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Anaerobic Treatment of Low Strength Wastewater

A dissertation submitted in partial satisfaction of the
requirements for the degree Doctor of Philosophy
in Civil Engineering

by

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To my parents,

Şerife and İbrahim Çakır

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PUBLICATIONS

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ABSTRACT OF THE DISSERTATION

Anaerobic Treatment of Low Strength Wastewater

by

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Anaerobic wastewater treatment is becoming an accepted technology for treating various types of wastewater. It is frequently used for medium to high strength wastewater (2,000 to 20,000 mg/L COD), but has had fewer applications to low strength wastewater (< 1,000 mg/L COD) especially in developed countries. In order to understand the applicability of anaerobic treatment for low strength wastewater, such as domestic wastewater, a literature review was performed. The review showed two main variations of anaerobic wastewater treatment techniques (anaerobic filter and upflow anaerobic sludge blanket reactor) and a number of modifications of these two themes in 34 different countries with influent COD ranging from 58 mg/L to 62,000 mg/L. To better understand and extend this technology to low strength wastewater, a dynamic model that predicts treatment efficiency as well as gas production and composition was developed. The model is composed of 12 ordinary differential equations and uses Monod-type kinetics. The model was verified using experiment results from pilot scale studies.

Anaerobic wastewater treatment has the potential to reduce greenhouse gas emissions. A model was developed to estimate greenhouse gas production from domestic wastewater treatment and to compare the differences between aerobic and anaerobic methods. The greenhouse gas contributors of CO_2 and CH_4 are included in the analysis, while N_2O is neglected. Anaerobic wastewater treatment reduces carbon dioxide emissions through energy conservation but the dissolved methane in the anaerobic reactor effluent can offset this reduction when treating low strength wastewater. The cross over points range from 300 to 700 mg/L influent wastewater COD, and are dependent on the relative efficiency of the aerobic treatment system. Anaerobic wastewater treatment becomes favorable for concentrations higher than the cross over values. A technology to recover dissolved methane would make anaerobic treatment favorable at nearly all influent strengths.

This research documents the opportunities for anaerobic wastewater treatment. The results of the extensive literature review and model predictions suggest that anaerobic treatment is promising and favorable for low strength wastewaters under certain conditions.

CHAPTER 1

INTRODUCTION

1.1 DISSERTATION ORGANIZATION

This dissertation includes a compilation of papers written during dissertation research. Chapter 2 is an extensive literature review of anaerobic treatment processes and modeling of anaerobic filters. Chapter 3 includes a model on greenhouse gas production from wastewater treatment and compares the aerobic and anaerobic methods. The manuscripts in Chapter 2 and Chapter 3 have been submitted to Water Research and are currently under review for publication. The Appendix A includes an earlier version of a paper on modeling work that is published in Journal of Environmental Health and Science in 2003.

1.2 OVERVIEW OF THE PROBLEM

Wastewater treatment is essential to maintain public health and prevent surface and groundwater pollution. The Clean Water Act Amendments passed in 1972 established national goals and objectives for wastewater management in US, requiring all municipalities, with only few exceptions, to have biological secondary wastewater treatment. The activated sludge process is the most common method of secondary wastewater treatment in US. The process provides high treatment efficiency but has some disadvantages such as high investment and operational costs, high maintenance and high energy requirement due to aeration.

Many people in the developing countries lack basic sanitation services due to limitations in financial resources, lack of environmental awareness and less incentive to meet regulations such as low fines. A great portion of the wastewater in these countries is either discharged untreated into surface waters or disposed in uncontrolled ways. The rest is mostly treated with simple methods such as stabilization ponds and septic tanks. Stabilization ponds require large amounts of land and might not be feasible if the land is expensive or not available, such as in urban locations. There are a small number of activated sludge plants but these may not operate properly due to lack of maintenance, and many do not have sludge treatment facilities. There is a need for simple, economical and energy efficient systems to eliminate organic contaminants, suspended solids and pathogens.

Anaerobic treatment has traditionally been used to treat sludges and high strength wastewaters. Escalation of energy costs during the energy crisis in the early 1970's stimulated an interest in research and application of new anaerobic systems such as the anaerobic filter (AF) and the upflow anaerobic sludge blanket reactor (UASB) for domestic and industrial wastewater treatment (Bowker, 1983; Switzenbaum, 1995). The new anaerobic systems are able to treat low strength wastewaters by maintaining high solids retention time, which is independent of the hydraulic retention time.

The new anaerobic systems may provide economical and efficient solutions for domestic wastewater when compared to conventional aerobic systems. They have several advantages over the conventional systems: they are simple without the need for highly trained staff, energy efficient as they do not require aeration and have the potential to produce methane, produce less sludge, do not require complex equipment and are easy to operate. These systems and especially the UASB have had practical applications during the past 20 years in lesser developed countries such as Brazil, Colombia, India and Mexico. Anaerobic treatment for domestic wastewater is especially suitable for tropical and sub tropical regions, for rural areas such as villages or small communities and for coastal and tourist cities (Vieira, 1988; Yu *et al.*, 1997; Seghezzo *et al.*, 1998; Monroy *et al.*, 2000 Foresti, 2002).

In developed countries such as US anaerobic reactors have rarely been used for treatment of low strength wastewaters. The main obstacle for this is the preexisting wastewater treatment infrastructure and the lack of experience with anaerobic systems. Also cost of energy, landfilling sludge, fuel and surcharge rates are low and companies require a quick payback. One potential use of anaerobic reactors is to upgrade overloaded activated sludge plants (Switzenbaum, 1995; Alderman *et al.*, 1998). The European Union's new environmental legislation related to sludge disposal and renewable energy generation may spur interest in anaerobic treatment in the future (Zakkour *et al.*, 2001).

The greenhouse gas emissions have increased during the last 200 years due to anthropogenic activities. Wastewater treatment can contribute to greenhouse gases through production of CH_4 or CO_2 from treatment processes or from CO_2 produced from the energy required for treatment. Although anaerobic treatment is more energy efficient than aerobic treatment and seems to produce less greenhouse gas, a detailed study of these systems shows that the dissolved methane lost in the anaerobic reactor effluent can offset any reductions in carbon dioxide emissions for low strength wastewaters. If a technique to capture dissolved methane can be developed then anaerobic treatment would be favorable at nearly all influent strengths.

1.3 OBJECTIVES

This dissertation's main concern is treatment of low strength wastewaters. The objectives are:

1. To review the previous anaerobic treatment processes especially focusing on anaerobic filters, upflow anaerobic sludge blanket reactors, modified upflow anaerobic sludge blanket reactors and modified anaerobic filters.
2. To evaluate the potential use of anaerobic filters, upflow anaerobic sludge blanket reactors and modified reactors for low strength wastewater.
3. To develop a model that can be used for anaerobic filters.
4. To demonstrate treatment efficiency and gas production and composition of domestic wastewater using this model.
5. To estimate greenhouse gas production from domestic wastewater treatment
6. To investigate the differences between aerobic and anaerobic methods in greenhouse gas production.

1.4 REFERENCES

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CHAPTER 2

ANAEROBIC TREATMENT OF LOW STRENGTH WASTEWATER

ABSTRACT

Anaerobic filters (AFs) and upflow anaerobic sludge blanket (UASB) reactors are finding wide-scale acceptance for treating various types of wastewater. They are frequently used for medium to high strength wastewater (2,000 to 20,000 mg/L COD), but have fewer applications to low strength wastewater (< 1,000 mg/L COD). In order to understand the applicability of anaerobic treatment for low strength wastewater, such as domestic sewage, a literature review was performed and a dynamic mathematical model was developed. The review showed two main variations of anaerobic wastewater treatment techniques (AF and UASB) and a number of modifications of these two themes. A total of 136 references were found that documented anaerobic wastewater treatment, ranging in strength from 58 mg/L to 62,000 mg/L COD in 34 different countries. A Monod-type kinetic model, which predicts treatment efficiency and gas production, was developed to describe some of the literature observations. The results of the extensive literature review and model predictions suggest that anaerobic treatment is promising and economical for treating low strength wastewater, although production of a useful biogas fuel by-product is doubtful. This is contrary to experience in the United States where anaerobic wastewater treatment is seldom performed.

Keywords: Anaerobic filter; domestic wastewater; EGSB reactor; hybrid anaerobic filter; low strength wastewater; modeling; UASB

2.1 INTRODUCTION

Anaerobic treatment has traditionally been used for treatment of sludges and especially those derived from wastewater treatment plants. Treatment is provided to reduce sludge mass, increase dewaterability, and reduce pathogen content while producing a useful energy by product - methane gas. The restriction to sludges or high strength wastewater existed because elevated temperatures were required for the slow growing methanogens, and the methane produced from the concentrated sludges was required for heating. Figure 2.1 shows the heating value of digester gas using the stoichiometric methane yield from chemical oxygen demand (COD) destruction. The heating value is plotted as a function of wastewater strength, and specific points for mesophilic (37°C) and thermophilic (55°C) conditions are shown, assuming an ambient temperature of 20°C. The graph shows two lines: 100% heat conversion efficiency with 100% COD oxidation efficiency, and 60% COD oxidation with 83% heat conversion efficiency (overall 50%), which is typical of modern boiler and heat exchanger efficiency (the graph neglects any heat recovery that might be obtained from the digested sludge). For example increasing the temperature of a wastewater with an ambient temperature of 20°C to 37°C requires over 11,000 mg/L COD destruction at overall 50% conversion efficiency. For this reason, it is uneconomical to heat low strength wastewaters for anaerobic treatment.

Young and McCarty (1969) and others (Witt *et al.*, 1979; Lettinga & Vinken, 1980; Braun & Huss, 1982) extended anaerobic treatment to high and medium strength wastewater by developing methods to retain cells in the reactors. The new anaerobic systems such as the anaerobic filter (AF), upflow anaerobic sludge blanket (UASB) and hybrid reactors (a combination of UASB and AF) allow treatment of low strength wastes such as domestic wastewater by maintaining long solids retention time (SRT) independent of the hydraulic retention time (HRT). This reduces or eliminates the need for elevated temperatures.

The new anaerobic systems may provide economical and efficient solutions for domestic wastewater when compared to conventional aerobic systems. They have several advantages over the conventional systems: they are simple, energy efficient, produce less sludge, do not require complex equipment and are easy to operate. These systems have had worldwide practical applications. Anaerobic treatment for domestic wastewater is especially suitable for tropical and sub tropical regions, for rural areas such as villages or small communities with a need for compact, simple systems without highly qualified staff and sophisticated equipment and for coastal and tourist cities.

The objective of this paper is to review the previous anaerobic treatment processes, evaluate their potential use for low strength wastewater, describe a model that can be used for AFs, and demonstrate treatment efficiency of domestic wastewater.

2.2 ANAEROBIC TREATMENT PROCESS

Anaerobic treatment of waste is a complex biological process involving several groups of microorganisms (Cha & Noike, 1997; Harper & Pohland, 1997; Jianrong *et al.*, 1997). In general complex wastes are stabilized in three basic steps: hydrolysis, acid fermentation and methanogenesis. In the acid fermentation step the organic waste is decomposed into lower fatty acids such as acetic and propionic by acid forming bacteria. In methanogenesis these fatty acids are broken down into CO_2 and CH_4 by methanogens (Speece, 1996). The growth rate of the methanogens is low and is usually the rate-limiting step. Long SRT is required to retain the slow growing methanogens. For complex or particulate substrates, hydrolysis to fermentable compounds can be rate limiting (Pavlostathis & Giraldo-Gomez, 1991; Valentini *et al.*, 1997).

Conventional anaerobic digestion uses a completely mixed reactor and is mainly used to digest municipal sludge. This process is limited because the HRT is equal to the SRT, which results in large reactor volumes and low volumetric loading rates. The minimum SRT required is approximately 10-15 days at $35^{\circ}C$. The history of early work regarding anaerobic wastewater treatment can be found in Jewell (1987) and Seghezzo *et al.* (1998).

A major improvement over complete mixing was the anaerobic contact process (Schroepfer *et al.*, 1955, 1959). This process used a completely mixed reactor followed by a settling tank, analogous to the activated sludge process, to separate and recycle cells to maintain high SRT with low HRT. The mixing of the reactor was done either with mechanical stirrers or by recirculating the biogas. A major disadvantage of the process was the need for a degasifier between the digester and the settling tank to prevent gas lifting of sludge particles. This process has been used for treating sugar, distillery, yeast, dairy and meat processing wastewater. The removal efficiencies ranged between 65-98% depending on different substrates and operational conditions (Nahle, 1991).

Coulter (1957) was the first to develop AF process. Wastewater flows through rock or synthetic media, which retains biomass on the surfaces and/or in the voids. This process was largely forgotten until Young and McCarty (1969) studied the treatment of a protein-carbohydrate wastewater (1500-6000 mg/L COD) at 25°C, at organic loading rates (OLR) of 0.96-3.40 kg COD/m^{3}d. Pretorius (1971) used a modified digester (similar to a UASB) followed by a biophysical filter to treat 500 mg/L of raw sewage at 24 hr retention time at 20°C. The digester concentrated the suspended solids and hydrolyzed the complex molecules, which were broken down to methane and carbon dioxide in the filter. He achieved COD removal efficiencies as high as 90%, and concluded that hydraulic loading was a better design parameter than waste concentration for low strength wastewater.

The UASB process was later developed, which employs a dense granular sludge bed at the bottom. A gas solids-separator is used at the top to capture digester gas while preventing solids from leaving the reactor (Lettinga & Hulshoff Pol, 1986, 1991; Souza, 1986). Lettinga (1980) treated raw domestic sewage (140-1100 mg/L COD) at ambient temperatures of 8-20°C using a UASB. Removal efficiency of 65-90% was achieved for influent COD greater than 400 mg/L and an efficiency of 50-65% was obtained for COD's less than 300 mg/L. Temperature had limited effect on removal efficiency.

More recently the UASB and AF processes have been modified to use the best features of each. The expanded granular sludge bed (EGSB) reactors use recycle to improve wastewater/sludge contact. EGSB reactors are designed with a higher height/diameter ratio as compared to UASB reactors, to accommodate an upward recycle flow (liquid superficial velocity) of 4 to 10 m/h (Seghezzo *et al.*, 1998). The hybrid reactor is a combination of UASB and AF reactor concepts. Packing media is placed in the top of a UASB (Guiot & Van den Berg, 1985; Di Berardino, 1997).

There are some other anaerobic processes such as anaerobic expanded/fluidized bed (Jewell, 1987; Jewell *et al.*, 1981) and anaerobic sequencing batch reactor (Banik & Dague, 1997; Ndon & Dague, 1997), which were also used for the treatment of low strength wastewater. These processes will not be covered in this paper.

The following sections describe the early development of each process with a detailed list of the published demonstrations or applications of each technology. The tables are divided by classifying the studies into laboratory, pilot, demonstration or full-scale application.

2.3 ANAEROBIC FILTERS

Table 2.1 shows 24 previously published studies of laboratory scale (< 10 L) AFs. Wastewater strengths ranged from 54,000 mg/L COD highest (Veiga *et al.*, 1994) to 207 mg/L COD lowest (Viraraghavan & Varadarajan, 1996). Pilot and large pilot scale (10 to 100 L, and 100 to 1000 L, respectively) investigations are shown in Table 2.2 and there are 24 citations. They range in concentrations from 26 mg/L TOC (~ 65 mg/L COD) to

62,000 mg/L COD. Table 2.3 shows the demonstration and full-scale installations (13 citations), influent wastewater strengths ranged from 60 mg/L BOD to 68,400 mg/L soluble TOD. Only 20 citations were found for low strength wastewater (< 1000 mg/L COD), and none were full-scale installations. Three (Chung, 1982; Kobayashi *et al.*, 1983; Abramson, 1987) were from our laboratory and the partial results will be used later in the model calibration. The lack of full-scale installations suggests that the technology is not yet accepted or requires further development. This may be in part due to lack of experience or preference for UASBs.

Hudson *et al.* (1978) used an AF to treat low strength shellfish processing wastewater with COD removal efficiencies ranging from 33 to 81% with 8 to 75 hr HRT with two different packing media. Koon *et al.* (1979) used an AF to treat domestic wastewater, and found BOD removal efficiency from 43 to 60 % at 12-48 hr HRT. His cost analysis showed that for a design flow of 189 m^3/d about 20% reduction in total annual costs could be achieved over the activated sludge process. Genung *et al.* (1979) reported 55% BOD removal from domestic wastewater in a demonstration facility. Kobayashi *et al.* (1983) evaluated a 16 L AF treating domestic wastewater at three temperatures (20, 25 and 35°C), and found an average COD removal of 73%. Abramson (1987) showed 40 to 90% TOC removal in large pilot scale reactors. Iyo *et al.* (1996), Kim *et al.* (1997), Bodik *et al.* (2000), Elmitwalli *et al.* (2000), Kondo and Kondo (2000), Camargo and Nour (2001) also had varied success in treating low strength waste in anaerobic filters.

In contrast to low strength wastewater, AF treatment of medium and higher strength wastewater has been more extensively investigated. Chian and DeWalle (1977), Frostell (1981), Bowker (1983), Guerrero *et al.* (1997), Leal *et al.* (1998), Wilson *et al.* (1998), Ince *et al.* (2000), Alves *et al.* (2001), Garrido *et al.* (2001) are some notable examples.

2.4 UPFLOW ANAEROBIC SLUDGE BLANKET REACTORS

Tables 2.4 and 2.5 show the laboratory, pilot, demonstration and full-scale investigations of UASBs for wastewater treatment. There are 56 citations and 44 of them address low strength wastewater. More than 20 are full-scale investigations. The UASB has had much greater acceptance but not in the United States. The cited full-scale installations are in Europe, South America and Southeast Asia (Bowker, 1983; Frankin, 2001).

Lab scale studies using UASBs to treat low strength wastewater began as early as 1976, with Lettinga *et al.* (1983) performing many of the early studies. De Man *et al.* (1986) and Campos *et al.* (1986) were among the first to demonstrate low strength wastewater treatment in UASBs in large-scale reactors. Table 2.5 shows many recent investigations using low strength wastewater. All are outside the United States. Draaijer *et al.* (1992) used a 1200 m^3 UASB reactor to treat municipal wastewater in Kanpur, India. The highest removal efficiency obtained was 74%. Vieira *et al.* (1994) performed a full-scale study on sewage discharged from low-income community in Sumare, Brazil, obtaining 74% removal efficiency. In another Brazilian study, Chernicharo and Cardoso

(1999) treated domestic sewage from small villages using a partitioned UASB reactor. The partitioned reactor included three digestion chambers working in parallel to accommodate influent flow rate fluctuations. Removal efficiency reached 79% at HRT of 7.5 hr. The cost evaluation showed that partitioned UASB reactor was much less expensive than the conventional UASB reactor. Karnchanawong *et al.* (1999) investigated UASB domestic wastewater treatment in Thailand obtaining 53-69% BOD removal efficiency. Karnchanawong *et al.* (1999) also studied domestic wastewater treatment from apartment complexes in Bangkok. The removal efficiency ranged from 60 to 76%. He suggested an HRT of 10-12 hr as a design criterion for full-scale UASB reactors to achieve 75% BOD removal.

2.5 MODIFIED UPFLOW ANAEROBIC SLUDGE BLANKET REACTORS AND ANAEROBIC FILTERS

Tables 2.6 and 2.7 show the modified reactor studies. Kennedy and Van den Berg (1982) among others, investigated downflow AFs with varying success. Guiot and Van den Berg (1985) were the first to use packing above a UASB to improve efficiency. After 1989 there are 16 reported investigations using a hybrid AF, and 4 used low strength wastewater. Elmitwalli *et al.* (1999, 2001) used the hybrid concepts to treat domestic wastewater. Again, the experience is all outside of the United States, and there are currently no full-scale installations treating low strength wastewater.

Table 2.7 lists the modified UASBs for 9 investigations for domestic or low strength wastewater and several more treating septic tank effluents. Only one study was at full-scale for low strength wastewater, and all were outside the United States. De Man *et al.* (1988) was the first to use an EGSB to treat low strength wastewater, obtaining 20 to 60% soluble COD removal. Van der Last and Lettinga (1992) investigated an EGSB reactor treating domestic sewage, obtaining about 30% COD removal efficiency. EGSB reactors have also been used for industrial wastewater (Kato *et al.*, 1997).

