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Secondary Municipal Wastewater Treatment Using the UASB/ Solids Contact Technology

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SECONDARY MUNICIPAL WASTEWATER TREATMENT USING THE UASB/SOLIDS CONTACT TECHNOLOGY

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LIST OF ABBREVIATIONS

AFBR	Anaerobic Fluidized Bed Reactor
ASCC	Aerobic Solid Contact Chamber
BOD	Biological Oxygen Demand
COD	Chemical Oxygen Demand
DO	Dissolved Oxygen Levels
ESS	Effluent Suspended Solids
HRT	Hydraulic Retention Time
MLSS	Mixed-Liquor Suspended Solids
PCOD	Particulate COD
RAS	Return Activated Sludge
SCC	Solid Contact Chamber
SRT	Solid Retention Time
SS	Suspended Solids
TCOD	Total Chemical Oxygen Demand
TF/SC	Trickling filter-solids contact processes
TSS	Total Suspended Solids
UASB	Upflow Anaerobic Sludge Blanket
VFA	Volatile Fatty Acids
VSS	Volatile Suspended Solids
WAS	Waste Activated Sludge
WWTP	Wastewater treatment plant

ABSTRACT

Anaerobic pretreatment and aerobic post-treatment of municipal wastewater is being used more frequently. Recent investigations in this field using an AFBR / aeration chamber combination demonstrated the technical feasibility of this process. The investigation presented herein describes the use of a combined UASB / aeration chamber system for the treatment of municipal wastewater and attempts to demonstrate the technical feasibility of using the UASB process as both a pretreatment unit and a waste activated sludge digestion system. The results indicate that the UASB reactor has a TSS removal efficiency of about 37%. Of the solids removed by the unit, 33 % were degraded by the action of microorganisms, and 4.6 % were accumulated in the reactor. The results also show that accumulation of solids in the UASB reactor took place in the upper zone of the sludge bed.

1. INTRODUCTION

Anaerobic digestion is among the oldest biological wastewater treatment processes. With the development of digester heating and mixing, the anaerobic digestion process was initially used for the stabilization of sludge from wastewater treatment plants. Originally, the reactors volumes were large, the hydraulic retention time was frequently greater than 30 days, and the reactors were not mixed.

The modern high-rate anaerobic reactor uses a large solids retention time while maintaining a short hydraulic retention time. This differentiation allows the slowly growing microorganisms to remain within the reactor independent of the wastewater flow; therefore, higher volumetric loading rate can be applied.

Anaerobic processes are extremely attractive for the treatment of high-strength wastewater as well as dilute wastewater, such as domestic sewage. The principal reasons are their lower energy requirements and lower excess sludge production. Different anaerobic technologies have been applied to the treatment of domestic wastewater, providing good efficiencies at low HRTs. One technology is the Upflow Anaerobic Sludge Bed (UASB), which is the most frequently used reactor in full-scale installations for the anaerobic treatment of domestic wastewater (Lettinga et al., 1980, 1993).

Anaerobic biological treatment alone cannot achieve the performance levels required for direct discharge. However, it can be employed as a cost-effective pretreatment ahead of aerobic treatment. This technology combining anaerobic pretreatment and aerobic post-treatment of wastewater is being used more and more frequently. The marriage of these processes brings two

advantages: simple design technology and minimization of sludge production (Jenicek et al., 1999).

The University of New Orleans UWMRC has conducted an important research in this field using an Anaerobic Fluidize Bed Reactor (AFBR) / aeration chamber combination. Corzo (2001) studied the feasibility and the efficiency of chemical and biological flocculation treating municipal wastewater using AFBR. Corzo concluded that the AFBR/aeration chamber system is a very attractive alternative for municipal wastewater treatment because it has low operation and maintenance costs, and no costs associated with sludge stabilization. Later, Bustillos (2002) studied a combined anaerobic/aerobic treatment for treating municipal wastewater in which the waste sludge generated during the process was recycled to the anaerobic units to be digested. Bustillos reported that total COD, filtered COD, and total SS removal efficiencies by the anaerobic/aerobic process were higher at 100-min. hydraulic retention time in the aerated solids contact chamber. Finally, Padron (2004) studied the total SS removal, accumulation rates and degree of stabilization solids in the AFBR. Padron concluded that the AFBR was highly efficient in both, solids stabilization and in reduction of sludge production. His results indicate that at a solids load of 1.09 kg SS/m3.d, 0.173 kg SS/m3.d was consumed by the action of the anaerobic bacteria and 0.173 kg SS/m3.d were accumulated at the top of the fluidize bed.

The investigation presented herein describes the use of a combined UASB / aeration chamber system for the treatment of municipal wastewater; quantifies the SS removal and accumulation in the system, and determines the degree of stabilization of solids in the UASB unit. This work attempts to demonstrate the technical feasibility of using the UASB process as both a pretreatment unit and a waste activated sludge digestion system.

1.1. Objectives and Scope

- Determine the rate of accumulation of suspended solids in the UASB reactor.
- Analyze the UASB effectiveness in removing TCOD, TSS, and VSS.
- Evaluate the technical feasibility of the combined treatment without wasting sludge to obtain secondary effluents.

2. LITERATURE REVIEW

2.1. Combined Anaerobic/Aerobic Treatment

Most of the conventional wastewater treatments processes developed and applied so far are base on energy intensive aerobic technology. The most well-known aerobic processes are the activated sludge process, and the trickling filter. Besides being energy intensive, aerobic treatment processes generate excess sludge that requires proper treatment and disposal. In contrast, anaerobic treatment has long been the traditional method for stabilizing municipal sludge. The principal advantages of anaerobic treatment are: energy savings, lower biomass yield, fewer nutrient requirements, and high volumetric loadings (Metcalf and Eddy, 2003). However, conventional anaerobic treatment has little effect on the concentrations of nitrogen and phosphorus (Collivignarelly, 1990), and the effluent quality does not meet secondary effluent characteristics. Consequently, anaerobic treatment can be considered as an effective pretreatment technique.

A considerable number of researchers have investigated the combined anaerobic-aerobic technology with emphasis mainly on nutrient (N and P) removal. The most common process used for biological nitrogen removal in municipal wastewater is the modified Ludzak-Ettinger (MLE) process (Metcalf and Eddy, 2003). The process consists of an anoxic tank followed by and aeration tank where nitrification occurs. Nitrate produced in the aeration tank is recycled to the anoxic tank.

Biological phosphorus removal is based on the so-called luxury uptake (Sedlak, 1991), that can be summarized as follows:

1. Numerous bacteria are capable of storing excess amounts of phosphorus as polyphosphates in their cells.
2. Under anaerobic conditions, phosphorus accumulating organisms (PAOs) will assimilate fermentation products (e.g., volatile fatty acids) into storage products within the cell with the associated release of phosphorus from stored polyphosphates.
3. Under aerobic conditions, energy is produced by the oxidation of storage products and polyphosphate storage within the cell increases at levels that are higher than the original ones.

Techniques developed in recent years have made the anaerobic process economically feasible for pretreatment of high-strength organic industrial wastewater, as well as dilute wastewaters, such as domestic sewage. The combined anaerobic treatment of the wastewater with the excess aerobic sludge could be beneficial for this technology (Jenick et al., 1999).

Van Haandel et al., (1994) compares aerobic treatment of raw sewage, using a conventional sludge process, a sludge thickener and anaerobic sludge digester, with an anaerobic-aerobic treatment system. For the combined system they used a UASB reactor with complementary secondary treatment in an activated sludge process and stabilization of the excess active sludge in the UASB unit. According to Van Haandel et al., (1994) the advantages of the combined UASB-activated sludge system are:

1. As a result of the removal organic material and suspended solids achieved in the UASB reactor, the sludge mass in the subsequent activated sludge process becomes relatively small and consequently the reactor volume is reduced. In practice, the total volume for the anaerobic-aerobic treatment will often be less than half that required for aerobic treatment (including the sludge digester)

2. The presence of the anaerobic reactor dispenses the need for a sludge stabilization unit. The excess activated sludge can be conveyed to the UASB reactor.
3. The stabilized sludge production will be smaller in an anaerobic-aerobic system because of the comparably smaller sludge production in the anaerobic system. In addition, the stabilized sludge in the anaerobic reactor has a high concentration. Therefore, the liquid-solid separation is simpler.
4. By removing part of the organic load anaerobically, the oxygen demand of the aerobic stage becomes lower. Consequently, less power is required. Moreover, the methane produced would be amply sufficient to make the combined system independent of external energy. However, in practice it depends on the efficiencies of the biogas capture and methane combustion.

Souza and Foresti (1996) worked with a system composed of an UASB reactor followed by two aerobic sequencing batch reactors in parallel. The system received synthetic wastewater. Souza and Foresti reported removal efficiencies averaging 95% for COD and 85% for TKN.

Silva et al. (1995) investigated a UASB-activated sludge system, receiving an influent composed of 90% of industrial wastewater. The UASB reactor achieved removal efficiencies around 70% for COD and 80% for BOD. The aerobic system was unstable and subjected to filamentous bulking. On stable periods, the removal efficiencies of the activated sludge system alone averaged 42% for COD and 63% for BOD. Silva attributed the instability to the high percentage of industrial wastewater flow.

Extensive research conducted by Parker (1983) and La Motta et al., (2003) using a trickling filter / solids contact process for treating municipal wastewater has demonstrated that the process is able to produce effluents of excellent quality that can consistently have monthly

average BOD and SS concentration below 20 mg/L using HRTs as low as 15 minutes in the aeration chamber.

Bustillos (2001) studied the combined anaerobic fluidized bed reactor/aerated contact chamber treatment for treating municipal wastewater in which the sludge generated during the process was recycled to the anaerobic units to be digested. Bustillos tried five different hydraulics retention times and reported that total COD, filtered COD, and total SS removal efficiencies by the anaerobic/aerobic process were higher at 100-min. hydraulic retention time in the aerated solids contact chamber. She reported that the recirculation of sludge to the anaerobic step is a significant contribution of organic matter, which improved the performance of the anaerobic reactor.

Pontes et al., (2003) studied the performance of an UASB reactor used for combined treatment of domestic sewage and excess sludge from a trickling filter. The researchers operated the system in two principal phases: (1) the UASB reactor/TF system was fed with domestic sewage pumped directly from the sewer collection stream and (2) besides feeding the reactor with domestic sewage, the UASB reactor was also fed with the aerobic sludge from the trickling filter. The researches reported no adverse effects on the performance of the UASB reactor due to the return of the aerobic sludge produced in the trickling filter. On the contrary, the COD results indicated better removal efficiencies.

Padron (2004) studied the total SS removal, accumulation rates and degree of stabilization of solids in the AFBR. Padron concluded that the AFBR was highly efficient in both, stabilizing the solids and reducing the amount of sludge produced. His results indicate that at a solid load of 1.09 kg SS/m3.d, 0.173 kg SS/m3.d were consumed by the action of the anaerobic bacteria and 0.173 kg SS/m3.d accumulated at the top of the fluidize bed. His results

also show that the solids entrapped in the fluidized bed reactor were almost completely stabilized, and due to their position at the top of the fluidize bed they can be removed without affecting the reactor operation.

The investigation presented herein describes the use of a combined UASB / aeration chamber system for the treatment of municipal wastewater. The author quantifies the SS removal and accumulation in the system and determines the degree of stabilization of solids in the UASB unit, demonstrating the feasibility of avoiding separate sludge stabilization units.

2.2. Anaerobic Degradation of Organic Matter.

Anaerobic digestion is among the oldest processes used for the stabilization of solids and biosolids (Metcalf and Eddy, 2003). Anaerobic degradation of organic matter is a complex microbial process consisting of several interdependent consecutive and parallel reactions. Four different phases can be distinguished in the overall conversion process: (Van Haandel et al., 1994)

1. Hydrolysis: complex particulate matter is converted into dissolved compounds with a lower molecular weight.
2. Acidogenesis: dissolved compound are taken up in the cells of fermentative bacteria and after acidogenesis are excreted as simple organic compounds.
3. Acetogenesis: the products of acidogenesis are converted into the final products for methane production: acetate, hydrogen and carbon dioxide.
4. Methanogenesis: methane is produce from acetate or from the reduction of carbon dioxide by hydrogen.

Until the 1960s the anaerobic digestion process was used basically for the stabilization of sludge from wastewater treatment plants. During that time the reactor volumes were large and the hydraulic retention time was frequently greater than 30 days. Better understanding of the biological process and the possibility of increased solids retention time independently of the hydraulic retention time increased the range of using the anaerobic digestion processes (Maragno et al., 1992)

Anaerobic digestion generally produces a well-stabilized sludge. Raw sludge is introduced into a biological reactor. The digester contents are kept at a near constant temperature of 35 °C and mixed. The organic loading fed to the digester and type of mixing dictate the required detention time of the sludge. A reasonable loading and well mixed digester can produce a well-stabilized sludge in 15 days. A lesser-mixed tank may take from 30 to 60 days to achieve the same degree of stabilization (Robinson et al., 1997). The solids retention time is a fundamental design and operating parameter for all anaerobic processes. In general, SRT values greater than 20 days are needed for anaerobic processes at 30 °C for effective treatment performance. (Metcalf and Eddy, 2003)

2.3. Upflow Anaerobic Sludge Bed (UASB)

The upflow anaerobic sludge bed (UASB) reactor was developed in the 1970s by Gatze Lettinga and his group at the University of Wageningen in the Netherlands (Van Haandel et al., 1994). The UASB reactor is by far the most widely used high rate anaerobic system for anaerobic sewage treatment. Several full-scale plants have been put into operation in developing countries. The basic UASB reactor is illustrated on Figure 1. As shown in figure 1, influent wastewater is distributed at the bottom of the UASB reactor and travels in an upflow mode

through the sludge blanket. Critical elements of the UASB reactor design are the gas-solids separator, the influent distribution system and the effluent withdrawal design (Metcalf & Eddy 2003). The phase separator is placed at the top of the reactor and divides it into a lower part, the digestion zone, and an upper part, the settling zone.

