Honduras Wastewater Treatment: Chemically Enhanced Primary Treatment and Sustainable Secondary Treatment Technologies for Use with Imhoff Tanks

by

Robert C. McLean

B.S. Civil Engineering
Environmental Engineering Concentration
California State Polytechnic University – Pomona, 2008

SUBMITTED TO THE DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF

MASTERS OF ENGINEERING IN CIVIL AND ENVIRONMENTAL ENGINEERING
AT THE
MASSACHUSETTS INSTITUTE OF TECHNOLOGY

JUNE 2009

Signature of Author……………………………………………………………………………………
Department of Civil and Environmental Engineering
May 8, 2009

Certified by………………………………………………………………………………………………
E. Eric Adams
Senior Research Engineer and Lecturer of Civil and Environmental Engineering
Thesis Supervisor

Accepted by……………………………………………………………………………………………...
Who is this person??????????

Comment [EA1]: Prof. Daniele Veneziano; see previous memo:
Accepted by………. Daniele Veneziano
Chairman, Departmental Committee for Graduate Students

Honduras Wastewater Treatment: Chemically Enhanced Primary Treatment and Sustainable Secondary Treatment Technologies for Use with Imhoff Tanks

by

Robert C. McLean

Submitted to the Department of Civil and Environmental Engineering
on May 8, 2009 in partial fulfillment of the
requirements for the Degree of Masters of Engineering in
Civil and Environmental Engineering

ABSTRACT

Wastewater treatment within Honduras is indicative of the state of water and sanitation services throughout the developing portions of Central America. One technology which comprises approximately 40 percent of all treatment facilities within Honduras is the Imhoff tank. First patented in 1907 the Imhoff tank has long been out of favor within the developed world as newer technologies and large centralized processing of wastewater have developed. However, Imhoff tanks are still considered appropriate primary treatment technology for decentralized facilities found within Honduras. A large number of systems have fallen into various states of disrepair due to neglect through lack of proper maintenance. One system within the municipality of Las Vegas, Honduras was examined extensively to expand upon the appropriateness of rehabilitating these systems through various enhancement technologies. Water quality measurements were obtained for the Las Vegas system and it was found to be providing only negligible removals of wastewater constituents. Two large factors which figure into this are: measured flow rates were approximately 50 percent higher than originally anticipated in design and ~~neglect in performing~~ routine maintenance on the system has been neglected.

Comment [EA2]: Parallel construction

Utilizing the coagulant ferric chloride it was possible to increase removal efficiencies and achieve regulatory effluent standards with chemically enhanced primary treatment despite the high flows. However, it is doubtful that the costs associated with dosages required to achieve these removals are sustainable for communities such as Las Vegas. To address these deficiencies further sustainable practices for optimizing the Imhoff tanks as well as designs for both pre-treatment and secondary treatment options appropriate for use in Honduras were developed. With the recommended system it is possible to achieve regulatory effluent levels while maintaining low annual operating costs for the system.

Comment [EA3]: Were all regs able to me met?

Comment [EA4]: Well-written abstract

Thesis Supervisor: E. Eric Adams
Title: Senior Research Engineer and Lecturer of Civil and Environmental Engineering

Table of Contents

Table of Figures

Listing of Tables

Chapter 1: Introduction

The provision of adequate water and sanitation services within a society is vital to the quality of life for it members. Supplies of clean potable water are critical in preventing the spread of water borne diseases such as cholera, typhoid, and dysentery; all of which can be fatal, especially in locations lacking advanced medical practices. Processing and treatment of water once it has been contaminated through use is required before it can be returned to the water supply. Therefore, treatment of wastewater is paramount to continuing the supply of future fresh water for use.

Wastewater treatment within Honduras is indicative of that found throughout the developing portions of Central America. As the second poorest nation within the region, Honduras lacks much of the funding necessary to improve its water and sanitation services. This unfortunate fact is also an indication that wastewater sanitation is not well funded throughout Central America. Relief of poverty is a main objective for the Honduran government and non-governmental organizations (NGOs) seeking to provide aid to the nation. One of the fundamental principals in relieving that poverty is the provision of adequate water and sanitation services to the citizens of Honduras. However, ~~a dichotomy is created by attempting~~it is challenging to fund a poverty reduction project within an impoverished nation.

Scarcity of resources has required that treatment technologies within the country consist of simple infrastructure systems which provide treatment through sustainable practices. Unfortunately a large portion of the systems which exist within Honduras have not been properly designed or maintained. Collaborative efforts between international consultants and the Honduran agencies which make up its water sector are focused on finding ways of improving existing infrastructure, disseminating knowledge about sanitation, and identifying appropriate new decentralized technologies for use in wastewater treatment.

One such technology that was utilized within the United States into the 1950s is the Imhoff tank. As explained in Chapter 3, this system provides primary treatment and digestion of settled sludge, making available a basic form of wastewater sanitation with minimal required inputs. Imhoff tanks comprise approximately 40 percent of the nation's wastewater infrastructure; of the tanks found throughout Honduras, most are in various states of disrepair. Construction of new treatment systems throughout the country will phase out the use of Imhoff tanks. However, efforts must also be made to improve the condition and treatment efficacy for Imhoff tank systems which possess remaining service life.

The municipal township of Las Vegas currently utilizes Imhoff tanks for its treatment of wastewater. Topics concerning wastewater within Las Vegas have been researched by several teams of graduate students attempting to assess water pollution sources within the region, water quality, wastewater generation patterns, enhancement options for the Imhoff tanks, and potential ways in which the system may be expanded to include other forms of treatment.

1.1 Background of Recent Studies Performed within the Las Vegas Region

In the academic year of 2005 – 2006, Tia Trate and Mira Chokshi, Master of Engineering graduate students within the Department of Civil and Environmental Engineering (CEE) at Massachusetts Institute of Technology (MIT) focused their research on the water quality of Lake Yojoa, Honduras' largest natural freshwater lake ~~.~~ (Trate, 2006; Chokshi, 2006). Their examination studied the impact upon the lake of wastewater effluent discharged into it from tributary streams. The students' efforts involved water quality sampling, identifying point and non-point pollution sources, and locating stakeholders within the region that had a vested interest in keeping the lake unpolluted. One of the largest identified polluters of the lake was the community of Las Vegas, located approximately 8 km upstream from Lake Yojoa.

Comment [EA5]: You need to formally cite the study. Similar comment may apply elsewhere.

Also during academic year 2005 – 2006 a graduate student from the University of Texas-Austin, Aridaí Herrera, was conducting research within the township of Las Vegas (Herrera, 2006).~~.~~ Herrera's focus was on assessing the current wastewater treatment needs of the community and investigating the status of the existing infrastructure, the Imhoff tanks. Herrera, a native to Honduras, brought an intimate knowledge of the country and was aware of the deficiencies in sanitation services. Herrera's efforts detailed the city of Las Vegas' need for improved wastewater treatment and the development of proper maintenance protocols for the existing Imhoff tank system. Herrera shared this information with MIT's CEE Dept. in an attempt to encourage graduate study into ways of improving the existing infrastructure performance for the community of Las Vegas.

The CEE Dept. at MIT had two graduate students in 2008, Matthew Hodge and Anne Mikelonis, who, accompanied by Herrera, were provided the opportunity to research and build upon the initial efforts made within the community of Las Vegas. These students coordinated with local municipal officials to conduct water quality testing of influent and effluent samples, obtain flow measurements, pilot test the use of chemically enhanced primary treatment[1], and begin the preliminary phase of recommending processes which could expand treatment within Las Vegas.

In January 2009 a group of three MIT graduate students within the CEE Dept. again went to Honduras; Mahua Bhattacharya, Lisa Kullen, and Robert McLean were provided the opportunity to perform individual topic research and conduct a collaborative field survey investigating the status of wastewater treatment within a larger portion of Honduras. Through this work, the students were provided an opportunity to learn about wastewater and sanitation services within a developing nation, attempting to identify areas in need of improvement and recommending strategies to achieve these goals. The work of the 2009 team was undoubtedly bolstered by the efforts of previous research endeavors which provided insight into possible alternatives for improving wastewater infrastructure within Honduras.

Comment [EA6]: Reference to group report.

[1] Defined in Chapter 4

1.2 Scope of this Work

This work seeks to expand upon the body of knowledge pertinent to enhancing the treatment efficacy of Imhoff tanks within Honduras. The identification of options for improving Imhoff tank performance is critical to communities such as Las Vegas which currently use this type of system for wastewater treatment. Building upon the previous creative efforts of Herrera, Mikelonis, and Hodge, this work seeks to: further assess the state of the Las Vegas Imhoff tanks, make recommendations with regard to chemically enhanced primary treatment as a solution to increase efficacy, introduce modifications to the current infrastructure in order to improve plant performance, and recommend preliminary designs for secondary treatment processes appropriate for use on the decentralized scale of Las Vegas.

The next three chapters of this work provide: a general examination of wastewater treatment within Honduras (Chapter 2), definition for what an Imhoff tank is and explain how the system works (Chapter 3), and information which elucidates the chemically enhanced primary treatment process (Chapter 4). These chapters are meant to provide background and define information useful to understanding the remainder of this work.

The remaining chapters examine topics on a site specific level pertinent to the Imhoff tanks of Las Vegas. Chapter 5 presents a detailed assessment of the Las Vegas Imhoff tanks and future planned development by the municipality. Contained within Chapter 6 are the methodologies and results for bench scale tests of chemically enhanced primary treatment using ferric chloride and aluminum sulfate as coagulants. Chapter 7 examines conceptual designs for improving the efficacy of the existing infrastructure and considers the possibility of adding secondary treatment systems. Finally, recommendations and conclusions from this work are contained within Chapter 8[2].

[2] With the understanding that this work may ~~find usefulness~~ be useful within various countries, attempts have been made to utilize *Systèm International* (SI) units wherever possible. U~~, u~~nfortunately this is not the case for certain empirically derived design calculations which utilize US customary units. To alleviate potential confusion a conversion table of units has been provided within Appendix VI.

Chapter 2: Survey of Water and Sanitation Within Honduras

This chapter presents excerpts from the report created through the collaborative efforts of the Masters of Engineering students from Massachusetts Institute of Technology who researched within Honduras during the academic year of 2008-2009. During this time, the team met with leaders of agencies within the water and sanitation sector of Honduras to coordinate the study of ten representative facilities located across the country. The study culminated ~~into a deliverable~~ in a report ~~passed along~~ provided to the Honduran agencies within this sector in addition to the CEE Dept. at MIT. The report presents a summary of the investigation findings including: a background on the Honduran water sector, a description of each facility visited, and trends and recommendations based upon these observations. Within this work, material from the survey has been condensed to contain information which is applicable specifically to wastewater regulation and the agencies involved within the sector, trends observed which are applicable to Imhoff tank systems, and recommendations made by the team which may foster improvement for these systems[3].

Comment [EA7]: Cite by author and date.

2.1 Honduras General – Introduction

The Republic of Honduras is the second largest country in Central America. With a population of 7.7 million people, Honduras covers an area of 112,000 square kilometers, roughly the area of the state of Tennessee. As detailed in Figure 1 the country is bordered by Guatemala and El Salvador to the west, Nicaragua to the southeast, and possesses access to both the Pacific Ocean and Caribbean Sea.

[3] For the complete report please see (Bhattacharya et al, 2009)

Figure 1: Map of Honduras and Neighboring Region

Honduras is a Spanish-speaking nation comprised of 18 departments or political territories, which are further divided into a total of 298 municipalities. The nation is democratic, with universal mandatory voting by all citizens over the age of 18 years (U.S. CIA, 2008). The country's capital of Tegucigalpa is also its largest city; approximately 12 percent of the population resides within Tegucigalpa. Overall the country's population is divided into 43% urban dwellers and 57% rural (WHO, 2001).

As mentioned previously in Chapter 1, Honduras has one of the highest levels of poverty in Central America; sixty-five percent of the population lives on less than two dollars a day (Water for People, 2006). The nominal per capita GDP is $1,635 (FCO, 2008). Literacy rates in the nation were reported at 80% on the 2001 census. The median age in the country is 20 years with life expectancy at birth of 69 years (U.S. CIA, 2008).

2.2 Water and Sanitation within Honduras

Poverty reduction, through the provision of essential services such as adequate water and sanitation, has been a primary development initiative in Honduras (Mikelonis, 2008). However, poverty levels have also been a factor in the historical lack of sewerage fee collection, with current service providers facing cultural and economic challenges in levying rates or tariffs on sanitation services. As a result, sanitation is largely inadequate throughout the country; in urban areas, 41% of all residences lacked sanitation services as of 2001. Rural sanitation connection rates were reportedly below 20% (WHO, 2001). Similar investigative work performed by the organization Water for People five years later (Table 1) found improvement in these number but services are still lacking across both urban and rural populations.

Table 1: Sanitation Coverage within Honduras

Sanitation Coverage Honduras 2001 Groups of Population	2001 Population	Population with sewerage service	Population with latrines	Total population served	Coverage %
Rural	3,113,304	N/A	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

Source: Water for People - Honduras 2006

Inadequate sanitation holds severe consequences for the population of Honduras with regards to water-related diseases. With a high infant mortality rate of 42 out of 1000 births, the leading cause of infant mortality is reported as intestinal infectious diseases. For children under the age of ~~5~~five, the second leading cause of death is diarrheal diseases. Water-related diseases include waterborne (e.g. bacterial diarrhea, hepatitis A, typhoid fever) as well as vector-borne illnesses (e.g. malaria and dengue fever) whose transmission is exacerbated by unsanitary conditions. Cholera, a waterborne illness previously eradicated from Honduras, re-emerged with an outbreak in 2001. Proper sanitation is critical to raising the standards of health in the nation (WHO, 2001).

Comment [EA8]: Usual to spell out numbers smaller than ten (some say 100)

2.3 Regulatory Framework of Wastewater Sector within Honduras

Multiple agencies attempt to work across several layers of government in the oversight, regulation, administration, and promotion of water and sanitation provision within Honduras. Unfortunately effective communication between these varying agencies has been lacking in some instances. These lapses in communication have led to difficulties in changing regulation, obtaining necessary permitting for new projects, and declines in operation and maintenance of existing systems. Reorganization and delegation of responsibilities for these agencies is ongoing in response to several factors including pressures from funding agencies and recent statutory reform in the water sector. This restructuring aims to improve the efficiencies of communication between agencies and to improve the water and sanitation infrastructure of the country. A summary of the agencies with description is provided within this section and in Table 2.

ERSAPS

Compliance and enforcement in the sanitation sector is handled by the Regulator of Potable Water and Sanitation Sector, or ERSAPS (Herrera, 2006). This agency is charged with the task of acting as a regulatory overseer for municipalities of all sizes with regard to water and sanitation. The agency offers this information in an attempt to disseminate knowledge about the laws governing water and sanitation to local levels. Examples of this are the technical manuals provided through their website which include guidelines for meeting regulatory requirements (Mikelonis, 2008).

SANAA

Historically, the oversight responsibility for sanitation in Honduras fell to the National Autonomous Water and Sanitation Service (SANAA), which was charged with all aspects of sanitation including planning and construction as well as operation of facilities. A legislative change in 1990 created the Law of Municipalities, granting Honduras' 298 municipalities the independent responsibility for sanitation services within their borders. A subsequent legislative change in 2003 created the Framework Law for the Water and Sanitation Sector of Honduras. This new law detailed the implementation of the restructuring called for in the Law of the Municipalities.

This transference of responsibility from SANAA to the municipalities was set to be completed in 2008. Progress has been slow due to confusion over the change in jurisdiction, and SANAA's position that some municipalities are not ready to manage these responsibilities. SANAA still operates roughly half of all urban water sanitation services, despite the mandate to terminate this function by 2008; the remainder of these services is provided by a combination of municipalities and private utility ventures. In the current configuration, SANAA's role is as technical secretary to CONASA, described below (Water for People, 2006).

CONASA

The agency of CONASA was created by the Honduran government to assist in implementing the changes mandated by the Law of the Municipalities, as well as the UN Millennium Development Goals and the Poverty Reduction Goals set by the national government. As specified in the Framework Law for the Water and Sanitation Sector of Honduras of 2003, the National Water and Sanitation Council (CONASA) was created to set policy for the sector. CONASA seeks to expand sanitation coverage to 95% by the year 2015 (WHO, 2001).

SERNA and CESCCO

Approvals and permitting for wastewater treatment systems are mainly carried out by SERNA, the Department of Natural Resources and the Environment. The agency is specifically involved in the formulation and evaluation of policies pertaining to water resources, renewable energy sources, geothermal and hydropower, and mining. CESCCO, the Center for the Study and Control of Contaminants, is the technical research arm of SERNA. Its responsibilities include the assessment of pollutant impact on human health and ecosystems, providing laboratory analysis assistance and services to communities, as well as monitoring air pollution in major urban centers (SERNA, 2009).

FHIS

Funding for many water sanitation projects is channeled through the Honduran Social Investment Fund (FHIS), an agency designed to mitigate the economic effect of governmental restructuring on local communities. This agency selects priority projects and transfers funds to municipalities to support those projects with funding from both the Honduran government and international aid agencies. The capital funding provided by FHIS is critical for the implementation of a large portion of Honduras' wastewater facility projects (Water for People, 2006).

RAS-HON

The Honduran Network of Water and Sanitation (RAS-HON) facilitates the efforts of the various entities in the water sanitation sector. This non-governmental organization (NGO) consists of a group of advising environmental engineers and others with technical expertise in the field of sanitation who work with the various agencies listed above to provide technical support and exchange of ideas within this sector.

Juntas

The provision of services in rural areas falls almost exclusively to the Water Boards or the Juntas (Water for People, 2006). Many of these Juntas are organized into a national association, the Honduran Association of Water Boards, which lobbies for the interests of the rural water boards and allows for pooling of technical knowledge (RAS-HON, 2008).

Table 2: Summary of Established Agency Roles within Water and Sanitation Sector

Agency Name	Established Agency Role
ERSAPS	Compliance and enformcement in the saniation sector
SANAA	Releasing authority as urban service provider, becoming technical secretary to CONASA
CONASA	Establishment of policy
FHIS	Channels national and international funds for infrastructure projects
RAS-HON	NGO allos for exchange of ideas and technical support
Juntas	Regional water boards charged with providing rural santation services
SERNA	Approvals and permitting for water resources projects
CESCCO	Technical branch of SERNA providing research and laboratory services

2.4 Identified Trends

Facilities were found to range greatly on a number of measures such as adequacy of design, financial budgeting, and operation and maintenance. Some facilities observed routine water quality sampling and maintenance protocols while others were found to be less maintained or completely abandoned. The trends discussed below are general in nature and do not necessarily apply to all facilities visited. While the survey presents an informative glimpse into typical wastewater management systems, it may not be fully representative of the wider state of wastewater treatment throughout the country.

2.4.1 Design Trends

Comment [EA9]: If you are going to describe problems with systems other than the LV Imhoff tanks, you should probably provide a list of the facilities (either a table or in the text).

Two main issues were observed with respect to design of wastewater treatment facilities. These fell under the categories of design oversight and the implementation of inappropriate technologies.

Examples of design oversights were observed while surveying the Imhoff tank system of Las Vegas. While surveying this system, it was noted that flows greater than originally designed for were being sent to the system. This suggests that the original design was undersized or did not account for increases in water consumption, or alternately that planned expansion was never implemented.

In addition to design oversights, the use of inappropriate technology is a prominent design concern as demonstrated by the activated sludge package plant systems of Amarateca. High energy inputs associated with aeration demands led to excessive operational costs. It was indicated that, in the process of selecting the systems, the community was not made aware of these substantial ongoing expenses or their eventual need to pay for these. According to SANAA, a similar problem was encountered at an activated sludge plant in Tegucigalpa. Due to extremely high operational costs, the plant is routinely shut down during intervals of peak energy demand. This shows that suitability of technologies is not necessarily assessed prior to implementation. The use of inappropriate technology has serious consequences which can often lead to system abandonment, a clear sign of system failure.

2.4.2 Operation and Maintenance Trends

Several important patterns of operation and maintenance schemes were identified over the course the study. Proper operation and maintenance plans are crucial to ongoing system performance and broadly fall under the categories of general maintenance, water quality monitoring, and sludge management.

2.4.2.1 General Maintenance

General maintenance activities include routine tasks such as surface scum removal, cleaning of bar screens, clearing flow obstructions, and grounds keeping. Overall 6 out of 10 facilities visited were maintained to some degree and appeared to be in acceptable operating condition. The extent of general maintenance conducted varied from site to site. Some systems which existed within larger population centers were found to be maintained to a greater degree than others located within less populous locations such as Las Vegas.

2.4.2.2 Water Quality and Monitoring

Flow monitoring practices were reported at several locations. However, at other facilities flow monitoring devices were found to exist but system operators were not familiar with how they should be used. One clear example within the group survey report accounts a plant manager who was aware that a Parshall flume existed at the facility but was unaware of its purpose, measuring flow!

Similar to the general maintenance practices observed, routine water quality monitoring was found to occur within the larger population centers. Smaller decentralized systems were found to have received proper maintenance and water quality monitoring while under the supervision and care of SANAA. However, it was revealed that when responsibility for these smaller systems was handed over to the Juntas such practices were seldom perpetuated.

2.4.2.3 Sludge Management

Of the facilities visited, a number (mainly waste stabilization ponds) were recently brought into operation and have not yet needed to carry out desludging. Some facilities have been monitoring sludge depth and are reportedly in the process of developing a sludge management plan. Among the systems that were found to have been desludged, all were reportedly to have been desludged on a non-routine basis. None of the facilities surveyed has been successful in implementing or marketing sludge for beneficial reuse.

2.4.3 Trends in Community Issues

Both social and economic community involvement can affect plant performance. Two general areas of concern were observed relating to service connections and system financing. Of the facilities surveyed, one received extremely low flows with only 15% of the community connected to the sanitary sewer system. This low connection rate was attributed to high connection fees. In other locations, illegal hook-ups from storm drains and industrial wastewater sources were of concern. Several of these systems reported high flows during rainfall events due to illegal storm sewer connections.

For others, financial hurdles led to system neglect and eventual abandonment. The most technologically advance Imhoff tank observed, the facility at Villa Linda Miller, was reportedly well maintained and met effluent discharge requirements while it was managed by SANAA. Upon handover to the Junta, funds were not allocated for the system's ongoing maintenance. Subsequently it fell into disrepair due to neglect. Similarly, at Amarateca (a packaged activated sludge plant), when responsibility for the treatment plants was ceded to the community, the Junta was unable to fund operational expenses and consequently the facilities are being replaced (with funding from outside sources) at a near total loss of initial capital invested.

The situations at Villa Linda Miller and Amarateca suggest a disconnection between the communities and their wastewater management systems. In both cases, the treatment works were funded, installed, and initially operated by external agencies with limited community involvement. In the long term, when responsibilities were passed over to the Juntas, community leaders were not prepared to handle the maintenance and financial burdens of these systems. Ultimately, this led to disrepair or abandonment requiring renewed capital investment; clearly this is an unsustainable pattern of resource allocation.

2.5 Recommendations from Survey Report

The purpose of this section is to propose recommendations for addressing the identified problematic trends from the study. As previously outlined, the trends pertinent to design, operation and maintenance, and community issues have been shown to hinder the performance of wastewater treatment within Honduras. Addressing these concerns has the potential to improve current infrastructure performance and to ensure adequate development of new systems.

Many of the issues encountered in this survey pertained to technical aspects which could be precluded by the active involvement of overseeing agencies such as SANAA. Detailed technical considerations should be included in the design approval process to ensure that systems are technically sound. These could include examining the appropriateness of facility site location, sizing, and technology employed. The inclusion of performance clauses within consulting or vendor contracts will also act to guarantee appropriateness of technologies by creating a system of accountability, ensuring consistent performance. In addition vendors and consultants should provide proper operations and maintenance procedural manuals to be kept onsite for facility operator reference at any new facility.

Operation and maintenance issues could be mitigated through appropriate regulatory and community involvement. The enforcement of meeting water quality monitoring goals will require that proper operation and maintenance procedures are followed.

The involvement of regulatory agencies such as SANAA or SERNA in mandating proper water quality monitoring and reporting could serve to enforce effluent compliance. Where proper water quality monitoring protocols have not been developed such agencies could provide guidance in creating procedures to achieve regulatory wastewater standards. A system of periodic reporting to regulatory agencies could act to sustain plant performance and identify areas of concern on a regional scale. Implementation of a discharging permit regime could establish penalties for non-compliance with regulatory requirements.

Successful management of wastewater systems requires adequate involvement by the communities for which they serve. This could be established through a number of ways. During the selection and approval phase for designs active participation and feedback from community leaders could ensure involvement and identify critical issues such as potential odor problems or lack of maintenance funds for certain types of systems. Additionally, this early involvement could develop a community sense of ownership for its wastewater management system. This sense of ownership preemptively tackles future issues such as lack of ongoing maintenance funding. This is particularly important at this critical phase of water sector reform to decentralize management responsibilities within Honduras.

A number of external measures can also be taken to improve the upkeep of wastewater treatment facilities. The annuity generated from an escrow account established at the time of the project capital investment could help ensure funds for ongoing operation and maintenance activities. The involved presence of a circuit rider could be beneficial for information dissemination, helping different facilities resolve their issues based on lessons learned elsewhere. This would prove an invaluable resource in stopping the perpetuation of mistakes which ultimately leave ramifications of their consequences across the nation as a whole.

Comment [EA10]: Nice summary. My only major comment for this chapter is a summary table or text listing the 10 plants.

Chapter 3: Examination of Imhoff Tank Systems

Imhoff tanks are a primary form of wastewater treatment with a long history of use. The technology is applicable for situations in developing countries and communities where decentralized wastewater treatment is desirable. This chapter seeks to explain what an Imhoff tank is and how the process works, the basic design guidelines for Imhoff tanks, maintenance requirements for these systems, and an examination of the advantages and limitations of such systems.