2.6 SUMMARY OF PREVIOUS WORK

UASBs, AFs and modified reactors have demonstrated excellent performance for high and medium strength wastewater. There are fewer but significant examples for low strength wastes, in different parts of the world but mostly in developing countries with tropical to moderate climates.

The efficiencies ranged from 5% COD removal to as high as 99% COD removal. Temperatures were as low as 2°C. Hydraulic retention times ranged from 1.5 hrs to 10 days for UASBs and 1.5 hrs to 74 days for AFs. The anaerobic systems alone were usually insufficient to meet secondary discharge definitions (less than 30 mg/L BOD_5 and 30 mg/L TSS), and to achieve nutrient removal.

In order to overcome these shortcomings, aerobic reactors (such as sequencing batch reactors (SBRs), tricking filters, activated sludge, stabilization ponds, packed columns, biofilters, rotating biological contactors (RBCs), hanging sponge cubes, etc.) were used for polishing. Also, partitioned or staged anaerobic reactors were suggested for wastewater with high suspended solids or with high influent fluctuations, and for better colloidal suspended solids removal.

Gas composition and production have been less frequently reported (Donovan *et al.*, 1979; Koon *et al.*, 1979; Kobayashi *et al.*, 1983; Abramson, 1987; Noyola *et al.*, 1988; Barbosa & Sant'Anna, 1989; Vieira & Garcia, 1992; Singh *et al.*, 1996; Agrawal *et al.*, 1997; Chua *et al.*, 1997; Lomas *et al.*, 1999; Lacalle *et al.*, 2001), but are a function of different factors such as temperature, waste type and strength. Figure 2.2 shows gas composition as a function of wastewater strength or substrate concentration. For low substrate concentration, less methane and carbon dioxide are produced, and they are insufficient to displace the nitrogen that is stripped from solution. Methane is more soluble than nitrogen (Henry's coefficients are 0.0017 mM/mmHg and 0.000845 mM/mmHg for CH_4 and N_2, respectively), which means the dissolved methane is a large fraction of the total produced, for low strength applications. Therefore biogas from low strength processes will have measurable to medium nitrogen content.

The reactors for low strength wastewater have usually been operated at low HRTs ranging from 3 to 24 hrs. Waste type, OLR, HRT, start up conditions, temperature, porosity, media configuration, feeding policy, flow pattern, and gas separation devices are some of the factors that need special attention in order to obtain good solids retention and prevent operational problems. Generally, the daily fluctuations in influent wastewater did not have an adverse effect on removal efficiency.

The previously cited studies show good success with anaerobic wastewater treatment at ambient temperatures, but there are few full-scale implementations, especially in the United States and especially for anaerobic filters. This review and the following research were performed in order to better understand anaerobic treatment and in the hopes that it can be more frequently adopted. In order to better understand the application for low strength wastewater, we developed a model that can predict reactor efficiency, gas production and gas composition as a function of key process variables.

2.7 MODEL DEVELOPMENT

The model developed is a dynamic model describing anaerobic treatment using anaerobic filters. The model is based in part on earlier models developed by Andrews (1969, 1971). The model predicts treatment efficiency as well as gas production and composition. The model includes an acid fermentation step and does not require methanogenesis to be rate limiting. Hydrolysis or break down of particulates and complex substrates were not considered, since the model is limited to domestic wastewater treatment following primary clarification. Applications to industrial wastewaters or wastewaters with high solids should be examined by other techniques. This issue of which step is rate limiting can be complicated, related to types of substrate and the way "rate limiting" is defined; the reader is directed to Pavlostathis and Giraldo-Gomez (1991) for a more complete discussion. Lindgren (1983) and others have used similar approaches. The model is restricted to low strength influents, and does not require the more advanced concepts that separate substrates and biomasses into different pools (Mosey, 1983; Moletta *et al.*, 1986; Suidan *et al.*, 1994; Jeyaseelan, 1997; Batstone *et al.*, 2000; Karama *et al.*, 2000; Batstone *et al.*, 2002).

The model includes the physical, chemical and biological interaction between gas, liquid and biological phases, which are shown in Figure 2.3. The model is composed of

12 ordinary differential equations. The general material balance equation (Accumulation = Input – Output + Production – Utilization) was used for the corresponding 12 state variables: substrates and biomasses in the biological phase for acid fermentation and methanogenesis steps; CO_2, N_2 and CH_4 partial pressures in the gas phase; alkalinity, dissolved CO_2, N_2, CH_4 and NH_3 in the liquid phase.

2.8 STOICHIOMETRY

A generalized stoichiometric relationship showing the conversion of acetic acid to methane and carbon dioxide with the synthesis of biomass and the decay of biomass is given respectively in equations (2.1) and (2.2). For acetic acid the carbon dioxide and methane yield will be equal to each other as shown in equation (2.5).

$$CH_3COOH + Y_{XS2}.Y_{NH_3X_2^1}NH_3 \rightarrow Y_{XS2}C_5H_7NO_2 + Y_{XS2}.Y_{CO_2X_2^1}CO_2 + Y_{XS2}.Y_{CH_4X_2^1}CH_4 + aH_2O \tag{21}$$

$$C_5H_7NO_2 + bH_2O \rightarrow Y_{CO_2X_2^2}CO_2 + Y_{CH_4X_2^2}CH_4 + Y_{NH_3X_2^2}NH_3 \tag{2.2}$$

$$a = 2 - 2Y_{XS2}(1 + Y_{CO_2X_2^1}) \tag{2.3}$$

$$b = 3 \tag{2.4}$$

$$Y_{CO_2X_2^1} = Y_{CH_4X_2^1} = 0.5(\frac{2}{Y_{XS2}} - 5) \tag{2.5}$$

from oxidation-reduction balance

2.9 BIOLOGICAL PHASE

The rate of change of complex substrate and acetate concentration in the reactor at any time depends on the influent and the utilization of complex substrate and acetate for biomass growth (eqs. 2.7 and 2.8). Monod-type kinetics in equation (2.6) was used to describe the utilization of substrate. The growth rates are a function of temperature.

$$\mu_1 = \frac{\mu_{\max 1} S_1}{(K_{S1} + S_1)} \qquad \mu_2 = \frac{\mu_{\max 2} S_2}{(K_{S2} + S_2)} \tag{2.6}$$

$$\frac{dS_1}{dt} = \frac{Q}{V}(S_{1O} - S_1) - \frac{\mu_1 X_1}{Y_{XS1}} \tag{2.7}$$

$$\frac{dS_2}{dt} = \frac{Q}{V}(S_{2O} - S_2) - \frac{\mu_2 X_2}{Y_{XS2}} + \frac{(1 - Y_{XS1})}{Y_{XS1}}\mu_1 X_1 \tag{2.8}$$

The rate of change of acidogen and methanogen concentrations in the reactor is a function of the influent and effluent biomass concentrations and the biomass growth and decay in the reactor (eqs. 2.9 and 2.10). In AF the biomass concentration in the reactor is much higher than the effluent biomass concentration as the biomass is retained in the packing media.

$$\frac{dX_1}{dt} = \frac{Q}{V}(X_{1O} - X_{1E}) + (\mu_1 - k_{1d}) X_1 \tag{2.9}$$

$$\frac{dX_2}{dt} = \frac{Q}{V}(X_{2O} - X_{2E}) + (\mu_2 - k_{2d}) X_2 \tag{2.10}$$

The production and utilization of dissolved CO_2, CH_4 gases and NH_3 during the biological reactions are given in equation (2.11). The production and utilization rates of CO_2, CH_4 and NH_3 during methanogen growth are shown by r_1, r_3 and r_5 respectively.

Similarly the production rates of CO_2, CH_4 and NH_3 during methanogen decay are represented by r_2, r_4 and r_6. The production of NH_3 during acidogen growth and decay is shown by r_7 and r_8.

$$\begin{aligned}
r_1 &= \mu_2 X_2 Y_{CO_2 X_2^1} \\
r_2 &= k_{2d} X_2 Y_{CO_2 X_2^2} \\
r_3 &= \mu_2 X_2 Y_{CH_4 X_2^1} \\
r_4 &= k_{2d} X_2 Y_{CH_4 X_2^2} \\
r_5 &= -\mu_2 X_2 Y_{NH_3 X_2^1} \\
r_6 &= k_{2d} X_2 Y_{NH_3 X_2^2} \\
r_7 &= \mu_1 X_1 Y_{NH_3 X_1^1} \\
r_8 &= k_{1d} X_1 Y_{NH_3 X_1^2}
\end{aligned} \tag{2.11}$$

2.10 LIQUID PHASE

The net rate of CO_2, CH_4 and N_2 transfer between the liquid and gas phases can be expressed by two-film theory in equation (2.12). Henry's Law was used to determine the concentration of the gases in the liquid phase at equilibrium with the partial pressure of the gases in the gas phase. Henry's Law constants are a function of the temperature.

$$T_{Gi} = K_L a_i \left(C_i^* - C_i\right) \tag{2.12}$$

$$C_i^* = K_{Hi} P_i$$

$$K_{Hi} = f(Temp)$$

The alkalinity balance is shown in equation (2.13):

$$Z = \left[HCO_3^-\right] + 2\left[CO_3^{-2}\right] + \left[NH_3\right] + \left[OH^-\right] - \left[H^+\right] \tag{2.13}$$

The mass balance for the total carbonic acid system is shown in equation (2.14)

$$\left[CO_2\right]_T = \left[CO_2\right]_D + \left[HCO_3^-\right] + \left[CO_3^{-2}\right] \tag{2.14}$$

$$f_{HCO_3^-} = \frac{1}{\left(1 + \frac{K_2}{\left[H^+\right]} + \frac{\left[H^+\right]}{K_1}\right)} \tag{2.15}$$

$$f_{CO_3^{-2}} = \frac{1}{\left(1 + \frac{\left[H^+\right]}{K_2} + \frac{\left[H^+\right]^2}{K_1 K_2}\right)} \tag{2.16}$$

$$f_{CO_2} = \frac{1}{\left(1 + \frac{K_1}{\left[H^+\right]} + \frac{K_1 K_2}{\left[H^+\right]^2}\right)} \tag{2.17}$$

$$f_{NH_3} = \frac{1}{\left(1 + \frac{\left[H^+\right]}{K_{NH_3}}\right)} \tag{2.18}$$

The rate of change of alkalinity in the reactor (eq. 2.19) depends on the influent alkalinity and change in the ammonia concentration in the liquid phase.

$$\frac{dZ}{dt} = \frac{Q}{V}(Zo - Z) + \frac{Q}{V}\left[f_{NH_3}(NH_3)_T - (NH_3)_0\right] \tag{2.19}$$

The rate of change of total carbonic acid concentration in the reactor (eq. 2.21) is a function of the influent carbonic acid concentration and gas transfer rate of dissolved carbon dioxide (eq. 2.20) and the rate of dissolved carbon dioxide production during biological growth and decay.

$$T_{GCO_2} = K_L a_{CO_2}\left((CO_2)_D^* - f_{CO_2}(CO_2)_T\right) \tag{2.20}$$

$$\frac{d(CO_2)_T}{dt} = \frac{Q}{V}\left[(CO_2)_{TO} - (CO_2)_T\right] + T_{GCO_2} + r_1 + r_2 \tag{2.21}$$

The rate of change of dissolved N_2 in the reactor (eq. 2.22) depends on the influent N_2 and the gas transfer rate of N_2. The N_2 gas does not undergo any biological or chemical reaction in the reactor.

$$\frac{d(N_2)_D}{dt} = \frac{Q}{V}\left[(N_2)_{DO} - (N_2)_D\right] + T_{GN_2} \tag{2.22}$$

The rate of change of dissolved methane gas in the reactor (eq. 2.23) is a function of the influent methane concentration and the gas transfer rate of methane and the rate of dissolved methane production during biological growth and decay.

$$\frac{d(CH_4)_D}{dt} = \frac{Q}{V}\left[(CH_4)_{DO} - (CH_4)_D\right] + T_{GCH_4} + r_3 + r_4 \tag{2.23}$$

The rate of change of total ammonia in the reactor (eq. 2.24) depends on the influent ammonia concentration and the reaction rates during biological growth and decay.

$$\frac{d(NH_3)_T}{dt} = \frac{Q}{V}\left[(NH_3)_{TO} - (NH_3)_T\right] + r_5 + r_6 + r_7 + r_8 \tag{2.24}$$

2.11 GAS PHASE

The partial pressures of CO_2, CH_4 and N_2 gases in the gas phase are a function of the gas transfer rate and the outflow from the gas phase (eq. 2.25).

$$\frac{dP_i}{dt} = -P_T D T_{Gi}\left(\frac{V}{V_g}\right) - P_i\left(\frac{Q_g}{V_g}\right) \tag{2.25}$$

$$D = R(273.15 + Temp) \tag{2.26}$$

$$Q_i = -DVT_{Gi} \tag{2.27}$$

$$Q_g = \sum_{i=1}^{i=3} Q_i + Q_{H_2O} \tag{2.28}$$

2.12 MODEL RESULTS

Kobayashi *et al.* (1983) and Abramson's (1987) AF data were used to calibrate the model. The input values for specific growth rate, decay rate, saturation constant, yield coefficients and similar values were chosen according to the literature review (Speece, 1996; Batstone *et al.,* 2002). Figure 2.4 shows the calibration graph for removal rate and effluent substrate concentration as a function of solids retention time. Pairs of points are shown, with one pair representing the observed data, and the second pair representing the simulation for those conditions. The simulations are not on a smooth line, as shown in later figures, since each observed data point was collected at different temperatures, hydraulic retention times and influent substrate concentrations. Model predictions of the gas composition of the effluent as a function of solids retention time and influent concentration are given in Figures 2.5 and 2.6. The model accurately predicts the high nitrogen partial pressure for low strength wastewater. This is due to the dissolved nitrogen in the influent wastewater.

2.13 DISCUSSION

A survey of producers of anaerobic processes in both Europe and the United States revealed no full-scale examples of any anaerobic technology for treating low strength or domestic wastewater in the North America. The manufacturers concluded that energy costs were inadequate to justify the additional capital cost of anaerobic processes, which generally required longer hydraulic retention times than aerobic processes. Additional savings is required to motivate owners or operators to overcome the economic disincentives or prejudice against using anaerobic technology in this new role. Odors and stability problems are routinely assumed.

The literature review and the model prediction suggest that AFs can achieve sufficient efficiency to approach secondary treatment standards. For example Hudson *et al.* (1978), Genung *et al.* (1979), Koon *et al.* (1979), Noyola *et al.* (1988), Bodik *et al.* (2002), Elmitwalli *et al.* (2002) found AFs to successfully treat low strength wastewaters to COD and TSS concentrations of 37-160 mg/L and 10-20 mg/L at ambient temperatures (13- 29°C) in laboratory, pilot or demonstration scale facilities (Tables 2.1, 2.2 and 2.3). The range of retention times was 10 to 24 hours for successful treatment. This compares to reactor retention times in the 4 to 6 hours for activated sludge plant aeration tanks and 1 to 2 hours in secondary clarifiers. A comparison of capital costs would need to consider the various reactor sizes as well as reactor complexities (e.g., clarifier internal mechanisms, pumps, etc.) and the cost associated with aeration equipment (e.g., blowers, diffusers, etc.). Energy for pumping for the two competing systems would be similar for wastewater flows, although the activated sludge process would have a small disadvantage because of sludge recycle pumping.

Aeration costs are the main disadvantage of the activated sludge process. A complete economic comparison would be difficult to make and site specific, but the energy cost of aeration is not difficult to calculate. Modern, fine pore diffusers can have Standard Aeration Efficiencies of 3.6 to 4.8 kg O_2/kW-hr, which becomes 0.7 to 2.6 kg O_2/kW-hr depending upon process conditions (Rosso *et al.*, 2001).

These costs can be used to calculate the potential savings for an anaerobic treatment plant. A 1 m^3/sec plant treating primary effluent with a COD of 300 mg/L might require 17,300 kW-hr per day, assuming 60% of the COD is oxidized with 40% being converted to biomass or untreated. The energy value at US$0.10/kW-hr is US$1730/day. The net present worth of this daily savings at 5% interest over 5 years is US$2.8 million. This is the capital that could be used to defer the additional reactor cost. At 1 m^3/sec flow rate, and increase in combined reactor retention time from 6 hr to 12 hr is 21,600 m^3. The savings is not generally adequate to justify the likely cost of the additional reactor volume, but could be adequate in some situations, due to site-specific conditions. Additionally, if a 20-year payback were feasible, the savings would be US$ 7.9 million, which might well justify the additional capital costs. Additional savings will accrue with higher energy costs, and many places in the United States already experience higher costs than US$0.10/kW-hr. Furthermore, energy costs are not expected to decrease. Routine increases are expected for the foreseeable future.

Another factor is the excess sludge production. The anaerobic system would produce less sludge for disposal and would have less biogas production. This should be a net credit, since biogas production from anaerobic digestion is rarely adequate to cover sludge disposal costs. It is assumed that the biogas production from the AF would not have value due to previously cited issues with nitrogen and methane solubility.

The model suggests that lower HRTs than 12 hours might be possible. Also a hybrid filter might further reduce requirements for longer HRTs. Another issue is nutrient removal. All of the anaerobic systems described herein are not useful for biological nutrient removal (BNR), and if BNR is required, other treatment options are probably favorable; however, in the case of the large coastal plants, such as the Hyperion Plant in Los Angeles, BNR is not required and is unlikely to be required in the near future.

A final potential usage for anaerobic wastewater treatment is for large, existing treatment plants. In the United States, these plants invariably use primary clarification followed by the activated sludge process. As the plant reaches its rated capacity, the plant is usually expanded to anticipate increased loads by expanding each process as needed. An additional alternative can be considered, if anaerobic wastewater treatment is used. An anaerobic process can be used to treat primary effluent and a 70 to 90% reduction in BOD or COD may be obtained, which can reduce the organic loading rate of the activated sludge process. If a hybrid of modified anaerobic process is used that can function both as primary treatment as well as secondary treatment (e.g., hybrid system with adequate provisions for sludge storage, treatment and disposal), it may be possible to increase plant capacity without expanding either the existing primary or secondary treatment processes.

2.14 CONCLUSIONS

The literature review showed that anaerobic treatment using AFs, UASBs and modified reactors is an efficient method for treating various types of wastewater, and there are some examples of low strength wastewater treatment, such as domestic wastewater. World wide, there is an increase in the number of pilot scale investigations and full-scale applications. For example many UASB reactors were built in the last 20 years to treat domestic sewage in tropical and sub tropical countries (Monroy *et al.*, 2000). There are fewer large scale AFs and modified reactors treating low strength wastewater, and no full-scale facilities in the North America were identified.

The developed dynamic model was able to predict treatment efficiency from previous pilot scale AF studies. Furthermore the model simulates the gas composition of the effluent from influent characteristics. The previous data and the model suggest that 12 to 24 hr HRT is required to achieve greater than 60% COD removal. Methane composition will be less than 50% below influent substrate concentrations of 130 mg/L COD at ambient temperature of 20^{o}C.

The literature review, model predictions and economics suggest that anaerobic wastewater treatment, for wastewaters in the range of domestic wastewaters, may soon be practicable in developed areas. Additional research is needed and should address hybrid approaches that exploit the well-known advantages of granular sludge in UASBs with additional surface area available using anaerobic filter. Such hybrid designs are already being investigated. There may also be an application for anaerobic wastewater treatment in areas that use “ocean disposal,” meaning primary treatment followed by ocean disposal in a long, deep outfall. Finally, the use of anaerobic treatment to reduce the loading on existing large treatment plants should also be pursued.

