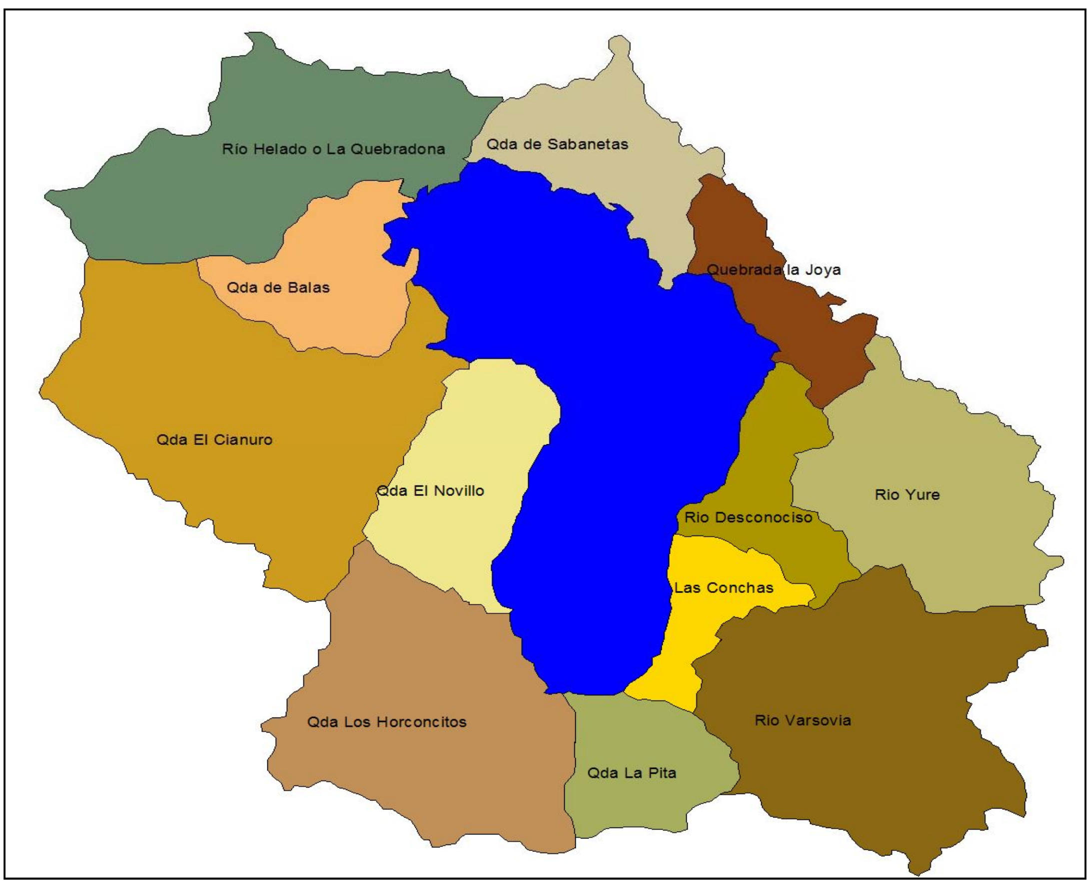

Figure 1.2: Microwatersheds of the Lake Yojoa Subwatershed
(Herrera, 2010)

1.3.1 Eutrophication of Lake Yojoa

The heavy nutrient load to Lake Yojoa has led to eutrophication throughout the lake. Eutrophication is the result of increased productivity in a water body, characterized by excessive growth of aquatic plant life. The driving force behind eutrophication is increased availability of the limiting nutrients essential to plant growth, usually nitrogen and phosphorus. Shallow lakes are particularly vulnerable to nutrient enrichment due to conversion of lowlands to agricultural or urban land. Overall, domestic sewage, agricultural practices, and industry are major contributors to eutrophication, the key negative impact on lakes worldwide (Ansari et al, 2011).

Eutrophication interferes with natural nutrient cycles and can lead to drastic biological changes and corresponding negative effects on ecological systems (Ansari et al, 2011). In the case of Lake Yojoa, this is seen most visibly in the increased growth of what the locals refer to as water lilies, as well as decreased transparency due to increased turbidity, potentially a result of phytoplankton growth. Both of these effects are shown in Figure 1.3.

Figure 1.3: Growth of Water Lilies (Left) and Discoloration of Water in the Lake (Right)
(Walker, 2011)

In her thesis work, MEng student Tia Trate identified many of the point and non-point sources of nutrient loading to the lake. Point sources are largely wastewater discharged from Las Casetas as well as the municipality of Las Vegas, while non-point sources include groundwater infiltration and urban, agricultural, and forest runoff. Other sources, not classified as either point or non-point, are fish farms, including Aquafinca (see Section 1.3.2.1), and use of detergents to wash clothes in the rivers and streams flowing into the lake. In her analysis, Trate concluded that fish farms were a major contributor of both nitrogen and phosphorus to the lake (Trate, 2006). However, this analysis should be considered preliminary, given the many assumptions that were made and the complications in assessing pollution from non-point sources. A complete nutrient load analysis, with contribution and cooperation from each stakeholder and improved data, particularly from non-point sources such as agricultural practices, should be performed in order to determine how best to proceed.

1.3.2 Stakeholders

There is a wide variety of stakeholders throughout the Subwatershed – governmental and non-governmental organizations and agencies, companies, and people. This section describes some of the primary players and Figure 1.4 indicates the locations of some of these stakeholders.

1.3.2.1 Aquafinca

Aquafinca is a tilapia farming operation based in Borboton and owned by Regal Springs Tilapia. The company is a world leader in tilapia production and employs around 1,600 people, many of whom are native Hondurans; this number represents significant recent growth. Aquafinca is generally considered a model company among business leaders, largely due to its positive interactions with its employees; there are no labor unions, just a good system and good relationships, where employees are generally given more than is guaranteed by law, according to Aquafinca's CEO, Israel Snir. They have even received requests from other companies to visit and explain how they interact with their employees, since it has proven to be effective and productive (Snir, 2011). As the company's slogan goes, "It's not about the fish, it's about the people."

Social responsibility is taken very seriously by Aquafinca, and the company has recently undertaken a variety of projects that reinforce this mentality. For example, Aquafinca is currently working with 14 different schools, campaigning for literacy and providing financial support. In exchange, students "adopt" parts of local forests, engaging in efforts to restore and revitalize them. This program has already produced substantial results in Cajón (Snir, 2011).

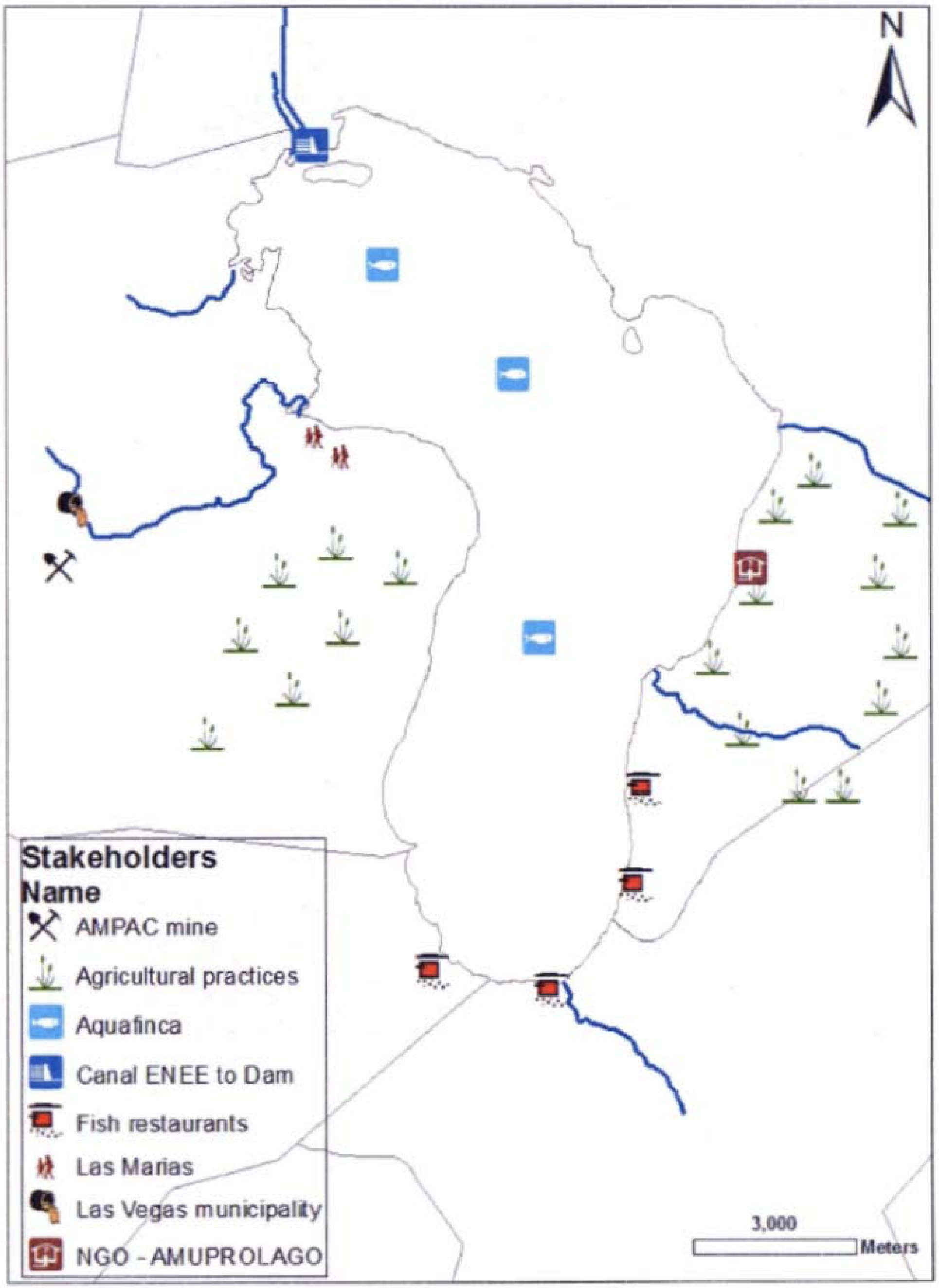

Figure 1.4: Map of the Study Site Showing Locations of Some of the Stakeholders
(Trate, 2006)

In addition, Aquafinca employs a variety of technologies that help make the operation sustainable. The plant produces little wastewater, as most of the waste from the operation is put to use. Fish remains are turned into meal, which is sold as food for livestock. Scales and skin are exported to be used in beauty products and gelatin, respectively. Glycerin from the fish oil is turned into soap for the company's employees, and most of the rest of the oil is converted to biodiesel. The processing plant produces about 5,000 gallons of biodiesel each day, which is used to fuel the plant's generator (Ramírez, 2011). Overall, the company is consistently compliant with the standards it is legally bound to adhere to; a recent audit by SERNA (the Ministry of Natural Resources and the Environment, described in Section 1.3.2.12) found only one of 35 items was not compliant, and it was in a category unrelated to the company's actual operation, according to the CEO (Snir, 2011).

1.3.2.2 AMPAC

AMPAC, the American Pacific Corporation, is currently operating a mine in El Mochito, Honduras. The mine produces lead and zinc concentrates, and waste from the mining operations is either reused as fill (the mine uses a cut-and-fill process) or sent to a tailings pond. There are currently three tailings ponds on site, the oldest of which has been capped. The newest tailings pond was installed about two years ago and is built on a geotextile foundation; it is currently in the first phase of its life. A third pond has been in operation since the 1980s and is nearing the end of its useful life, as the lifetime of a tailings pond is about 20 years. It is estimated that this pond can hold another two feet of material, which could take up to two years to accumulate. Once it is full, it will be capped with a layer of clay and organics. It is currently accepting effluent from the newest tailings pond as AMPAC awaits final governmental approval of its newest pond (Gomez, 2011).

The tailings ponds act as sedimentation lagoons, with a 6-8 hour settling time before water is discharged. Effluent from the tailings ponds is monitored by AMPAC on a monthly basis and by the government about 6-8 times per year. The effluent is tested for a variety of metals, and according to AMPAC representatives, the mine is consistently in compliance with all applicable standards. Iron is normally the most difficult metal to comply with; the maximum concentration allowed is 1 ppm, and effluent from the tailings ponds is usually at around 0.5-0.8 ppm iron (Gomez, 2011).

Tailings are sent into the ponds at a rate of about 800 gallons per minute, though the total amount sent each day varies considerably. These wastes are comprised of approximately 60% solids and 40% liquids. The waste is treated with lime and copper sulfate before entering the ponds. A small, virtually undetectable amount of cyanide also goes into this process, in much lower amounts than in the past (Gomez, 2011). Impacts of AMPAC's former practices with regards to cyanide can be seen in the name given to the microwatershed in which Las Vegas is located and to which AMPAC discharges wastes. Mentioned previously in Section 1.3, "Quebrada El Cianuro" literally translates to "Cyanide Creek."

After settling in AMPAC's tailings ponds has occurred and the solids have dried, the deposited waste resembles concrete. Tailings can be entering the pond at the same time that effluent is being discharged, so there is never a need to interrupt the overall process of treatment and removal of effluent (Gomez, 2011).

1.3.2.3 AMUPROLAGO

Asociación de Municipios del Lago de Yojoa y su Área de Influencia, the Association of Municipalities of Lake Yojoa and Catchment Area, is a non-governmental, non-profit organization directed by the mayors of the municipalities around Lake Yojoa. The organization has four categories of strategic objectives and action (AMUPROLAGO, 2005):

1. Environment: Promote the conservation and environmental protection of Lake Yojoa and catchment area through the implementation of practices and measures to ensure the sustainable use of resources.
2. Local Economic Development: Promote local business development by encouraging the inclusion of producers at the local, national, and regional level, and improve the competitiveness of the tourism industry of the Commonwealth through its positioning in the local and international market as one of the most important tourist destinations in Honduras.
3. Institution: Strengthen management capacity of municipalities belonging to the Commonwealth, supporting them in efficient performance of administrative management.
4. Infrastructure: Improve the conditions of social infrastructure that will translate into improved living conditions in terms of health, education, and housing for residents of the Commonwealth.

AMUPROLAGO works with SERNA (the Ministry of Natural Resources and the Environment, described in Section 1.3.2.12), the Conservation Institute of Forestry, water boards of communities around the lake, UMAs (environmental municipal units), and other stakeholders to identify projects and procure funding to mitigate negative impacts to Lake Yojoa. The organization also offers tours of the lake to interested citizens and visitors. Part of the reason for this is to increase awareness of the eutrophication of the lake, as described in Section 1.3.1. Two of the biggest difficulties faced by AMUPROLAGO are the political aspect of environmental conservation activities and the impact of such activities on agriculture (Marcía, 2011).

The organization is regulated by HONDULAGO (described in Section 1.3.2.4), and this chain of command has led to some political challenges. For example, the National Congress of Honduras recently approved $3 million to clean the lake, but AMUPROLAGO has not yet been given the opportunity to put this money to use. In addition, a group of volunteers from Spain had been working on a recycling project in La Guama, but the work is currently on hold due to difficulties AMUPROLAGO has encountered in acquiring a permit. This situation may be improved upon by increasing channels of communication between AMUPROLAGO and HONDULAGO (Marcía, 2011).

1.3.2.4 HONDULAGO

HONDULAGO was created as part of a 2007 law for the protection of Lake Yojoa. It is a regulating body responsible for enforcing laws, ordinances, agreements, contracts, and regulations applicable to development activity in the Lake Yojoa Subwatershed. While legally ordained to perform oversight of activities in the subwatershed, there is some concern that HONDULAGO has also been acting as an implementing agency in certain respects (Marcía, 2011).

HONDULAGO is currently working with the United Nations Development Programme (UNDP) to generate a plan to intervene and restore water quality in Lake Yojoa. UNDP identified HONDULAGO as a source of funding for such initiatives; other donors include an Italian corporation that has already contributed 26 million Euros to the project. Thus far, the information-gathering phase has been completed. Development of the plan itself, including costs, was scheduled to take place in the 45 days following our visit on January 11, 2011. HONDULAGO, in addition to serving as a source of funding for the project, has the power to design and implement tariffs for wastewater systems, although how well this prescribed power translates into action has yet to be seen (UNDP, 2011).

1.3.2.5 Las Casetas

The fish restaurants, Las Casetas, are a major tourist attraction for the region, although bad publicity surrounding Aquafinca's supposed pollution of the lake caused a recent drop in visitors to the restaurants, according to the association of restaurant owners (Santos, 2011). There are over 50 casetas along the shore of Lake Yojoa, and while they do not all serve food (some are simply small homes), they are a collective source of pollution to the lake, as cooking oil, greywater and sanitary waste are discharged directly to the lake. Hopefully, this will change once the Las Casetas wastewater treatment system is rehabilitated.

The treatment plant at Las Casetas, proposed in 2006 and constructed in 2008, consists of an upflow anaerobic sludge blanket (UASB) system for primary treatment, a trickling filter for secondary treatment, a final clarifier tank, and a sludge drying bed. Each of these features is explained and analyzed in Section 3.8. There is a pumping station to bring the wastewater into the treatment plant, though the pumps were never installed. From what we learned during our visits to Las Casetas, the treatment system was never used largely due to the failure of the piping system; the pipe that was installed was apparently not to specification and sagged significantly in the sun, making it unusable. Currently, there are plans to rehabilitate the Las Casetas plant; PRONADERS, the National Program for Sustainable Rural Development (described in Section 1.3.2.6), is in charge of this process (Castillo, 2011).

As an entirely separate project, the Las Casetas owners are also looking into installing grease traps in their restaurants, which would connect to the piping system for the treatment plant. For this reason, any plans to install new piping behind the restaurants should include "Y"-connections for the grease traps, as well as for periodic cleaning of the piping if necessary (Castillo, 2011).

1.3.2.6 PRONADERS

Programa Nacional de Desarrollo Rural Sostenible, the National Program for Sustainable Rural Development, through the Secretary of Agriculture and Livestock (SAG), is responsible for the coordination, integration, implementation, monitoring, and evaluation of rural development projects, with active participation from local governments, civil society, and the beneficiary population. The mission of PRONADERS is to improve the lives of rural families and communities, with a focus on sustainable management of natural resources. Between 2006-2010, the program executed 18 different projects, resulting in 1,313 sub-projects benefiting 212,000 poor families. These projects included access to irrigation, access to plants and animals to initiate new investments, care of natural resources, training and technical assistance in the field, small domestic infrastructure works, and making technology available to small producers (SAG, 2011).

Within PRONADERS, there is a single engineer, Jessica Castillo, in charge of infrastructure around Lake Yojoa. PRONADERS is currently funding the completion of the Las Casetas treatment plant, although government approval was expected to take another 4-5 months from the time of our visit. Management of the Las Casetas project will also fall to PRONADERS; they plan to keep the existing design with a few small changes, including covering the existing pipe with concrete and making some alterations to the collection tank. Our research will hopefully help to inform these decisions. PRONADERS also hopes to teach the Las Casetas community how to properly maintain the system; the company that wins the bidding process will be responsible for this aspect of the project. The restaurant owners will be expected to pay a fee to cover repairs and maintenance. They will also be expected to guard against theft of the pumps, which have not yet been installed for this very reason. Furthermore, the restaurant owners will be involved in the process of rehabilitating the plant, including connecting the piping system (Castillo, 2011).

PRONADERS also expects that AMUPROLAGO will be very involved in this project moving forward; HONDULAGO, on the other hand, does not currently have a strong presence, but has indicated a desire to be involved in the project and may also play a role in regulating the water once it is completed (Castillo, 2011).

1.3.2.7 RAS-HON

Red de Agua y Saneamiento de Honduras, the Honduran Water and Sanitation Network, is a network created by the Honduran government that includes government entities, private sector agencies, civil organizations, academic institutions, NGOs, international agencies (as donors), and other funding organizations (Water For People, 2006). Government agencies involved in RAS-HON include ERSAPS (the Regulatory Body for Water and Sanitation, described in Section 1.3.2.9), CONASA (the National Council for Drinking Water and Sanitation), SANAA (the National Water and Sanitation Service, described in Section 1.3.2.10), SERNA (the Ministry of Natural Resources and the Environment, described in Section 1.3.2.12), and the Secretary of Health. Watersheds are increasingly being used as the basis for water resources management planning in Honduras, an approach that complements water laws being developed, and RAS-HON is leading the effort to implement this approach throughout the country.

There are 21 hydrographic watersheds in Honduras. RAS-HON has developed criteria to assess the negative impact to each watershed, and based on these criteria selected and prioritized those watersheds where the impact was felt most strongly. Four main watersheds were prioritized: the Chamelecón watershed, near San Pedro Sula; the Choluteca watershed, in the region that includes Tegucigalpa, Honduras' capital city; the Islas de la Bahía (Bay Islands) watershed, a region central to Honduras' tourism industry, where the second largest coral reef in the world is located (and being heavily damaged by pollution); and the Ulua watershed, which includes the Lake Yojoa Subwatershed (Garcia, 2011).

For each of the four watersheds identified, a hydraulic balance was performed, which included evaluating the yield of each watershed and its organic loading due to receipt of untreated wastewater. A simulation capacity was determined for each watershed based on these factors. For Lake Yojoa, the estimated loading of biochemical oxygen demand (BOD) was 1.66 tons per day; it is this increased nutrient loading that has led to the growth of water lilies throughout the lake, as described in Section 1.3.1 (Garcia, 2011).

RAS-HON's plan, moving forward, is to increase awareness of the problem of overloading of watersheds through institutional strengthening, to monitor control of effluents, and to come up with a conceptual design of what is suitable for each site in terms of wastewater treatment. In the case of Las Vegas, RAS-HON is considering a wide variety of options for treatment, including continuing the use of the Imhoff tanks already in place, described in Section 1.3.2.8. There is also some potential for "green fill", a method of improving the natural attenuation of nutrients, which has already been implemented to some extent in the Bay Islands restaurants, Las Conchas. RAS-HON also wants to come up with an investment plan to get an idea of how much money will be required to intervene in each watershed. For Lake Yojoa, the current estimate is about $4.5 million (Garcia, 2011).

1.3.2.8 Municipalities

In 1990, the Law of Municipalities gave Honduran municipalities autonomy. This included responsibility for provision of water and sanitation to those communities within the municipalities. Nonetheless, thus far the municipalities' role in planning, development, and operation of water and sanitation systems has been very limited (Water For People, 2006). However, many municipalities are eager to take on more responsibility, as evidenced by our experience with the municipality of Las Vegas. The new mayor is highly interested in the wastewater situation; part of her campaign platform included leaving Las Vegas with cleaner, treated water. A challenge for her administration will be setting specific goals for wastewater treatment. Las Vegas' budget for 2011 does not include money for any major wastewater treatment infrastructure development, but does include room for conducting planning and studies and producing documents for consultants and bidding (Las Vegas, 2011).

Currently, Las Vegas has a pair of Imhoff tanks that serve approximately 10,000 of the municipality's estimated 26,000 residents (Las Vegas, 2011). Wastewater that passes through the tanks enters the adjacent Raices Creek, a tributary of Lake Yojoa. A second collection system running parallel to Raices Creek conveys wastewater to the location of the Imhoff tanks, but discharges the wastewater directly to the creek without any treatment. Lake Yojoa is located approximately 8 kilometers downstream of the discharge point from the Imhoff tanks and collector system. The Imhoff tanks were already overloaded with regards to flow at the time of the MEng students' visit in 2009; furthermore, flow control gates were missing, causing a large portion of the flow to bypass treatment altogether, and sludge was not being removed at the suggested 6-month interval (Bhattacharya et al, 2009).

There is some level of management of the Imhoff tanks performed by Las Vegas' department of water and blackwater, although at present nothing is being done to significantly improve upon the tanks' performance. There has been talk of constructing additional treatment facilities in the area, and several proposals for wastewater treatment plant designs in Las Vegas are currently under review. The companies developing these proposals include ARITA, 5INHCO, and some North American investors. However, while there is a good amount of land available for construction (specifically, about 5,200 m^2 next to the Imhoff tanks), there are foreseeable difficulties when it comes to complying with siting regulations (Las Vegas, 2011).

San Juan and El Mochito, two urban centers within the Las Vegas municipality, are both served by septic tanks; however, the majority of these tanks are in poor condition, with many of them having collapsed. It has been suggested that these two locales be connected to Las Vegas' wastewater treatment system, though at present this would only cause further overburdening of the system, with little to no treatment actually occurring.

It is evident from our experience in the field that involvement of the municipal governments in the management of wastewater infrastructure is integral to ensuring that systems are adequately constructed, monitored, and maintained, and it will also be essential to establish communication among the different municipalities in order that they may share their problems, experiences, and knowledge with regards to wastewater treatment development.

1.3.2.9 ERSAPS

Ente Regulador del Sector Agua Potable y Saneamiento, the Regulatory Body for the Water Supply and Sanitation Sector, is one of the entities created by the 2003 Framework Law for the Water and Sanitation Sector of Honduras. Its role includes the regulation and quality control of water and sanitation services (Water For People, 2006). ERSAPS is currently working with RAS-HON through member NGOs and private consultants to create a forum for discussion as an alternative to a chain-of-command form of management. This included working closely with RAS-HON to evaluate human impact on Honduras' watersheds, as outlined in Section 1.3.2.7. The information-gathering phase of this work has been completed, and ERSAPS is currently in the process of developing a summary report, with assistance from the World Bank. They are also looking for support from the political sector and groups that develop infrastructure in order to create an institutional and legal framework for this process (Garcia, 2011).

1.3.2.10 SANAA

Servicio Autonomo Nacional de Acueductos y Alcantarillados, Honduras' National Autonomous Water and Sanitation Service, is the agency in charge of water services throughout Honduras. SANAA consists of three groups: a legal administration commission, a sectoral planning commission, and an engineering and support commission, which together comprise seven administrative divisions and four support divisions for the implementation of policies and projects. There are six regional offices throughout the country (SANAA, 2010).

Created by the Government of Honduras in 1961, SANAA has traditionally been in charge of promoting policies, planning, construction, and operation of Honduras' water and sanitation projects. The agency operated about 42 urban water systems until 1990, when the Law of Municipalities was implemented. This law mandated that management of water and sanitation services fall under the purview of the individual municipalities, and as such, some municipalities began requesting the transference of responsibility from SANAA to their own governments. SANAA resisted the release of these responsibilities initially, but the Framework Law of 2003 required the agency to complete the transference of responsibility for about 32 water systems by 2008 (Water for People, 2006). It is unclear to us whether SANAA has fulfilled this obligation.

1.3.2.11 AMHON

La Asociación de Municipios de Honduras, the Association of Municipalities of Honduras, is a civil entity composed of mayors representing the 298 municipalities in Honduras. The organization serves as an advocate for municipal interests and acts to defend municipal autonomy, decentralization, and national reconstruction.

The organization's basic strategy, as quoted from their website (AMHON, 2011), is as follows:

1. AMHON creates, analyzes, and updates the strategic model for Honduran municipalities.
2. AMHON encourages every mayor to adopt the strategic model for Honduran municipalities as a reference model for its role and supports them politically and technically to do so.
3. AMHON promotes and supports the strategic model for Honduran municipalities, and the demands of the mayors consistent with this model, with the branches of government and international bodies.
4. AMHON develops its own political and technical capacity for reflection and implementation designed to meet the above objectives.

In 2008, an environmental management department was added to AMHON under the social, environmental, and economic department. This department uses an integrated approach to investigate social and environmental impacts as a whole, with a focus on strengthening local capacity through strengthening of municipal policy. The department is also interested in developing strategic alliances at the national, regional, and local level to coincide with national and local environmental policies (AMHON, 2011).

As part of the National Plan for Honduras, AMHON is working to help municipalities take control of their wastewater treatment systems and strengthen municipalities in the decision-making process. In 2010, 14 systems were decentralized, with control shifting from SANAA to the municipalities. Many of the systems that have already been decentralized are working well, providing a model for financial independence (AMHON, 2011).

There are a number of weaknesses within the municipalities, of which AMHON is well aware. For this reason, AMHON's goal is to have a strong legislative and policy input while working in line with national policies already in place and currently in development. This will involve specific work with individual municipalities in an attempt to strengthen and build local capacity (AMHON, 2011).

AMHON is also partnering with UPI in this work. At the time of our discussion with AMHON, a workshop had been scheduled for February of 2011 at which representatives of the municipalities that have successfully gained control of their wastewater systems would meet to discuss what worked and what didn't and to discuss potential changes to their systems. Luis Eveline, Rector of UPI, proposed appointing an engineer to oversee activities in these municipalities following the meeting to ensure proposed changes actually happened in each municipality and to provide a report to AMHON based on these changes (AMHON, 2011). However, the meeting never occurred, highlighting the need for improved communication between municipalities and a stronger emphasis on wastewater treatment as a collaborative effort.

One problem that AMHON has identified is that there is not much political gain to be had for mayors investing in wastewater. Sanitation is not a popular topic, and most people are happy simply not talking about it. As such, fees charged for improper maintenance of wastewater systems are either nonexistent or too small to be of significance. Furthermore, there is no continuity when the political situation changes every four years, and AMHON recognizes the need for a technical unit to address this problem. There is also an overall need for citizen awareness, a significant challenge that should more appropriately be addressed by the Honduran Social Fund (FHIS) (AMHON, 2011).

An opportunity for municipalities to work together to address the wastewater problem is through development of regional wastewater treatment plans based on watershed management. Honduras has been broken up into 7 distinct regions, each associated with a different hydrologic watershed; each region now has the opportunity to develop its own plan of action. There is a problem with visibility – people never actually see pollutants entering the watershed, and as the old adage goes, "out of sight, out of mind." By making an initial assessment report for each watershed, complete with data on the loadings that each watershed is suffering from, municipalities could be empowered to make sure that something gets done. Some municipalities have already made headway in this process and can be used as a model to bring the rest in (AMHON, 2011).

AMHON is now looking to create a new program to specifically address and procure funding for wastewater issues in a national context. In order to procure funding, Environmental Municipal Units (UMAs) will need an initial assessment of the needs of each municipality. AMHON is planning to work with SERNA (the Ministry of National Resources and the Environment, described in Section 1.3.2.12) to make this happen. As a case study, there is a treatment facility in San Marcos de Colón from which pollution is entering a natural reserve in Nicaragua via a river which runs across the border between the two countries, creating a trans-boundary water issue. A technical analysis of this treatment plant is currently being performed by Aridai Herrera, SANAA, and an environmental engineering student from UPI. This analysis will help to identify why the system is not being maintained and to justify putting more money into it. This is the sort of analysis that AMHON hopes to eventually provide for all wastewater treatment facilities throughout the country in order to create a matrix that will allow politicians to make more informed choices by comparing costs based on flows at each location and determine where there is the greatest need and the greatest potential for improvement (AMHON, 2011).

1.3.2.12 SERNA

Secretaría de Recursos Naturales y Ambiente, the Ministry of Natural Resources and the Environment, is in charge of the formulation, coordination, implementation, and evaluation of policies related to: protection and use of water resources; new and renewable sources of energy;

transformation of hydroelectric and geothermal energy; mining activities; exploration and exploitation of hydrocarbons; environment, ecosystems, national parks, and protection of flora and fauna; and research and control of pollution in all of its forms. Its mission is to promote the sustainable development of Honduras through such policies in order to improve the quality of life of the inhabitants of Honduras (SERNA, 2010).

Chapter 2: State of Wastewater Treatment in the Lake Yojoa Subwatershed

The state of wastewater treatment in Lake Yojoa Subwatershed of Honduras is very limited. During our site investigation of the Lake Yojoa Subwatershed we were not able to visit all of the wastewater treatment systems. We focused our research on the municipalities of Las Vegas and Santa Barbara, and the Las Casetas wastewater treatment plant. We found that all three locations were discharging untreated wastewater into the local receiving water bodies. The municipalities of Las Vegas and Santa Barbara, which did have wastewater treatment for a portion of their wastewater flow, were discharging poor quality effluent due to several different factors: lack of adequate maintenance, primary treatment only, and hydraulic overloading. Despite these facts, sanitation in the Lake Yojoa Subwatershed has improved in some ways over the past five years since MIT began working in Honduras. In particular, the amount of coverage has improved in many locations. The term "coverage" is used to describe buildings in local communities that are tied into to sewer systems. The word coverage and treatment are not synonymous. Many of the systems that are covered by local piping systems are still discharging untreated wastewater into local receiving water bodies.

2.1 Mayoral Meeting

During the investigation period in January 2011, our team attended a mayoral meeting held in the municipality of Santa Barbara. In attendance were over twenty mayors from different municipalities in the State of Santa Barbara, Honduras. We were given the opportunity to introduce our project, explain what we hoped to accomplish, and describe how these leaders could support our research work to improve wastewater treatment in the Lake Yojoa Subwatershed. There was an overwhelming response from many of the mayors as they described their wastewater problems. Many of them had insufficient treatment, lack of qualified plant operators, and a lack of funding to address system deficiencies. After listening to some of the mayors describe their various wastewater problems, we posed the question, "Who does NOT have problems with their wastewater treatment?" Of the over twenty mayors in the meeting, only one of the municipalities was not having problems with their wastewater treatment. This is very indicative of the state of wastewater treatment in Lake Yojoa Subwatershed of Honduras.

The local political structure plays a key role in the state of wastewater treatment in the region. Many of these mayors have more pressing needs to address than the lack of adequate wastewater treatment, and with insufficient funds many of these wastewater problems endure. When a new mayor is elected to a municipality, which occurs every four years, many of the municipal personnel, including the wastewater treatment plant operators, change. The new municipal labor force generally has no previous experience or understanding of the municipal wastewater treatment system they inherit. Furthermore, the lack of system maintenance records complicates this transition.