3.1 Introduction to Imhoff Tanks:

The Imhoff tank is a primary treatment system that utilizes the force of gravity to seperate solids from wastewater, a process known as primary sedimentation. These solids are then degraded under anaerobic digestion within a lower chamber of the tank prior to sludge disposal. Imhoff tanks are often characterized as two-storey tanks that provide for sedimentation processes to occur in the upper storey and anaerobic digestion of settled particles in the lower storey (Crites, 1998). Originally designed and patented by Dr. Karl Imhoff of Germany in 1906, this system overcame certain difficulties associated with septic systems (Metcalf, 1935). Septic systems of the time would shock load receiving water bodies when effluent heavily loaded with solids was purged as they became full (Babbitt, 1922). The Imhoff tank offered a solution by separating these two processes and allowing sludge to develop a higher quality with removal to controlled areas. Properly designed Imhoff tanks are capable of achieving removals of 50 – 70% suspended solids and 30 – 50% B.O.D. (Barnes, 1976). The system is one example of technology implemented at wastewater treatment works within the United States into the 1940s. At present they are still used in developing parts of the world as a treatment technology that requires minimal maintenance and no energy inputs other than hydraulic gradients.

3.2 Design Guidelines for an Imhoff Tank:

The design of an Imhoff tank is controlled to a large extent by the same factors which govern all primary sedimentation processes; these are the overflow rate, detention time, and horizontal velocity (Lee, 2007). The surface overflow rate is defined as the ratio of wastewater flow to surface area of tank; detention time, the ratio of tank volume to wastewater flow; and horizontal velocity as the ratio of wastewater flow to the product of width and depth, see Appendix I for examples of calculations. Table 3 provides a listing of typical design parameters for these and other factors for Imhoff tanks. Controlling these parameters ensures optimum conditions for the sedimentation and sludge digestion processes to occur within. Typical plan view and cross section schematics of main Imhoff tank components are provided in Figures 2 and 3.

Table 3: Typical Design Criteria for Imhoff Tanks, Source: (Crites, 1998)

Typical design criteria for Imhoff Tanks			
		Value	
Design Parameter	Unit	Range	Typical
Settling Compartment			
Overflow rate peak hour	$m^3/m^2 \cdot d$	25 to 40	33
Detention time	hrs	2 to 4	3
Length to width ratio		2:1 - 5:1	3 to 1
Slope of settling compartment	ratio	1.25:1 - 1.75:1	1.5 to 1
Slot Opening	mm	150 to 300	250
Slot overhang	mm	150 to 300	250
Scum baffle			
Below surface	mm	250 to 400	300
Above surface	mm	300	300
Freeboard	mm	450 to 600	600
Gas Vent			
Area (percent of total area)	%	15 to 30	20
Width of gas vent opening	mm	450 to 760	600
Sludge Digestion Section			
Storage capacity	month	4 to 8	6
Volume	m^3/capita	0.06 to 0.1	0.07
Sludge withdrawal pipe	mm	200 to 300	250
Depth below slot to top of sludge	m	0.3 to 1	0.6
Total water depth (surface to tank bottom)	m	7 to 9.5	9

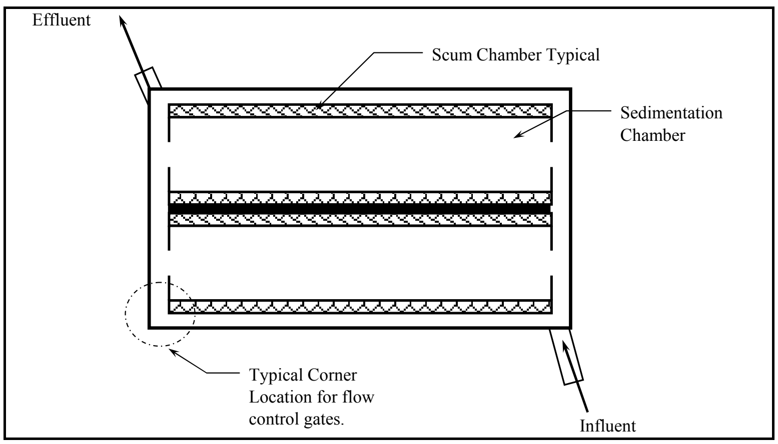

Figure 2: Typical Plan View of Parallel Imhoff Tank System

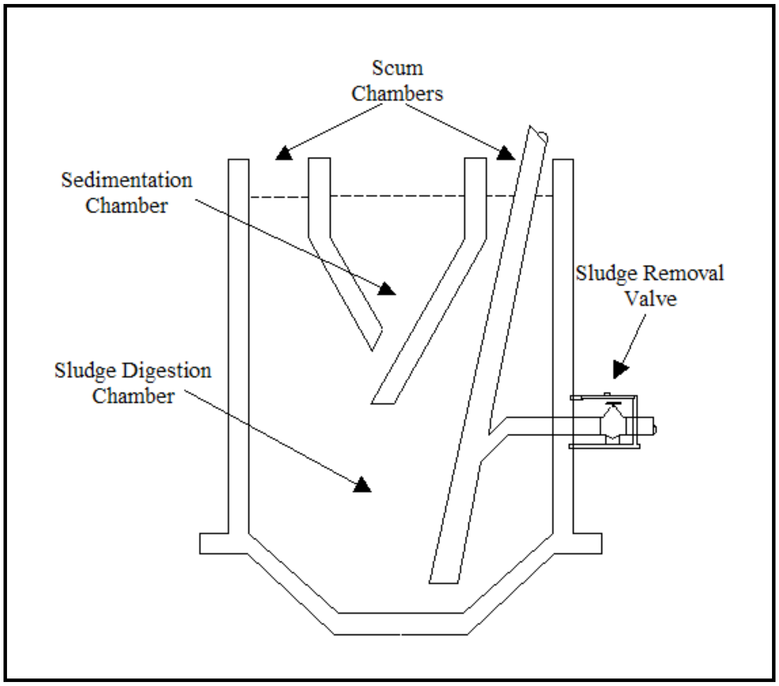

Figure 3: Typical Cross Section of Imhoff Tank (Mikelonis, 2008)

Properly designed, Imhoff tanks should exhibit plug flow through the system. This plug flow ensures that velocity is uniform everywhere within the cross section of the tank. The use of baffles at the inlet of each sedimentation chamber will increase the tendency of plug flow and should always be accounted for in design. This in turn allows for maximum settling time for all particles and efficient use of tank volume through eliminating the problems associated with short circuiting.

Achieving a plug flow regime allows for particles with a settling velocity greater than a certain threshold to be removed predictably. The settling velocity of particles suspended within laminar flow is governed by Stokes Law and is dependent upon particle size, specific gravity, and fluid viscosity (Crites, 1998). For all sedimentation tanks this settling velocity can be back calculated to achieve an approximate minimum removed particle size. Theoretically particles below this size will pass through the system untreated. Alternatively the settling velocity can be utilized to develop a characteristic design length for the tank. Particles that successfully settle to the bottom of the tank are termed primary sludge.

Differing from standard sedimentation tanks, Imhoff tanks require an additional separate compartment, the sludge digestion chamber, in which the primary sludge is collected and digested. It is recommended that the volume of the lower compartment be designed to accommodate sludge accumulation for a 6 month time period. This time frame will allow for sufficient anaerobic digestion of the sludge materials thereby improving their quality.

Specific to Imhoff tanks, flow gates should be in place to regulate the direction of flow through the system and normalize the distribution of solids within the digestion chamber (Herrera, 2006). This practice ensures proper sludge digestion throughout the chamber by ensuring that the sludge blanket is of uniform thickness. The digestion process produces methane and carbon dioxide gases which will need to be vented out from the lower compartment. Proper Imhoff tank design allows for these gases to vent through isolated parallel scum chambers from the sludge digestion chamber. In this way the gases do not disturb the plug flow regime of the sedimentation chamber.

Aside from the main components previously described, a properly designed Imhoff tank should include typical pre-treatment systems such as a grit-chamber and bar screens. A grit-chamber will remove the larger readily settling particles from suspension before they reach the main tank, thereby freeing up more digestion space within the sludge digestion hopper. Bar screens are recommended for the preservation of plug flow by eliminating large floating objects which may become lodged within the system changing tank hydraulics. In addition a flow bypass system should be included to isolate the system for purposes of maintenance.

3.3 Maintenance Requirements for Imhoff Tanks:

Maintenance requirements for Imhoff tanks are minor and no skilled labor is required. Care should be used in developing a sense of operator ownership for smaller systems such as the Imhoff tank. An understanding that the system serves a fundamental purpose to both those who have used the water and those who are downstream from its discharge will go along way in getting a community to take ownership and interest in its care. Fundamental to any good maintenance program is the record keeping of when an action was last performed and when it is slated to be performed again. Recordkeeping is emphasized because it provides for a structure to base efforts upon.

Records should be kept for the following activities as these are performed regularly. Hydraulic flow measurements should be performed daily and documented to watch for changes in wastewater production. Proper inspection that the tank is not clogged should be performed routinely. All influent, effluent, and sedimentation flow paths must be free from obstructions for efficient treatment of wastewater. Grit-chambers and bar screens should be inspected daily and cleaned out weekly or daily respectively[4]. Scum chambers should be cleaned once a week to promote proper venting of digestion gases (Mikelonis, 2008). Sludge levels within the digestion chamber should be monitored for uneven distributions and depth of sludge. The direction of flow is recommended to be switched using the flow gate controls once every two weeks to promote even distribution (Herrera, 2006). The sludge digestion chamber should maintain a minimum clearance of 0.6 meters freefall between top of sludge blanket and entrance from above. Table 4 summarizes the regular maintenance intervals for a typical Imhoff tank system.

---

[4] Depending upon loading conditions inherent to the wastewater these items may need additional cleaning at more frequent intervals.

Table 4: Typical Maintenance Intervals for Imhoff Tanks

Typical Imhoff Tank System Maintenance Schedule		
Process	Task	Interval
Grit Chamber	Check for flow obstructions Clean out settled debris	Daily Weekly
Bar Screens	Clean obstructions to flow	Daily
Scum Chambers	Scrape and Remove Scum from surface	Weekly
Digestion and Sedimentation Chamber	Check for flow obstructions Inspect sludge level Scrape sloping wall free of residue Switch direction of sediment deposit Empty the sludge digestion chamber	Daily Weekly Weekly Bi-Weekly Semi-Annually
Site as whole	Clear away any litter debris and plants	As needed

3.4 Advantages and Limitations of Imhoff Systems:

Existing and new Imhoff tanks when designed properly and in the right setting provide adequate primary treatment of wastewater. They posses several key advantages allowing for them to be considered a sustainable treatment option. The most obvious advantage to the Imhoff tank is the fact that no moving parts, motors, or electricity are required for its operation. The entire operation is controlled by the hydraulic gradient across the treatment works. For this reason the operating costs for these systems are extremely low. In addition the limited maintenance that is required to operate an Imhoff tank allow for very low overhead. Often these systems can be operated by one individual who works part time on maintaining the plant and simple repairs would consist of replacing valves, flow gates, and bar screens.

They are primarily applicable as decentralized wastewater treatment options. Decentralization allows for treated wastewaters to remain close to their point of use minimizing the transportation of water offsite. This in turn minimizes costs and keeps the local environment from experiencing water scarcity. Provided that adequate treatment is being realized through the Imhoff tank this local water can contribute to receiving waters. In addition the nutrients from properly digested sludge have potential benefit in the community as an agricultural amendment to soil.

The largest limitation of an Imhoff tank system is that it is only a primary form of treatment. When used alone it does not meet the newer requirements for effluent guidelines. Removal of pathogens is not accomplished except for those which are trapped in settling particles. However, existing Imhoff tanks lend themselves nicely to retrofitting and adaptation. With a small footprint these systems can be added to in stages if adjacent land is available. In this way an existing Imhoff tank system can become the primary treatment works of a multi-stage treatment process that allows for secondary systems downstream. This allows for communities to make capital improvements to infrastructure incrementally. In addition, retrofitting in this way can extend plant life and allow for options such as discounting and other factors to limit the cost of such systems.

3.5 Summary:

Although the technology of the Imhoff tank dates back to over one-hundred years it does provide a sufficient form of primary treatment within a simple framework. The ability to run the system in a sustainable fashion makes it a model to base improved designs upon and encourages implementing enhancements to existing systems. Low operation and maintenance costs are associated with these systems. The routine maintenance that is required for an Imhoff tank is simple and does not demand skilled labor. Utilizing this system in tandem as the primary treatment works for a multi-stage process plant allows for newer technologies to be coupled to old; saving on capital expenditures and lowering the demand for resources in developing better wastewater treatment. Enhancements or coupled systems to Imhoff tanks will be addressed throughout the remainder of this work.

Chapter 4: Chemically Enhanced Primary Treatment

The name, chemically enhanced primary treatment (CEPT), provides insight into the goal of this technology: that is, to increase the efficiencies of primary wastewater treatment through a physical chemical process. This chapter seeks to introduce the subject of CEPT, address feasibility for use with typical Imhoff tanks, and discuss the concerns and limitations of this technology.

4.1 Introduction to CEPT:

The practice of utilizing chemical coagulants in the treatment of water containing suspended solid material is not a new technology. It has been utilized for over a century in the removal of suspended solids in applications ranging from potable water supply to industrial manufacturing process effluent water. Some of these processes seek the formation of precipitates which readily settle out from solution once formed (Crites, 1998). Others, as examined in this work, seek to increase the size of suspended particles and hence their removal rate within a primary treatment system.

Colloidal and suspended solid particles within wastewater often possess negatively charged anionic materials. For this reason they will tend to repel one another and remain in suspension if small enough, overcoming both the effects of gravity and the attractive Van-der-wall forces. Coagulation is the destabilization of the charge that exists on colloidal and suspended particles by a coagulant (Kawamura, 2000). These coagulants are the chemical additions of the CEPT process. The most commonly utilized coagulants are the metal salt coagulants: aluminum sulfate (alum), ferric chloride, and ferric sulfate (Kawamura, 2000)[5].

With the destabilization of particle charge, the suspended materials are allowed to come into close enough contact with one another to cling and form larger particulates or floc. This is the flocculation process, in which particles are allowed to collide with one another through motion and adhere into larger masses. Fortunately, the relatively dilute suspended particle concentrations that exist in most municipal wastewater allow for flocculation to occur naturally during the settling operation (Crites, 1998).

Chemically enhanced primary treatment (CEPT) seeks to catalyze this natural process through the introduction of coagulants such as metal salts which release positively charged metal ions that speed up the process of coalescence or flocculation. This increases the size of particles that exist in wastewater as the positively charged metal cations attract the negatively charged colloidal suspended solids (Lee, 2007). These enlargements in size contribute to an increase in settling velocities and result in greater removal efficiencies for the facility as exemplified in Table 5.

[5] Synthetic polymers are also available but will not be discussed here, see Kawamura, 2000 for more details on synthetic polymer coagulants.

Table 5: Typical Removal Efficiencies Obtained Using Various Treatment Methods

Comparison of Removal Efficiencies (National Research Council, 1992)		
Type of System	TSS (%)	BOD (%)
Conventional Primary Treatment	55	35
Conventional Primary + Biological Secondary Treatment	91	85
CEPT	85	57

4.2 Feasibility of CEPT use with Imhoff Tanks:

CEPT may present a solution to the problems associated with realizing flows greater than originally anticipated in design of a primary treatment system. These increased flows lead to a loss of detention time, decreasing the time allowed for particles to settle out of the sewage. These losses due to increased flows are currently a problem in areas experiencing increases in wastewater production without the expansion of new treatment installations. Developing countries are presently facing this situation as they grow in dependence on a lifestyle that produces increased wastewater without proportional capital improvements. This is the case across much of Central America including Honduras.

Increases in removal efficiency are one solution for situations where overloaded plants are failing to meet the regulatory requirements for effluent quality, especially for situations where plans do not exist to expand current treatment systems. Implementing CEPT at existing facilities is one possible way in which project life might be extended at modest levels of investment until further capital improvements can be realized.

Understanding what is required for a CEPT process reveals the level of investment needed. When conducting a full feasibility study assessing the options of various treatment technologies it important to consider the following: desired water quality, affordability, practical aspects of implementation, alternative technologies, and cost considerations (Parker, 2001).

Specific to the developing world, attention must be given to the feasibility of obtaining readily available supplies of chemicals locally. This will prevent the exhaustive costs of importation, which could render the technology unsustainable. A caveat to this would be to conduct bench scale testing of native wastewater to identify which chemical coagulant performs the best given the wastewater being treated. A cost benefit analysis can then be conducted to reveal if greater expenditure is appropriate to bring in less of a more effective chemical or if CEPT is a viable option at all. In addition it is important to develop skilled operators in identifying optimum dosing of chemicals so that expense is not wasted in over dosing. A study of approximately 100 treatment facilities within the United States revealed that the expenses required for operating a CEPT system are approximately half of those for typical secondary biological plants, and initial capital expenditures are only one fourth; see Table 6 for averaged cost details (Chagnon, 2004).

Table 6: Cost Comparisons for Applicable Treatment Technologies (Chagnon, 2004)

Treatment Cost Comparisons		
Type of System	Construction Costs* (US$M per m3/s)	O&M Costs** (US$M per year per m3/s)
Primary Treatment (no disinfection)	1.5	0.2
CEPT & Disinfection	1.3	0.5
Primary & Activated Sludge & Disinfection	5.0	1.0

* Construction costs are based on the maximum plant flow capacity
** Operation and Maintenance costs are based on the average yearly flow, approx. 1/2 max

Existing primary treatment infrastructure can easily be retrofitted to allow for CEPT; the main capital components of a CEPT system exist inherently within their design. Further fabrication would be required for installation of a location to properly dose and mix chemicals into the wastewater stream if not already present (Chagnon, 2004). This location is readily accessible at most plants and small ~~apparatuses~~ appurtenances can be created to allow for proper pre-mixing of chemicals and dosage administration. In addition most primary treatment systems are easily convertible to handle the increased sludge production that one would experience using CEPT.

4.3 Concerns and Limitations of CEPT:

~~The~~ This study focuses on ~~concerns examined within this work will center upon~~ the physical requirements of a CEPT system and the limitations that may arise from them. Three separate physical concerns are most prominent in regard to CEPT use: the requirement of skilled operational and maintenance personnel; the cost associated with obtaining, transporting, and dosing chemicals; and the additional loading on systems through increased sludge production.

Of the three areas of physical concern described above two are quantifiable simply through a conservation of mass. These are the cost associated with chemicals and the additional sludge production expected. As mentioned previously, proper dosing requirements should be obtained with site specificity to optimize coagulant usage. The lowest effective dosage to achieve the desired removal efficiency is then selected. This will guarantee a minimum amount of chemical requiring delivery and the least amount of new solids mass input to the system. The expenses associated with skilled operators are not controlled by the quantity of chemical used but instead by the maintenance needs of the system. However, it must be noted that proper maintenance is critical to obtaining effective treatment of wastewater and is therefore invaluable.

Chapter 5: Las Vegas Wastewater Treatment Status

This chapter presents the efforts and findings of field work conducted during the month of January 2009 at the Imhoff tank system of Las Vegas, Honduras. Previous MIT MEng students, Mikelonis (2008) and Hodge (2008) and University of Texas at Austin student Herrera (2006) developed a detailed review of the status of this Imhoff tank system. One of the aims of this work is furthering the body of knowledge for the Las Vegas system. This chapter conveys the assessment work for the following at Las Vegas: physical survey of Las Vegas Imhoff Tank, flow measurements, review of recent maintenance activities performed or description of those lacking, water quality assessment, and community wastewater infrastructure planning.

5.1 Introduction:

The township of Las Vegas is located within the department of Santa Barbara, Honduras approximately 10 kilometers west of Lake Yojoa, 130 kilometers northwest of Tegucigalpa, and 80 kilometers southwest of San Pedro Sula as shown in Figure 4.

Figure 4: Map of Las Vegas, Honduras Region (Google, 2009)

The community of Las Vegas is comprised of approximately 17,000 residents who are located within Las Vegas proper as well as the surrounding communities of El Mochito and San Juan. It has been estimated that approximately 6 people live per residence equating to approximately 2,850 homes within the governance of the municipality. Of these homes approximately 600 are connected to the Imhoff tank of Las Vegas (Bhattacharya et al., 2009). The remaining residences use septic systems or discharge directly into Raices Creek, a tributary to Lake Yojoa (Mikelonis, 2008).

The Imhoff Tanks are located in the southeastern most corner of the watershed for Las Vegas Central and consist of two tanks running in parallel. Designed and built in 1992 by SANAA with funding from FHIS this system provides wastewater treatment for approximately 3,600 people, a figure which accounts for known illegal connections (Mikelonis, 2008). Figure 5 provides a view of the watershed for Las Vegas Central and relative location of the Imhoff tanks.

Service revenues for sanitation within Las Vegas are obtained from a combination of assessing sewerage connection fees to residents, the government organizations FHIS and SANAA, and support funding from American Pacific Mining Corporation (AMPAC) (Hodge, 2008). The later is a large scale mining operation with a Central American office located within El Mochito and hence contributing to the municipality of Las Vegas. It is unknown how the municipality assigns priority of expenditure of these funds.

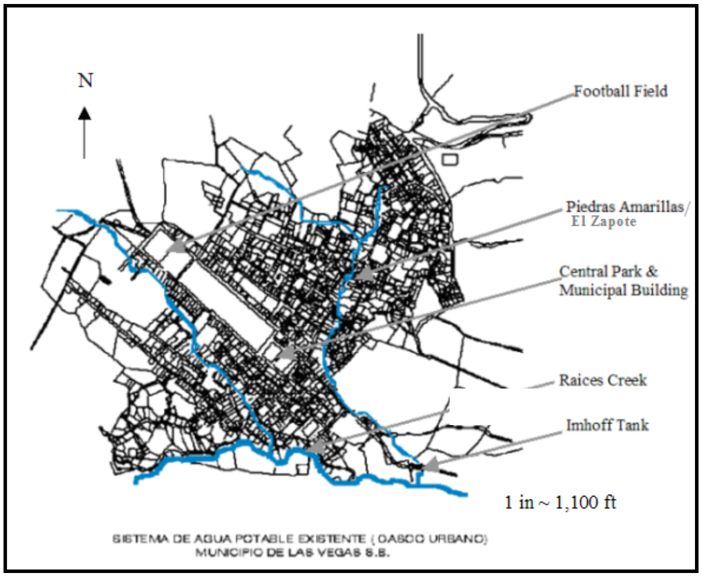

Figure 5: Schematic Representation of Las Vegas Watershed (Mikelonis, 2008)

5.2 Physical Survey of Las Vegas Imhoff Tanks:

The system consists of two tanks running in parallel which receive influent from a single trunk line running beneath the earthen road providing access to the site. This trunk line is exposed for the last 100 meters up to the facility. The treatment facility is located approximately 75 meters south of the nearest residence. The site is not connected to the electrical grid, although power is provided to the nearby residences. The system discharges primary treated effluent into Raices Creek located approximately 50 meters to the east of the tank. As built, the system does not provide any form of pre-treatment or secondary treatment. A grit chamber is not present in the design and a bypass does not exist for the system. Figures 6 and 7 provide plan view and elevation views respectively for the facility.

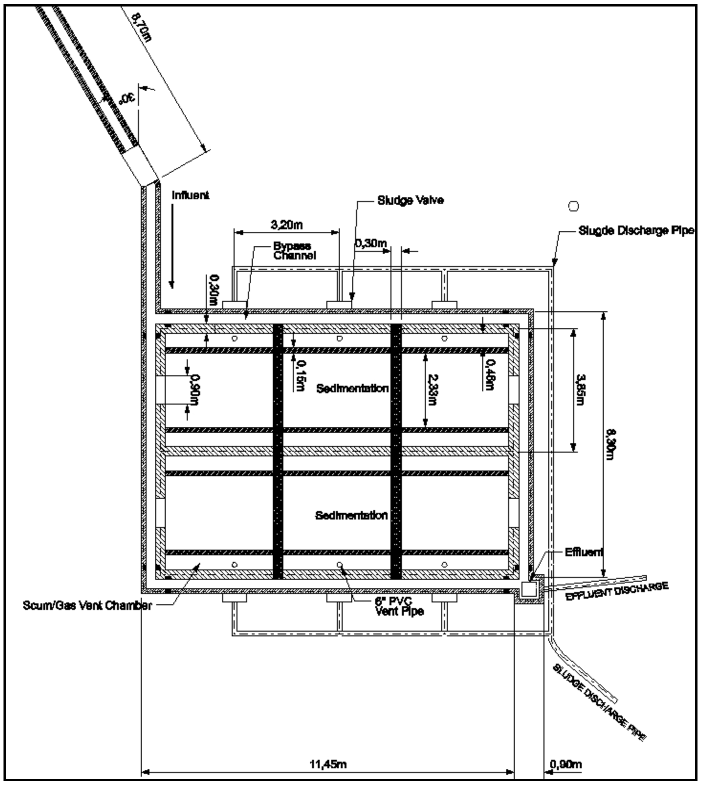

Figure 6: Plan View of Las Vegas Imhoff Tanks (Herrera, 2006)

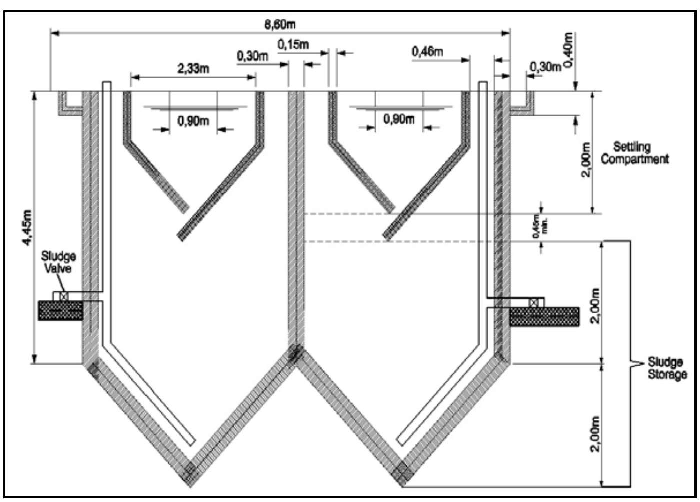

Figure 7: Elevation Cross Section of Las Vegas Imhoff Tanks (Herrera, 2006)

Overall structural integrity is considered sound upon visual inspection. The concrete walls of the structure do not appear to have any cracks nor show any signs of spalling away. It is important to note that only two thirds of this system is above grade. The soundness of structure below grade is unknown. The section of trunk line which is exposed leading up to the facility is in good condition. Repairs had been made to an access box along the trunk line within the last three months of 2008 according to local accounts.