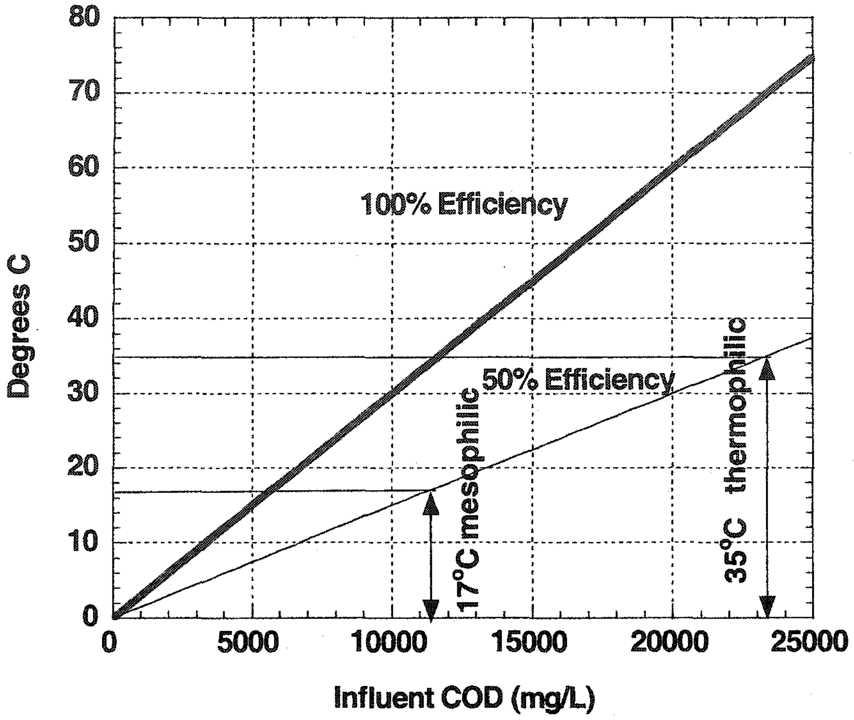

Fig. 2.1. Heat value of influent wastewater as a function of influent COD

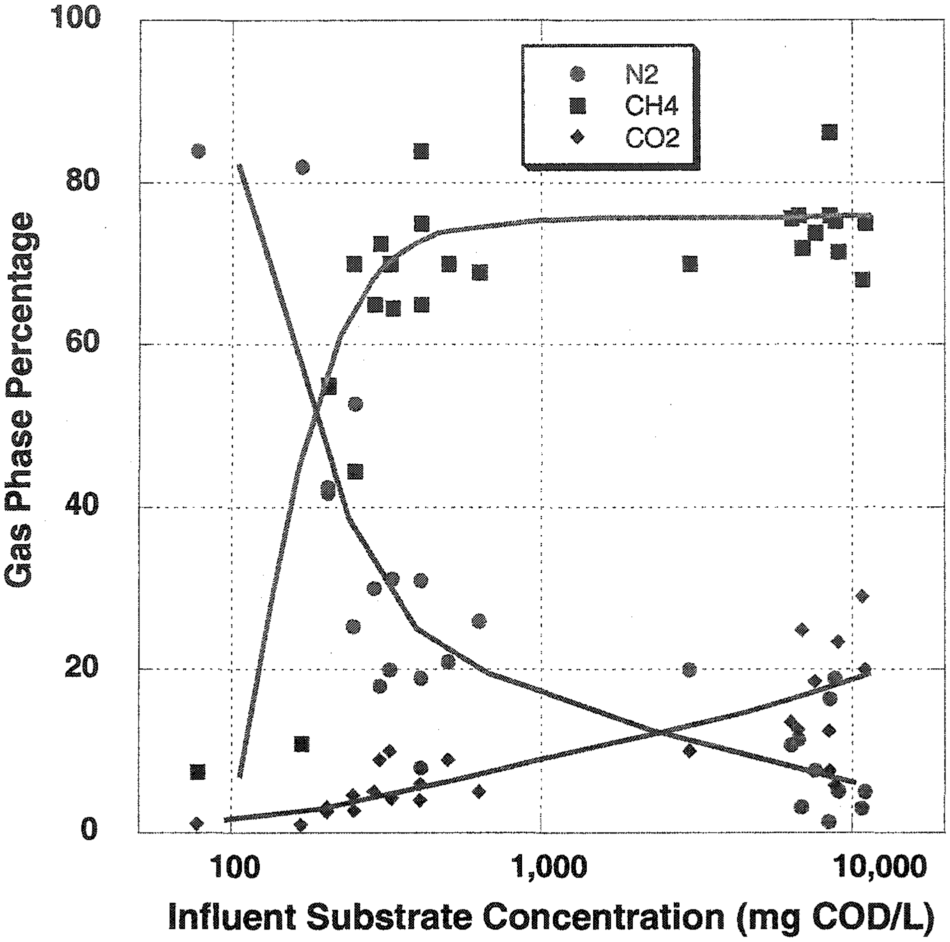

Fig. 2.2 Gas composition versus substrate concentration (literature references)

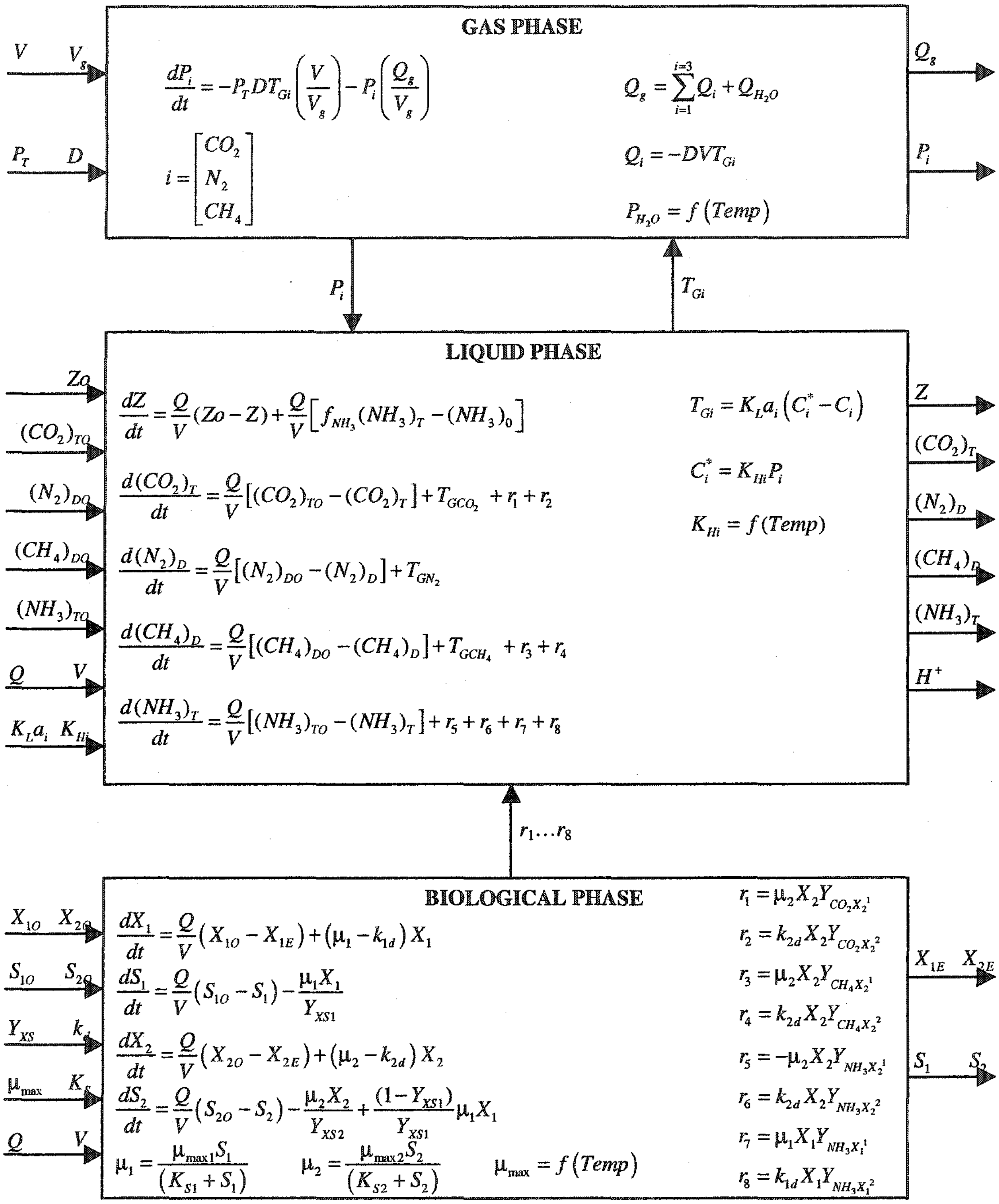

Fig. 2.3. Model Flow Diagram

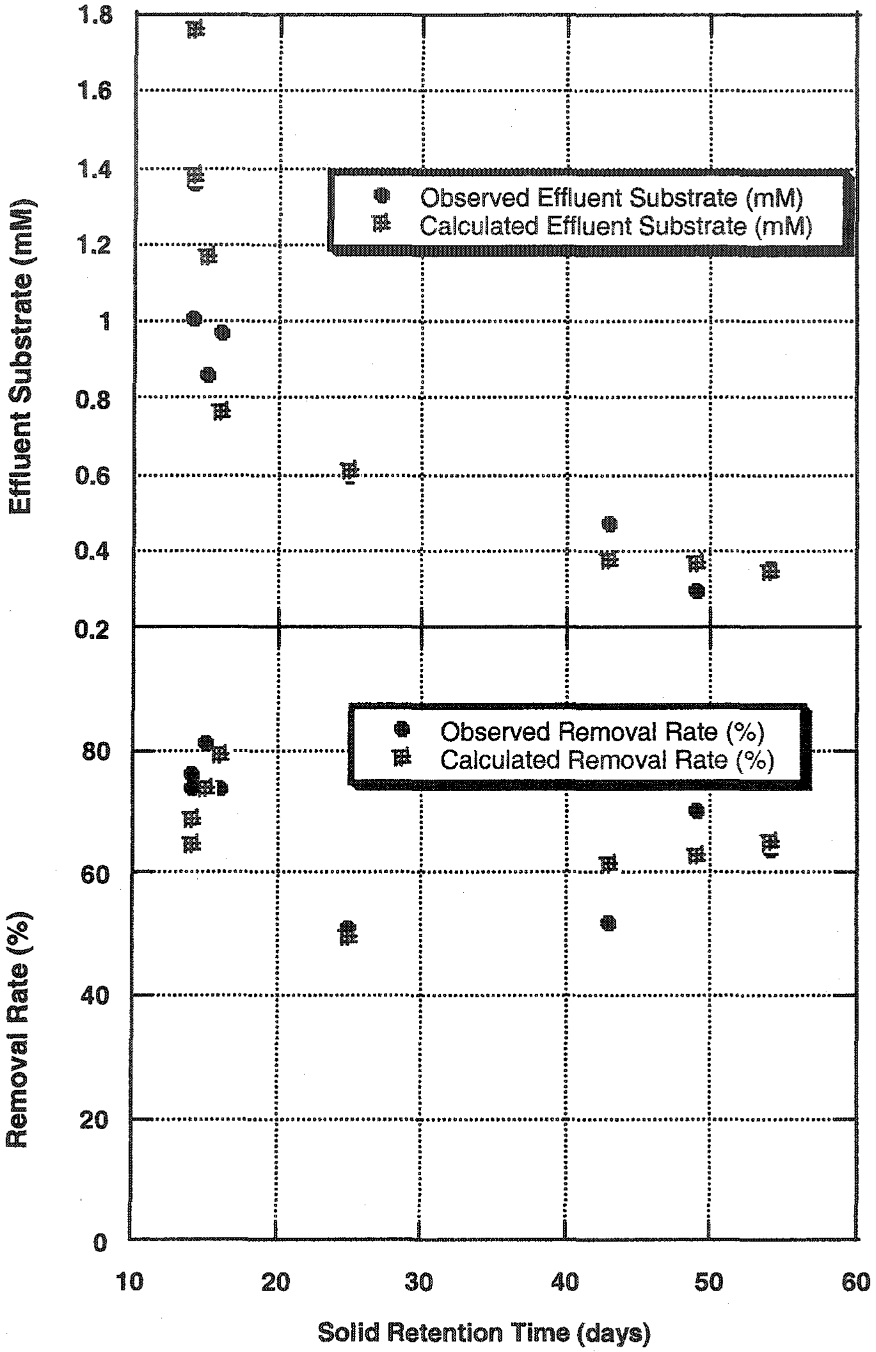

Fig. 2.4. Effluent substrate and removal rate as a function of SRT

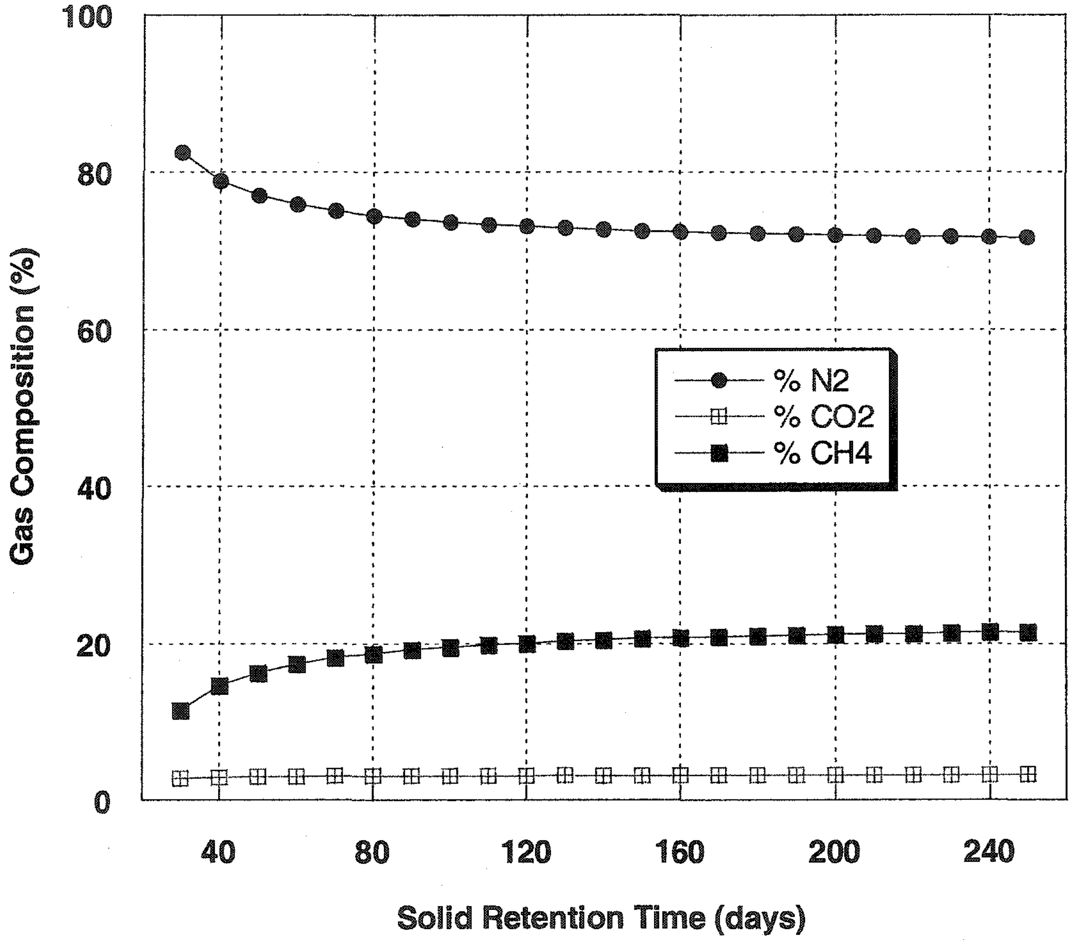

Fig. 2.5. Simulated gas composition as a function of SRT
(symbols represent calculated values)

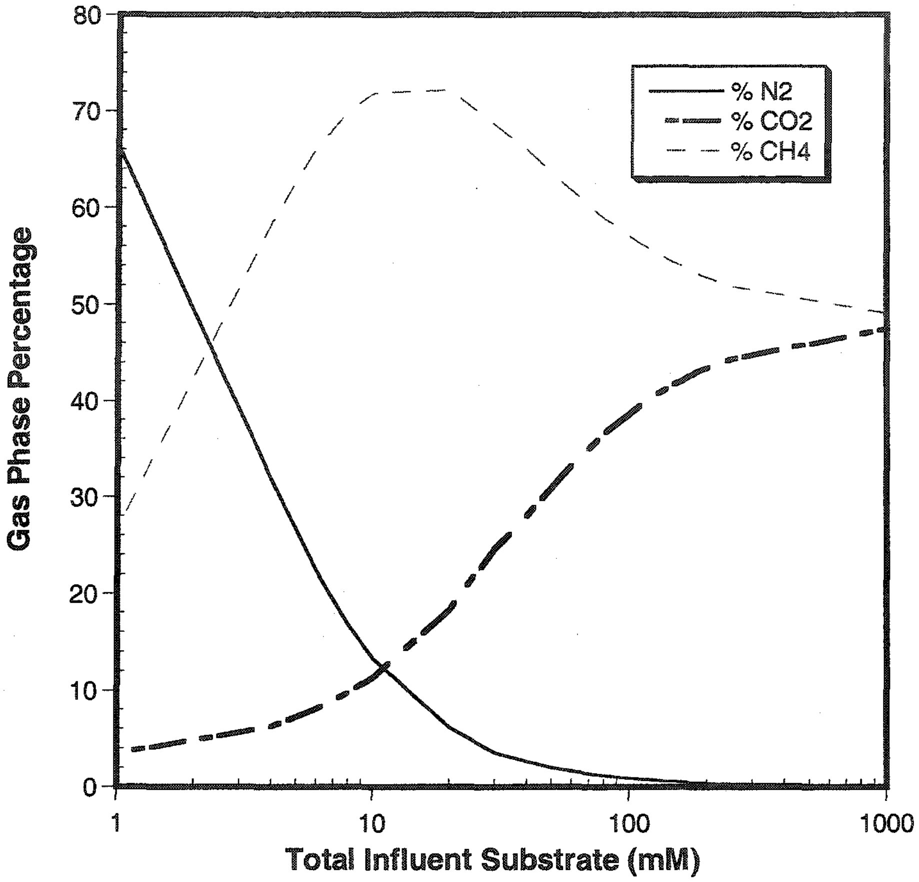

Fig. 2.6. Simulated gas composition as a function of influent substrate concentration

Table 2.1. Laboratory Scale Studies of Anaerobic Filter on Wastewater Treatment

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Packing material	Temp. (C)
Plummer *et al.* (1968) USA	Synthetic waste (1500-3000 mg/L)	0.424-3.392	4.5-72	36.7-92.1	Raschig rings and berl saddles mixture n=0.65-0.70	35
Pretorius (1971) South Africa	Raw sewage (500mg/L)	0.48	24-45	90	Stone n =0.6	20
El-Shafie & Bloodgood (1973) USA	Metrecal (10 g/L)	40.96	3-18	70.5	Hand-graded gravel	30
Frostell (1981) Sweden	Synthetic (8700 mg/L)	0.757-0.992	7.2-29	79-93	Polyurethane plastic material 200 m^2/m^3 * n=0.96	30
Landine *et al.* (1982) Canada	Potato processing wastewater	0.47-1.28	4-10 days	45-68	Rock media	22
Hanaki *et al.* (1990) Japan	Cafeteria (1300-2500 mg/L) 30% lipids		>1.3 days (SPS)[a] 3.3-10.1 days (TPS)[b]	80	Ring type plastic media 206 m^2/m^3 n=0.89	20
Viraraghavan *et al.* (1990) Canada	Dairy wastewater (4000 mg/L)	0.63-4.03	1-6 days	45-78 (12.5 C) 55-85 (21 C) 76-92 (30 C)	Plastic ballast rings 114 m^2/m^3 n=0.965	12.5-30
Hamdi & Garcia (1991) France	Olive mill wastewater (30 g/L)	2	15 days	60	PVC rings n=0.83	35
Hamdi & Ellouz (1993) France	Olive mill wastewater (9.22 g/L)	1.31	7 days	67	PVC rings n=0.83	35
Van der Merwe & Britz (1993) South Africa	Baker's yeast wastewater (5-30 g/L)	1.8-10	3 days	43-74	Synthetic rings 230 m^2/m^3 n=0.95	35