2.2 Typical Wastewater Systems- Honduras

2.2.1 Imhoff Tank

The Imhoff tank is a primary treatment wastewater system that has been used in the Municipality of Las Vegas in the Lake Yojoa subwatershed. An Imhoff tank consists of two levels, an upper primary sedimentation chamber and a lower anaerobic digester for the settled solids (Ujang and Henze, 2006). The Imhoff tanks rely on gravity for the particles to settle out. In principle this system works well for developing countries where decentralized wastewater treatment is desirable because it does not require the waste flow to be pumped long distances. The Imhoff tanks also require no electricity, only a hydraulic gradient to drive the flow. They work well in mountainous regions where there are ample slopes to drive the flow of wastewater. This is one of the main reasons Imhoff tanks are so prevalent in Honduras and the developing world in general. Another advantage of the Imhoff tank is that they require very little maintenance and no skilled labor (Crites, 1998). Only basic record keeping and daily visits are required to keep the Imhoff tanks running properly. Sludge removal should occur on a bi-annual basis or as necessary.

If the Imhoff tanks are maintained properly they provide an excellent low cost sustainable option for primary treatment. They can also be easily retrofitted with other secondary systems to produce a higher quality of effluent. However, if these systems are not maintained properly, their treatment capacity is reduced. In Honduras there have been problems with consistent maintenance regarding Imhoff tanks. In 2009, a group of MIT students assessed the condition of the Las Vegas Imhoff tank and during their visit it was determined that the Imhoff tanks were online and physically intact. However, it was also apparent that the tanks had not been maintained properly. The flow gates, which are critical to alternating the flow pattern through the tanks and thus spreading the sludge evenly among the tanks, were not in place. There were no baffles in place at any of the tanks, thus affecting the sedimentation process in the tanks. It was also reported that the sludge had likely never been emptied until just prior to the visit – a period of about 15 years. This would explain the higher coliform concentration in the effluent leaving the tanks.

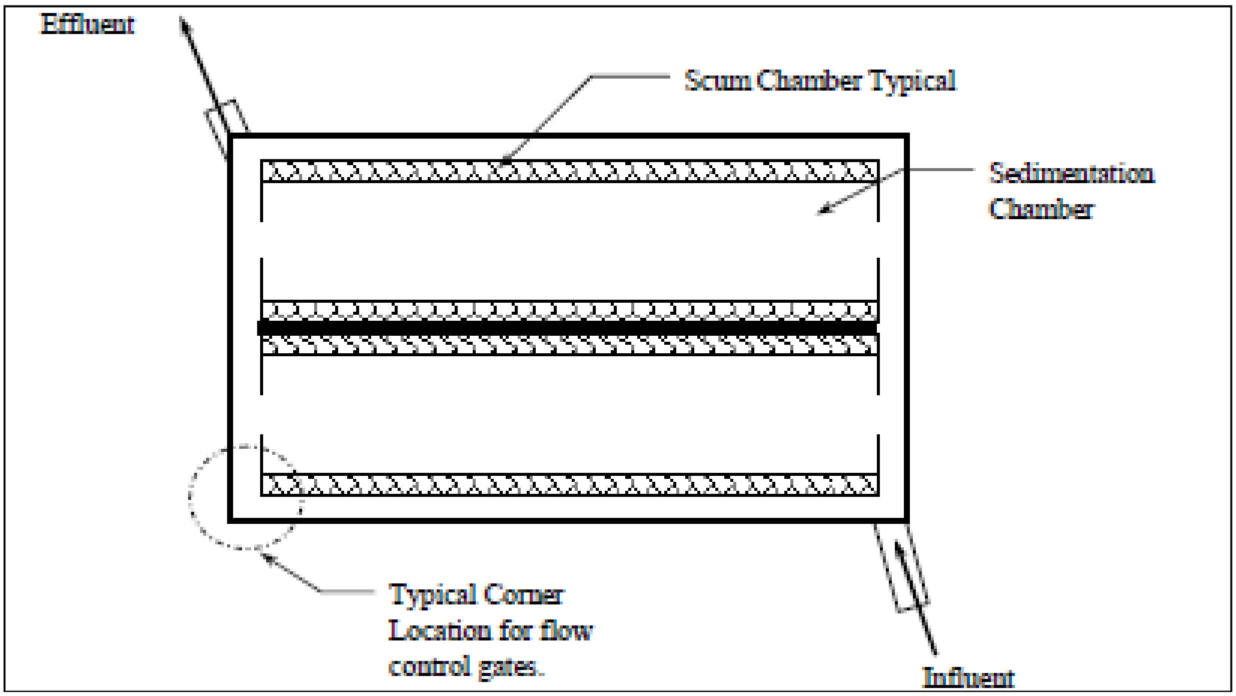

Figure 2.1: Typical Plan View of Parallel Imhoff Tank System
(McLean, 2009)

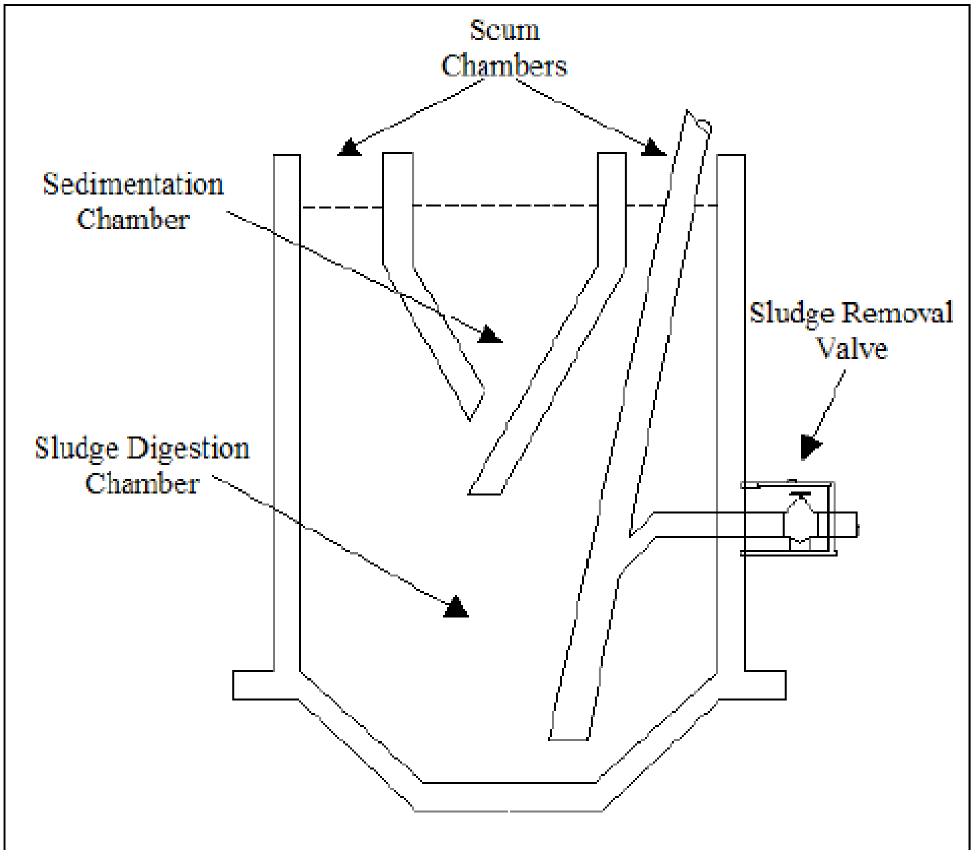

Figure 2.2: Imhoff Tank Schematic
(Mikelonis and Hodge, 2008)

2.2.2 Waste Stabilization Ponds

Waste Stabilization Ponds (WSPs) have been widely accepted as a means of wastewater treatment in Honduras. There are many advantages of WSPs including their physical simplicity, low cost, low maintenance requirements, and excellent pathogen removal (Ujang and Henze, 2006). The ponds are typically large excavated areas with shallow depths and embankments built from the excavated material. The ponds rely on many natural processes to promote wastewater treatment including photolysis, bacteria, and algae. The wastewater treatment occurs with very little human interaction. These systems generally work best in series, with subsequent ponds having improved water quality (Ujang and Henze, 2006). The first type of pond in the series is called a facultative pond, which removes both organic matter and excreted pathogens. These ponds are generally followed by maturation ponds, which are used to reduce pathogens. An anaerobic third pond is often used, but due to odor problems, these ponds are generally not found in Honduran systems. If maintained properly, WSP systems can provide an excellent low cost source of primary and secondary treatment.

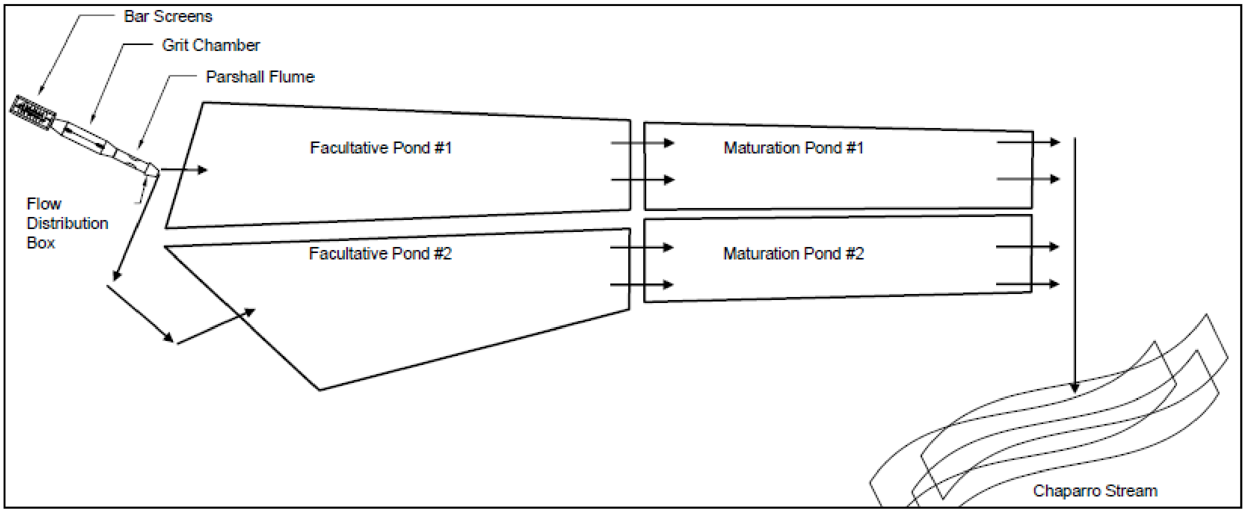

Figure 2.3: Schematic of Coloma Waste Stabilization Pond
(Bhattacharya et al., 2009)

Over the past four years there have been assessments of waste stabilization ponds in Honduras by MIT students. Several of the ponds that were visited were in good working condition. Some of the problems encountered were uneven flow distribution through systems running in parallel and no de-sludging of most systems for over five to ten years, which is the recommended removal time. This led to accumulation of sludge, which broke through the surface of the water, at the outlets of the ponds.

2.2.3 Packaged Activated Sludge Plants

The packaged activated sludge plants are single tank systems which consist of two main zones: aeration and secondary settling (Bhattacharya et al., 2009). The influent wastewater enters through the outer aeration zone of the system. In this outer zone air from a blower system is applied through disk diffusers (Bhattacharya et al., 2009). Effluent from the aeration zone enters the center of the tank in which the secondary settling occurs. The effluent from the settling chamber is then discharged from the system.

In 2009, MIT students assessed the condition of packaged plants in Ciudad Divina. There were three packaged plants at Ciudad Divina, of which only one was operational at the time. The operational plant had no apparent provision for de-sludging of the system, although there appeared to be an outlet that was capped on the bottom of the tank. The second plant was abandoned about a month and a half before the assessment. It appeared to be in very poor condition and was abandoned due to high operational costs resulting from high energy consumption of the air blower. The third plant had apparently never been used. It seems that without skilled labor and adequate operation and maintenance budget, these packaged plants are not a viable option.

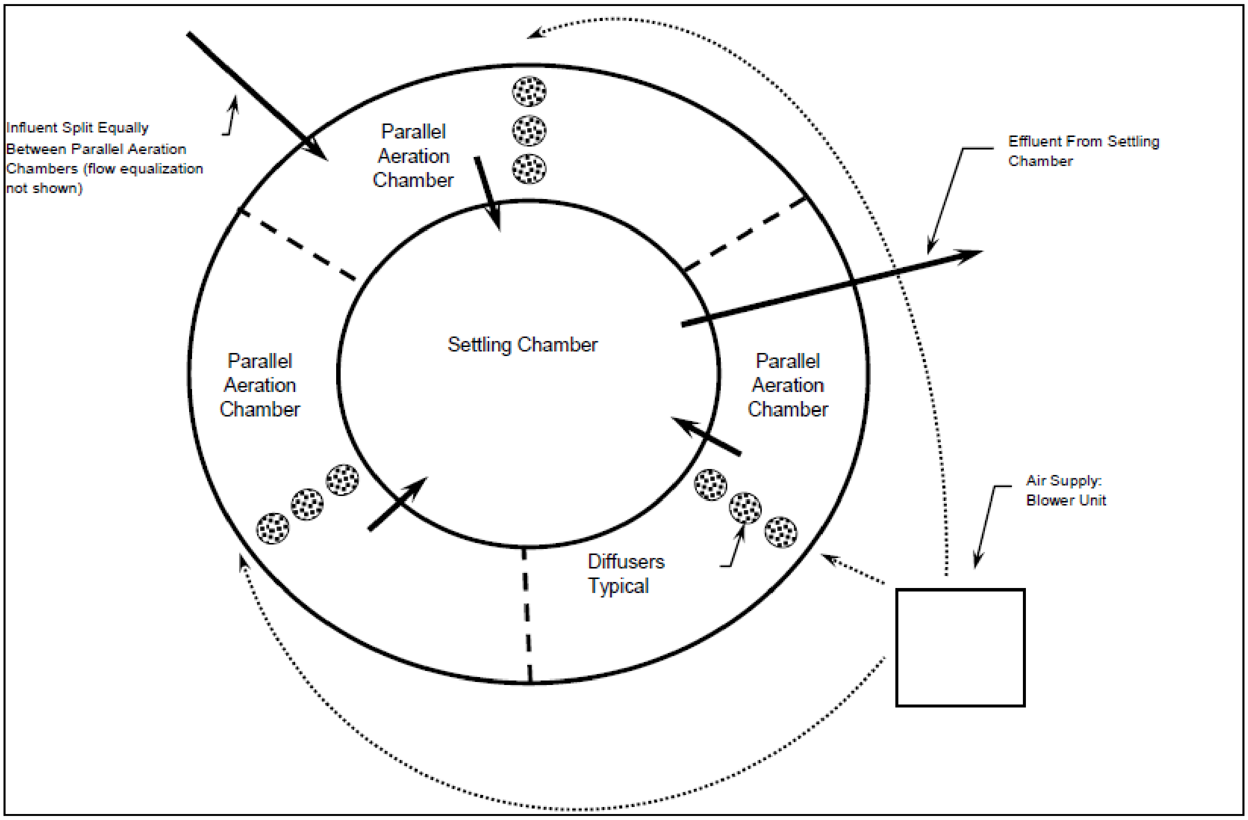

Figure 2.4: Schematic of Ciudad Divina Packaged Activated Sludge Plant
(Bhattacharya et al., 2009)

2.2.4 Upflow Anaerobic Sludge Blanket Reactors

Upflow Anaerobic Sludge Blanket Reactors (UASBs) are reinforced concrete structures that produce a high rate anaerobic wastewater treatment (Mara, 2005). UASBs have been tested in tropical and subtropical regions and are widely used in Europe and South America (Crites, 1998). These systems are a viable option as an efficient form of primary treatment for the Lake Yojoa Subwatershed and are just beginning to see implementation in the region, with one UASB having been built recently to service the fish restaurants (Las Casetas). The critical elements of the system are an influent distribution system, a gas-solid separator, an effluent withdrawal design, and in most case a settling zone (Metcalf and Eddy, 2003). Some UASBs consist of two chambers: a lower digestion zone and an upper settling zone. The system operates by driving the influent evenly through the bottom of the unit through a layer of sludge (sludge "blanket") and effectively removing BOD through anaerobic biochemical reactions (Mara, 2005). Once the water passes through the sludge blanket, it moves upward to the settling zone along with some suspended activated sludge particles. In the next step the phase separator decreases the upward flow of the water, allowing the activated sludge particles to settle out to the bottom of the tank and thus keeping a dense concentration of activated sludge on the bottom (Mara, 2005).

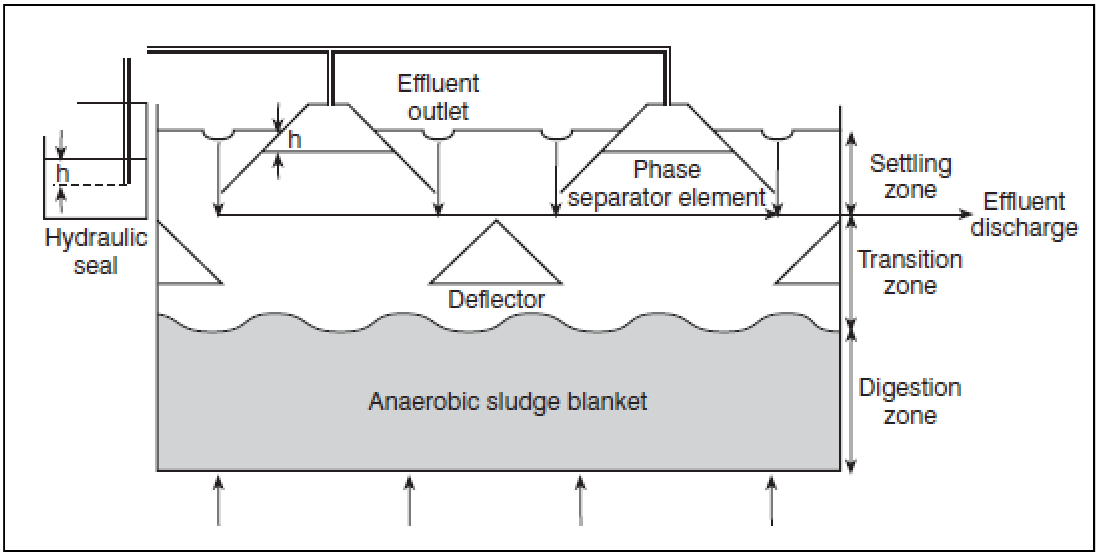

Figure 2.5: Schematic of a Basic UASB
(Mara 2005)

The UASBs do require a significant amount of energy, but they also produce bio gas in the form of methane that could possibly be burned off or used to generate electricity. The estimated energy usage for the UASB built for Las Casetas wastewater treatment plant is 795 KWh per month (Patronato de Playas de Maria, 2006). It seems feasible that areas in Honduras may be able to use UASBs if they have sufficient funding and can provide adequate operation and maintenance for the system.

2.3 Wastewater Treatment in Las Vegas

2.3.1 Introduction

Las Vegas is located in the county of Santa Barbara in the northwest of Honduras, approximately 10 kilometers west of Lake Yojoa (Figure 2.6) and 130 kilometers northwest of Tegucigalpa, the capital of Honduras.

Figure 2.6: Map of Las Vegas Region, Honduras
(Google 2011)

The municipality of Las Vegas includes the surrounding communities of El Mochito and San Juan. In total, it is estimated that between the three communities there are 17,000 residents (Joel & Adalberto, 2011) residing in 2,850 homes (McLean, 2009). It is also estimated that 600 homes (3,600 people) are connected to the Las Vegas Imhoff tank (Bhattacharya et al., 2009). Many of the remaining residents of Las Vegas are connected to a recently built collector system which discharges directly into the Raices Creek without any form of treatment (Joel & Adalberto, 2011). The residents of El Mochito and San Juan are served mainly by septic tanks or they discharge their waste directly into Raices Creek. Figure 2.7 provides a schematic representation of the Las Vegas watershed and the location of the Imhoff tank.

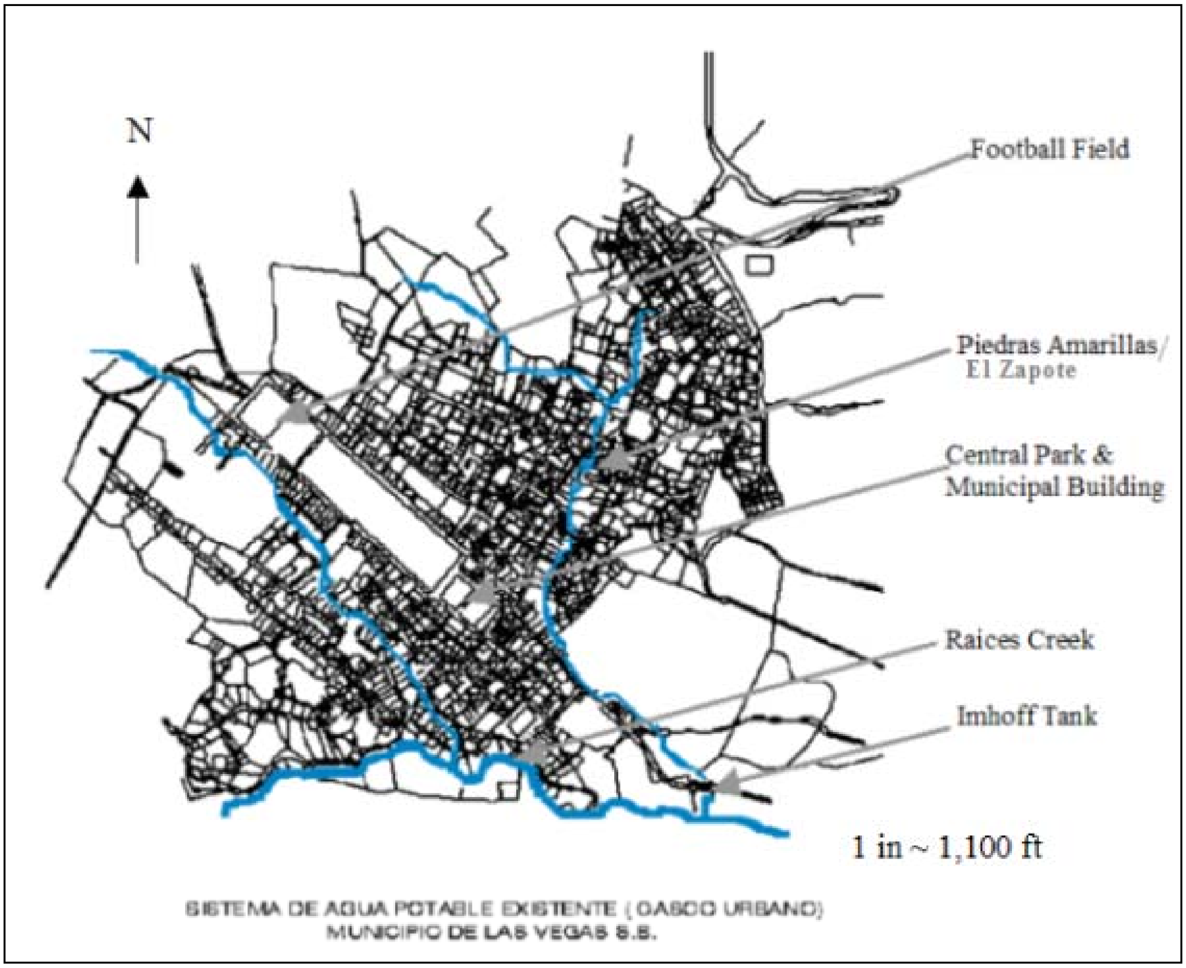

Figure 2.7: Schematic Representation of Las Vegas Watershed
(Mikelonis, 2008)

2.3.2 Description of Las Vegas Imhoff Tank

In 1992 an Imhoff tank, consisting of two identical tanks running parallel to each other, was built for the municipality of Las Vegas. It receives wastewater from the municipality of Las Vegas and discharges the effluent into the Raices Creek which flows into Lake Yojoa eight kilometers downstream of the discharge point as shown in Figure 2.8. The Imhoff tank is located in the southern part of Las Vegas and it uses natural slopes to drive the flow of wastewater to the treatment plant. Figure 2.9 shows the location and area surrounding the Imhoff tank. Figure 2.10 and 2.11 show a plan and elevation view for the Las Vegas Imhoff tank.

2.3.3 Previous Work

Since MIT began studying wastewater treatment in the municipality of Las Vegas in 2005, there have been three research trips to Honduras, as well as a study by a University of Texas student who has been a major contributor to MIT's work.

In 2006, while attending the University of Texas, Aridaí Herrera published his thesis "Rehabilitation of the Imhoff Tank Treatment Plant in Las Vegas, Santa Barbara, Honduras, Central America." The objective of his thesis was to create a guide for officials in the municipality of Las Vegas for the operation and rehabilitation of the Imhoff tank. Herrera found that since 1992, when the system was built, there had been no record of any maintenance or de-sludging of the system. There were also no control gates which allowed some of the flow to bypass the two tanks and thus leave the system untreated. It was assumed that combined sewers drain the city, which means that during a heavy rainfall, large amounts of sand from many of the dirt roads in Las Vegas drain into the Imhoff tank. This sand was observed by Herrera in the channels between the control gates and also assumed to be present at the bottom of the tank.

Sludge removal was also observed to be a significant problem. At the time of Herrera's visit all six de-sludging valves were opened and no sludge flowed out of the tank. The lack of sludge flow was attributed to hardened sludge in the bottom of the tank, a large amount of sand in the digestion chambers, plugging of de-sludging pipes in the tanks connected to the valves, or a combination of all of these conditions (Herrera, 2006). The pollutant removal efficiency was evaluated based on wastewater analysis from Experco, a Canadian environmental engineering consulting firm that evaluated the tank in 2003. The Total Suspended Solids (TSS) removal was 46% and the BOD_5 results showed a 12% increase of the influent concentration at the effluent point (Herrera, 2006). These values indicated that the Imhoff tank was not performing up to standards and that it was also in violation of the national discharge regulations.

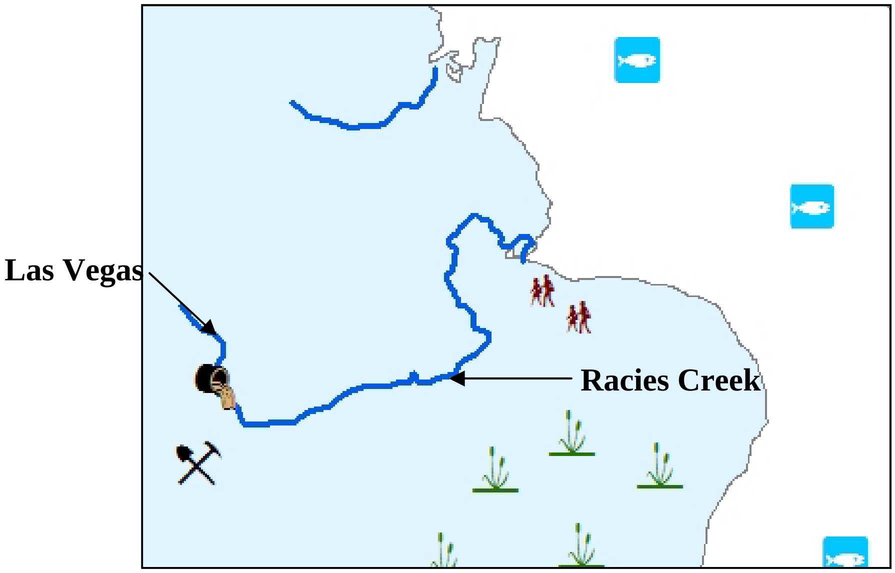

Figure 2.8: Various Stakeholders in and Around Lake Yojoa
(Chokshi and Trate, 2006)

Figure 2.9: Imhoff Tank and Surrounding Area
(Walker 2011)

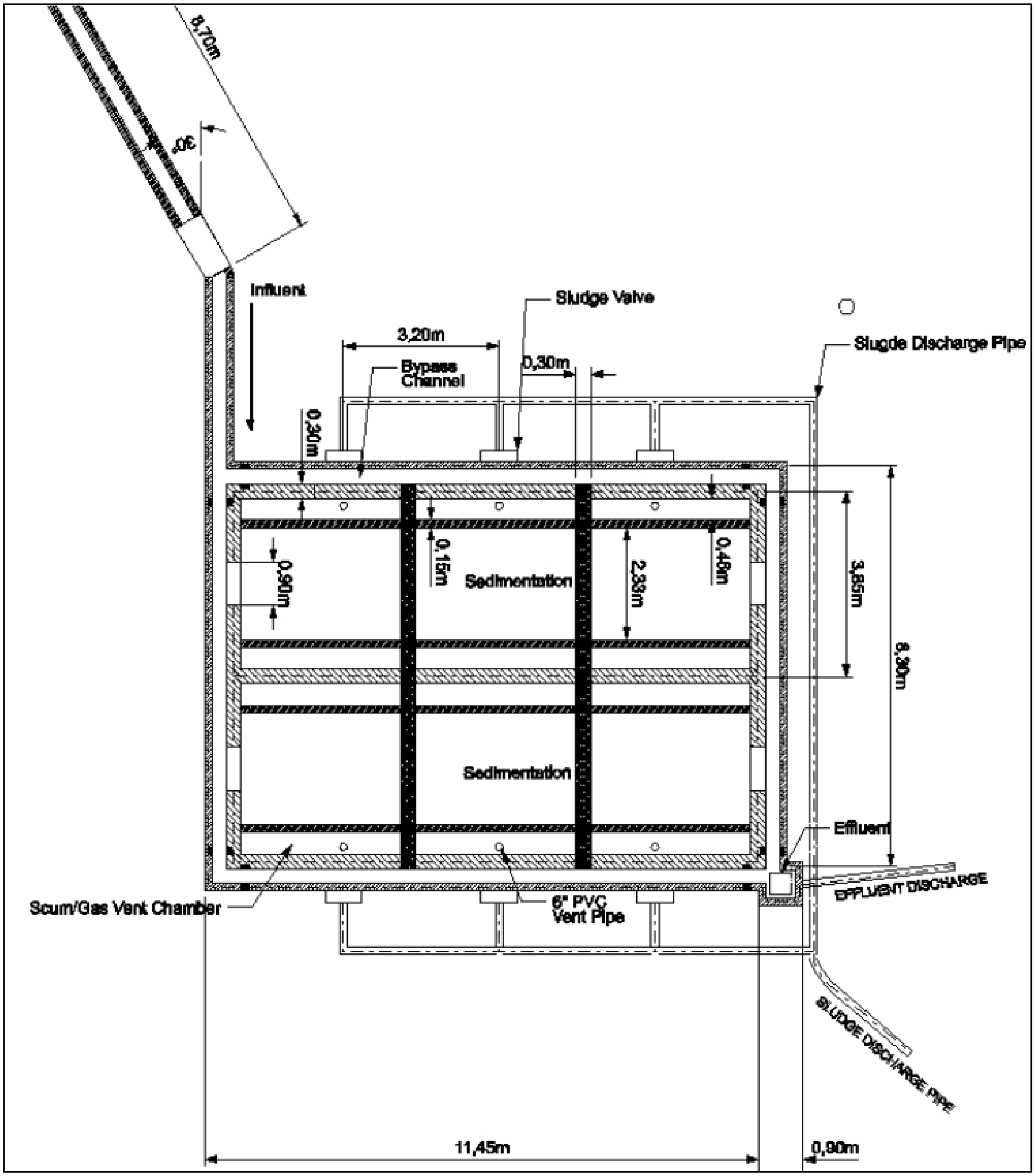

Figure 2.10: Plan View of Las Vegas Imhoff Tanks
(Herrera 2006)

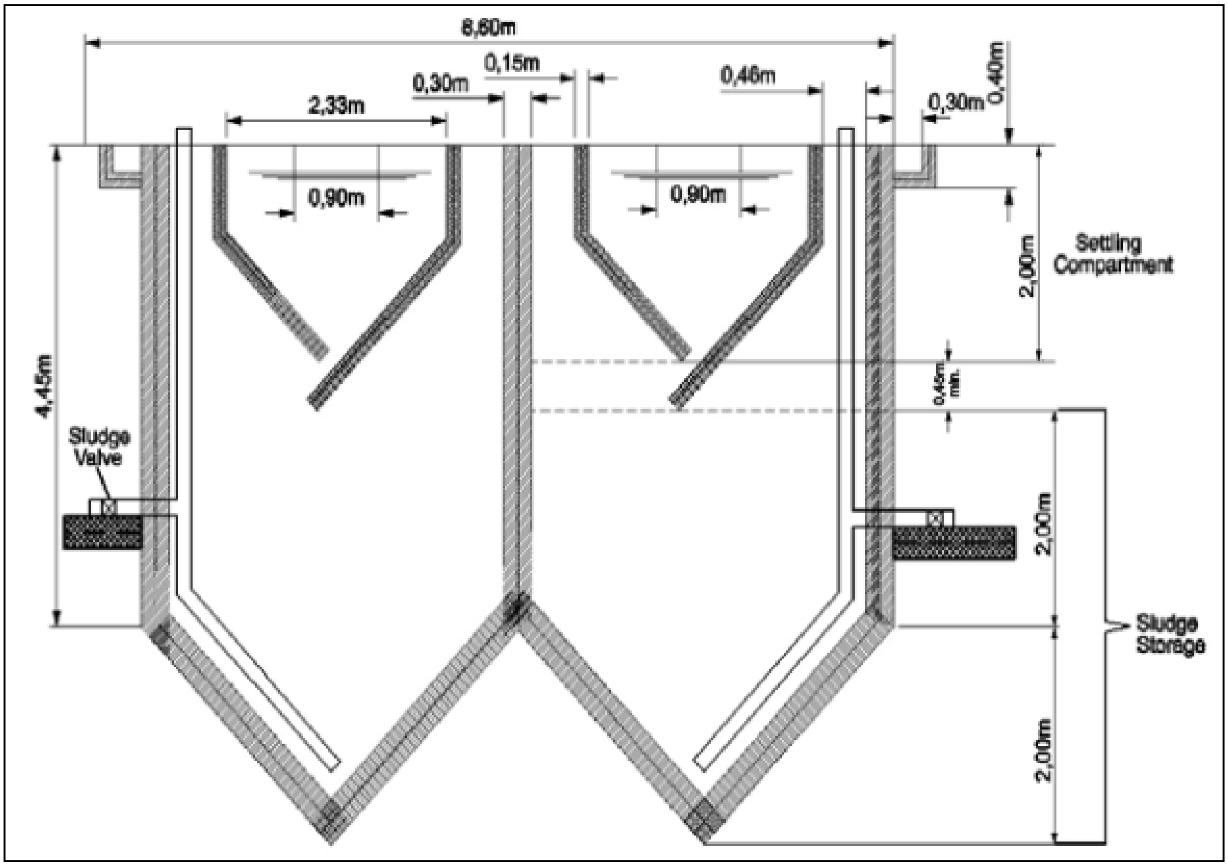

Figure 2.11: Cross Section of Las Vegas Imhoff Tanks
(Herrera 2006)

In 2006, the first group of MIT students performed research work in Honduras. Mira Chokshi and Tia Trate investigated the nutrient loading and temperature analysis of Lake Yojoa. During their trip to Honduras they looked at many of the contributors and stakeholders of Lake Yojoa which included the municipality of Las Vegas. However, their research was mainly focused on an estimation of nutrient loading from the effluent of the Imhoff tank. It was assumed during the time of their research that the Imhoff tank did not provide any substantial nutrient removal.