The site is situated with an adjacent hillside to the west and predominately level terrain adjacent to the three other faces. Within the surrounding terrain a gradual southeasterly slope exists, approximately 30 to 1, toward the confluence of Raices Creek and one of its tributary creeks. The access road, between the tanks and Raices Creek, continues southward toward another community down gradient. Overall site availability is approximately 3 hectares[6], with the Imhoff Tanks occupying a small portion of this land space. This provides the opportunity for plant expansion since the land is owned by the municipality. In December 2007 a portion of the land was utilized for sludge drying when the facility was desludged (Mikelonis, 2008).

5.3 Flow Measurements of Las Vegas Imhoff Tanks:

Flows for the system were measured over a 24 hour period spanning January 13th and 14th of 2009[7]. A Global Flow Probe model no. FP101 was utilized to obtain measurements of both velocity and depth at the trunk line access point immediately upstream from the tanks. This device had been calibrated to two other forms of measurement within the labs at MIT and was found to be within +/- 5% error. The influent velocity and water depth within the 0.3 meter diameter inlet channel were recorded. Measurements were taken along the centerline of the channel attempting to place the hub of the impeller at mid-depth; each data collection sequence developed a time averaged velocity and depth over a one minute time period. The measurements provided a peak flow of 1,240 m^3/day and an average flow of 1,060 m^3/day[8]. The latter implies a per capita usage of almost 300 liters/day. A sharp spike in flow was noted around 7 am January 14th. Flows were considered diurnal with a greater flow noted in morning hours, tapering off toward noon, and then rising to a local maximum in the afternoon before the lowest flows overnight as shown in Figure 8. This resulted in a peak surface overflow rate of 52 m/day which is approximately 25% higher than the maximum design recommendation (Section 3.2, Table 1).

These flow rates are dramatically less than those obtained in previous studies. In 2008 a peak flow rate of 4,600 m^3/day (191 m^3/hr) and subsequent surface overflow rate of 193 m/day were estimated, nearly four times design specification (Hodge, 2008). Hodge had qualified this estimation by stating that the measuring device, a timed floating tangerine, would remain within the fastest moving surface waters of the channel. The method utilized by Hodge is common in developing a rough estimate for flows and differences this large would not be attributable to location of measurement. Instead, emphasis is placed on potential changes in water consumption as a driving factor, a subject which both Hodge and Mikelonis addressed during their studies. As

[6] Approximately sixty percent of this is level usable terrain in present condition.
[7] Three collection times were forfeited in the middle of the night (assumed not to govern design) due to safety concerns expressed by members of the team regarding the remoteness of the site and access issues.
[8] Refer to Appendix I for collected data and calculation steps in obtaining these values.

such, the values obtained in the 2008 study will serve as the upper bound for flow estimation to Las Vegas' Imhoff tanks.

~~Similarly,~~ Experco International, a Canadian environmental engineering consulting firm also studied the Las Vegas Imhoff Tank in April of 2003 obtaining a peak flow of approximately 3,400 m^3/day (143 m^3/hr), as shown in Figures 9 and 10 (Herrera, 2006). The values of the Experco study also exceed the design recommendations for an Imhoff tank dimensioned as the one in Las Vegas. Methodology of the Experco study is unknown, although it does support a higher past water usage condition.

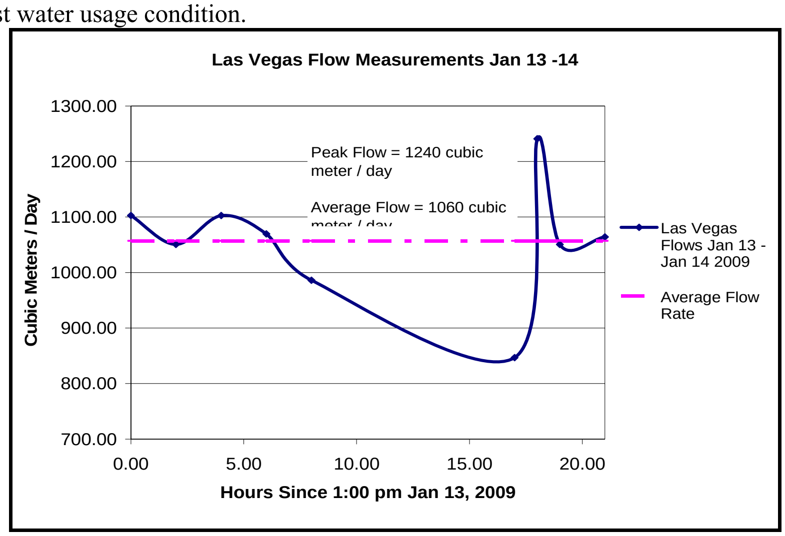

Figure 8: Flow Measurements Obtained January 13th -14th, 2009

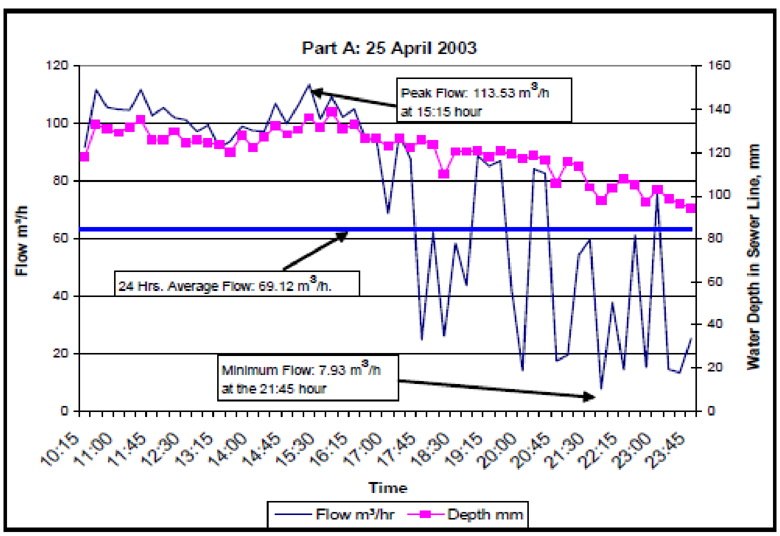

Figure 9: Flow Measurements Experco International Study April 25, 2003 (Herrera, 2006)

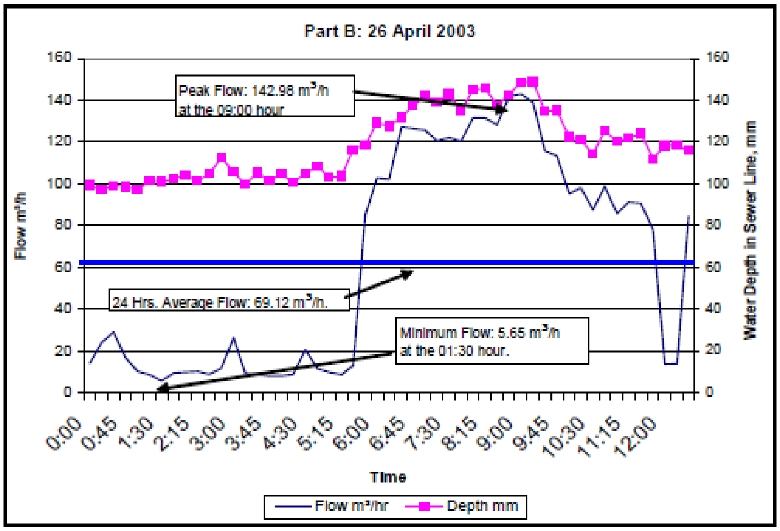

Figure 10: Flow Measurements Experco International Study April 26, 2003 (Herrera, 2006)

As alluded to previously, the discrepancies between flow measurements by all three groups can possibly be accounted for by examining the season of measurement, the methodologies used, and the efforts of the 2008 group to inform the community about the severe over burden on the system. The methodology utilized in the present study measured flow velocity directly and tried to obtain measurement at a location where average velocities within a water column may be observed. Methodologies utilizing floating devices can create subtle over estimations (approximately 30%) in velocities since they reside on the water surface and are transported with the fastest moving source as explained by Hodge. The combined effects of more precise measurement in 2009 and the efforts to promote a decline in water usage by Hodge and Mikelonis in 2008 may account for the larger discrepancy in values. During their work, they had obtained information that flow rates may have been high due to coffee cultivation. One step in the processing of harvested coffee is to wash the beans thoroughly for long durations (Hodge, 2008). The team had observed coffee beans within the scum chambers of the Imhoff Tank in 2008, and made recommendations that these flows not be sent to the sewer. Coffee beans were not observed within the system during January 2009.

5.4 Review of Maintenance Activities:

Section 3.3 outlined a series of activities which should be observed as standard maintenance procedures for Imhoff Tanks. ~~,~~ Table 4 provides a reference summary for these activities. In 2008 time was spent in creating and installing flow control gates and baffles to regulate the flow between tanks (Mikelonis, 2008). During the present period of study the facility did not receive any observed maintenance, and the recommended components from 2008 were found to not exist as exemplified in Figure 11. No information was provided with regard to the whereabouts of these except for the flow control gates. The flow gates were wood planks which as a potential fuel source could have been taken for household fire needs. This was not confirmed. Lack of flow control measures allowed for short circuiting of wastewater to proceed untreated through the facility. It was estimated that short circuiting flow was approximately forty percent of overall flow to the system.

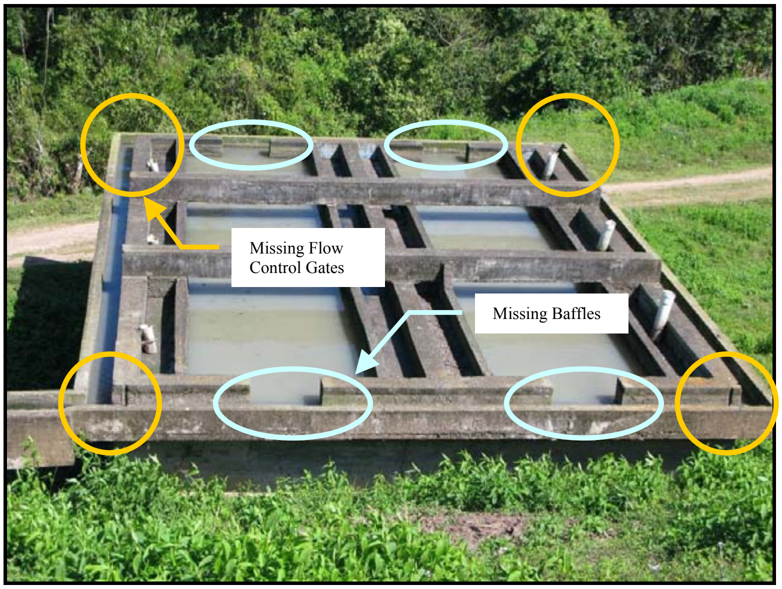

Figure 11: Las Vegas Imhoff Tank, Photo Taken January 2009 Denoting Missing Flow Controls

Aside from the routine maintenance, two of the most critical maintenance needs of Imhoff tanks are the removal of sludge from the digestion chamber and scum from the scum chambers. For the tanks at Las Vegas desludging has only occurred once, December 2007 (Mikelonis, 2008). Since this time no desludging operations have taken place and the plant is beyond the suggested six month interval for sludge removal. Gas venting was observed within the main sedimentation chamber as seen in Figure 12. This is an indication that the sludge digestion chamber may be at capacity and desludging of the system necessary. Scum chambers were found to be caked over with approximately 5 centimeters of hardened scum, Figure 13 reflects this finding. The last known recorded scrapping for these chambers was in January 2008 (Mikelonis, 2008). It is unknown if scrapping has occurred since.

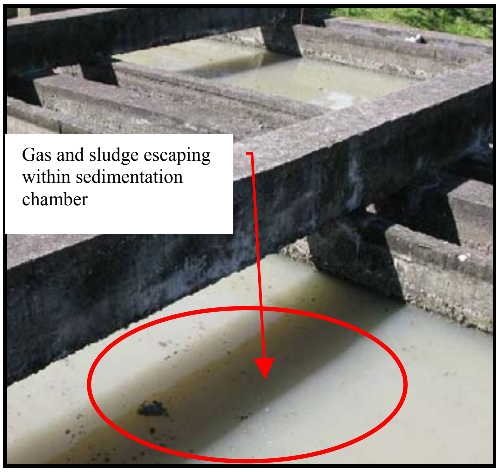

Figure 12: Gas Venting Within Sedimentation Chamber

Figure 13: Scum Caked Gas Vents

5.5 Water Quality Assessment:

The combination of excessive flows, short circuiting, and inadequate maintenance has affected the facility's ability to provide adequate treatment of wastewater prior to discharge. During the study, water samples were obtained from the existing treatment works to determine what level of treatment the system was providing. This consisted of collecting samples to measure the following characteristics of influent and effluent water: turbidity, chemical oxygen demand (COD), pH, and *E. coli*[9]. Sludge quality tests were not performed in this assessment. Samples for influent were obtained from the supply line access point approximately 4 meters from the tank inlet, effluent samples were taken from the discharge point into Raices Creek. Figure 14: reflects sample collection of influent to the system.

[9] Turbidity and COD testing data from initial assessment tabulated in Appendix I.

Figure 14: Influent Sample Collection

These samples were transported to and tested within a secured field laboratory provided by the municipality and set up by MIT students. Laboratory instrumentation utilized in these experiments consisted of a HACH Spectrophotometer (turbidity and COD), HACH COD incubation reactor, pH test strips, and 3M Petri-films (E. coli)[10]. These laboratory supplies had been brought into the country from the United States by the MIT team. Emphasis in selecting these water quality parameters for measurement and the methods used to test them took into account the number of samples needed, ease of transporting the equipment into the country, and the time schedule within which work needed to be completed. It should be emphasized that all measurements were collected over a period spanning four days (January 14 – 17, 2009) and reflect the state of the system at that time

5.5.1 Turbidity Measurements for Las Vegas Imhoff Tank

Turbidity measurements were made using the HACH Spectrophotometer which provided readings in Nephelometric turbidity units (NTU). The meter was zeroed using a blank that consisted of local tap water available within the laboratory space. Timing of sample collection between influent and effluent were spaced by a detention time of 90 minutes. The results of the turbidity survey are expressed in Figure 15, and show that at time of testing the plant was providing an approximated 18% drop in the wastewater turbidity. These values seem abnormally low for a surface overflow rate that is only 25% higher than design capacity; however, in the light of the high short circuiting percentages observed without the necessary flow gates these values seem justifiable.

[10] Specific methods and instrumentation are described within the sections for each observed property.

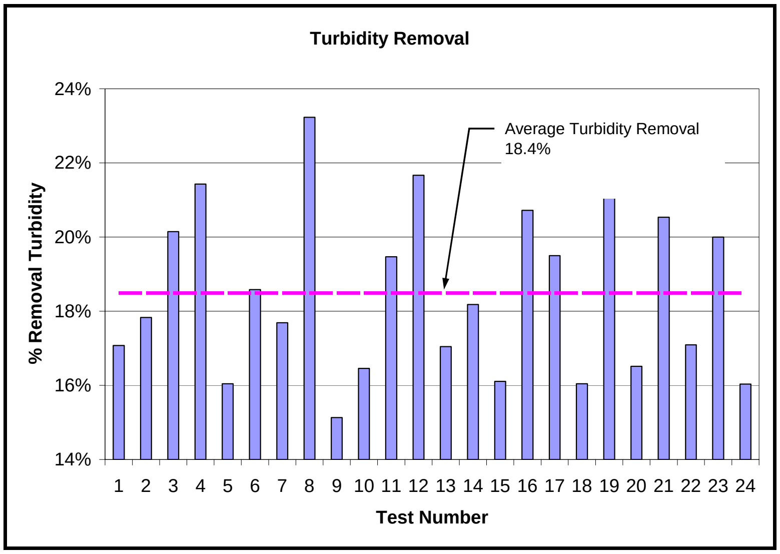

Figure 15: Turbidity Removal Efficiency for Las Vegas Imhoff Tanks, January 2009

5.5.2 COD Removal for Las Vegas Imhoff Tanks

Two measurements are typically made in regard to oxygen demand. These are the ~~biological~~ biochemical oxygen demand (BOD) and chemical oxygen demand (COD). The work conducted in the lab utilized assessment of COD only. The COD test was selected because procedures for BOD testing require longer incubation periods and greater demands for laboratory equipment. COD tests were conducted using the Reactor Digestion Method (Method 8000) as defined by the United States Environmental Protection Agency. Each test represents the removal of COD between influent and effluent samples offset by the approximate detention time of 90 minutes. The results of the COD tests are provided in Figure 16 and reflect an average COD removal efficiency of approximately 7 percent, well below rates~~. The samples collected and tested showed COD removal rates below those~~ expected for ordinary primary treatment systems. This is again attributed to problems associated with short circuiting through the system, excessive flow rates, and loss of detention time due to sludge accumulation.

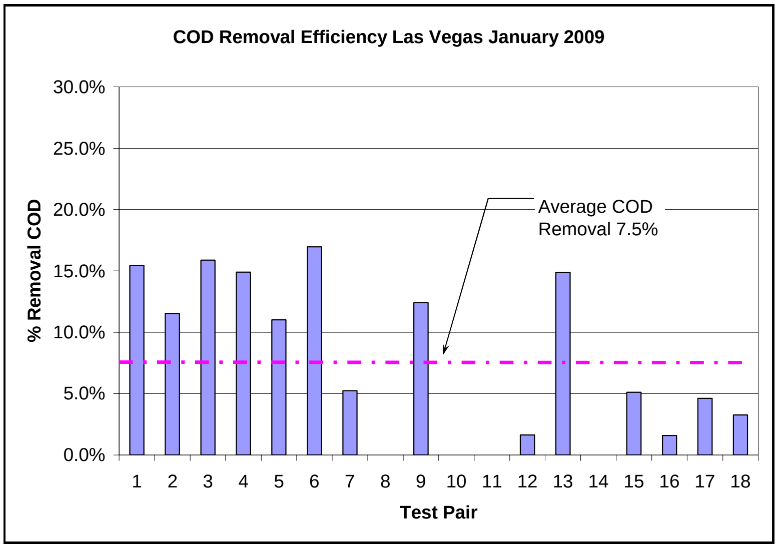

Figure 16: COD Removal Efficiency Observed Las Vegas January 2009

5.5.3 pH and E. coli Measurements of Las Vegas Wastewater

The pH of all wastewater samples varied from 8 to 8.5 and was within the range of effluent quality standards established by ERSAPS. These samples were tested utilizing pH test strips which measured the influent and effluent directly. *E. coli* testing was performed utilizing 3M Petri-films and 1 milliliter sampling quantities. The test required utilization of numerous dilutions to obtain a sample that would prove countable. The dilutions ranged from pure sample to a 10,000 to 1 dilution. The only countable sample was that of the 10,000 to 1 which provided counts of 9 colonies (90,000/mL) and 8 colonies (80,000/mL) for influent and effluent respectively. This reflects a negligible removal of approximately 10 percent, well outside the requirements of ERSAPS.

5.6 Community Wastewater Infrastructure Planning:

The municipality is looking for ways to obtain higher connection rates for its wastewater treatment infrastructure. As mentioned previously in the sections which examine flow, the Imhoff Tank is already receiving excessive amounts of flow. This, coupled with maintenance deficiencies, is contributing to lower overall effluent quality. Municipal engineers are admittedly aware of this fact, understanding that the increases in flow can not be sent to the Imhoff Tanks. During the January 2009 visit the mayor and municipal engineers were eager to explain their plans to connect the surrounding communities of El Mochito and San Juan to a wastewater collection grid. The trunk line of this system utilizes a concrete encased 8 inch PVC connector line. Construction is currently underway on sections of this project. However, the final upstream and downstream termination sites for the connector have not been detailed. Estimated distance from current construction and the surrounding communities is several kilometers. The municipality is unsure whether this system will simply transport untreated wastewater to a remote site for discharge or to a treatment works.

The community is interested in finding either alternative treatment technologies with which to replace the Imhoff tank system or ways to improve upon the current infrastructure. However, it is vital that the municipality first take an active role in understanding the operational and maintenance needs of the current system before any additional technological improvements are going to prove of merit. The design capacities of the current Imhoff tanks are exceeded. Efforts at improving infrastructure should focus on continuing programs to decrease water usage and developing good maintenance practices. This will allow for increases in facility life for the Imhoff tanks and a realization of better use for the capital initially spent upon this system. The next two chapters examine improvement technologies that may provide the community of Las Vegas with ways of enhancing its current Imhoff tank system.

Chapter 6: CEPT, Applicability to Las Vegas Imhoff Tanks

Chapter 4 introduces the practice of chemically enhanced primary treatment (CEPT) and proposes its potential use as a means of improving wastewater treatment efficiencies for systems such as Imhoff tanks. Research and testing of CEPT within Las Vegas' Imhoff tanks occurred on a bench scale in Honduras during January of 2009. Additional testing to compare two of the most common coagulants for efficacy side by side took place at MIT in March of the same year. The aim of this research is to reveal whether CEPT provides a viable option for the community of Las Vegas in achieving higher quality effluent. This chapter focuses on: consideration of CEPT as a site specific option, methodology for testing CEPT in Las Vegas, results of the Las Vegas experimentation, and low dosage efficacy comparisons between aluminum sulfate and ferric chloride.[11]

6.1 Introduction, Site Specificity:

Under the right conditions CEPT may be a viable option as an enhancement technology for Imhoff tanks. Truly assessing this notion would require testing CEPT in a setting that provides no technology other than an Imhoff tank. The site in Las Vegas provides this opportunity. Observing the effectiveness of CEPT as a means of improving Imhoff tank performance in this ideal setting will allow for determination of whether this technological pairing is appropriate in its own right. CEPT may provide the municipality an option to increase the efficiency of this system with only modest levels of investment, until such time as further capital improvements are possible.

As Chapter 5 mentions, the Las Vegas municipality is eager to examine options for improving the current infrastructure, though they understand that not all technologies are necessarily right for their situation. As such, they provided access to laboratory facilities, to the site, and to its resources, for the purposes of developing a working body of knowledge on the subject of CEPT and its application to the Imhoff tanks in Las Vegas. Previous efforts by Mikelonis in 2008 provided a piece of this information using alum as a possible coagulant, with the conclusion that the chemical may be cost prohibitive at the necessary dosages. The current effort focuses on a different chemical, ferric chloride, as another possible alternative.

6.2 Methodology for Testing CEPT in Las Vegas:

The initial starting point when considering the use of any CEPT system is bench scale testing, as it provides the engineer valuable insight into expected behavior of the system within a short period of time (Metcalf and Eddy, 2003). However, the results achieved with CEPT are wastewater specific; that is, each site must be tested individually to account for response differences attributed to varying water chemistries. Properly performed bench scale studies of CEPT provide insight into: optimization of chemical coagulants, chemical application sequence, confirmation of mixing conditions for flocculation, and estimations of hydraulic surface loading through measurement of settling velocities (Kawamura, 2000).

[11] Chemical specific information found in Appendix II.

The next step in a testing regimen would be to run pilot scale tests to confirm correlations between the predicted results of bench scale and observed results of pilot scale. Contrary to the time commitments demanded of bench scale tests (approximately 200 hrs), pilot studies require 6 to 12 months to obtain reliable data and are expensive to perform (Kawamura, 2000).

The scope of the CEPT testing performed in Las Vegas during January 2009 was limited to short duration bench scale testing as schedule and budget constraints in this study prohibited pilot scale efforts. This differs from the previous work of Mikelonis in 2008 that used alum to conduct a small pilot scale test of CEPT within the Imhoff tank. The work in 2008 indicated good agreement between the bench test and single pilot scale test. Similar outcomes would be expected with ferric chloride. The reason for the different testing scopes is that available supplies large enough to perform pilot tests of ~~Alum~~alum[12] could be obtained within Honduras (Mikelonis, 2008). The same could not be said for ferric chloride. Attempts to obtain ferric chloride within Honduras proved futile; industrial processes within the country do not readily use the substance, and inability to ship substances listed hazardous by the United Nations into the country without proper paperwork both act to keep the chemical scarce. The small supply of ferric chloride utilized for the bench scale tests was hand carried into the country from the United States.

The present bench scale testing utilized influent samples from the Imhoff tanks, the Phipps and Bird jar tester (Figure 17), and the field laboratory instrumentation (see Section 5.5). Samples were collected and immediately transported for processing in the lab. Processing and testing consisted of dosing the samples at various levels with a premade solution of ferric chloride, $FeCl_3$, dissolved into the local tap water. Equivalence between bench scale modeling and full scale Imhoff tank conditions can be accomplished by allowing the device to input a given mixing energy representative of possible injection sites and then provide a comparable time of settling that would reflect the surface overflow rates of the Imhoff tanks.

Samples were drawn from the jar mixer system and tested at times reflecting the measured and ideal surface overflow rates of Las Vegas' Imhoff tanks. These samples were tested for turbidity, COD, and pH using the methods detailed in Section 5.5[13]. The values, reflecting the concentration of dosing solution, mixing times, mixing energies, and detention times, have been tabulated in Table 7[14]. Results for each type of test are discussed in the following paragraphs.

[12] approximately 20 kilograms
[13] Total suspended solids (TSS) were not tested while in Honduras due to an inability to calibrate the HACH meter to local TSS levels.
[14] For development and calculation of these parameters see Appendix III.

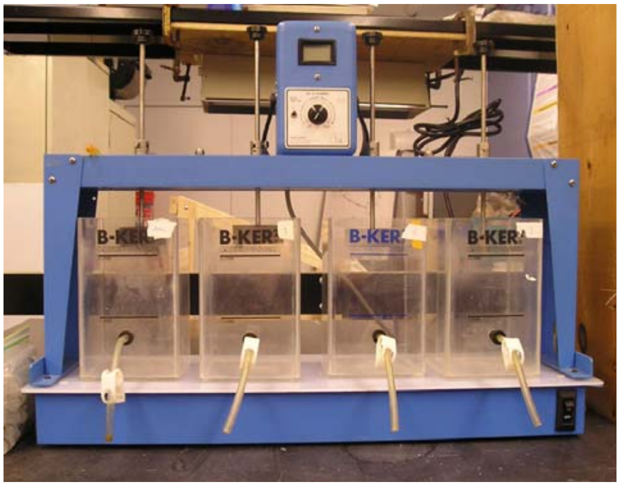

Figure 17: Phipps and Bird Jar Tester (Mikelonis, 2008)

Table 7: Parameters Utilized in Bench Scale CEPT Testing in Las Vegas

Parameters Constant throughout Bench Scale Testing in Las Vegas	
Concentration of Dosing Solution (mg/ml)	100
Volume of each mixing Jar (liters)	2
Mixing Time, t for Phipps and Bird Jar Tester (sec)	30
Phipps and Bird Jar Tester motor speed (RPM)	100
Mean Velocity Gradient, G associate with RPM above (sec^{-1})	100
Product of Gt (unitless)	3000
Detention Time correlating with Las Vegas surface overflow rate (sec)	250
Detention Time correlating with Ideal Imhoff surface overflow rate (sec)	400

6.3 Examination of Results for CEPT Testing in Las Vegas:

Turbidity

Part of the present study focuses on the observation of turbidity removal efficiency using varying doses of ferric chloride. The Las Vegas turbidity removal study consists of twenty-four separate tests. Each test examines the removal efficiencies achieved for a specific dosage at two distinct times, which correlate with the current surface overflow rate of 52 m/day (Section 5.3) and the ideal surface overflow rate of 33 m/day (Crites, 1998). Observations from the turbidity study are plotted in Figures 18 and 19.