Reference and Region	Waste[1]	Organic loading rate[2] ($kg/m^3.d$)	Retention time (h)	Efficiency (%)	Packing material	Temp. (C)
Borja & Gonzalez (1994) Spain	Olive mill wastewater (30 g/L)	2	15 days	70	Sepiolite rings n=0.69	35
Hanaki *et al.* (1994) Japan	Synthetic wastewater (2000-2500 mg/L)	0.27-0.82	3-9 days	81-90	Plastic tubes n=0.83	20
Veiga *et al.* (1994) Spain	Tuna processing wastewater (20-54 g/L) 80% protein 20% fatty acids + fats	3-13	24-96	75	PVC Raschig rings 300 m^2/m^3	37
Smith (1995) USA	Hazardous landfill leachate (3628 mg/L)	2.8	31.2	66-82	Plastic pack 331 m^2/m^3 n=0.88	36
Viraraghavan & Varadarajan (1996) Canada	Septic-tank effluent (207-286 mg/L)	0.09-0.17	1.20-3.17 days	5-52 (5 C) 25-62 (10 C) 49-65 (20 C)	Plastic ballast rings 114 m^2/m^3 n=0.965	5, 10, 20
Viraraghavan & Varadarajan (1996) Canada	Whey wastewater (3400-5200 mg/L)	2-10.1	0.52-1.7 days	69-93	Ceramic saddles n=0.57	16- 30
Guerrero *et al.* (1997) Spain	Fish meal processing wastewater (10.4-34 g/L)	1.62-5.26	4.41-12.22 days	80-90	PVC rings 450 m^2/m^3 n=0.94	37
Punal *et al.* (1999) Spain	Cheese whey wastewater (9000 mg/L)	0-35	8.4	60-95 (SFR)[c] 85-95 (MFR)[d]	PVC Raschig rings 228 m^2/m^3 n=0.94	
Reyes *et al.* (1999) Spain	Piggery wastewater (941 mg/L) (five upflow and downflow mode)		1,2,4 days 8,12	70 (BOD) 60	Waste tyre rubber 5 m^2/m^3 n=0.66	30-35

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Packing material	Temp. (C)
Yilmazer & Yenigun (1999) Turkey *CSTR+ AF*	Cheese whey powder (11 g/L)	(1) 3.67 (2) 2.75 (3) 1.83	24h+3 day 24h+4 day 24h+6 day	(1) 63 (2) 95 (3) 67	Plastic pall rings 322 m^2/m^3 n=0.90	35
Bodik *et al.* (2000) Slovak Republic	Municipal wastewater (490-780 mg/L)		10, 20, 46	46-90	Plastic filling	9, 15, 23
Di Berardino *et al.* (2000) Italy	Food industry wastewater (0.53-2.62 g/L)	0.41-1.23	31-133	81.7-92.5	PVC tubes	35
Elmitwalli *et al.* (2000) Netherlands	(1) Raw sewage (772 mg/L) (2) Synthetic sewage (595 mg/L) (3) Skimmed milk		0.5-8	(2) 53-68	Reticulated polyurethane foam sheets 500 m^2/m^3	18-22
Ince *et al.* (2000) Turkey	Dairy wastewater (2000-6000 mg/L)	5-21	12	80	Raschig rings of glass media	35
Punal *et al.* (2000) Italy	Synthetic wastewater (1) 7200 mg/L (nitogen limited) (2) 6900 mg/L (nitrogen balanced)	1.5-4.5	1.5-4.6 days	(1) 76-86 (2) 80-90	PVC Raschig rings 228 m^2/m^3 n=0.85	35

[1] mg/L COD if not otherwise indicated

[2] COD unless otherwise indicated

[*] Specific surface area

[a] Single-phase system, [b] Two-phase system, [c] SFR: Single fed reactor, [d] MFR: Multiple-fed reactor

Scale: 0-10 liter Laboratory

Table 2.2. Pilot Scale Studies of Anaerobic Filter on Wastewater Treatment

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Packing material	Temp. (C)
Young & McCarty (1969) USA	Synthetic waste (1500-6000 mg/L)	0.96-3.392	4.5-72	36.7-98	Smooth quartzite stone	25
Lovan & Foree (1971) USA	Brewery press liquor (6000-24000 mg/L)	0.8	15-330	90	Crushed limestone	34
Jennett & Dennis (1975) USA	Pharmaceutical wastes 95% methanol (1250-16000 mg/L)	0.221-3.52	12-48	94-98	Hand-graded quartzitic gravel n =0.47	37
Chian & DeWalle (1977) USA	Leachate (19.5-62 g/L)		7.5-74 days	94-98		
Hudson *et al.* (1978) USA	Shellfish processing wastewater (121-466 mg/L)	a. 0.18-0.34 b. 0.15-0.36	7.92-74.4	a. 33-55 b. 45-81	a. Granitic stone packing 130 m^2/m^{3*} n=0.53 b. Oyster shells n=0.82	9.8-26
DeWalle *et al.* (1979) USA	Landfill leachate (0.027-430 mg/L ions)		4.2-34	75 metal ion	Plastic medium 206 m^2/m^3 n=0.94	Room temp
Braun & Huss (1982) Austria	Molasses distillery slops (45-50 g/L)	30-50 VS	26.4-38.4	34-50	Plastic-ball packing material	42
Kobayashi *et al.* (1983) USA	Domestic wastewater (288 mg/L)	0.32	24	73	PVC pack 44 ft^2/ft^3 n=0.97	20-35
Lindgren (1983) Sweden	Synthetic (150-600 mg/L)				Polyurethane plastic material n=0.95	20-35
Noyola *et al.* (1988) France	Domestic sewage (407 mg/L)	0.5-12	4-72	45-80	PVC packing 170 m^2/m^3 n=0.85	16, 29

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Packing material	Temp. (C)
Abe *et al.* (1991) Japan *Aerobic soil column + AF (denitrifying reactor)*	Livestock wastewater (200 TOC mg/L)		1.8-2.6 days		a. Carbonized rice husks b. Carbonized rice husks with 20% straw c. Volcanic ash soil d. Charcoal chips	25
Akunna *et al.* (1994) France	Synthetic wastewater (5318 mg/L)	0.53-5.55	23h-10 days	60-77 99 (overall)	PVC rings	37
Viraraghavan & Varadarajan (1996) Canada	Potato-processing wastewater (220-840 mg/L)	0.14-0.35	1.5 days	17-56	Stone n=0.42	2-20
Wilson *et al.* (1998) Singapore	a. Domestic (0.26-0.54 g/L) b. Soy-bean processing (7.52-11.45g/L)	a. 0.96-2.04 b. 4.41-22.25	a. 0.42-0.21 day b. 1.04-0.42 days	a. 75-52 b. 92-75	a. Cylindirical plastic rings b. Soft fibrous media 1560 m^2/m^3	a. 17-28 b. 35
Show & Tay (1999) Singapore	Synthetic waste (2500-10000 mg/L)	2-16	15-30	a. 78-97 b. 77-95 c. 57-95	a. Glass Raschig ring 187 m^2/m^3 n=0.75 b. PVC Raschig ring 132 m^2/m^3 n=0.90 c. PVC Raschig ring 187 m^2/m^3 n=0.75	35
Jawed & Tare (2000) South Africa	Synthetic feed (2.30-8.74 g/L)	2-12	0.8-1.1 days	40-80	PVC module 102 m^2/m^3 n>0.97	34-36
Alves *et al.* (2001) Portugal	Synthetic dairy wastewater (3-12 g/L)	3.33-8.6	0.9-1.4 days	>90	PVC Raschig ring 230 m^2/m^3 n=0.925	35

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Packing material	Temp. (C)
Picanco *et al.* (2001) Brazil	Synthetic wastewater (1267 mg/L)	1.27	24	68	a. Polyurethane foam n=0.92 b. PVC n=0.015 c. Special ceramic n=0.64 d. Refractory brick n=0.35	30
---------- Large Pilot Scale Studies ----------						
Donovan *et al.* (1979) USA	Heat treatment liquor (10-11 g/L)	1.56-9.39	16.56-152.64	17-68	Plastic media n=0.95	35
Chung (1982) USA	Domestic wastewater (25.6 TOC mg/L)	0.16	24	60	PVC pack 44 ft^2/ft^3 n=0.97	22.4
Abramson (1987) USA	Domestic wastewater (30-500 TOC mg/L)		6-60	40-90 TOC	PVC packing material	27.2
Sarner (1990) Sweden	Sodium based sulphite pulp mill wastewater (10-26 g/L)	20-40		85 inorganic sulphur removal	Plastic medium 140 m^2/m^3	
Kim *et al.* (1997) Japan	Sewage a. (222 BOD mg/L) b. (200.9 BOD mg/L)	a. 0.73 BOD b. 0.85 BOD	a. 7.3 b. 5.7	a. 96.1BOD b. 97 BOD	Polypropylene foam tube	
Camargo & Nour (2001) Brazil	Sewage (996 mg/L)	2.66-11.95	2-9	60-80	Whole and cut bamboo rings	

[1] mg/L COD if not otherwise indicated
[2] COD unless otherwise indicated
[*] Specific surface area
Scale: 10-100 liter Pilot, 100-1000 Large Pilot

Table 2.3. Demonstration and Full Scale Studies of Anaerobic Filter on Wastewater Treatment

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Packing material	Temp. (C)
Genung *et al.* (1979) USA	Sewage (60-220 BOD mg/L)	0.048-0.608 BOD	2.5-10.5	55 BOD	Raschig unglazed ceramic rings	15-20
Koon *et al.* (1979) USA	Domestic sewage (92-209 BOD mg/L)	0.24-0.608 BOD	12-48	43-59.8 BOD	Raschig unglazed ceramic rings	13-25
Harper *et al.* (1990) USA	Poultry processing wastewater (2478 mg/L)	2.8	21	70 92 FOG (fat, oil and grease)	Polyethylene random pack	35
Hogetsu *et al.* (1992) Japan	Wool scouring wastewater (68.4 g/L soluble TOD)	3-45 TOD	Several days	60	Polypropylene media 65 m^2/m^{3*} n=0.95	37-53
Watanabe *et al.* (1993) Japan	Sewage (13 g BOD/c.d blackwater)** (27 g BOD/c.d graywater)			90 BOD		
Iyo *et al.* (1996) Japan	Domestic sewage a. 141.6 BOD mg/L b. 180.4 BOD mg/L c. 166.7 BOD mg/L	a. 0.06 b. 0.08 c. 0.075 (BOD)	a. 57 b. 54 c. 53 (overall)	a. 94.4 BOD b. 91.8 BOD c. 95.1 BOD (overall)	Polypropylene 82 m^2/m^3 n=0.39	a. 22-27 b. 16-22 c. 16-20
Viraraghavan & Varadarajan (1996) Canada	Slaughterhouse wastewater (1194-5900 mg/L)	0.47-2.98	0.8-4.9 days	37-77	Plastic ballast rings 105 m^2/m^3 n=0.90	23.6-27.1
Leal *et al.* (1998) Venezuela	Brewery wastewater (1400-3900 mg/L)	8	10	96	PVC Raschig rings	34-39
Kondo & Kondo (2000) USA	Domestic wastewater (130-550 BOD mg/L)	a. 0.68 b. 0.136 (BOD)	a. 9.6 hr b. 2 days (overall)	a. 97 BOD b. 98 BOD (overall)	Plastic media	14-21

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Packing material	Temp. (C)
			Full Scale Studies			
Witt *et al.* (1979) USA	Guar (9140 mg/L)	7.52	24	60		36.6
Campos *et al.* (1986) Brazil	Meat processing wastewater (1878 mg/L)	1.4	13	76	Broken stones n=0.40	24-25
Defour *et al.* (1994) Ireland	Citric acid wastewater (16.6 g/L)	11.3	1.46 days	65		
Garrido *et al.* (2001) Spain	Dairy wastewater (6-15 g/L)	0.5-8	1.5	50-85	PVC packing	37

[1] mg/L COD if not otherwise indicated

[2] COD unless otherwise indicated

[*] Specific surface area, [**] g/c.d refers to gram per capita per day

Scale: 1000-10000 liter Demonstration, >10000 liter Full

Table 2.4. Laboratory and Pilot Scale Studies of UASB on Wastewater Treatment

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Temp (C) and Scale
Pretorius (1971) South Africa	Raw sewage (500 mg/L)	0.5	24	90	20, L
Frostell (1981) Sweden	Synthetic	2.5-10	20.6-53.3	68-87	30, L
Kato *et al.* (1997) Brazil	Synthetic (whey and ethanol) (113-722 mg/L) (127-675 mg/L)	0.2-6.8	2.6-29	30-99	30, L
Ruiz *et al.* (1997) Spain	Slaughterhouse wastewater (5200-11400 mg/L)	1.03-6.58	28.8-156	93-59	37, L
Kalyuzhnyi *et al.* (1998) Mexico	Potato-maize (raw) (5500-18100 mg/L)	0.63-13.89	15.6-144	63.4-81.3	35, L
Kalyuzhnyi *et al.* (1998) Mexico	Potato-maize (preclarified) (3600-9000 mg/L)	5.02-15	14.4-43.2	71.1-93.6	35, L
Elmitwalli *et al.* (1999) Netherlands	1. Raw sewage (456 mg/L) 2. Pre-settled sewage (344 mg/L)	1.37 1.03	8	65 59	13, L
Bodik *et al.* (2000) Slovak Republic	Municipal wastewater (310 mg/L)	0.62	12	37-48	9,15, L
Syutsubo *et al.* (2000) Japan	1. Alcohol distillery wastewater 2. Synthetic acetate wastewater 3. Sucrose wastewater (3000 mg/L)	9	8	94-99	55, L
Kalogo *et al.* (2001) Belgium	Raw domestic sewage (320 mg/L)	1.99	4.0	65	29, L
Kalyuzhnyi *et al.* (2001) Russia	Winery wastewater (2000-4200 mg/L)	1.7-4.7	0.86-1.15 days	57-68	4.8-10.3, L

Reference and Region	Waste[1]	Organic loading rate[2] $(kg/m^3.d)$	Retention time (h)	Efficiency (%)	Temp (C) and Scale
Kalyuzhnyi *et al.* (2001) Russia *Two-stage UASB+UASB*	Winery wastewater (1500-4300 mg/L)	1.3-2.2	1.8-2.0 days (overall)	71-78 (overall)	3.9-10.2, L
Lacalle *et al.* (2001) Spain *UASB+ Upflow Aerated Filter*	Food industry wastewater (10.4 g/L)	1.27-2.76	4.51-13.0 days (overall)	96-99 (overall)	33, L
Nadais *et al.* (2001) Portugal	Dairy wastewater 1. 5.9, 11.9 g/L 2. 5.9, 5.8 g/L 3. 5.9, 5.6 g/L	 1. 11.8, 23..8 2. 11.8, 11.6 3. 11.8, 22.4	 1. 12 2. 12 3. 12, 6	 1. 93, 85 2. 93, 93 3. 93, 74	35, L
Nunez & Martinez (2001) Spain *UASB+Activated Sludge Process*	Slaughterhouse wastewater (1533-1744 mg/L)	2.62-6.73	6-16	85 (overall)	35, L
Lettinga *et al.* (1983) Netherlands	Raw domestic sewage (520-590 mg/L)	1.39-1.57	9	57-79	21, P
Gnanadipathy & Polprasert (1993) Thailand	Domestic wastewater (450-750 mg/L)	0.9-6.0	3-12	90	30, P
Sayed & Fergala (1995) Egypt *Two-stage UASB reactor system*	Domestic sewage (200-700 mg/L)	1.22-2.75[a] 1.70-6.20[b]	10 (8+2) 8 (6+2) 6 (4+2)	61-66[a] 32-46[b] 74-82 (overall)	18-20, P and L
Tang *et al.* (1995) Puerto Rico	Domestic wastewater (782 mg/L)	0.782-3.128	6-72	70.9	~20, P
Agrawal *et al.* (1997) Japan *UASB+ Hanging Sponge Cubes*	Raw sewage (300 mg/L)	1.03	7	(70 mg/L)	7-30, P

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m³.d)	Retention time (h)	Efficiency (%)	Temp (C) and Scale
Cheng *et al.* (1997) Taiwan	PTA manufacturing wastewater (4.66 g/L)	0.39-3.25	1.5-4.6 days	21-73	35, P
Gonzalez *et al.* (1998) Cuba	Sugar cane molasses (3640-3820 mg/L)	2.3-7.15	0.52-1.65 days	59.9-91	24-32, P
Goncalves *et al.* (1999) Brazil *UASB+ Aerated Biofilter*	Domestic wastewater (297-463 mg/L)	1.39-1.84	4-8 4.11-8.23 (overall)	68-73 82-92 (overall)	P
Lettinga *et al.* (1983) Netherlands	Raw domestic sewage (420-920 mg/L)		32-40	48-70	12-18, LP
Lettinga *et al.* (1983) Netherlands	Raw domestic sewage (248-581 mg/L)		12	72	18-20, LP
De Man *et al.* (1986) Netherlands	Municipal wastewater (100-900 mg/L)		4-14	45-72	7-18, LP
Vieira & Souza (1986) Brazil	1.Settled sewage (341 mg/L) 2.Raw sewage (424, 406 mg/L)	1.2.05 2.2.54, 2.44	4	1. 65 2. 60, 65	1. 35, LP 2. 20, 23, LP
De Man *et al.* (1988) Netherlands	Low strength wastewater (190-1180 mg/L)		7-8	30-75	12-20, LP
Monroy *et al.* (1988) Mexico	Sewage (465 mg/L)		12-18	65	12-18, LP
Barbosa & Sant'Anna (1989) Brazil	Raw domestic sewage (627 mg/L)	3.76	4	74	19-28, LP
Singh *et al.* (1996) Thailand	Synthetic wastewater (500 mg/L)	4 3 2 1.2	3 4 6 6	90-92	20-35, LP
Chernicharo & Machado (1998 Brazil *UASB/AF system*[c]	Domestic sewage (640 mg/L)		4-6 1.5-24 (AF)	80 85-90 (overall)	LP
Castillo *et al.* (1999) Spain *UASB+ two RBC reactors*	Domestic sewage 1. 363-625 mg/L 2. 613-666 mg/L	1. 1.45-10 2. 2.13-9.81	1.5-7.5 3-10 (overall)	1. 27-70 2. 22-55 82-99 (overall)	1. 18-20, LP 2. 12-13, LP

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Temp (C) and Scale
Chernicharo & Nascimento (2001) Brazil *UASB+Trickling Filter*	Domestic sewage (420-666 mg/L)	0.44-2.52	4	65-77 74-88 (overall)	LP
Torres & Foresti (2001) Brazil *UASB + SBR*	Domestic sewage (103-250 mg/L)	0.412-1	6	65 92 (overall)	14-25, LP
Von Sperling *et al.* (2001) Brazil *UASB+ Activated Sludge Process*	Municipal wastewater (386-734 mg/L)	2.32-4.4	4 7.9-11.2 (overall)	68-84 85-93(overall)	LP

[1] mg/L COD if not otherwise indicated

[2] COD unless otherwise indicated

[a]-This corresponds to the first stage which consists of two flocculent sludge UASB reactors working alternately (one at a time)

[b]-This corresponds to the second stage which consists of one granular sludge UASB reactor

[c]- The system consists of a UASB reactor followed by downflow and upflow anaerobic filters in parallel with blast furnace slag media

Scale: 0-10 liter Laboratory (L), 10-100 liter Pilot (P), 100-1000 liter Large Pilot (LP)

Table 2.5. Demonstration and Full Scale Studies of UASB on Wastewater Treatment

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Temp (C)
Demonstration Scale Studies					
Craverio *et al.* (1986) Brazil *Two-stage (CSTR+ UASB)*	Brewery/soft drink wastewater (1.3-8 g/L)	2-13	6-8	80.9 84.4 (overall)	35
De Man *et al.* (1986) Netherlands	Municipal wastewater (100-900 mg/L)		9-16	46-60	10-18
Karnchanawong *et al.* (1999) Thailand	Domestic wastewater (64.6-94.7 BOD mg/l)	0.13-0.51	4.5-12	52.6-69.4 BOD	
Martinez *et al.* (2001) Uruguay	Malting wastewater	0.25-6		85	15, 28, 30
Full Scale Studies					
Campos *et al.* (1986) Brazil	Vegetable/fruit processing wastewater (394-872 mg/L)	0.78-1.36	7.5-24	66-76	29-30
De Man *et al.* (1986) Netherlands	Municipal wastewater (100-900 mg/L) (150-5500 mg/L)		6.2-18	31-49	11-19
Hulshoff Pol & Lettinga (1986) Netherlands	a. Brewery wastewater (1-1.5 g/L) b. Alcohol distillery wastewater (4-5 g/L) c. Maize starch wastewate (10 g/L) d. Paper industry wastewater (3 g/L) e. Paper mill wastewater (~1 g/L)	a. 4.5-7.0 b. 11.5-14.5 c. 15 d. 10.5 e. 4.4-5	a. 5.6 b. 8.2 c. 18.3 d. 8-10 e. 5.5	a. 75-80 b. 92 c. 90-95 d. 75 e. 70-72	a. 20-24 b. 32-35 c. 40 d. 30-40 e. 26-30
Louwe Kooijmans & van Velsen (1986) Lettinga *et al.* (1987) Colombia	Domestic sewage (267 mg/L)	2	6-8	75-82	25