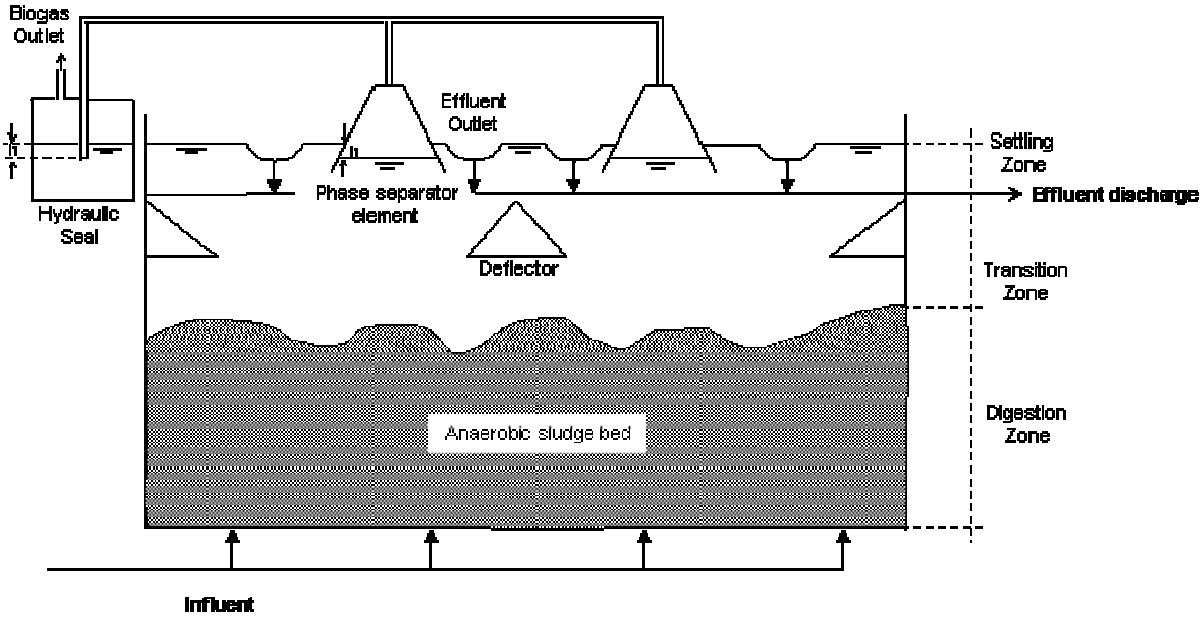

Figure 1 Schematic representation of an upflow anaerobic sludge bed (Adapted from: Van Haandel et al., 1994)

Due to the inclined design of the wall in the phase separator, the upflow velocity decreases when the liquid flows towards the discharge point. Because of the decreasing upflow velocity, sludge flocs drawn into this zone can settle back to the reactor. The presence of a settler on the top of the digestion zone enables the system to maintain a large sludge mass in the UASB reactor (Van Haandel et al., 1994).

The UASB configuration satisfies the main characteristics required for biological treatment system to be simple and efficient: (Foresti, 2002)

1. High biomass concentration inside the reactor, promoting high cellular retention times.
2. Development of structured multi-cellular aggregates in the form of granules, composed by microorganisms groups and different species.
3. Low excess sludge production and low requirements of nutrients.
4. High stability in response to normal fluctuations of influent composition and concentrations.
5. Capacity of accommodating high organic loading rates.
6. Lower cost of construction, installation and operation compared with conventional aerobic units.

In the UASB process biodegradable substances are quickly acidified and the converted into methane and other biogas components. In UASB systems, the sludge bed acts as a filter to the suspended solids, thereby increasing their specific retention time. Due to this increase in retention time, the UASB reactor may achieve high COD and SS removal at very short HRT. However, the accumulation of SS and adsorbed soluble organic matter in the sludge blanket may provoke the displacement of active cells, promoting the formation of sludge with low methanogenic activity. Also the accumulation of inhibitory or toxic substances in domestic wastewater could contribute to the loss of methanogenic activity of the sludge (Ruiz et al,. 1998)

The first results reported for the application of UASB technology for domestic sewage treatment instead of industrial wastewater were quite contradictory. In fact, there are significant differences between domestic sewage and industrial wastewater. Although considered low-strength wastewater, domestic sewage is quite complex due to the high fraction of particulate COD, presence of fatty acids compounds, proteins, and detergents, among other barely known compounds. The aforementioned reasons imposed limitations on the anaerobic process in respect

to COD removal efficiencies, and also in terms of maximum organic loading rates and hydraulic loading rates to be applied. These limitations enforce the need for post-treatment in many cases (Foresti, 2002).

2.3.1. Start-up of a UASB reactor treating sewage.

Reactor start-up is an important economic process step, because during this period the productivity of the wastewater supplied must be adapted to the capacity of the wastewater treatment plant. The duration of the start-up periods depends on numerous biological, chemical and physical parameters. As shown in Figure 2, the start-up is influenced by the wastewater composition and strength, the volume, the activity and adaptation of the inoculum, environmental parameters such as temperature, pH, nutrients and operational parameters like loading rate and retention times (Weiland et al., 1991).

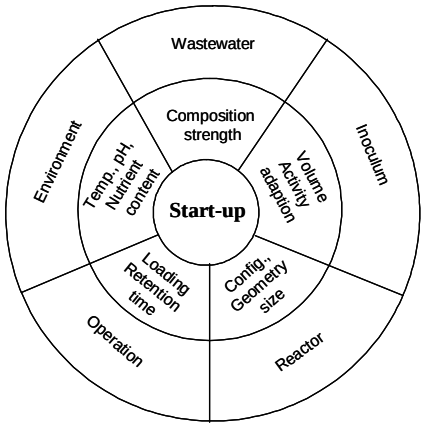

Figure 2 Important parameters for reactor start-up. (Adapted from: Weiland et al., 1991)

During this period the danger of overloading exists and if this occurs, acid fermentation can become predominant over methanogenic fermentation. Sewage contains the bacterial population necessary for the anaerobic digestion. Thus, a UASB reactor for anaerobic sewage

treatment can be started up without the need for inoculation. However, if inoculum is used it should be as high as possible, and the seed should have a sufficient activity and adaptation to the wastewater properties. It is preferable to use a mix of several sources of active biomass instead of biomass from one source (Weiland, 1990). The addition of some municipal anaerobic sludge may be beneficial, due to the increased spectrum of different methanogenic genera (Weiland et al., 1991).

Based on the experimental result obtained from extensive investigation conducted at the Agricultural University of Wageningen and in Cali pilot plant, three UASB reactors were started without using seed sludge and the steady states conditions were reached within 12-20 weeks of operations (Van Haandel et al., 1994).

The start-up process is usually achieved by a progressive increase of the organic loading rate (OLR) up to the design value. Low OLR causes poor hydraulic mixing, especially in UASB reactors, and impeding a proper mass transfer between the phases in the reactor. In fact, it may take 4-8 months before a steady state is reached in this kind of process (Franco et al., 2002).

It is important to mention that the release of biogas in an anaerobic reactor treating sewage is small because: (1) the biodegradable organic material concentration is low. (2) A considerable part of the produced biogas remains dissolved in the liquid phase. Typically, in biogas from sewage about 75% is methane, and the remainder is made up of a mixture of carbon dioxide, nitrogen, water vapor and a small fraction of hydrogen sulfide (Van Haandel et al., 1994)

2.3.2. Anaerobic granular sludge.

Sludge granules are at the core of UASB technology. A sludge granule is an aggregate of microorganisms formed during wastewater treatment in an environment with a constant upflow hydraulic regime. In the absence of any support matrix, the flow conditions create a selective environment in which only those microorganisms, capable of attaching to each other, survive and proliferate. Due to their large particle size (generally ranging from 0.5 to 2 mm in diameter), the granules resist washout from the reactor, permitting high hydraulic loads.

The advantageous performance of the UASB reactor compared to the traditional anaerobic treatment system is due to the granulation of the active biomass. Granulation is a natural self-immobilization process that proceeds in digesters fed with mainly soluble organic matter and operated in an up-flow manner (Lettinga, 1995). Although the precise mechanism of the granule formation remains unknown, their composition, structure and factors influencing their formation are understood to a great extent. The microstructure of the granules is dependent on the chemical composition of the wastewater fed to the UASB reactor (Skiadas et al., 2002). Other factors affecting the development of granulated solids are pH, upflow velocity, and nutrient addition (Annachhatre, 1996).

The presence of other suspended solids in the sludge bed can also inhibit the density and formation of granulated sludge (Lettinga et al., 1991). Granulation has been observed in UASB reactors treating domestic sewage. Barbosa and Sant' Anna (1989) reported granulation in a 120-L UASB reactor treating sewage at temperatures ranging from 18 to 28°C, at HRT of only 4 h with an upflow velocity of 0.48 m/h. According to Seghezzo et al. (2000), more than 30% of the sludge developed in the UASB reactor was granulated. The authors attributed the granulation process to the low concentration of suspended solids in the influent.

2.3.3. Upflow Velocity.

The upflow velocity, based on the flow rate and the reactor area, is a critical design parameter. For weak wastewater the allowable velocity and reactor height will determine the UASB reactor volume, and for stronger wastewater it will be determine by the volumetric COD loadings (Metcalf and Eddy, 2003).

The upflow velocity has repeatedly been the subject of experimentation. A range of 0.3 to 0.9 m/h tested by Viera and Garcia (1992), showed no significant effect on the treatment results. On the other hand, Lettinga and Van Haandel (1994) reported a nearly linear decrease in efficiency while increasing the upflow velocity. These contradictory results may be explained by differences in the occurrence of short-circuiting through the bed. Since, in the most UASB reactors the solids retention time is sufficiently high for degradation of all anaerobically degradable compounds, one would expect low effluent values. The increase of the effluent concentration of soluble compounds at higher upflow velocities may indicate little contact between the bed and the wastewater components (Wiegant, 2001).

Josse (1996) studied the performance of an UASB and anaerobic biofilm fluidized bed (ABFB) reactors with emphasis on fecal Coliform removal. Josse reported that upflow velocities between 1 and 2 m/hr, with organic loading rates around 2 Kg/m^3.day, and HRTs no lower than three hours were the most desirable conditions for efficient pathogen removal.

Hydraulic retention time and upflow velocity may influence the distribution of the sludge, eventually segregating layers based on density differences or concentration gradients (Alphenaar et al., 1993). Since upflow velocity will have an influence on the effluent TSS and BOD concentrations, the effluents requirements, or the requirements of the post-treatment unit

following the UASB reactors is one of the most important parameters to be considered dealing with UASB reactors.

2.4. Sewage Characteristics.

By 1940 most municipal wastewater was generated from domestic sources. After 1940, as industrial development grew significantly, increasing amounts of industrial wastewater have been and continue being discharged to municipal collection systems. The composition and concentration of the impurities of the wastewater will depend on the contributing population.

Wastewaters may contain substances that can adversely affect the performance of an UASB reactor: sludge granulation, foaming or scum formation. The fraction of particulate versus soluble COD is important in determining the design loading for UASB reactors as well as determining the applicability of the process (Metcalf and Eddy, 2003). Invariably, the objectives of sewage treatment include removal of suspended solids and organic material and for this reason these parameter will be discussed in some detail.

2.4.1. Solids

Based on the physical size of the particles, three categories of solids can be distinguished: (1) dissolved; (2) colloidal; and (3) particulate matter. The last two form the suspended solids. One of the important wastewater characteristic with reference to reactor design is the presence of non-biodegradable material. In most wastewaters the organic solids are for the most part biodegradable. In this case, the rate of hydrolysis and the SRT will determine if there will be an accumulation of biodegradable organic suspended solids. If the non-biodegradable material is soluble, non-toxic or toxic but present in sub-inhibitory concentrations, it will not adversely

affect the reactor performance. Particulate non-biodegradable material, on the other hand, will remain within the reactor producing a solids build-up, thereby occupying reactor volume (Iza et al., 1991).

It is important to address not only the ability of the anaerobic reactor to degrade wastewater suspended solids, but also the likely effects of solids entering and remaining inside the reactor on its long-term operation. Experiences in full-scale UASB reactor have shown that a low SS concentration in the influent is preferred and conversion of such low concentrations can be achieved if the SS are biodegradable. However, at higher concentrations, influent SS can cause granular sludge deterioration. UASB reactors containing flocculent sludge can accommodate higher SS concentrations, although the specific sludge activity may be reduced (Iza et al., 1991).

Ruiz et al., (1998) investigated the performance of a UASB reactor treating domestic wastewater and also characterized the biomass in the unit. These researchers observed that the accumulation of organic matter in the upper zone of the UASB was significant, but was not converted to methane because the methanogenic activity in this zone was very low. In order to achieve complete stabilization of the wastewater components, recirculation or external digestion of the accumulated organics in the upper zone of the UASB would be necessary.

2.4.2. Organic material

Organic matter in municipal sewage is present in two forms: particulate and dissolved. Therefore, total chemical oxygen demand (COD) can be defined as the sum of particulate COD (PCOD) and soluble COD (DCOD). It is important to mention that the PCOD is made up of

organic suspended solids and organic colloidal particles, and that the dissolved COD is the truly soluble biodegradable material present in the wastewater.