The second group of MIT students, Anne Mikelonis and Matthew Hodge, conducted research in Honduras in 2008. Their primary focus was on the improvement of wastewater treatment for the municipality of Las Vegas. Hodge's work focused on the assessment and rehabilitation of the Las Vegas Imhoff tank. He found that the Imhoff tank was hydraulically overloaded and that the wastewater was being severely diluted by sources of clean water. It was estimated that the wastewater production for Las Vegas was approximately 1,000 L/day/person and approximately

556 legal connections to the sewer system (Hodge, 2008). This number of connections could be greatly higher due to illegal connections to the sewer system. The high amount of wastewater production produced diluted wastewater which was very difficult to treat. At the time of Hodges's study, the Imhoff tank was removing 26% TSS and 19% of biochemical oxygen demand and chemical oxygen demand (Hodge, 2008). It was determined that that maintenance could improve the level of treatment, but only through the reduction of clean water dilution of the wastewater could real gains be made in treatment efficiency.

Mikelonis' work focused on the addition of chemicals to improve the primary treatment of Honduran Imhoff tanks. She did this by testing the addition of solid aluminum sulfate to the influent of the Las Vegas Imhoff tank. "Bench scale testing and pilot testing during January 2008 in the Las Vegas Imhoff Tanks found that a dosage of 150 mg/l alum (17% Al_2O_3) was necessary to treat Las Vegas' domestic wastewater" (Mikelonis, 2008). Due to the high costs of alum and the difficulty of chemical preparation and chemical injection, it was determined that chemical addition was not a viable option for the municipality of Las Vegas. Her overall recommendations were similar to those of Hodge in that only through the conservation of water and the improved maintenance would the Las Vegas Imhoff Tank provide high quality primary wastewater treatment.

In 2009, Mahua Bhattacharya, Lisa Kullen, and Robert McLean, three MIT students, evaluated wastewater treatment options for Honduras. They investigated ten wastewater treatment facilities in Honduras that were considered to be typical types of treatment, including the Las Vegas Imhoff tank. It was determined that the tank was online and functional at the time of their trip. They determined, through flow measurements over a twenty-four hour period that the average flow into the Imhoff tank was approximately 1,050 m^3/day with a peak flow of 1,250 m^3/day (Bhattacharya et al., 2009). These flows were approximately three times less than the flows observed by the MIT students in 2008. This could be due to the reduction of water recommendations made by Mikelonis and Hodge, or to the lack of stormwater runoff during their investigation. At the time of the trip in 2009, there was no extreme rain event reported prior or during the visit (Bhattacharya et al., 2009). Also, in 2008 the MIT students found evidence of coffee beans in the wastewater. During the de-pulping of coffee beans, water is allowed to run over the picked beans for upwards of 24 hours (Mikelonis and Hodge, 2008). This could be another reason for the difference in flow measurements from 2008 and 2009. There was no evidence of coffee bean de-pulping during the 2009 investigation.

2.3.4 Assessment of Las Vegas Imhoff Tank

2.3.4.1 Physical Condition

During our investigation we found that the Las Vegas Imhoff tank was structurally sound with no major cracks in the concrete. There were some areas where the concrete was starting to wear away (Figure 2.13), but none of these areas are affecting the capability of the system to function properly. The effluent PVC pipe, shown in Figure 2.12, did have some sort of leak, presumably at the connection to the horizontal PVC pipe running underground, but the leak did not appear to be losing a significant amount of water. There was also some substantial damage to the sludge removal valves which will be described in section 2.3.4.4.

2.3.4.2 Vehicle and Human Traffic

When the tank was built in 1992, there were no residents living to the south of the tank, and thus very little traffic near the tank. However, at some point over the years, "squatters" set up on the land just south of the tank. Today, the number of people living beyond the tank is fairly substantial and leads to a fairly constant flow of traffic by the tank. The vehicle and human traffic directly next to the Imhoff tank could be an issue (Figure 2.14). These squatters now have documentation of their residency thus making it unlikely they will be moved. This traffic does not necessarily affect the performance of the Imhoff tank, but adverse health implications can result from direct human contact with the raw sewage or the partially treated effluent. Also, the current traffic through the site limits the options for expanding the plant to incorporate secondary treatment. There are rumors from some of the locals that children sometimes place rocks into the influent chamber, thus clogging the system, and resulting in the overflow of wastewater across the road. At the time of our investigation there were children playing on top of the Imhoff tank.

Figure 2.12: Effluent PVP Pipe
(Walker 2011)

Figure 2.13: Concrete Damage near the Imhoff Tank Outlet
(Walker 2011)

2.3.4.3 Missing Flow Gates and Baffles

Consistent with previous studies of the Las Vegas Imhoff tank, there were no control flow gates and baffles at the time of our investigation. The flow gates are used to switch the flow direction in the tanks, as well as prevent wastewater from flowing through the system untreated. Figure 2.16 shows a schematic drawing of the Imhoff Tank including the locations of the missing flow gates. The proper operation of an Imhoff tank includes the switching of the flow path approximately every two weeks (Crites, 1998) to distribute the sludge evenly in the bottom of the tank. This operation is very easy to accomplish with minimal labor. Figure 2.15 shows a picture of the slots for the control flow gates.

Figure 2.14: Location of Imhoff Tank in Reference to the Road
(Walker 2011)

In 2008, time was spent creating and installing control flow gates to regulate the flow between the two batteries (Mikelonis, 2008). These flow gates originally failed because they did not form a tight seal with the concrete slots in the channel. Subsequently sand bags were placed behind the wooden flow gates to tighten the seal (Mikelonis & Hodge, 2008). This set up, shown in Figure 2.17, was successful in creating a good seal and is recommended for the future use of control flow gates.

The missing baffles cause irregular velocity of the wastewater within each tank and thus lead to uneven deposition of solids and unequal residence times in each tank. The uneven flow of wastewater into the first battery of the Imhoff tank without the baffles can be seen in Figure 2.18.

Figure 2.15: Control Flow Gate Slots
(Walker 2011)

In 2008 an effort was made to correct this uneven flow by installing baffles at the inlet of each Imhoff tank. Wooden baffles were used with 13 holes per row which were hand drilled and approximately one inch in diameter (Mikelonis & Hodge, 2008) as shown in Figure 2.19. The baffles are used to balance flow through the entire width of the Imhoff tank. Because there is no preliminary treatment such as grit removal or bar screens, the holes can become easily clogged by plastic bags or other objects causing the flow to come over the top of the baffle. However, this is still better than the alternative of uneven flow without the baffles. The baffles should be cleaned on a daily basis to insure their proper function.

It is assumed that the baffles installed in 2008 were stolen shortly after the MIT team left Honduras and used as fire wood. If new baffles and control flow gates are installed, steps would need to be taken to insure that they are not stolen. This reiterates the disadvantages of having a wastewater treatment plant in such close proximity to human and vehicle traffic without any fencing or protection.

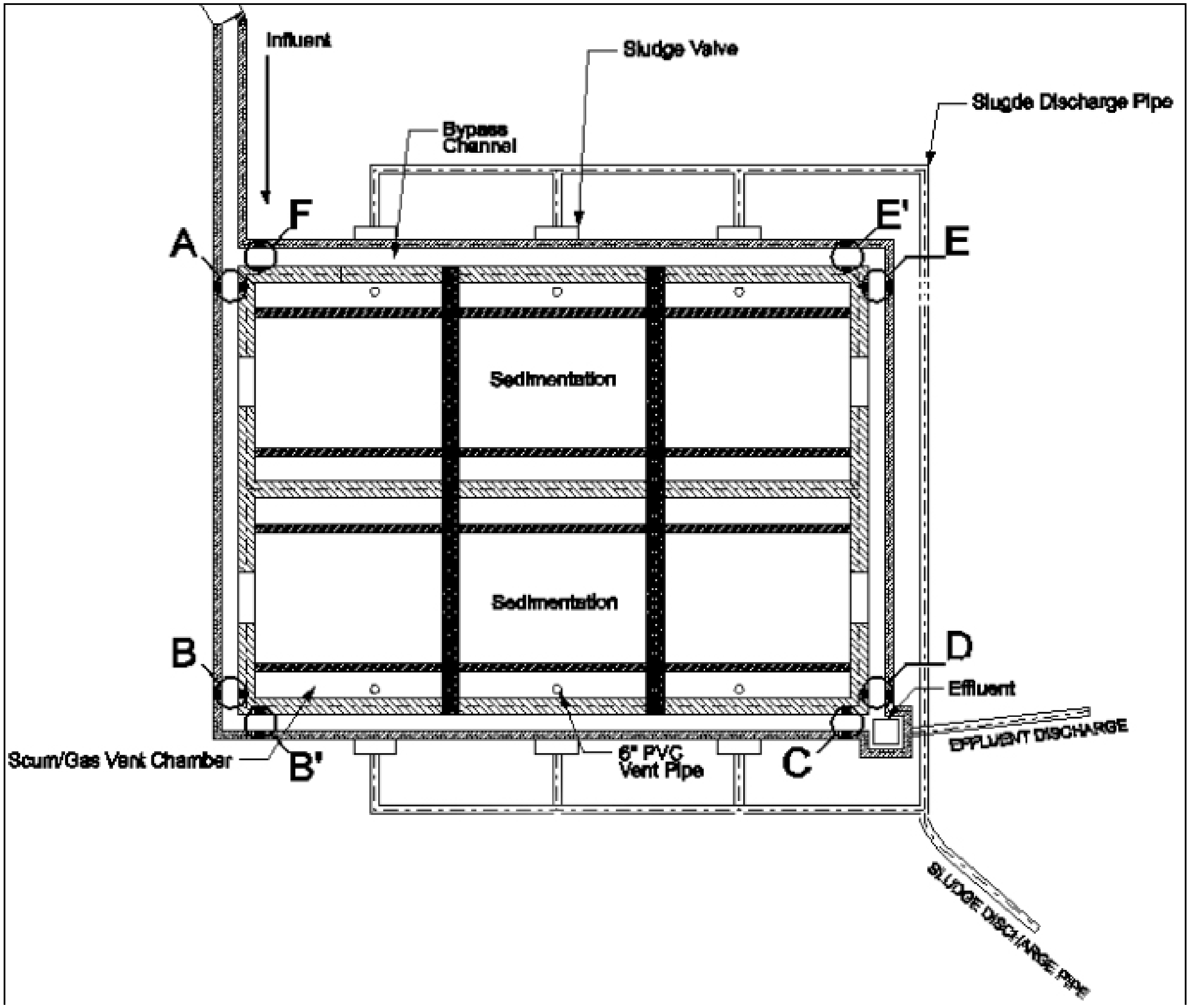

Figure 2.16: Imhoff Tank Compartments and Control Gate Locations
(Herrera 2006)

Figure 2.17: Control Flow Gate with Sand Bag to Create a Tighter Seal
(Mikelonis & Hodge, 2008)

Figure 2.18: Uneven Flow into First Battery of the Imhoff Tank
(Walker 2011)

Figure 2.19: Wooden Baffles
(Mikelonis & Hodge, 2008)

2.3.4.4 Sludge Removal

The state of sludge removal for the Las Vegas Imhoff tank is very poor. The only time the sludge is known to have been emptied was in December of 2007. The process of removing all of the sludge from the Imhoff tanks took three men about two days of work (Mikelonis & Hodge, 2008) because the sludge had to be removed by rope and buckets from the digestion chambers. The sludge was then buried along side of the Imhoff tank. The sludge had to be physically removed due to sand and other compacted soils that had clogged the removal valves. The presence of sand may have been due to the combined sewer and storm drains, in which storm runoff carried massive amounts of sand from the local unpaved roads into the Imhoff tank.

At the time of our investigation three of the six sludge valves were missing (Figure 2.20 & 2.21). It is likely that valves were stolen from the tank.

Figure 2.20: Location of Missing Sludge Valve
(Walker 2011)

Figure 2.21: Sludge Valve
(Walker 2011)

Our investigation included an interview with Santiago Hernandez, the head of the utilities group for Las Vegas, who is in charge of maintenance for the Imhoff tank. During the interview Hernandez explained the municipality's plan for removing the sludge as follows: On the southern side of the tank, where there are no operational sludge removal valves (Figure 2.20), they are planning on digging out the three locations where the sludge valves were located, and gaining access to the outlet pipes. The outlet sludge pipes converge underground and discharge into stream. Once they have dug a trench, they are going to open the pipe and let the sludge empty out of the tank. They also plan to put gates where the valves had been to control the flow of sludge from the tank (Hernandez, 2011).

It was unclear from our interview what they planned on doing with the sludge once it is removed. There is a fair amount land to the south of the Imhoff tank which could be used for a sludge drying bed. Much like in 2007, we expect that compacted sand and other soils will prevent the sludge from flowing freely. The only way for the sludge to be removed from the tank is by the rope and bucket method. It is possible that if the sludge is removed on a bi-annual basis and the sludge valves are fixed, that the sludge may flow freely from the tank.

2.3.4.5 Scum Removal

During our first visit to the Las Vegas Imhoff tank in January of 2011, it appeared that scum removal had not taken place in quite some time (Figure 2.21). After spending a few days in Las Vegas and meeting with city officials a work crew was sent out to the Imhoff tank to clean the scum. Figure 2.22 shows a picture of the channel after the cleaning.

Figure 2.22: Scum Build-up in Channel
(Walker 2011)

2.3.4.6 Maintenance

Based on the lack of scum removal, the missing baffles and control flow gates, and the missing sludge valves, it does not appear like the Las Vegas Imhoff tank is receiving regular maintenance. Also, because the system was built without a by-pass, it is very hard to perform major maintenance or repairs on the system. During our visit to the mayoral office we found that there were also no basic records kept of maintenance or repair on the system. This is consistent with the past MIT investigations. As previously mentioned, every time the mayor of the city changes, those who are in charge of the wastewater system inherit a system which they know very little about. Without consistent records it will be very hard for the new regimes to come in and properly maintain the Imhoff tank.

Figure 2.23: Scum Channel after Cleaning
(Walker 2011)

2.3.4.7 Estimated Flow Rates

To get an idea of the flow into and out of the Imhoff tank we performed a rough flow measurement using a 4.5 gallon bucket and a stop watch. We conducted six trials where we held the bucket under the effluent pipe from the Imhoff tank that discharges into the stream. We started the stopwatch when we began filling the bucket, and stopped it when the bucket was full. The volume of the bucket was divided it by the average time it took to fill up the bucket, generating an average flow of approximately 0.8 gallons per second (11 cubic meters per hour). The measurements were taken at 10:45 am on Monday, January 18th.

In January of 2008, Hodge conducted flow measurements on the Las Vegas Imhoff tank. His results showed much higher values of flow. The flow was measured to be 173 cubic meters per hour on January 17, 2008 and 170 cubic meters per hour on January 24, 2008 (Hodge 2008). Both of these values, taken at similar times in the day as our trials, were much higher than our measured flow. It is unclear what would account for the order of magnitude difference in flow.

2.3.5 Meeting with Las Vegas Officials

During our investigation we were able to meet with Joel and Adalberto who both work for the municipality of Las Vegas. They started working for the municipality only a couple of months before our visit when the new mayor was elected. They explained to us that there was no budget this year for any major wastewater treatment work, but that they were planning on conducting studies and even possibly preparing documents for bidding for a new wastewater treatment plant (Joel & Adalberto, 2011). One of the current mayor's major platforms for election was water quality and she said she would like to leave office with a new wastewater treatment system in place (Joel & Adalberto, 2011). The municipality has had visits from two different treatment plant representatives: SinHCO and Arita.

2.3.6 Las Vegas Collector System

The Las Vegas Collector system was not present during any of the other investigations by MIT students. The system discharges wastewater directly into the same receiving body (Raices Creek) that receives effluent form the Imhoff tank, only with no form of treatment. The flow for the Las Vegas Collector system was estimated using the same process as the effluent from the Imhoff tank, and it was determined that the flow was 3 gallons per second (40 meters cubed per hour) at 10:45 am on January, 18 2011. This flow is four times greater than the effluent from the Imhoff tank and could account for the discrepancy in flows between our measurements and the 2008 measurements. The collector system may receive flow which used to travel to the Imhoff tank, although we could not verify this assumption during our investigation. Whatever the case may be, this is a large amount of wastewater that is being discharged without any form of treatment. Figure 2.23 shows a picture of the effluent pipe from the collector system.

Figure 2.24: Effluent Pipe from the Las Vegas Collector System
(Walker 2011)

2.3.7 Recommendations for Las Vegas Wastewater Treatment

The major recommendation for the municipality of Las Vegas is performance of regular maintenance on their wastewater systems. We believe regular maintenance will greatly improve the quality of effluent from the Imhoff tank. It will also prevent performance of major rehabilitation just to get the system to work properly. The Imhoff tank is an incredibly simple wastewater system, requiring extremely minimal amounts of maintenance in comparison to other wastewater systems. Over the past six years, during MIT's visits to Honduras, and specifically the municipality of Las Vegas, similar recommendations have been made. However, the municipality has not been able provide consistent maintenance to the Imhoff tank during that period of time. We believe that this lack of maintenance is due to insufficient funds to pay people to maintain the tank, and also due to lack of knowledge of the system maintenance requirements.

The Imhoff tank by itself is not a sufficient means of treatment for the amount of wastewater produced by the municipality, and we believe they should eventually invest in a new wastewater system. However, we are extremely skeptical that they will be able to maintain a new system which will inevitably be more complex than the current Imhoff tank. It is our belief that before a significant amount of money is invested in a new wastewater treatment plant, the municipality should first ensure that they have the means to properly maintain the new system. Although not examined thoroughly in our study, the reduction of water use by the municipality could help with the hydraulic overloading of the Imhoff tank and could lead to better wastewater treatment.

2.4 Wastewater Treatment in Santa Barbara

2.4.1 Introduction

The municipality of Santa Barbara, Honduras is located northwest of Lake Yojoa (Figure 2.24). Santa Barbara also utilizes an Imhoff tank, built in 1998, as their form of wastewater treatment. The Imhoff tank is very similar to that of Las Vegas except that it has four batteries instead of two. The effluent from the Santa Barbara Imhoff tank discharges into the Cececapa River which does not flow into Lake Yojoa. However, the municipality of Santa Barbara does lie within the boundaries of the Lake Yojoa Subwatershed and is thus part of our study area. There has been no previous work done on the Santa Barbara Imhoff tank by MIT students.

There are approximately 4,000 homes with a total population of about 24,000 people in the municipality of Santa Barbara. Of those 4,000 homes approximately 1,000 homes are connected to the Imhoff tank (Rodriguez, 2011). The other 3,000 homes are either connected to septic tanks or to a sewer system which receives no wastewater treatment (Rodriguez, 2011). About three years ago the municipality of Santa Barbara, under the previous administration, received 2.3 million lempira to do a study on the “master plan” for wastewater in the city (Rodriguez, 2011). Two million of the lempira came from United States Agency for International Development (USAID), while the rest came from the municipality (Rodriguez, 2011). The master plan included connecting everybody in the urban Santa Barbara area to a new wastewater treatment plant. The study concluded that it would take 7.5 million U.S. dollars to complete the project (Rodriguez, 2011).

Figure 2.25: Map of Santa Barbara, Honduras
(Google 2011)

2.4.2 Description of Santa Barbara Imhoff Tank

The Santa Barbara Imhoff tank has four tanks in parallel and appears to be in good physical condition even though it was built in 1998 before Hurricane Mitch, one of the most devastating hurricanes of all time, which cost Honduras billions of dollars in damage. Figure 2.25 is a picture of the Santa Barbara Imhoff tank.

2.4.3 Assessment of Santa Barbara Imhoff Tank

2.4.3.1 Condition of Imhoff tank

The concrete of the Imhoff tank did not appear to have any structural damage. However, the site was littered with trash and resembled a dump more than a treatment plant site (Figure 2.26). Even the stream to which the Imhoff tank discharges was covered in garbage (Figure 2.27). At the time of our investigation a dead bird was found in the scum chamber. These observations imply that no minimum maintenance is being provided to the plant. The only major damage we could find on the tank was a broken bypass pipe which can be seen in Figure 2.28 and a leaky effluent pipe which can be seen in Figure 2.29. The solution to the broken bypass was to place a rock above the inlet to the bypass pipe which created a tight seal and thus prevented the water from flowing through. A more permanent solution will need to be considered in order to rehabilitate the system. The Imhoff tank also had a V-notch weir in the influent channel, but due to extreme hydraulic overloading the weir was completely submerged under the wastewater and thus unusable to determine influent flow measurements.

Figure 2.26: Santa Barbara Imhoff Tank
(Puckett 2011)

Figure 2.27: Area Adjacent to Santa Barbara Imhoff Tank
(Walker 2011)

Figure 2.28: Trash in the Receiving Stream
(Walker 2011)


Figure 2.29: Broken Bypass Pipe
(Walker 2011)


Figure 2.30: Leaky Effluent Pipe
(Walker 2011)

2.4.3.2 Missing Control Flow Gates and Baffles

Similar to the Las Vegas Imhoff tank there were no baffles or control flow gates present at the time of our site visit to the Santa Barbara Imhoff tank. This could be the reason for the incredibly uneven distribution of sludge which will be described in the next section. Until baffles and control flow gates are added to the Imhoff tank, the solids distribution will continue to be uneven. Figure 2.30 shows a picture of the slots intended to hold the control flow gates.

Figure 2.31: Slots for the Control Flow Gates
(Walker 2011)

2.4.3.3 Sludge Removal

Sludge removal is a major problem for the Santa Barbara Imhoff tank. It is unclear when, if ever, the sludge has been removed from the tank. At the time of our investigation all four tanks had extreme sludge backup. In the first and second tanks, closest to the influent channel, the sludge was only four inches from the top of the water (Figure 2.31). In tanks three and four the sludge was approximately four and five feet from the surface respectively. The sludge appeared to have a very high sand content much like the sludge from the Las Vegas Imhoff tank. This could be due to the stormwater runoff carrying sand into the system from the dirt roads. It is unlikely that, even if the sludge removal system was working properly, sludge would flow from the tank. It has probably become so compacted that the only means of removal will be by shoveling the sludge out.

Figure 2.32: Sludge Backup in the First Bay of the Imhoff Tank
(Walker 2011)

The Santa Barbara Imhoff tank was also missing all six of its sludge valves (Figure 2.32). These valves will need to be replaced in order to be able to open the sludge gates and effectively remove the sludge. It is possible that the valves may be able to be opened by a large wrench.

The Santa Barbara Imhoff tank does have adjacent area for a sludge drying bed as shown in Figure 2.33. This area is enclosed by a one foot high concrete wall, and all of the sludge removal pipes from both sides of the tank converge in this area. The sludge drying bed was a little overgrown with grass and weeds, but could be easily rehabilitated with a minimal amount of ground work.

Figure 2.33: Missing Sludge Valves
(Walker 2011)

Figure 2.34: Sludge Drying Bed
(Walker 2011)

2.4.3.4 Scum Removal

Much like the Las Vegas Imhoff tank, maintenance at the Santa Barbara Imhoff tank seems to be lacking, including removal of scum. At the time of our site visit there was a very thick layer of scum in the scum chamber as shown in Figure 2.34. A dead bird was found in of the scum chambers shown in Figure 2.35.

Figure 2.35: Layer of Scum in the Scum Chamber
(Walker 2011)

2.4.4 Meeting with Santa Barbara Official

During our investigation of the Santa Barbara Imhoff tank we were able to meet with Martin Rodriguez who works for the municipality and is in charge of the Imhoff tank. Unfortunately, Martin Rodriguez does not have the staff or the resources to spend any time working on the Imhoff tank (Rodriguez, 2011). He is very aware the tank has major problems, but with no budget to work on the system, the Imhoff tank continues to lack adequate maintenance.

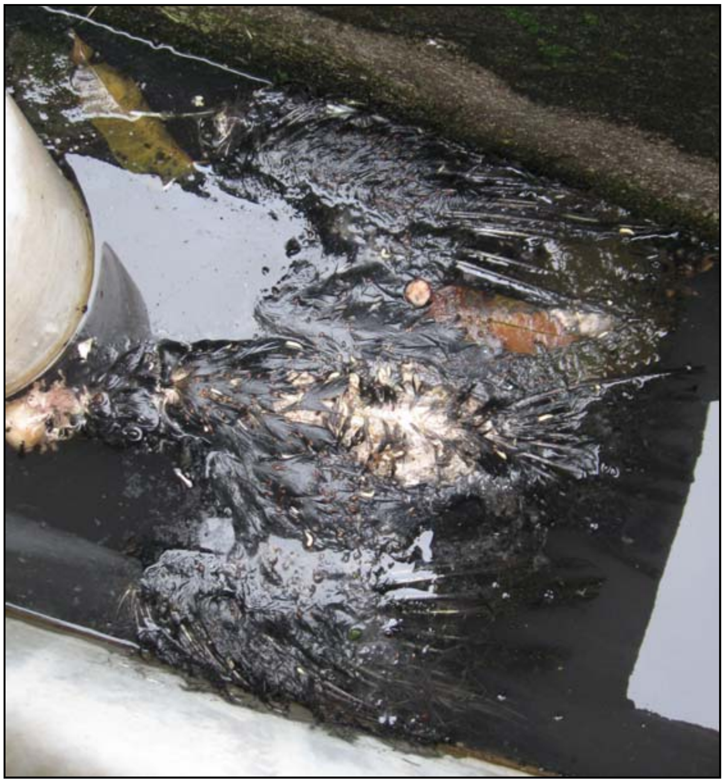

Figure 2.36: Dead Bird in the Scum Chamber
(Walker 2011)

2.4.5 Other Wastewater Issues in Santa Barbara

The Imhoff tank was not the only treatment plant in Santa Barbara. A wastewater treatment plant was built for the local hospital a few years ago; however due to extreme hydraulic overloading, the system collapsed and was no longer working (Rodriguez, 2011). We were unable to visit the system during our site investigation, but we were told that it was a combined sewer and overflow system, consisting of wastewater and stormwater. During an extreme storm event, the runoff overloaded the system and it collapsed (Rodriguez, 2011).

2.4.6 Recommendations for Santa Barbara Imhoff tank

Although the Santa Barbara Imhoff tank was not functioning properly at the time of our investigation, we feel could be easily rehabilitated because it seems to be structurally sound. In order to rehabilitate the system, the first thing that must be fixed is the broken bypass pipe. Once this pipe is fixed, the system can be bypassed and the sludge can be physically removed. However, the wastewater that bypasses the system may have to be directed in a different direction than it was intended to go. Inside the sludge drying bed there is a hole in the concrete which is meant to release the wastewater when it is bypassed. There is a small bridge for human traffic that is located directly below this hole in the sludge drying bed as shown in Figure 2.36. If the wastewater is bypassed without redirecting the flow, it is possible that the bridge could wash out.

Figure 2.37: Human Traffic Bridge Located Below the Imhoff Tank
(Walker 2011)

Once the bypass has been fixed, and the sludge removed from the tank, we believe that the Santa Barbara Imhoff tank can still provide good primary wastewater treatment effluent. The addition of control flow gates and baffles will help to equally distribute the solids throughout the four tanks. With a minimal amount of regular maintenance we see no reason why the Santa Barbara tank could not function properly. However, without financial support from the municipality of Santa Barbara which will allow for the regular maintenance, the treatment will very quickly deteriorate and go back to being a non functioning system.

Chapter 3: Las Casetas Wastewater Treatment Plant

3.1 Introduction

"Las Casetas", literally "the little houses", are a group of 51 fish restaurants along the southeast shore of Lake Yojoa. The proprietors of the restaurants catch, fry, and serve tilapia to their customers, largely tourists numbering approximately 15,000 each month (Chokshi and Trate, 2006). The relative success of this business depends on local attitudes toward water quality in the lake. During periods when the lake receives bad publicity for the apparent poor quality of its water, patronage of the Casetas tends to dwindle.

This problem of decreased business associated with negative public perceptions of water quality will likely be partially alleviated through highly visible treatment of wastewater from the restaurants. This was part of the motivation for SAG, DINADERS, and MARENA's proposal for a combined sanitary sewer system and treatment plant for Las Casetas, presented by the community board of Playas de Maria, where the restaurants are located, in March of 2006 (Patronato de Playas de Maria, 2006). The proposal was well-received and resulted in the construction of a treatment plant in 2008 and accompanying sewer system to bring waste from the restaurants to the plant. However, problems with the piping, as outlined in section 3.3, prevented the plant from ever being utilized. Since that time it has merely sat, abandoned, as a remnant of another failed public works project.

Our investigation during January convinced us that the project was not a failure, but rather held great potential for rehabilitation. The treatment plant appeared to be in good shape, and it seemed that rehabilitating or replacing the piping system may be enough to allow the project's original goals to be met. What follows is an analysis of the system as it currently stands, a requisite first step in determining how to move forward with the Las Casetas treatment plant and sewer system.

3.2 PRONADERS

PRONADERS is the agency in charge of Las Casetas wastewater treatment plant rehabilitation. Within PRONADERS, there is a single engineer, Jessica Castillo, in charge of infrastructure around Lake Yojoa. PRONADERS is currently in charge of procuring the funding for the completion of Las Casetas treatment plant, although government approval is expected to take another 4-5 months from the time of our visit (Castillo, 2011). Management of Las Casetas project will also fall to PRONADERS; they plan to keep the existing design with a few small changes, including covering the existing pipe with concrete and making some alterations to the collection tank (Castillo, 2011). Our research will hopefully help to inform these decisions. PRONADERS also hopes to teach Las Casetas community how to properly maintain the system; the company that wins the bidding process will be responsible for this aspect of the project. The restaurant owners will be expected to pay a fee to cover repairs and maintenance. They will also be expected to guard against theft of the pumps, which have not yet been installed for this very reason. Furthermore, the restaurant owners will be involved in the process of rehabilitating the plant, including connecting the piping system.

PRONADERS also expects that AMUPROLAGO will be very involved in this project moving forward; HONDULAGO, on the other hand, does not have any presence but may play a role in regulating the water once it is completed.

3.3 Las Casetas Sewer and Piping System

When Las Casetas wastewater treatment plant was built in 2008, the piping system to convey the wastewater from the restaurants to the plant was never connected, and thus the treatment plant has been sitting unused for over two years. Figure 3.1 shows a picture of the black influent pipe, and the white effluent pipe that lie just outside of the fenced in area that contains the treatment plant. From our initial analysis of the actual treatment plant, which will be introduced in Section 3.8, the treatment plant seems to be a viable system that, if connected, would provide a decent level of treatment to the incoming wastewater. Until the piping and distribution system is fixed, the plant will continue to sit unused.

Figure 3.1: Influent and Effluent Pipes just Outside the Treatment Plant
(Walker 2011)

3.3.1 Current Condition

The piping system, in its current condition, is completely unusable. There are many sections of pipe that are missing, and the pipes that are there are not connected to anything. Of the pipes that are still in place, many of them have sagged and would be unable to convey water even if they were connected. Also, the pipes used were not specified in the design documents (see Section 3.4.7 for further detail on pipe specifications). It is unclear if any design changes were made to allow the current type of piping used, though it seems highly unrealistic that such a change would have been authorized. From interviews conducted during our investigation the sagged pipes seem to be the main reason the system was never connected to the treatment plant. It appears that the type of piping which was installed was not strong enough to withstand the ultraviolet rays from the sunlight, and soon after construction began the pipes sagged. This sagging could cause the flow to back up and clog the pipes.

In January of 2010, when it was evident to the owners of the restaurants that the piping system was not going to be finished, a letter was sent to Engineer Carlos Montes, whose company installed the pipes, stating that the owners of the restaurants, along with members of DINADERS, were going to remove some of the piping that was getting washed away by the waters of the lake, and take inventory of all the pipes and parts that were removed (FERRUFINO, 2010). This is why during our investigation we found that many sections of pipe were missing. We could not determine where the stock pile of piping and parts were located and if this material remained onsite. Figure 3.2 illustrate the current problems with the piping system.

Figure 3.2: Current Condition of Piping System
(Walker 2011)

There are a few locations where the pipe was connected to effluent pipes from the houses (Figure 3.3). These pipes coming from the houses were a four inch PVC pipe which is consistent with the design specifications (Patronato de Playas de Maria, 2006). It did not appear that these effluent PVC pipes from the houses were carrying any wastewater at the time of our investigation due to the absence of water flowing from the pipes and lack of evidence that any water had traveled through the pipes. It is unclear what, if anything, these PVC pipes were connected to.

Figures 3.3: Pipe Connections
(Walker 2011)

There were concrete posts built to support the piping system which was one of the methods specified to support the pipe and maintain the slope required for gravity to deliver the wastewater to the distribution box (Figure 3.2). The other method used was the attachment of pipes directly to the houses (Figure 3.4). Both of these methods are specified in the design documents (Patronato de Playas de Maria, 2006). During our investigation, Mario Santos, the president of the restaurant association, said that all of the residents were willing to allow these pipes to be attached to their houses if it could help the project (Santos, 2011). Attaching the pipes to the houses could be useful in some situations and may save construction costs if additional concrete columns do not have to be built.