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Temp (C)
Collivignarelli *et al.* (1991) Maaskant *et al.* (1991) Italy	Municipal wastewater (205-326 mg/L)		12-42	31-56	7-27
Draaijer *et al.* (1992) India	Municipal wastewater (563 mg/L)	2.25	6	74	20-30
Kiriyama *et al.* (1992) Japan	Municipal sewage a. (297 mg/L) b. (286 mg/L) c. (394 mg/L)	a. 0.65 b. 0.73 c. 0.97	1.8	a. 58 b. 69 c. 73	a. 12 b. 24 c. 28
Van der Last & Lettinga (1992) Netherlands	Pre-settled domestic sewag (391 mg/L)	1.34-4.69	2-7	16-34	>13
Schellinkhout & Collazos (1992) Colombia *UASB+ facultative pond/lagoon*	Raw sewage	 b. 2.0	a. 5-19 b. 5.2	a. 66-72 b. 18-44	
Vieira & Garcia (1992) Brazil	Domestic wastewater (188-459 mg/L)	0.62-1.88	5-15	60	18-28
Defour *et al.* (1994) Belgium	Potato wastewater (2600 mg/L)	8	7	90	
Defour *et al.* (1994) Belgium	Potato wastewater (12,500 mg/L)	12	18	78	
Defour *et al.* (1994) France	Brewery wastewater (4200 mg/L)	5	17	89	
Defour *et al.* (1994) Netherlands	Starch wastewater (5500 mg/L)	18	7.5	82	
Schellinkhout & Osorio (1994) Colombia	Sewage (380 mg/L)	1.82	5	45-60	24
Vieira *et al.* (1994) Brazil	Sewage (402 mg/L)	1.38	7	74	16-23
Tare *et al.* (1997) India	Domestic wastewater (1183 mg/L)	3.55	8	51-63	18-32
Tare *et al.* (1997) India	Domestic wastewater (404 mg/L)	1.21	8	62-72	18-32

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Temp (C)
Chernicharo & Borges (1997) Brazil	Domestic sewage (600 mg/L)	1.11	13	68	
Vinod *et al.* (1997) India	Domestic sewage (133-254 mg/L)	1.1	8	49-65	
Vinod *et al.* (1997) India	Domestic sewage (551-730 mg/L)	5.63	8	24-50	
Yu *et al.* (1997) India	Municipal wastewater	0.7	12	49-78	15-25
Chernicharo & Cardoso (1999) Brazil *Partitioned Reactor*	Domestic sewage (712 mg/L)	2.28	7.5	79	
Karnchanawong *et al.* (1999) Thailand	Domestic wastewater (409.5-517.7 mg/L)	0.41-2.16	4.5-24	59.9-76.4	30.9
Del Nery *et al.* (2001) Brazil *DAF+UASB reactors R1, R2*	Poultry slaughterhouse wastewater (2631 mg/L)	0.51-2.11	1.47-5.29 days	47.8-84.4 (R1) 54.5-83.4 (R2)	
Florencio *et al.* (2001) Brazil *UASB+polishing pond*	Domestic sewage (290-563 mg/L)	0.79-1.40	8.8-9.7	71-83 79-84 (overall)	30.2-31
Rodriguez *et al.* (2001) Colombia	Domestic sewage (463-538 mg/L)	0.037-1.81	6.7-24.9	73-84	24-27

[1] mg/L COD if not otherwise indicated

[2] COD unless otherwise indicated

Scale: 1000-10000 liter Demonstration, >10000 liter Full

Table 2.6. Studies Using Modified Anaerobic Filter Process on Wastewater Treatment

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Packing material	Temp(C) and Scale
Kennedy & Van den Berg (1982) *Downflow Fixed Film Reactors* Canada	Bean blanching	0.5-7.5	1-25 days	79-83	Clay packing 120 m^2/m^3* n=0.52-0.55	35, P
Kennedy & Droste (1983) *Downflow Fixed Film Reactors* Canada	Sucrose substrate (5-20 g/L)	4-4.5	0.5-15 days	56-85	NPP (needle punched polyester) packing n=0.92	L
Guiot & Van den Berg (1985) *Upflow Blanket Filter* Canada	Sugar wastewater (2500 mg/l)	5-51	2-18	96 (5-25 kg/m^3.d) 63 (36 kg/m^3.d) 64 (51 kg/m^3.d)	Plastic rings 2/3 sludge blanket 235 m^2/m^3	27, L
Kennedy & Guiot (1986) *Upflow Blanket Filter* Canada	Synthetic sucrose wastewater a. 2.5-10.6 g/L b. 300 mg/L c. 5000 mg/L	a. 10 b. 2.85-4.9 c. 5-16	a. 7.2-24 b. 1.6-3 c. 7.2-24	a. 96 b. 73-93 c. 77-97 (8%) 79-97 (16%) 72-97 (32%)	Plastic rings n=0.80 2/3 sludge blanket (a, b) 8, 16, 32% packing depth (c)	27, L
Kennedy & Guiot (1986) *Upflow Blanket Filter* Canada	Landfill leachate (15-25 g/L)	4.8-14.7	1.5-4.2 days	97-98	Plastic rings n=0.80 2/3 sludge blanket	35, P
Chang (1989) *Hybrid Upflow Anaerobic Filter* Taiwan	Leachate from solid waste landfill (11-58.4 g/L)	1.43-21.97	1.25-7.67 days	92 (OLR< 13kg/m3.d) 70 (OLR=21.97kg/m3.d)	Ceramic raschig rings 312 m^2/m^3 n=0.59	35, P
Chung & Choi (1993) *Hybrid Upflow Anaerobic Filter* Korea	Naked barley distillery wastewater (3-6 g/L)	1-3	72-144	89-94 (AUBF-1/7)** 91-94 (AUBF-1/2) 93-95 (AF)	Polyethylene rings 280 m^2/m^3 n=0.88	35, L

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Packing material	Temp(C) and Scale
van der Merwe & Britz (1993) *Hybrid Upflow Anaerobic Filter* South Africa	Baker's yeast wastewater (5-30 g/L)	1.8-10	3 days	42-84	Polyethylene foam 0.77 kg/m^3	35, L
Austermann-Haun & Seyfried (1994) *Hybrid Upflow Anaerobic Filter* Germany	Industrial wastewater (11.4 g/L)	1.7	6.8 days	81	BIONET 100 m^2/m^3 34% packed	36.1, F
Miyahara & Noike (1994) *Hybrid Upflow Anaerobic Filter* Japan	Synthetic wastewater (550 mg/L)	0.55	24	75	Vinylidene chloride looped fibre (Ring Lace)	20, L
Tilche *et al.* (1994) *Hybrid Upflow Anaerobic Filter* Italy	Piggery wastewater	8.5-9.7	72	55	BIO-ECO polypropylene random pack	31-36, F
Borja *et al.* (1995) *Hybrid Upflow Anaerobic Filter* UK	Slaughterhouse wastewater (2450 mg/L)	5-45	2-12	69 (45 g/ l.d) 75 (32 g/ l.d) 98 (5-22 g/ l.d)	1/3 clay-ring support medium (bentonite) 250 m^2/g n=0.63 2/3 sludge blanket	35, L
Cordoba *et al.* (1995) *Hybrid Upflow Anaerobic Filter* Argentina	Dairy wastewater (1.82-8.39 g/L)	1.8-8.4	24	89.9-95.8	Polyurethane foam n=0.91 8/75 sludge blanket	30, L
Fang &Kwong (1995) *Hybrid Upflow Anaerobic Filter* Hong Kong	Corn starch Wastewater (3-25 g/L)	3-50	9.6-24	40-90	Plastic rings 235 m^2/m^3 21/31 sludge blanket	37, L
Di Berardino *et al.* (1997) *Hybrid Upflow Anaerobic Filter* Portugal	Food processing wastewater (300-2200 mg/L)	0.17-0.42	2.5 days	a. 60 b. 83	Plastic rings	a. 25, P b. 30, P

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Packing material	Temp(C) and Scale
Timur *et al.* (1997) *Hybrid Upflow Anaerobic Filter* Turkey	Landfill leachate (14.9-19.98 g/L)	0.77-16.53	0.9-5.1 days	81.4 TOC	Plastic pall rings 322 m^2/m^3 n=0.90	35, L
Bello-Mendoza & Castillo-Rivera (1998) *Anaerobic Hybrid Reactor* Mexico	Coffee processing wastewater (2030 mg/L)	0.21-2.59	10-59	22.4-88.6	Volcanic rocks 2/3 sludge blanket	18-23, D
Borja *et al.* (1998) *Hybrid Upflow Anaerobic Filter* Spain	Slaughterhouse wastewater (3.74-10.41g/L)	2.49-20.82	0.5-1.5 days	90.2-93.4	Polyurethane foam n=0.5 2/3 sludge blanket	35, L
Elmitwalli *et al.* (1999) *Anaerobic Hybrid Reactor* Netherlands	a. Raw sewage (456 mg/L) b. Pre-settled sewage (344 mg/L)		8	a. 66 b. 61	Reticulated polyurethane foam sheets 500 m^2/m^3	13, L
Hutnan *et al.*(1999) *Anaerobic Hybrid Reactor* Slovakia	Synthetic wastewater (6000 mg/L)	0.5-15	0.4-12 days	80-90	Tubular plastic carrier 544 m^2/m^3 n=0.93	37, L
Wu *et al.* (2000) *Anaerobic Hybrid Reactor* Singapore	Synthetic wastewater (5000 mg/L)	1-24	5-60	71-98	Raschig rings 20%, 40%, 60% and 75% packing height	35, L
Elmitwalli *et al.* (2001) *AF + Anaerobic Hybrid Reactor* Egypt	Raw domestic sewage		a. 4+8 b. 2+4 c. 3+6	a. 70.9 b. 58.6 c. 63 (overall)	Vertical sheets of RPF 2400 m^2/m^3 n=0.97	13, P

[1] mg/L COD if not otherwise indicated

[2] COD unless otherwise indicated

* Specific surface area, **AUBF-1/7 refers to 1/7 packed anaerobic upflow sludge bed filter

Scale: 0-10 liter Laboratory (L), 10-100 liter Pilot (P), 100-1000 liter Large Pilot (LP), 1000-10000 liter Demonstration (D), >10000 liter Full (F)

Table 2.7. Studies Using Modified UASB Process on Wastewater Treatment

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Temp(C) and Scale
De Man *et al.* (1988) *EGSB reactor* Netherlands	Low strength wastewater (150-600 mg/L)		2-3	20-60 CODs	12-20, LP
Van der Last & Lettinga (1992) *EGSB reactor* Netherlands	Domestic sewage (391 mg/L)	2.7-9.4	1.5-5.8	~30	16-19, F
Bogte *et al.* (1993) *UASB-septic-tank* Netherlands	Domestic wastewater (976 mg/L)	0.53	44.3	33	13.8, D
Bogte *et al.* (1993) *UASB-septic-tank* Netherlands	Domestic wastewater (821 mg/L)	0.34	57.2	3.8	12.9, D
Bogte *et al.* (1993) *UASB-septic-tank* Netherlands	Domestic wastewater (1716 mg/L)	0.20	202.5	60	11.7, D
Lettinga *et al.* (1993) *UASB-septic-tank* Indonesia	Domestic sewage Black water		360	90-93	LP
Lettinga *et al.* (1993) *UASB-septic-tank* Indonesia	Domestic sewage Grey + black water		34	67-77	LP
Wang (1994) *HUSB reactor* Netherlands	Sewage (650 mg/L)	5.2	3	37-38	15.8, LP
Wang (1994) *EGSB reactor* Netherlands	Sewage (397 mg/L)	4.76	2	27-48	15.8, LP
Kato *et al.* (1997) *EGSB reactor* Brazil	Synthetic wastewater with ethanol (127-675 mg/L)	3.9-32.4	0.2-2.1	56-97	30, P

Reference and Region	Waste[1]	Organic loading rate[2] (kg/m^3.d)	Retention time (h)	Efficiency (%)	Temp(C) and Scale
Kato *et al.* (1997) *EGSB reactor* Brazil	Brewery wastewater (666-886 mg/L)	9-14.4	1.3-2.4	70-91	15-30, LP
Van Lier *et al.* (1997) *EGSB reactor* Netherlands	Synthetic wastewater (550-1100 mg/L)	5.1-6.7	4	97	8, L
Yu *et al.* (1997) *ABR reactor*[a] Britain	Municipal wastewater (338-516 mg/L)		2-16	67.8-83.5 (overall)	18-28, L
Driessen & Yspeert (1999) *IC reactor* Netherlands	Dairy industry wastewate (820-2950 mg/L)	8.5-24	2.6-4	51	37, F
Driessen & Yspeert (1999) *IC reactor* Netherlands	Food industry wastewat (1000-7500 mg/L)	5-42	3.6-9.1	80	27, F
Driessen & Yspeert (1999) *IC reactor* Netherlands	Brewery wastewater (3000-23000 mg/L)	4-36	8-24	70-90	35, F

[1] mg/L COD if not otherwise indicated

[2] COD unless otherwise indicated

[a] ABR is the shortcut for Anaerobic Baffled Reactor. The system consists of three chambers. The first is a UASB reactor without a gas-solid-liquid separator, the second is a down flow fixed film reactor with plastic packing and the third one is a hybrid UASB-AF with plastic media at the top 3/5

Scale: 0-10 liter Laboratory (L), 10-100 liter Pilot (P), 100-1000 liter Large Pilot (LP), 1000-10000 liter Demonstration (D), >10000 liter Full (F)

2.15 NOMENCLATURE

a	stoichiometric coefficient from oxidation-reduction balance
b	stoichiometric coefficient from oxidation-reduction balance
CH_3COOH	molecular formula for acetic acid
$C_5H_7NO_2$	empirical molecular formula for biomass
C_i^*	saturation concentration of gases in liquid phase at equilibrium (mM)
C_i	concentration of gases in liquid phase (mM)
$\left[CO_2\right]_T$	total concentration of all forms of carbonic acid (mM)
$\left[CO_2\right]_D$	concentration of carbonic acid and dissolved carbon dioxide (mM)
$\left[CO_3^{-2}\right]$	carbonate ion concentration (mM)
D	conversion factor (L gas/mole gas)
$f_{CO_3^{-2}}$	fraction of carbonate ion in the carbonic acid system
f_{CO_2}	fraction of dissolved carbon dioxide in the carbonic acid system
$f_{HCO_3^-}$	fraction of bicarbonate ion in the carbonic acid system
f_{NH_3}	fraction of ammonia in the total ammonia system
$\left[H^+\right]$	hydrogen ion concentration (mM)
$\left[HCO_3^-\right]$	bicarbonate ion concentration (mM)
k_{1d}	acidogenic decay rate (d^{-1})
k_{2d}	methanogenic decay rate (d^{-1})
K_La_i	overall gas transfer film coefficient (d^{-1})
K_{Hi}	Henry's Law constant (mM/mmHg)
K_{S1}	saturation constant for complex substrate (mM)
K_{S2}	saturation constant for acetate (mM)
μ_1	specific growth rate for acidogens (d^{-1})
μ_2	specific growth rate for methanogens (d^{-1})
$\mu_{\max 1}$	maximum specific growth rate for acidogens (d^{-1})
$\mu_{\max 2}$	maximum specific growth rate for methanogens (d^{-1})
$\left[NH_3\right]$	ammonia concentration (mM)
$\left[OH^-\right]$	hydroxyl ion concentration (mM)
P_i	partial pressure of gases in the gas phase (mmHg)
P_T	total gas pressure of CO_2, CH_4, N_2 gases and water vapor (760 mmHg)
Q	liquid flowrate (L/d)

Q_g	gas outflow from the reactor (L/d)
Q_i	gas outflow of CO_2, CH_4 and N_2 gases (L/d)
Q_{H_2O}	gas outflow of water vapor (L/d)
R	universal gas constant (0.082057 L-atm/mole. ° K)
S_1	complex substrate concentration in the reactor (mM)
S_2	acetate concentration in the reactor (mM)
S_{1O}	influent complex substrate concentration (mM)
S_{2O}	influent acetate concentration (mM)
$Temp$	temperature (° C)
T_{Gi}	gas transfer rate (mM/d)
V	liquid volume in the reactor (L)
V_g	gas volume in the reactor (L)
X_1	acidogen concentration in the reactor (mM)
X_2	methanogen concentration in the reactor (mM)
X_{1O}	influent acidogen concentration (mM)
X_{2O}	influent methanogen concentration (mM)
X_{1E}	effluent acidogen concentration (mM)
X_{2E}	effluent methanogen concentration (mM)
Y_{XS1}	acidogen yield per complex substrate utilized (mole/mole)
Y_{XS2}	methanogen yield per acetate utilized (mole/mole)
$Y_{CO_2X_2^1}$	carbon dioxide yield per methanogen produced (mole/mole)
$Y_{CO_2X_2^2}$	carbon dioxide yield per methanogen decay (2.5 mole/mole)
$Y_{CH_4X_2^1}$	methane yield per methanogen produced (mole/mole)
$Y_{CH_4X_2^2}$	methane yield per methanogen decay (2.5 mole/mole)
$Y_{NH_3X_1^1}$	ammonia yield per acidogen produced (mole/mole)
$Y_{NH_3X_1^2}$	ammonia yield per acidogen decay (mole/mole)
$Y_{NH_3X_2^1}$	ammonia yield per methanogen produced (mole/mole)
$Y_{NH_3X_2^2}$	ammonia yield per methanogen decay (mole/mole)
Z	alkalinity in the reactor (meq/L)
Zo	influent alkalinity (meq/L)

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CHAPTER 3

GREENHOUSE GAS PRODUCTION: A COMPARISON BETWEEN AEROBIC AND ANAEROBIC WASTEWATER TREATMENT TECHNOLOGY

ABSTRACT

Anaerobic wastewater treatment offers improved energy conservation with potential reduction in greenhouse gas emissions. Pitfalls exist in that the methane produced in anaerobic treatment can offset any reductions in carbon dioxide emissions, if it is released to the environment. This paper analyzes greenhouse gas emissions from both aerobic and anaerobic treatment systems, including sludge digestion and the losses of dissolved methane in digested biosolids and process effluents. There exists cross over points, ranging from 300 to 700 mg/L influent wastewater BOD_u, which are functions of the efficiency of the aerobic treatment system. Anaerobic treatment becomes favorable when treating influents higher in concentrations than the cross over values. A technology to recover dissolved methane would make anaerobic treatment favorable at nearly all influent strengths.

Keywords: Aerobic, anaerobic; carbon dioxide; global warming, greenhouse gas, methane, wastewater treatment

3.1 INTRODUCTION

During the last 200 years atmospheric concentrations of greenhouse gases, CO_2, CH_4 and N_2O have increased due to anthropogenic activities such as production and use of fossil fuels and other agricultural and industrial activities (El-Fadel and Massoud, 2001). To compare the effect between different gases, their global warming potentials (GWP) were estimated and referenced to CO_2. For a 100-year horizon, CO_2, CH_4 and N_2O have GWPs of 1, 21 and 310 respectively (Keller and Hartley, 2003). Wastewater treatment can contribute to greenhouse gases through production of CH_4 or CO_2 from treatment processes or from CO_2 produced from the energy required for treatment. CH_4 produced from sewage treatment was found to constitute 5% of the global methane sources (El-Fadel and Massoud, 2001).

The objective of this paper is to estimate greenhouse gas production from domestic wastewater treatment and to investigate the differences between aerobic and anaerobic methods. In anaerobic treatment complex wastes are stabilized in three basic steps: hydrolysis, acid fermentation and methanogenesis. Anaerobic treatment processes can release the more harmful greenhouse gas CH_4. Aerobic treatment involves the conversion of organic waste to biomass and CO_2 by an aerobic bacterial culture, and generally does not produce CH_4 unless biosolids are digested anaerobically. One key difference between this analysis and previous analyses is consideration of anaerobic wastewater treatment technologies, such as upflow anaerobic sludge blanket reactors (UASBs) and anaerobic filters (AFs). Previous analyses have usually considered only aerobic wastewater treatment and anaerobic sludge treatment. The greenhouse gases, CO_2,

CH_4 and N_2O can all be produced in wastewater treatment but only CO_2 and CH_4 are included in this analysis, while N_2O is neglected.