The primary purpose of wastewater treatment is to remove the suspended and soluble organic constituents measured as COD or BOD in the incoming liquid streams. Levine et al. (1985, 1991) have pointed out that the contaminants that must be removed from the wastewater are a complex mixture of particulate and soluble substances, inorganic and organic, ranging in size from less than 0.001 μm to well over 100 μm. A major fraction of the organic material in municipal wastewater is in the particulate form.

Research performed at the University of New Orleans has determined that in the New Orleans Metropolitan area most of the organic matters present in domestic sewage is particulate material that can be removed by flocculation (La Motta et al., 2003). In the case of Jefferson Parish, more than 80% of the TCOD is in the form of organic particles, while only 20% is truly dissolved organic material. The reported data demonstrates that colloidal COD and suspended solids removal cannot be achieved unless there is successful biological flocculation and sedimentation of the well-formed floc particles (La Motta et al., 2003).

2.5. Solids contact process

Biological processes are used to convert the finely divided and dissolved organic matter into settleable biological flocs that can be removed in sedimentation tanks (Metcalf and Eddy, 2003). In its simple form, the activated sludge process is composed of a reactor, a sedimentation tank, and a sludge recirculation system. In the reactor, the influent organic material is metabolized by microorganism present in the sludge floc. Aerobic bacteria excrete exocellular polymers which destabilize colloidal particles, both organic and inorganic. The gentle agitation

induced by the aeration system provides the required oxygen for aerobic metabolism and promotes particle flocculation (Steiner et al., 1976). Once the mixed liquor enters into the sedimentation tank, liquid-solid separation occurs due to gravity, so that an effluent substantially free of organic material and suspended solids is discharged (Van Haandel et al., 1994). Part of the settled sludge is returned to the aeration basis in order to maintain the desired mixed liquor suspended solids concentration. Because more activated sludge is produced than can be used in the process, some of it is wasted from the aeration basin or from the returned sludge line to the sludge-handling systems for treatment and disposal (Qasim, 1985).

According to Metcalf and Eddy (2003) the principal factors used in process control for an activated sludge system are: (1) maintaining dissolved-oxygen levels in the aeration tanks, (2) regulating the amount of return activated sludge (RAS), and (3) controlling the waste activated sludge (WAS). The mixed-liquor suspended solids (MLSS) concentration is also used as a control parameter. Dissolved oxygen (DO) levels between 1.0 and 1.4 mg/l are recommended to consistently produce a very good final effluent.

An interesting aspect of the biological flocculation process in aerobic systems is the short hydraulic retention time needed in the aeration chamber. Extensive research conducted by Parker (1993) and La Motta et al., (2003) has demonstrated that final effluent SS concentrations of around 20 mg/l can be obtained using HRTs as low as 15 minutes in the aeration tank.

As mentioned before, once the mixed liquor enters into the sedimentation tank, liquid-solid separation will occur due to gravity settling of the flocs, and a well-clarified effluent can be discharged. The two principal functions of the sedimentation tank are: (1) provide clarification to produce high quality effluent, and (2) provide thickening of settled solids. Therefore, sufficient depth must be provided in order to ensure enough space for storage of the settled solids for

thickening and, at the same time, to prevent the solids from being lost in the effluent (Quasim et al., 1994).

2.5.1. Excess sludge

One of the major concerns in aerobic wastewater treatment plants is the need to stabilize and dewater the excess sludge before disposal. According to Metcalf and Eddy (2003) solids and biosolids are stabilized to (1) reduce pathogens, (2) eliminate offensive odors, and (3) inhibit, reduce, or eliminate the potential for putrefaction. Dewatering is a physical operation used to reduce the moisture content of sludge and biosolids.

The possible return of excess sludge to the UASB reactor was originally proposed by Van Haandel and Lettinga (1994) and Souza and Foresti (1996), and tested by Goncalves et al. (1999), who studied the association between UASB reactors and submerged aerated biofilters for domestic sewage treatment. The impact of the aerobic sludge on the anaerobic reactor was studied by Jenicek et al. (1997), who observed a lower specific methanogenic activity of the anaerobic sludge when the UASB reactor was fed with the excess aerobic sludge.

Pontes et al., (2003) studied the performance of an UASB reactor used for combined treatment of domestic sewage and excess sludge from a trickling filter. These researchers reported no adverse effects on the performance of the UASB reactor due to the return of the aerobic sludge produce in the trickling filter. On the contrary, the COD results indicated better removal efficiencies.

Padron (2004) demonstrated that the WAS can be successfully treated in an AFBR. In an experiment using an AFBR linked to an aeration tank with a HRT of 100 min, Padron observed

complete sludge stabilization in the AFBR, and a slow solids accumulation in the anaerobic reactor. Sludge has to be wasted from this reactor after three months of continuous operation

3. EXPERIMENTAL SETUP AND DESIGN

The University of New Orleans UWMRC has been conducting important research aimed at determining the feasibility and efficiency of combined anaerobic/aerobic treatment of domestic wastewater. The present project analyzes the potential use of a combined UASB / aeration chamber system for the treatment of municipal wastewater. The pilot plant is located within the University of New Orleans facility at the Marrero Wastewater Treatment Plant, 6250 Lapalco Boulevard, Marrero, Louisiana. Raw and treated wastewater samples from different stages of the combined treatment process were taken in order to evaluate the performance of the system.

The principal parameters measured during the experimental phase were: total COD, total suspended solids (TSS), volatile suspended solids (VSS), biogas and methane generation.

The full-scale plant processes wastewater by passing it through bar racks. Next, the wastewater goes to an aerated grit chamber and then is diverted to a box that divides the flow to two primary clarifiers. The splitter box is the point from where wastewater was extracted to feed the pilot plant.

3.1. Pilot Plant Description

The UASB/aeration chamber pilot plant consists of the following components:

- Rotating Screen
- UASB reactor
- Aerated solids contact chamber
- Secondary Clarifier

The pilot plant treats 7 L/min of effluent coming from the Marrero full-scale plant splitter box. It is important to mention that the excess sludge was pumped from the clarifier to the UASB unit. A better understanding of the treatment unit train can be seen in the Figure 3.

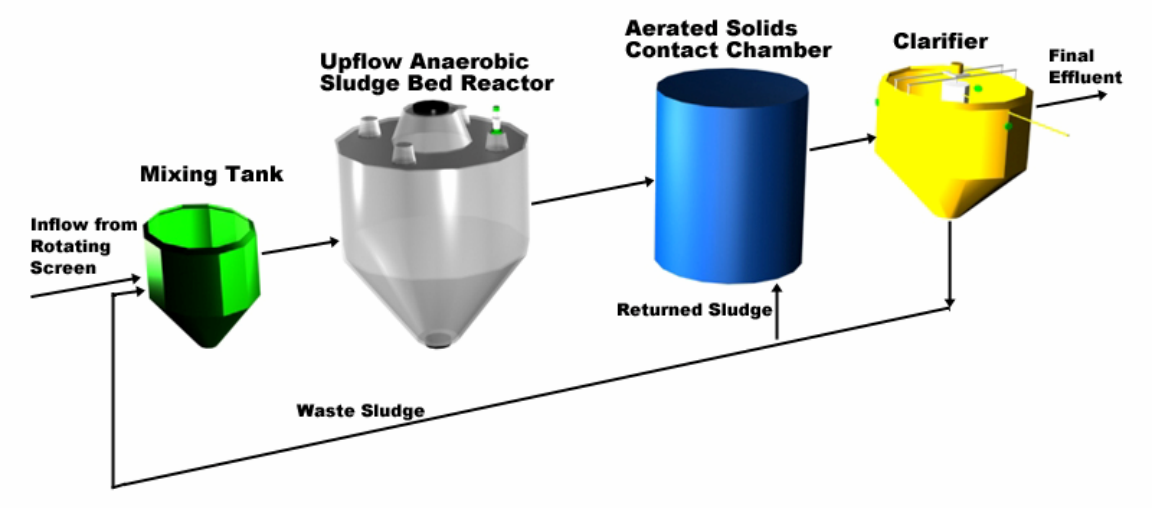

Figure 3 Pilot plant diagram

3.1.1. Feeding System

As mentioned before, the pilot plant influent was taken from the splitter box by a 372.5 W (½ hp) centrifugal pump (1). (Equipment specifications are presented in Table 2). The pump suction (1) has a filtering mechanism, which consist of a perforated 0.91 m (3 ft) section of 10.2 cm (4 in) diameter pipe, with 5.08-cm (2-in) orifices. This perforated pipe is covered with a metal screen with 9.5 mm (3/8 in) mesh size and is encased in a 20.32 cm (8 in) PVC pipe, in order to prevent large solids from entering into the suction pipe.

Pump (1) delivers the wastewater through 50 m of a 2.54 cm (1 in) diameter pipe to a rotational screen (2). Since the flow rate handled by the plant is small compared to the capacity of the pump (1), excess wastewater is wasted to one of the full-scale clarifiers.

The rotational screen (2) has 0.5 mm (0.020 in) openings and 248.5 W (⅓ hp) electrical motor. As wastewater enters the rotating cylindrical screen, solids larger than 0.5 mm ride over the top of the screen and are removed by a blade assembly located along its upper part.

The effluent from the rotational screen is pumped out (3) from an effluent holding tank to a 120-L distribution tank located on the roof of the pilot plant. An electrical mixer (4) continuously stirs the liquid in the distribution tank. Screened effluent flows by gravity from the distribution tank to a 57-L (15 gal) conical-bottom tank. This tank has two inlets, one from the distribution tank with the screened wastewater and the other from the clarifier with the waste sludge. These two streams are mixed by a submersible pump (5) located inside the conical tank.

A diaphragm pump (6) feeds the system with the liquid mixture. It is important to mention that the mixing tank feeds both the AFBR and UASB reactors at a constant flow rate of 125 L/h (33GPH) with individual diaphragm pumps. The effluent of the AFBR was discarded.

3.1.2. Upflow Anaerobic Sludge Bed

The UASB reactor is a 400 L (110 gal) cylindrical polyethylene tank with a 60 degree conical bottom. The tank has a diameter of 0.86 m (33.85 in) and a height of 1.16 m (45.67 in). During a previous research project conducted at the pilot plant, the reactor was used as an anaerobic upflow packed filter (AUPF) filled with rashing rings. For this project, all the rashing rings were removed and sludge was introduce again in order to use it as inoculum for the UASB

reactor. Four ports are arranged along the UASB reactor height allowing samples to be taken. Figure 4 shows the UASB reactor.

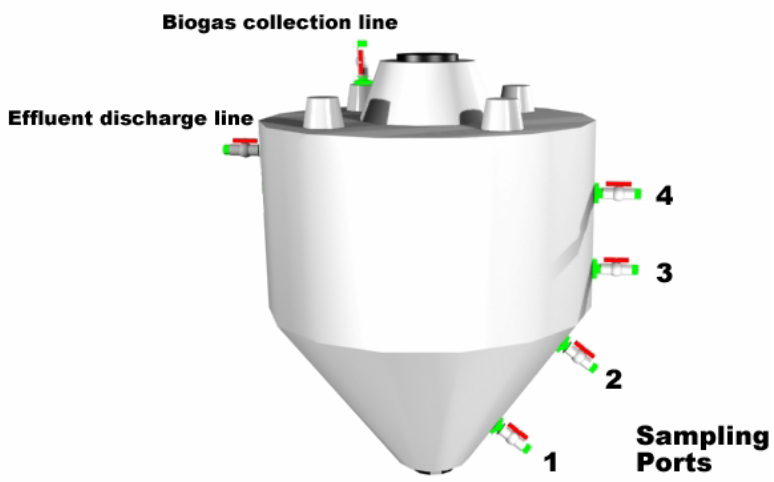

Figure 4 Upflow anaerobic sludge bed reactor

As explained in the literature review the UASB unit is fed from the bottom of the reactor to ensure fluidization of the sludge bed. An internal recirculation system was used to maintain an upflow velocity of around 1 m/h in the cylindrical section. The recirculation is achieved using a 29.8 W (1/25 hp) magnetic pump (7). Figure 5 shows the internal recirculation of the UASB reactor.

Unfortunately, due to low performance of the UASB reactor during the following months after start-up, the UASB unit was emptied and cleaned. The unit was inoculated again with 132.5 L (35 gal) of anaerobic sludge from the digester units of Terrace Avenue Wastewater Treatment Plant located at 2800 Terrace Av. Slidell, Louisiana. It is important to highlight that in order to fill up the rest of the volume of the reactor, raw wastewater was added.

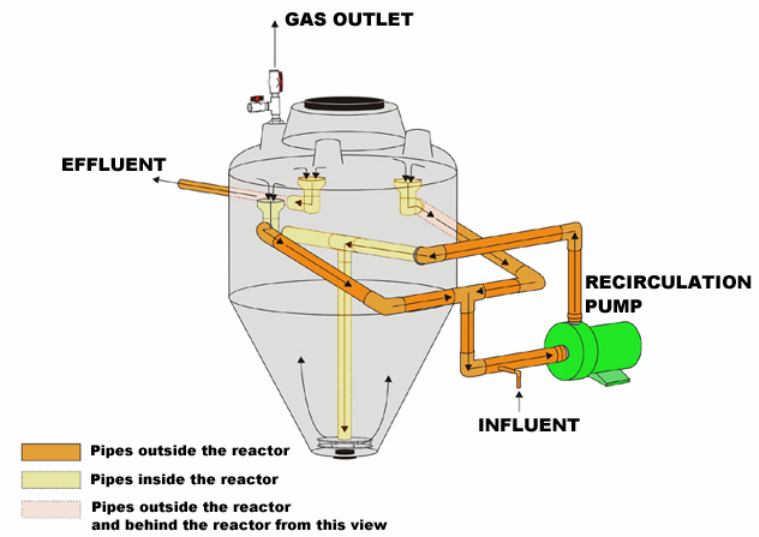

Figure 5 Schematic representation of the UASB reactor (Adapted from: Padron 2004)

3.1.3. Aerated solids contact chamber

During the experimental phase of this research, two aerated solids contact chambers (ASCC) were used in order to work with different hydraulic retention times. The ASCC was fed by gravity from the UASB reactor. The contact chambers consisted of a polyethylene tank equipped with a fine bubble diffuser system at the bottom fed by a 559.3 W (¾ hp) compressor (8). The first ASCC had a volume of 240-L (63.5 gal) and was used to study the effect of using 120-min hydraulic retention time. The volume of the second tank, used to study the effect of 180-min hydraulic retention time, was 360-L (95.1 gal). The blower provided air to maintain an optimum dissolved oxygen (DO) concentration and velocity gradient for uniform mixing and flocculation. Table 1 presents the specifications of the air diffuser system.