The data plot represents the removal efficiencies achieved at various ferric chloride dosages[15,16] and its curves are typical of those used in optimizing dosages of coagulant chemicals to achieve the desired removal efficiencies. The data demonstrates that a maximum removal efficiency of approximately 92% is achieved when dosing with 325 mg/L of ferric chloride for the current surface overflow rate and that a maximum removal efficiency of 93% is achieved for a dose of 300 mg/L with the ideal overflow rate. These doses are very high and would prove cost prohibitive should they need to be ongoing. However, in considering the removal efficiencies achieved with lower range dosing between 100 – 125 mg/L, removal efficiencies are approximately 20% better than those expected with an optimized Imhoff tank alone[17].

Comparison of the turbidity removal values in this study with the values obtained using ~~Alum~~ alum in the study by Mikelonis (Figure 20) [18] shows that ferric chloride is providing approximately fifty percent higher removal of turbidity at the same dosage.[19] Unfortunately, similar low dosage data does not exist for comparison purposes within the present Las Vegas study. The laboratory testing performed at MIT in March 2009 was conducted to address this deficiency in data collected (Section 6.4).

Comment [EA11]: Spellling of removed on y-axis. Both figures.

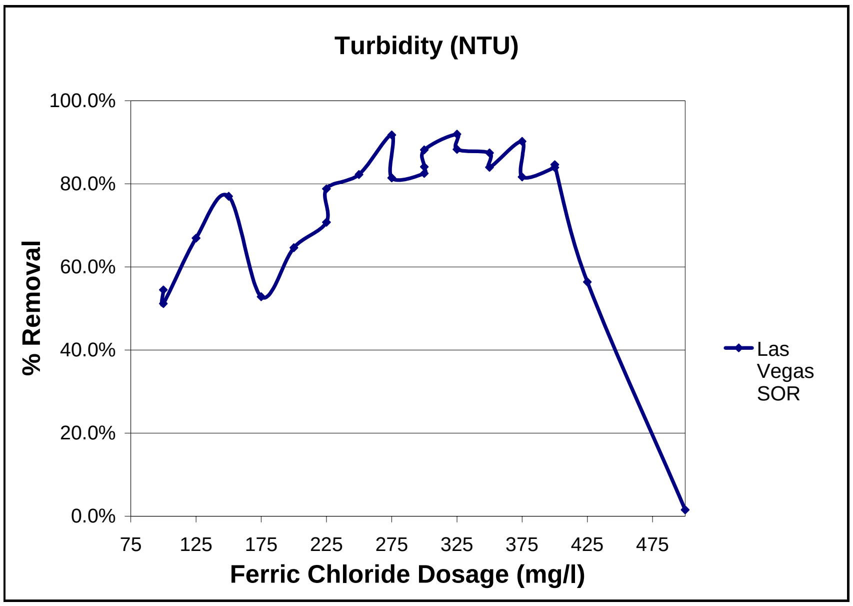

Figure 18: Turbidity Removal Efficiency with Ferric Chloride and Measured Overflow Rate

[15] Ferric chloride dosages reflect the quantity of dry weight product; Appendix II provides information pertinent to percent by weight of active Fe.
[16] Raw data results of the study in Appendix III.
[17] Idealized Imhoff tank results are provided in Chapter 3.
[18] A 6.5 minute detention time equates to ideal Imhoff tank design, 2.5 minute detention time with the observed surface overflow rate of 191 m/day (Mikelonis, 2008).
[19] Water chemistry variability is thought to exist between the two studies. These were not addressed.

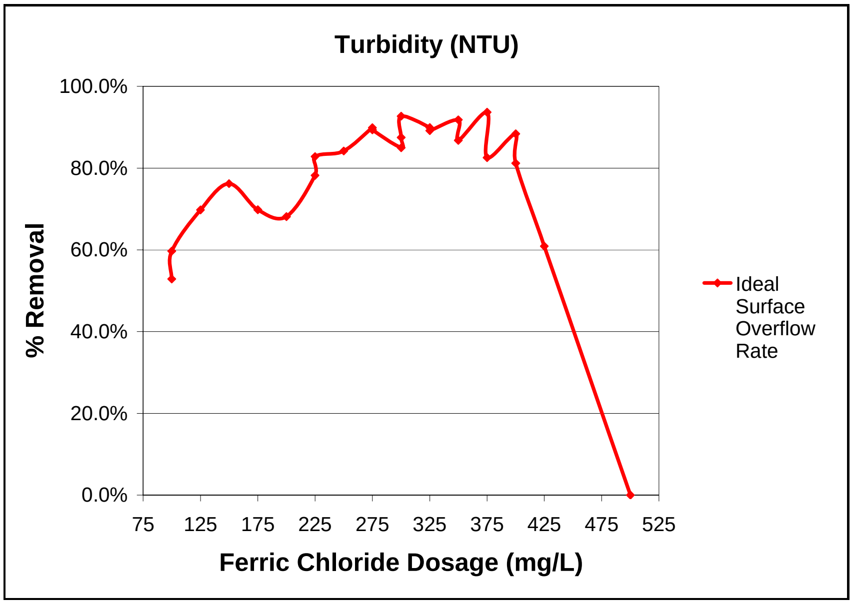

Figure 19: Turbidity Removal Efficiency with Ferric Chloride and Ideal Overflow Rate

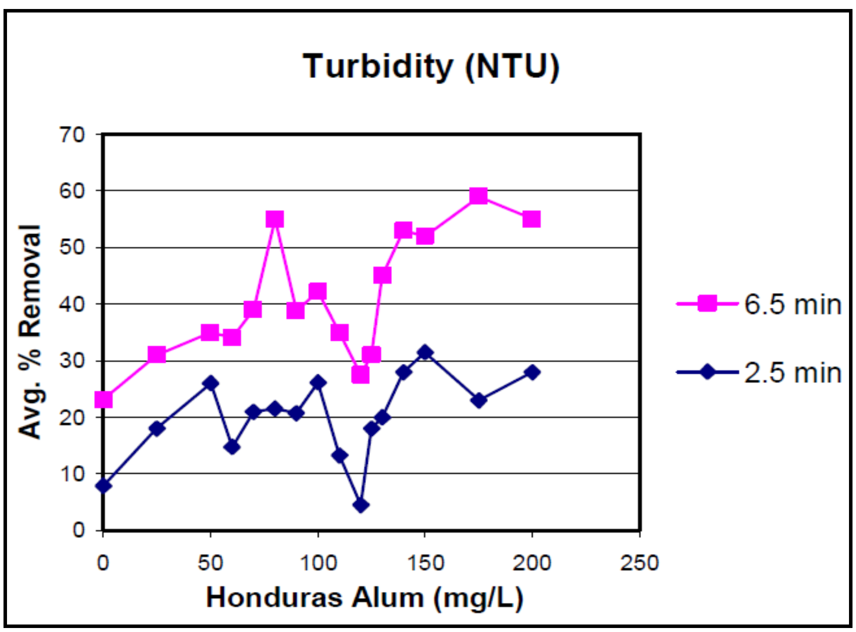

Figure 20: Turbidity Removal Efficiencies Using Alum (Mikelonis, 2008)

Chemical Oxygen Demand

Another part of the present study focuses on the observation of chemical oxygen demand (COD) reduction. The process of sedimentation produces a decrease in the COD found within wastewater effluent, which is attributable to a certain fraction settling out with the primary sludge. For this reason it is important to assess the COD reductions that are obtainable utilizing CEPT.

Las Vegas' bench scale testing for COD involved the collection and analysis of eighteen samples tested for COD[20] removal efficiency at varying ferric chloride dosages. The test utilized the same correlations as turbidity testing between the observed surface overflow rate at the Imhoff tanks and the jar mixer. The results of this testing reflect COD removals oscillating between forty and sixty-five percent[21]. It is hypothesized that larger suspended particles could harbor more COD constituents; these would be the first to fall from suspension once the coagulation process is underway and may be contributing to the removal efficiencies observed at any dosage. Over the range of the dosages tested the average removal efficiency was 52%. Results of the experiment are plotted in Figure 21. The work of Mikelonis' study in 2008 also seems to reflect a sudden upswing in COD removal efficiencies at a threshold dosage similar to the present study (Figure 22), but unfortunately not enough data points exist to make a true comparison across the complete range of dosages tested.

Comment [EA12]: Spellling of removed on y-axis (looks like it has been cut off).

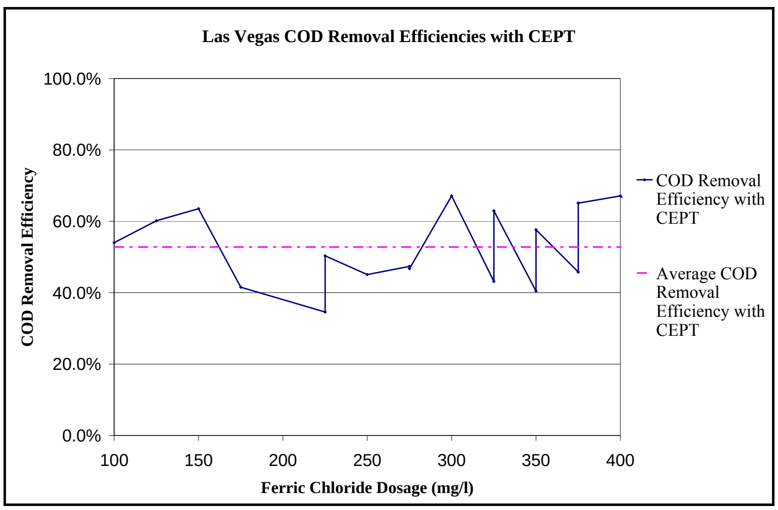

Figure 21: COD Removal Efficiency with Ferric Chloride in Las Vegas

[20] Section 5.5 provides method of COD analysis.
[21] Raw data results of the study in Appendix III.

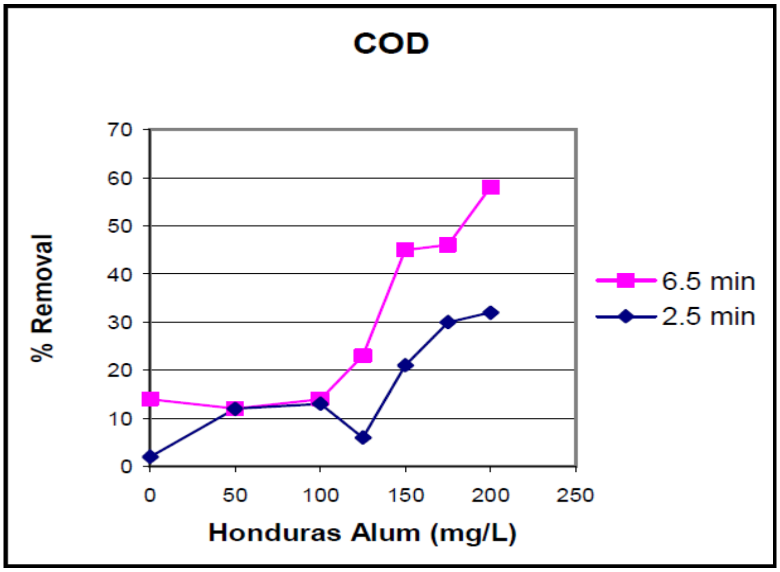

Figure 22: COD Removal Efficiency with Alum in Las Vegas (Mikelonis, 2008)

pH

Part of the present study also focuses on the testing of water pH. A concentration indicator of free hydrogen ions, pH is an important wastewater property and is regulated as such. Concentrations of free hydrogen ions also play an important role in water chemistry processes. The current pH value of a given influent can have dramatic affects on how effective a particular coagulant may be. Substances such as metal salts lower pH when they are utilized in CEPT processes which complicate water chemistry further. For ferric chloride the following reaction occurs with water lowering the pH (Metcalf and Eddy, 1991):

$$FeCl_3 + 3H_2O \Leftrightarrow Fe(OH)_3 + 3H^+ + 3Cl^-$$

It is important to monitor pH of both influent and effluent waters; aside from being a regulatory requirement, pH of influent will affect chemical efficacy and pH of effluent waters can be detrimental to infrastructure components. Consideration must be given to the changes in pH resulting from dosing with a coagulant at a given quantity. If pH is found to be out of range it may be appropriate to neutralize effluent waters prior to discharge. Neutralization technologies require careful monitoring and as such present a demand for skilled labor in operation.

The testing for pH is generally a simple matter; it can be tested often and with little effort on the part of operators. The bench scale study tested the pH of samples prior to and immediately after CEPT administration. The tested wastewater was found to be slightly basic (with an average pH of 8.0) prior to CEPT and slightly acidic (with an average pH of 6.2) after treatment with ferric chloride. Both were acceptable within the regulatory guidelines for Honduras.

Sludge Production with CEPT in Las Vegas

Creation of increased sludge is the desired characteristic when implementing CEPT. Increases are indicative that the process is achieving greater removal efficiencies of wastewater contaminants. However, careful accounting of this increased sludge production is necessary so that sufficient removal and treatment mechanisms are coordinated. In the case of typical primary clarifiers this may amount to simply increasing the removal rates of sludge sent off for processing elsewhere. This option is not available for the system in Las Vegas; instead, this system provides for sludge processing through the digestion of sludge. Unfortunately, chemically precipitated sludge (e.g. use of ferric chloride, alum, lime) requires longer periods of time for digestion. In addition, this sludge tends to be gelatinous; making it potentially difficult to flow through removal mechanisms (Lee, 2007). Specific testing is required to provide insight into increased digestion time periods and the difficulties that may be encountered with respect to mobilizing sludge.

In their existing state, Las Vegas' Imhoff tanks provide for removal of approximately 20 percent of all TSS which passes through the system. The generation of sludge from this removal amounts to approximately 40 kg per day. Alternatively, the use of CEPT to achieve the Honduran regulatory effluent requirement of 100 mg/L for TSS requires a removal of 47 percent, based upon observations during the water quality study. Achieving this level of treatment requires a minimum dosing of 100 mg/L ferric chloride and generates approximately 140 kg per day of sludge, 68% of which is removed TSS (95 kg/day). Appendix IV provides information pertinent to the calculation of sludge quantities.

The mass of sludge increases nearly four fold when generation from all sources is included in the accounting. For this reason, it is critical that studies ascertain the length of digestion time required when CEPT is used. This information should be utilized to decide if CEPT is appropriate for the given volume of the Imhoff tank digestion chamber, and what modifications may be necessary for sludge removal intervals.

6.4 Low Dosage Efficacy Testing of Alum and Ferric Chloride:

Examining low dosage efficacy is spurred by the fact that cost considerations associated with CEPT are particularly crucial when designing plants for developing countries (Kawamura, 2000). Accurate side by side comparison of the efficacy between the dominant coagulants, aluminum sulfate (alum) and ferric chloride requires their use as CEPT coagulants with the same source wastewater.[22] This process is ordinarily carried out during bench scale studies performed by engineers and plant operators in the chemical optimization phase.

Unfortunately opportunity for this type of analysis was not provided while in Honduras. Further complicating a comparison, wastewater from Las Vegas could not be transported to the United States and stored for use when testing could be carried out. However, a laboratory was set up at MIT which utilized the testing equipment from the Honduras study to test efficacy using

[22] Same source wastewater defined as water taken during one sampling collection, temporal and spatial congruency.

wastewater sourced locally. This source wastewater, raw primary influent, was obtained from the Massachusetts Water Resources Authority (MWRA) treatment facility located on Deer Island, Massachusetts[23].

Tests performed in the MIT comparison study repeated the bench scale testing procedures utilized in Honduras[24], with the additional inclusion of measuring reductions in total suspended solids (TSS). Approximately 50 liters of raw primary influent were collected and transported from the Deer Island facility to MIT approximately 15 km away. Samples were processed and tested shortly after delivery to preserve the representative integrity of results. Each coagulant was tested on 10 separate samples with varying low dosages ranging from zero to 250 mg/L (Appendix III). From the results of this test Alum was found to be a better performer in reducing both turbidity and TSS at low dosages. Ferric chloride was found to be slightly better than Alum in all cases for removal of COD. The results of the turbidity, TSS, and COD removal efficiency comparison study are plotted in Figure 23 - 25 respectively[25].

Comment [EA13]: Use Word's find tool to be consistent with capitalization. Alum is most commonly in lower case, but I have seen it in upper

These results do not necessarily support all of the conclusions obtained during the Las Vegas study. Ferric chloride does not outperform alum with the same ratio. However, there is support for increased removal efficiencies of COD utilizing CEPT. Adequate correlation between the two coagulants and their performance at increasing removal efficiencies for the Las Vegas Imhoff tanks would require that these studies be carried out in country with similar samples as outlined previously. A bench scale study followed by pilot scale study of this matter could be carried out over a few months and is a recommendation for further review.

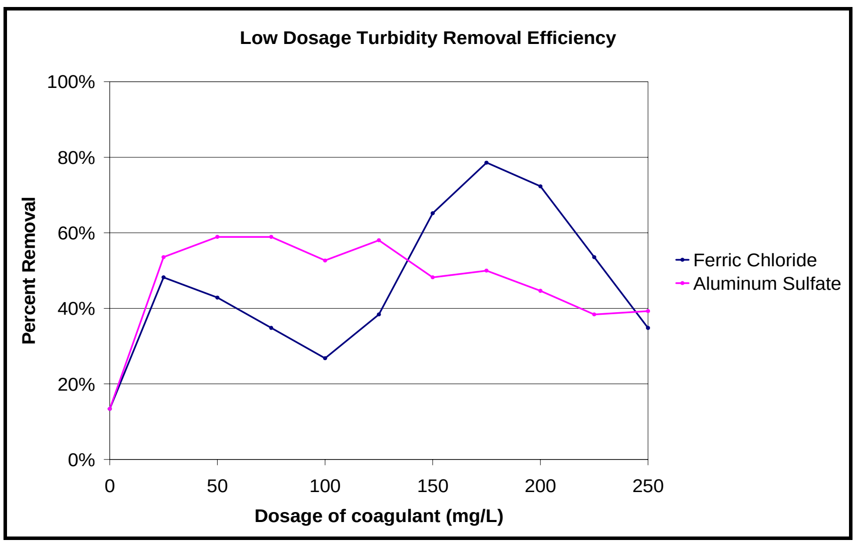

Figure 23: Plotted MIT Coagulant Comparison for Turbidity

[23] Influent to facility is greatly diluted due to combined sewer/storm and recycling in plant processes (Tyler, 2009).
[24] Table 7: Parameters utilized in bench scale CEPT testing in Las Vegas, provides reference.
[25] Dosages listed represent dry weight of chemical, for percent active ingredient see Appendix II.

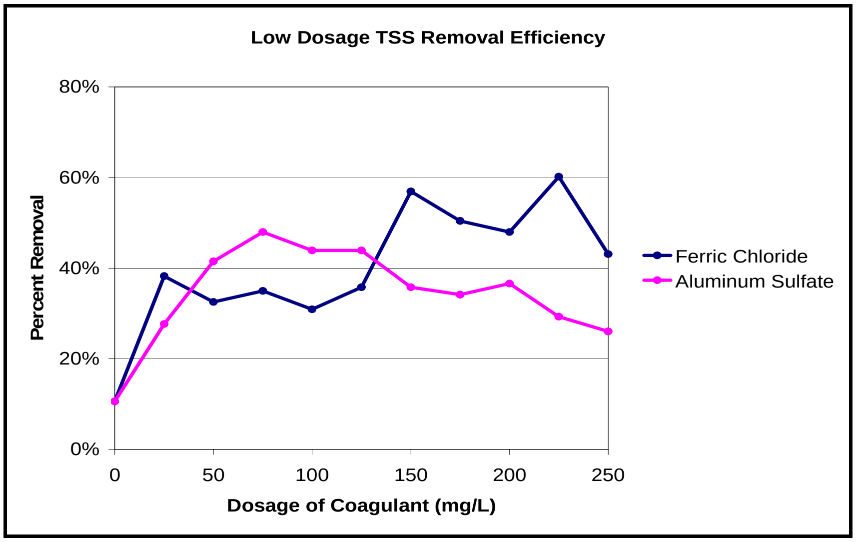

Figure 24: Plotted MIT Coagulant Comparison for TSS

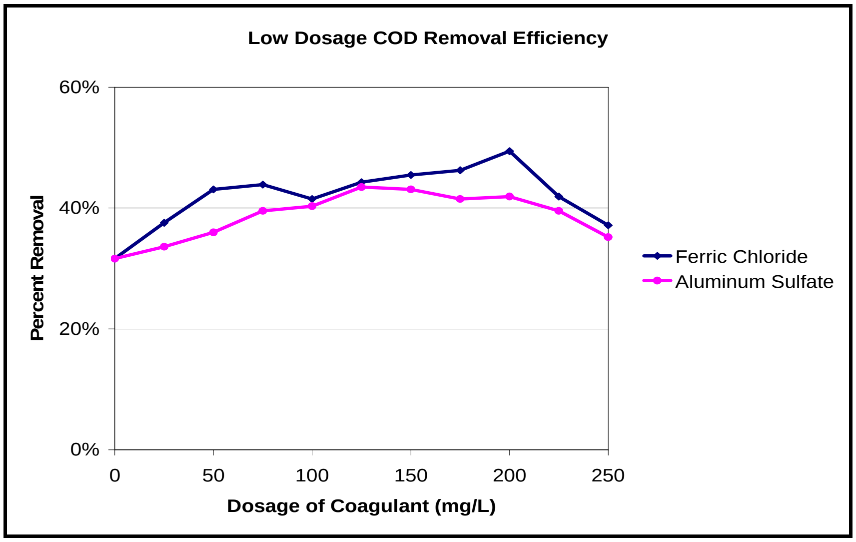

Figure 25: Plotted MIT Coagulant Comparison for COD

Chapter 7: Enhancement Infrastructure for Las Vegas Imhoff Tanks

As detailed in Chapter 5, the largest piece of wastewater treatment infrastructure within the township of Las Vegas is the Imhoff tank. Serving some 3,600 residents, this system is currently overloaded, receiving greater than capacity flows long before its planned service life has expired. This condition is reducing effluent quality which does not meet the current regulatory standards. Planned new-construction of wastewater treatment infrastructure throughout Honduras will eliminate the use of Imhoff tanks as a treatment option in an attempt to bring treatment processes up to newer technological standards (Ortiz, 2009). Unfortunately, this plan does not present a solution for improving Imhoff tank systems which presently exist, have unrecovered capital expenditures from construction, and still possess a theoretical service life.

This chapter seeks to introduce appropriate technologies which can be coupled to the Las Vegas Imhoff tanks in an attempt to salvage its treatment purpose and allow it to effectively remain online until such time that capital investment is again justified on newer technology. These enhancements are separate from the use of CEPT as described in Chapter 6; they involve alternative technologies which connect in series to the current infrastructure: pre-treatment technologies (flow bypass and control, bar-screens, and grit chambers), secondary treatment processes (trickling filters and constructed wetlands), and nuisance odor reduction systems (odor reducing bio-filters). These enhancement processes have proven effective for use in decentralized treatment systems such as those found in Las Vegas (Crites, 1998).

7.1 Pre-Treatment Technologies:

Raw primary influent to systems such as the Imhoff tank should ordinarily be first processed through some means of pre-treatment. Applicable to primary systems, such as Imhoff tanks, the purposes of pre-treatment are to regulate flows of the raw sewerage influent, reduce potential flow obstructions, and achieve a reduction of course solid debris (Metcalf and Eddy, 1979). In addition to these treatments, a simple method of flow measurement should be provided; one such method is outlined for use with grit chambers (Section 7.1.3). These processes allow for efficient use of tank volumes both within the sedimentation and sludge digestion chambers.

7.1.1 Flow Bypass and Flow Regulation Measures

The ability to bypass and control the distribution of wastewater flow through any treatment facility is vital for purposes of efficient operation and in performing system maintenance. The current configuration of the Imhoff tanks in Las Vegas does not provide any method of flow control. For this facility, flow control measures should consists of at least four distinct forms: a bypass allowing flow to circumvent~~ing~~ ~~flow entirely to~~ the entire system, baffles to reduce short circuiting and increase plug flow condition, and flow control gates to regulate the direction of flow and hence the distribution of sludge within the digestion chamber.

Comment [EA14]: I only count 3

Flow Bypass

The current feed line to the system is above grade at locations immediately upstream from the Imhoff tank. Such exposure allows for easy retrofitting of the current trunk line for purposes of installing a bypass. This could be accomplished by excavating the soil beneath the influent feed line and building the bypass system within this excavation. Once completed the original section of trunk line could be demolished allowing flow to pass through the new bypass location. This construction technique would allow for work to be completed on this section without creating a need to divert flow from the existing system.

Important aspects in the design of this bypass include minimizing both the head loss through discharge pathways and the difference in elevation between inverts of existing and new construction. Appropriate freeboard will ensure adequate flow through the bypass discharge orifice by providing necessary head. The use of an adequate slope for the discharge channel downstream of the ball valve will reduce the opportunity for backwater conditions to develop within the bypass. A ball valve is recommended at the discharge orifice to reduce head losses, encouraging open channel flow, and allow for easy cleaning if clogging materials become entrained. To ensure that entrapment of particles does not become a problem within the bypass the difference between invert elevations of adjacent trunk line sections and the bypass should be minimized. Plan and elevation views of this proposed construction are provided within Figures 26 and 27 respectively.