3.2 METHODOLOGY

The approach taken in this analysis is to model two different types of treatment systems providing the equivalent of secondary treatment. Most secondary treatment plants consist of primary clarification, aerobic biological treatment such as the activated sludge process, or an anaerobic treatment technology such as a UASB reactor, and anaerobic digestion for biosolids treatment. Three cases were considered for the aerobic technology: a conventional activated sludge process with a solids retention time (SRT) of 10 days, an extended aeration activated sludge process with a SRT of 30 days and a high-rate activated sludge process with a SRT of 5 days. The anaerobic technology was assumed to have an SRT of 30 days, which requires that biomass can be accumulated in the reactor, as occurs with a UASB or AF. The process parameters used in the model are shown in Table 3.1.

Figure 3.1 shows a general layout of the treatment plant including the energy and greenhouse gas contributions. The energy produced from digester and anaerobic reactor biogas is used for digester heating and power generation. The total greenhouse gas production is calculated from each part of the system as shown below in equation 3.1, using equivalence factors for the greenhouse potential of CH_4.

$$T_{CO_2}^{Equivalent} = T_{CO_2}^{ASP\_or\_AR} + T_{CO_2}^{Methane\_Burning} + T_{CO_2}^{Power\_Generation} + 21 \times T_{CH_4}^{D\_Effluent} + 21 \times T_{CH_4}^{AR\_Effluent} \tag{3.1}$$

Figure 3.2 shows the aerobic treatment system, and the model includes CO_2 production from biological oxidation of substrates in the wastewater, biomass decay and CO_2 production from anaerobic digestion and biogas combustion. The CO_2 production from power consumption is included, and the reduction in CO_2 production from energy conservation by using digester gas for heating and power generation is also considered. Figure 3.3 shows the anaerobic treatment system and the model includes CO_2 contributions that are similar to the aerobic case, as well as CH_4 production from anaerobic wastewater treatment and dissolved CH_4 in the treatment plant effluent. Also for both technologies the dissolved CH_4 in the digester effluent is included.

The model equations are included in Figure 3.4. Subscripts and superscripts are used for simplicity and clarity of understanding. Subscripts show what the variable stands for and the superscripts show the process (Benefield and Randall, 1980; Cakir and Stenstrom, 2003; Gujer *et al.*, 1999).

3.3 RESULTS AND DISCUSSION

The treatment efficiency of the aerobic and anaerobic technology can be seen in Figure 3.5. Influent wastewater concentrations ranged from 100 to 1100 mg/L BOD_u (BOD_u ~ COD) for both technologies. The activated sludge plant had a mean effluent concentration of 15 mg/L BOD_u compared to 28 mg/L BOD_u for the anaerobic reactor.

The treatment efficiency for aerobic and anaerobic treatment was 75-98 % and 51-96 % respectively. The anaerobic treatment might require some post treatment to achieve discharge limits, depending on where it is located.

A key difference in this analysis and previous analyses is the impact of the CH_4 contained in treated wastewaters. For anaerobic digesters, this term is generally small, because anaerobic digesters treat low flows of concentrated biosolids. For anaerobic wastewater treatment, the mass of dissolved CH_4 in the process effluent can be as large as the recovered CH_4, especially for low strength wastewaters. Previous analyses have generally ignored dissolved CH_4. For example for an influent concentration of 100 mg/L BOD_u, mass of CH_4 in digester effluent is 0.01 ton/day which is very small compared to 3.27 ton/day of CH_4 in anaerobic reactor effluent (Cakir and Stenstrom, 2003). CH_4 dissolved in the anaerobic reactor effluent is a major contributor for greenhouse gas production especially for low strength wastewaters.

The dissolved CH_4 in the anaerobic effluents is calculated using Henry's Law and the partial pressure of CH_4 in the anaerobic reactor gas. The CH_4 partial pressure in the anaerobic reactor biogas ranged between 0.18 to 0.76 atm and increased with increasing influent strength. The CH_4 in the digested sludge effluent is calculated assuming 0.65 atm CH_4 partial pressure. The treatment efficiency and methane partial pressures in the anaerobic reactor are calculated using the authors' previously developed mathematical model.

The equivalent CO_2 production from aerobic and anaerobic wastewater treatment is shown in Figure 3.6. At higher influent concentrations, the anaerobic process produces "negative" CO_2 due to biogas combustion that supplies energy that would have otherwise required fossil fuel combustion. Cross over points for extended aeration, conventional and high-rate activated sludge process are about 300 mg/L, 500 mg/L and 700 mg/L BOD_u respectively. For influent wastewater concentrations above cross over points, anaerobic technology becomes a feasible technology for wastewater treatment that produces lower greenhouse gas emissions. A technique to capture CH_4 lost in the anaerobic reactor effluent so that it could be used as a fuel gas would make the anaerobic treatment more favorable even for very low influent wastewater strengths.

There exist significant differences between this analysis and previous analyses. El-Fadel and Massoud (2001) considered a variety of treatment systems but did not assume that CH_4 produced from anaerobic sludge digestion is routinely combusted. In United States and other developed countries methane from anaerobic digestion is combusted to produce energy at large treatment plants and is at least flared and converted to CO_2 at smaller plants (Monteith *et al.*, 2003). El-Fadel and Massoud's analysis will overestimate CH_4 release from many treatment plants. Monteith *et al.* (2003) considered anaerobic sludge digestion and combustion of the biogas, but did not consider anaerobic wastewater treatment. Keller and Hartley (2003) considered anaerobic wastewater treatment, but assumed that the dissolved CH_4 in the effluent is captured and not released to the environment. It is not clear how this can be economically achieved for all processes.

The analysis presented in this paper considers both anaerobic wastewater treatment as well as discharge of dissolved CH_4 in both the digested biosolids as well as the anaerobic wastewater effluents. This provides a more realistic evaluation of process alternatives. It is shown that there exists a cross over point as a function of influent wastewater strength where anaerobic wastewater treatment becomes favorable with respect to greenhouse gas emissions. The cross over points range from 300 to 700 mg/L influent BOD_u, depending upon the type of aerobic process being used for comparison.

3.4 CONCLUSIONS

This paper compares greenhouse gas production by aerobic and anaerobic treatment systems, including anaerobic wastewater treatment by processes such as the upflow anaerobic sludge blanket reactor. Methane recovery as well as methane losses in process effluents and digested biosolids are considered. The analysis shows that for very low strength wastewaters (less than 300 mg/L BOD_u), aerobic processes will emit less greenhouse gas. At higher strengths, anaerobic wastewater treatment is more favorable, and the cross over point depends upon the relative efficiency of the aerobic system. A technology to economically recover dissolved CH_4 from process effluents could make anaerobic wastewater treatment more favorable in reducing greenhouse gas emissions at all influent strengths.

Table 3.1. Process parameters used in the model

Overall Facility
Flow (Q) = 37,850 m^3/d (10 MGD)
Influent BOD_u (S_{INF}^{PC}) = 100-1100 mg/L
Influent TSS (SS_{INF}^{PC}) = 49-543 mg/L
Primary Clarifier
BOD_u removal ($RR_{BOD_u}^{PC}$) = 40 %
TSS removal (RR_{TSS}^{PC}) = 60 %
Activated Sludge Process
Mean Effluent BOD_u (S_{EFF}^{ASP}) = 15 mg/L
SRT (θ_C^{ASP}) = 5, 10, 30 d
Yield (Y^{ASP}) = 0.5 g VSS/g BOD_u
Decay rate (k_d^{ASP}) = 0.06 d^{-1}
CO_2 yield ($Y_{CO_2}^{ASP}$) = 1.375 kg CO_2/kg BOD_u
Anaerobic Reactor
Mean Effluent BOD_u (S_{EFF}^{AR}) = 28 mg/L
SRT (θ_C^{AR}) = 30 d
Yield (Y^{AR}) = 0.05 g VSS/g BOD_u
Decay rate (k_d^{AR}) = 0.03 d^{-1}
CO_2 yield ($Y_{CO_2}^{AR}$) = 0.6875 kg CO_2/kg BOD_u
CH_4 yield ($Y_{CH_4}^{AR}$) = 0.25 kg CH_4/kg BOD_u
Anaerobic Digestion
SRT (θ_C^{D}) = ~12 d
Yield (Y^{D}) = 0.04 g VSS/g BOD_u
Decay rate (k_d^{D}) = 0.015 d^{-1}
CH_4 yield ($Y_{CH_4}^{D}$) = 0.25 kg CH_4/kg BOD_u
Energy
Heat conversion efficiency ($Eff_{heat-conversion}$) = 0.83
Electrical conversion efficiency ($Eff_{electrical-conversion}$) = 0.5
Aeration efficiency ($Eff_{aeration}$) = 2 kg O_2/kW.h
CO_2 production in power generation ($Y_{CO_2}^{Power\_Generation}$) = 0.96 kg CO_2/kW.h

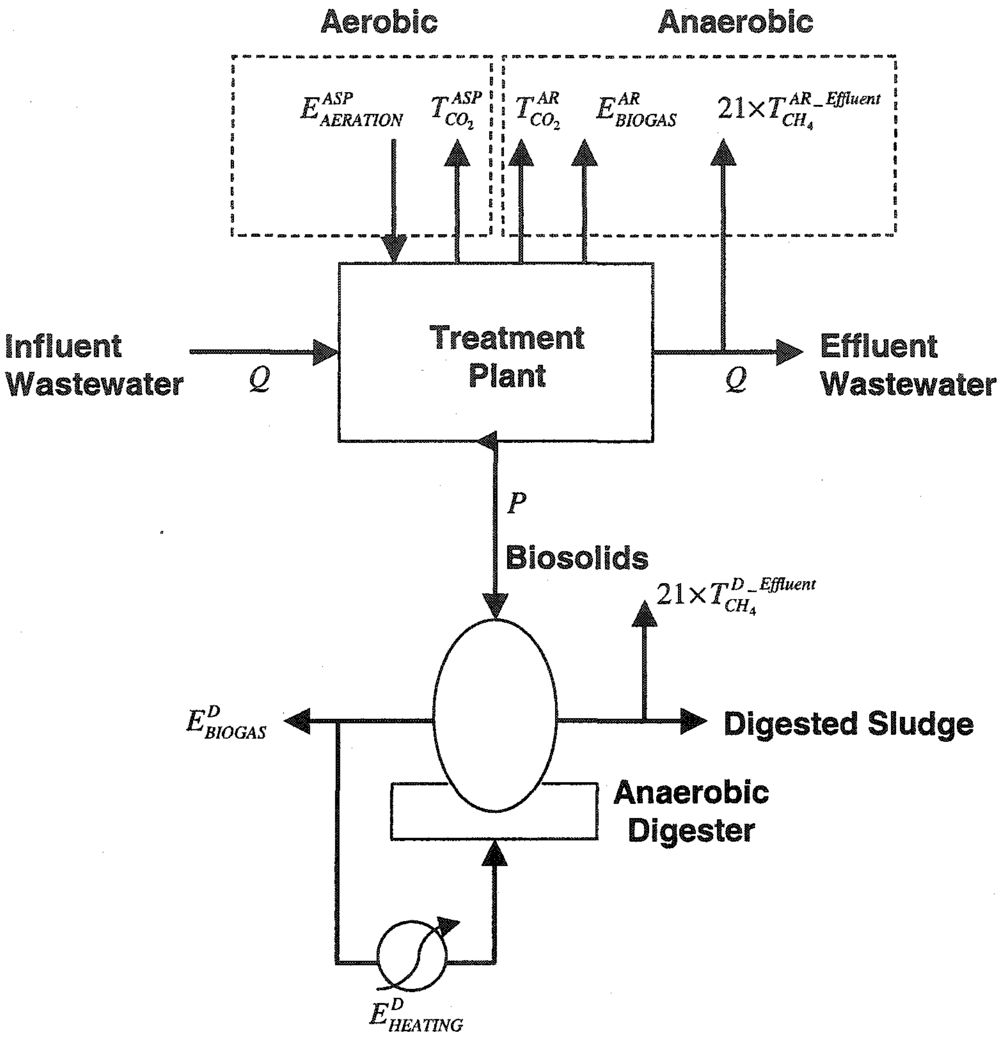


Fig. 3.1. General layout of the treatment plant

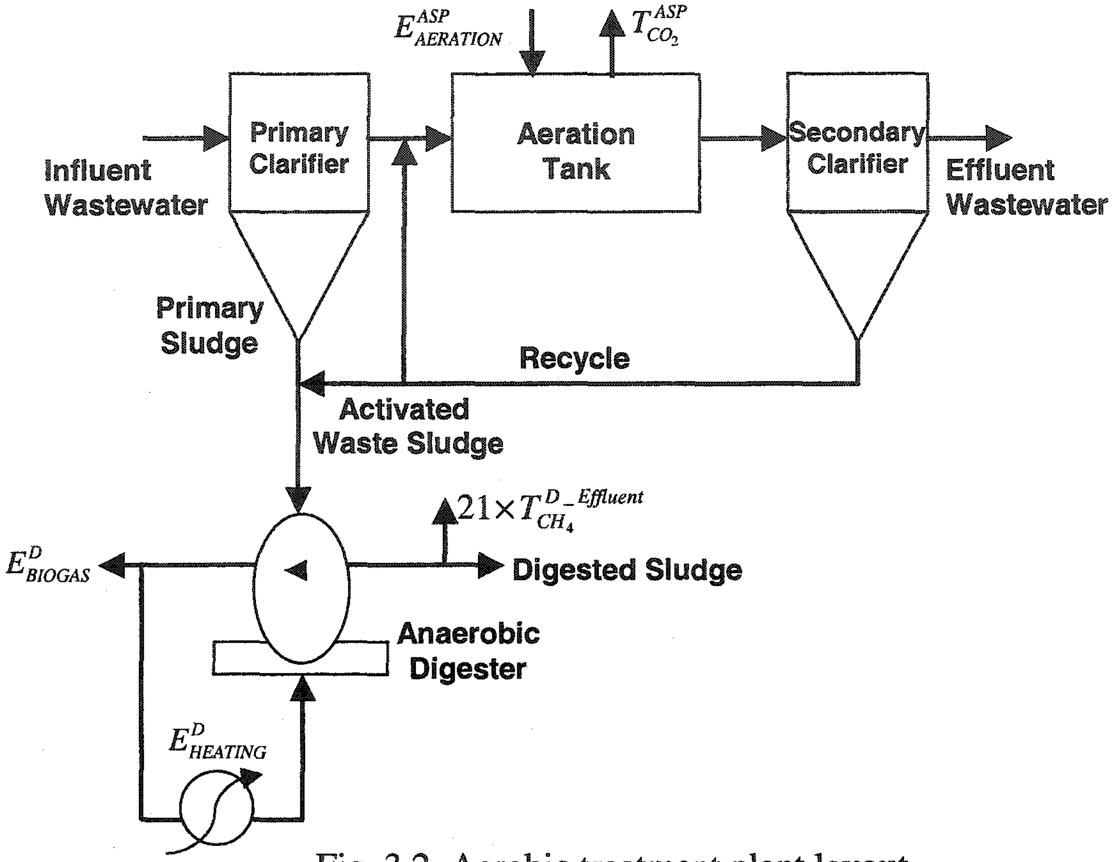


Fig. 3.2. Aerobic treatment plant layout

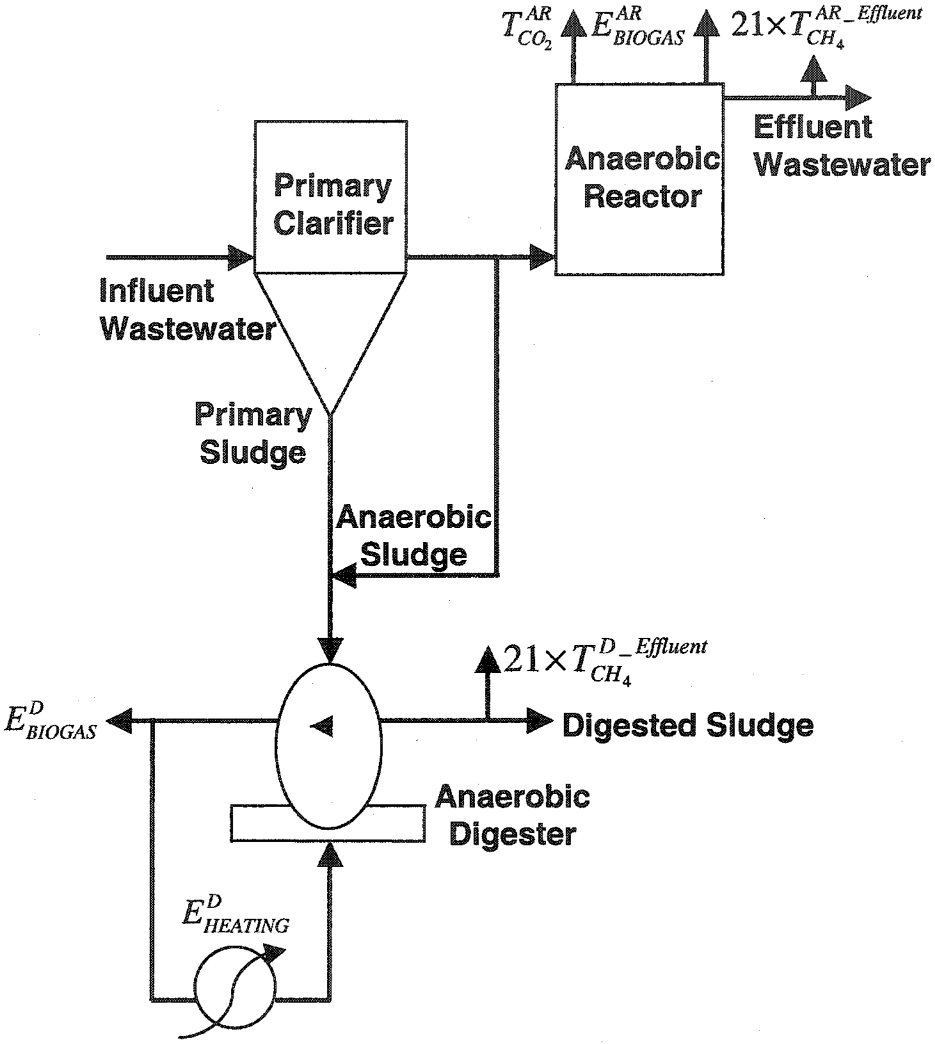
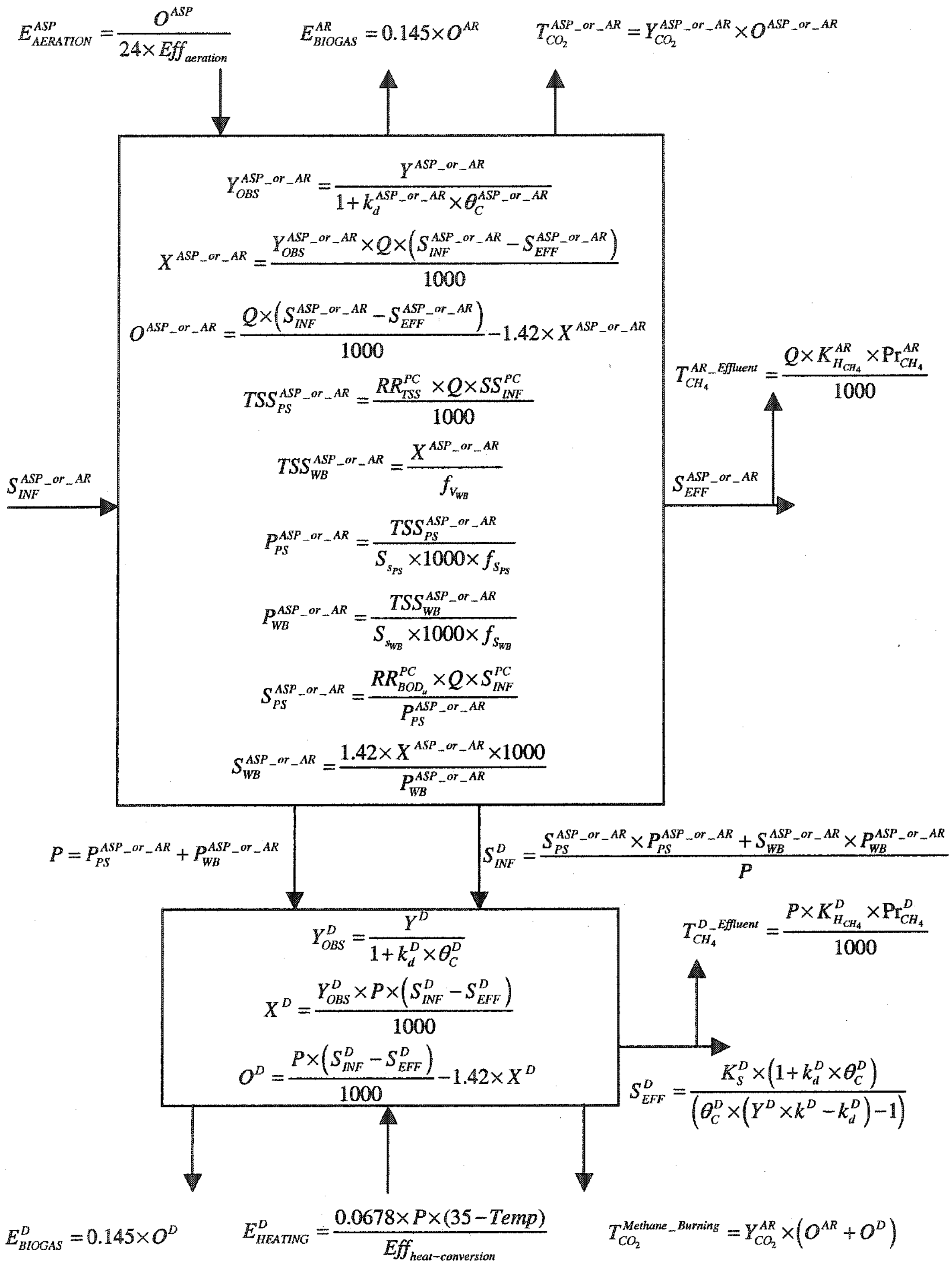