Table1 Specifications of the air diffuser system (Bustillos, 2002)

Heat Bonded Silica Fine–Pore Diffuser System	
Length (cm)	15
Width (cm)	4
Number of Diffusers	6
Suggested Flow (m3/h)	0.852
Max. Pore Size (μm)	80
Bubble Size (mm)	0.5 - 2.0

The aerated solids contact chamber is fed at its bottom from both, the effluent of the UASB reactor and the sludge recycled from the clarifier. The reactor contents are completely mixed by the air bubbles, and the mixed liquor leaves the ASCC through the center well, as shown Figure 6.

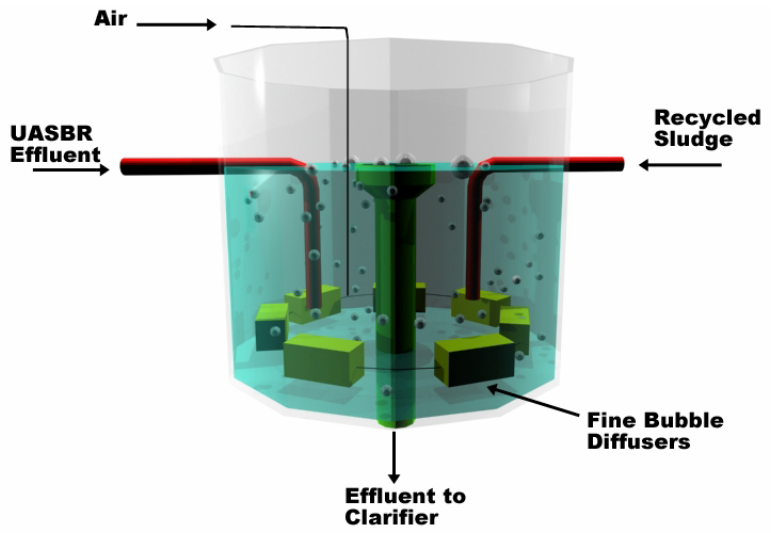

Figure 6 Graphical representation of the solids contact chamber

The air provided by the blower in the ASSC allows development of aerobic bacteria and, consequently, the production of exocellular polymers that cause solids to bind together and form flocs.

3.1.4. Clarifier

The clarifier unit consists of a 280 L (70 gal.) polyethylene tank with a conical bottom section. The water from the ASSC is discharged into the clarifier tangentially in a 20.3-cm (8-in) center well in order to reduce the inflow energy and to provide optimum condition for flocculation. Additionally, a rotary arm moved by a 1 rpm gear motor (9) scrapes the bottom of the conical section with the main purpose of preventing the sludge from compacting.

The clarified effluent leaves the unit through three 38.1 mm (1 ½-in) PVC pipes located radially along the top of the clarifier, and is discharged into the final effluent line. Part of the sludge retained at the bottom of the unit was recycled to both, the ASCC and the mixed tank by centrifugal pumps (10) driven by cycle timers (11). Figure 7 shows a complete diagram of the secondary clarifier unit, including the features mentioned above.

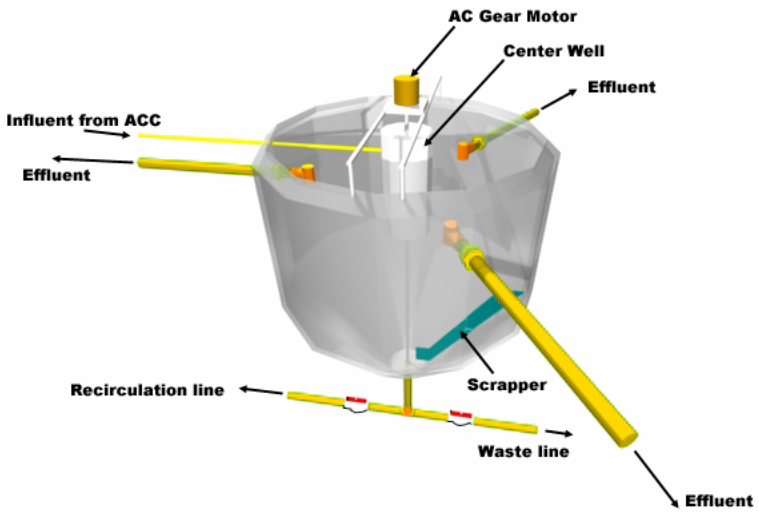

Figure 7 Graphical representation of the clarifier

Table 2 Description of the electric equipment used at the pilot plant

Equipment Number and name	Manufacturer / Model	Characteristics
(1) Centrifugal pump	TEEL / 3P551	Self-priming, ½ HP, 115/230 volts, 58 GPM at 10 ft of head
(2) Rotational strainer	WaterLink Rotostrainer® / RSA2512UBCR	Rotating cylinder screen, 1/3 HP, 120 volts
(3) Centrifugal pump	TEEL / 1P809	Submersible, 1/10 HP, 115 volts, 900 GPH at 1 ft of head
(4) Open drum mixer	Neptune mixer company / B-10	¼ HP, 115/220 volts, 316 Stainless Steel Shaft and 3 Blade Propeller
(5) Centrifugal pump	TEEL / 1P808	Open air/submersible 1/50 HP, 115 volts,400 GPH at 1 ft of head
(6) Diaphragm pump	Cole-Parmer / 76302-50	Single head, 115 volts, 16.5 GPH of maximum flow 125 strokes / minute
(7) Centrifugal pump	TEEL / 2P390	Self-priming, ½ HP, 115/230 volts, 2280 GPH at 10 ft of head
(8) Air compressor	GAST / 4F742	¾ HP, 115/230 volts, free air flow at 10 Inches Vacuum 6.3 CFM
(9) Gear motor	Dayton® / 2Z804	AC Parallel shaft gearmotor, 115 volts
(10) Centrifugal pump	TEEL / 1P808	Open air/submersible 1/50 HP, 115 volts,400 GPH at 1 ft of head
(11) Timer	OMRON® / 2A179	Repeat cycle timer independent on/off times 120/240 volts
(12) Submersible pump	LittleGIANT / 2P352	Submersible, 1/6 HP, 110 volts, 20 GPM at 1 ft of total head

3.1.5. Biogas collection system

The biogas collection system is used to collect and quantify the biogas produced by the UASB reactor. The system is the same one used by Padron (2004), and it consists of:

1. A 114-L (30 gal) gas collection tank, closed to the atmosphere with two valves at its top (release valve and sampling valve)
2. An 8-L leveling container, open to the atmosphere. The leveling container is connected to the gas collection tank and also to the liquid collection tank through an overload connection to discharge the excess liquid and maintain a constant level of retaining fluid.
3. A 200-L liquid collection tank which is open to the atmosphere.

It is important to highlight that the retaining fluid used was a saturated sodium chloride solution containing 5% H_2SO_4 and methyl orange for color (Metcalf and Eddy, 1972). Figure 8 shows a complete diagram of the biogas collection system.

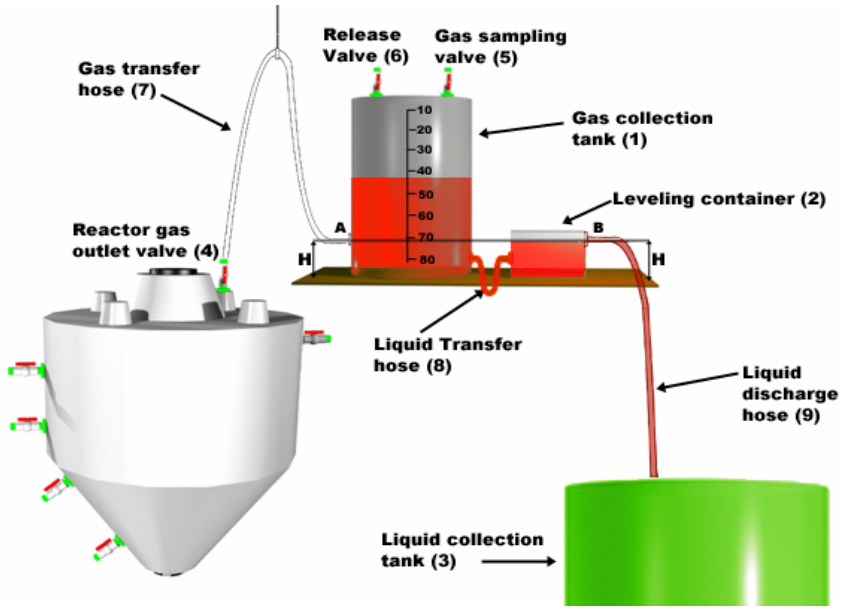

Figure 8 Schematic representation of the biogas collection system

3.2. Sampling and Laboratory Analysis

The sampling phase was initiated in January 2004 and lasted through November 2004. Samples were taken as often as possible depending on weather and plant operating conditions. After collection, samples were taken to the Environmental Laboratory at the University of New Orleans to be analyzed.

3.2.1. Sampling

Water samples were collected from four points within the combined system:

1. Mixing Tank (UASB influent)
2. UASB effluent discharge line
3. Clarifier effluent discharge line
4. Sludge recycle line

As mentioned before, the intermittent discharge of sludge from the clarifier to the mixed tank along with a continuous variation of the pilot plant influent, made it necessary to collect 24-hour composite samples instead of punctual samples. Two automatic composite wastewater samplers from Global Water, model WS300, were used to collect the water samples (150 mil of sample every 60 minutes). In order to preserve the samples, sulfuric acid was added to the collection tank to ensure the final pH was maintained below 2 (AWWA, 1995).

For the UASB effluent, raw samples and their supernatants were stored and analyzed separately. The samples were stored in glass bottles of 500 ml each.

3.2.2. Laboratory tests

Three parameters were measured, namely, total chemical oxygen demand (TCOD), total suspended solids (TSS) and volatile suspended solids (VSS). The analyses were performed in the Environmental Engineering Laboratories located at the Center for Energy Resources Management (CERM)

- Total oxygen demand (TCOD)

The TCOD test is used to measure the oxygen equivalent of the organic material in wastewater that can be oxidized chemically using dichromate in an acid solution (Metcalf and Eddy, 2003). The temperature is increased to the boiling point of the mixture to accelerate the redox reaction. After two hours, the concentration of organic material originally present in the sample can be calculated from the decrease in the dichromate concentration. The COD of a waste is, in general, higher than the biochemical oxygen demand (BOD) because more compounds can be chemically oxidized than can be biologically oxidized.

The TCOD was selected to estimate the amount of organic material in the samples because the TCOD test is easier and takes less time than the BOD test. Method 5220D of the Standard Methods (APHA, 1999) was used to analyze the samples.

- Total Suspended Solids (TSS)

The total suspended solids (TSS) test is used to quantify the amount of suspended organic and inorganic matter present in the water samples. The analyses were performed using Method 2040D of the Standard Methods (APHA, 1999). After filtration, the solids remaining in the 0.45-μm pore size filter paper were dried at 105°C ± 1°C.

- Volatile Suspended Solids (VSS)

The volatile suspended solids (VSS) test is used to quantify the amount of total organic solids, biodegradable and non-biodegradable, present in the water samples. The analyses were performed using Method 2540E of the Standard Methods (APHA, 1999). Solids remaining in the 0.45-µm pore size filter after TSS test were ignited at 550°C. The remaining solids represent the fixed fraction, while the weight lost represents the volatile faction.

3.2.3. Biogas collection and analysis

The methane concentration, which is an important indicator of the anaerobic activity inside the reactor, was measured during the experimental phase. A portable gas analyzer model LMS manufactured by CEA Instruments, Inc, was the device used to monitor the methane concentration in the biogas.

As mentioned before, biogas produce by the UASB reactor was collected and quantify. Biogas sampling was done on a 24-hour basis. Padron (2004) explained the procedure used in this research for collecting biogas samples. Nomenclature refers to figure 8.

1. The gas outlet valve in the reactor (4) was closed.
2. The leveling container (2) was raised until the level of its liquid was equal to the top surface of the gas collection tank (1).
3. The gas release and gas sampling valves in the gas collection tank (5, 6) were open.
4. The retaining liquid was pumped to the gas collection tank (1) from the liquid collection tank (3) by using a 124.2 W (1/6 hp) submersible pump (12).
5. After the gas collection tank (1) was completely filled with the retaining fluid, the gas sampling and gas release valves (5, 6) were closed.

6. The leveling container was lowered until its overload connection (B) was at the same height as the connection of the gas transfer hose (7) to the gas collection tank (point A).
7. Finally the reactor gas outlet valve (4) was open.

It is important to mention that the biogas leaves the UASB reactor at atmospheric pressure. The reason for this lay on the fact that the gas collection tank (1) is interconnected to the leveling container (2), and point A is at the same height H as the surface of the liquid in the leveling container (point B), consequently both points are at the same pressure, which is atmospheric pressure.

4. RESULTS AND DISCUSSION

As mentioned before two automatic composite wastewater samplers were used to collect 24-h composite samples of the UASB reactor influent and effluent.