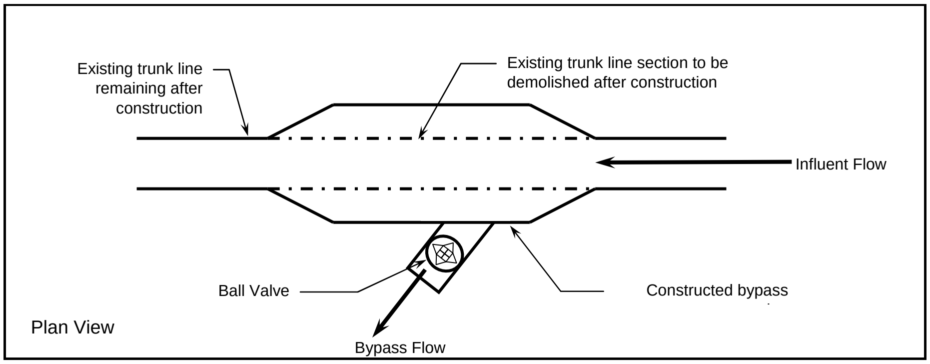

Figure 26: Plan View of Flow Bypass Construction

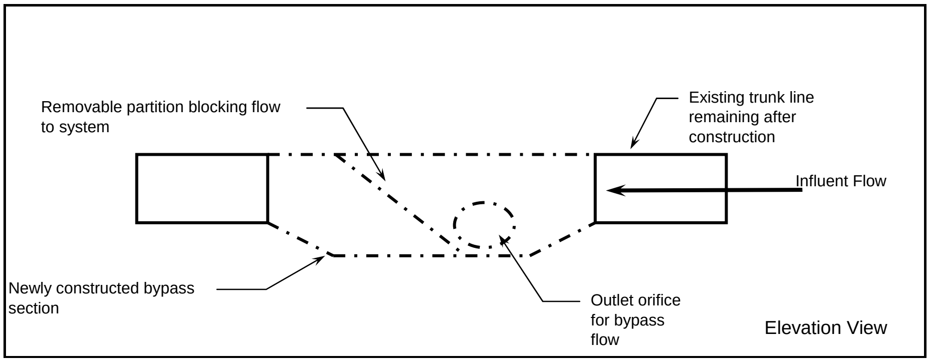

Figure 27: Elevation View of Flow Bypass Construction

Flow Equalization Baffles

Proper sedimentation requires that conditions approaching plug flow be maintained within the sedimentation chamber. A simple technology often employed to create this condition are perforated baffles which regulate and equalize the flow of water to the sedimentation tanks. The inlets to the sedimentation chambers of the Las Vegas Imhoff tank are not baffle controlled. At approximately 1 meter (0.9 m) wide, these unregulated inlets provide pathways which preferentially transport wastewater through the system. This preferential movement of water allows for short circuiting and does not promote plug flow. The municipality is aware of this missing technology through the efforts of Mikelonis, who conducted small scale tests to familiarize the municipality with the use of perforated baffles[26]. The tank does not presently have a system for permanently installing these baffles. However, it is possible to chisel away portions of the concrete at the inlet to allow for notches that these baffles may be slid into. To ensure that these baffles are not stolen they should be constructed of metal and fitted with a lock that connects to an eyelet grouted into the concrete of the Imhoff tank.

For the Las Vegas Imhoff tanks, two baffles are required to be constructed, one for the influent side of each tank. Baffles are recommended as an influent control measure only and do not need to be included at the effluent side of the sedimentation chamber. However, notching and grouted eyelets should be provided at either end of the sedimentation chambers since the direction of flow will be switched biweekly as recommended within Chapter 3; baffles should be switched at the time of changing flow direction with baffles being placed at the appropriate inlet side of sedimentation chambers.

Design of the baffles and appropriate freeboard should be done using the peak volumetric flow for the system. Use of peak flow will ensure that baffle perforation is adequate for the prevention of overflow from the distribution channel. Of equal importance, baffles should be fabricated as thin as possible to decrease frictional losses associated with through flow. Schematics of as recommended flow baffles for the Imhoff tank of Las Vegas are provided in Figures 28 and 29.

[26] See (Mikelonis, 2008)

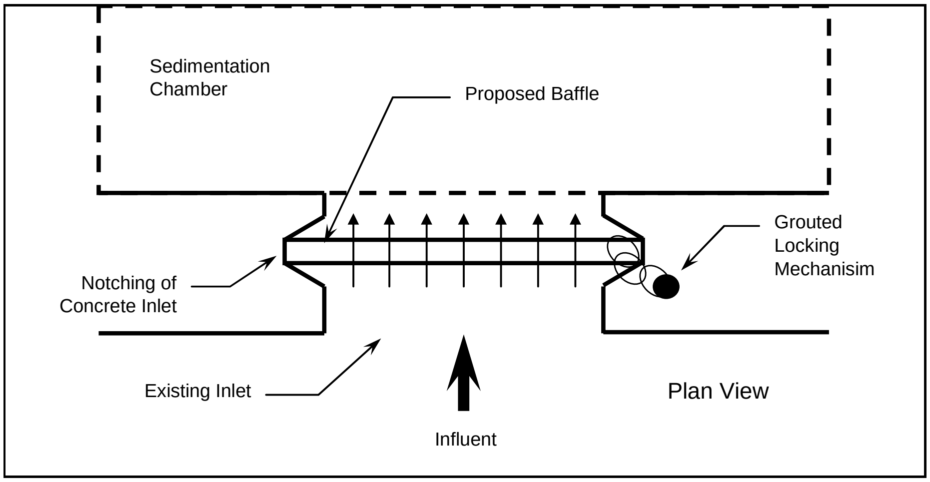

Figure 28: Plan View of Proposed Baffle Wall for Sedimentation Chamber Inlet

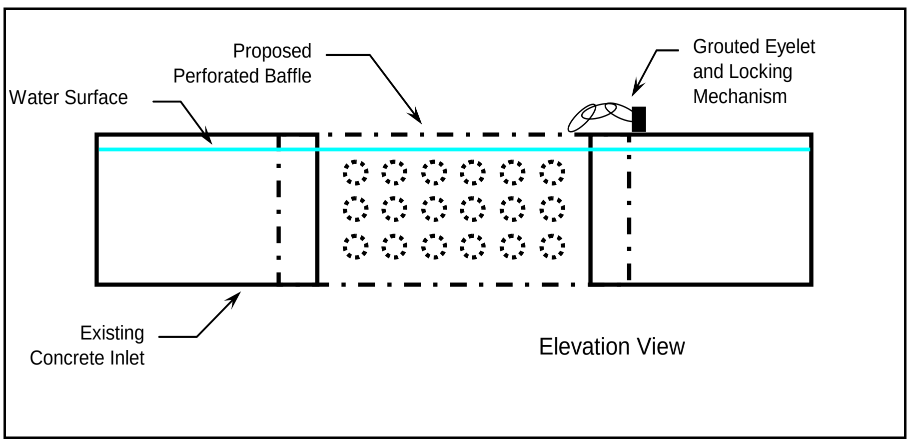

Figure 29: Elevation View of Proposed Baffle Wall for Sedimentation Chamber Inlet

Flow Control Gates

Adequate digestion of sludge within an Imhoff tank is dependent upon a number of factors, one of which is the time allowed for this digestion process to occur. Normalizing the distribution of sludge thickness within the digestion chamber will optimize detention times for sludge within the chamber. To achieve even distribution of sludge within the digestion chamber it is important to regulate the direction of flow to the sedimentation chamber. This is accomplished through the use of flow control gates as mentioned in Chapter 3. These control gates differ from the baffles in that they are not perforated and thereby prevent flow in a given direction.

Las Vegas' Imhoff tanks were originally designed with these in place; the work of Herrera in 2006 provides detail regarding their configuration to achieve desired flow patterns. A schematic depicting location for the flow control gates and configurations for the two possible flow patterns have been provided in Figure 30 and Table 8 respectively. However, as described in Chapter 5 these flow control gates are no longer present. Notching within the distribution channel is present for these gates to fit within.

Similar to the flow control baffles, it is recommended that new gates be fabricated from either metal or concrete with a means of locking the gates to the main Imhoff tank structure through the use of grouted eyelets and locks or bolts. Of the two flow configurations, flow pattern one requires use of four gates; it is recommended that a minimum of 6 gates be fabricated for redundancy. A minimum of four grouted eyelets, with long enough chains, are required for properly securing the flow control gates. These gates should always be in place to prevent wastewater from bypassing the Imhoff tank untreated.

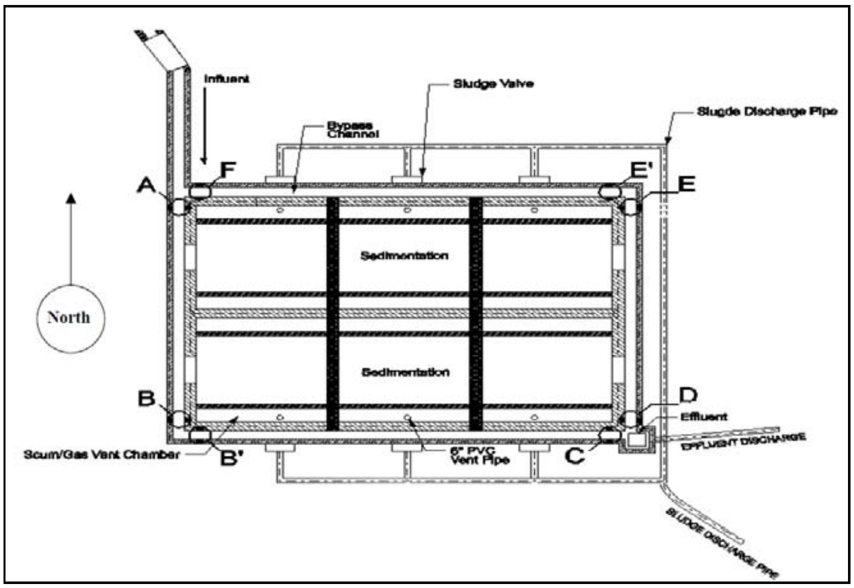

Figure 30: Potential Locations of Flow Gates (A through F)
Source: (Herrera, 2006)

Table 8: Flow Gate Configuration for Flow Patterns

Gate	Flow Pattern I	Flow Pattern II
A	Open	Closed
B/B'	Closed	Open
C	Closed	Open
D	Open	Closed
E/E'	Closed	Open
F	Closed	Open

Source: (Herrera, 2006)

7.1.2 Bar Screens

The existing Imhoff tanks at Las Vegas do not provide a means of screening incoming wastewater for rags or large objects which may obstruct flow through the various orifices of the system. Screens are classified according to the method utilized to clean them: manual or mechanical. For the flows observed within a decentralized system such as Las Vegas the appropriate type are manually cleaned course screens. Screening with manually cleaned course screens is a relatively simple process, provided some simple logic is utilized in their construction.

Manually cleaned bar screens should require no greater raking distance than can be easily achieved by hand. Sufficient freeboard should exist, so that if the bar screens become clogged, flow is allowed to proceed over the bar screens and not bypass the treatment works through overflow. The floor of the channel section which contains the bar screens should be sloped and not present pockets which could collect grit behind screened solids (Metcalf and Eddy, 2003). Raking of the screens should be performed daily at a minimum and more frequently if clogging becomes a problem. Collected screenings should be disposed of through burial in an onsite pit or landfill disposal.

Locating the bar screen within the slot of the bypass partition (Section 7.1.1) is possible when flow is not being bypassed from the tanks, see Figure 27 for detail. This configuration allows for one location to create two service points for the facility. Materials found within Honduras that can be utilized for fabrication of bar screens include concrete reinforcing bars welded into a grid or PVC pipes connected in a grid pattern. Table 9 provides a summary of typical design information for manually cleaned bar screens.

Table 9: Typical Design Parameters Manually Cleaned Bar Screens

Design information for manually cleaned bar screens				
	US Customary Units		SI Units	
Parameter	Unit	Value	Unit	Value
Bar size				
Width	in	0.2-0.6	mm	5.0 - 15.0
Depth	in	1.0-1.5	mm	25-38
Clear Spacing Between Bars	in	1.0-2.0	mm	25-50
Slope from Vertical	°	30-45	°	30-45
Approach Velocity				
Maximum	ft/sec	1.0-2.0	m/sec	0.3-0.6
Allowable Headloss	in	6	mm	150
Adapted from Wastewater Engineering Treatment and Resuse (Metcalf and Eddy, 2003)				

7.1.3 Grit Chamber Design

Grit is composed of sand, gravel, cinders, or similar heavy solids with settling velocities and specific gravities (typically 2.65 or higher) significantly greater than the organic degradable material found within wastewater (Crites, 1998). These materials, which do not readily degrade during sludge digestion, can significantly affect the sludge quality of Imhoff tanks. Valuable digestion volume within the sludge digestion chamber and hence detention time is presently lost to grit accumulation. Taking advantage of the differential settling characteristics of the grit from the putrescible organics allows for these materials to be disposed of without nuisance (Camp, 1942). The purpose of the grit chamber is to remove these materials prior to the primary settling provided by the Imhoff tank.

Grit chambers vary greatly in their design and complexity of operation. Some use vortices to separate these materials, while others use aeration. The simplest types use gravity and reductions in flow velocity to remove grit. This latter type of grit chamber was observed during the group survey at several wastewater treatment facilities in Honduras, and seems appropriate for use with small decentralized scale facilities such as Las Vegas. Locations such as Las Vegas suffer large infiltrations of groundwater into the sewerage collection grid after heavy rainfall events. Undoubtedly this infiltration carries large quantities of this grit material which would clearly settle within the tanks; the use of grit chambers coupled to this system would greatly reduce the impact of grit to the Imhoff tanks.

The simplest grit chambers are considered the channel-type rectangular horizontal-flow grit chambers which are customarily located downstream from the bar screen operation (Metcalf and Eddy, 1991). These systems utilize channel geometries coupled with inlet and outlet controls to maintain a horizontal flow velocity close to 0.3 m/sec, and provide a long enough detention time within the grit chamber to allow for settling of these particles. Flow controls also provide the added benefit of being able to measure flow if calibration marks are provided. Table 10 provides design recommendations for grit chambers. Typical designs include a minimum of two grit chambers in parallel for purposes of taking one off-line to perform maintenance removal of collected grit. This collected grit can either be buried in an onsite pit or disposed of to a sanitary landfill[27]. Figure 31 provides a schematic plan view and examples of outlet control geometries.

Table 10: Typical Design Parameters for Horizontal-Flow Grit Removal Chambers

		Value	
Item	Unit	Range	Typical
Detention Time	sec	45-90	60
Horizontal Velocity	m/sec	0.25 - 0.4	0.3
Settling velocity for removal of:			
50-mesh material (0.30 mm)	m/min	2.8 - 3.1	3
100-mesh material (0.15 mm)	m/min	0.6 - 0.9	0.8
Added length allowance for inlet/outlet turbulence	%	25-50	30
Adapted from Small and Decentralized Wastewater Management Systems (Crites, 1998)			

[27] If the later method is used it is suggested that grit be dried prior to disposal to save on transportation costs.

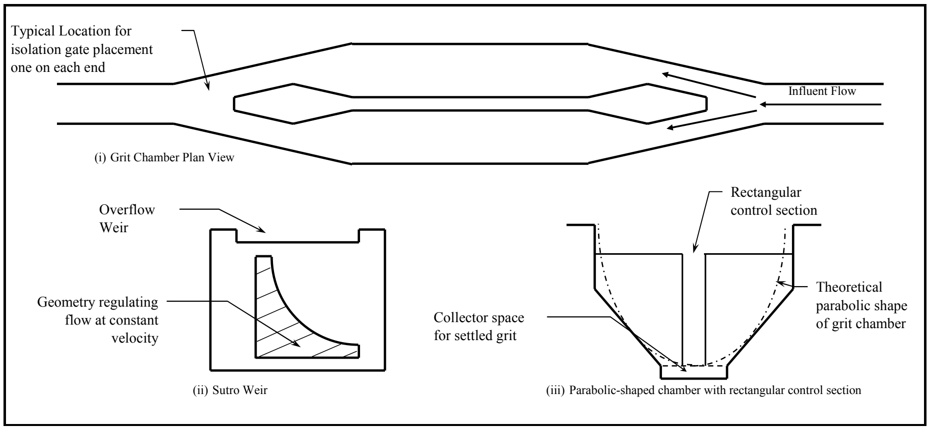

Figure 31: Schematic of Grit Chamber and Flow Regulating Controls

7.2 Secondary Treatment Technologies:

The previous section dealt with pre-treating wastewater flowing to Las Vegas' Imhoff tanks in order to improve treatment at the start of the primary treatment process. This section deals with how to treat the effluent from the Imhoff tank (i.e., secondary treatment). As mentioned previously the Las Vegas Imhoff tank system does not provide any form of secondary treatment. Applied to the current infrastructure, secondary treatment processes must provide a measurable improvement to final effluent quality that justifies their expense. They must be appropriate for geographic and resource constraints. In addition they must not be too sensitive to primary effluent quality so that they effectively process the residual loading from the Imhoff tanks. Finally they must not require heavy operational and maintenance budgets or technically skilled labor. This section considers alternative types of secondary systems which may be applicable to the Las Vegas Imhoff tank system: trickling filter systems and constructed wetlands (both free water surface and submerged flow).

7.2.1 Trickling Filter Systems

Trickling filters provide secondary treatment through the use of attached biological growth processes and are in fact not actual filters at all (Lee, 2007)[28]. In the truest sense of the word, a filter uses a physical process to separate a solute from a solvent, in this case waste from water. However, trickling filters provide this separation process through employing the use of biologically active microorganisms attached to media within the filter. The microorganisms that are attached to the filter media (rock, slag, or plastic) utilize the high surface area to volume ratios[29] of the media to grow in abundance; as a result these systems provide secondary treatment within a moderate sized unit.

---

[28] To avoid confusion the common nomenclature of trickling filter will be used within this work.

[29] Surface area per unit volume media 12 - 30 m^2/m^3 (Rock / Slag), 24 - 60 m^2/m^3 (Plastic) (Crites, 1998)

The attached biological growth consists of aerobic, anaerobic, and facultative bacteria; fungi; algae; and protozoa. All of these are found to a varying degree throughout the filter media. Aerobic conditions exist within the upper portions of filters and anaerobic conditions exist within the lower portions. Within the trickling filter, facultative species dominate the decomposition of organic material (Crites, 1998).

Delivery of wastewater to the filter is accomplished through direct spraying above the media; this contributes to the oxygenation of secondary influent. The process is usually carried out through the use of a distribution arm which sprays wastewater onto the media through nozzles as it is swept angularly across the tank. Rates at which wastewater influent are applied conventionally define the type of trickling filter (low rate, intermediate rate, and high rate). The mechanics of mobilizing the sprayer arm can be accomplished with motors or may utilize the momentum of water sprayed angularly downward to propel the arms forward. Because the latter method does not require special components and utilizes hydraulic head rather than electrical energy for propulsion, it is considered the better design alternative for Las Vegas. Sufficient head should be available given Imhoff tank effluent elevation and site topography.

In common practice, a portion of this wastewater distributed across the top of the filter is recirculation water made up of effluent from the trickling filter[30]. Recirculation provides an equalization step in wastewater applied, dilutes the effluent from the primary treatment system preventing shock conditions for the attached microorganisms, and contributes continual seeding of biological growth. This portion of the process requires the use of a mechanical pump to lift the water back to the head level it possessed prior to distribution through the rotary arm. This lifting distance will require accounting of the following: depth of filter media, depth of underdrain system, height of freeboard, travel to meet primary effluent, and minor losses. Understanding that particulate organic and grit matter will be present in this recirculation water it is important that an appropriate pump be used which will accommodate this debris.

This debris, which is present to some extent within the effluent from all trickling filters, is typically removed through the use of a secondary clarifier. Made up of both residual untreated organics and sloughed microorganisms from the filter media, this material can be problematic if not properly settled and disposed of. The installation of a secondary clarifier should complement any trickling filter system. Several different types of clarifier exist which would be appropriate to the decentralized needs of Las Vegas. All can be modeled after the sedimentation processes outlined within Chapter 3.

The National Research Council performed an extensive study of trickling filter design and performance within military installations after World War II (NRC, 1946). These studies utilized collaborative efforts between both designers and operators of facilities, and led to the development of efficiency performance formulas, summarized in Table 11, which predictably calculate BOD removal percentages for conventional trickling filters. This information has since become the backbone of most trickling filter design calculations. Typical design parameters for conventional trickling filters have been compiled within Table 12.

[30] All trickling filters except low rate filters require recirculation; higher rate filters typically require greater recirculation ratios.

Table 11: Efficiency Formulas Developed by NRC for Trickling Filter Design

Equation	Definition of Terms
Single-stage or first-stage of a two-stage trickling filter: $$E_1 = \frac{100}{1 + 0.0561\sqrt{\frac{W_1}{VF}}}$$	E_1 = efficiency of BOD removal for process at 20ºC, including recirculation and sedimentation, percent W_1 = BOD Loading to filter, lb/day V = volume of filter media, $10^3 ft^3$ F = recirculation factor
Recirculation factor: $$F = \frac{1+R}{(1+R/10)^2}$$	R = recirculation ratio Q_r/Q Q_r = recirculation flow Q = wastewater flow
Second-stage filter: $$E_2 = \frac{100}{1 + \frac{0.0561}{1-E_{if}}\sqrt{\frac{W_2}{VF}}}$$	E_2 = efficiency of BOD removal for second-stage filter at 20ºC, including recirculation and settling % E_{1f} = fraction of BOD removed in first-stage filter W_2 = BOD loading applied to second-stage filter, lb/day
Adapted from NRC study data courtesy Crites, (1998)	

Table 12: Recommended Design Parameters for Trickling Filters

Item	Low Rate	Intermediate Rate	High Rate	High Rate
Filter Medium	Rock/Slag	Rock/Slag	Rock/Slag	Plastic
Size, cm	3-12/5-12	3-12/5-12	3-12/5-12	Vendor Spec
Specific Surface, m^2/m^3	12--30	12--30	12--30	24--60
Void Space %	40--55	40--55	40--55	92--97
Specific density, kg/m^3	3800--7000	3800--7000	3800--7000	150--450
Hydraulic Loading Rate				
m^3/m^2•day	1.2--3.5	3.5--9.4	9.4--37.5	12--70
Organic Loading Rate kg $BOD_5/10^3 m^3$•day	80--400	240--480	480--1250	800--3200
Depth, m	2--3	2--3	2--3	3--12
Recirculation Ratio	0	0--1	1--2	1--2
Sloughing	Intermittent	Intermittent	Continuous	Continuous
BOD_5 Removal Efficiency, %	80--90	50--80	65--90	65--90
Filter Flies	Many	Some	Few	Few or None
Adapted from: Small and Decentralized Wastewater Management Systems (Crites, 1998)				

Proposed Design for Trickling Filter

Design of a trickling filter for the municipality of Las Vegas should utilize native materials[31]. The schedule of work would probably require a timeline no greater than three months with proper organization. This figure includes time estimates for the following: excavation and foundation work, set-up of formwork for concrete placement, curing time for placed concrete, and connection of necessary plumbing and electrical components. Establishment time for microorganism growth is also required. Dependent upon loading to the system, this time can be identified and recorded by monitoring removal of BOD_5. This value should be utilized in situations where re-establishment maintenance is needed.

The following preliminary stage design is an estimate of what is required to meet the secondary treatment needs of Las Vegas. It is based upon limited measurements of both quantity of flow and water quality. Further pilot scale studies, coupled with long term monitoring of water quality, flow patterns, and future treatment needs should be conducted prior to any finalization of design. With that caveat, the following is a draft design for providing secondary treatment by a trickling filter for the community of Las Vegas. Table 13 provides dimensions, loading, efficiency, and power consumption estimates for the draft designed trickling filter to service Las Vegas. See Appendix V for calculation.

Table 13: Preliminary Design Estimates for Las Vegas Trickling Filter

National Research Council (NRC) Design Method for Trickling Filters with Rock Media	
Parameter	Value
Average Flow rate Q_{avg} (Mgal/day)	0.5
Recirculation Flow Q_R (Mgal/day)	0.40
Pumping Wattage (watts)	747.6
Pumping Costs (lempira/day)	89.7
BOD Influent (mg/L)	175
BOD unit loading rate to filter W (lb/d•1000ft^3)	60.8
Volume of filter (1000 ft^3)	12.0
Depth of Filter Media (ft)	7.0
Surface Area of Filter (ft^2)	1714.3
Diameter of Filter (ft)	46.7
Recirculation Ratio r	0.8
Recirculation Factor F	1.5
BOD Removal Efficiency (%) (US customary units)	74%
BOD Effluent (mg/L)	46
Hydraulic Loading Rate (SOR) (gal/ft^2•min)	0.36
Hydraulic Loading Rate (SOR) (gal/ft^2•day)	525
Filter Classification (low, intermediate, or high)	High Rate

[31] Reinforced concrete is recommended for foundation and structure, and rock for the filter media.

Utilizing a High-Rate trickling filter scheme, the preliminary design is capable of achieving approximately 75% removal efficiency of BOD, producing an effluent within regulatory guidelines. This is a conservative value; proper maintenance of the Imhoff tanks will increase the removal efficiency of primary system and decrease secondary loadings. The use of a high-rate system greatly reduces the levels of filter flies which for the climate of Honduras would be highly problematic (Metcalf and Eddy, 1991). However, the use of a high rate system carries with it continuous sloughing of slime layer materials from the filter media, approximately 40 kg per day of this sludge[32]. A slope within the base of the filter toward the effluent discharge will encourage water transport and carry out this deleterious material. The development of an appropriate measure to dispose of these solids from the wastewater is recommended. Appropriate small scale secondary clarification techniques exist for removal of these materials. Discharging them with secondary effluent would be detrimental to Raices Creek and Lake Yojoa. Recommendation for such a system follows the outline of the proposed trickling filter design.