Fig. 3.3. Anaerobic treatment plant layout



$$E_{AERATION}^{ASP}=\frac{O^{ASP}}{24\times Eff_{aeration}}$$

$$E_{BIOGAS}^{AR}=0.145\times O^{AR}$$

$$T_{CO_2}^{ASP\_or\_AR}=Y_{CO_2}^{ASP\_or\_AR}\times O^{ASP\_or\_AR}$$

$$Y_{OBS}^{ASP\_or\_AR}=\frac{Y^{ASP\_or\_AR}}{1+k_d^{ASP\_or\_AR}\times\theta_C^{ASP\_or\_AR}}$$

$$X^{ASP\_or\_AR}=\frac{Y_{OBS}^{ASP\_or\_AR}\times Q\times\left(S_{INF}^{ASP\_or\_AR}-S_{EFF}^{ASP\_or\_AR}\right)}{1000}$$

$$O^{ASP\_or\_AR}=\frac{Q\times\left(S_{INF}^{ASP\_or\_AR}-S_{EFF}^{ASP\_or\_AR}\right)}{1000}-1.42\times X^{ASP\_or\_AR}$$

$$TSS_{PS}^{ASP\_or\_AR}=\frac{RR_{TSS}^{PC}\times Q\times SS_{INF}^{PC}}{1000}$$

$$TSS_{WB}^{ASP\_or\_AR}=\frac{X^{ASP\_or\_AR}}{f_{V_{WB}}}$$

$$P_{PS}^{ASP\_or\_AR}=\frac{TSS_{PS}^{ASP\_or\_AR}}{S_{s_{PS}}\times1000\times f_{S_{PS}}}$$

$$P_{WB}^{ASP\_or\_AR}=\frac{TSS_{WB}^{ASP\_or\_AR}}{S_{s_{WB}}\times1000\times f_{S_{WB}}}$$

$$S_{PS}^{ASP\_or\_AR}=\frac{RR_{BOD_u}^{PC}\times Q\times S_{INF}^{PC}}{P_{PS}^{ASP\_or\_AR}}$$

$$S_{WB}^{ASP\_or\_AR}=\frac{1.42\times X^{ASP\_or\_AR}\times1000}{P_{WB}^{ASP\_or\_AR}}$$

$$S_{INF}^{ASP\_or\_AR}$$

$$S_{EFF}^{ASP\_or\_AR}$$

$$T_{CH_4}^{AR\_Effluent}=\frac{Q\times K_{H_{CH_4}}^{AR}\times\mathrm{Pr}_{CH_4}^{AR}}{1000}$$

$$P=P_{PS}^{ASP\_or\_AR}+P_{WB}^{ASP\_or\_AR}$$

$$S_{INF}^{D}=\frac{S_{PS}^{ASP\_or\_AR}\times P_{PS}^{ASP\_or\_AR}+S_{WB}^{ASP\_or\_AR}\times P_{WB}^{ASP\_or\_AR}}{P}$$

$$Y_{OBS}^{D}=\frac{Y^{D}}{1+k_d^{D}\times\theta_C^{D}}$$

$$X^{D}=\frac{Y_{OBS}^{D}\times P\times\left(S_{INF}^{D}-S_{EFF}^{D}\right)}{1000}$$

$$O^{D}=\frac{P\times\left(S_{INF}^{D}-S_{EFF}^{D}\right)}{1000}-1.42\times X^{D}$$

$$T_{CH_4}^{D\_Effluent}=\frac{P\times K_{H_{CH_4}}^{D}\times\mathrm{Pr}_{CH_4}^{D}}{1000}$$

$$S_{EFF}^{D}=\frac{K_S^{D}\times\left(1+k_d^{D}\times\theta_C^{D}\right)}{\left(\theta_C^{D}\times\left(Y^{D}\times k^{D}-k_d^{D}\right)-1\right)}$$

$$E_{BIOGAS}^{D}=0.145\times O^{D}$$

$$E_{HEATING}^{D}=\frac{0.0678\times P\times(35-Temp)}{Eff_{heat-conversion}}$$

$$T_{CO_2}^{Methane\_Burning}=Y_{CO_2}^{AR}\times\left(O^{AR}+O^{D}\right)$$

Fig. 3.4. Equations used in the model
(wastewater treatment reactor, activated sludge process or anaerobic reactor, top; anaerobic digester, bottom)

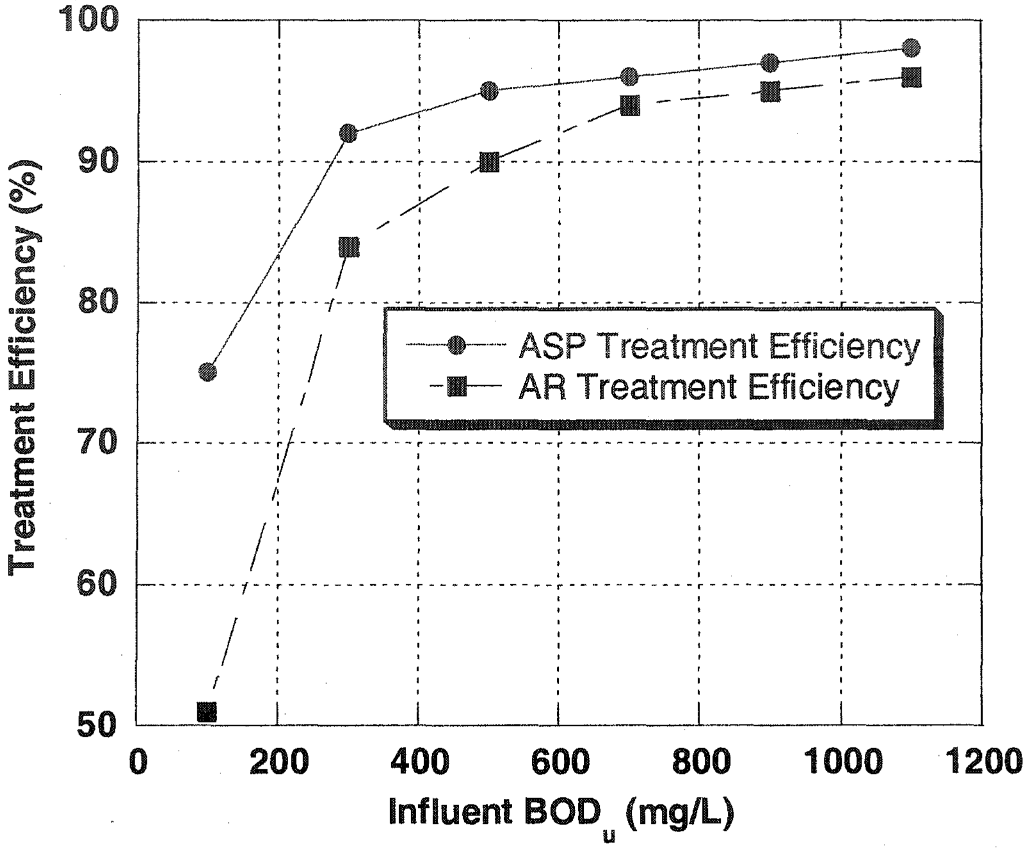


Fig. 3.5. Treatment efficiency versus influent BOD$_u$

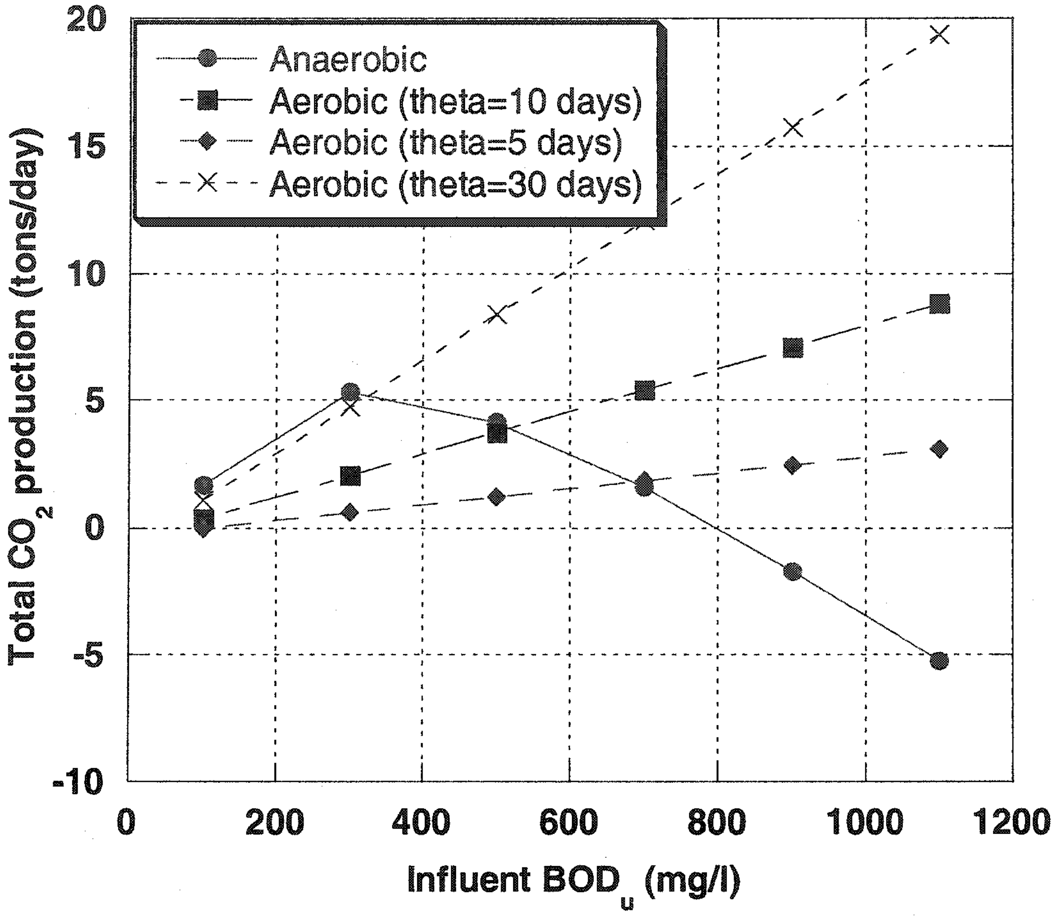


Fig. 3.6. Total equivalent CO_2 production versus influent BOD_u

3.5 NOMENCLATURE

AR:	Anaerobic reactor
ASP:	Activated sludge process
D:	Digester
E_{BIOGAS} :	Energy produced from biogas (kW)
$E_{AERATION}^{ASP}$:	Energy required for aeration (kW)
$E_{HEATING}^{D}$:	Energy required for digester heating (kW)
$Eff_{aeration}$:	Aeration efficiency (kg/kW.h)
$Eff_{electrical-conversion}$:	Electrical conversion efficiency
$Eff_{heat-conversion}$:	Heat conversion efficiency
f_S :	Weight fraction of sludge that is solids
f_V :	Weight fraction of solids that are volatile
k :	Maximum rate of substrate utilization per unit mass of biomass ($gBOD_u/gVSS.d$)
k_d :	Decay rate (d^{-1})
K_H :	Henry's Law constant (mg/L.atm)
O :	Oxygen requirement (kg/d)
θ_C :	Solids retention time (d)
P :	Sludge flow rate (m^3/d)
$\Pr$:	Partial pressure of gas (atm)
PS:	Primary sludge
Q:	Wastewater flow rate (m^3/d)
RR :	Removal rate
S :	Substrate (mg/L)
S_s :	Specific gravity of sludge
SS :	Total suspended solids (mg/L)
$Temp$:	Influent wastewater temperature (^{o}C)
TSS :	Total suspended solids removal (kg/d)
T :	Greenhouse gas production (kg/d)
WB:	Waste biosolids
X :	Biomass production (kg/d)
Y_{OBS} :	Observed biomass yield (g VSS/g BOD_u)
Y :	Biomass yield (g VSS/g BOD_u)
Y_{CH_4} :	CH_4 yield (kg CH_4/kg BOD_u)
Y_{CO_2} :	CO_2 yield (kg CO_2/kg BOD_u)

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CHAPTER 4

CONCLUSIONS

The literature review showed that anaerobic treatment is feasible and economical for low strength wastewaters. Anaerobic treatment is much more likely to be applicable in developing countries for several reasons. These countries lack wastewater infrastructure, so they need low cost, simple treatment systems which can be operated and maintained easily with less trained staff. Also the energy costs in these countries are usually high or there is no available energy. For their applications, treatment systems should be energy efficient and require less energy. Anaerobic treatment methods such as anaerobic filter (AF), upflow anaerobic sludge blanket reactor (UASB) or hybrid reactor might be sufficient for these countries as there is less concern for effluent quality and reclamation requirements.

Aerobic methods, especially the activated sludge process, are the conventional way of treating low strength domestic wastewaters in US. No full scale anaerobic technology has been used for low strength wastewater and it is not expected in the near future. Energy, sludge disposal, fuel costs and surcharge rates have been low so there has been too little incentive to use alternative energy efficient systems. Also anaerobic reactors require longer hydraulic retention times than activated sludge plants and for low strength wastewaters the payback period for the additional reactor size might be too long for companies wanting quick payback.

Anaerobic wastewater treatment systems increased during the last 20 years primarily in developing countries in tropical regions and anaerobic treatment has become a more mature technology. Further research is needed using pilot and full scale anaerobic filters and hybrid reactors treating low strength wastewaters at lower temperatures.

A dynamic model was developed to predict treatment efficiency, gas production and composition from an anaerobic filter and previous pilot scale data was simulated with using this model. The model accurately predicts the high nitrogen partial pressure for low strength wastewater. This is due to the dissolved nitrogen in the influent wastewater. For very low strength wastewater methane gas composition was low and dissolved methane is a large fraction of the total methane produced. A hydraulic retention time of 12-24 hr is suggested to achieve greater than 60% COD removal.

Another model was developed to estimate greenhouse gas production from domestic wastewater. Aerobic and anaerobic processes were compared with this model. Cross over points for extended aeration, conventional and high-rate activated sludge process are about 300, 500 and 700 mg/L COD respectively. Above these cross over points anaerobic processes emits less greenhouse gas. At higher influent concentrations the anaerobic process produces “negative” CO_2 or CO_2 credit due to biogas combustion that supplies energy that would have otherwise required fossil fuel combustion. For anaerobic processes the mass of dissolved methane in the reactor effluent can be as large as the recovered methane for very low strength wastewaters. If the methane lost in the anaerobic reactor effluent could be captured to be used as a fuel gas then anaerobic treatment would be more favorable even for very low influent wastewater strengths.

Appendix A

A DYNAMIC MODEL FOR ANAEROBIC FILTER

ABSTRACT

Anaerobic filters are frequently used to treat medium to high strength wastewater (2,000 to 20,000 mg/L COD), but have fewer applications to low strength wastewater (< 1,000 mg/L COD). In order to understand the applicability of anaerobic treatment for low strength wastewater, such as domestic sewage, a dynamic mathematical model was developed. This Monod-type kinetic model, which predicts treatment efficiency and gas production, was used to describe literature observations and is offered as a predictive tool.

Keywords: Anaerobic filter; domestic wastewater; dynamic model; low strength wastewater; mathematical model; UASB reactor

INTRODUCTION

Anaerobic treatment has traditionally been used for treatment of sludges and high strength wastewater. This was due to the need for elevated temperatures for slow growing methanogens. The invention of new anaerobic systems such as the anaerobic filter (AF), upflow anaerobic sludge blanket (UASB) and hybrid reactors (a combination of UASB and AF) has reduced or eliminated the need for elevated temperatures by maintaining long solids retention time (SRT) independent of the hydraulic retention time (HRT). This change enabled treatment of low strength wastes such as domestic wastewater.

The anaerobic filter uses a rock or synthetic media for biomass growth. The wastewater flows through the media, and biomass is retained on the media, which makes the hydraulic and solids retention time independent. Coulter (1957) was the first to develop AF process, but it was not used until 1969 when Young and McCarty studied the treatment of a protein-carbohydrate wastewater (1500-6000 mg/L COD) at 25°C, at organic loading rates (OLR) of 0.96-3.40 kg COD/m^{3}d. The UASB is an alternative process, developed primarily by Lettinga and his associates (1980, 1993) and retains biomass in granular sludge beds, which decouples the hydraulic and solids retention times. The UASB process is now widely used except in the United States, where it and other anaerobic wastewater treatment technologies are unpopular.

The objective of this paper is to describe a dynamic model that can be used for anaerobic filters, and demonstrate treatment efficiency of domestic wastewater. This predictive tool is offered to improve process acceptance.

MODEL DEVELOPMENT

Anaerobic treatment of waste is a complex biological process involving several groups of microorganisms (Cha & Noike, 1997; Harper & Pohland, 1997; Jianrong *et al.*, 1997). In general complex wastes are stabilized in three basic steps: hydrolysis, acid fermentation and methanogenesis. In the acid fermentation step the organic waste is decomposed into lower fatty acids such as acetic and propionic by acid forming bacteria.

In methanogenesis these fatty acids are broken down into CO_2 and CH_4 by methanogens (Speece, 1996). The growth rate of the methanogens is low and is usually the rate-limiting step. Long SRT is required to retain the slow growing methanogens.

The following model is a dynamic model describing anaerobic treatment using anaerobic filters. The model predicts treatment efficiency as well as gas production and composition. The model assumes methane formation from acetate is the rate-limiting step. Therefore, the model is simplified to methanogenesis, and hydrolysis and fermentation steps are not considered. This is a valid assumption for highly degradable low strength wastewater. The model is based in part on earlier models developed by Andrews (1969, 1971). The model is restricted to low strength influents, and does not require the more advanced concepts that separate substrates and biomasses into different pools (Mosey, 1983; Moletta *et al.*, 1986; Suidan *et al.*, 1994; Jeyaseelan, 1997; Batstone *et al.*, 2000; Karama *et al.*, 2000). The model includes the physical, chemical and biological interaction between gas, liquid and biological phases. The model is composed of 10 ordinary differential equations. The general material balance equation (Accumulation = Input – Output + Production – Utilization) was used for the corresponding 10 state variables: substrate, and biomass in the biological phase; CO_2, N_2 and CH_4 partial pressures in the gas phase; alkalinity, dissolved CO_2, N_2, CH_4 and NH_3 in the liquid phase. The equations are shown in Figure 1.