Figures 3.4: Pipe Connected to House
(Walker 2011)

3.3.2 Condition of Distribution Tank

At the end of the piping system, approximately 500 meters from the treatment plant, a distribution tank was built (Figure 3.5). The tank seems to be structurally sound but, like the rest of the piping system, it is currently not connected to anything. The investigation and interviews determined that there were some problems with the tank. It was rumored that the tank had actually been lifted up from its original position by a buoyancy force due to the high water level of the lake (Galeana, 2011). White staining can be seen on the concrete tank (Figure 3.5) where the high water in the lake has been over the past couple of years. We could not find any physical evidence that the tank had ever popped up. However, it is possible that the tank could have been moved backed to its original place after having been lifted up by the buoyancy force from the lake water. The buoyancy of the tank during the high water levels in the lake will be addressed in the design section. Another possible problem with the distribution tank is the entry location of the pipe. Based on the height of the water staining on the tank, the influent wastewater pipe is below the water level of the lake for some period of time during the year. This means that the influent pipe must be well sealed in order to avoid intrusion of lake water into the tank, or the possibility of untreated wastewater escaping from the tank.

Figure 3.5: Distribution Tank
(Walker 2011)

3.3.3 Design Documents

The majority of our design documents were obtained by Mario Santos, the president of the Las Casetas restaurant association (Santos, 2011). Mr. Santos had compiled all of the documents he had and gave them to us electronically for our review. During our meeting with Jessica Castillo from PRONADERS, we were able to verify that we had the correct design documents (Castillo, 2011). The final design document is titled:

SECRETARIA DE AGRICULTURA Y GANADERIA - SAG

DIRECCIÓN NACIONAL DE DESARROLLO RURAL SOSTENIBLE DINADERS

PROGRAMA MULTIFASE DE MANEJO DE RECURSOS NATURALES EN CUENCAS PRIORITARIAS - MARENA

DISEÑO FINAL

SISTEMA DE ALCANTARILLADO SANITARIO

CON PLANTA DE TRATAMIENTO

Comunidad de Playas de María

Municipio de Santa Cruz de Yojoa, Cortés

Código LY-131

SUBCUENCA DEL LAGO DE YOJOA

Presentado por el Patronato de Playas de Maria

We translated the design documents into English for our analysis. Also included in the documents that we received from Mr. Santos was a set of AutoCAD drawings for the design of the piping system. These drawings can be found in Appendix A. From the final design documents and the AutoCAD drawings of the piping system, we set out to determine if the design of the piping system was adequate, and if what was built matched what was called for on the design.

3.4 Piping System Design

3.4.1 Design Flows

The design flows for Las Casetas treatment plant were calculated based on the population estimates that would be served by the system. Table 3.1 contains the population estimates used for the design flows.

Table 3.1: Population Estimates for Las Casetas Treatment Plant
(Patronato de Playas de Maria, 2006)

Population Estimates				
Location	*Population Density (ppl/house)*	*National Institute of Statistics*		*Population*
		2001 Inhabitants	*2006 Inhabitants*	*2026 Inhabitants*
Playas de Maria	6	308	348	571

The increase of population recorded from 2001 to 2006 was approximately 2.5% per year. This annual increase of 2.5% was assumed to be constant for the next twenty years and was used to calculate the estimated population for 2026. These population estimates listed in Table 3.1 were for the permanent residents only. Due to the limited amount of area for houses and business on the shore of the lake, our assumption is that this population increase is conservative. Unless other houses or businesses are built, the population density of the residents would need to increase in order to hold the estimated 571 permanent residents in 2026. The number of houses and businesses at the time of the study was 58.

In addition to the fixed population of permanent residents, a floating population consisting of visitors to the restaurants and houses was considered. It was determined through observation that approximately 3,000 visitors per week visited the 58 houses and businesses of Las Casetas (Patronato de Playas de Maria, 2006). It was also approximated that 80% of that floating population was concentrated on the weekends (Patronato de Playas de Maria, 2006). Table 3.2 summarizes the current and floating population estimates. Also included in Table 3.2 are the final set design population and floating design population used for the calculations of the wastewater flow.

Table 3.2: Design and Floating Population Used for Flow Calculations
(Patronato de Playas de Maria, 2006)

Current Population (December 2005)	
Number of current houses	58
Inhabitants per house	6
Total	348
Growth rate	2.50%
Floating Population Estimate	
Recorded weekly average	3000
Weekdays (20% of weekly average)	120
Weekends (80% of weekly average)	1200
Daily weighted	429
Design Period	20 years
Projection Method: Arithmetic	
$P_f = P_i \text{ x } (1 + i \text{ x } n/100)$	
Fixed Design Population (year 2026)	522
Floating Design Population (year 2026)	643.5

`

The difference between the set design population of 522 and the original population estimate 571 for the year 2026 is unclear. From the arithmetic equation given using an *i* value of 2.5%, an *n* value of 20 years, and a P_i value of 348, the final fixed population is equal to 522. The floating population of 3000 was split into 20% on weekdays and 80% on weekends. However, the daily average of 429 people, for the floating population, was calculated simply by dividing the 3,000 visitors by the seven days of the week. The floating population estimate for 2026 was also calculated by the arithmetic method using the daily average of 429 people for *Pi* and the same *n* and *i* values used for the fixed design. The fixed and floating design populations used for the calculation of the design flow are listed in the bottom of Table 3.2.

3.4.2 Quantity of Water Used Per Person

The quantity of water used per person was estimated by examining the current functioning of the drinking water system. It was estimated that the permanent residents use 200 liters per person per day and the floating population uses 75 liters per person per day (Patronato de Playas de Maria, 2006).

3.4.3 Contributions to the System

Sewage flow is mainly composed of four different sources of water: wastewater, groundwater infiltration, stormwater runoff, and illegal or illicit connections to the system. For the purpose of the design it was assumed that there was only the contribution of wastewater to the system (Patronato de Playas de Maria, 2006). This seems like a valid assumption do to the limited size of the project and the ease of examining all of the connections and contributions. Also, per the design specifications, the PVC or HDPE pipes that were specified in the design have very low infiltration rates and more than 50% of the pipes are elevated above the ground (Patronato de Playas de Maria, 2006). The possibility of infiltration should be considered in the final design. However, based on our conservative estimate of the peak flow in Section 3.4.4, we feel we have been very conservative in our analysis of the piping system and the treatment plant.

3.4.4 Design Flows

The design flow, which is assumed to be an average flow, was calculated using the fixed and floating population estimates from the year 2026 (Table 3.2) and the estimates for the quantity of water used per person. Since the design flow was estimated based on average daily consumption of water, it was assumed that 80% of the water used enters the sanitary sewer (Patronato de Playas de Maria, 2006). Table 3.3 contains a summary of the design flows.

Table 3.3: Design Flow Summary
(Patronato de Playas de Maria, 2006)

Calculated Design Flow		
Set Design Population	522	
Floating Design Population	643.5	
Fixed Population Allocation	200	lts/p/day
Floating Population Allocation	75	lts/p/day
Daily Expenditure	152,662.50	lts/day
Daily Variation Coefficient	1.5	
Time Variation Coefficient	1.5	
Supply Flow	343,490.63	Liters/day
Rate of Return	80%	
Design Flow (Q) =	274,792.50	lts/day
	3.18	lts/sec

Included in the calculation of the design flow are a daily variation coefficient and a time variation coefficient (Patronato de Playas de Maria, 2006). It is unclear where either of these two coefficients comes from and there is no explanation in the design documents why these two coefficients were used. We assume that these factors are related to the variation of people per day (weekday or weekend) and the variation of people per time of the day. It would make sense that floating population is concentrated over the lunch and dinner hours with very little contribution during other times. Based on the assumptions of an 80% rate of return of water into the sewer system and the two variation coefficients, we verified that the calculated value of 3.18 liters per second was correct. We assume that this calculated design flow is an average flow for the year 2026 although it is never explicitly stated as an average flow in the design documents. There is also no mention of peak flow or low flow in the design documents. Generally when wastewater systems are designed there is a peak, average, and minimum flow calculated for the current date and the estimated design life of the system.

3.4.5 Peak Flow Calculations

Because of the assumption that the design flow is an average flow, we estimated a peak flow for the purpose of validating the design. For this peak flow estimate the following assumptions were made by our team:

- The peak amount of water used per person occurs during the dinner time hours of a weekend day.
- Of the 200 liters per person per day of the fixed residents, 110 liters are used per person during an assumed four hour dinner period. We believe this is a high estimate, but it could be valid for all of the water needed to cook and clean the food which is prepared for dinner. Also, the washing of dishes will require the use of significant of water.
- Of the 75 liters per person per day for the floating population, 60 liters are used during an assumed four hour dinner period. This assumption was made based on the comparative popularity of the restaurants for dinner versus lunch.
- It was estimated from the 80% weekend floating population that approximately 1,200 people visit the restaurants per weekend day.
- It was assumed that 100% of the water used enters the sewer system.

From these assumptions a peak flow estimate of 9.0 liters per second was calculated. This peak flow value will be used to determine the viability of the current design for the piping system as well as the wastewater treatment plant. Again, we believe this estimate to be very high and thus conservative for the evaluation of the design.

3.4.6 Topography and Physical Characteristics

The topography, longitudinal distances, and available head are of extreme importance for the piping system design. Based on the construction documents on the AutoCAD files, the piping system is approximately 1.1 km long with 3.35 meters of available head. The construction documents call for a constant slope of 0.305% over the entire length of the system (Patronato de Playas de Maria, 2006).

3.4.7 Piping Specifications

The pipe specifications in the design documents call for PVC or HDPE piping. PVC is a cheap and durable plastic piping that has many construction uses. There are two PVC pipe wall thicknesses: schedule 40 and schedule 80. The thickness of the PVC pipe was not specified in the design documents. HDPE or High-density polyethylene is a similar type of piping which is a little stronger and more expensive than the PVC piping. Both of these pipes were selected based on their low roughness coefficient because of the minimal amounts of hydraulic head available. PVC has the advantage of being more available and cheaper, but when exposed to ultraviolet rays it can deteriorate quickly. Due to this consideration it called for periodically coating the PVC with a protective paint (Patronato de Playas de Maria, 2006). HDPE does not have the same drawbacks since it is protected by a black membrane.

The type of piping used for construction was a black corrugated pipe that did not match either of the design specifications (Figure 3.6). This piping was not strong enough to withstand the ultraviolet rays from the sun, and after being exposed to the sun for some time, the pipe sagged and became unusable. This black corrugated piping is significantly less rigid than the PVC and HDPE piping. With the small slope, it would be nearly impossible to keep this pipe from sagging over long distances without the addition of a massive amount of support columns or attachments to the houses.

3.4.8 Velocity of Wastewater in the Piping System

SANAA standards stipulate a maximum speed of 3.00 meters per second and a minimum speed of 0.60 meters per second (Patronato de Playas de Maria, 2006). Due to the small slope available it was determined that a minimum velocity of 0.50 meters per second would be acceptable (Patronato de Playas de Maria, 2006). These standards were created to insure that solid materials would not accumulate in the piping system. Locations to provide cleaning to the system will be installed at shorter increments as a precautionary measure due to the possibility of low velocity wastewater (Castillo, 2011).

Figure 3.6: Black Corrugated Pipe
(Walker 2011)

3.4.9 Junction Box Cleaning

It is unclear from the design documents what the normal spacing is for access to clean the piping system. The construction documents label only three locations for junction box cleaning. During our interview, Jessica Castillo from PRONDANDERS stated that the cleaning device for the pipes could be used up to 50 meters in each direction. Based on this she said they were going to install junction cleaning boxes, which are access points to clean the pipes, every one hundred meters of the piping system (Castillo, 2011).

3.4.10 Pipe Sizing

There are two different diameter pipes specified on the construction plans. The first section is a four inch diameter pipe that runs from the furthest upstream point for a distance of 400 meters. After that, the plans call for a six inch diameter pipe to run the remaining 700 meters to the distribution box. It is reasonable to use a smaller diameter pipe upstream because the flows will be significantly lower in that reach.

3.5 Design Verification

During our investigation on the Las Casetas treatment plant we tried to be as thorough as possible in acquiring all necessary information and obtaining all design and construction documents. We believe we have every document available to us, although there is the possibility that there exist other construction documents. It is our goal to access the design that we have and to verify if it is a good design. It was apparent from our site investigation that the construction does not match the specifications in the design documents and construction plans, with the main difference being the type of piping used. It is our opinion that the black corrugated pipe which was used will not suffice and that PVC or HDPE piping must be used unless other measures are taken to prevent the black corrugated pipe from sagging.

3.5.1 Available Hydraulic Head

One of the major concerns of the piping system is the minimal amount of hydraulic head available to drive the flow of the wastewater. The design parameters are as follows:

- 1.1 kilometers long
- 3.35 meters of available head
- Constant pipe slope of 0.305%

We believe that the constant slope is the best way to approach the construction. The elevation of the piping system will need to be very precise to maximize the amount of available head, and changing the slope down the length of the piping system will be a very difficult construction task.

3.5.2 Head Loss Due to Friction

The first step in verifying the design is to calculate the head loss due to friction in the pipes (minor losses are addressed later). This was done using the Darcy-Weisbach equation:

$$h_f = f\frac{L}{D}\frac{V^2}{2g} = f\frac{L}{4R_h}\frac{V^2}{2g}$$

where h_f is the head loss due to friction, V is the velocity of the fluid, D is the diameter of the pipe, R_h is the hydraulic radius, L is the length of the pipe, f is the friction factor and g is the acceleration due to gravity (Crowe, Elger, & Roberson, 2005). A priori it is unknown if the pipes would be flowing full and thus have the possibility of being pressurized. Due to the small flows it was assumed that the pipe would not be flowing full. The calculations were made under this assumption and four times the hydraulic radius was substituted into the Darcy-Weisbach equation for the diameter of the pipe.

To determine if the head loss due to friction was greater than the available head, a spreadsheet was created in which the head loss due to friction, and other minor losses, would be equated to the available head. To do this there were two different cases that must be considered: a pipe flowing less than half full and a pipe flowing more than half full. The flow (Q), the area (A) and diameter (D) of the pipe were known. Next, the area of the water flowing through the pipe (A_w) must be determined in order to calculate a velocity. The area of the water flowing through the pipe can be set up as a function of the depth of water in the pipe (d) and the diameter of the pipe. In order to calculate the hydraulic radius, which is needed because of the assumption that the pipe is flowing less than full, the wetted perimeter (W_p) must be determined.

$$R_H = \frac{A_w}{W_p}$$

$$W_p = f(d, D)$$

The wetted perimeter can also be determined as a function of the depth of water in the pipe and the diameter of the pipe. Once all of the unknowns are a function of the depth of water in the pipe and the diameter of the pipe, the head loss due to friction can be equated to the available head to determine if it is sufficient. The available head is equal to the slope of the pipe (S) multiplied by the length of the pipe (L).

$$Available\ head\ (h) = LS$$

From this equation it is now possible to set the head loss due to friction equal to available head.

$$LS = f\frac{L}{4R_h}\frac{V^2}{2g}$$

The last unknown variable in the equation is the friction factor (f). The friction factor is generally determined from the Moody diagram which relates the friction factor to the Reynolds number (Re), while also incorporating the relative roughness. The relative roughness is determined from the equivalent sand grain roughness (k_s) and the diameter of the pipe (Crowe, Elger, & Roberson, 2005). Again, since the pipe is assumed to be flowing less than half full, four times the hydraulic radius is substituted for the diameter. The Reynolds number is a function of velocity, diameter and kinematic viscosity (ν).

$$Re = \frac{VD}{\nu} = \frac{V(4R_h)}{\nu}$$

$$Relative\ Roughness = \frac{k_s}{D} = \frac{k_s}{4R_h}$$

Because the velocity and the hydraulic radius can be determined as a function of the height of water flowing in the pipe and the diameter of the pipe, the only other value needed is the kinematic viscosity. For this analysis a value for the kinematic viscosity of water at 20° Celsius was used (Crowe, Elger, & Roberson, 2005).

$$\nu = 1.00\ X\ 10^{-6}\,\frac{m^2}{s}$$

In order to calculate through iteration the values of the hydraulic head related to the height of wastewater flowing in the pipe, it is convenient to use an equation for the friction factor instead of reading the value off of the Moody diagram for each iteration. Swamee and Jain came up with an equation that relates the friction factor to the equivalent sand grain roughness, the diameter, and the Reynolds number (Crowe, Elger, & Roberson, 2005). Once again, the diameter is substituted by four times the hydraulic radius.

$$f = \frac{0.25}{\left[log_{10}\left(\frac{k_s}{3.7D} + \frac{5.74}{Re^{0.9}}\right)\right]^2} = \frac{0.25}{\left[log_{10}\left(\frac{k_s}{3.7(4R_h)} + \frac{5.74}{Re^{0.9}}\right)\right]^2}$$

For the following values of the Reynolds number it is reported that the friction factors vary by less than 3% from those on the Moody diagram (Crowe, Elger, & Roberson, 2005).

$$4\ X\ 10^3 < Re < 10^8$$

The values for the Reynolds number and friction factor determined through this analysis will be verified to determine if the Swamee and Jain equation is valid for this investigation. The value of the equivalent sand grain roughness must also be determined. The equivalent sand grain roughness for new PVC or HDPE pipe is considered smooth (Crowe, Elger, & Roberson, 2005). However, after a long period of use the pipe could have sludge build up that would cause the equivalent sand grain roughness to increase. To account for the possibility of sludge build up a value of k_s equal to 1 millimeter, which is typical for a concrete pipe, will be used for analysis (Crowe, Elger, & Roberson, 2005).

The last thing to consider before making the calculations is other minor losses due to bends and transitions. The Darcy-Weisbach equation can be written as follows to include the addition of other minor losses (Crowe, Elger, & Roberson, 2005).

$$h_f = f \frac{L}{4R_h} \frac{V^2}{2g} = \frac{V^2}{2g} \left[\frac{fL}{4R_h} + \Sigma K \right]$$

The term (ΣK) is used to represent the sum of all the various minor losses. In our calculations there will be three types of losses considered: loss due to expansion of the pipe, loss due to pipe entrances from the houses, and loss due to bends.

There are three different bends specified in the construction documents: 90°, 45°, and 22.5°. There is also one expansion from a four inch to a six inch pipe. The values of the respective minor loss coefficients are listed below in Table 3.4. There are no specifications for the radius of the bends so the worst case scenario is used for each bend. The 90° bend is assumed to be a miter bend with no vanes which has a K value of 1.1 (Crowe, Elger, & Roberson, 2005). The values of K for the 45° bends and the 22.5° bends are taken as three quarters and one half of the 90° bend K value respectively (King, 1954). The pipe expansion loss coefficient is a function of the ratio of the two diameters and the angle of the expansion. Since the angle of expansion is not specified it is assumed to be 180 degrees (i.e., sudden expansion) and the ratio of the diameters is 0.67. From these two values a K of 0.41 will be used (Crowe, Elger, & Roberson, 2005). The value of K for each pipe entrance from the houses is assumed to be 0.5 (Tapley, 1990).

Table 3.4: Various Loss Coefficients
(Walker 2011)

Loss Coefficients		
90° Bend	K =	1.1
45° Bend	K =	0.825
22.5° Bend	K =	0.55
Pipe Expansion	K =	0.41
Pipe Entrance	K =	0.5

3.5.3 Friction Loss Calculations

For ease of calculations the piping system was split up into eleven equal length segments all with a constant slope of 0.305% and a length of 100 meters. The flow for each section was set equal to the fraction of homes for that particular section. For example, for section eleven, which goes from 1,100 meters upstream to 1,000 meters upstream, one eleventh of the flow was used. For section ten, which goes from 1,000 meters upstream to 900 meters upstream, two elevenths of the flow was used. Because all of the equations introduced in the previous section were functions of flow, height of water in the pipe, and diameter of the pipe, an initial value for the height of water in the pipe must first be assumed. Also, based on this estimated height, there were two different geometries for the pipe. Either the pipe was flowing more than half full or less than half full. Once a value of the height of water in the pipe was assumed, the Excel spreadsheet could solve for the final value through iteration using the Solver function. If the final value of water in the pipe did not match the original assumption of water flowing more or less than half full, the spreadsheet would return an error and the section could be recalculated using the proper assumption.

The minor losses were added to the calculations based on the section of pipe analyzed. All of the bends and expansions were specified on the construction documents so they could be broken up into their respective pipe section. The pipe entrances were broken up equally over the eleven pipe sections; each 100 meter section was assumed to have a total of five entrances.

For each section of pipe, the spreadsheet would return a value of the height of water in the pipe, the velocity of the water in the pipe, the friction factor, and the Reynolds number. From these values the initial assumptions of using the Swamee and Jain equation could be validated and the

Reynolds number could be checked using the Moody diagram. For each section of pipe these values were calculated using the design flow of 3.18 liter per second (which is assumed to be an average flow) and our estimated peak flow of 9 liters per second.

3.6 Results

3.6.1 Available Hydraulic Head

Based on our design verification using the Darcy-Weisbach equation and the design and estimated peak flows, it appears that there is enough available hydraulic head to drive the flow of the wastewater to the distribution box. Based on our conservative estimates of the head loss due to friction and other minor losses such as bends, expansions, and entrances, we believe the preliminary design for the piping system will work.

3.6.2 Friction Factor

As can be seen in Tables 3.4 and 3.5 the friction factor obtained from the Moody diagram is very similar to the friction factor calculated from Swamee and Jain equation. All of the values of the Reynolds number, for both the peak and design flows, are within the range reported to produce a value of f within 3% of that which would be obtained from the Moody diagram. The friction factor is a function of the diameter of the pipe, the Reynolds number, and the equivalent sand grain roughness. We believe, based on our conservative estimate of k_s, that the friction factor is also a conservative estimate.

3.6.3 Minor Losses

The sum of the minor losses (ΣK) included in our analysis is much smaller than the value of ($fL/4R_h$) for each pipe section for both the design and peak flows (Tables 3.5 & 3.6). In the Darcy-Weisbach equation, re-written to include the sum of the minor losses, the values of ($fL/4R_h$) and ΣK are added and multiplied by the velocity squared divided by two times the acceleration due to gravity to find the total head loss. This means that the head loss is more dependent on the friction factor than other minor losses. The value of the ΣK varies from 3% to 23% of the value of ($fL/4R_h$).

$$h_f = \frac{V^2}{2g}\left[\frac{fL}{4R_h} + \Sigma K\right]$$

Table 3.5: Calculation Summary for Design Flow (Q = 3.18 l/s)

Section	*Stations*	*Flowing More or Less Than Half Full*	*Pipe Diameter (in)*	ΣK	$fL/4R_h$	*Flow (l/s)*	*Velocity (m/s)*	*Height of Wastewater in Pipe (in)*	*Re*	*Calculated f*	*Moody Diagram f*
1	1 + 000 to 1 + 100	Less	6	3.60	26.76	3.18	0.44	2.5	59,401	0.036	0.035
2	0 + 900 to 1 + 000	Less	6	4.15	28.10	2.89	0.43	2.4	55,604	0.036	0.036
3	0 + 800 to 0 + 900	Less	6	2.50	30.24	2.60	0.43	2.2	52,236	0.037	0.036
4	0 + 700 to 0 + 800	Less	6	3.60	32.07	2.31	0.41	2.1	47,947	0.038	0.037
5	0 + 600 to 0 + 700	Less	6	5.25	34.25	2.02	0.39	2.0	43,435	0.038	0.037
6	0 + 500 to 0 + 600	Less	6	4.70	37.67	1.73	0.38	1.8	39,101	0.039	0.038
7	0 + 400 to 0 + 500	Less	6	6.35	41.60	1.45	0.35	1.7	34,205	0.040	0.039
8	0 + 300 to 0 + 400	Less	4	2.50	45.50	1.16	0.35	1.7	31,985	0.041	0.040
9	0 + 200 to 0 + 300	Less	4	3.05	53.37	0.87	0.33	1.5	26,293	0.043	0.041
10	0 + 100 to 0 + 200	Less	4	3.05	67.71	0.58	0.29	1.2	19,800	0.046	0.045
11	0 + 000 to 0 + 100	Less	4	3.05	103.38	0.29	0.24	0.8	11,985	0.052	0.052

Table 3.6: Calculation Summary for Estimated Peak Flow (Q = 9.0 l/s)

Section	*Stations*	*Flowing More or Less Than Half Full*	*Pipe Diameter (in)*	*ΣK*	*$fL/4R_h$*	*Flow (l/s)*	*Velocity (m/s)*	*Height of Wastewater in Pipe (in)*	*Re*	*Calculated f*	*Moody Diagram f*
1	1 + 000 to 1 + 100	More	6	4.15	17.84	9.00	0.52	5.4	94,542	0.032	0.032
2	0 + 900 to 1 + 000	More	6	3.60	17.42	8.18	0.53	4.7	98,450	0.032	0.032
3	0 + 800 to 0 + 900	More	6	2.50	17.99	7.36	0.54	4.2	97,231	0.032	0.032
4	0 + 700 to 0 + 800	More	6	3.60	18.76	6.55	0.53	3.9	91,866	0.033	0.032
5	0 + 600 to 0 + 700	More	6	5.25	19.45	5.73	0.49	3.7	83,663	0.033	0.033
6	0 + 500 to 0 + 600	More	6	4.70	21.03	4.91	0.48	3.3	77,260	0.034	0.033
7	0 + 400 to 0 + 500	Less	6	6.35	22.77	4.09	0.45	3.0	68,496	0.035	0.035
8	0 + 300 to 0 + 400	More	4	0.00	33.45	3.27	0.41	3.9	46,273	0.038	0.037
9	0 + 200 to 0 + 300	More	4	0.00	31.38	2.45	0.42	2.7	49,575	0.037	0.038
10	0 + 100 to 0 + 200	More	4	3.05	37.70	1.64	0.38	2.1	39,815	0.039	0.039
11	0 + 000 to 0 + 100	Less	4	3.05	55.20	0.82	0.32	1.4	25,259	0.044	0.043

3.6.4 Height of Wastewater in Pipe

Figure 3.7 contains the height (head) of the wastewater flowing in the pipes based on our analysis using the Darcy-Weisbach equation for both the design and estimated peak flows. Section One refers to the furthest downstream reach that will connect to the distribution box, and Section Eleven refers to the furthest upstream reach at the start of the piping system. Pipe sections one through seven are six inch diameter pipes, and pipe sections eight through eleven are four inch diameter pipes. Looking at this graph the wastewater would be flowing right to left with the highest amounts of wastewater flow in the downstream sections. As can be seen from the graph, the values of the height of the wastewater flowing in the pipe all seem to be very reasonable. However, neither case accounts for the possibility of low flow which would be expected to have much lower heights and could possibly cause problems with solids backing up in the pipes.

3.6.5 Velocities

Figure 3.8 shows the estimated velocities of the wastewater based on our analysis using the Darcy-Weisbach equation for the design and estimated peak flows. The sections of the pipes are consistent with Figure 3.7 with the wastewater flowing right to left on the graph. Also plotted on this graph is a line for the minimum velocities requirements used for the design of this system. The majority of pipe sections fall below this line, especially those values that are furthest upstream from the distribution box. This is not the worst case scenario for the low velocity. A minimum flow would need to be used to determine the lowest possible velocity for the system. Because of our conservative estimate of k_s, the velocities of the system were approximately 30-40% lower than if we had assumed that the piping was smooth. In the case of a smooth pipe, many of the velocities would exceed the minimum velocity standard used for design.

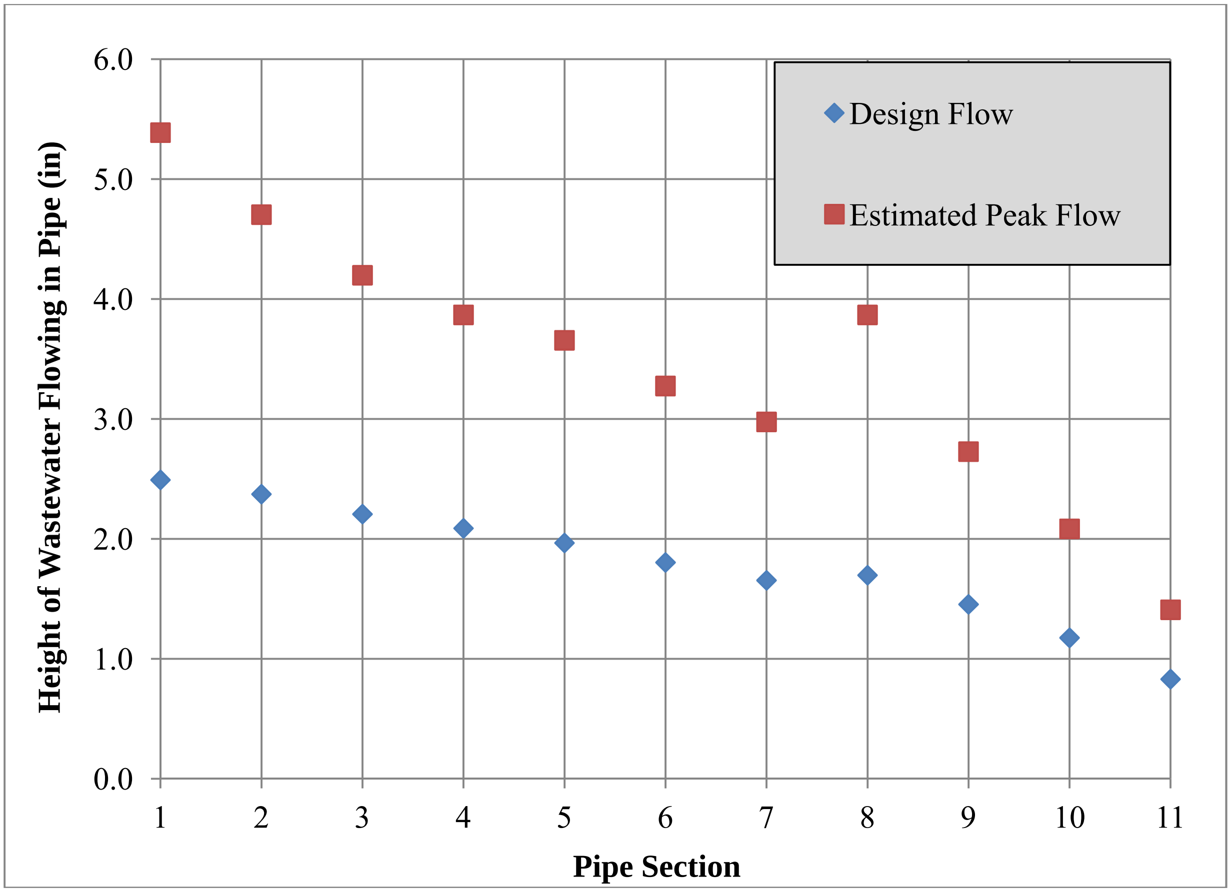

Figure 3.7: Estimated Height of Wastewater Flowing in Pipe Calculated From the Darcy-Weisbach Equation

It is our belief that it will be impossible to meet this minimum velocity standard at low flow conditions. A negative effect of not meeting the minimum velocity requirements could be increased possibility of clogging in the piping system due to the solid material in the wastewater. This effect may be mitigated by the addition of more cleaning boxes or an upstream pumping system designed to flush the pipes. Natural peak flows may also be strong enough to flush the piping system and decrease the susceptibility of clogging.

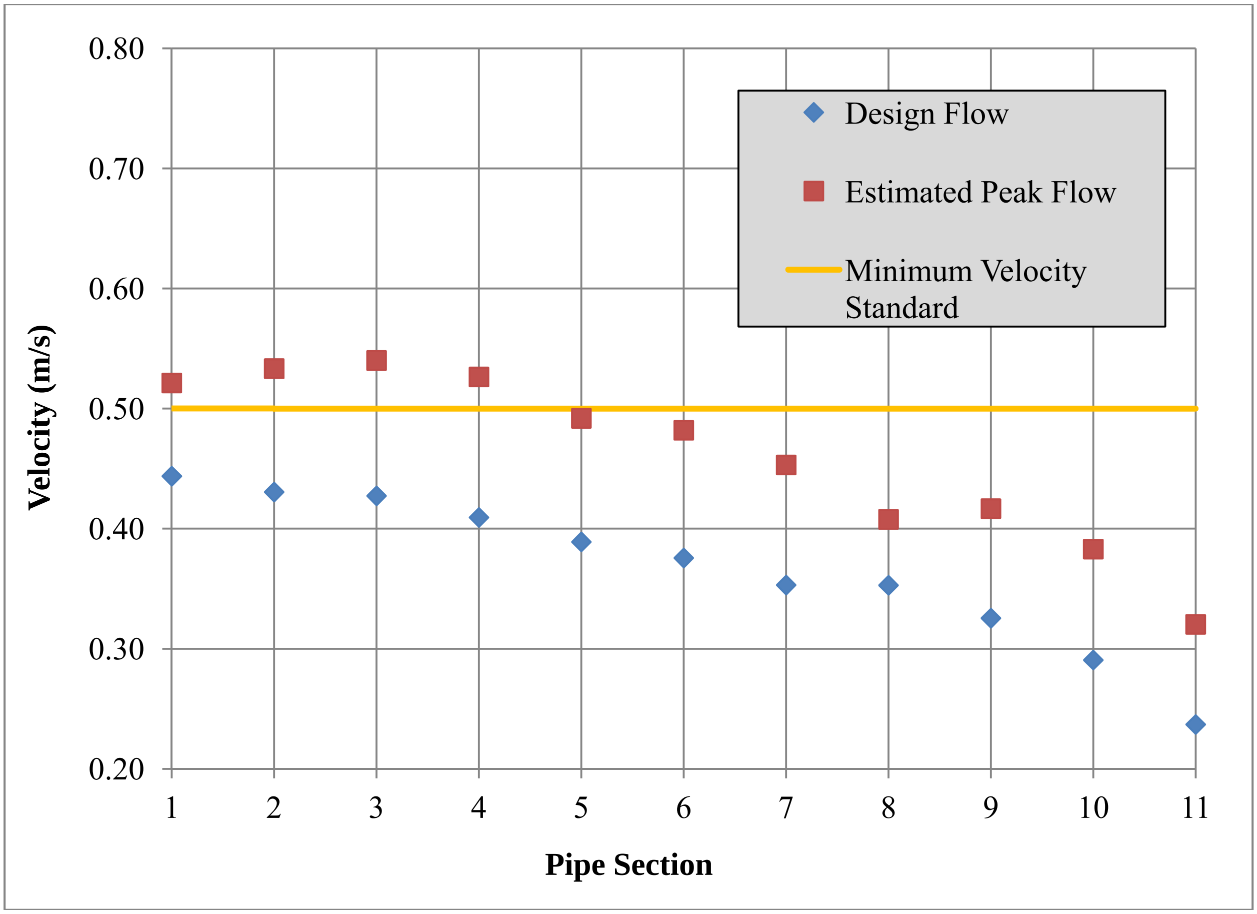

Figure 3.8: Estimated Velocities of Wastewater Calculated From the Darcy-Weisbach Equation

3.6.6 Low Flow Conditions

As outlined earlier, nowhere in our analysis or in any of the design documents that we have obtained, is there any mention or account of low flow conditions in the piping system. We believe that low flow would be nearly impossible to calculate due to the limited amount of entries, through the form of individual businesses and residents, to the system. The furthest point upstream, where the piping system begins, the flow will consist entirely of the flow from one particular business or resident. The low flow for this section of pipe will be zero because at some points during the day or night it is assumed that that particular resident will not be using water. Again, one solution could be to install additional cleaning access points in order to keep the pipes clear. Another option is to periodically flush the pipes. This could be done by pumping water from the lake into the furthest upstream point in the piping system, and flushing out all of the solids or it may be accomplished by the natural peak flows in the system. The last option would be to adjust the slope of the pipe for the furthest upstream sections. An increase in slope at the furthest upstream reach will cause an increase in velocity and will help to mitigate the problem of clogging. The drawbacks to adjusting the slope in the upstream section are twofold. First, the slope of the downstream sections will have to be decreased which will make the velocities more uniform, but it will decrease the velocity near the distribution tank which could cause additional problems with clogging. Second, construction of a multiple slope piping system will be very difficult. There would need to be a great precision in construction especially when you are constructing a slope less than 0.305%.