4.1. UASB Influent

During the experimental phase the UASB reactor was fed with a mixture of screened wastewater from the rotational screen and excess sludge wasted from the bottom of the secondary clarifier. The characteristics of this influent are presented in Table 3.

Table 3 Characteristics of the UASB influent

Parameter	Value
Total COD, mg/L	306
Total suspended solids, mg/L	170
Volatile suspended solids, mg/L	152

Figure 9 shows a linear relationship between TCOD and TSS in the UASB influent. A linear regression analysis generated the following equation:

$$TCOD = 1.54 \times TSS + 47.4 \quad (1)$$

Where TCOD and TSS are in mg/L and the coefficient of determination (R^2) for the data is 0.72.

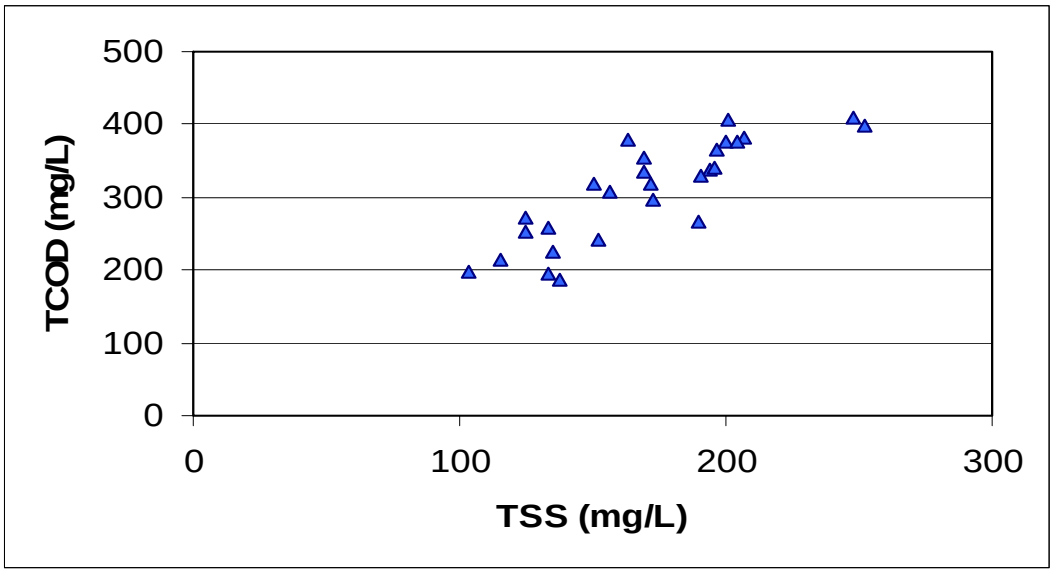

Figure 9 UASB influent TCOD vs. UASB influent TSS

Similarly, figure 10 shows a linear relationship between TCOD and VSS in the UASB influent. A linear regression analysis of the TCOD and VSS data generated the following equation:

$$TCOD = 2.01 \times \mathrm{VSS} + 6.85 \qquad (2)$$

Where TCOD and VSS are in mg/L and the coefficient of determination (R^2) for the data is 0.67.

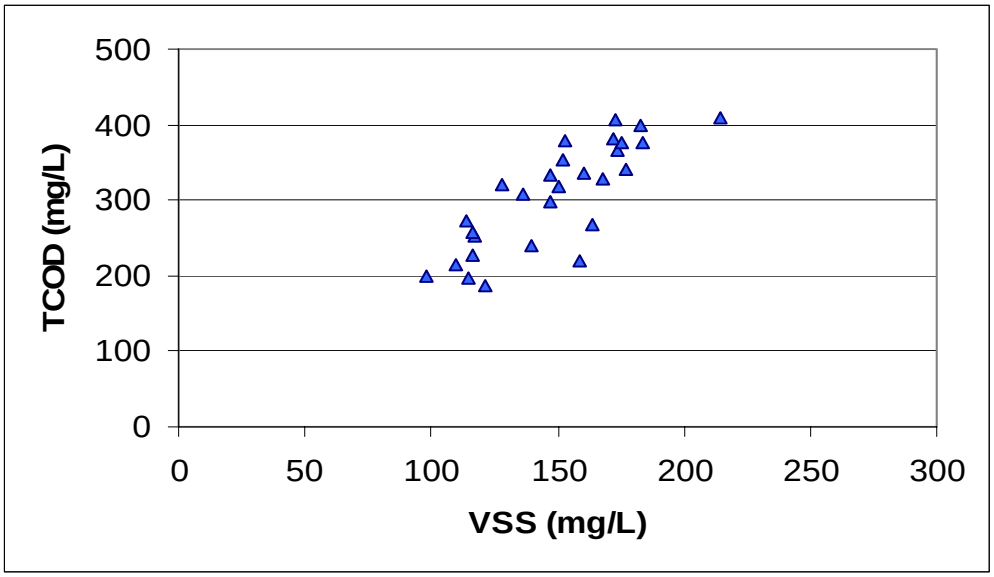

Figure 10 Relationship between UASB influent TCOD and influent VSS

A similar analysis was performed by Jimenez (2000) using a lower range of data. He reported a linear relationship between suspended solids and TCOD, with a coefficient of determination (R^2) equal to 0.98. The linear regression equation was the following:

$$TCOD = 1.9325 \times SS + 31.91 \quad (3)$$

Combining the data of both researches a linear relationship between suspended solids and TCOD could be obtained as shows figure 11. A linear regression analysis of the TCOD and SS data generated the following equation:

$$TCOD = 1.6082 \times SS + 40.09 \quad (4)$$

Where TCOD and SS are in mg/L and the coefficient of determination (R^2) for the data is 0.96.

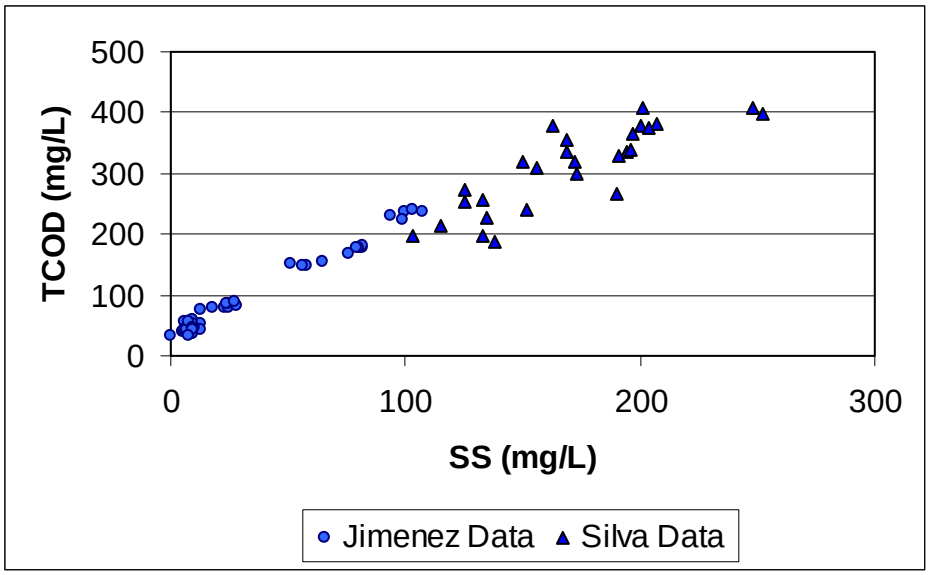

Figure 11 Relationship between TCOD and SS

4.2. UASB effluent

The UASB reactor was fed at a constant flow rate of 120 L/h (31.7 GPH). Therefore, the hydraulic retention time of the reactor based on the sludge bed volume was 3.3 h, the average organic load applied was 2.22 kg TCOD/m^3.d, and the average solids load was 1.23 kg TSS/m^3.d.

Table 4 UASB raw effluent average removal efficiencies

Measured Parameter	Removal
Total chemical oxygen demand (TCOD)	33 %
Total suspended solids (TSS)	37 %
Volatile suspended solids (VSS)	35 %

Table 4 shows the average percent removals of TCOD, TSS and VSS obtained in the UASB based on the mixed effluent. These results indicate a low performance of the UASB unit

compared to typical values reported for UASB reactors treating municipal wastewater using HRT between 6-8 hours. However, in order to maintain a constant HRT in the SCC, no major changes in the UASB operational parameters were done to explore the effect of HRT on TCOD removal.

Table 5 shows the average percent removals of TCOD, TSS and VSS obtained in the UASB based on the settled effluent, and demonstrates that a significant fraction of the effluent TCOD is due to TSS.

Table 5 UASB settled effluent average removal efficiency

Measured Parameter	**Removal**
Total chemical oxygen demand (TCOD)	59 %
Total suspended solids (TSS)	72 %
Volatile suspended solids (VSS)	73 %

Van Haandel et al. (1994) defined the removed load as the load of organic material that is removed from the liquid phase (converted into sludge or methane). They also defined the digested load as the load that is actually converted into methane. The aforementioned analysis states that the degraded load corresponds to the removal considering the raw effluent while the removed load corresponds to the removal obtained considering the settled effluent. Using Van Haandel's criteria, Table 6 shows the average percent removals and degradation in the UASB reactor.

Table 6 Average performance of the UASB

Parameter	Percent removed	Percent degraded
TCOD	59 %	33 %
TSS	72 %	37 %
VSS	73 %	35 %

In order to evaluate the performance of the UASB reactor, the UASB mixed effluent TCOD (mg/L) was plotted against the influent TCOD (mg/L). Figure 12 shows a linear relationship with a coefficient of determination (R^2) of 0.79. A linear regression analysis generated the following equation:

$$TCOD_{Mixed\ Effluent} = 0.66 \times TCOD_{Influent} \quad (5)$$

This equation yields a removal of 34%. The actual average value obtained with the experimental data is 33%. This TCOD removed corresponds to organic matter that is converted to CH_4 and CO_2.

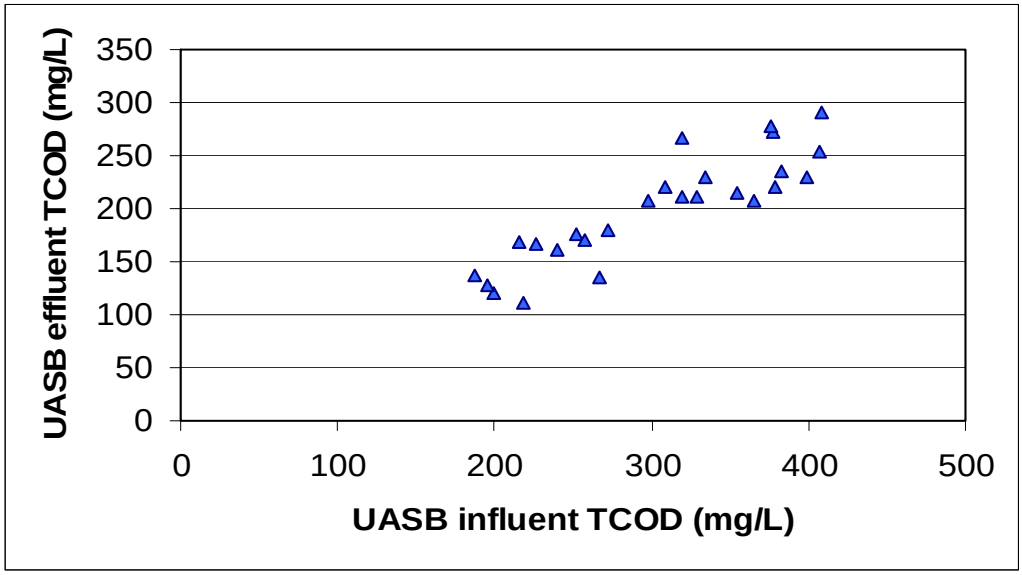

Figure 12 Relationship between UASB Effluent TCOD and influent TCOD

A similar performance was observed with regard to the removal of TSS in the UASB unit. Figure 13 shows a linear relationship between TSS in the influent and mixed effluent of the UASB, with a coefficient of determination (R^2) equal to 0.69. The linear regression equation is the following:

$$TSS_{Effluent} = 0.64 \times TSS_{Influent} \quad (6)$$

This equation yields a TSS removal of 36%. The actual average value obtained with the experimental data is 37%.

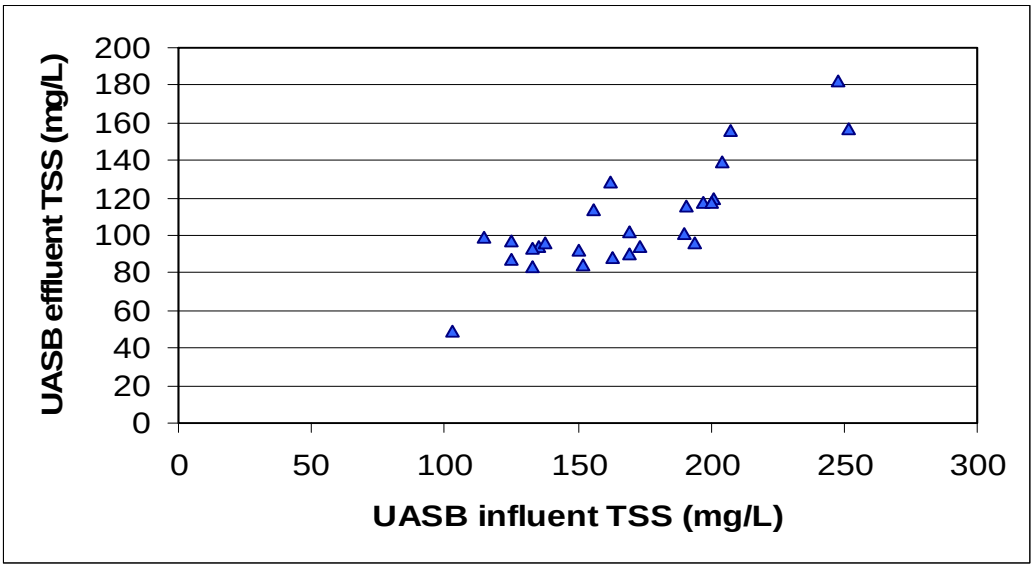

Figure 13 Relationship between UASB Effluent TSS and influent TSS

Figure 14 shows a similar relationship between the UASB mixed effluent and influent VSS concentration, with the following best-fit equation:

$$VSS_{Effluent} = 0.63 \times VSS_{Influent} \quad (7)$$

Where VSS is in mg/L and the coefficient of determination (R^2) for the data is 0.66. This equation yields a removal of 37%. The actual average value obtained with the experimental data is 35%.