The footprint of the trickling filter would be approximately 210 m^2 (2,300 ft^2) with a diameter of approximately 17 m (55 ft) when wall thicknesses are estimated. Operating costs (electricity) for the pumping of recirculation water with this system are estimated to be less than 3,000 Lempira per month (approximately $150). Adequate recirculation flows should provide the necessary momentum to allow the distributor arms to rotate without motors. To facilitate this, an adjustable counter weight should be provided to change rotational inertia[33]. With recirculation pumping being the only major operational expense outside of paying for plant monitoring attendants, the trickling filter seems like a viable economic solution to increase the wastewater treatment efficiency for the municipality[34]. It must again be noted that this is a preliminary estimate utilizing the input parameters outlined previously in Table 13 and that final design should be based upon an extensive study into the wastewater needs specific to location. Schematics of both plan and elevation views are provided in Figures 32 and 33 respectively.

Secondary clarification to remove sludge from the effluent of the trickling filter could be accomplished utilizing a rectangular sedimentation tank with a travelling bridge collector. This system would allow for secondary sludge to both settle to the bottom of the tank and float to the surface. Sludge would be transported toward the influent end by the bridge collector which would be fashioned with two blades, one to skim the water surface and another to scrape the bottom. An Archimedes screw contained within a sump at the influent end of the tank would provide for removal of settled sludge and surface collected sludge could simply be skimmed from the tank with nets. This system could be operated mechanically or manually with gears actuated by motors or levers respectively. Sludge from this system could be spread out for drying and onsite burial or used as a soil amendment if sludge quality is high enough. Further study into the settling characteristics of sludge from the trickling filter would be required to accurately design the secondary clarifier and make recommendations for maintenance procedures.

[32] See Appendix IV for sludge generation calculations with trickling filter.
[33] Final design of the distribution arms should verify this through calculation and testing.
[34] Estimates have been made with regard to electricity costs within Honduras, further study is warranted.

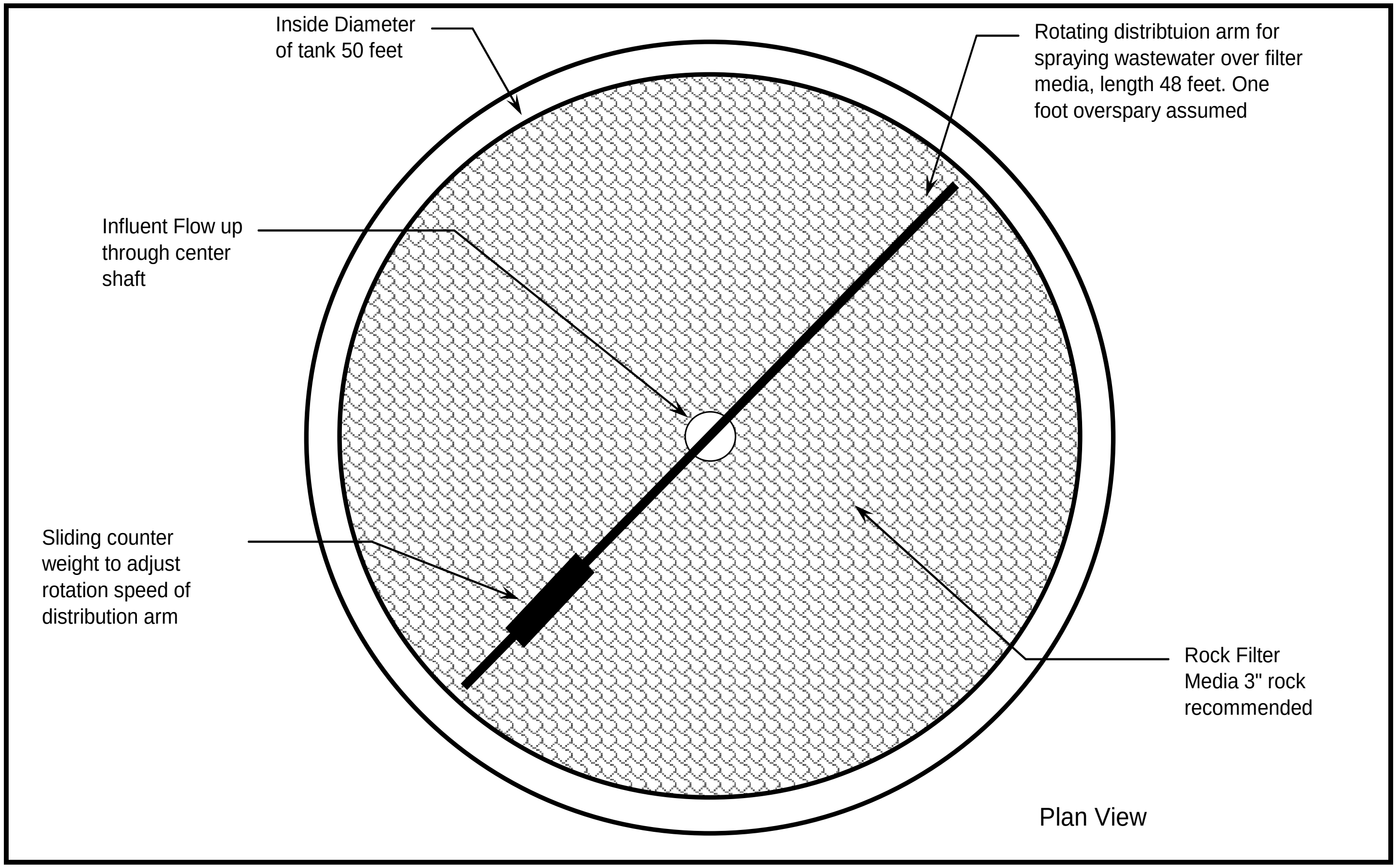

Figure 32: Plan View of Preliminary Design for Las Vegas Trickling Filter

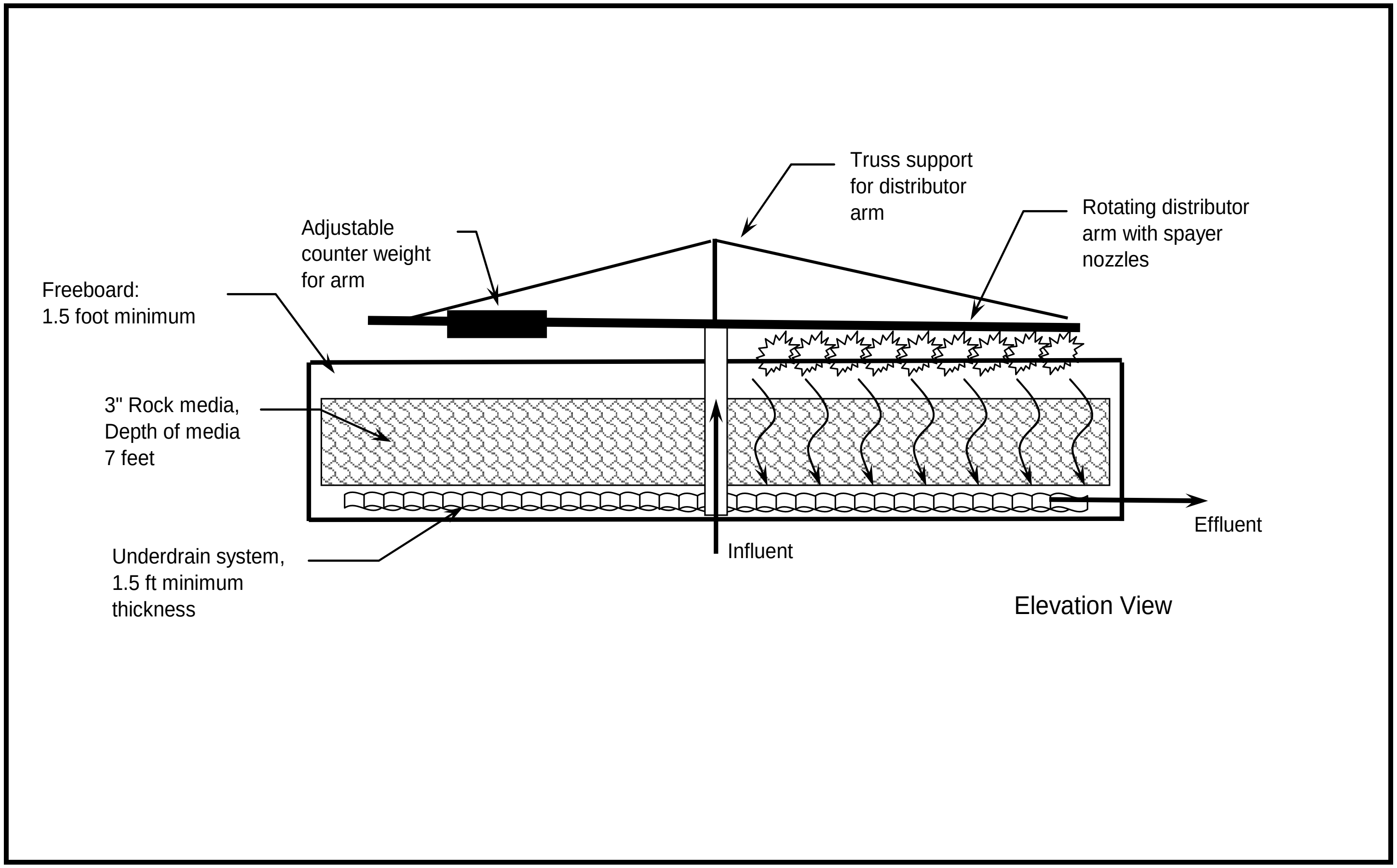

Figure 33: Typical Cross Section for Preliminary Design of Las Vegas Trickling Filter

7.2.2 Constructed Wetland Systems

Wetland environments provide methods for cleaning wastewater through natural attenuation processes if sufficient dilution of waste is provided. The levels of dilution are often the dividing line between a polluted environment and a thriving eco-system. Wastewater loading from a sewerage collection system is often too great to be remediated through natural wetland attenuation processes. The purpose of engineering or constructing wetlands is to create an environment that can provide proper treatment for these loadings. Numerous case studies have shown that constructed wetland systems can successfully treat a varied assortment of wastes including secondary treatment of municipal sewerage, acid waste sites, and heavy metals contamination (Lorion, 2001).

These systems require virtually no maintenance other than monitoring of flow patterns, water quality testing, and occasional harvesting of plant matter from the wetland system. Low maintenance is ideal for decentralized wastewater treatment since these systems are often located where skilled operators are cost prohibitive. As such, they may provide a solution for use in Las Vegas as an alternate secondary treatment system with the Imhoff tanks. However, as with other design alternatives, constructed wetlands do carry with them certain requirements which limit their applicability.

Typically these systems require large enough land area to allow for sufficient plant, bacterial, and animal growth to accommodate the wastes without overloading. As such, they may not be appropriate for use within certain geographic regions if large tracts of available land do not exist or terrain is to undulating. Two major categories of constructed wetlands will be considered and preliminarily sized for use in Las Vegas: the free water surface (FWS) and subsurface flow (SF) types. Attention will be given to sizing as a priority since the region is largely mountainous and site specificity is a priority (Lorion, 2001). Table 14 provides a summary of the treatment mechanisms that exist within constructed wetland systems.

Table 14: Treatment Mechanisms that Exist Within Constructed Wetland Systems

Constituent	Free water system	Subsurface flow
Biodegradable organics	Bioconversion by aerobic, facultative, and anaerobic bacteria on plant and debris surfaces of soluble BOD, adsorption, filtration, and sedimentation of particulate BOD	Bioconversion by facultative and anaerobic bacteria on plant and debris surfaces
Suspended Solids	Sedimentation, filtration	Filtration, sedimentation
Nitrogen	Nitrification/denitrification, plant uptake, volatilization	Nitrification/denitrification, plant uptake, volatilization
Phosphorus	Sedimentation, plant uptake	Filtration, sedimentation, plant uptake
Heavy metals	Adsorption of plant and debris surfaces, sedimentation	Adsorption of plant roots and debris surfaces, sedimentation
Trace organics	Volatilization, adsorption, biodegradation	Adsorption, biodegradation
Pathogens	Natural decay, predation, UV irradiation, sedimentation, excretion of antibiotics from roots of plants	Natural decay, predation, sedimentation, excretion of antibiotics from roots of plants

Adapted from: *Small and Decentralized Wastewater Management Systems* (Crites, 1998)

Free Water Surface Wetlands

Free water surface (FWS) wetlands consist of shallow flooded regions (approximately 10 – 45 centimeters deep) which contain vegetation rooted to the bottom of the flood zone. Vegetation for these systems typically consists of cattails, reeds, sedges, taro, and rushes, several of which grow within Honduras (Lorion, 2001). This vegetation uptakes contaminants directly, and provide growth media for microorganisms and animals which consume organic contaminants (Figure 34). Typically impermeable barriers prevent seepage into the underlying strata (Shanahan, 2009). However, some systems exist that utilize seepage and evapotranspiration as an outlet measure for treated effluents rather than direct collection and discharge (Crites, 1998).

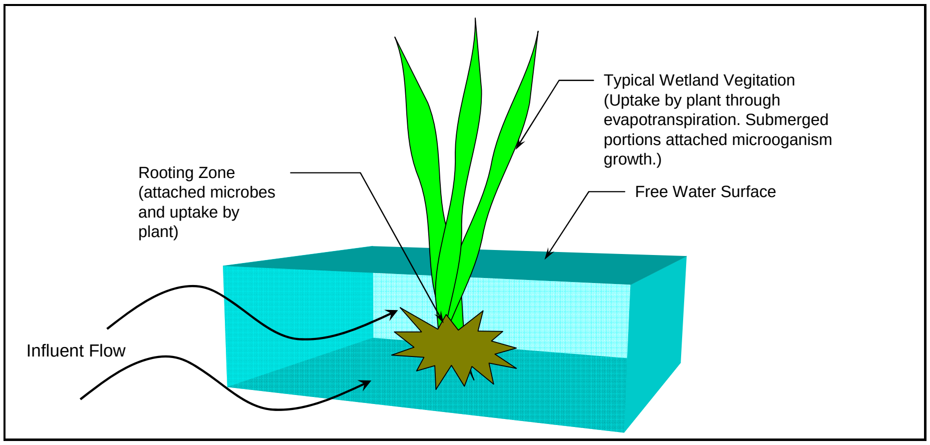

Figure 34 : Elemental Volume of Free Water Surface Wetland

The large surface area and exposed water surface for these systems require that care be given to maintain detention times adequate even during rainfall events – something which for Las Vegas may be difficult due to direct rainfall and groundwater infiltration into the sewerage network. The flows which result due to these events can greatly reduce detention times by opening short circuit pathways for the volume of water requiring mobilization. Often non-ideal flow rather than plug flow is observed for these systems under normal loading, due to the natural variability of plant growth patterns and stagnation points created by regions of entrapped solids.

Modeling techniques are often utilized to approximate actual detention times and flow patterns. These complications greatly increase the uncertainty inherent in the design of constructed wetlands and increase the need for redundancy. However with careful planning in design phases and attention to site specificity, FWS wetlands can potentially provide great cost savings over conventional treatment methods (Lorion, 2001).

The sizing of FWS wetlands requires that the following parameters be quantified: detention time for desired BOD_5 removal efficiency[35], organic loading rate, required surface area, water balance, and aspect ratio. Appendix V contains examples of the formulae and calculations for these parameters utilized in drafting a design FWS wetland system for Las Vegas. Table 15 summarizes typical design criteria and expected effluent qualities for FWS wetlands.

Table 15: Useful Parameters in FWS Wetland Design

Item	Unit	Value
Design parameter		
Detention time	day	2--5 (BOD)
		7--14 (N)
BOD loading rate	lb/ac•day	<100
Water depth	ft	0.2--1.5
Minimum size	ac/Mgal•day	5--10
Aspect ratio		2:1 to 4:1
Mosquito control		Required
Harvesting interval	yr	3--5
Expected effluent quality		
BOD_5	mg/L	<20
TSS	mg/L	<20
TN	mg/L	<10
TP	mg/L	<5
Adapted from (Crites, 2008)		

[35] Detention times for Nitrogen removal can be used as an alternative.

Sizing of Free Water Surface Constructed Wetland in Las Vegas

The preliminary sizing of a free water surface constructed wetland system for the municipality of Las Vegas has been conducted. The input parameters (BOD, flow rates, TSS) utilize averages of values obtained during the MIT study trips in both 2008 and 2009[36]. These values are limited and detailed studies should be conducted to document them over longer time intervals. Table 16 provides input values utilized in design, expected removal efficiencies, and required sizing needs. The most critical of parameters in this preliminary analysis is the surface area sizing requirement for the wetland system, approximately 1.7 hectares (4.0 acres) when calculated for detention times. However this sizing exceeds the recommended loading rates of 110 kg BOD per hectare per day (100 pounds of BOD per acre per day). Therefore, an adjusted total area of 3.9 hectares (9.6 acres) is required to provide a conservative 80% BOD removal.

Table 16: Preliminary Design Las Vegas Free Water Surface Wetland

Free Water Surface Constructed Wetland Design Calculator	
Parameter	Value
Flow Into FWS Q_{in} (Mgal/day)	0.5
Flow Out of FWS Q_{in} (Mgal/day)	0.55
Average Flow Q_{avg} (Mgal/day)	0.53
Calculated Average Flow Q_{avg} (ft^3/day)	7.0E+04
Effluent BOD Desired (mg/L)	50.00
Influent BOD (mg/L)	220.00
BOD removal constant $k_{apparent}$ (1/day)	0.68
Detention Time t (days)	2.19
Plant Based Void Ratio η	0.70
Depth of Flow d_w (ft)	1.25
Organic Loading Rate L_{org} (lb BOD/ac•day)	239.31
Required Surface Area BOD Removal (acres)	4.0
BOD Loading (lb/ac•day)	239.3
Area to Reduce *BOD Loading <100* (acres)	9.6
Aspect Ratio (length/width)	4
Width of FWS (ft)	209
Length of FWS (ft)	837
Influent TSS (mg/L)	150
Wastewater hydraulic loading rate (in/day)	4.8
Effluent TSS (mg/L)	18
Headloss through FWS (ft/ft)	2.2E-05

The indicated pond area is nearly 4 hectares. Unfortunately, the municipality does not own sufficient land adjacent to the Imhoff tanks to provide for this spacing[37]. Additional land of this acreage might be obtainable, but would require extensive earthwork to make it level. In addition, shoring up adjacent excavations would be required to prevent slope failures from inundating the

[36] See Chapter 5 water quality assessment (BOD approximated as 0.6 times COD).
[37] See Chapter 5 physical site survey, approximately 3 hectares (60 percent usable).

wetland system with sediments. While technology is available within Honduras to accommodate both of these tasks, land acquisition, excavation earthwork, and soil stabilization are likely cost prohibitive expeditions in comparison to other alternative treatments available.

Subsurface-Flow Constructed Wetlands

Subsurface-Flow (SF) wetlands are characterized by a lack of a free water surface. Instead, flow is through media located below grade (typically rock, gravel, and sand) that make up the root zones of surficial plantings (similar species to FWS wetlands). The surficial plantings in turn bring oxygen to the root zone for biological processes and root decay which contributes to denitrification (Crites, 1998). Figure 35 provides a cross sectional schematic of a SF wetland and its features.

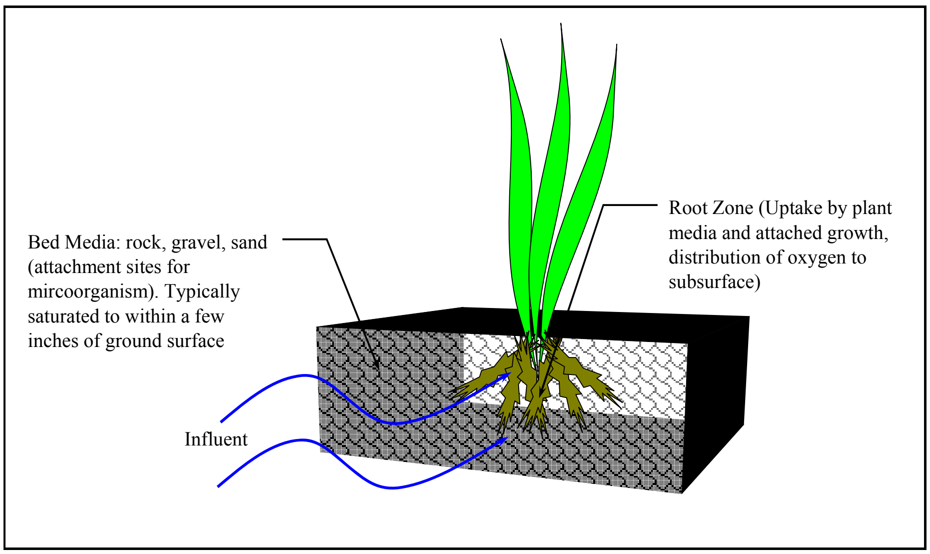

Figure 35: Elemental Volume for Subsurface-Flow Wetland

These systems typically provide contact sites with greater surface to volume ratios (roots and soils media) than FWS wetlands. Increases in surface contact area provide for greater sites upon which microorganism growth and consumption of organic constituents within primary effluent can occur. This generally allows for smaller tracts of land to be required for these systems. Underground treatment also provides advantages through reducing disease spreading vectors such as mosquitoes, aiding in odor control, and reduction of human contact exposure potential (Reed, 1995).

Acting somewhat as a filter media, these systems carry the danger of becoming clogged with solids; no effective technology has come forward which is available for backwashing. As such, design life for a typically loaded system is governed by its potential clogging time (approximately 5 – 7 years), after which the system must be excavated, replaced, and reestablished, with additional earthwork and replanting requirements. However, the time between wetland replacements may be extended by the previously outlined forms of pre-treatment and primary settling coupled with careful selection of subsurface flow media.

The sizing of typical SF wetlands utilizes parameters similar to their FWS counterparts with the introduction of two additional terms, the depth of media and TSS entry loading rate. The first is the vertical depth of rock, gravel, and sand media which wastewater will flow through. The second is the quantity of TSS expressed in pounds per day per square foot which will entry through the cross sectional plane of the wetland face. Reducing this term is critical to prevent clogging and achieve successful longevity for the SF wetland. Similarly the BOD per acre per day loading specified for FWS wetlands is also applicable for SF wetland systems. Both of these parameters ensure proper efficacy in treating wastewater for these systems (Reed, 1995). Table 17 summarizes the parameters useful in design of SF wetlands.

Table 17: Useful Parameters for SF Wetland Design

Item	Unit	Value
Design parameter		
Detention time	day	3--5 (BOD)
		6--10 (N)
BOD loading rate	lb/ac•day	<100
TSS entry loading rate	lb/ft^2•day	0.008
Water depth	ft	1--2
Medium depth	ft	1.5--2.5
Mosquito control		Not needed
Harvesting interval		Not needed
Expected effluent quality		
BOD_5	mg/L	<20
TSS	mg/L	<20
TN	mg/L	<10
TP	mg/L	<5
Adapted from (Crites, 1998)		

Sizing of Subsurface-Flow Constructed Wetland in Las Vegas

Over arching objectives similar to FWS wetlands were used in analyzing the appropriateness of SF wetland technology for use in Las Vegas; these were the observed BOD loading to the system, expected effluent water quality, and observed flow measurements. This similarity ensured proper comparison of treatment efficacy across the two systems. Table 18 provides the results of the preliminary sizing design; Appendix V provides the assumptions and sample calculations utilized in sizing the SF wetland system for Las Vegas.

Table 18: Preliminary Design Las Vegas Subsurface-Flow Wetland

Submerged-Flow Constructed Wetlands Design Calculator	
Parameter	Value
Average Flow Q_{avg} (Mgal/day)	0.55
Effluent BOD (mg/L)	50.00
Influent BOD (mg/L)	220.00
BOD removal constant $k_{apparent}$ (1/day)	1.10
Detention Time t (days)	1.35
Effective Porosity η	0.30
Depth of Medium d_m (ft)	2.50
Depth of Fluid in Bed d_w (ft)	2.00
Area Required for BOD removal A_s (ac.)	3.79
BOD Loading (lb/ac•day)	266.23
Area to Reduced *BOD Loading <100* (acres)	10.09
Width of SF wetland w (ft)	200.00
TSS mass loading (lb/day)	100.00
TSS entry zone loadings (lb/ft^2•day)	0.20
Conversion Factor F ft^2 to acres	43560.00
Hydraulic Conductivity k (ft/day)	100.00
Headloss through SF (slope expressed as decimal)	1.4E-05
Calculated Length of SF for BOD removal (ft)	825.56
Calculated Aspect Ratio SF for BOD removal	4.13

In this preliminary design, the system is over 4 hectares (10 acres), which is higher than its FWS counterpart. Sizing was again increased due to the BOD loading rate adjustment; however, the reason for larger size than FWS is attributable to the effective porosity of soil media which is approximately half that of FWS systems. This SF wetland system is not recommended for use in Las Vegas since the costs associated with acquisition of adjacent lands, earthwork, and slope stability could greatly increase the cost of this system over other proposed technologies. Further, the additional costs associated with excavation and placement of fill in initial construction, potentially needing to excavate and replace a subsurface flow wetland when it becomes clogged, and the need for greater acreage makes this the least cost effective solution for use in Las Vegas.

7.3 Nuisance Odor Reduction Technologies

Residents downwind from the Imhoff tank system in Las Vegas have expressed concern over the nuisance odors associated with the treatment operation (Garcia, 2009). A constant flow of odorous gases, typically hydrogen sulfide and ammonia, escapes from the sludge digestion chambers through the gas venting system. The warm tropical climate of Honduras which is good for the digestion of sludge increases the presence of these gases. This problem is not specific to Las Vegas, it is probably the largest complaint associated with any wastewater treatment facility!

Hydrogen sulfide is denser than air and ammonia forms denser than air vapors in the presence of moisture; these gases do not simply vent off to the atmosphere. Large centralized treatment plants utilize blowers to encapsulate odorous gases within confined spaces. These gases are then piped off to air scrubbers which may use any of numerous technologies to remove the culprit gases from the air (Metcalf and Eddy, 2003). An elaborate system of blowers and air scrubbers is impractical for use with the small decentralized system found in Las Vegas. Instead a passive form of odor containment, transport, and air scrubbing would be appropriate for use with this system.

The tropical climate and abundant sun exposure offer opportunities for creating such a passive air collection and scrubbing system. Specific to the Imhoff tank site at Las Vegas, an abandoned cistern is located approximately 6 meters (20 vertical feet) up the adjacent hill. Providing approximately 14 cubic meters (500 cubic feet) of volume, the cistern can be converted into a location for air scrubbing. Encapsulation of the odorous gases can be accomplished through the use of lightweight corrugated metal sheeting, to construct a low-rise superstructure upon the Imhoff tanks. This corrugated metal sheeting is a common material found throughout Honduras, and numerous sheet metal workers exist for sizing and welding of materials.