3.6.7 Pipe Supports

The pipe supports are specified in the design documents to span a length of six meters for the coated PVC pipe (Patronato de Playas de Maria, 2006). However, the CAD drawings we received indicate the span as approximately three meters. Based on our analysis of the PVC pipe span we believe the span from the CAD drawings is more appropriate. To determine the maximum support spacing for a PVC pipe flowing full with water, which would be the worst case scenario, we used the following equation:

$$L = \left[\frac{0.154EI}{W}\right]^{\frac{1}{3}}$$

where L is the support spacing in inches, E is the modulus of elasticity for PVC pipe material (lb/in^2), I is the pipe moment of inertia (in^4), and W is the weight of the pipe filled with water (lb/in) (PWEagle, 2004). For our analysis we assumed schedule 40 PVC pipe, which is more commonly used than schedule 80. The modulus of elasticity was assumed be 372,000 lb/in^2 for a temperature of 90° Fahrenheit (PWEagle, 2004). The pipe moment of inertia and the weight of pipe filled with water were calculated to be 28.2 in^4 and 1.3 lb/in respectively. The maximum span length was calculated to equal 2.7 meters which is less than half of the specified span length in the design documents. However, based on observations during our site investigation and pictures of the concrete columns, it appears that the concrete columns which were built to support the pipe were spaced at a distance of approximately three meters, which is consistent with the CAD drawings. The equation used for the maximum support spacing is based on common industry practice of limiting the vertical sag of the pipe to be less than 0.2% of the span length (PWEagle, 2004). Pipe sag calculations were made by the following equation:

$$Y = \frac{0.013WL^4}{EI}$$

where Y is the maximum vertical sag (inches) and all of the other variables are consistent with the maximum sag length equation (PWEagle, 2004). Based on the span length of 2.7 meters the maximum vertical sag, which occurs at the midpoint between supports, is 0.21 inches, or 0.2% of the span length. This sag is comparable to the natural drop in elevation between a column and the next mid-span point (slope of 0.35% times 2.7/2 m = 0.19 in.) meaning that a portion of the pipe sections downstream from the mid-points between spans could be essentially flat.

A span length of three meters is still larger than the maximum span distance calculated, but it is pretty close. The maximum span value of 2.7 meters was calculated based on the assumption that the pipe was flowing full. Based on our analysis of the peak and design flows, the pipe will never be flowing completely full although it was close to full near the distribution box for the peak flow. Because the decrease in water in the pipe would lead to a larger calculated span length, we believe that a pipe spacing of three meters is reasonable, and it would be impractical and costly to build new pipe supports at a distance of 2.7 meters if the columns that are already there are spaced at three meters. However, we note that the sag is proportional to the span length raised to the power four, so a seemingly modest increase in span from 2.7 to 3 m increases the sag by 52% to about 0.3 inches. This means that downstream portions of each pipe section would have a slightly negative slope under extreme conditions. We do not believe this will be a serious problem, but caution against using spans any larger than 3 m.

3.6.8 Distribution Box

As outlined in section 3.3.2, the buoyancy of the distribution tank must be checked to determine if the tank could possibly float at high water conditions. Nowhere in the design documents are there any specifications or sizes of the distribution tank so the measurements of the tank were assumed based on pictures of the tank. The tank is assumed to have six inch walls and base. The footprint of the tank is assumed to be 61 square feet and the height of the walls is assumed to be five and a half feet. The density of concrete is assumed to 150 pounds per cubic feet and the density of water, at 70° Fahrenheit, is assumed to be 62.3 pounds per cubic feet (The Engineering Toolbox). The total weight of concrete was calculated to be 20,000 pounds. The worst case scenario for the tank would be if the water was all the way to the top of the walls, and the tank was completely empty. In this case the volume of water displaced would equal 336 cubic feet. This would equal a weight of water displaced of 21,000 pounds, which is barley greater than the weight of concrete. For this worst case scenario, with the estimated dimensions of the tank, the buoyancy force would be greater than the weight of the concrete and the tank would float. It seems very unlikely that this situation would ever occur, but further analysis should be conducted to insure that the tank will not float. Another thing to consider is the cover to the distribution tank which was not included in the initial calculation. This additional weight of concrete could help to counteract the buoyancy force, and thus keep the tank from floating.

3.7 Piping System Conclusions

Based on our analysis of the design and construction documents which we have obtained, we feel the piping system design is satisfactory, and the system should work properly if built to spec. Obviously what was built of the piping system in 2008 did not match what was specified in the construction documents.

The potential problems we see with the design are as follows

- Precision construction of pipe elevations
- Lack of low flow calculations
- Low velocities of the wastewater
- Spacing of the piping supports
- Potential damage to the piping system along the shoreline

3.7.1 Precision Construction of Pipe Elevations

Because of the limited amount of available hydraulic head, and the small slope of the pipe, it is essential that the piping system is built with the utmost precision. We are assuming that the topographic information, which includes the horizontal and vertical distances of the piping system, is correct. Additional surveying work may be required to check that the construction plans are accurate.

3.7.2 Lack of Low Flow Calculations and Low Wastewater Velocities

As mentioned before, it will be difficult to accurately predict the low flow condition, so steps should be taken to compensate for low velocities and the potential for clogging in the pipes. It is possible that the periodic natural peak flows will adequately flush the system and prevent build up from occurring. Other solutions are as follows:

- Constructing additional junction cleaning boxes to allow for the pipe to be cleaned at regular intervals.
- Creation of a pumping system which uses lake water to periodically flush the pipe at the furthest upstream reach.

Whatever the solution is for the low flow velocities in the piping system, it will still be important to maintain proper operation and maintenance. Even at some of the sections of pipe closest to the distribution box, with the highest flow, there will be times during the middle of the night when the velocities will be very low. It will be essential to make sure the entire system is cleaned periodically to maintain proper function.

3.7.3 Spacing of Piping Supports

Based on our calculations of the maximum support spacing of 2.7 meters for schedule 40 PVC pipe, we believe the support specification of six meters on the design documents is far too large. However, the spans which were actually built are closer to three meters, which is consistent with the CAD drawings. This is still slightly larger than our estimate for the maximum span length. We believe it is not practical to re-build all of the supports at a distance of 2.7 meters and that the three meters spacing supports should be used. Since the span length was calculated based on sag and not pipe failure, the worst case scenario is that some of the pipes have sections of zero or slightly negative slope.

3.7.4 Potential Damage to the Piping System Along the Shoreline

The location of some of the piping system along the shoreline of the lake could also be a problem. As can be seen in Figure 3.9, the concrete columns meant to hold the piping system are surrounded by a bunch of boats. If the pipe were to be mounted onto the columns, there is always the potential for damage to occur as people try to maneuver their boats around the pipes. The solution that Jessica Castillo of PRONADERS had suggested was to encapsulate the pipe in concrete to help protect it from these types of potential damage (Castillo, 2011). This solution could also help to protect the pipe from damage due to the ultraviolet rays of the sun.

Figure 3.9: Potential Hazards for the Piping System
(Walker 2011)

3.8 Treatment

The Las Casetas treatment plant includes an upflow anaerobic sludge blanket (UASB) system for primary treatment, as described in Section 2.2.4, and a trickling filter for secondary treatment. A sludge drying bed is also provided for dewatering of the sludge produced in the UASB, and effluent from the trickling filter also undergoes final clarification and chlorination before being discharged to Lake Yojoa or used for irrigation water. Each of these processes is described in the following sections.

3.8.1 Upflow Anaerobic Sludge Blanket (UASB)

The primary treatment tank at the Las Casetas site is an upflow anaerobic sludge blanket (UASB), pictured in Figure 3.10. These systems are described in some detail in Section 2.2.4. Design flows provided in the initial proposal for the treatment plant and sewer system were used in conjunction with design drawings compiled by Entech Environmental Technology and provided to us by PRONADERS in order to determine if the plant, as constructed, is sufficient to treat the wastewater from Las Casetas.

Figure 3.10: UASB Tank at Las Casetas
(Walker, 2011)

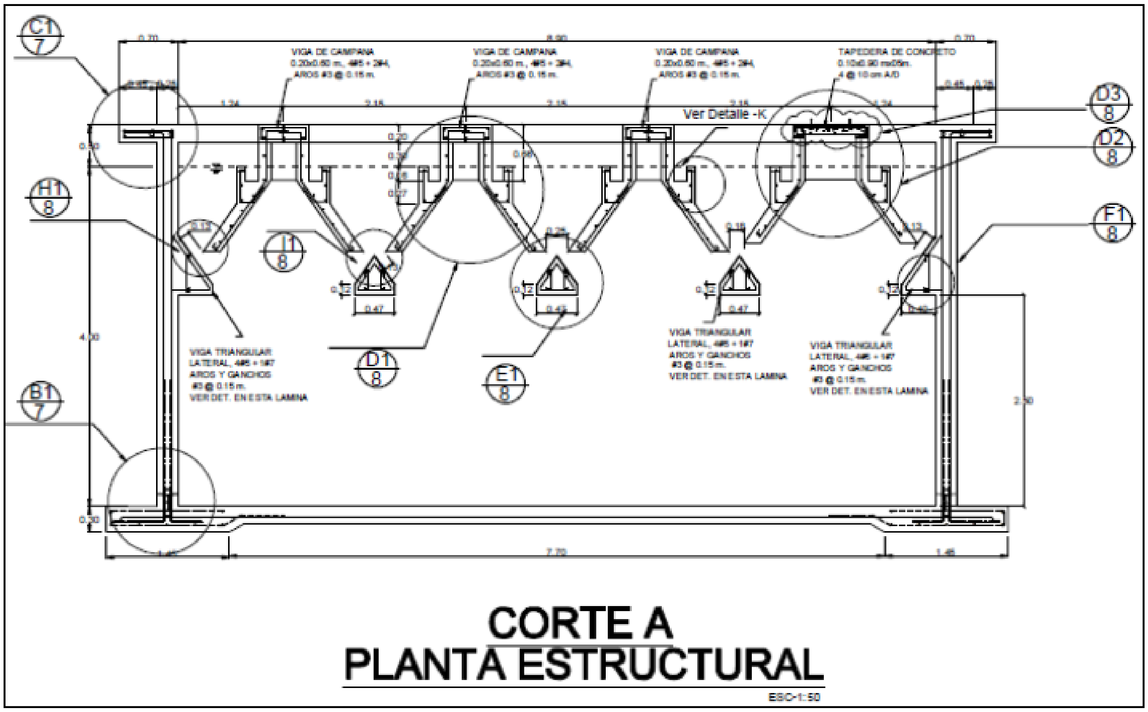

Figure 3.11: Design Drawing for the UASB Tank, Front View
(Entech, 2007)

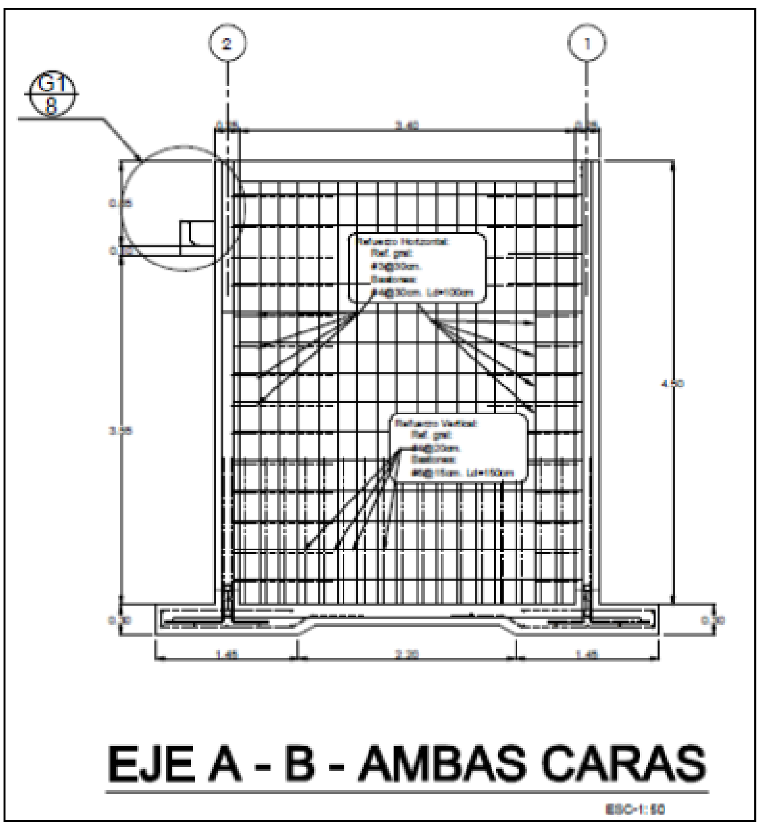

Figure 3.12: Design Drawing for the UASB Tank, Side View
(Entech, 2007)

The drawings shown in Figures 3.11 and 3.12 were used to calculate the total volume of the UASB reactor as designed. With a length of 8.9 meters, a width of 3.4 meters, and a height of 4.0 meters, the volume of the tank was calculated to be 121 m^3. This is significantly less than 1000 m^3, the maximum suggested size for such a reactor. The length-to-width ratio of the reactor is also an important consideration in UASB design, and from the given parameters was found to be 2.6 to 1, also less than the suggested maximum ratio of 4 to 1 (Mara, 2005).

Next, the volume and design flow were used to calculate the upflow velocity through the reactor as well as the mean hydraulic retention time. This was done using the relation

$$V_{up} = \frac{Q}{A} = \frac{D}{\theta}$$

in which V_{up} is the upflow velocity, Q is the design flow rate, A is the surface area of the reactor, D is the depth of wastewater in the reactor, and θ is the mean hydraulic retention time (Mara, 2005). Given a design flow of 3.18 liters per second, as calculated in the proposal for the system, and a surface area of 30.3 m^2 as calculated from the dimensions of the tank given in the design drawings, and assuming that D is approximately equal to the height of the tank (4.0 meters), V_{up} was estimated to be 0.4 meters per hour and θ was estimated to be 10.6 hours. Both of these values are within the range of acceptable values, given in Table 3.7 (Mara, 2005).

The calculations for upflow velocity and retention time were repeated using the estimated peak flow rate of 9 liters per second, which yielded values of 1.07 meters per hour and 3.74 hours, respectively. This upflow velocity is barely outside the range of acceptable values, but the retention time is very low, about two-thirds of the minimum recommended retention time. Thus, it may be wise to install a bypass system in case of such a flow, which could overload the system. Whether or not this is necessary will depend on actual values of peak flow, which should be monitored during the early phases of implementation. Table 3.7 summarizes the UASB design parameters and calculations.

Table 3.7: UASB Design Parameters
(Mara, 2005)

Parameter	Calculated Value Under Design Flow	Calculated Value Under Peak Flow	Acceptable Value	Acceptable Under Design Flow?	Acceptable Under Peak Flow?
Reactor volume, m^3	121.04	121.04	< 1000	Yes	Yes
L:W ratio of reactor	2.6:1	2.6:1	< 4:1	Yes	Yes
Upflow velocity, m/h	0.4	1.07	< 1	Yes	No
HRT, hours	10.6	3.74	6 to 12	Yes	No

Finally, the percentage of chemical oxygen demand (COD) removed can be approximated using the equation

$$R = 100(1 - \theta^{-0.68})$$

in which R is the fraction of COD removed and θ, as before, represents the mean hydraulic retention time in hours. In domestic wastewaters, percent COD removal essentially equals percent BOD removal (Mara, 2005), though the composition of the wastewater from Las Casetas may invalidate this assumption since a significant portion of it is restaurant waste containing grease and cooking oil (Chokshi and Trate, 2006). Using the design mean hydraulic retention time of 10.6 hours, the Las Casetas UASB tank should theoretically remove approximately 80 percent of COD; this number indicates high efficiency and also coincides with the value given in the initial analysis of the system outlined in the proposal (Patronato de Playas de María, 2006).

Estimates of BOD in restaurant wastewater vary widely. One study of 28 restaurants in Texas found a mean five-day BOD (BOD_5) of 1584 mg/L before trimming the data (that is, removing outliers), and 1040 mg/L after trimming (Lesikar et al, 2006). Another study determined that restaurants primarily serving seafood, as is the case for Las Casetas, were generally in the middle of the range of BOD values, below primarily Mexican and Asian restaurants but higher than restaurants primarily serving American cuisine. In this study, the restaurants evaluated that primarily served seafood, of which there were only two, had wastewater with BOD values between 415 and 1600 mg/L (Garza, 2004). From these studies, we assumed a moderately conservative BOD value for the Las Casetas restaurant wastewater of 1000 mg/L.

However, the casetas that will be served by the treatment plant are not all restaurants, nor do those that serve food produce only restaurant waste. Based on the proposal for the system, of the 58 currently occupied casetas, 12 are used solely for housing or other purposes and do not serve as restaurants (Patronato de Playas de María, 2006). These houses will produce only domestic sewage. BOD is a characteristic indicator of the strength of raw sewage, and typical values are outlined in Table 3.8. In cities and towns in developing countries, common BOD values for raw sewage are 400-800 mg/L, usually containing about 40 grams of BOD per person per day (Cairncross and Feachem, 1993). Using a design population of 571 and a design flow rate of 3.18 liters per second, as given in the proposal for the system, we approximate a BOD loading of only 83 mg/L. Given the relatively small design population, this may be a reasonable approximation; however, to be conservative, we will use an estimate of 200 mg/L for the domestic portion of the wastewater, on the lower end of the spectrum of BOD values for raw sewage.

Table 3.8: BOD Characterization of Raw Sewage
(Cairncross and Feachem, 1993)

Strength	BOD (mg/L)
Weak	200 or less
Medium	350
Strong	500
Very Strong	750 or more

As a first guess, we approximate that half of the waste from the casetas serving food is restaurant waste, and the other half is domestic waste. Further assuming that each caseta produces the same amount of domestic waste, and that each restaurant produces the same amount of restaurant waste, this gives us a restaurant waste to domestic waste ratio of 46 to 58. Thus, using our estimate of 1000 mg/L for the restaurant portion of the waste and 200 mg/L for the domestic portion, we can approximate an overall BOD concentration of about 550 mg/L. If we assume that BOD removal efficiency is effectively equal to COD removal efficiency, and therefore the UASB removes approximately 80% of the BOD in the wastewater, this results in an estimated BOD concentration of 110 mg/L after primary treatment.

3.8.2 *Trickling Filter*

A trickling filter is an attached growth process, in which microorganisms form a film over filter media to which wastewater is applied. The microorganisms biodegrade the organic material in the wastewater, with the filter media providing large amounts of surface area on which the microorganisms can grow, and oxygen is delivered to the microorganisms via natural or forced draft ventilation of air through the pore spaces in the filter media. Historically, the most popular design consisted of a bed of stones 1 to 3 meters deep, with a rotating arm to distribute water over the rocks (Davis, 2011); the trickling filter at the Las Casetas plant is an example of this style of trickling filter, with the exception that rather than utilizing a rotating filter arm, the trickling filter has eight stationary arms through which the water is distributed. The filter is shown in Figure 3.13.

Figure 3.13: Trickling Filter at the Las Casetas Treatment Plant
(Puckett, 2011)

Trickling filters are classified according to the applied hydraulic and organic load; see Table 3.9. Organic loading is expressed as kilograms of BOD_5 per day per cubic meter of bulk filter volume (Davis, 2011). There is no data currently available on the BOD content of the Las Casetas wastewater, but the organic loading can be approximated using the estimated BOD loading derived in Section 3.8.1. The hydraulic loading is a function of the flow rate and the dimensions of the filter and filter media, and since these parameters are all known, at least approximately, the hydraulic loading can be estimated as well.

Table 3.9: Trickling Filter Classification
(Metcalf and Eddy, 2003)

Design Characteristic	Low or Standard Rate	Intermediate Rate	High Rate	High Rate	Roughing
Type of packing	Rock	Rock	Rock	Plastic	Rock/Plastic
Hydraulic loading, m^3/m^2-d	1-4	4-10	10-40	10-75	40-200
Organic loading, kg BOD/m^3-d	0.07-0.22	0.24-0.48	0.4-2.4	0.6-3.2	> 1.5
Recirculation ratio	0	0-1	1-2	1-2	0-2
Filter flies	Many	Varies	Few	Few	Few
Sloughing	Intermittent	Intermittent	Continuous	Continuous	Continuous
Depth, m	1.8-2.4	1.8-2.4	1.8-2.4	3.0-12.2	0.9-6
BOD removal efficiency, %	80-90	50-80	50-90	60-90	40-70
Effluent quality	Well nitrified	Some nitrification	No nitrification	No nitrification	No nitrification
Power, kW/10^3 m^3	2-4	2-8	6-10	6-10	10-20

Hydraulic loading is typically expressed as cubic meters of wastewater applied per day per square meter of bulk filter surface area, or depth of water applied per unit of time. In order to determine the filter media surface area, the average size of the stones in the filter must be approximated. Then, using an estimate for the porosity, φ, of the filter media, the specific area of the filter media can be approximated as

$$\alpha = \frac{6}{D_p}(1 - \varphi)$$

in which D_p represents the particle diameter of the filter media. The specific area is defined as the surface area per unit volume of filter bed, and so when multiplied by the volume of the filter the result is the total surface area of filter material available (Hirasaki, 2004). It is important to note that this equation assumes that the medium is of uniform spherical composition; however, since we only need an estimate of the hydraulic loading in order to classify the trickling filter, this should not affect our analysis substantially.

As mentioned previously, the filter media at Las Casetas consists of a bed of stones. Some examples are depicted in Figure 3.14.

Figure 3.14: Las Casetas Trickling Filter Media
(Puckett, 2011)

From these images, a rough estimate of the average particle diameter D_p was found. The stones are clearly nowhere close to being of uniform size or shape, but an average diameter is good enough for our analysis and was approximated as 5 centimeters. The porosity was estimated to be around 50 percent, based on reported values of porosity for various filter media (Liu and Lipták, 1999). Using this value of porosity and the estimated average particle diameter of 5 centimeters, the specific area was calculated as 60 m^2 per m^3.

From the design drawings, and the fact that around 0.6 meters at the top of the filter was unfilled, the total filter volume was determined to be approximately 17 m^3. Thus, the total surface area of filter media was estimated to be 1,020 m^2 – the estimated specific area multiplied by the volume of the filter.

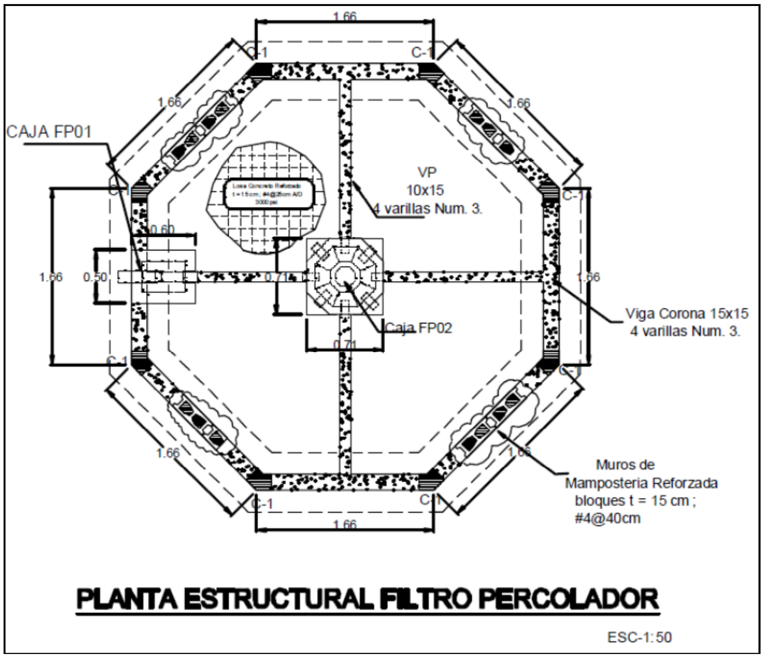

Figure 3.15: Design Drawing for Trickling Filter, Plan View
(Entech, 2007)

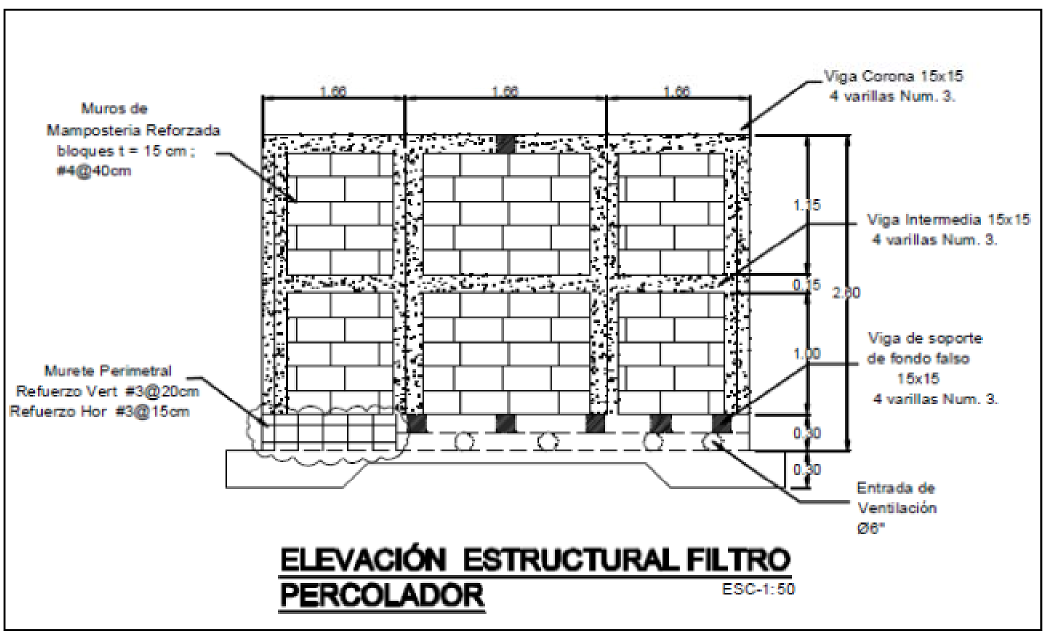

Figure 3.16: Design Drawing for Trickling Filter, Side View
(Entech, 2007)

Finally, the estimated hydraulic loading was calculated for the design flow rate of 3.18 liters per second. The hydraulic loading was determined by dividing this flow rate by the bulk surface area of 1,020 m^2, calculated above. This yielded a value of 0.27 meters per day, which is significantly less than the lowest typical hydraulic loading value for low rate trickling filters, as outlined in Table 3.9 (Metcalf and Eddy, 2003).

The organic loading was approximated based on the estimated BOD content, 110 mg/L, of the effluent after primary treatment in the UASB. Using the bulk filter volume of 17 m^3, as calculated previously, and the design flow of 3.18 liters per second, the organic loading was estimated to be about 1.8 kilograms per day per cubic meter. This is a very high organic loading rate, clearly inappropriate for a low-rate filter based on the parameters given in Table 3.9. However, there is a good possibility that the BOD content of the wastewater is significantly lower than the estimated value. In order to perform a more accurate analysis of the Las Casetas trickling filter, then, measurements of the BOD in the wastewater must be made. The easiest way to do this may be to use a 5-day BOD jar test (American Public Health Association, 1992).

As described in Metcalf and Eddy (2003), The National Research Council offers the following equation for the BOD removal efficiency of a first-stage rock filter:

$$E = \frac{100}{1 + 0.4432\sqrt{\frac{W}{VF}}}$$

in which E is BOD removal efficiency at 20 degrees Celsius, W is the BOD loading rate in kg per day, V is the volume of filter packing in cubic meters, and F is the recirculation factor. The recirculation factor is a unitless parameter given by

$$F = \frac{1 + R}{(1 + \frac{R}{10})^2}$$

where R is the unitless recycle ratio (Metcalf and Eddy, 2003). Thus, for a system with no recycling, as is the case for the Las Casetas plant, the recycle ratio is zero and the recirculation factor is simply one.

Assuming the calculated organic loading of 1.8 kilograms per day per cubic meter is correct, and given that the volume of filter packing is equal to the bulk filter volume multiplied by one minus the porosity, the BOD removal efficiency of this system at 20 degrees Celsius was calculated to be 54 percent. However, the removal efficiency varies with temperature according to

$$E_T = E_{20}(1.035)^{T-20}$$

where E_T is the removal efficiency at temperature T in degrees Celsius, and E_{20} is the removal efficiency at 20 degrees Celsius (Metcalf and Eddy, 2003). The average annual temperature in Honduras at elevations below 1500 feet is generally between 26-28 degrees Celsius (Encyclopædia Britannica, 2011). Using an average value of 27 degrees Celsius, the above equation, and a 54% removal efficiency at 20 degrees Celsius, the average removal efficiency of the Las Casetas trickling filter should be around 69 percent. In this case, the final BOD of the effluent would be approximately 34 mg/L. This is well below the Honduran national effluent standard of 50 mg/L for BOD (Mikelonis and Hodge, 2008), a drastic improvement over the level of BOD that would be discharged otherwise. In addition, removal efficiency may be even higher if BOD levels in the raw wastewater are lower than the values estimated here.

Removal efficiency, as well as hydraulic loading, may also be increased by incorporating recirculation into the process, or by making the trickling filter an intermittent process. Both of these options, however, would require significant adjustments to the facility that may be outside the realm of feasibility. It is recommended that a second analysis of the filter, using measured values of bulk filter surface area and BOD loading, first be performed. Based on the results of this analysis, a cost-benefit analysis may be the best method to determine whether it would be worthwhile to implement either strategy.

3.8.3 Sludge Drying

The proposal for the treatment plant indicated that two sludge drying beds should be constructed in order to ensure adequate dehydration time. Under this scenario, once sludge removal is initiated, dried sludge would be removed every week, i.e., every two weeks for each individual bed (Patronato de Playas de María, 2006). However, based on our visual assessment of the facility, it appears that only one sludge drying bed was actually built, shown in Figure 3.17.

Figure 3.17: Las Casetas Sludge Drying Bed as Constructed
(Puckett, 2011)

Sludge drying is a combination of drainage and evaporation. A significant amount of runoff occurs during the first 12 to 18 hours after removal of sludge from the primary tank, and subsequent drying is primarily a result of evaporation. The depth of sludge should generally range from 20 to 30 cm, according to the proposal, with thinner layers allowing for faster removal of dried sludge. “Fresh” sludge should never be added on top of dried or partially dried sludge. In addition, dried sludge can be piled outside of the drying bed for several months before disposal, and can be used for fertilizer for the site of the treatment plant or elsewhere (Patronato de Playas de María, 2006). If the sludge is to be used as fertilizer for agricultural purposes, it should be stored for at least three months before being applied to cropland as it may contain *Ascaris* eggs (Mara, 2005).

For a UASB system, the amount of sludge produced is typically around 0.2 kg per kg of BOD removed (Mara, 2005). Given the estimated values of 550 and 110 mg/L, before and after primary treatment, respectively, and the design flow of 3.18 liters per second, this would mean a sludge production rate of approximately 24 kilograms per day.

In general, at temperatures above 20 degrees Celsius, the area required for a sludge drying bed is in the range of 0.01 to 0.015 m^2 per person (Mara, 2005). Given the population estimates from the Playas de María proposal, then, an initial guess of the required area can be made. The design population consists of a set population of 522 permanent residents and a floating population of 643.5 visitors, totaling about 1166 people (Patronato de Playas de María, 2006). The design diagrams produced by Entech (Environmental Technology) delineate a sludge drying bed area of 6.00 meters by 5.01 meters, or approximately 30 m^2 (Entech, 2007). Using the design population of 1166, this represents about 0.026 m^2 per person – twice as much as the required area suggested by Mara (2005). Thus, the sludge drying bed appears to be more than large enough as constructed to handle the amount of sludge that would be generated, based on this perhaps oversimplified approximation, and could in fact be separated into two sludge drying beds as initially designed in order to better accommodate new sludge at all times.