Stoichiometry

A generalized stoichiometric relationship showing the conversion of acetic acid (CH_3COOH) to methane and carbon dioxide with the synthesis of biomass ($C_5H_7NO_2$) and the decay of biomass is given respectively in equations (1) and (2).

$$CH_3COOH + Y_{XS}.Y_{NH_3X^1}NH_3 \rightarrow Y_{XS}C_5H_7NO_2 + Y_{XS}.Y_{CO_2X^1}CO_2 + Y_{XS}.Y_{CH_4X^1}CH_4 + aH_2O \tag{1}$$

$$C_5H_7NO_2 + bH_2O \rightarrow Y_{CO_2X^2}CO_2 + Y_{CH_4X^2}CH_4 + Y_{NH_3X^2}NH_3 \tag{2}$$

Biological Phase

The rate of change of substrate concentration (S) in the reactor at any time depends on the influent (S_o) and the utilization of substrate for biomass growth. Monod-type kinetics is used to describe the utilization of substrate.

The rate of change of biomass (X) concentration in the reactor is a function of the influent (X_o) and effluent (X_E) biomass concentrations and the biomass growth and decay in the reactor. In AF the biomass concentration in the reactor is much higher than the effluent biomass concentration as the biomass is retained in the packing media.

The production and utilization rates of dissolved CO_2, CH_4 gases and NH_3 during biomass growth and decay are shown by r_1, r_3, r_5, r_2, r_4 and r_6 respectively.

Liquid Phase

The net rate of CO_2, CH_4 and N_2 transfer (T_{Gi}) between the liquid and gas phases can be expressed by two-film theory. Henry's Law was used to determine the concentration of the gases in the liquid phase at equilibrium with the partial pressure of the gases in the gas phase. Henry's Law constants (K_{Hi}) are a function of the temperature.

The rate of change of alkalinity (Z) in the reactor depends on the influent alkalinity (Zo) and the change of bicarbonate, carbonate and ammonia concentrations in the liquid phase. The rate of change of total carbonic acid concentration in the reactor is a function of the influent carbonic acid concentration and gas transfer rate of dissolved carbon dioxide and the rate of dissolved carbon dioxide production during biological growth and decay. The N_2 gas does not undergo any biological or chemical reaction in the reactor, and it is stripped from the influent.

Methane gas is produced by the biomass and is stripped from the liquid to the gas phase. For low strength systems, the mass of dissolved nitrogen in the influent can be large compared to methane that is produced. Nitrogen is stripped from the influent to the gas phase in the reactor because it changes from being in equilibrium with 79% nitrogen gas fraction to essentially zero in the digester gas. The mass of stripped nitrogen is sufficient to reduce the methane percentage when using low strength (i.e., low methane production) wastewater.

Gas Phase

The rate of change of partial pressures of CO_2, CH_4 and N_2 gases (P_i) in the gas phase are a function of the gas transfer rate and the outflow (Q_g) from the gas phase. The partial pressure of H_2O (P_{H_2O}) changes with the temperature.

RESULTS AND DISCUSSION

Kobayashi *et al.* (1983) and Abramson's (1987) AF data were used to calibrate the model. Figure 2 show the calibration graph for removal rate and effluent substrate concentration as a function of solids retention time. Pairs of points are shown, with one member representing the observed data, and the second member representing the simulation for those conditions. The simulations are not on a smooth line, as shown in later figures, since each data point was collected at different temperatures, hydraulic retention times and influent substrate concentrations. Model predictions of the gas composition of the effluent as a function of influent substrate concentration and solids retention time are given in Figures 3 and 4. The model accurately predicts the high nitrogen partial pressure for low strength wastewater.

CONCLUSIONS

The dynamic model was able to predict treatment efficiency from previous pilot scale AF studies. Furthermore, the model simulates the gas composition of the effluent from influent characteristics. The previous data and the model suggest that 24 hr HRT is required to achieve greater than 60% COD removal. Methane composition will be less than 50%, when influent substrate concentrations are less than 130 mg/L COD at ambient temperature of 20°C.

Hopefully, the reported predictions from the dynamic model will enhance the use of anaerobic treatment in the United States and other areas where anaerobic wastewater treatment is less frequently used.

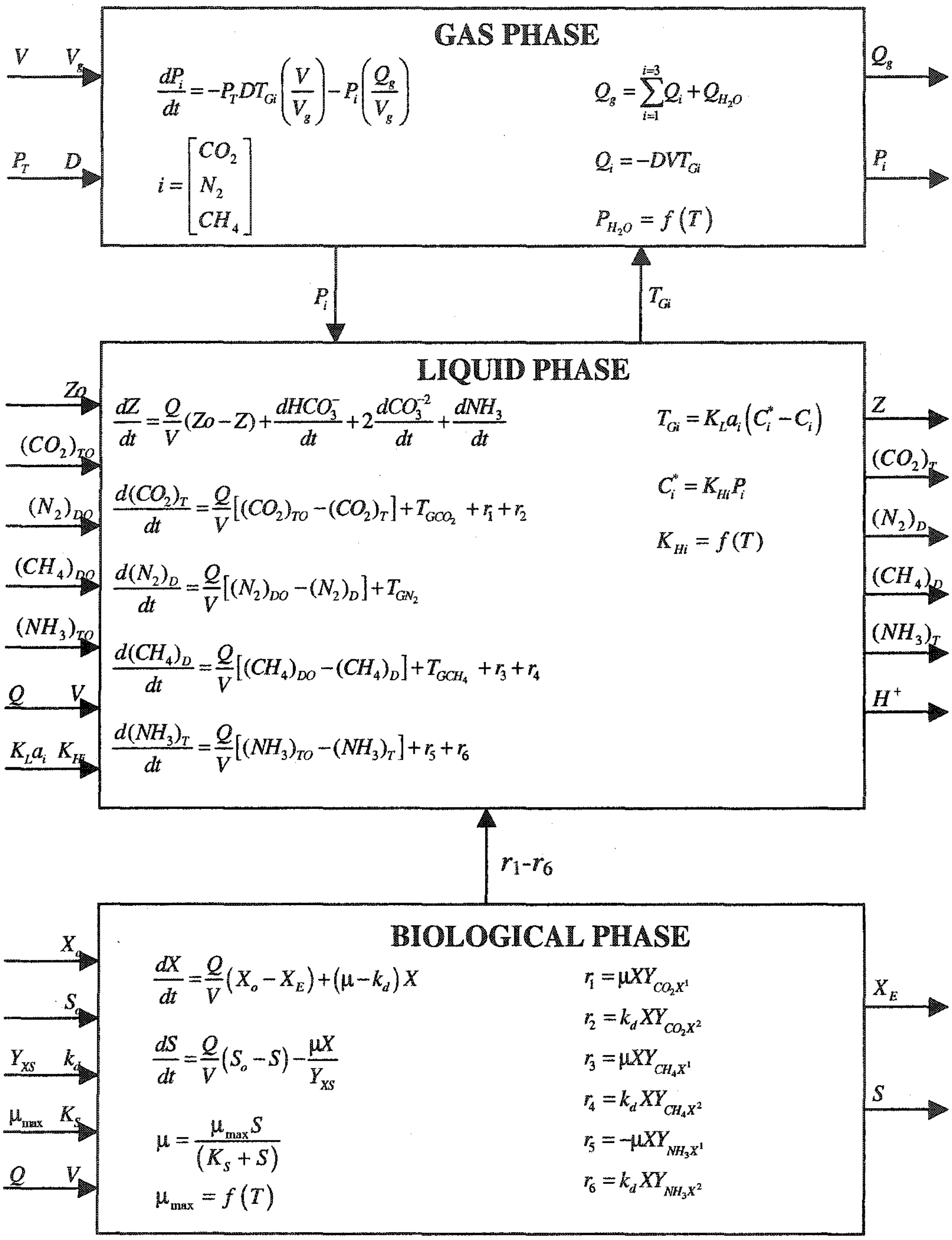

Fig. 1. Model Flow Diagram

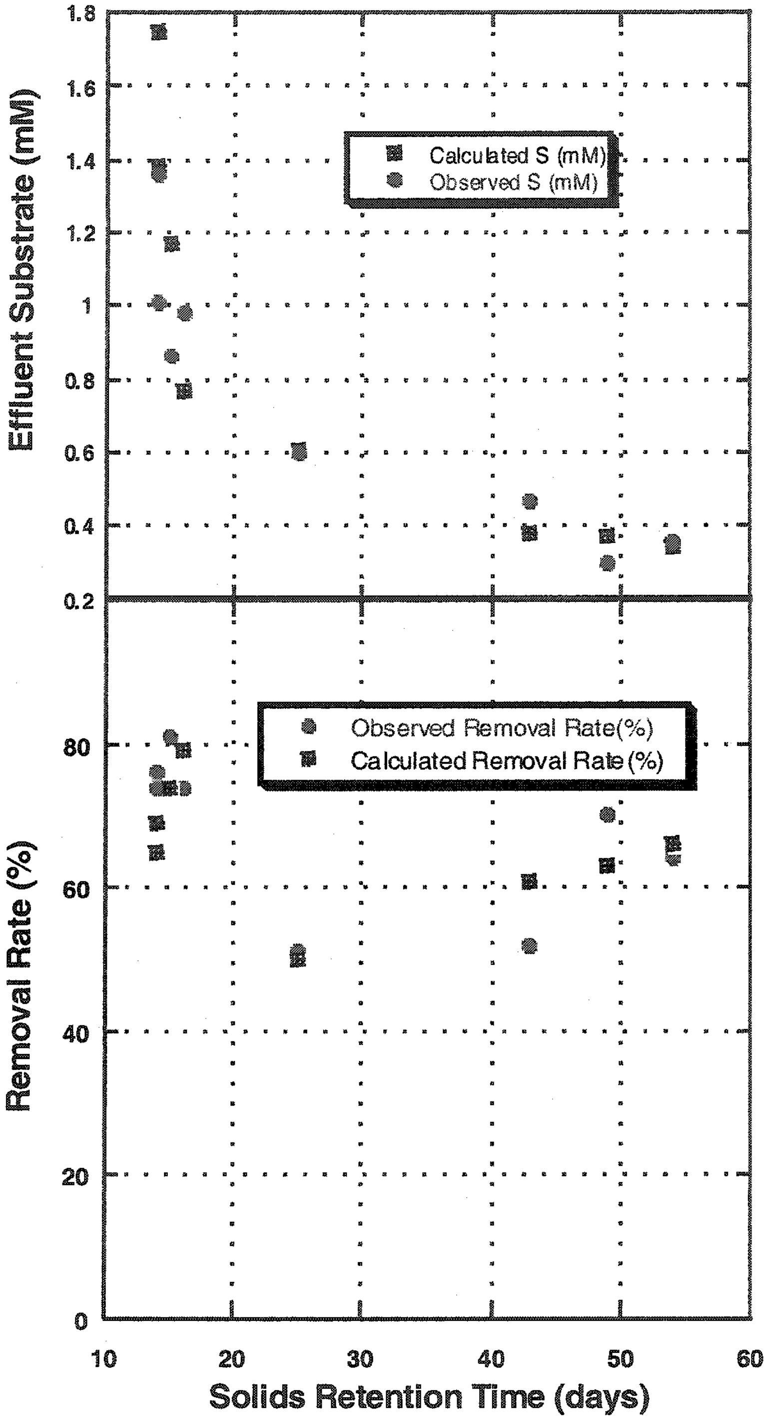

Fig. 2. Effluent substrate concentration (top) and removal rate (bottom) versus SRT

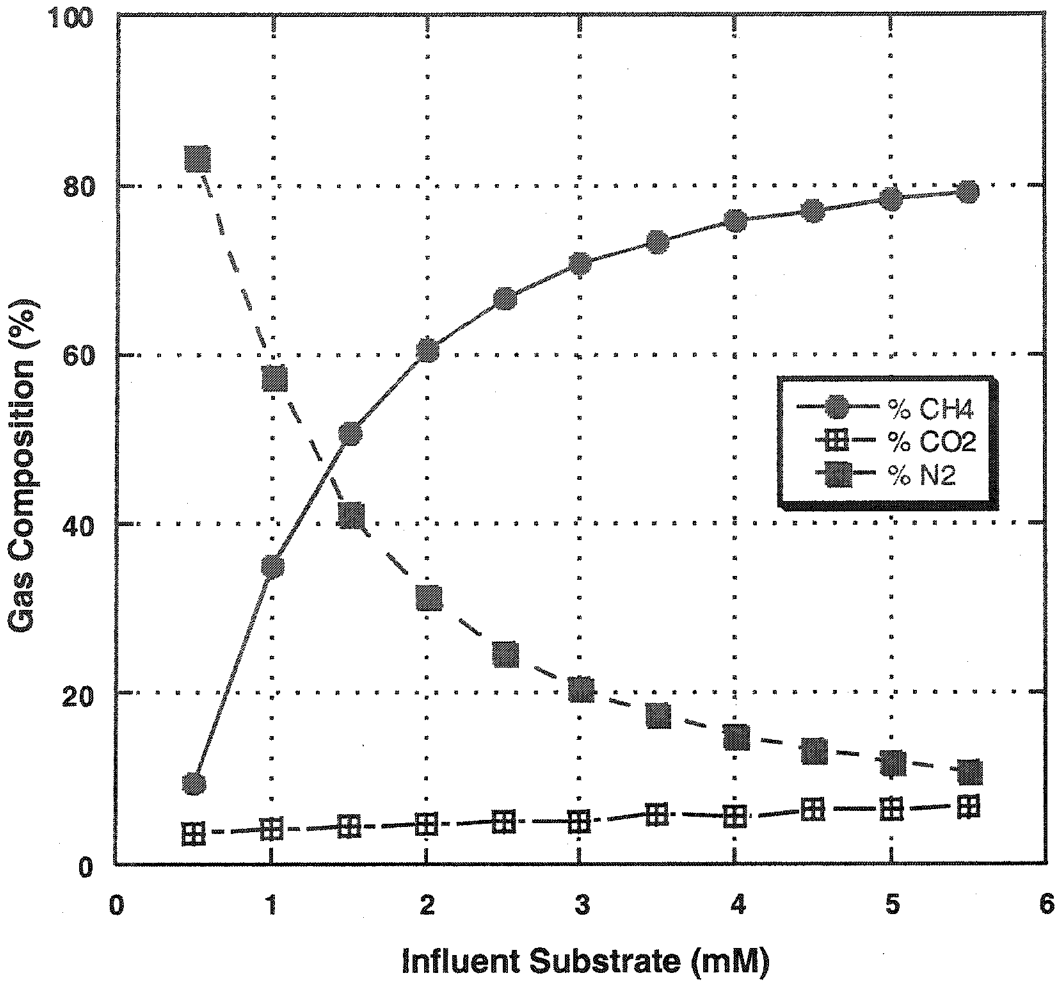

Fig. 3. Simulated gas composition versus influent substrate concentration
(Symbols represent calculated values)

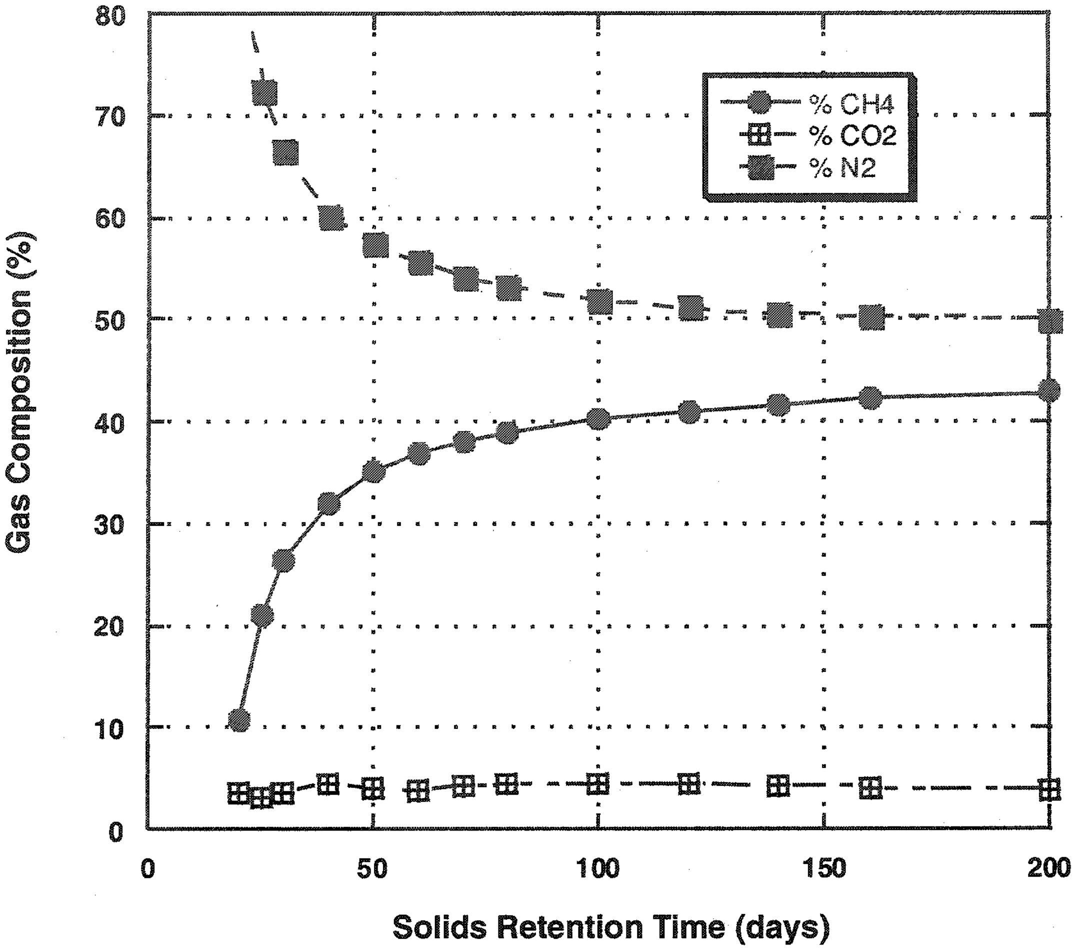

Fig. 4. Simulated gas composition versus SRT
(Symbols represent calculated values)

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Appendix B

PROCESS PARAMETERS USED IN THE ANAEROBIC FILTER MODEL

$k_{1d} =$ 0.011 d^{-1}
$k_{2d} =$ 0.011 d^{-1}
$(K_L a)_{N_2} =$ 10 d^{-1}
$(K_L a)_{CO_2} =$ $1.01 * (K_L a)_{N_2}$ d^{-1}
$(K_L a)_{CO_2} =$ $0.89 * (K_L a)_{N_2}$ d^{-1}
$(K_H)_{CO_2} =$ $0.073 / (0.72206 + 0.02969 * \theta + 0.00026693 * \theta^2)$ mM/mmHg
$(K_H)_{N_2} =$ $0.0073 / (5.2726 + 0.14661 * \theta - 0.00045931 * \theta^2)$ mM/mmHg
$(K_H)_{CH_4} =$ $(39.335 - 1.1167 * \theta + 0.017826 * \theta^2 - 0.00011491 * \theta^3) / 12160$ mM/mmHg
$K_{S1} =$ 2.5 mM
$K_{S2} =$ 0.34 mM
$(\mu_{\max 1})_{20} =$ 0.20 d^{-1}
$(\mu_{\max 2})_{20} =$ 5.4 d^{-1}
$\mu_{\max 1} =$ $(\mu_{\max 1})_{20} * 1.03^{(\theta - 20)}$ d^{-1}
$\mu_{\max 2} =$ $(\mu_{\max 2})_{20} * 1.03^{(\theta - 20)}$ d^{-1}
$P_{H_2O} =$ $5.0538 - 0.021092 * \theta + 0.030783 * \theta^2$ mmHg
$P_T =$ 760 mmHg
$R =$ 0.082057 L-atm/mole.° K
$\theta =$ 20 °C
$Y_{XS1} =$ 0.02 mole/mole
$Y_{XS2} =$ 0.0117 mole/mole
$Y_{CO_2 X_2^1} =$ 28.8 mole/mole
$Y_{CO_2 X_2^2} =$ 2.5 mole/mole
$Y_{CH_4 X_2^1} =$ 28.8 mole/mole
$Y_{CH_4 X_2^2} =$ 2.5 mole/mole

$Y_{NH_3X_1^1}$ = 1 mole/mole

$Y_{NH_3X_1^2}$ = 1 mole/mole

$Y_{NH_3X_2^1}$ = 1 mole/mole

$Y_{NH_3X_2^2}$ = 1 mole/mole