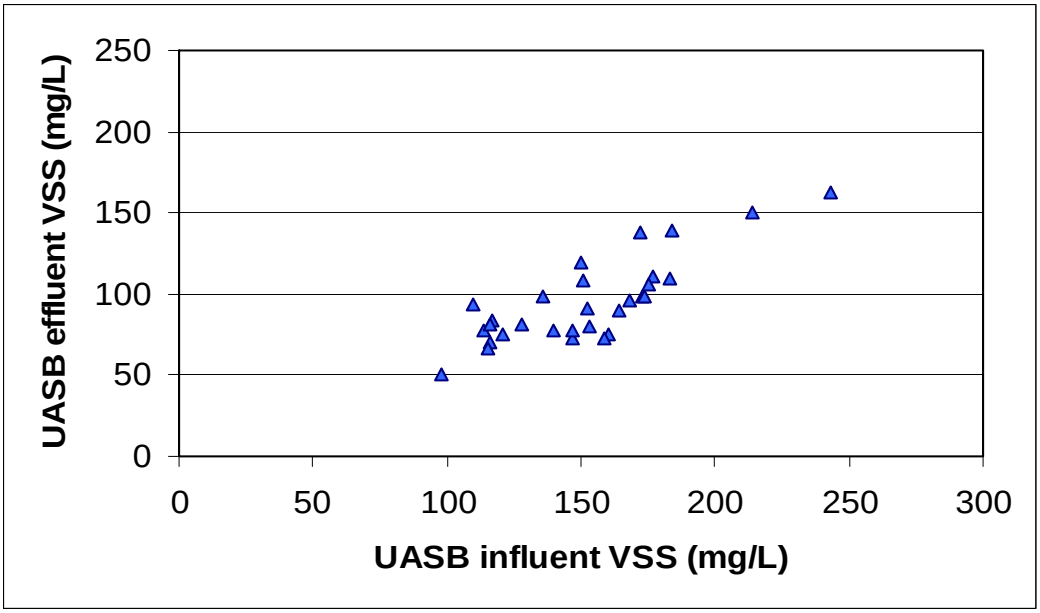

Figure 14 Relationship between UASB Effluent VSS and Influent VSS

It is important to highlight that similar linear relationship between TSS and TCOD were observed in both the influent and effluent values:

$$TSS = 0.65 \times TCOD - 30.78 \quad (8)$$

4.3. UASB biogas

Vieira (1987) reported that the biogas produced in a full-scale UASB reactor treating domestic sewage had an average composition of 70 % methane, 22% nitrogen, and 8% carbon dioxide. Typically, the biogas in an anaerobic reactor treating domestic sewage is about 70-80 percent methane, and the remainder is made up of a mixture of carbon dioxide, nitrogen, water

vapor and a small fraction of hydrogen sulfide (Van Haandel et al., 1994). In the present research, the methane content in the biogas produced had an average of 59.8%.

According to Yoda et al., (1985), given the partial pressure of methane in the overlaying gas phase, the amount of methane dissolved in the effluent can be calculated using Henry's law.

Unfortunately, only a few points could be recorded during September and August 2004 on biogas production. Table 7 shows the production of CH_4 observed, and an estimation of the total production according to Henry's law (Appendix B).

Table 7 Methane production in the UASB unit (25ºC and one atmosphere of pressure)

Date	**Observed CH_4 (ml gas/L sewage)**	**Dissolved CH_4 (ml gas/L sewage) Using Henry's Law**	**Total CH_4 (ml gas/L sewage)**
7/23/2004	5.9	21.61	27.51
7/28/2004	6.55	22.54	29.09
7/29/2004	5.34	23.6	28.94
8/1/2004	4.98	22.45	27.44
8/3/2004	8.73	16.57	25.30
8/4/2004	7.72	15.01	22.74
8/6/2004	6.88	21.79	28.68
8/9/2004	5.57	22.11	27.69
8/11/2004	6.57	21.43	28

4.4. Sludge concentration and accumulation in the reactor

The behavior of the sludge bed in the reactor was analyzed by taking sludge samples from ports at different elevations. Four sampling ports were arranged over the UASB reactor height: P_1=0.1 m, P_2=0.34 m, P_3=0.57 m and P_4=0.67 above the bottom, as indicated in Figure 15.

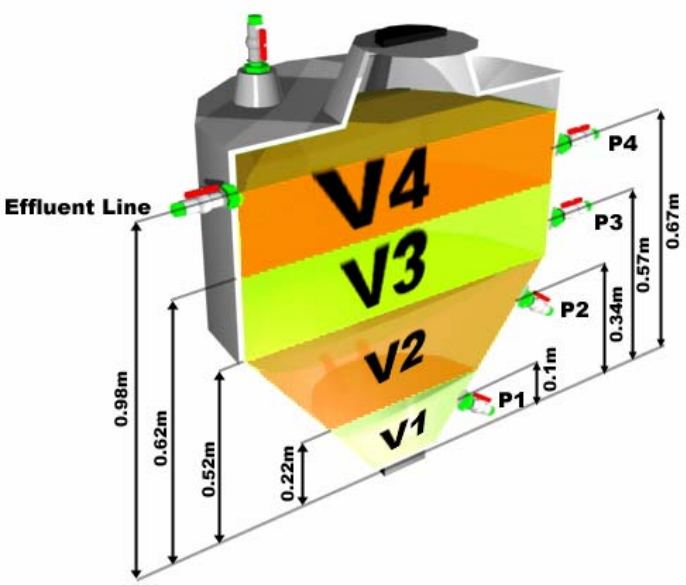

Figure 15 Schematic representation of the imaginary sections used to determine the sludge hold-up in the reactor.

Table 8 and figure 16 show the results of the sludge concentration profile regarding the TSS. As can be seen in figure 16, the distribution of TSS changed along the four tests.

Table 8 TSS concentration profile

Port / Date	7 April 2004 Concentration (mg/L)	27 May 2004 Concentration (mg/L)	26 June 2004 Concentration (mg/L)	22 July 2004 Concentration (mg/L)
1	36016	41667	38970	36899
2	34340	44764	35014	31054
3	13115	3511	28098	28656
4	724	57	108	8788

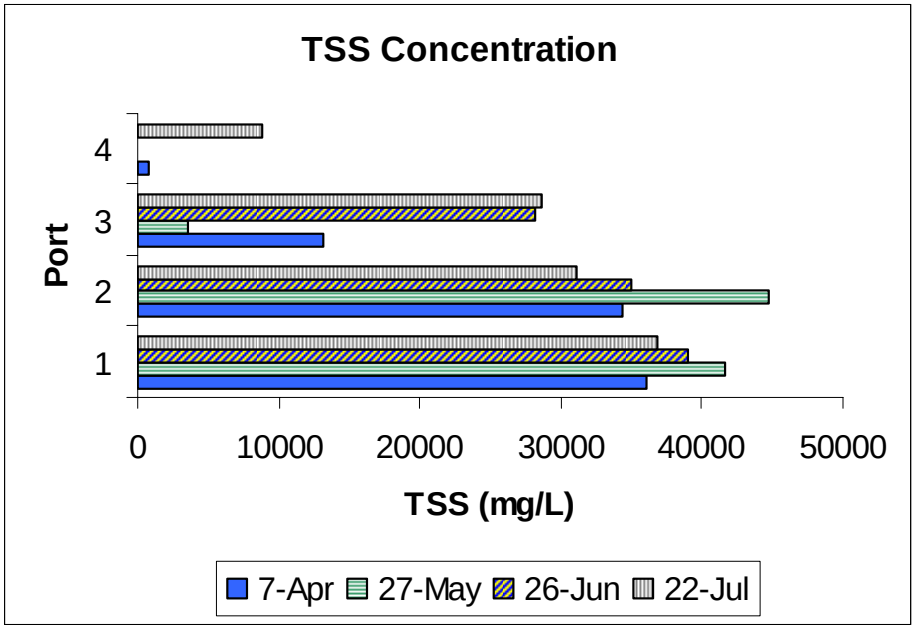

Figure 16 Evolution of TSS concentrations

The results of the first TSS profile test (04/07/2004) shows that the concentration of particles in the bed decreases gradually from P_1 to P_4. The results of the second test (05/27/2004) show a different distribution of concentrations, showing an accumulation of solids in P_2, and a relatively low concentration in P_3. The results of the third test (06/26/2004) show a homogeneous

distribution of the sludge blanket among ports 1, 2 and 3. The relatively high concentration found in P_3 seems to indicate that the height of the sludge bed was increasing and the low concentration found in P_4 (108 mg/L) indicate that the boundary of the sludge bed was somewhere between P_3 and P_4. The results of the last test (07/22/2004) show the tendency of accumulating solids in upper zone of the sludge bed.

Table 9 and figure 17 present the results of the sludge concentration profile regarding to VSS. Using the same analysis, it can be observed that accumulation of solids took place in the upper zone of the sludge bed.

Table 9 VSS concentration profile

Port / Date	7 April 2004 Concentration (mg/L)	27 May 2004 Concentration (mg/L)	26 June 2004 Concentration (mg/L)	22 July 2004 Concentration (mg/L)
1	24390	27955	25576	23411
2	24906	29663	22145	20817
3	9180	2835	18133	18540
4	495	47	86	5636

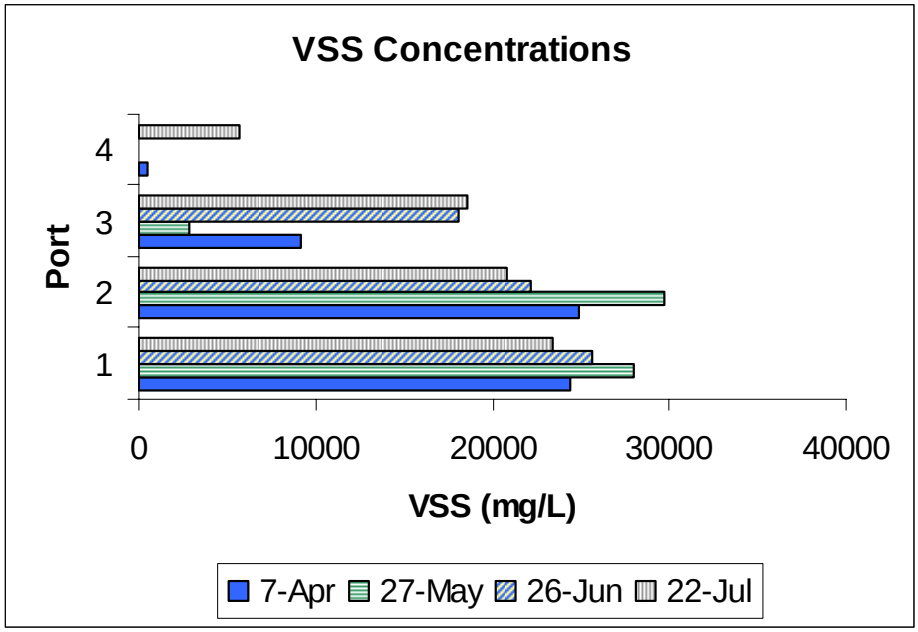

Figure 17 Evolution of VSS concentrations

To estimate the sludge hold-up of the reactor, the sludge concentration at each section was assumed to be equal to the concentration found at its port. Table 11 presents the results based on the section volumes given in table 10.

Table 10 Characteristics of the imaginary sections

Section Volume	Lower limit (Meters from the bottom)	Upper limit (Meters from the bottom)	Volume, m^3
V_1	0.0	0.22	0.021
V_2	0.22	0.52	0.108
V_3	0.52	0.62	0.058
V_4	0.62	0.98	0.209

These results show that between the first and the last solids profile tests (106 days), 2,252 g of TSS and 1,155 g of VSS accumulated in the reactor.

Table 11 Sludge build-up in the UASB

Date	TSS, g	VSS, g
04/07/2004	5,404	3,856
05/27/2004	5,955	3,985
06/26/2004	6,282	4,018
07/22/2004	7,656	5,011

Figure 18 shows the solids build-up in the UASB reactor. These results demonstrate that the solids retained inside the UASB unit tend to accumulate in the upper part of the sludge bed. It is important to mention that during the 106 days between the first and the last profile, no sludge was removed from the reactor except for sampling.