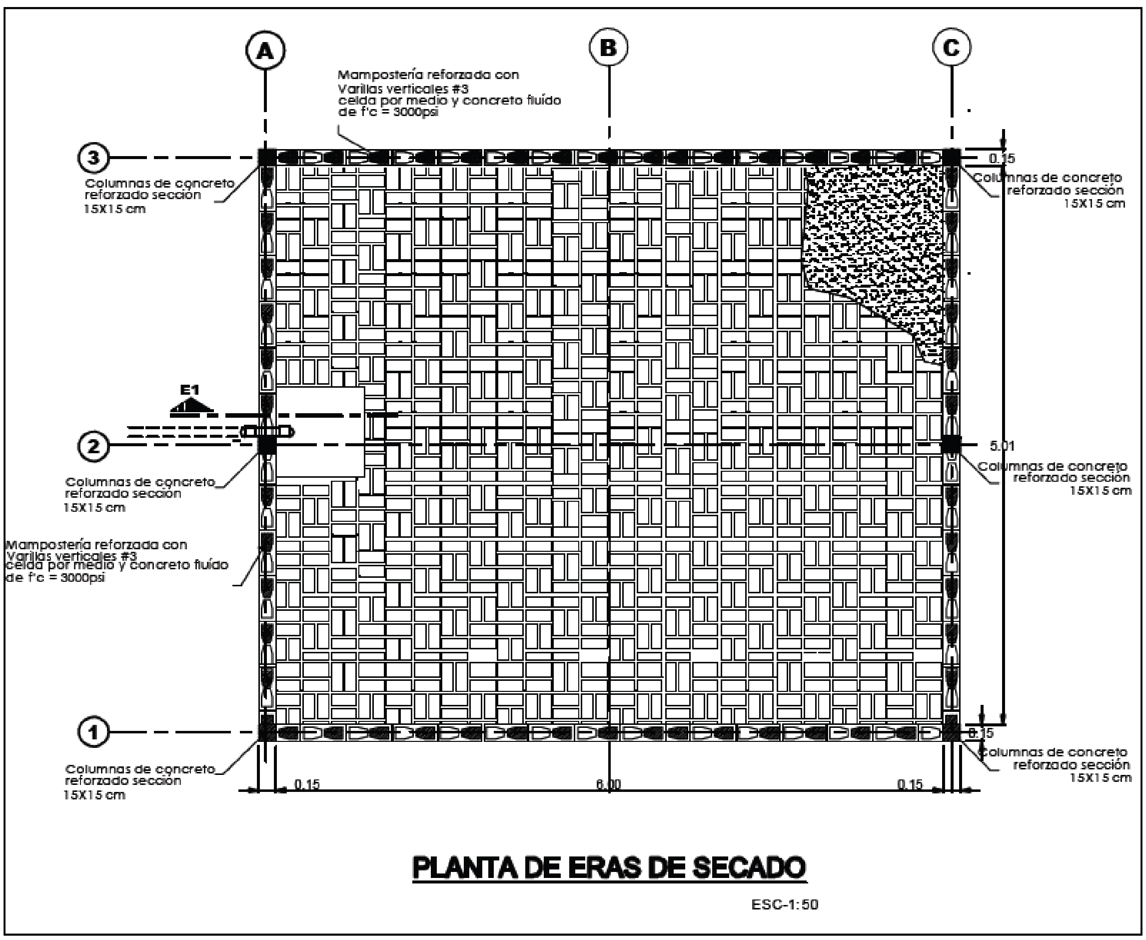

Figure 3.18: Design Drawing for the Sludge Drying Bed, Plan View
(Entech, 2007)

3.8.4 Final Treatment and Disposal

Before the effluent from the Las Casetas plant is discharged, it is sent to a secondary clarifier. Secondary clarifiers are generally a requirement in plants where trickling filters are used, because excess growths of microorganisms in the filter can slough off and enter the effluent as BOD and suspended solids (Davis, 2011). The secondary clarifier at Las Casetas is pictured in Figure 3.19.

Figure 3.19: Secondary Clarifier at Las Casetas Treatment Plant
(Puckett, 2011)

The dimensions of the clarifier as given in the design drawings were used to calculate the surface area and determine the sidewater depth of the clarifier. Using these parameters, the overflow velocity can be calculated according to

$$v_0 = \frac{Q}{A}$$

where v_0 is the overflow velocity, A is surface area and Q is flow rate. Based on the sidewater depth of the clarifier, recommended average and maximum trickling filter secondary clarifier overflow rates are available, as outlined in Table 3.10 (Davis, 2011).

Table 3.10: Recommended Trickling Filter Secondary Clarifier Overflow Rates
(Davis, 2011)

Sidewater depth, m	Average overflow rate, m/h	Maximum overflow rate, m/h
2	0.4	0.75
3	0.8	1.6
4	1.2	2.2
5	1.4	2.8

The design drawings for the clarifier indicate a length of 2.63 meters and a width of 2.70 meters, yielding a surface area of 7.1 m^2. The drawings also suggest that two clarifying tanks in series were planned, though only one was actually constructed (Entech, 2007).

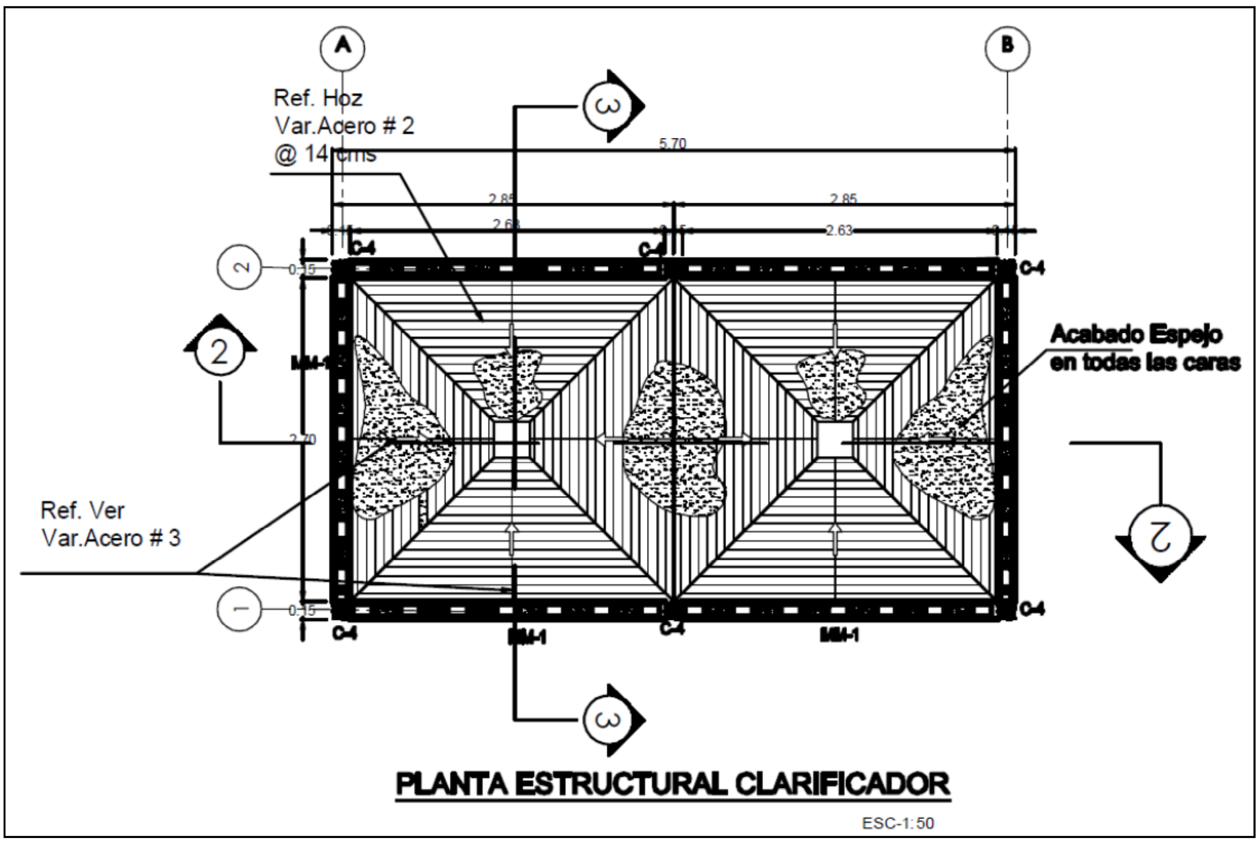

Figure 3.20: Design Drawing for Secondary Clarifier, Plan View
(Entech, 2007)

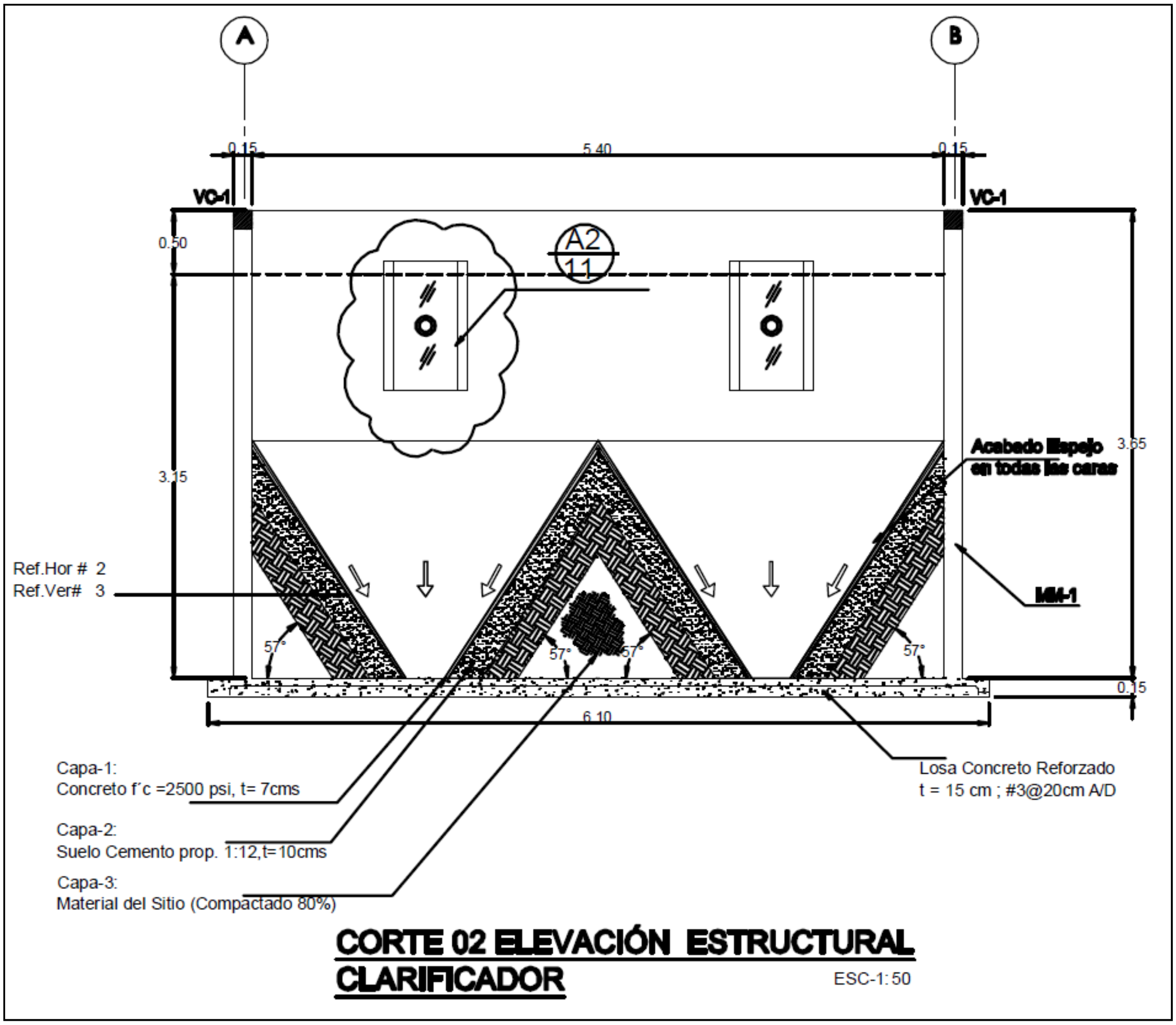

Figure 3.21: Design Drawing for Secondary Clarifier, Side View
(Entech, 2007)

Using the given surface area, the design flow rate of 3.18 liters per second, the peak flow rate of 9 liters per second, and the equation for overflow velocity given above, the average and maximum overflow velocities were calculated. The average overflow velocity was found to be approximately 1.6 meters per hour, and the maximum was approximately 4.6 meters per hour. With a sidewater depth of 3.15 meters, based on the design drawings, these values are clearly much higher than the recommended values given in Table 3.10. However, if both clarifiers had been constructed, this would double the surface area available and thus halve the overflow rates, resulting in an average overflow rate of about 0.8 meters per hour and a maximum overflow rate of about 2.3 meters per hour. While this maximum overflow velocity is still relatively high, the average overflow velocity is consistent with the values given in Table 3.10. Therefore, it is recommended that the second clarifier, which was planned but not constructed, be built in order to ensure efficient removal of solids after the wastewater has exited the trickling filter. Based on diagrams of the plant done by UPI students during our visit, there is enough space for a second clarifier within the confines of the treatment plant, though construction would require relocating the small distribution box that conveys effluent from the trickling filter to the clarifier, depicted in Figure 3.22.

Figure 3.22: Distribution Box Between Trickling Filter and Secondary Clarifier
(Puckett, 2011)

The plans for the plant also call for chlorination of the effluent, although specific details as to how the chlorine is to be applied are not available. Since chlorine requires at least half an hour of contact with water in order to effectively disinfect it, and because exposure to sunlight would destroy the protective chlorine residual, the effluent should be held in a storage tank blocked off from sunlight while chlorine is applied (Cairncross and Feachem, 1993). Such a storage tank does not appear to currently exist at the Las Casetas facility, and should be constructed if chlorination is to be applied.

Several relatively simple possibilities exist for the application of chlorine to the effluent. Solution feeders can be operated either electrically or via energy produced by the flow of water, provided a minimum head is available, and supply chlorine (usually in the form of hypochlorite solution) at a rate proportional to the flow. Alternatively, a drip-feed device can be used to apply chlorine at a more or less constant rate, although such devices can be troublesome to stop when there is no flow in the pipe, and tend to be less reliable over time as they can become encrusted with chloride deposit. There are also devices available that add chlorine to water in the form of calcium hypochlorite tablets, which are convenient and easy to use but generally more expensive than hypochlorite powder (Cairncross and Feachem, 2003).

There are some concerns with the use of chlorine that should be taken into account before disinfection is implemented. In particular, chlorine residuals can prove toxic to some species of fish, and tilapia may be at risk. Partial kills of tilapia have been reported at chlorine residual levels between 0.35-1 mg/L (Young, 1975). Care must therefore be taken to ensure that the level of chlorine residual in the final effluent will not significantly affect the lake's tilapia population.

3.8.5 Alternatives Considered

An extended aeration / activated sludge system was considered and analyzed as an alternative to the UASB. The system investigated included an aeration chamber with a retention period of 1.75 hours, a re-aeration area with a retention period of 7.07 hours, a 200-cubic-foot aerated digester, a settling chamber, and chlorination as a final process.

The advantages of this type of system were readily apparent, as it has seen widespread use globally. These advantages included the relatively small amount of equipment required, a simple solution to the sludge problem, and low requirements for number of and training for operating staff. However, there were also a number of disadvantages, which included the high energy consumption of such a system, the large space requirements, and potential for production of sludge in excess of what the filtration mechanism could handle, making it impossible to utilize sludge drying beds in large facilities. A comparison between the extended aeration system and the UASB system was performed and is outlined in Table 3.11. While the monetary and spatial requirements for the UASB system were actually found to be greater than those of the extended aeration system, the excessive energy requirements for the extended aeration system were deemed to outweigh the other considerations. Thus, the extended aeration system proposal was abandoned in favor of a UASB system, which also requires less maintenance in terms of electrical equipment and motors (Patronato de Playas de María, 2006).

Table 3.11: Extended Aeration vs. UASB System
(Patronato de Playas de María, 2006)

Characteristic	Extended Aeration	UASB
Capacity, m^3/day	2 x 227	300
Area Required, m^2	180	220
Initial Cost, USD	$138,000.00	$118,496.00
Operating Costs (excluding electricity), Lempiras/month	2,000.00	3,000.00
Energy Consumption, KWh/month	4,200	795

Chapter 4: Conclusions and Recommendations

4.1 The Lake Yojoa Subwatershed

Inadequate wastewater treatment is a problem throughout the Lake Yojoa Subwatershed, as identified by previous MIT M.Eng. teams and evidenced by our own experience in the field. Despite indications that many of the municipal governments in the Subwatershed are interested in improving wastewater treatment, progress has been slow and minimal, for a variety of reasons. Visibility of the sanitation problem is one issue that should be addressed, since many Honduran citizens simply do not see where their waste ends up and are therefore uninformed of the overall wastewater situation. There are also impediments to progress when it comes to political turnover with little to no continuity within the sanitation sector. In order for wastewater treatment to improve, new lines of communication must be opened between incoming and outgoing political officials, stakeholders, and the workers who maintain the treatment systems.

The extent to which this lack of treatment contributes to eutrophication of the lake is not known, since a variety of other sources also contribute pollutants to the lake, and each of these sources must be evaluated independently in order to form a complete picture. To this end, a full nutrient load analysis of the lake, based on reliable data from all point and non-point sources, is essential to understanding how best to reverse the ecological damage that has been done. Such an analysis will require involvement with and cooperation among all parties responsible for any pollution to the lake. This dialogue has already begun through the MIT contacts with municipal governments, civil entities, and other stakeholders in the Subwatershed, and it is our hope that it will continue in the coming years.

4.2.1 Las Casetas

This study focused on a collector system and treatment plant for Las Casetas, a group of small homes and fish restaurants on the southeastern shore of Lake Yojoa. In general, both the sewer system and treatment plant were well-designed and, in the case of the treatment plant, well-built. If the system is connected and maintained, there is no reason why it should not work as intended. Nonetheless, we do have several specific recommendations for the system, introduced in Chapter 3 and enumerated here.

4.2.1.1 Sewer System

For the piping network, it was determined that the original design should be sufficient to convey the wastewater to the treatment plant, although low velocities at the upstream end of the system may create an increased risk of clogging. This risk will be greater during periods of low flow. The following recommendations are proposed:

- Replace the black corrugated piping that was originally used for the sewer system with either PVC or HDPE piping, as designed;
- Ensure that the piping is installed at a constant slope, as designed, in order to maximize available head;
- Install junctions to enable periodic cleaning of the pipe, especially at the upstream end of the system;
- Investigate the potential for installing a pumping system to flush the system at the upstream end in order to help mitigate the risks of clogging due to low velocities;
- Calculate the buoyancy of the distribution box to insure that it will be lifted up during the high water period for the lake; and
- Ensure that the pipe supports were built at three meter intervals.

4.2.1.2 Treatment Plant

The treatment plant was, overall, both well-designed and well-constructed. Under the design flow, the UASB should be able to remove around 80% of COD in the wastewater, and the trickling filter, secondary clarification, and chlorination will enhance the quality of the effluent as well. To maximize the efficiency of the plant, the following suggestions are proposed:

- Measure BOD_5 in the influent, and/or obtain original design reports including measurements of BOD_5 in the influent, in order to more accurately assess BOD removal rates during primary and secondary treatment;
- Install grease traps, as currently planned, as preliminary treatment;
- Re-evaluate the trickling filter, with an eye on the potential for recirculation or intermittency in order to increase the hydraulic load;
- Separate the sludge drying bed into two sections, to be used alternately;

- Install another secondary clarifier, as designed, in order to ensure sufficient removal of suspended solids following the trickling filter.
- Investigate potential methods for application of chlorine to the final effluent, making sure to keep chlorine residual levels below those that may be harmful to the lake's tilapia.

If these recommendations are heeded, the system should become a viable and sustainable collection and treatment system for Las Casetas' wastewater and will not only help to improve the quality of effluent entering Lake Yojoa, but will also improve public perception of the health and cleanliness of the restaurants. Hopefully this will, in turn, lead to increased business for the region.

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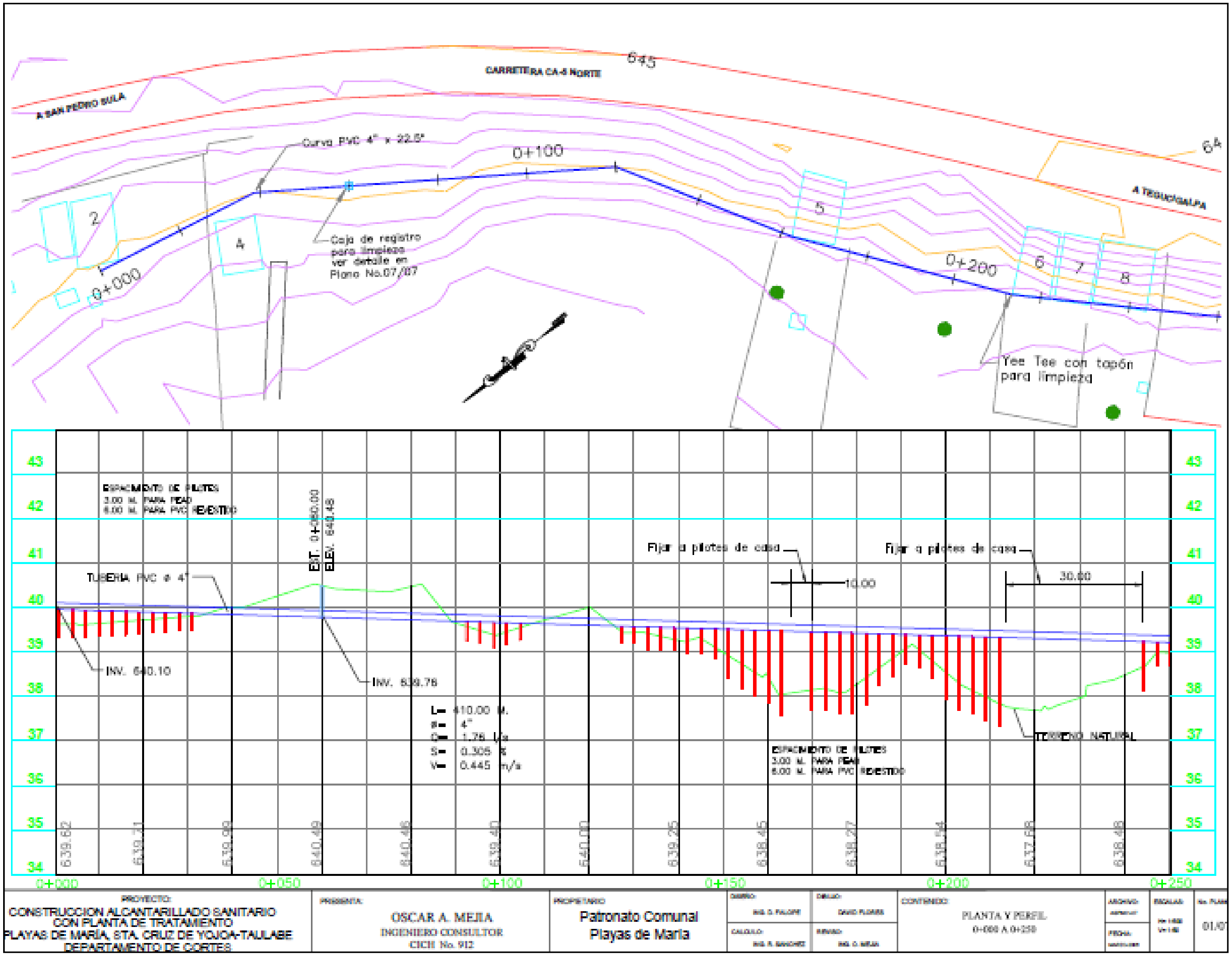
Appendix A: Las Casetas Piping System Plans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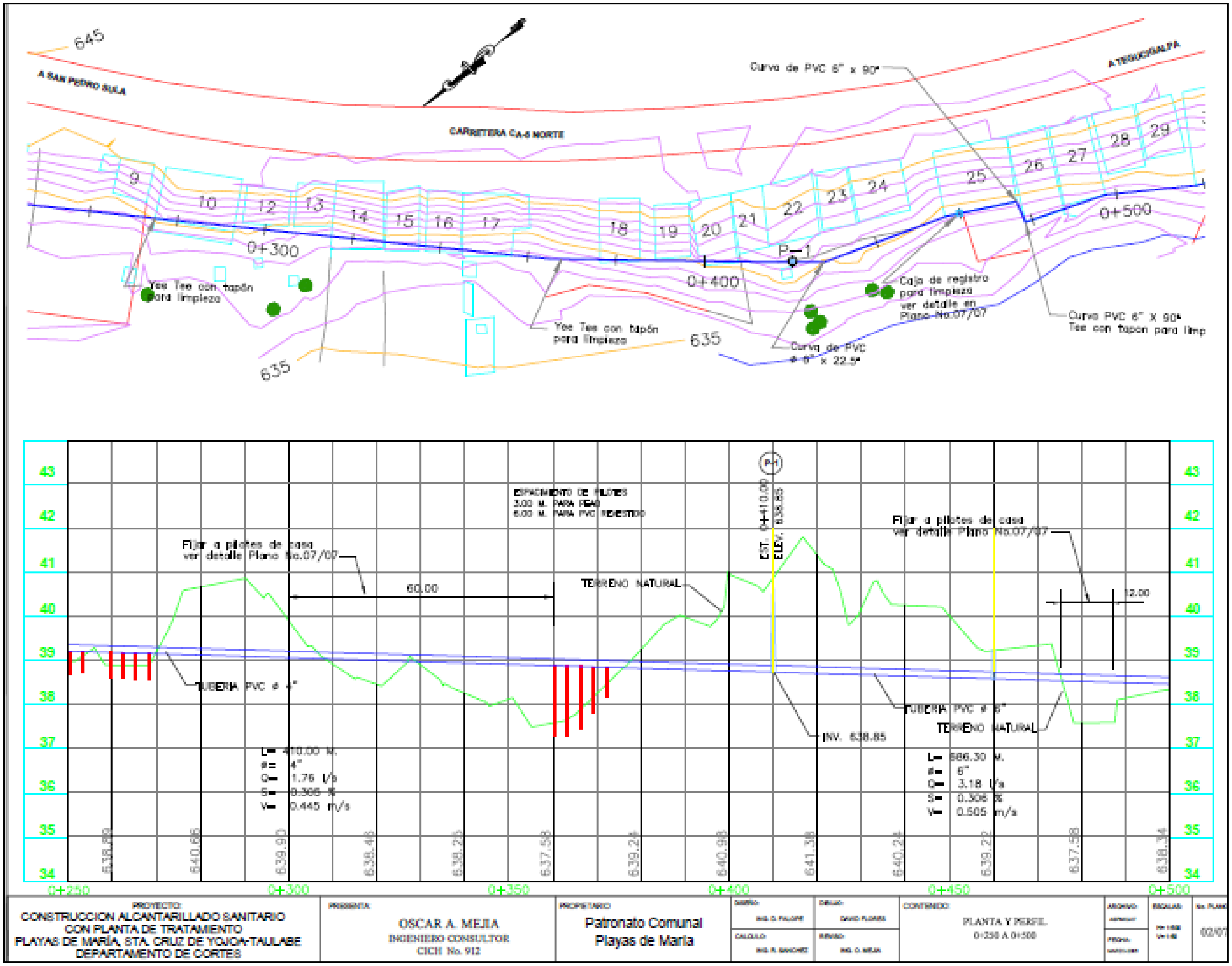
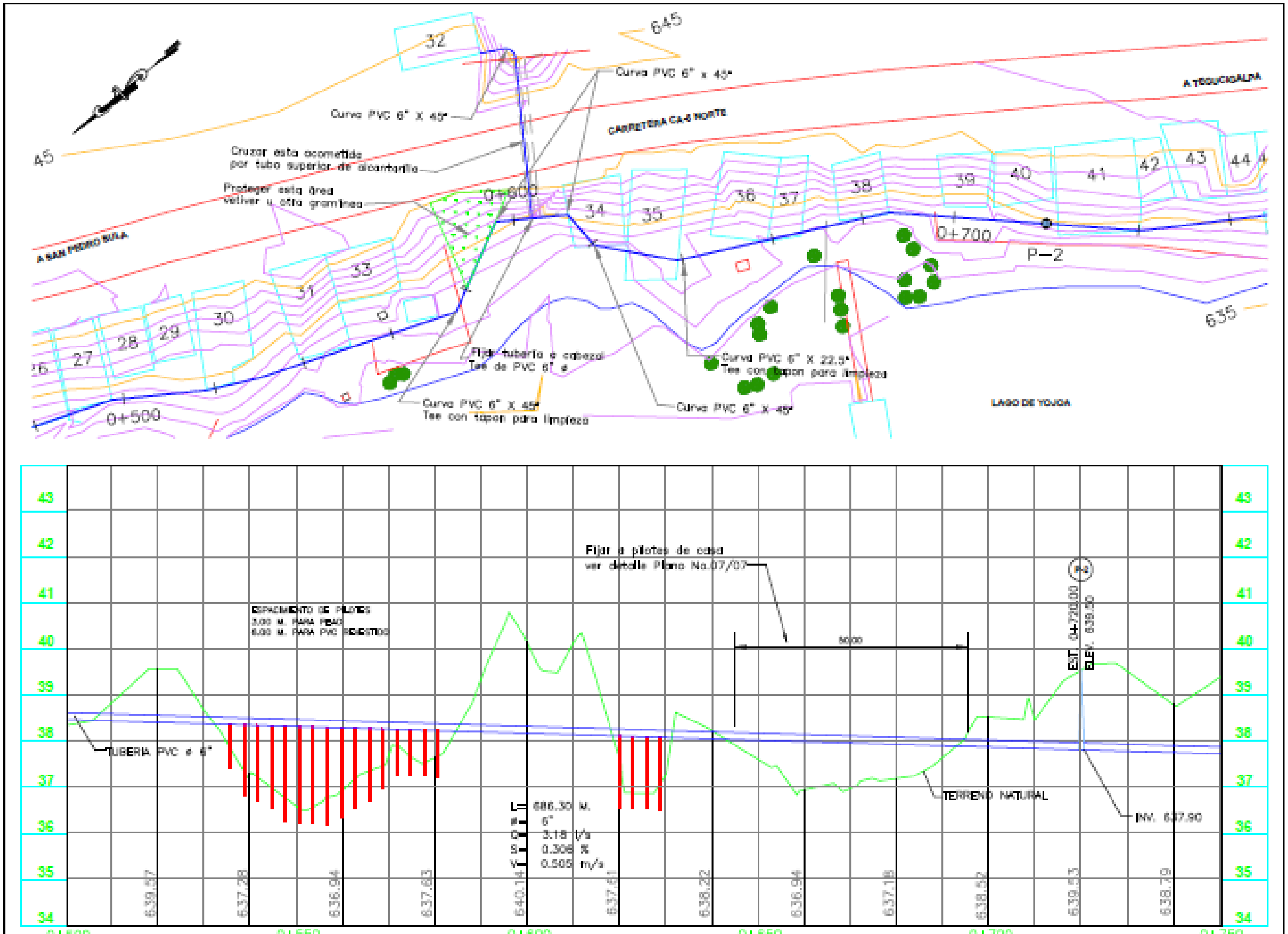
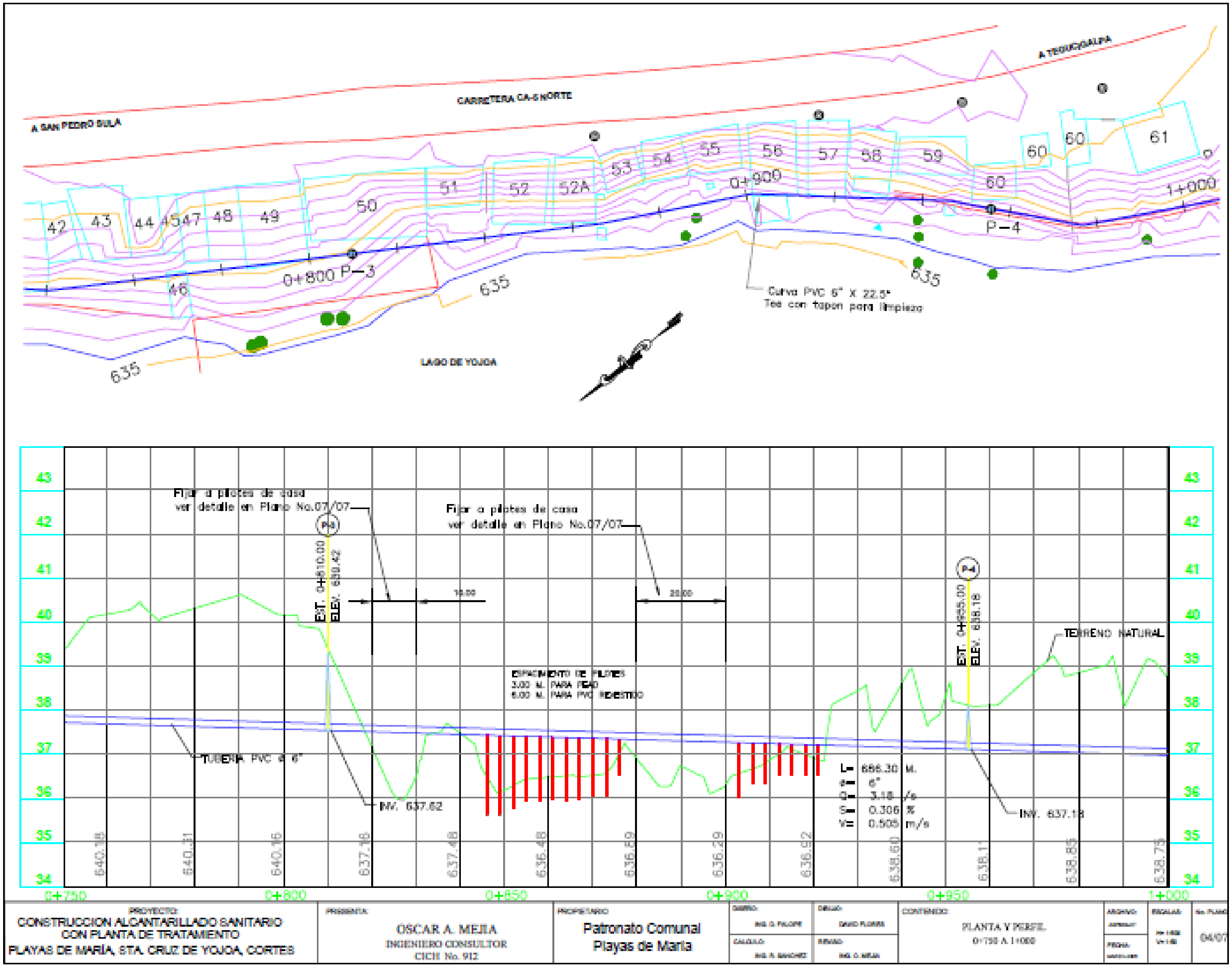
A SAN PEDRO SULA
A TEGUCIGALPA
Curva PVC 4" x 22.5°
Caja de registro para limpieza ver detalle en Plano No.07/07
Yee Tee con tapón para limpieza
0+000
0+100
0+200
ESPACIAMIENTO DE PILOTES
3.00 M. PARA PEAD
6.00 M. PARA PVC REVESTIDO
TUBERIA PVC ø 4"
INV. 640.10
INV. 639.76
EST. 0+060.00
ELEV. 640.48
Fijar a pilotes de casa
Fijar a pilotes de casa
10.00
30.00
TERRENO NATURAL
L= 410.00 M.
ø= 4"
Q= 1.76 l/s
S= 0.305 %
V= 0.445 m/s
PROYECTO:
CONSTRUCCION ALCANTARILLADO SANITARIO CON PLANTA DE TRATAMIENTO
PLAYAS DE MARIA, STA. CRUZ DE YOJOA-TAULABE
DEPARTAMENTO DE CORTES
PRESENTA:
OSCAR A. MEJIA
INGENIERO CONSULTOR
CICH No. 912
PROPIETARIO:
Patronato Comunal
Playas de Maria
CONTENIDO:
PLANTA Y PERFIL
0+000 A 0+250