A passive means of lifting these gases which are denser than air (i.e., at 20ºC: air 1.20 grams per liter, hydrogen sulfide 1.363 grams per liter, ammonia 1.23 grams per liter wet) can be accomplished through heating the encapsulated air and allowing it to flow up a riser stack pipe to the air scrubber. The source of heat for this system would come from the abundant sunshine; painting the corrugated sheet metal black can increase the effect. Additional sources for buoyancy could come from V~~v~~enturi effects due to wind blowing across the top of the cistern or a small compressor to lift the gasses. Inlet vents located around the base of the superstructure would allow for fresh air to flow in as odorous air is transported out the stack to the scrubber. It is important that the superstructure be of reduced height to ensure capture of dense gases. Access points must be provided for necessary maintenance and inspection of the Imhoff tank. A system of hinged access doors or the overlapping of removable sheets on framing beneath can provide this access. The piping for transporting of venting gases should be as centralized as possible.

Pilot scale studies have shown that biological processes which occur within compost piles can be utilized in air scrubbers for removal of these odorous gases. Removals of 99% for hydrogen sulfide gas and 80 % for ammonia gas were achieved when compost methods were tested for use with a municipal wastewater facility[38]. A pilot system was attempted by SANAA for another facility within Honduras; unfortunately this system had several design flaws that created leaks and prevented efficient operation. Compostable materials such as banana, orange, and other fruit peels are readily available within the region and the tropical climate of Honduras does provide ideal conditions for their use. The inclusion of ash can provide additional carbon sites upon which these gases will be removed. It is important to introduce the gases below the composting material for adequate flow through the media. In addition, an underdrain and port with access low enough to provide for collection of any accumulated fluids within the scrubber should be provided. Figure 36 provides a conceptual schematic for the proposed air collection and scrubber system.

[38] (Chen, 2004)

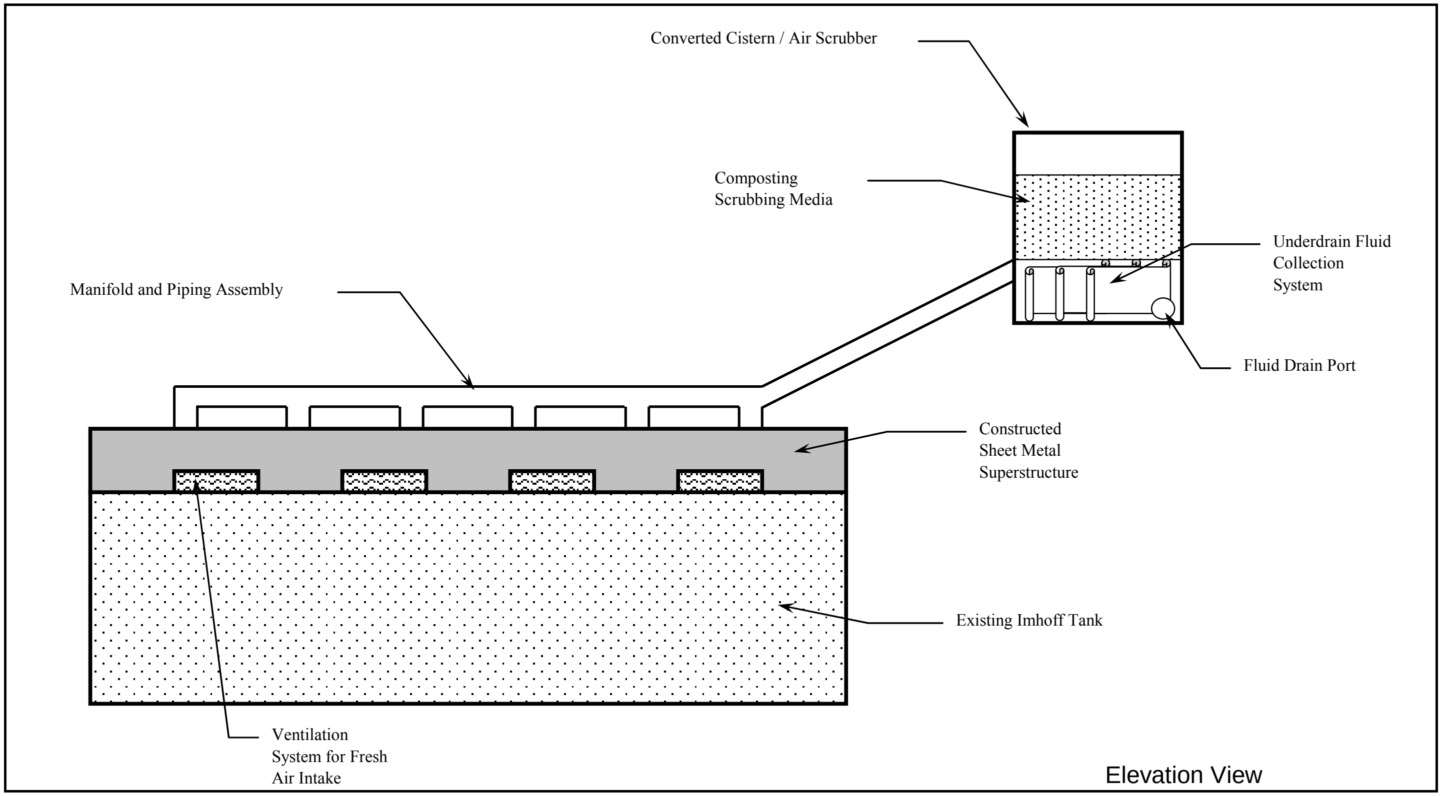

Figure 36: Conceptual Elevation View of Odorous Gas Collection System and Air Scrubber

Chapter 8: Conclusions and Recommendations

The present state of water and sanitation services across much of Honduras and Central America is at a cross roads and in need of improvement. Numerous systems have been implemented that are not necessarily appropriate for the communities which they are meant to serve. Designs which have been permitted by the Honduran wastewater sector have been found: ill-suited to handle the volumetric flows of wastewater, too costly for ongoing operation to continue, or not appropriate for the site chosen. Wastewater treatment recommendation and implementation has been often initiated by NGOs with good intentions but lacking awareness of the outcome of their efforts when they have departed from a region. Most of the communities they seek to aid lack the resources to accept the final responsibility for these systems.

The myriad of recent changes within the water and sanitation sector of Honduras have yet to be worked through fully. The sector is aware of the need for change and seems poised to implement a feasible strategy to improve water and sanitation services throughout the country, what is needed now is further input appropriate to Honduras. During a meeting with agencies from the water and sanitation sector, Pedro Ortiz, the Technical Director of SANAA, stated that Hondurans are studying treatment technologies at universities all across the globe and trying to bring what they have learned back to Honduras. Mr. Ortiz further added that not all of these technologies are necessarily appropriate for Honduras for various reasons, the largest being cost of bringing the technology to Honduras and ongoing operation and maintenance expenditures. Acutely obvious is a need for the various agencies to fully understand and disseminate knowledge about the operational and maintenance procedures for the systems which currently exist within the country. Without mastery and understanding of these fundamental concepts new systems are likely to experience failure as well.

This present work has sought to examine technologies which may be appropriate for use in enhancing treatment of wastewater within Honduras without sacrificing existing technologies. Nearly forty percent of the country's wastewater infrastructure is comprised of Imhoff tank systems. Although these systems do not provide final effluent quality which meets regulatory guidelines, they are very appropriate for use as primary treatment processes in more advanced systems. The Imhoff tanks of Las Vegas serve as both a representative example of this type of technology and a platform for demonstrating ways of improving upon these systems.

8.1 Technical Recommendations

Prior to incorporating secondary treatment technologies with Imhoff tanks, these systems should be brought up to a uniform standard that provides optimum performance of the Imhoff tank systems themselves. Initial factors within the scope of an improvement plan would include the following: initiate proper operation and maintenance procedures for the system with record keeping, begin installation of appropriate pre-treatment technologies for Imhoff tanks which lack these systems, and optimize flow distribution through the Imhoff tanks to provide ideal detention times for removal of settling particles and sludge digestion. These actions will go along way to improve both the performance of the Imhoff tanks and usable service life.

Once these systems have been brought up to an optimized condition and proper operation and maintenance has been implemented work can begin on the next phase of an improvement scope, implementing technologies which have been tested to improve final effluent qualities when coupled to the Imhoff tank system. The appropriateness of a technology will need to have been demonstrated through the use of bench and pilot scale testing which took into account long period water quality monitoring and flow measurement. Without this level effort the systems are likely to not be appropriate to meet the demands of treating a community's wastewater.

Based upon the examination of chemically enhanced primary treatment within Las Vegas it seems doubtful that this technology could prove viable as a means of economically improving Imhoff tank performance over a long period. The transportation and supply costs associated with using either ferric chloride or aluminum sulfate are too high. Estimates of these costs can be obtained by examining required dosages in the Las Vegas study and the associated chemical costs. These have been summarized in Table 19. Ferric chloride bought in bulk is estimated at 34 Lempira per kilogram (approximately $1.70 USD per kilogram) and would amount to annual expenses over 1,500,000 Lempira per year (approximately $76,000 per year); this figure does not including wages paid to plant operators. Values for using aluminum sulfate are only 5 percent less. These are astronomical values for the community and the municipality does not generate this revenue from connection and service fees. The calculated annual costs associated with these chemicals would decrease if locally available sources became available.

Comment [EA15]: Center table caption

Table 19: Daily Costs Associated with CEPT Use in Las Vegas

Chemical Substance	Unit Cost (Lempira/kg)	Dosage Required to Meet Effluent Guidelines (mg/L)	Assumed Flow Rate (m3/day)	Daily Cost Chemical (Lempira/Day)	Transportation and Storage Estimate (Lempira/Day)	Total Daily Cost (Lempira/Day)	Total Daily Cost ($USD/Day)
Ferric Chloride	34	100	1060	3604	541	4145	207
Aluminum Sulfate	25	130	1060	3445	517	3962	198

In contrast to concepts which enhance the removal efficiency of the Imhoff tanks themselves is the idea of partnering existing Imhoff tanks with secondary treatment technologies. This strategy compartmentalizes treatment and allows for individual plant processes to be utilized in tandem with one another to improve final effluent. Specific to the Las Vegas study two types of sustainable secondary treatment systems were examined. These were the trickling filters and constructed wetland systems. Both of these systems provide secondary treatment utilizing very low operational and maintenance budgets. Unfortunately in the case of Las Vegas land availability does not allow for the use of constructed wetland systems. However, trickling filters have proven amenable technology for use with the system and operating budgets after initial capital investment are very low (approximately 32,000 Lempira per year). Utilized with a properly maintained and configured Imhoff tank system, a trickling filter system would allow the community of Las Vegas to achieve the regulatory effluent guidelines for Honduras. It must again be emphasized that proper system maintenance is critical to maintaining the expected effluent qualities. Without proper maintenance no system functions properly!

8.2 Areas of Further Research

The community of Las Vegas is appreciative of the research efforts afforded its wastewater treatment situation. Specific to developing further recommendation to improve treatment within this region, extensive data is needed to accurately obtain flow and water quality measurements over a longer study period. This may be accomplished through a rotating team of graduate students or a small group which stays for a long duration. It is recommended that both rainy season and dry season data be collected to assess the effects of these cycles on wastewater quality and quantity.

It is also recommended that a system of best practices in collecting this information be developed and shared with the agencies that make up the wastewater sector of Honduras. They in turn should be encouraged to disseminate this knowledge throughout the various municipalities. This will be critical to meeting the objectives of the Law of Municipalities which turns over control of wastewater treatment to individual municipal Juntas. Documentation of this type of information is also critical to being able to provide improved water and sanitation throughout Honduras.

References:

Adams, E. Eric (2008). Laboratory training exercises for testing coagulants. Massachusetts Institute of Technology, Department of Civil and Environmental Engineering.

Babbitt, Harold E (1922). *Sewerage and Sewage Treatment*, New York: John Wiley & Sons, Inc.

Barnes, D. and Wilson, F. (1976). *The Design and Operation of Small Sewage Works*, New York: John Wiley & Sons, Inc.

Bhattacharya, M et al (2009) *Evaluation Wastewater Treatment Options for Honduras*, unpublished work completed 2009.

Camp, T. R. (1942) Grit Chamber Design, *Sewage Works Journal*, Vol. 14, p. 368.

Chagnon, F. and Harleman, D. (2004). *An Introduction to Chemically Enhanced Primary Treatment*, Department of Civil and Environmental Engineering Massachusetts Institute of Technology.

Chen, YX et al (2004) *Biological removal of air loaded with a hydrogen sulfide and ammonia mixture*. Journal of Environmental Sciences (China), published in Netherlands. 2004-; vol 16 (issue 4) : pp 656-61

Crites, R. and Tchobanoglous, G. (1998). *Small and Decentralized Wastewater Management Systems*, New York: McGraw-Hill, Inc.

FCO (2008). United Kingdom Foreign and Commonwealth Office (FCO), "North &Central America and Caribbean: Honduras", 2008, http://www.fco.gov.uk/en/about-the-fco/country-profiles/north-central-america/honduras?profile=economy&pg=2

Garcia, E. (2009). Personal communication with Eber Garcia resident of Las Vegas, Honduras January 16, 2009.

Herrera, A. (2006). *Rehabilitation of the Imhoff Tank Treatment Plant in Las Vegas, Santa Bárbara Honduras, Central América*. MS thesis, Dept of Civil, Architectural and Environmental Engineering, U. Texas, Austin, 2006.

Hodge, M. (2008). *Wastewater Treatment in Las Vegas, Santa Barbara, Honduras*, MEng Thesis, Dept of Civil and Environmental Engineering, Massachusetts Institute of Technology, Cambridge, MA 2008.

Kawamura, S. (2000). *Integrated Design and Operation of Water Treatment Facilities*, 2nd ed., New York: John Wiley & Sons, Inc.

Lee, C. C. (2007). *Handbook of Environmental Engineering Calculations*, 2^{nd} ed., New York: McGraw-Hill, Inc.

Lorion, R. (2001). *Constructed Wetlands: Passive Systems for Wastewater Treatment*, Technology Status Report prepared for the US EPA Technology Innovation Office.

Metcalf and Eddy, Inc (1979). *Wastewater Engineering Treatment, Disposal, and Reuse,* New York: McGraw-Hill, Inc.

Metcalf and Eddy, Inc (1991). *Wastewater Engineering Treatment, Disposal, and Reuse*, 3^{rd} ed., New York: McGraw-Hill, Inc.

Metcalf and Eddy, Inc (2003). *Wastewater Engineering Treatment and Reuse*, 4^{th} ed., New York: McGraw-Hill, Inc.

Metcalf, L. and Eddy, H. (1935). Imhoff tanks. *American Sewerage Practices*, New York: McGraw-Hill, Inc.

Mikelonis, A. (2008). *Chemically Enhance Primary Treatment of Wastewater in Honduran Imhoff Tanks*. MEng Thesis, Dept of Civil and Environmental Engineering, Massachusetts Institute of Technology, Cambridge, MA 2008

NRC (National Research Council) (1946) Trickling Filters in Sewage Treatment at Military Installations, *Sewage Works Journal,* Vol. 18, No. 5.

Ortiz, P. (2009). Communication between Honduran Wastewater Sector Agencies and members of the Massachusetts Institute of Technology Research Team during collaboration meeting headed by Pedro Ortiz, Technical Director of SANAA.

Parker, D. et al (2001). *The Future of Chemically Enhanced Primary Treatment: Evolution Not Revolution*, accessed November 18, 2008 www.cd3wd.com/CD3WD_40/ASDB_SMARTSAN/CEPT-Debate-2.pdf

Phipps & Bird (2009). G-Curves. Retrieved April, 2009 from: (http://www.phippsbird.com/gcurve.html)

RAS-HON (2008). RAS-HON website, "AHJASA", accessed 11/18/08, http://www.rashon.org.hn/web_ahjasa/index.html

Reed, S. C., R. W. Crites, and E. J. Middlebrooks (1995) *Natural Systems for Waste Management and Treatment,* 2^{nd} ed.. McGraw-Hill, New York.

SERNA (2009). SERNA website, accessed 3/30/09 http://www.serna.gob.hn/comunidad/historia/Paginas/default.aspx

Shanahan, P. (2009). Lecture notes from Waste Containment and Remediation course within the Department of Civil and Environmental Engineering, Massachusetts Institute of Technology April, 2009.

Tyler, C. (2009). Personal communication with Charles Tyler of Massachusetts Water Resources Authority while collecting samples from Deer Island Facility March 20, 2009.

U.S. CIA (2008). United States Central Intelligence Agency (US CIA), “Honduras”, 2008, https://www.cia.gov/library/publications/the-world-factbook/geos/ho.html

Water for People (2006). “Water for People - Honduras Country Strategy.” Water for People International Programs Committee, 2006.

World Health Organization (WHO), “Honduras”, 2001, http://www.paho.org/english/sha/prflhon.htm

Appendix I: Flow Observation and Water Quality Data for Las Vegas

Flow Observation Data Las Vegas Imhoff Tanks

Collection of velocity and depth data was performed over a 24 hour period beginning January 13th, 2009 at 1:00 pm using the instrumentation outlined in Section 5.3 (velocity meter attached to stadia rod). This data was utilized along with inlet channel geometries to develop a flow rate for the facility. The data collected during this measurement period along with the flow calculations is contained in Table20. Example formulae for calculations have been provided below.

Table 20: Data Collected During Flow Monitoring Las Vegas January 2009

Flow Observed Jan 13 -14 Beginning 1pm Jan 13							
Hrs Since 1pm Jan 13th	Pipe Diameter (ft)	Depth to water surface Below Sofet (ft)	Velocity (m/s)	θ	Cross Sectional Area of Flow (ft^2)	Flow (m^3/day)	SOR (m/day)
0.00	1	0.80	1.23	1.85	0.11	1102.55	46.47
2.00	1	0.80	1.17	1.85	0.11	1050.57	44.28
4.00	1	0.80	1.23	1.85	0.11	1102.55	46.47
6.00	1	0.80	1.19	1.85	0.11	1069.72	45.09
8.00	1	0.81	1.18	1.80	0.10	986.29	41.57
17.00	1	0.82	1.10	1.75	0.10	846.72	35.69
18.00	1	0.79	1.29	1.90	0.12	1240.82	52.30
19.00	1	0.80	1.17	1.85	0.11	1050.57	44.28
21.00	1	0.80	1.19	1.85	0.11	1064.25	44.86
Surface Area							
square meters	square feet				Average	1057.12	44.56
23.72	510.00				Peak	1240.82	52.30

Calculation for cross sectional area of circular pipe not flowing full:

Angle subtended by top-width of water in pipe: ***θ*** (radians)

$$\theta = 2\cos^{-1}\left(1 - 2\left(d / D\right)\right)$$

d = Depth of water flowing in pipe (Length)
D = Diameter of circular pipe (Length)

Cross sectional area of water flowing in channel: ***A*** ($Length^2$)

$$A = D^{\frac{2}{8}}\left(\theta - \sin\left(\theta\right)\right)$$

θ = Angle subtended by top-width of water in pipe (radians)
D = Diameter of circular pipe (Length)

Calculation of flow rate for water flowing in pipe:

Volumetric flow rate of wastewater in pipe: ***Q*** ($Length^3$/Time)

$$Q = V_{avg} \times A$$

V_{avg} = Measured average velocity (Length/Time)
A = Cross sectional are of water flowing in channel ($Length^2$)

Calculation of surface overflow rate (SOR):

SOR rate at which flow crosses water surface: ***SOR*** ($Length^3$/Time•$Length^2$)

$$SOR = \frac{Q}{SurfaceAreaTanks}$$

Q = Volumetric flow rate of wastewater ($Length^3$/Time)
SAT = Surface area of water within tanks ($Length^2$)

Water Quality Testing Data Las Vegas Imhoff Tanks

Pertinent wastewater quality data for the Imhoff tanks in existing condition was collected over the course of this study beginning on January 14, 2009 and terminating on January 17, 2009. Specific to the study in Las Vegas the following parameters were tested to serve as benchmarks for comparison to CEPT bench testing and for use in design calculations:

Turbidity (NTU)	Table 21
COD (mg/L)	Table 22

Turbidity Observations

Turbidity is a measurement of scintillation and reflection of light due to the presence of suspended particles within water. Turbidity measurements were performed on influent and effluent samples obtained form the Las Vegas Imhoff tanks and are reflected in Nephelometric Turbidity Units. These samples consist of 30 separate influent and effluent pairs; which were arbitrarily assigned to one another (individual removals not indicative of actual). The purpose of these samples was to develop the average influent and effluent turbidity values for wastewater going to and leaving Las Vegas' Imhoff tanks. Testing for turbidity utilized Method 2130 which was approved by the EPA Standard Methods Committee, 1994. Reading of NTU values was accomplished utilizing a programmed code within a HACH photo-spectrometer DR/2500.

Table 21: Existing Las Vegas Imhoff Tank Turbidity Measurements

Turbidity Testing Las Vegas Imhoff Tanks January 2009					
Test #	Date	Time Sample Obtained	Influent Turb	Effluent Turb	Removal %
1	16-Jan-09	8:30 AM	120	94	22%
2	16-Jan-09	8:30 AM	113	92	19%
3	16-Jan-09	8:30 AM	123	102	17%
4	16-Jan-09	8:30 AM	88	73	17%
5	16-Jan-09	8:30 AM	112	89	21%
6	16-Jan-09	8:30 AM	131	110	16%
7	16-Jan-09	10:30 AM	152	129	15%
8	16-Jan-09	10:30 AM	158	132	16%
9	16-Jan-09	10:30 AM	149	125	16%
10	16-Jan-09	10:30 AM	147	121	18%
11	16-Jan-09	10:30 AM	159	128	19%
12	16-Jan-09	10:30 AM	174	137	21%
19	17-Jan-09	9:20 AM	106	89	16%
20	17-Jan-09	9:20 AM	99	76	23%
21	17-Jan-09	9:20 AM	113	91	19%
22	17-Jan-09	9:20 AM	110	90	18%
23	17-Jan-09	9:20 AM	111	88	21%
24	17-Jan-09	9:20 AM	106	89	16%
25	17-Jan-09	11:00 AM	129	106	18%
26	17-Jan-09	11:00 AM	139	111	20%
27	17-Jan-09	11:00 AM	126	99	21%
28	17-Jan-09	11:00 AM	109	91	17%
29	17-Jan-09	11:00 AM	117	97	17%
30	17-Jan-09	11:00 AM	110	88	20%
		Averages	125	102	18%

COD Removal

Testing for COD removal efficiencies for the Las Vegas Imhoff tanks were conducted using the Reactor Digestion Method (Method 8000) as defined by the United States Environmental Protection Agency. Split between the 16^{th} and 17^{th} of January 2009, a collection of 18 random pairs of influent and effluent samples were collected and tested. All collection pairings were separated by a detention time of 90 minutes as calculated with measured flow rates. The purpose of this sampling was to develop sufficient numbers of pairings to obtain a picture of the average removal efficiency for COD through the Las Vegas Imhoff tanks.

Table 22: Existing Las Vegas Imhoff Tank COD Removals

COD Testing Las Vegas Imhoff Tanks January 2009						
Test #	Test Type	Date	Time Sample Obtained	COD Influent (mg/l)	COD Effluent (mg/l)	COD Removal Efficiency
13	COD Only (from imhoff)	16-Jan-09	11:55 AM	466	394	15.5%
14	COD Only (from imhoff)	16-Jan-09	11:55 AM	442	391	11.5%
15	COD Only (from imhoff)	16-Jan-09	11:55 AM	441	371	15.9%
16	COD Only (from imhoff)	16-Jan-09	11:55 AM	443	377	14.9%
17	COD Only (from imhoff)	16-Jan-09	11:55 AM	427	380	11.0%
18	COD Only (from imhoff)	16-Jan-09	11:55 AM	442	367	17.0%
31	COD Only (from imhoff)	17-Jan-09	2:00 PM	268	254	5.2%
32	COD Only (from imhoff)	17-Jan-09	2:00 PM	245	245	0.0%
33	COD Only (from imhoff)	17-Jan-09	2:00 PM	258	226	12.4%
34	COD Only (from imhoff)	17-Jan-09	2:00 PM	276	276	0.0%
35	COD Only (from imhoff)	17-Jan-09	2:00 PM	242	253	0.0%
36	COD Only (from imhoff)	17-Jan-09	2:00 PM	246	242	1.6%
37	COD Only (from imhoff)	17-Jan-09	2:00 PM	262	223	14.9%
38	COD Only (from imhoff)	17-Jan-09	2:00 PM	251	255	0.0%
39	COD Only (from imhoff)	17-Jan-09	2:00 PM	293	278	5.1%
40	COD Only (from imhoff)	17-Jan-09	2:00 PM	251	247	1.6%
41	COD Only (from imhoff)	17-Jan-09	2:00 PM	260	248	4.6%
42	COD Only (from imhoff)	17-Jan-09	2:00 PM	245	237	3.3%
			Averages	320	292	7.5%

Appendix II: Chemical Information for Substances Used in CEPT Testing

Chemical Supply Vendors

Ferric Chloride, $FeCl_3$

Name of Company: Cole-Parmer
Telephone Number: 1-888-358-4717
URL for product page:
http://www.coleparmer.com/catalog/product_view.asp?sku=8820995&pfx=WU
Name of Chemical: Iron (III) chloride hexahydrate
Product Number: WU-88209-95
Product Name: Iron (III) chloride hexahydrate, pure, granulated 99%
Quantity: 2.5kg
Price Listed: 95.20 USD

Aluminum Sulfate, $Al_2(SO_4)_3$

Name of Company: VWR International
URL for product page:
http://www.vwrsp.com/catalog/product/index.cgi?catalog_number=EM-AX0745-2&inE=1&highlight=EM-AX0745-2&reference_type=0&partnumber=17927-65-0&sim_code=1.0
Name of Chemical: Aluminum Sulfate n-hydrate
CAS Number: 7784-31-8
Product Name: Cake Alum, 54.0 – 59.0% as anhydrous
Quantity: 500 mg
Price Listed: 38.95 USD

Material Safety Data Sheets (MSDS)

Material safety data sheets are available from the vendor websites for the chemicals utilized in the bench scale testing of chemically enhanced primary treatment, ferric chloride (Honduras) and aluminum sulfate (Honduras and MIT laboratories). These MSDS provide specific information about molecular weights, active ingredients, transporting, and chemical handling procedures for these specific chemicals.