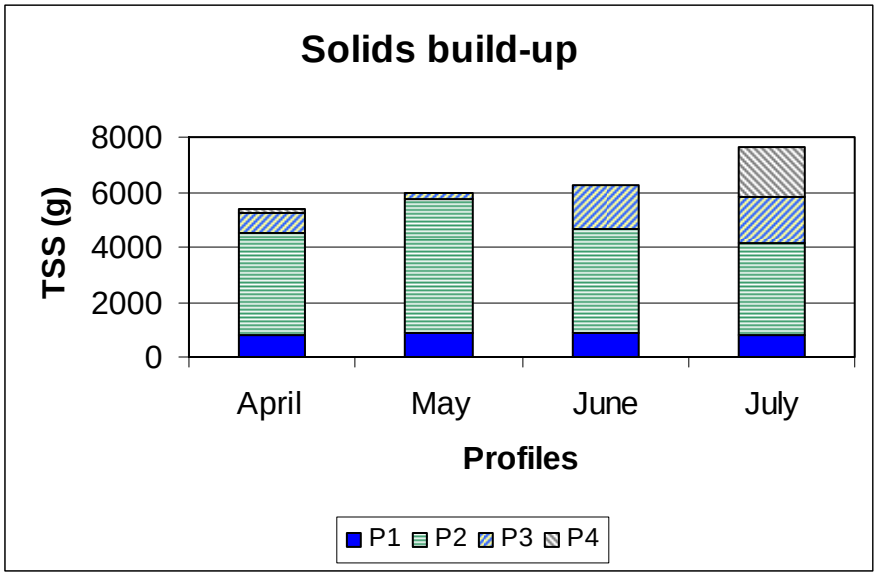

Figure 18 Solids build-up in the UASB

4.5. Mass Balance on Solids in the UASB

To establish a consumption rate and determine the amount of solids degraded inside the UASB reactor a mass balance on solids was performed. Figure 19 and table 12 show the required information.

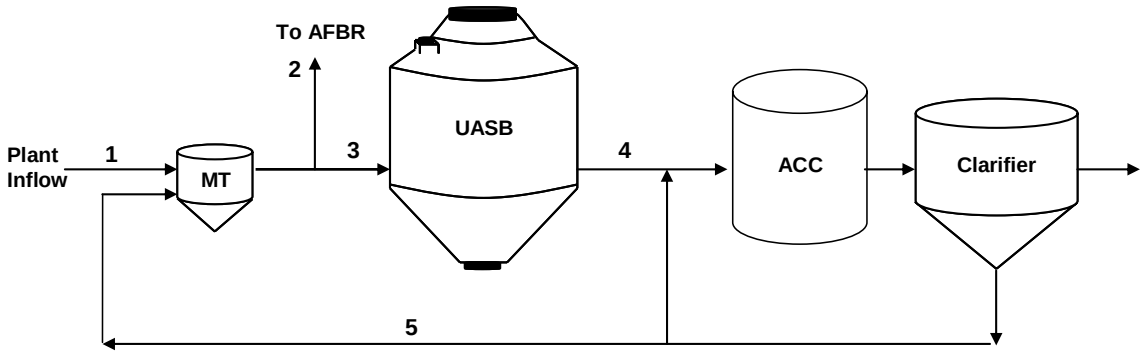

Figure 19 Mass balance diagram

Table 12 Information used to set-up the mass balance

Point	Flow Rate (L/d)	TSS (mg/L)
1	5743	130
2	2880	159
3	2880	159
4	2880	99
5	17.55	9765

The TSS values presented in table 12 are the average values of the readings taken between the first and last solid profile tests. Miss Jackeline Luque provided the information

related to stream 5. The flow rate and composition of stream 1 can be determined using the following analysis:

General mass balance on the mixing tank:

$$Q_1\rho_1 + Q_5\rho_5 = Q_2\rho_2 + Q_3\rho_3$$

Assuming constant density ($\rho_1 = \rho_5 = \rho_2 = \rho_3$)

$$Q_1 + Q_5 = Q_2 + Q_3$$

$$Q_1 = Q_2 + Q_3 - Q_5$$

$$Q_1 = 2{,}880\,L/d + 2{,}880\,L/d - 17.55\,L/d$$

$$Q_1 = 5{,}743\,L/d$$

TSS balance on the mixing tank:

$$Q_1 \times TSS_1 + Q_5 \times TSS_5 = Q_2 \times TSS_2 + Q_3 \times TSS_3$$

Then:
$$TSS_1 = \frac{Q_2 \times TSS_2 + Q_3 \times TSS_3 - Q_5 \times TSS_5}{Q_1}$$

$$TSS_1 = \frac{2{,}880\,L/d \times 159\,mg/L + 2{,}880\,L/d \times 159\,mg/L - 17.55\,L/d \times 9{,}765\,mg/L}{5{,}743\ mg/L}$$

$$TSS_1 = 130\,mg/L$$

TSS balance on the UASB reactor:

$$TSS_{feed} = TSS_{wasted} + TSS\ Accumulated + TSS\ Consumed.$$

$$TSS\ Consumed = TSS_{feed} - TSS_{wasted} - TSS\ Accumulated$$

Assuming a 106 days base for the mass balance (from April 7, 2004 to July 22, 2004), and a value equal to 2,252 g of TSS accumulated in the reactor during the 106 days (solids profiles):

$$TSS\ Consumed = 106\,d\left[Q_3\left(L/d\right)\times TSS_3 - Q_4\left(L/d\right)\times TSS_4\right] - TSS\ Accumulated$$

$$TSS\ Consumed = 106\,d\left[2{,}880\left(L/d\right)*\left(159\,mg/L\right) - 2{,}880\left(L/d\right)*\left(99\,mg/L\right)\right] - 2{,}252{,}000\,\text{mg}$$

$$TSS\ Consumed = 16{,}064{,}800\,\text{mg} = 16{,}064.8\,\text{g}$$

Table 13 Mass balance results

- TSS fed to the UASB reactor in 106 days: 48,540 g
- TSS Accumulated in the UASB reactor in 106 days: 2,252 g
- TSS degraded in the UASB reactor in 106 days: 16,065 g
- TSS recycled from the clarifier in 106 days: 18,166 g

According to these results, 33 % of the TSS fed were degraded by the action of microorganisms and 4.63 % accumulated in the UASB reactor. This yields an accumulation rate of 21.25 g/d and degradation rate of 151.6 g/d. Therefore, at the applied solids load of 1.15 kg TSS/m^3.d., 0.38 kg/m^3.d are consumed, and 0.054 kg/m^3.d are accumulated in the unit.

4.6. Operational problems

As mentioned before, due to low performance of the UASB reactor during the following months after start-up, the UASB unit was emptied and cleaned. The unit was inoculated again with 132.5 L (35 gal) of anaerobic sludge. After process stability was attained, the UASB reactor

was then operated at constant HRT (3.3 h). Table 14 shows the average value of the parameters measured.

Table 14 Average TCOD, TSS, VSS, CH_4 values before August 13, 2004

Parameter	**UASB influent**	**UASB raw effluent**	**UASB settled effluent**
Total COD, mg/L	306	205	126
Total suspended solids, mg/L	170	107	48
Volatile suspended solids, mg/L	152	99	41
Methane concentration % (V/V)	59.8		

On the other hand, the solids profiles demonstrate that continuous accumulation of solids took place in the upper zone of the sludge bed. However, after August 13, 2004 a sudden change in the UASB reactor performance was observed. Table 15 and 16 show the average value of the parameters measured and the solids profile performed on August 18, 2004, respectively.

Table 15 Average TCOD, TSS, VSS, CH4 values after August 13, 2004

Parameter	**UASB influent**	**UASB raw effluent**	**UASB settled effluent**
Total COD, mg/L	431	450	263
Total suspended solids, mg/L	239	231	81
Volatile suspended solids, mg/L	189	200	76
Methane concentration % (V/V)	25.1		

Table 16 Solids profile August 18, 2004

Port	TSS (mg/L)	VSS (mg/L)
1	25,538	16,615
2	5,723	3,897
3	203	150
4	179	133

When comparing these results with the solids profile performed on July 22, 2004, it can be noticed that large amounts of sludge inside the UASB reactor were lost. This explains the low performance of the UASB reactor and the low methane concentration at this time. Microscopic examination of the activated sludge in the ASCC which originates one of the sludge streams being fed to the UASB, detected the presence of a kind of free-swimming ciliates (crawlers). Ciliates are usually found under conditions of good floc formation and generally indicate satisfactory activated sludge operation. Ciliates are very sensitive and changes in their population can indicate the presence of toxic substances. One noticeable sign of toxicity conditions is the blooming of ciliates to higher numbers (Jenkins et al., 2003). The microscopic examination of the sludge performed in August 2004 showed an overpopulation of crawlers, and this led to the conclusion that a toxic stream had been introduced into the Marrero treatment plant.

Unfortunately, due to the low performance of the UASB reactor after August 13, 2004, the solids digestion test could not be done. However, different researchers have reported that the excess sludge from a UASB reactor treating municipal wastewater is very well stabilized and shows excellent drying characteristic (Lettinga et al., 1993). Padron (2004) using a combined

AFBR/ACC operated under the same influent conditions and a HRT of 3.4 h in the AFBR, observed complete sludge stabilization in the anaerobic unit.

4.7. Comparison between the performances of the AFBR and UASB reactors treating municipal wastewater.

As indicated before, Padron (2004) demonstrated the technical feasibility of using the AFBR/solids contact chamber for secondary treatment of municipal sewage. This research uses an UASB reactor instead of AFBR for the same purpose. The two investigations were conducted at the same location and both reactors were fed with similar flow rates and wastewater characteristics. Table 17 summarizes the results reported by both researchers.

Table 17 Summary of the results AFBR/UASB

Parameter	AFBR	UASB
Removal efficiency (%)	32	37
TCOD degraded (%)	23	33
VSS degraded (%)	32	35
Biogas composition (%V/V)	54	60
TSS removed by accumulation (%)	16.3	4.63
TSS removed by degradation (%)	15.8	33

When analyzing the results in the table 17, it can be concluded that the combined system UASB/ACC showed a better performance than the AFBR/ACC for the treatment of municipal

wastewater. While the recirculation in the UASB reactor required using a 29.8 W centrifugal pump, the AFBR needed a 372.5 W centrifugal pump to fluidize the media (activated carbon). Therefore, a significantly lower amount of energy is needed to operate the UASB. Other advantages of UASB system include the simple reactor construction, the absence of expensive support media and the long experience in full-scale plant.

Significant disadvantages of the UASB technology include:

- Long star-up periods.
- High sensitivity to toxic substances in the influent stream.
- Difficult recovery of the unit after toxic shock loads.
- Requirement of skilled operation.
- Difficult formation of granular sludge.

5. CONCLUSIONS AND RECOMMENDATIONS

The following summarizes the conclusions derived from the research conducted under this project:

- The UASB/Aeration chamber system is a very attractive alternative for municipal wastewater treatment because it has low operation and maintenance costs, and no costs associated with sludge stabilization.
- The recirculation of sludge to the UASB reactor provides an important contribution to the organic load and improves the reactor performance.
- The UASB reactor has a TSS removal efficiency of about 37%.
- Of the solids removed by the unit, 33 % were degraded by the action of microorganisms, and 4.6 % were accumulated in the reactor.
- An accumulation rate of 21.25 g/d and degradation rate of 151.6 g/d were observed in the UASB unit. Therefore, at the applied solids load of 1.15 kg TSS/m^3.d., 0.38 kg/m^3.d are consumed, and 0.054 kg/m^3.d are accumulated in the unit.
- Based on the solids profiles; it was concluded that accumulation of solids in the UASB reactor took place in the upper zone of the sludge bed. Consequently, they can be easily removed without shutting down the system and/or affecting the reactor operation.
- The UASB produces methane gas at an average rate of 6.47 ml of CH_4 per liter of sewage treated. Potentially, this energy could be reused.

- Based on the overall performance and cost associated, the combined system UASB/ACC shows to be more feasible than the combined AFBR/ACC for the municipal wastewater treatment.

The following items are suggested for further investigation.

- Improve the performance of the upflow anaerobic sludge bed, (upflow velocities, HRT)
- Perform a detailed mass balance on solids in the whole system (UASB-ASCC-Clarifier.)
- Analyze the possibility of feeding the UASB reactor with secondary sludge (high organic concentration) and use it as a biological digestion unit.
- Further research is required in order to assess the role of toxicity during the treatment of domestic wastewater in anaerobic systems.

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APPENDIX A

MARRERO WASTEWATER TREATMENT PLANT

Picture A-1 Rotating screen

Picture A-2 Distribution tank

Picture A-3 Mixing tank

Picture A-4 Upflow anaerobic sludge bed (UASB)

Picture A-5 Aeration chamber / Secondary clarifier

Picture A-6 Biogas collection and measuring system

APPENDIX B

CALCULATIONS USED TO ESTIMATE THE TOTAL PRODUCTION OF METHANE

B.1 Calculations used to estimate the total production of methane (according to Henry's law).

The following calculations are an example of the procedure used to estimate the total production of methane in a 24-hour period.

Initial conditions:

- Day: July 22-23 2004 (24-hour period)
- Volume of biogas produced: 26 L
- Temperature: 28 °C (301.15 K)
- Methane concentration: 68.8 % V/V
- Flow rate: 125 L/h

To calculate the volume of the gas at 25°C, the ideal gas law was used.

$$P \times V = n \times R \times T \qquad \Rightarrow \qquad n = \frac{P \times V}{R \times T}$$

Where:

P: Pressure

V: Volume

n: number of moles

R: constant

T: temperature

$$n_1 = n_2 \qquad \Rightarrow \qquad \frac{P_1 \times V_1}{R \times T_1} = \frac{P_2 \times V_2}{R \times T_2} \qquad \Rightarrow \qquad V_2 = \frac{V_1 \times T_2}{T_1}$$

$$V_2 = \frac{V_1 \times T_1}{T_2} = \frac{26\,L \times (298.15)K}{(301.15)K} = 25.74\,L$$

Therefore, the total volume of methane produced is:

$$Vol_{CH_4} = Vol_{gas} \times Concentration_{CH_4}$$

$$Vol_{CH_4} = 25.74\,L \times 0.688 \qquad \Rightarrow \qquad Vol_{CH_4} = 17.70L$$

The volume of sewage feed to the UASB reactor during the 24-hour period is

$$Vol_{sewage} = Q_{sewage} \times t = 125\,L/h \times 24\,h = 3000 \text{ L}$$

The volume of methane produced per volume of sewage feed to the reactor is:

$$\frac{17{,}700 ml CH_4}{3000\,L sewage} = 5.9\,ml\,CH_4/L\,sewage$$

To estimate the amount of methane gas dissolved in the effluent of the UASB reactor, it was assumed that the liquid was saturated with CH_4. The methane concentration was calculated using Henry's law. Metcalf and Eddy (2003), proposes the following equation to estimate the Henry's constant.

$$Log_{10}(H) = \left(\frac{-A}{T} + B\right)$$

Where:

H= Henry's constant at temperature T, K.