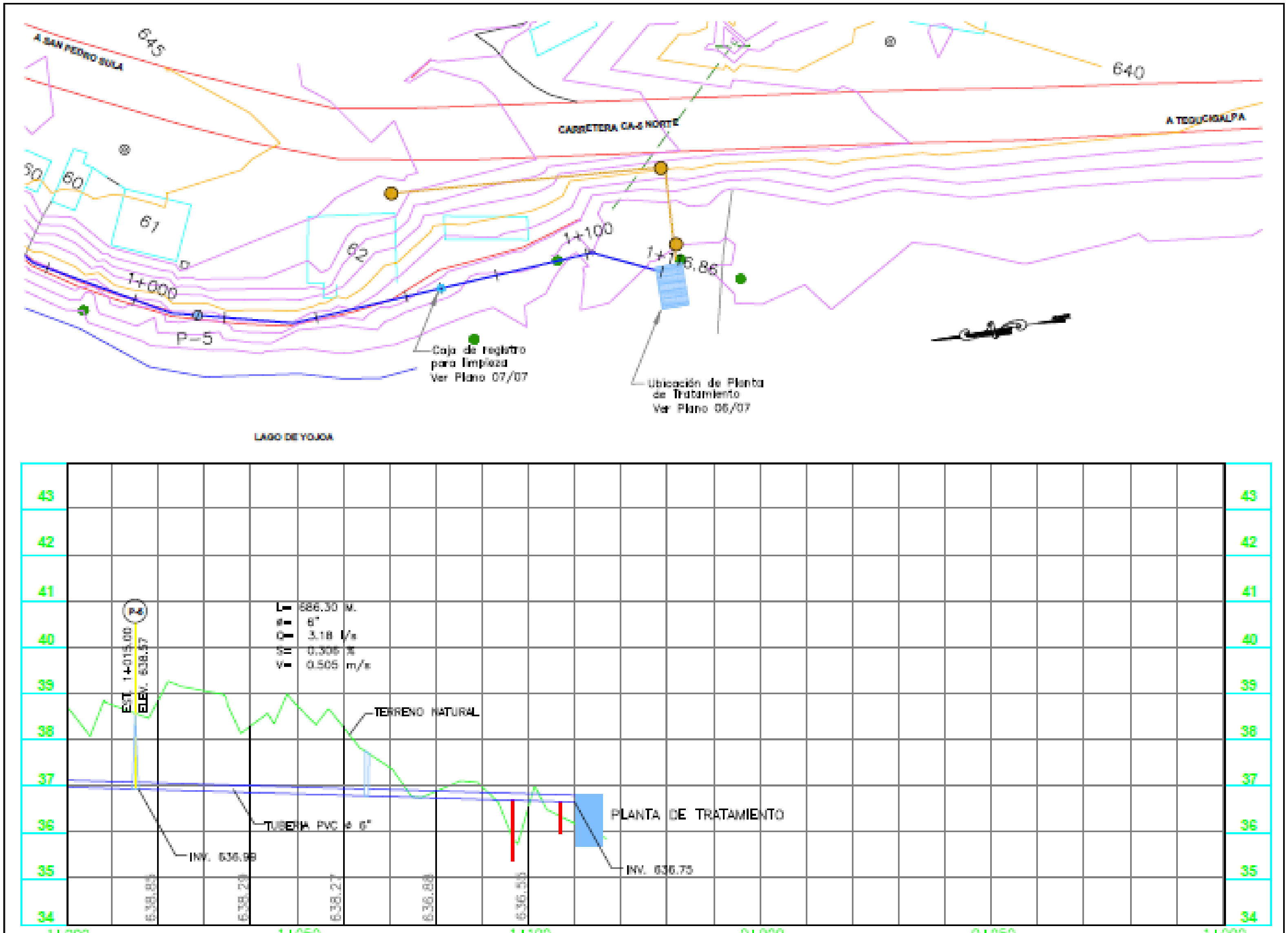


A SAN PEDRO SULA
CARRETERA CA-5 NORTE
A TEGUCIGALPA
Curva de PVC 6" x 90°
Yee Tee con tapón para limpieza
Caja de registro para limpieza ver detalle en Plano No.07/07
Curva de PVC ø 6" x 22.5°
Curva PVC 6" X 90° Tee con tapon para limp
0+300
0+400
0+500
P-1
ESPACIAMIENTO DE PILOTES
3.00 M. PARA PEAD
6.00 M. PARA PVC REVESTIDO
Fijar a pilotes de casa ver detalle Plano No.07/07
60.00
12.00
TERRENO NATURAL
TUBERIA PVC ø 4"
TUBERIA PVC ø 6"
EST. 0+410.00
ELEV. 638.85
INV. 638.85
L= 410.00 M.
ø= 4"
Q= 1.76 l/s
S= 0.306 %
V= 0.445 m/s
L= 586.30 M.
ø= 6"
Q= 3.18 l/s
S= 0.306 %
V= 0.505 m/s
0+250
0+350
0+450
PROYECTO:
CONSTRUCCION ALCANTARILLADO SANITARIO CON PLANTA DE TRATAMIENTO PLAYAS DE MARÍA, STA. CRUZ DE YOJOA-TAULABE DEPARTAMENTO DE CORTES
PRESENTA:
OSCAR A. MEJIA
INGENIERO CONSULTOR
CICH No. 912
PROPIETARIO
Patronato Comunal Playas de María
CONTENIDO:
PLANTA Y PERFIL
0+250 A 0+500
02/07



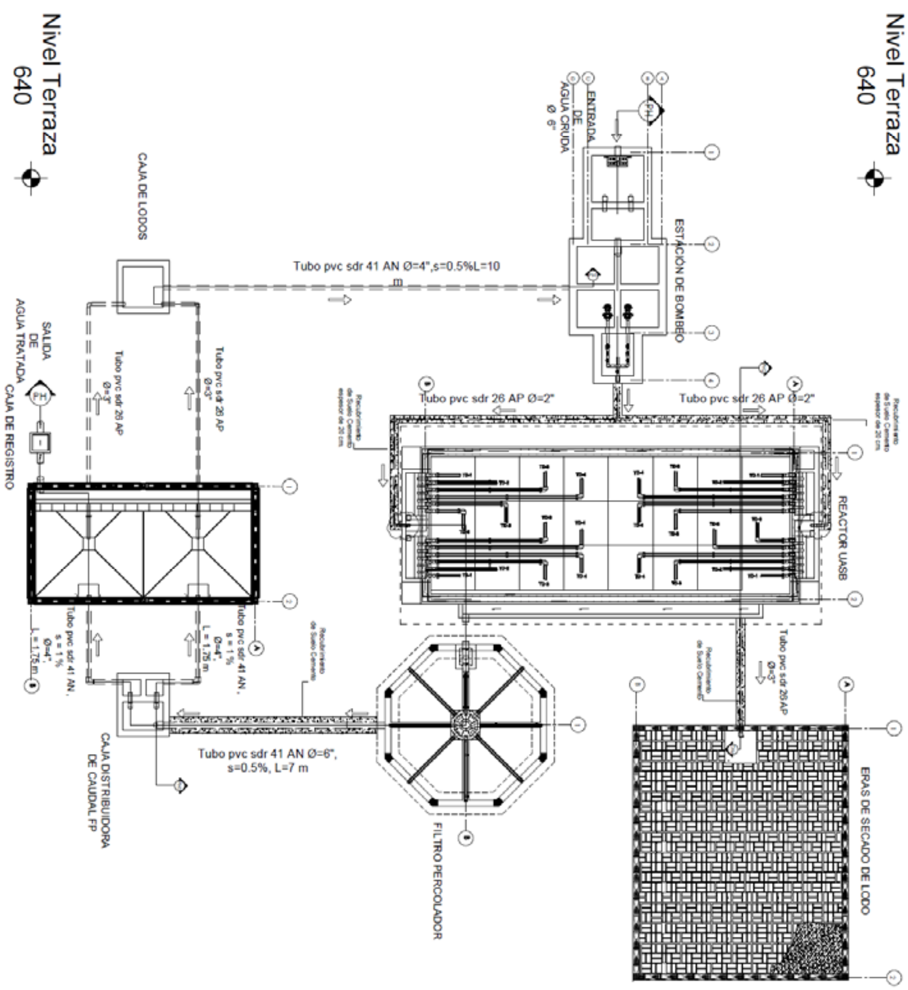
PROYECTO:	PRESENTA:	PROPIETARIO	DISEÑO: / CALCULO:	DIBUJO: / REVISO:	CONTENIDO:	ARCHIVO / FECHA:	ESCALAS	No. PLANO
CONSTRUCCION ALCANTARILLADO SANITARIO CON PLANTA DE TRATAMIENTO PLAYAS DE MARÍA, STA. CRUZ DE YOJOA-TAULABE DEPARTAMENTO DE CORTES	OSCAR A. MEJIA INGENIERO CONSULTOR CICH No. 912	Patronato Comunal Playas de Maria	ING. R. SANCHEZ	DAVID FLORES / ING. O. MEJIA	PLANTA Y PERFIL 0+500 A 0+750			03/07


A SAN PEDRO SULA
CARRETERA CA-5 NORTE
A TEGUCIGALPA
LAGO DE YOJOA
0+800 P-3
0+900
1+000
P-4
635
Curva PVC 6" X 22.5°
Tee con tapon para limpieza
Fijar a pilotes de casa
ver detalle en Plano No.07/07
EXT. 0+810.00
ELEV. 630.42
ESPACIAMIENTO DE PILOTES
3.00 M. PARA PEAD
6.00 M. PARA PVC REVESTIDO
TUBERIA PVC ø 6"
INV. 637.62
INV. 637.18
EXT. 0+955.00
ELEV. 638.18
TERRENO NATURAL
L= 686.30 M.
ø= 6"
Q= 3.18 l/s
S= 0.306 %
V= 0.505 m/s
0+750
0+800
0+850
0+900
0+950
1+000
PROYECTO:
CONSTRUCCION ALCANTARILLADO SANITARIO
CON PLANTA DE TRATAMIENTO
PLAYAS DE MARÍA, STA. CRUZ DE YOJOA, CORTES
PRESENTA:
OSCAR A. MEJIA
INGENIERO CONSULTOR
CICH No. 912
PROPIETARIO:
Patronato Comunal
Playas de Maria
CONTENIDO:
PLANTA Y PERFIL
0+750 A 1+000
04/07



PROYECTO: CONSTRUCCION ALCANTARILLADO SANITARIO CON PLANTA DE TRATAMIENTO PLAYAS DE MARÍA, STA. CRUZ DE YOJOA-TAULABE DEPARTAMENTO DE CORTES	PRESENTA: OSCAR A. MEJIA INGENIERO CONSULTOR CICH No. 912	PROPIETARIO: Patronato Comunal Playas de María	CONTENIDO: PLANTA Y PERFIL 1+000 A 1+100	No. PLANO: 05/07

Appendix B: Las Casetas Treatment Plant Plans



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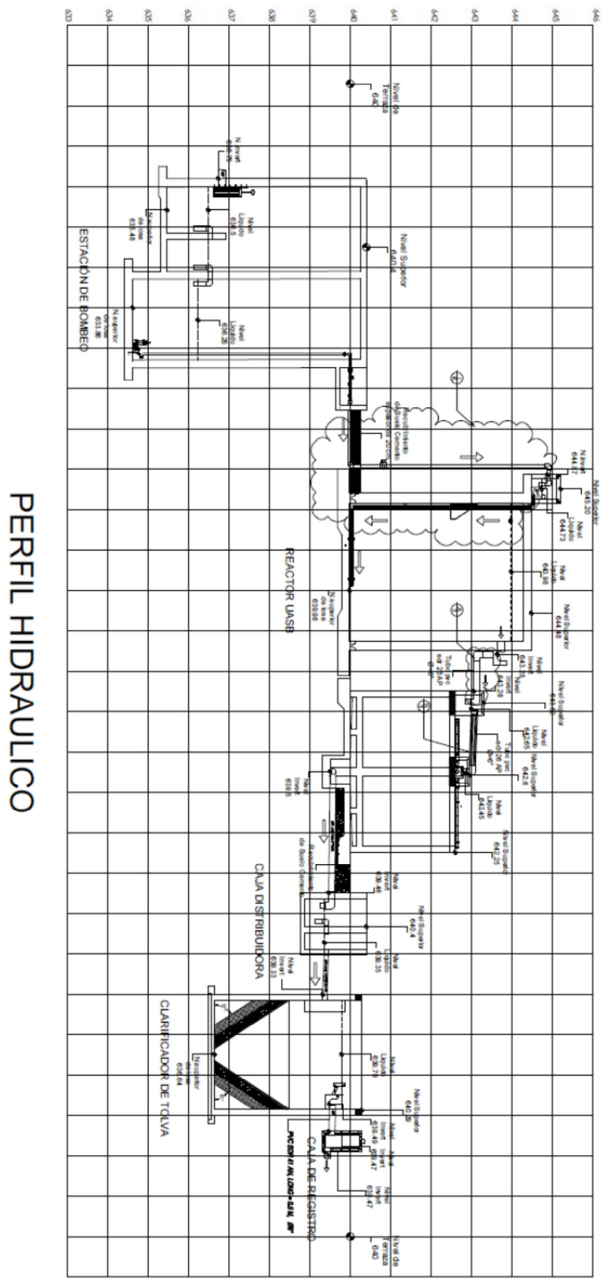
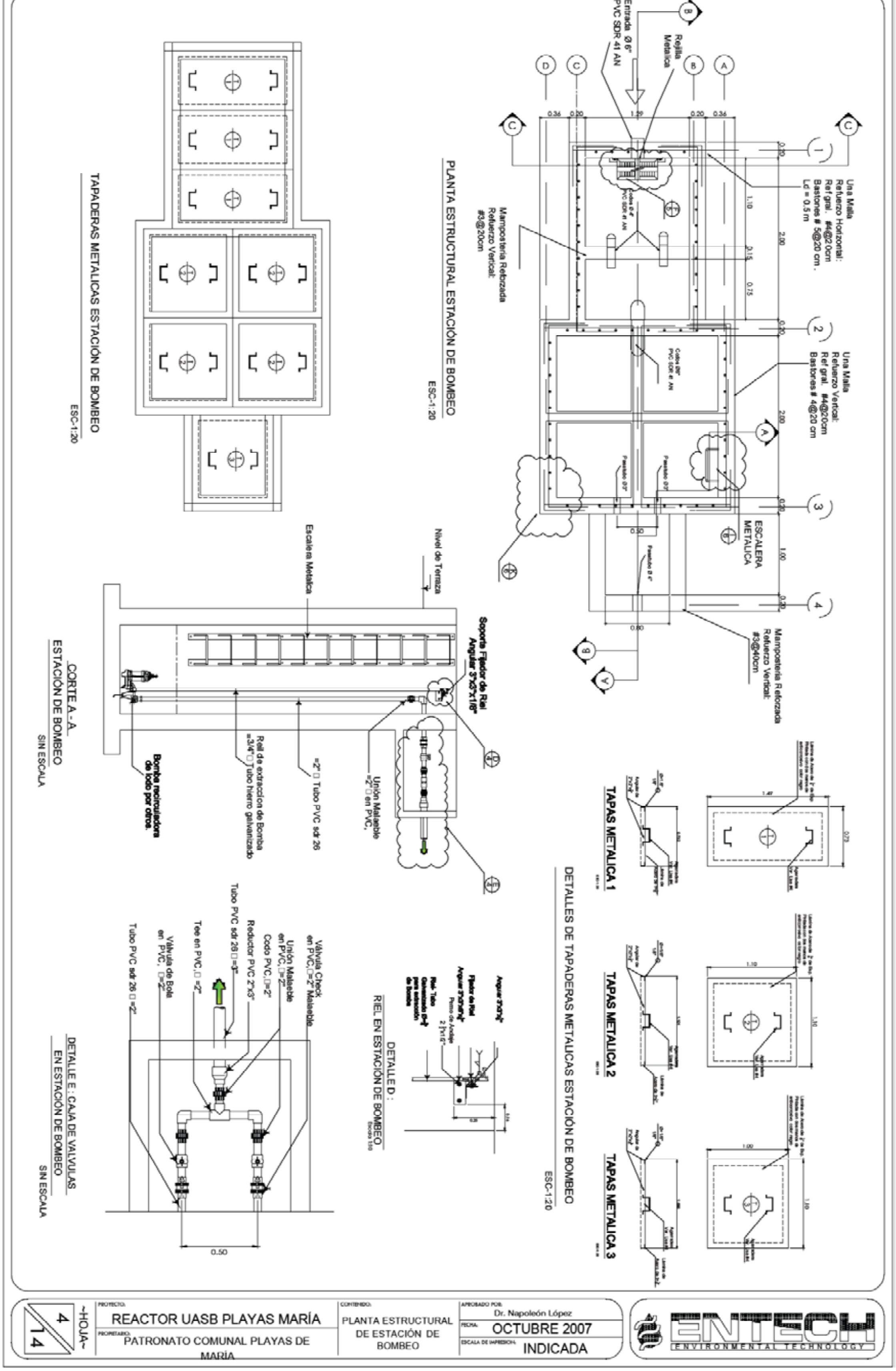
	PROPIETARIO: PATRONATO COMUNAL PLAYAS DE MARÍA		FECHA: OCTUBRE 2007
			ESCALA DE IMPRESIÓN: INDICADA

ENTECH
ENVIRONMENTAL TECHNOLOGY



~HOJA~ 2 / 14	PROYECTO: REACTOR UASB PLAYAS MARÍA PROPIETARIO: PATRONATO COMUNAL PLAYAS DE MARÍA	CONTENIDO: PERFIL HIDRÁULICO	APROBADO POR: Dr. Napoleón López FECHA: OCTUBRE 2007 ESCALA DE IMPRESION: INDICADA

ENTECH ENVIRONMENTAL TECHNOLOGY



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TAPADERAS METALICAS ESTACIÓN DE BOMBEO
ESC-1:20
DETALLES DE TAPADERAS METALICAS ESTACIÓN DE BOMBEO
ESC-1:20
TAPAS METALICA 1
TAPAS METALICA 2
TAPAS METALICA 3
CORTE A - A
ESTACIÓN DE BOMBEO
SIN ESCALA
DETALLE D :
RIEL EN ESTACIÓN DE BOMBEO
DETALLE E : CAJA DE VALVULAS
EN ESTACIÓN DE BOMBEO
SIN ESCALA
ESCALERA METALICA
Escalera Metalica
Nivel de Terraza
Entrada Ø 6" PVC SDR 41 AN
Rejilla Metalica
Mampostería Reforzada
Refuerzo Vertical:
#3@20cm
#3@40cm
Una Malla
Refuerzo Horizontal:
Refuerzo Vertical:
Bastones # 4@20 cm
Ld = 0.5 m
Soporte Fijador de Riel
Angular 3"x3"x1/8"
Unión Maleable
"2" en PVC.
"2" Tubo PVC sdr 26
Riel de extracción de Bomba
Tubo hierro galvanizado
Bomba recirculadora de lodo por ciclos.
Válvula Check en PVC "2" Maleable
Unión Maleable en PVC "2"
Codo PVC "2"
Reductor PVC 2"x3"
Tubo PVC sdr 26 "3"
Tee en PVC "2"
Válvula de Bola en PVC "2"
Tubo PVC sdr 26 "2"
0.50
PROYECTO:
REACTOR UASB PLAYAS MARÍA
PROPIETARIO:
PATRONATO COMUNAL PLAYAS DE MARÍA
CONTENIDO:
PLANTA ESTRUCTURAL DE ESTACIÓN DE BOMBEO
APROBADO POR:
Dr. Napoleón López
FECHA:
OCTUBRE 2007
ESCALA DE IMPRESIÓN:
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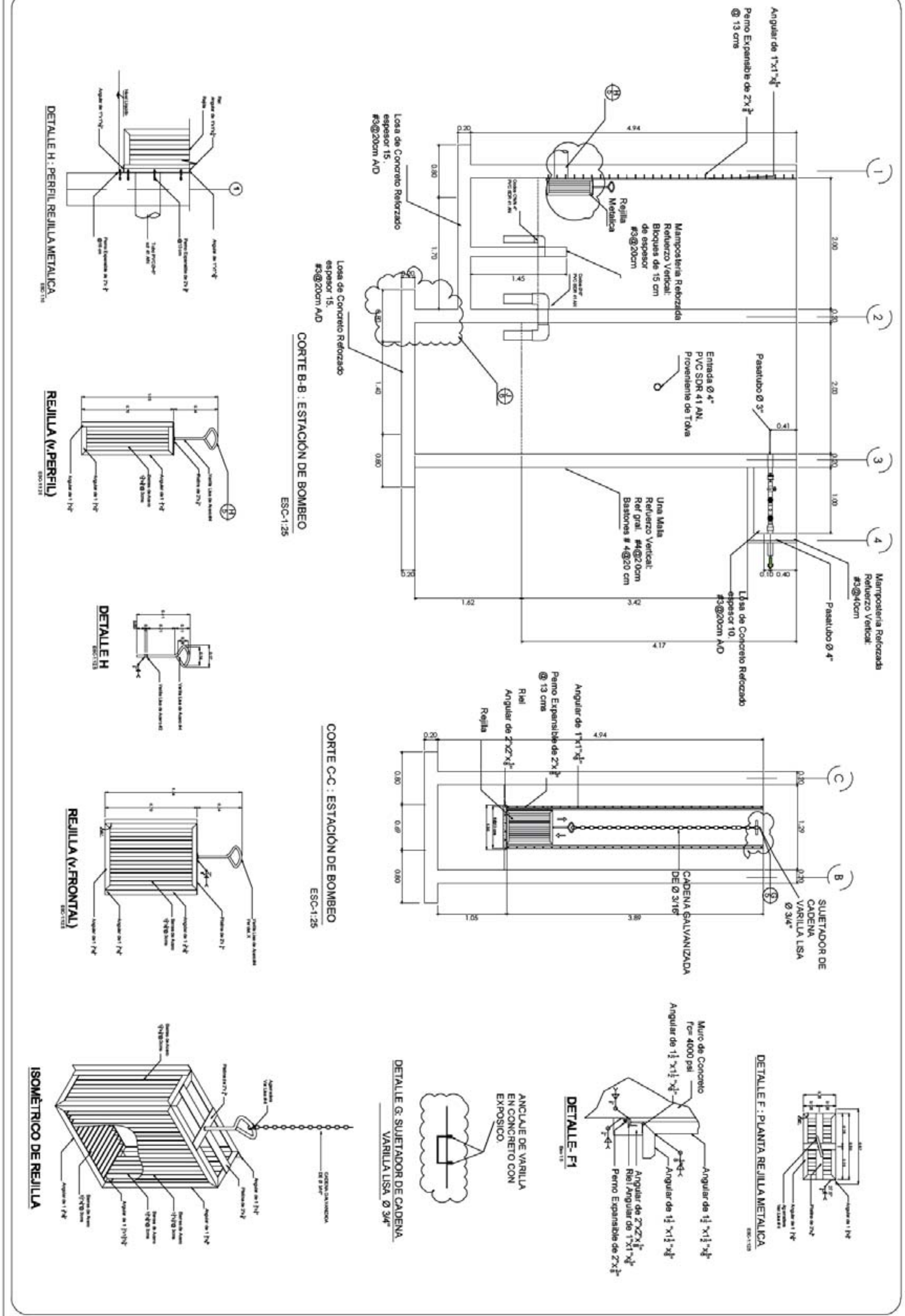
~HOJA~
4
14
ENTECH
ENVIRONMENTAL TECHNOLOGY



~HOJA~ 5/14

PROYECTO: REACTOR UASB PLAYAS MARÍA

PROPIETARIO: PATRONATO COMUNAL PLAYAS DE MARÍA

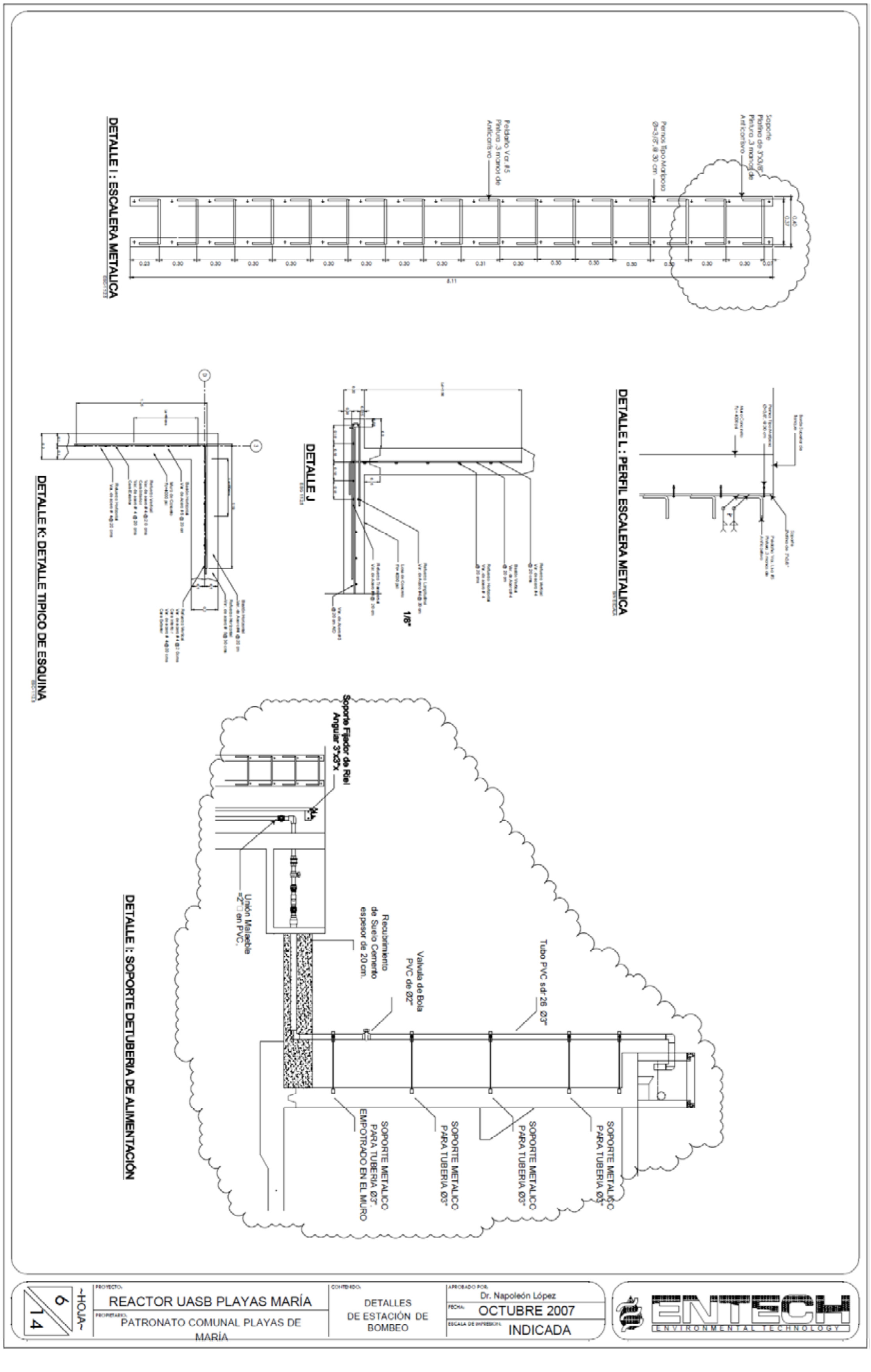
CONTENIDO: PERFIL ESTRUCTURAL DE ESTACIÓN DE BOMBEO

APROBADO POR: Dr. Napoleón López

FECHA: OCTUBRE 2007

ESCALA DE IMPRESION: INDICADA

ENTECH ENVIRONMENTAL TECHNOLOGY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DETALLE L : PERFIL ESCALERA METALICA
DETALLE J
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DETALLE K: DETALLE TIPICO DE ESQUINA
Soporte Fijador de Riel Angular 3"x3"x
Tubo PVC sdr 26 Ø3"
Valvula de Bola PVC de Ø2"
Recubrimiento de Suelo Cemento espesor de 20 cm.
Unión Maleable ±2" en PVC.
SOPORTE METALICO PARA TUBERIA Ø3"
SOPORTE METALICO PARA TUBERIA Ø3"
SOPORTE METALICO PARA TUBERIA Ø3"
SOPORTE METALICO PARA TUBERIA Ø3" EMPOTRADO EN EL MURO
DETALLE I: SOPORTE DETUBERIA DE ALIMENTACIÓN
~HOJA~
6
14
PROYECTO:
REACTOR UASB PLAYAS MARÍA
PROPIETARIO:
PATRONATO COMUNAL PLAYAS DE MARÍA
CONTENIDO:
DETALLES DE ESTACIÓN DE BOMBEO
APROBADO POR:
Dr. Napoleón López
FECHA:
OCTUBRE 2007
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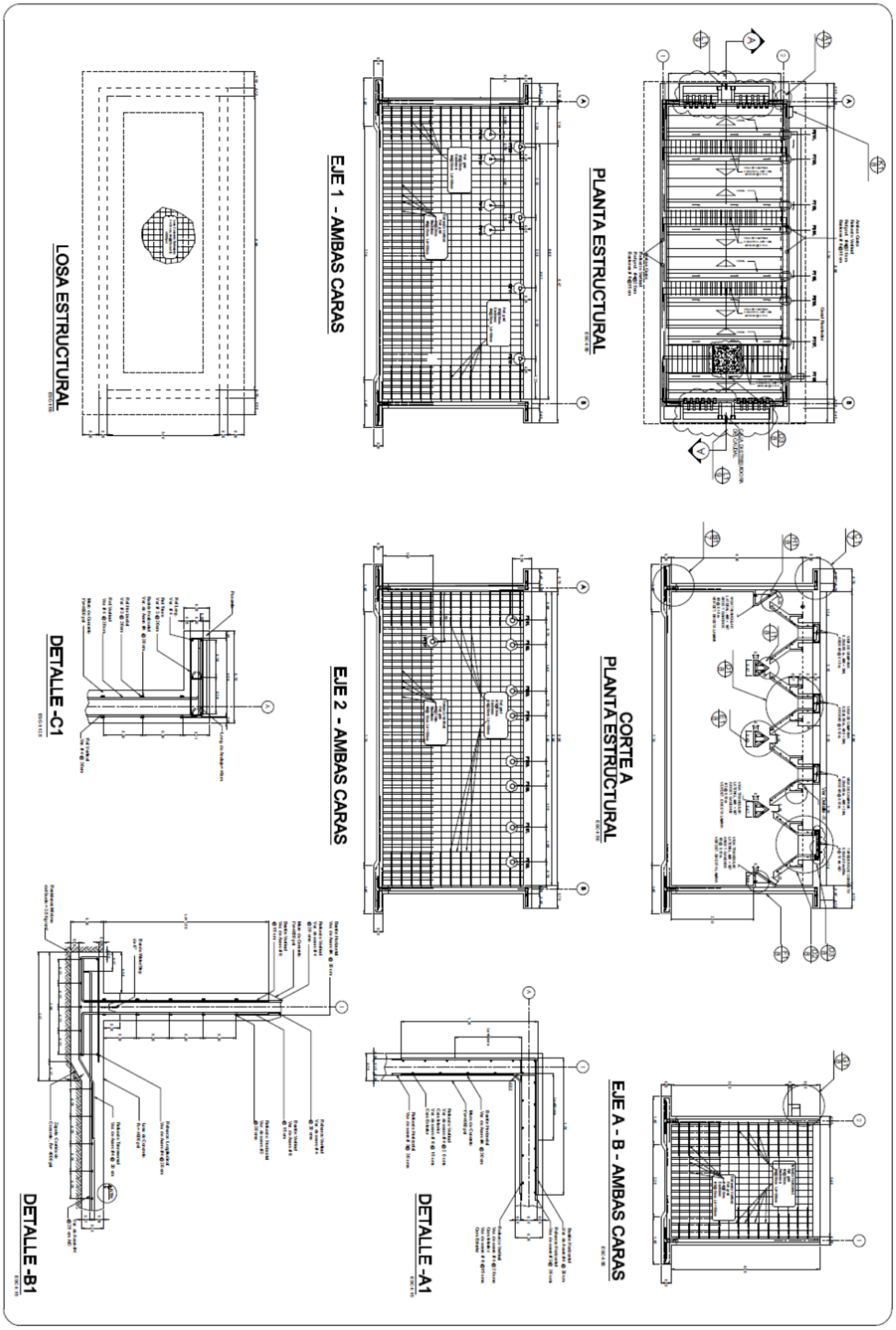
ENVIRONMENTAL TECHNOLOGY