Appendix III: Testing Data for CEPT Efficacy

Bench Scale Correlation between Phipps and Bird Jar Tester and Imhoff Tanks

Administration of coagulant dosing requires sufficient energies be utilized to ensure complete mixing of the chemical throughout the wastewater influent prior to arrival at the primary settling chamber. This can be accomplished through a number of ways. On the large scale, mixing can be accomplished within a flash mixer or jet mixer. The decentralized nature of Las Vegas' Imhoff tanks does not warrant expenditure on these methods. Ideal situations for mixing can also be accomplished by utilizing open channel flow over a sufficient distance; this allows for the head losses to impart a power input to the wastewater developing a characteristic velocity gradient (G) with units of sec$^{=1}$. This mixing energy distributed over the time travel interval in seconds (t) and forms a unitless factor known as G•t, a common indicator for mixing (Kawamura, 2000).

Calculations of Parameters in Mixing

Methodology developed from laboratory training exercise lecture at MIT (Adams, 2008)

N_R		= Reynolds Number (unitless)	$\left(\dfrac{Du\rho}{\mu}\right)$
	D	= Pipe Diameter	0.3 meters
	u	= Velocity	1.19 meters/second
	μ	= Dynamic Viscosity[39] H_2O	10^{-3} Newton•second/meter^2
	ρ	= Density of H_2O	998 kilograms/meter^3
N_R		**= 3.56 X 10^5**	*Turbulent Mixing*

H		= Headloss[40] (meters)	$f\left(\dfrac{L}{D}\right)\left(\dfrac{u^2}{2g}\right)$
	f	= friction factor[41] (unitless)	0.0025
	L	= Length[42] of travel within pipe	13 meters
	g	= gravitational constant	9.81 meters/second^2
H		**= 7.8 X 10^{-2} meters**	

[39] Water viscosity and density are temperature dependent variables. These values are for an assumed T = 20ºC
[40] This is the Darcy Weisbach form of the headloss equation. Other methods of calculating headloss exist.
[41] Typical values range between 0.002 and 0.003.
[42] Injection site taken correlates with sampling collection point, thought to be most favorable injection site.

P		= Power imparted to water (watts)	$\gamma \times Q \times H \times \frac{10^3\,Newton}{kiloNewton}$
	γ	= unit weight of water	9.81 kN/meter^3
	Q	= wastewater flow rate	0.012 meters^3/second
	H	= Headloss	7.8 X 10^{-2} meters
P		**= 9.2 watts**	

V		= Unit volume acted upon (meters^3)	$\frac{1}{u} \times L \times Q$
	u	= Average velocity of water	1.19 meters/second
	L	= Length of travel within pipe	13 meters
	Q	= wastewater flow rate	0.012 meters^3/second
V		**= 0.13 meters^3**	

G		= Velocity Gradient (sec^{-1})	$\left(\frac{P}{\mu V}\right)^{\frac{1}{2}}$
	P	= Power imparted to water	9.2 watts
	μ	= Dynamic viscosity of water	10^{-3} Newton•sec/meter^2
	V	= Volumetric prism	0.13 meters^3
	t	= Travel time L/u	10.9 seconds
G		= 270 sec^{-1}	
G X t		**= Representative Mixing Value**	**2950**

The value of G can be utilized to correlate bench scale with full scale facility operations. For the Phipps and Bird Jar Tester, bench scale can mimic full scale through the adjustment of two parameters, mixing time and angular rotation of mixing paddles. Figure 37 contains the manufacturer recommended curve relating revolutions per minute (RPM) of the motorized mixer with desired G values (Phipps, 2009). The mixing times are varied to develop the idealized GT value for the full scale system. The bench scale testing in all cases used a 100 RPM paddle speed for 30 seconds (Gt ~ 3000).

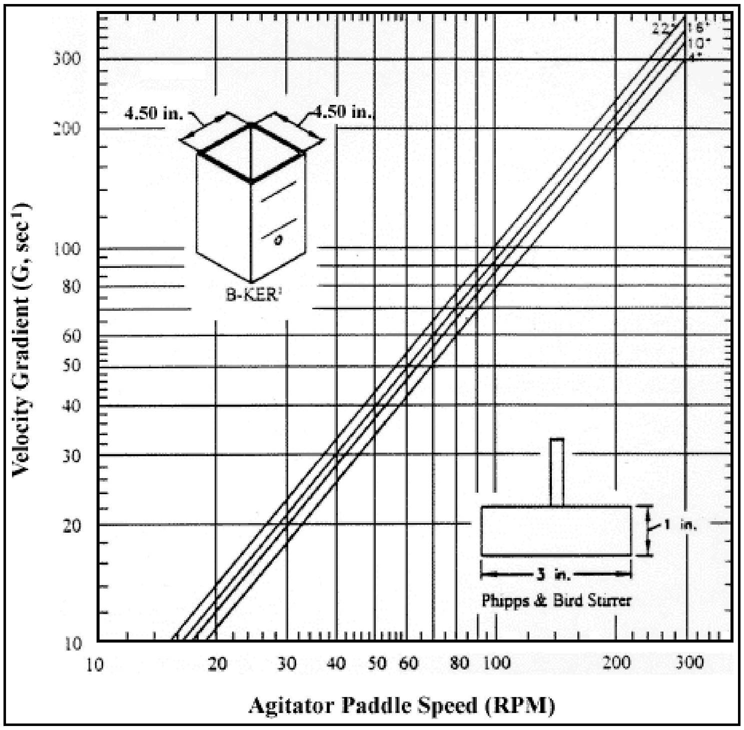

Figure 37: Phipps and Bird Velocity Gradient vs. RPM Curve for Mixing (Phipps, 2009)

CEPT Performance Bench Testing Efficacy Ferric Chloride in Las Vegas

The following data was collected from the efforts of bench scale testing ferric chloride efficacy when used in combination with the Imhoff tanks of Las Vegas. Turbidity and Chemical Oxygen Demand (COD) were assessed for comparison with the previously developed treatment efficiencies observed in the initial water quality study. Similar testing methods and equipment were utilized within this study. Dosages of coagulant were administered by pipette utilizing a solution of 10 grams (dry weight) of ferric chloride dissolved into 100 milliliters of tap water (available within the lab space). The Phipps and Bird test equipment was utilized with a mixing energy of 100 RPM for 30 seconds (GT value of approximately 3000). This was to simulate estimated GT value that would be achieved using the identified injection point.

Two separate surface overflow rates (SOR) were utilized in the examination of turbidity declines, the observed SOR of 52 meters per day, and an ideal SOR of 38 meters per day as established in Chapter 3 Table 3. COD test data reflect an examination of removal efficiencies for the observed SOR only; two constraints required this limitation, coordination of running numerous tests simultaneously and observed overflow rates governing the current efficacy of the treatment process. Detention times for both testing regimes were calculated using SOR values and the depth of a Phipps and Bird jar, 6 inches.

Bench Scale Turbidity Decline Data through Use of Ferric Chloride

Table 23 contains the results of bench scale testing data reflecting for turbidity declines observed using CEPT. These values were utilized in the development of Figures 18 and 19 in Chapter 6. Surface overflow rates of 4.2 minutes correlate with the observed overflow rate, while surface overflows of 6.6 minutes correlate with those recommended in designing Imhoff tanks.

Table 23: Turbidity Decline Efficacy Utilizing Ferric Chloride, Las Vegas January 2009

CEPT Efficacy Testing for Decline in Turbidity Las Vegas Imhoff Tanks, January 2009									
Test #	Test Type	Date	Time Sample Obtained	Coagulant Dosage (mg/l)	Turbidity t=0 (NTU)	Turbidity t=4.2 min (NTU)	Turbidity t=6.6 min (NTU)	Turbidity Removal Efficiency Las Vegas SOR	Turbidity Removal Efficiency Ideal SOR
3	Coagualtion Only	16-Jan-09	8:30 AM	100	123	56	58	54.5%	52.8%
25	Coag/COD	17-Jan-09	11:00 AM	100	129	63	52	51.2%	59.7%
26	Coag/COD	17-Jan-09	11:00 AM	125	139	46	42	66.9%	69.8%
27	Coag/COD	17-Jan-09	11:00 AM	150	126	29	30	77.0%	76.2%
19	Coag/COD	17-Jan-09	9:20 AM	175	106	50	32	52.8%	69.8%
2	Coagulation Only	16-Jan-09	8:30 AM	200	113	40	36	64.6%	68.1%
10	Coag/COD	16-Jan-09	10:30 AM	225	147	43	32	70.7%	78.2%
20	Coag/COD	17-Jan-09	9:20 AM	225	99	21	17	78.8%	82.8%
7	Coag/COD	16-Jan-09	10:30 AM	250	152	27	24	82.2%	84.2%
8	Coag/COD	16-Jan-09	10:30 AM	275	158	13	16	91.8%	89.9%
21	Coag/COD	17-Jan-09	9:20 AM	275	113	21	12	81.4%	89.4%
1	Coagulation Only	16-Jan-09	8:30 AM	300	120	21	18	82.5%	85.0%
4	Coagulation Only	16-Jan-09	8:30 AM	300	88	14	11	84.1%	87.5%
22	Coag/COD	17-Jan-09	9:20 AM	300	110	13	8	88.2%	92.7%
9	Coag/COD	16-Jan-09	10:30 AM	325	149	12	15	91.9%	89.9%
23	Coag/COD	17-Jan-09	9:20 AM	325	111	13	12	88.3%	89.2%
11	Coag/COD	16-Jan-09	10:30 AM	350	159	20	13	87.4%	91.8%
24	Coag/COD	17-Jan-09	9:20 AM	350	106	17	14	84.0%	86.8%
12	Coag/COD	16-Jan-09	10:30 AM	375	174	17	11	90.2%	93.7%
28	Coag/COD	17-Jan-09	11:00 AM	375	109	20	19	81.7%	82.6%
5	Coagulation Only	16-Jan-09	8:30 AM	400	112	18	13	83.9%	88.4%
29	Coag/COD	17-Jan-09	11:00 AM	400	117	18	22	84.6%	81.2%
30	Coag/COD	17-Jan-09	11:00 AM	425	110	48	43	56.4%	60.9%
6	Coagulation Only	16-Jan-09	8:30 AM	500	131	129	131	1.5%	0.0%

Bench Scale COD Removal Efficacy Data through Use of Ferric Chloride

The data contained within Table 24 reflect the COD removal efficacy observations made while conducting bench scale tests utilizing ferric chloride as a coagulant in tandem with the Las Vegas Imhoff tanks. Influent COD values reflect those of the wastewater prior to dosing with ferric chloride, effluent values reflect those obtained when correlating to a surface overflow rate of 52 meters per day (4.4 minutes detention time in jar).

Table 24: COD Removal Efficacy utilizing Ferric Chloride, Las Vegas January 2009

CEPT Efficacy Testing for COD Removal Las Vegas Imhoff Tanks, January 2009							
Test #	Test Type	Date	Time Sample Obtained	Coagulant Dosage (mg/l)	COD Influent (mg/l)	COD Effluent (mg/l)	COD Removal Efficiency
25	Coag/COD	17-Jan-09	11:00 AM	100	422	194	54.0%
26	Coag/COD	17-Jan-09	11:00 AM	125	479	191	60.1%
27	Coag/COD	17-Jan-09	11:00 AM	150	444	162	63.5%
19	Coag/COD	17-Jan-09	9:20 AM	175	308	180	41.6%
10	Coag/COD	16-Jan-09	10:30 AM	225	448	293	34.6%
20	Coag/COD	17-Jan-09	9:20 AM	225	304	151	50.3%
7	Coag/COD	16-Jan-09	10:30 AM	250	519	285	45.1%
8	Coag/COD	16-Jan-09	10:30 AM	275	513	270	47.4%
21	Coag/COD	17-Jan-09	9:20 AM	275	308	164	46.8%
22	Coag/COD	17-Jan-09	9:20 AM	300	316	104	67.1%
9	Coag/COD	16-Jan-09	10:30 AM	325	498	283	43.2%
23	Coag/COD	17-Jan-09	9:20 AM	325	329	122	62.9%
11	Coag/COD	16-Jan-09	10:30 AM	350	482	287	40.5%
24	Coag/COD	17-Jan-09	9:20 AM	350	302	128	57.6%
12	Coag/COD	16-Jan-09	10:30 AM	375	450	244	45.8%
28	Coag/COD	17-Jan-09	11:00 AM	375	364	127	65.1%
29	Coag/COD	17-Jan-09	11:00 AM	400	386	127	67.1%
30	Coag/COD	17-Jan-09	11:00 AM	425	369	155	58.0%

Low Dosage Efficacy Comparison Data Ferric Chloride and Aluminum Sulfate MIT

The same methods described within the Las Vegas CEPT bench scale testing sections were utilized in conducting the low dosage comparison efficacy for ferric chloride and aluminum sulfate; as such they will not be repeated here for brevity. Material safety data sheets for these chemicals along with suppliers are contained within Appendix II. Contained within Table 25 are the data collected during this comparison testing. Dosage solutions for both ferric chloride and aluminum sulfate consisted of 5 grams of dry weight chemical diluted per 100 milliliters of de-ionized water. The solution was administered with a pipette. Initial sampling was conducted measuring influent turbidity, total suspended solids (TSS), and chemical oxygen demand (COD). The methods for measuring turbidity and COD were outline in Appendix I; TSS measurements were made using the HACH DR/2500 spectrophotometer and method 8006.

Table 25: Data Collected During Low Dosage Efficacy Comparison MIT Labs

Data Collection Sheet for Coagulant Testing at MIT March 20, 2009													
	Chemical Data				Detention Time 4 Minutes						Removal Efficiencies		
Test #	Alum "check box"	Ferric Chloride "check box"	Dosage (mg/L)	Dosage Level of Active Metal Ion (mg/L)	Initial Turb @ t = 0 min (NTU)	Turb @ 4 min (NTU)	Initial TSS @ t = 0 min (mg/L)	TSS @ 4 min (mg/L)	Initial COD @ t = 0 min (mg/L)	COD @ 4 min (mg/L)	Turb % (NTU) Removal	TSS % (mg/L) Removal	COD % (mg/L) Removal
Raw Influent	NA	NA	0	0	112	97	123	110	253	173	13%	11%	32%
1		X	25	9		58	NA	76		158	48%	38%	38%
2		X	50	17		64	NA	83		144	43%	33%	43%
3		X	75	26		73	NA	80		142	35%	35%	44%
4		X	100	34		82	NA	85		148	27%	31%	42%
5		X	125	43		69	NA	79		141	38%	36%	44%
6		X	150	51		39	NA	53		138	65%	57%	45%
7		X	175	60		24	NA	61		136	79%	50%	46%
8		X	200	68		31	NA	64		128	72%	48%	49%
9		X	225	77		52	NA	49		147	54%	60%	42%
10		X	250	85		73	NA	70		159	35%	43%	37%
11	X		25	2		52	NA	89		168	54%	28%	34%
12	X		50	5		46	NA	72		162	59%	41%	36%
13	X		75	7		46	NA	64		153	59%	48%	40%
14	X		100	9		53	NA	69		151	53%	44%	40%
15	X		125	11		47	NA	69		143	58%	44%	43%
16	X		150	14		58	NA	79		144	48%	36%	43%
17	X		175	16		56	NA	81		148	50%	34%	42%
18	X		200	18		62	NA	78		147	45%	37%	42%
19	X		225	20		69	NA	87		153	38%	29%	40%
20	X		250	23		68	NA	91		164	39%	26%	35%
						Testing Apparatus Utilized	Phipps & Bird Jar Tester					Tests Performed By	R. McLean
						Mixing Energy Applied From Device	100 RPM					Sampling Location	Deer Island
						Time of Mixing	30 Seconds					Signature	

Appendix IV: Sludge Production Calculations

Sludge Production Calculations (Inherent and CEPT) for Las Vegas Imhoff Tanks

Inherent Sludge Production

Given:

Q_{avg} (meters^3/day), Flow rate of wastewater	= 1060 meters^3/day
Average Influent Turbidity (NTU)	= 125 NTU
Average Effluent Turbidity (NTU)	= 102 NTU

Assumptions:

Assumed total suspended solids conversion[43]	= 1.5 mg/L/NTU
Average Influent TSS_{inf} (mg/L)	= 190 mg/L
Average Effluent TSS_{eff} (mg/L)	= 150 mg/L
Removal % of TSS	= 20 %

Calculation:

Sludge produced (kg/day) $= Q_{avg} \times \frac{1000L}{m^3} \times TSS_{\inf} \times \%removal \times \frac{kg}{10^6 mg}$

Sludge produced (kg/day) = 40 kg/day

Sludge Production with CEPT

Given:

Q_{avg} (meters^3/day), Flow rate of wastewater	= 1060 meters^3/day
Average Influent Turbidity (NTU)	= 125 NTU
Average Effluent Turbidity (NTU)	= 102 NTU

Assumptions:

Assumed total suspended solids conversion	= 1.5 mg/L/NTU
Average Influent TSS_{inf} (mg/L)	= 190 mg/L
Average Effluent TSS_{eff} (mg/L)	= 100 mg/L
Removal % of TSS[44]	= 47%
Dosage of $FeCl_3 \cdot 6H_2O$ Required (mg/L)[45]	= 100 mg/L
Percent of $FeCl_3 \cdot 6H_2O$ Precipitated as $Fe(OH)_3$	= 40%

Calculation:

Sludge produced TSS (kg/day) + Sludge produced $Fe(OH)_3$ (kg/day)

$$Q_{avg} \times \frac{1000L}{m^3} \times \frac{kg}{10^6 mg}\left(TSS_{\inf} \times \%removal + DosageFeCl_3 \times \%precipitated\right)$$

Sludge produced (kg/day) = 140 kg/day, 68% as TSS (95 kg/day)[46]

---

[43] Assumed moderate level of conservatism due to lack of measurement for TSS in field, actual value will vary.
[44] Removal required achieving effluent guidelines established by Honduran regulation.
[45] Minimum dosage observed to achieve desired removal efficiency.
[46] It should be noted that chemically precipitated sludge matter takes longer to digest (Lee, 2007)

Sludge Production Estimates for Proposed Trickling Filters

Given[47]:

Typical sludge volume (liter/meter^{3}treated)	= 18 liter/meter^3
Percent solids content, %	= 1.5 %
Percent organic content, %	= 55%
Bacteria Growth Rate, Y	= 0.23kg/kg BOD removed

Assumptions:

Q_{avg} Flow rate of wastewater	= 1060 meter^3/day
Influent COD to trickling filter (mg/L)	= 250 mg/L
Conversion factor COD to BOD	= 0.7
Influent BOD to trickling filter (mg/L)	= 175 mg/L
Effluent BOD from trickling filter (mg/L)	= 46 mg/L

Calculation:

$$\text{BOD removed (kg/day)} = Q_{avg} \times \frac{\left(BOD_{\inf} - BOD_{eff}\right) \times 1000 \frac{L}{m^3}}{10^6 \frac{g}{kg}}$$

BOD removed = 140 kg/day

$$\text{Biological sludge (kg/day)} = BOD_{removed} \times Y$$

Biological sludge produced ~ 35 kg/day

[47] Given values taken from (Lee, 2007)

Appendix V: Design Calculations for Secondary Systems

Design Calculations for Las Vegas Trickling Filter

The following design calculations follow the recommended methods developed from National Research Council studies which assessed the performance of trickling filters used throughout World War II at military installations (Chapter 7, Table 11). The system recommended for Las Vegas utilizes only one trickling filter in series with the Imhoff tanks.

Efficiency of Proposed Trickling Filter

Assumptions:

Average Flow Rate Q_{avg} (Mgal/day)	= 0.5 MGD
Recirculation Flow Q_R (Mgal/day)	= 0.4 MGD
$COD_{influent}$, COD at effluent Imhoff (mg/L)	= 250 mg/L
Conversion factor COD to BOD	= 0.7
Depth of media (feet)	= 7 ft
Volume of media (feet^3)	= 12,000 ft^3
Type of media	= Rock

Calculations:

F, recirculation factor $= \dfrac{1+r}{(1+0.1r)^2}$

r, recirculation ratio, Q_R/Q_{avg}

***F* = 0.8**

E_1, Removal Efficiency, % $= \dfrac{100}{1+0.0561\sqrt{\dfrac{W}{VF}}}$ (US customary units)

E_1 = efficiency of BOD removal at 20ºC with recirculation
W = BOD loading to filter, lb/day
V = volume of filter media, 1000 ft^3
F = recirculation factor

$W = (COD_{\inf} \times 0.7) \times Q_{avg} \times 8.34 lb/(Mgal \bullet mg/L)$

***W* = 730 lb/day**

E_1 = 74 %

Pumping Requirements Proposed Trickling Filter

Assumptions:

Recirculation Flow Q_R (Mgal/day)	= 0.4 MGD
Height of freeboard (feet)	= 1.5 ft
Depth of underdrain (feet)	= 1.5 ft
Depth of media (feet)	= 7 ft
Pump Efficiency, percentage	= 70%

Calculations:

hp_{lift}, horsepower required $= \dfrac{Q_R \times (height(freeboard + underdrain + media))}{3960(MGD \bullet ft/hp)}$

hp_{lift} = 0.70

Brake horsepower required $= \dfrac{hp_{lift}}{pumpefficiency}$

Brake hp = 1.00

Pumping Costs $= \left(\dfrac{Brakehp \times 746 watts/hp \times 24 hrs/day}{1000 watts/kw}\right) \times \left(\dfrac{Xmoney}{kwh}\right)$

Pumping Costs ~ 90 lempira/day

Sizing Calculations for Las Vegas Constructed Wetlands

Preliminary calculations for sizing of constructed wetlands have been provided. Since these systems require large tracts of level terrain, which is limited in Las Vegas, area requirements are the predecessors to any additional design calculations. Calculations for wetland sizing based upon empirical design formulations (Crites, 1998).

Area Requirements for Free Water Surface Constructed Wetlands

Assumptions:

Average Flow Q_{avg} (Mgal/day)	= 0.53 MGD
BOD_{inf} (mg/L),	= 220 mg/L
BOD_{eff} (mg/L), regulatory level	= 50 mg/L
$k_{apparent}$ (day^{-1})	= 0.68
d_w, depth of flow (feet)	= 1.25 feet
η, plant based void ratio	= 0.70
Maximum BOD loading (lb/ac•day)	= 100 lb/ac•day

Calculations:

t, Req. Detention Time (days)= $\dfrac{-\ln \dfrac{BOD_{eff}}{BOD_{\inf}}}{k_{apparent}}$

***t* = 2.2 days**

A_{min}, min. surface area (acres)= $\dfrac{Q_{avg} \times t \times 3.07}{d_w \times \eta}$

A_{min} = 4 acres

Check against maximum BOD loading

BOD loading (lb/ac•day) $= \dfrac{BOD_{\inf} \times Q_{avg} \times 8.34 lb/(Mgal \bullet mg/L)}{A_{\min}}$

BOD loading ~ 240 lb/ac•day **Exceeds Recommended 100 lb/ac•day**

Adjust area with ratio $= \dfrac{BOD_{loading}}{MaxBOD_{loading}}$

$A_{REQUIRED}$ = 9.6 acres

Area Requirements for Submerged Flow Constructed Wetlands

Assumptions:

Average Flow Q_{avg} (Mgal/day)	= 0.55 MGD
BOD_{inf} (mg/L),	= 220 mg/L
BOD_{eff} (mg/L), regulatory level	= 50 mg/L
$k_{apparent}$ (day^{-1})	= 1.10
d_m, depth of medium (feet)	= 2.5 feet
d_w, depth of fluid in bed (feet)	= 2.0 feet
η, effective porosity medium	= 0.30
Maximum BOD loading (lb/ac•day)	= 100 lb/ac•day

Calculations:

t, Req. Detention Time (days)= $\dfrac{-\ln \dfrac{BOD_{eff}}{BOD_{\inf}}}{k_{apparent}}$

***t* = 1.4 days**

A_{min}, min. surface area (acres)= $\dfrac{Q_{avg} \times t \times 3.07}{d_w \times \eta}$

A

Check against maximum BOD loading

BOD loading (lb/ac•day) $= \dfrac{BOD_{\inf} \times Q_{avg} \times 8.34 lb/(Mgal \bullet mg/L)}{A_{\min}}$

BOD loading ~ 270 lb/ac•day **Exceeds Recommended 100 lb/ac•day**

Adjust area with ratio $= \dfrac{BOD_{loading}}{MaxBOD_{loading}}$

$A_{REQUIRED}$ = 10.1 acres

Appendix VI: Unit Conversion Table

Table 26: Common Conversions Between US Customary and SI units

Distance		
1 inch	= 2.54 centimeters	= 25.4 millimeters
1 foot	= 0.305 meter	= 30.48 centimeters
1 yard	= 0.9144 meter	
1 mile	= 1.61 kilometers	= 5,280 feet
1 kilometer	= 1,000 meters	= 0.6214 mile
1 meter	= 100 centimeters	= 1,000 millimeters
1 meter	= 3.28 feet	
1 centimeter	= 0.3937 inch	= 10 millimeters
1 millimeter	= 0.039 inch	= 0.1 centimeter
1 micron	= 10^{-4} centimeter	= 10^{-6} meter
10^{-6} meter	= 1micrometer	
Volume		
1 kiloliter	= 1,000 liters	= 1 cubic meter
1 liter	= 1,000 milliliters	= 1,000 cc
1 milliliter	= 1 cc (exactly 1.000027 cc)	
1 fluid ounce	= 29.57 milliliters	
1 US gallon	= 3.785 liters	
1 Imperial gallon	= 4.546 liters	
Weight		
1 kilogram	= 1,000 grams	= 2.2 pounds
1 gram	= 1,000 milligrams	= 0.035 ounce
1 milligram	= 1,000 micrograms	= 1/1,000 gram
1 microgram	= 10^{-6} grams	= 1/1,000 milligram
1 nanogram	= 10^{-9} grams	= 1/1,000 microgram
1 pound	= 0.45 kilogram	= 16 ounces
1 ounce	= 28.35 grams	

Source: GlobalSecurity.org
http://www.globalsecurity.org/security/library/policy/army/fm/3-11-22/taba-1.gif