A= empirical constant. For methane A= 675.74.

T= temperature, K

B= empirical constant. For methane B= 6.88

Rearranging,

$$H = 10^{\left(\frac{-A}{T}+B\right)} \qquad \Rightarrow \qquad H = 10^{\left(\frac{-675.74}{T}+6.88\right)}$$

Therefore the Henry's constant for methane at 28°C is

$$H = 10^{\left(\frac{-675.74}{301.15}+6.88\right)} = 43264.811 atm$$

The following form of Henry's Law provides the relationship between the mole fraction of the methane in the gas above the liquid and the mole fraction of the methane in the liquid:

$$Y_{CH_4} = H \times X_{CH_4} = \frac{V_{CH_4}}{V_T}$$

Where:

Y_{CH_4} = mole fraction of methane in the gas phase

H = Henry's constant in atm

X_{CH_4} = mole fraction of methane in the liquid phase

V_{CH_4} = volume of methane in the gas mixture

V_T = total volume of the gas mixture

Therefore:

$$X_{CH_4} = \frac{\frac{V_{CH_4}}{V_T}}{H} = \frac{0.688}{43{,}264.8} = 1.59 \times 10^{-5}$$

The molar fraction of methane in the in liquid phase is defined as:

$$X_{CH_4} = \frac{n_{CH_4}}{n_{CH_4} + n_{H_2O}}$$

It is important to highlight that the number of moles of dissolved gas in a liter of water is much less than the number of moles of water (One liter of water contains 1000g = 55.6 mole). Therefore:

$$n_{CH_4} = X_{CH_4} \times n_{H_2O} = 1.59 \times 10^{-5} \times 55.6 = 8.84 \times 10^{-4}\ mole\, CH_4/L$$

The volume occupied by the moles of methane dissolved in the liquid phase can be calculated by the ideal gas law (25 °C, 1 atm).

$$V = \frac{n \times R \times T}{P} = \frac{8.84 \times 10^{-4} \times 0.082\, atm.L/mole.K \times 298.15K}{1\, atm} \times 1000\, ml/L$$

$$V = 21.61 ml\ CH_4/L$$

Finally, the total volume of methane produced per liter of sewage is

$$V_{Total} = V_{observed} + V_{dissolved} = 5.9\, ml\, CH_4/L + 21.61 ml\ CH_4/L = 27.5 ml\ CH_4/L$$

APPENDIX C

PARAMETERS MEASURED

Table C-1 UASB Influent and Effluent TCOD, mg/L

	TOTAL COD (mg/L)			
Date	**Influent**	**Mixed Effluent**	**Settled Effluent**	**Final Effluent**
6/8/2004	240.5	162	110.5	N/A
6/17/2004	266.5	135.5	94	N/A
6/18/2004	308	220.5	175	N/A
6/22/2004	406.5	253.5	185.5	N/A
6/23/2004	334.5	230	173	N/A
6/24/2004	337	x	168	N/A
6/25/2004	298	207.5	161	N/A
7/12/2004	x	311.5	89.5	N/A
7/13/2004	199	119.5	79.5	N/A
7/14/2004	215	168.5	100.5	N/A
7/15/2004	252	175.5	136.5	N/A
7/16/2004	272.5	179	114	N/A
7/22/2004	382.5	235	126.5	N/A
7/23/2004	319	266	147.5	N/A
7/24/2004	340	x	110	57
7/25/2004	365.5	207.5	116	62.5
7/28/2004	408	290	168.5	69
7/29/2004	377	272.5	196	70
7/30/2004	328.5	211	99	61.5
7/31/2004	226.5	166	95.5	56
8/1/2004	257	170.5	107	53
8/2/2004	x	158.5	106.5	38.5
8/3/2004	196	127.5	73.5	54
8/4/2004	187.5	136.5	75	52
8/5/2004	218.5	112	78.5	47
8/6/2004	354	215.5	112.5	61
8/7/2004	320	211.5	122.5	89.5
8/8/2004	x	287	138.5	106
8/9/2004	378.5	220.5	155.5	131
8/11/2004	376	278	138.5	73
8/12/2004	398.5	230	152.5	102.5

Table C-2 UASB Influent and Effluent TSS, mg/L

	TOTAL SS (mg/L)			
Date	**Influent**	**Mixed Effluent**	**Settled Effluent**	**Final Effluent**
6/8/2004	152	84	40	N/A
6/17/2004	190	101	29	N/A
6/18/2004	156	114	99	N/A
6/22/2004	201	120	43	N/A
6/23/2004	169	90	52	N/A
6/24/2004	194	96	54	N/A
6/25/2004	173	94	43	N/A
7/12/2004	x	x	60	N/A
7/13/2004	103	49	27	N/A
7/14/2004	115	99	34	N/A
7/15/2004	125	97	43	N/A
7/16/2004	125	87	28	N/A
7/22/2004	207	156	47	N/A
7/23/2004	172	x	78	N/A
7/24/2004	196	x	33	39
7/25/2004	197	118	36	32
7/28/2004	248	182	73	38
7/29/2004	200	118	60	36
7/30/2004	191	116	29	31
7/31/2004	135	94	37	30
8/1/2004	133	83	42	18
8/2/2004	x	x	92	30
8/3/2004	133	93	41	46
8/4/2004	138	96	86	59
8/5/2004	x	x	24	27
8/6/2004	169	102	39	34
8/7/2004	150	92	44	61
8/8/2004	162	128	36	68
8/9/2004	163	88	44	79
8/11/2004	204	139	51	43
8/12/2004	252	157	50	70

Table C-3 UASB Influent and Effluent VSS, mg/L

	Volatile SS (mg/L)			
Date	**Influent**	**Mixed Effluent**	**Settled Effluent**	**Final Effluent**
6/8/2004	140	78	38	N/A
6/17/2004	164	90	27	N/A
6/18/2004	136	99	87	N/A
6/22/2004	173	98	38	N/A
6/23/2004	147	73	42	N/A
6/24/2004	160	75	46	N/A
6/25/2004	147	78	38	N/A
7/12/2004	x	206	53	N/A
7/13/2004	98	50	27	N/A
7/14/2004	110	94	28	N/A
7/15/2004	117	84	38	N/A
7/16/2004	114	78	24	N/A
7/22/2004	172	138	40	N/A
7/23/2004	150	119	71	N/A
7/24/2004	177	111	30	31
7/25/2004	174	99	33	24
7/28/2004	214	150	66	23
7/29/2004	175	106	57	28
7/30/2004	168	96	27	25
7/31/2004	116	81	29	23
8/1/2004	116	70	34	14
8/2/2004	243	163	78	21
8/3/2004	115	66	35	32
8/4/2004	121	75	32	28
8/5/2004	159	72.5	21	21
8/6/2004	152	91	25	28
8/7/2004	128	81	39	53
8/8/2004	151	108	34	60
8/9/2004	153	80	37	69
8/11/2004	184	139	44	38
8/12/2004	183	110	44	57

Table C-4 Methane concentration in the biogas produced in the UASB (% V/V)

Date	% CH_4
6/8/2004	51.50
6/17/2004	53.90
6/18/2004	56.90
6/22/2004	48.90
6/23/2004	50.60
6/24/2004	47.90
6/25/2004	50.00
7/12/2004	59.10
7/13/2004	41.20
7/14/2004	39.90
7/15/2004	43.50
7/16/2004	57.50
7/22/2004	61.60
7/23/2004	68.80
7/24/2004	60.10
7/25/2004	66.80
7/28/2004	69.10
7/29/2004	73.20
7/30/2004	72.10
7/31/2004	70.80
8/1/2004	72.10
8/2/2004	69.90
8/3/2004	52.30
8/4/2004	47.80
8/5/2004	55.00
8/6/2004	68.20
8/7/2004	67.50
8/8/2004	65.50
8/9/2004	70.40
8/11/2004	68.80
8/12/2004	73.50

Table C-5 Volume of biogas produced in the UASB

Date	Biogas Volume (L)	Temp (C)
7/23/2004	26	28
7/28/2004	28.5	25.5
7/29/2004	22	26.5
8/1/2004	21	28.5
8/3/2004	50.5	27.5
8/4/2004	49	28
8/6/2004	30.5	27
8/9/2004	24	28
8/11/2004	29	28.5

Table C-6 Solids profile test 04/07/2004

Port	Initial Vol (ml)	Final Vol (ml)	Sample Vol (ml)	Weight A (g)	Weight B (g)	Weight C (g)	TSS (mg/L)	VSS (mg/L)	Average TSS (mg/L)	Average VSS (mg/L)
1	82	1000	5	1.0953	1.1096	1.1002	34878	22927	36016	24390
	82	1000	5	1.094	1.1084	1.0985	35122	24146		
	82	1000	5	1.0978	1.1134	1.1027	38049	26098		
2	53	1000	5	1.0981	1.1083	1.1009	38491	27925	34340	24906
	53	1000	5	1.0982	1.1063	1.1001	30566	23396		
	53	1000	5	1.0987	1.1077	1.1015	33962	23396		
3	61	800	5	1.1007	1.1052	1.1022	11803	7869	13115	9180
	61	800	5	1.0985	1.1046	1.1002	16000	11541		
	61	800	5	1.1023	1.1067	1.1036	11541	8131		
4	70	800	10	1.0955	1.0961	1.0957	686	457	724	495
	70	800	10	1.0966	1.0973	1.097	800	343		
	70	800	10	1.1013	1.1019	1.1013	686	686		

Table C-7 Solids profile test 05/27/2004

Port	Initial Vol (ml)	Final Vol (ml)	Sample Vol (ml)	Weight A (g)	Weight B (g)	Weight C (g)	TSS (mg/L)	VSS (mg/L)	Average TSS (mg/L)	Average VSS (mg/L)
1	88	1000	5	1.0954	1.1129	1.1011	39773	26818	41667	27955
	88	1000	5	1.0971	1.1162	1.1033	43409	29318		
	88	1000	5	1.0955	1.1139	1.1017	41818	27727		
2	89	800	5	1.0957	1.1184	1.1036	40809	26607	44764	29663
	89	800	5	1.0998	1.1245	1.1079	44404	29843		
	89	800	5	1.098	1.1253	1.1072	49079	32539		
3	79	400	5	1.1002	1.1035	1.1006	3342	2937	3511	2835
	79	400	5	1.0951	1.0986	1.0959	3544	2734		
	79	400	5	1.0954	1.099	1.0962	3646	2835		
4	10	10	10	1.1008	1.1015	1.1009	70	60	57	47
	10	10	10	1.1011	1.1015	1.1012	40	30		
	10	10	10	1.1031	1.1037	1.1032	60	50		

Table C-8 Solids profile test 06/26/2004

Port	Initial Vol (ml)	Final Vol (ml)	Sample Vol (ml)	Weight A (g)	Weight B (g)	Weight C (g)	TSS (mg/L)	VSS (mg/L)	Average TSS (mg/L)	Average VSS (mg/L)
1	88	800	5	1.079	1.0974	1.085	33455	22545	38970	25576
	88	800	5	1.0881	1.1126	1.0967	44545	28909		
	88	800	5	1.0936	1.115	1.1011	38909	25273		
2	69	600	5	1.0964	1.1154	1.1029	33043	21739	35014	22145
	69	600	5	1.0932	1.1119	1.0992	32522	22087		
	69	600	5	1.0942	1.1169	1.1039	39478	22609		
3	60	800	5	1.0983	1.1086	1.1021	27467	17333	28089	18133
	60	800	5	1.0959	1.1064	1.0995	28000	18400		
	60	800	5	1.0979	1.1087	1.1017	28800	18667		
4	79	150	50	1.0952	1.0978	1.0957	99	80	108	86
	79	150	50	1.0951	1.0979	1.0956	106	87		
	79	150	50	1.0998	1.1029	1.1005	118	91		

Table C-9 Solids profile test 07/22/2004

Port	Initial Vol (ml)	Final Vol (ml)	Sample Vol (ml)	Weight A (g)	Weight B (g)	Weight C (g)	TSS (mg/L)	VSS (mg/L)	Average TSS (mg/L)	Average VSS (mg/L)
1	43	1000	5	1.092	1.1	1.0948	37209	24186	36899	23411
	43	1000	5	1.0907	1.0981	1.0937	34419	20465		
	43	1000	5	1.0968	1.1052	1.0997	39070	25581		
2	62	800	5	1.0988	1.1108	1.1032	30968	19613	31054	20817
	62	800	5	1.0956	1.1078	1.0998	31484	20645		
	62	800	5	1.0966	1.1085	1.0999	30710	22194		
3	63	400	5	1.1006	1.1241	1.1082	29841	20190	28656	18540
	63	400	5	1.0991	1.1223	1.1075	29460	18794		
	63	400	5	1.101	1.122	1.1089	26667	16635		
4	66	600	5	1.0954	1.1003	1.097	8909	6000	8788	5636
	66	600	5	1.0949	1.0994	1.0965	8182	5273		
	66	600	5	1.1007	1.1058	1.1027	9273	5636		

VITA

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