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	PROPIETARIO: PATRONATO COMUNAL PLAYAS DE MARÍA		FECHA: OCTUBRE 2007
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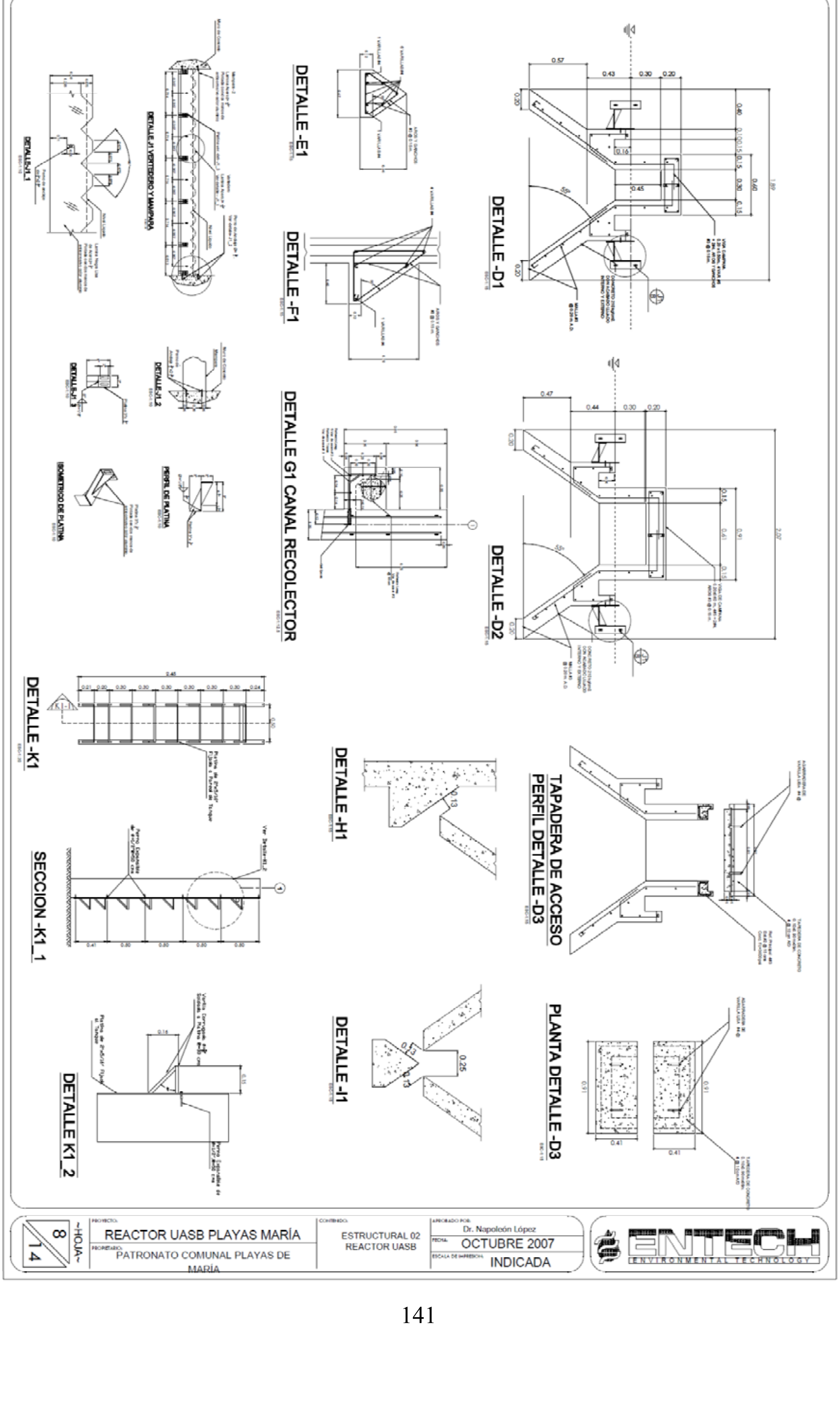
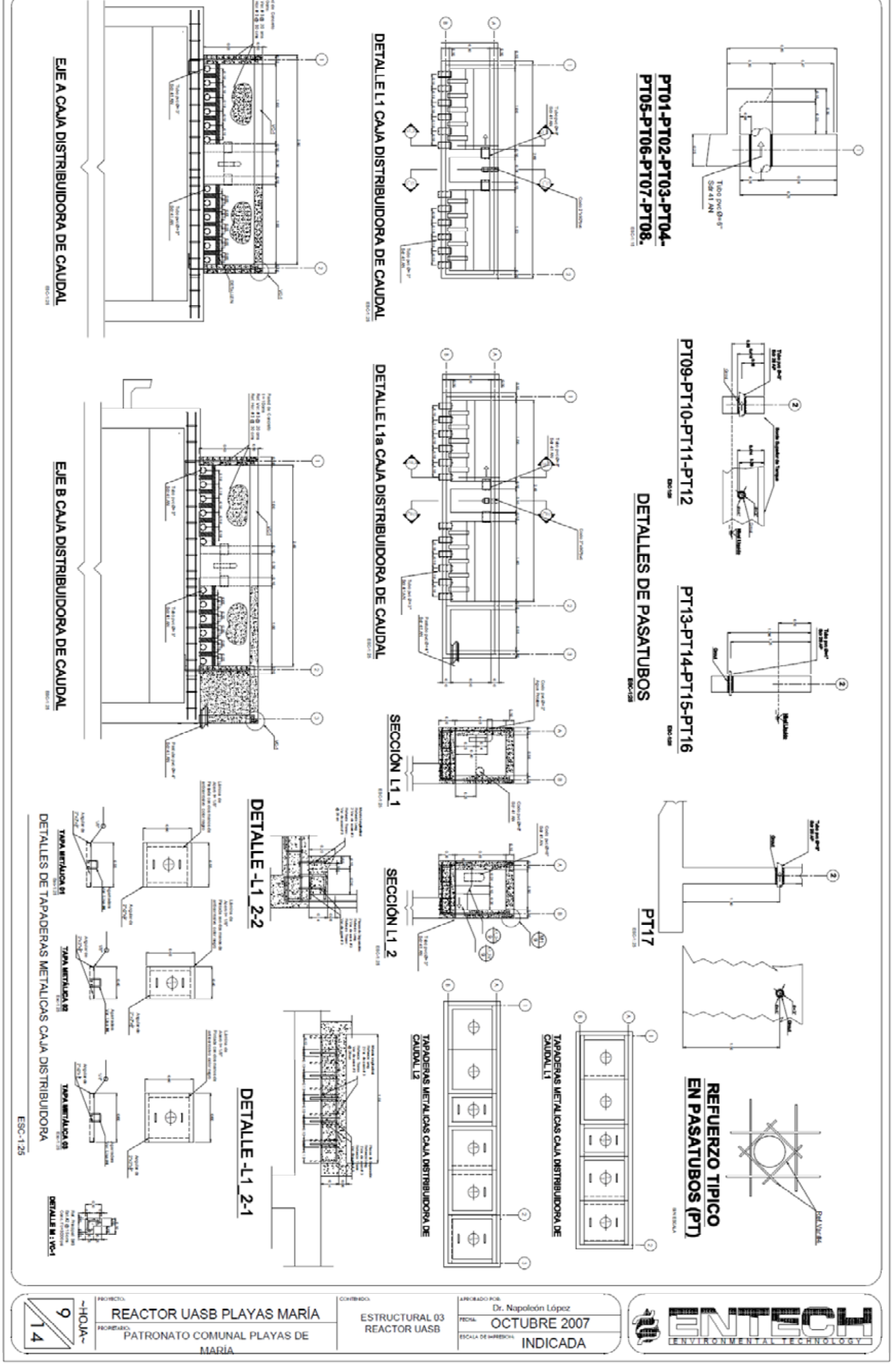
ENTECH
ENVIRONMENTAL TECHNOLOGY


DETALLE -D1
DETALLE -D2
DETALLE -E1
DETALLE -F1
DETALLE G1 CANAL RECOLECTOR
DETALLE-J1 VERTEDERO Y MAMPARA
DETALLE-J1_1
DETALLE-J1_2
DETALLE-J1_3
PERFIL DE PLATINA
ISOMETRICO DE PLATINA
TAPADERA DE ACCESO
PERFIL DETALLE -D3
PLANTA DETALLE -D3
DETALLE -H1
DETALLE -I1
DETALLE -K1
SECCION -K1_1
DETALLE K1_2
8
14
~HOJA~
PROYECTO:
REACTOR UASB PLAYAS MARÍA
PROPIETARIO:
PATRONATO COMUNAL PLAYAS DE MARÍA
CONTENIDO:
ESTRUCTURAL 02
REACTOR UASB
APROBADO POR:
Dr. Napoleón López
FECHA:
OCTUBRE 2007
ESCALA DE IMPRESION:
INDICADA
ENTECH
ENVIRONMENTAL TECHNOLOGY


PT01-PT02-PT03-PT04-PT05-PT06-PT07-PT08.
PT09-PT10-PT11-PT12
DETALLES DE PASATUBOS
PT13-PT14-PT15-PT16
PT17
REFUERZO TIPICO EN PASATUBOS (PT)
DETALLE L1 CAJA DISTRIBUIDORA DE CAUDAL
DETALLE L1a CAJA DISTRIBUIDORA DE CAUDAL
SECCIÓN L1_1
SECCIÓN L1_2
TAPADERAS METALICAS CAJA DISTRIBUIDORA DE CAUDAL L1
TAPADERAS METALICAS CAJA DISTRIBUIDORA DE CAUDAL L2
EJE A CAJA DISTRIBUIDORA DE CAUDAL
EJE B CAJA DISTRIBUIDORA DE CAUDAL
DETALLE -L1_2-2
DETALLE -L1_2-1
DETALLES DE TAPADERAS METALICAS CAJA DISTRIBUIDORA
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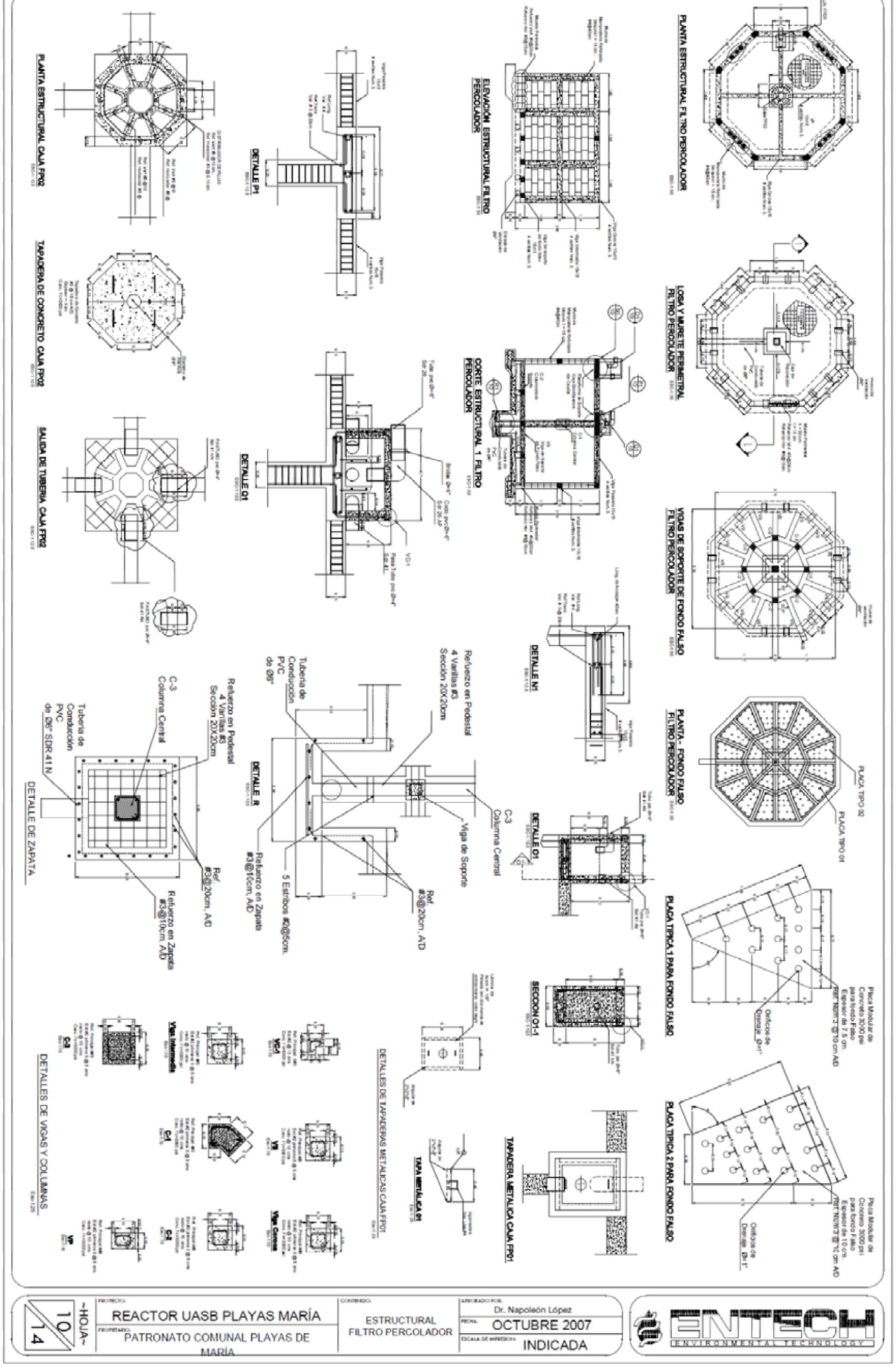
TAPA METALICA 02
TAPA METALICA 03
9
14
~HOJA~
REACTOR UASB PLAYAS MARÍA
PATRONATO COMUNAL PLAYAS DE MARÍA
ESTRUCTURAL 03
REACTOR UASB
Dr. Napoleón López
OCTUBRE 2007
INDICADA
ENTECH
ENVIRONMENTAL TECHNOLOGY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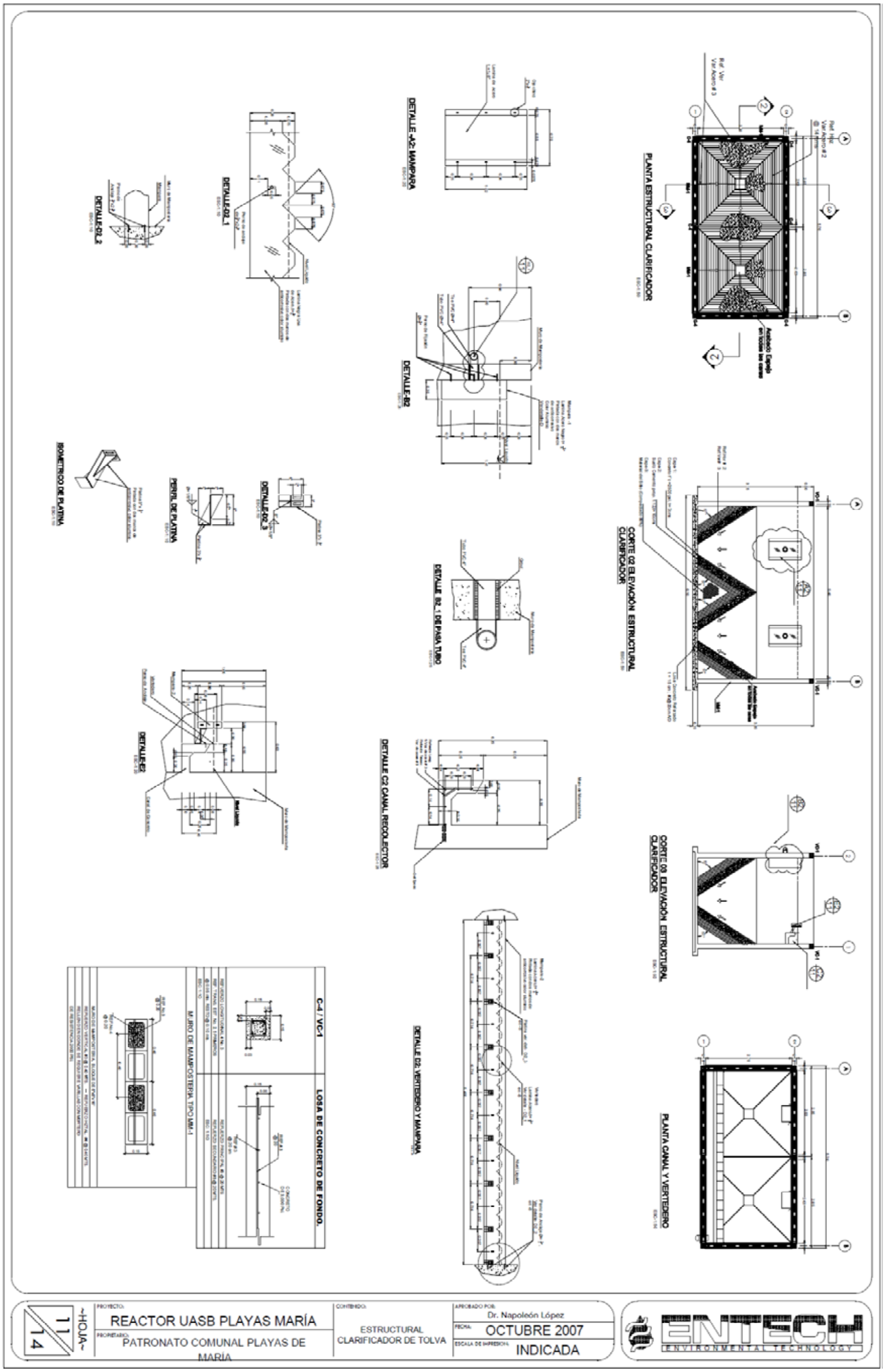
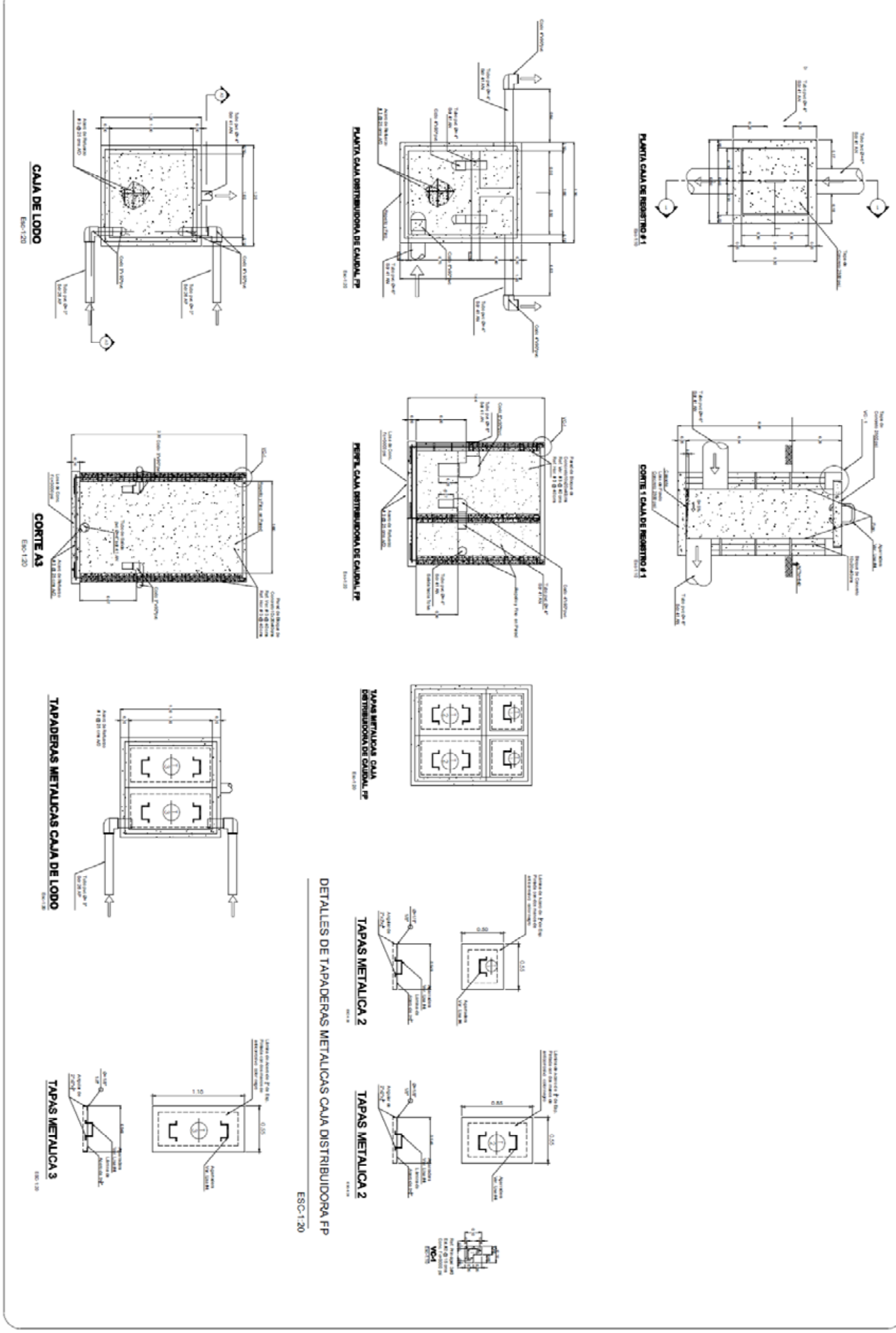
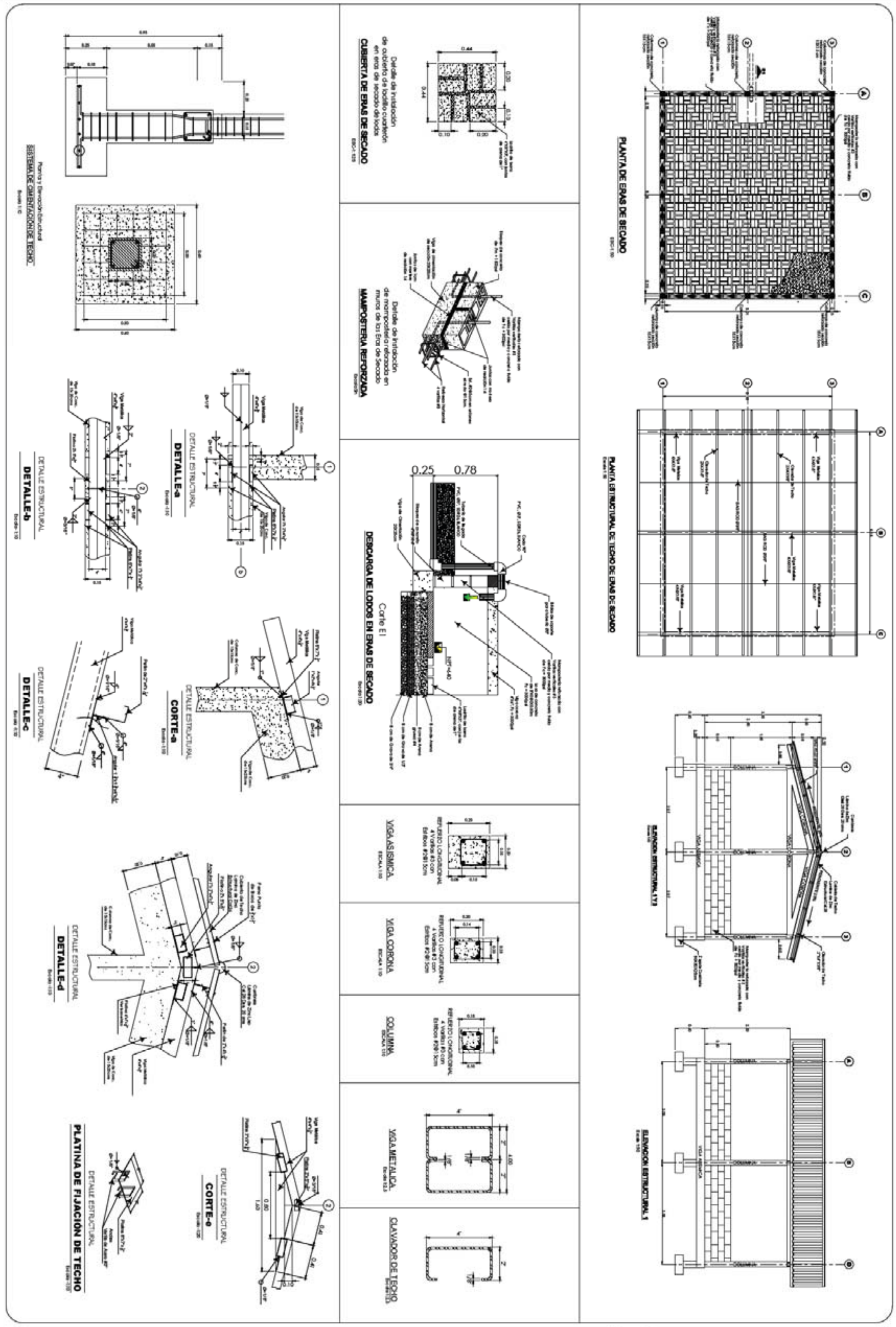
LOSA Y MURETE PERIMETRAL FILTRO PERCOLADOR
VIGAS DE SOPORTE DE FONDO FALSO FILTRO PERCOLADOR
PLANTA - FONDO FALSO FILTRO PERCOLADOR
PLACA TIPO 02
PLACA TIPO 01
PLACA TIPICA 1 PARA FONDO FALSO
PLACA TIPICA 2 PARA FONDO FALSO
Orificios de Drenaje Ø=1"
ELEVACIÓN ESTRUCTURAL FILTRO PERCOLADOR
CORTE ESTRUCTURAL 1 FILTRO PERCOLADOR
DETALLE N1
DETALLE O1
SECCION O1-1
TAPADERA METALICA CAJA FP01
TAPA METALICA 01
DETALLES DE TAPADERAS METALICAS CAJA FP01
DETALLE P1
DETALLE Q1
DETALLE R
Refuerzo en Pedestal 4 Varillas #3 Sección 20X20cm
Tubería de Conducción PVC de Ø6"
C-3 Columna Central
Viga de Soporte
Ref. #3@20cm. A/D
5 Estribos #2@5cm.
Refuerzo en Zapata #3@10cm. A/D
Tubería de Conducción PVC de Ø6" SDR-41 N
DETALLE DE ZAPATA
PLANTA ESTRUCTURAL CAJA FP02
TAPADERA DE CONCRETO CAJA FP02
SALIDA DE TUBERIA CAJA FP02
DETALLES DE VIGAS Y COLUMNAS
PROYECTO: REACTOR UASB PLAYAS MARÍA
PROPIETARIO: PATRONATO COMUNAL PLAYAS DE MARÍA
CONTENIDO: ESTRUCTURAL FILTRO PERCOLADOR
APROBADO POR: Dr. Napoleón López
FECHA: OCTUBRE 2007
ESCALA DE IMPRESIÓN: INDICADA
~HOJA~
10
14
ENTECH
ENVIRONMENTAL TECHNOLOGY


PLANTA ESTRUCTURAL CLARIFICADOR
CORTE 02 ELEVACIÓN ESTRUCTURAL CLARIFICADOR
CORTE 03 ELEVACIÓN ESTRUCTURAL CLARIFICADOR
PLANTA CANAL Y VERTEDERO
DETALLE A2 MAMPARA
DETALLE B2
DETALLE B2_1 DE PASA TUBO
DETALLE C2 CANAL RECOLECTOR
DETALLE D2 VERTEDERO Y MAMPARA
DETALLE E2
DETALLE E2_1
DETALLE E2_2
DETALLE E2_3
PERFIL DE PLATINA
ISOMETRICO DE PLATINA
C-4 / VC-1
LOSA DE CONCRETO DE FONDO.
MURO DE MAMPOSTERIA TIPO MM-1
~HOJA~
11
14
PROYECTO:
REACTOR UASB PLAYAS MARÍA
PROPIETARIO:
PATRONATO COMUNAL PLAYAS DE MARÍA
CONTENIDO:
ESTRUCTURAL
CLARIFICADOR DE TOLVA
APROBADO POR:
Dr. Napoleón López
FECHA:
OCTUBRE 2007
ESCALA DE IMPRESION:
INDICADA
ENTECH
ENVIRONMENTAL TECHNOLOGY

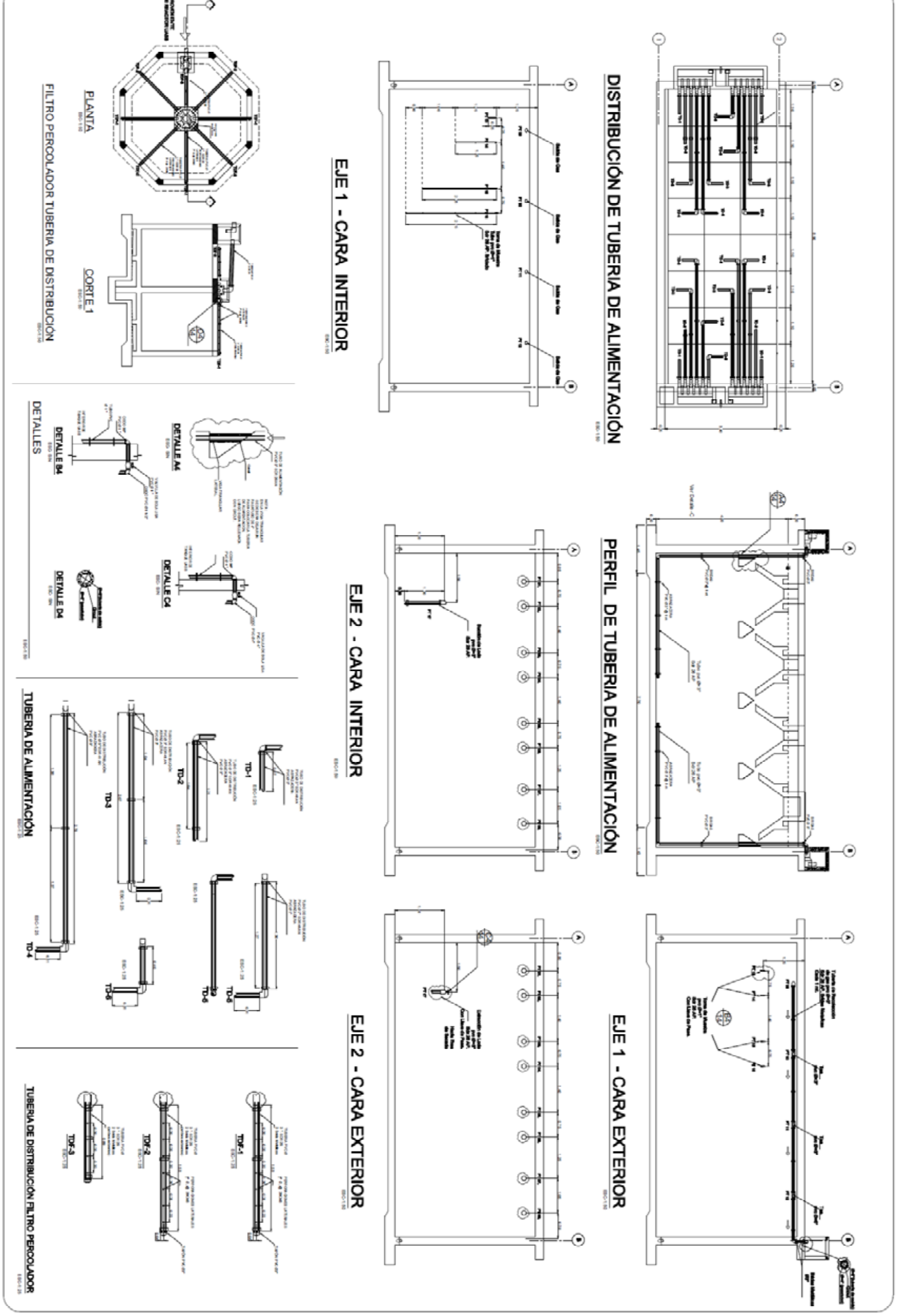


~HOJA~ 12/14	PROYECTO: REACTOR UASB PLAYAS MARÍA	CONTENIDO: CAJAS	APROBADO POR: Dr. Napoleón López	ENTECH ENVIRONMENTAL TECHNOLOGY
	PROPIETARIO: PATRONATO COMUNAL PLAYAS DE MARÍA		FECHA: OCTUBRE 2007	
			ESCALA DE IMPRESIÓN: INDICADA	



~HOJA~ 13 / 14	PROYECTO: REACTOR UASB PLAYAS MARÍA PROPIETARIO: PATRONATO COMUNAL PLAYAS DE MARÍA	CONTENIDO: ERAS DE SECADO	APROBADO POR: Dr. Napoleón López FECHA: OCTUBRE 2007 ESCALA DE IMPRESIÓN: INDICADA

ENTECH ENVIRONMENTAL TECHNOLOGY



~HOJA~ 14 / 14

PROYECTO: REACTOR UASB PLAYAS MARÍA

PROPIETARIO: PATRONATO COMUNAL PLAYAS DE MARÍA

CONTENIDO: INSTALACIÓN

APROBADO POR: Dr. Napoleón López

FECHA: OCTUBRE 2007

ESCALA DE IMPRESIÓN: INDICADA

ENTECH ENVIRONMENTAL TECHNOLOGY