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Community Onsite Anaerobic Sewage Treatment In Hybrid and UASB-Septic Tank Systems in Palestine

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The findings, interpretations and the conclusions expressed in this study don't express the views of Birzeit University, the views of the individual members of the MSc committee or the views of their respective.

DEDICATION

TO MY COUNTRY PALESTINE

TO MY PARENTS, MY SISTERS

MY BROTHERS

TO MY HUSBAND ZAHER, MY SON

KHALED

AND ALL MY FRIENDS

With my love and respect,

Noor Alhuda Al-Hindi
September, 2007

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Abstract

In countries like Palestine, where water is scarce and wastewater is dumped untreated to the environment, it is very important to develop low cost onsite sanitation systems in order to reduce the cost of the treatment process and to maximize the reuse of the treated effluent. The acquired knowledge and developed environmentally sound and financially feasible onsite wastewater treatment technologies are especially proper for application in Palestine, but will have general value and applicability in the worldwide.

This research consists of two parts, the first one is the UASB-septic tank performance in the long run, he second part is comparing the performance of the UASB-septic tank and the AH reactors. For the first part an onsite pilot scale UASB-septic tank reactor was monitored at Al-Bireh wastewater treatment plant in Palestine treating domestic sewerage under HRT of two days. The UASB- septic tank was operated for extra 45 days after the first year at ambient temperature fluctuates between 8 to 27°C with an average value 17.3 (5.4) °C. The wastewater in the study area is classified as (high strength) regarding to Metcalf and Eddy (1991) with average COD_{tot} concentration of 1062 mg/l with (COD/BOD_5) of 2.13 .The performance data obtained during operation of the reactor for the 45 days showed average removal efficiencies for COD_{tot}, COD_{ss}, COD_{col}, COD_{dis} of 72%, 82%, 58%, 55% respectively. Removal efficiency for BOD_5 was 68% and for TS the average removal efficiency 34%. For the second part of this research the UASB-septic tank and the hybrid UASB were operated at 2 days HRT in parallel achieved average removal efficiency of COD_{tot} 52% and 56% for BOD_5. The results showed that both systems have achieved the same nutrients removal efficiencies.

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List of Abbreviations

A	acidification
Af	anaerobic filter
AH	anaerobic Hybrid
AVG	average
BOD	biological oxygen demand
COD	chemical oxygen demand
COD_{col}	colloidal COD
COD_{dis}	dissolved COD
COD_{ss}	suspended COD
COD_{tot}	total COD
$COD\text{-}CH_4$	COD as CH_4 (methane)
COD-VFA	COD as VFA (volatile fatty acid)
eff	effluent

g	gram
H	hydrolysis
HRT	hydraulic retention
Inf	influent
L	liter
M	methanogenesis
m	meter
mg	milligram
ml	milliliter
Nkj	kjeldhal nitrogen
nm	nanometer
P	phosphorous
R	reactor
RPF	Reticulated Polyurethane foam
SRT	solid retention time
SS	suspended solids
STD	standard deviation
T	temperature
TS	total solids
TSS	total suspended solids
UASB	upflow anaerobic sludge blanket
V	volume
VFA	volatile fatty acids
VS	volatile solids
VSS	volatile suspended solids
V_{up}	upflow velocity (m/hr)
WWTP	wastewater treatment plant

Chapter 1

Introduction

1.1Background

Between 90 and 95% of the sewage production in the world is released into the environmental without any treatment. Water resources are polluted by varied sources, the most critical of which are city sewage and industrial waste discharge. Developing countries suffer from the lack of proper wastewater collection and treatment facilities, especially in rural areas (Elmitwalli *et al*., 2003).

Appropriate and sustainable sewage treatment technologies will help to preserve biodiversity and maintain healthy and freshwater. Among the different treatment systems now available worldwide, the anaerobic process is attracting more and more the attention of sanitary engineers. It is used successfully in tropical countries and there are encouraging results from subtropical and temperate regions (Zeeman *et al*., 2001). Anaerobic treatment methods are becoming increasingly popular for the treatment of various wastewaters. The possibility of using up flow anaerobic sludge blanket (UASB) reactor and anaerobic filter for sewage treatment is an attractive alternative especially for developing countries where there is a need for low cost reliable method for wastewater treatment.

Domestic and industrial wastewater in Palestine is mainly collected mainly in cesspits or, to a much lesser extent in sewerage networks. In many of the Palestinian villages and refugee camps, black wastewater is collected in cesspits, while grey wastewater is discharged via open channels. About 94%of the collected wastewater from the sewered localities in the West Bank, which resembles 24% of the population, is discharged into nearby wadis without being subjected to any kind of treatment. But less than 6% is connected to treatment plants (PWA, 1997). Meanwhile about 73% of the West Bank households have cesspit sanitation and almost 3% are left without any sanitation system (PCBS, 2000). The cesspits are left without lining, so sewage infiltrates into the earth layers and eventually to groundwater. Consequently, cesspits themselves pose increasing environmental pollution problems as sewage has begun to seep into water sources. Alarming signals have been reported in some places of groundwater pollution with high concentrations of chloride (e.g. 400 mg/l), sodium (e.g. 200 mg/l), potassium (e.g.35 mg/l) and nitrate (e.g. up to 250 mg/l) in both the West Bank and Gaza Strip. Those concentrations by far exceed the recommended guiding values for drinking water by World Health Organization (WHO).

As previously stated, the 'anaerobic' cesspits which are widely applied in Palestine have severe impact on groundwater quality. A proper system is the septic tank, which is the most known and commonly applied system for on-site anaerobic pretreatment of sewage. However, the performance of the septic tanks is rather poor horizontal flow mode of the influent sewage (Lettinga *et al*., 1991; Mgana, 2003). A significant improvement of the septic tank was achieved by applying upward flow and gas /solids/liquid separation device at the top, which resulted in the so called UASB- Septic tank system (Bogte *et al*., 1993; Lettinga *et al*., 1991; Zeeman *et al*., 2000).

Anaerobic treatment methods are becoming increasingly popular for the treatment of various wastewaters (Lettinga *et al.*, 1980). Anaerobic processes have been used for the treatment of concentrated municipal and industrial wastewaters for well over a century. In the absence of molecular oxygen, these processes convert organic materials into methane, a fuel that can yield a net energy gain from process operations. Because of recent advances in treatment technology and knowledge of process microbiology, applications are now extensive for treatment of dilute industrial wastewaters as well (McCarty and Smith, 1986). The Palestinian Water Authority (PWA) stimulates the application of anaerobic treatment technologies, which hardly require any energy, on the contrary they produce energy source, i.e. methane gas (Mahmoud, 2002).

The possibility of using up flow anaerobic sludge blanket (UASB) reactor and anaerobic filter for sewage treatment is an attractive alternative especially for developing countries where there is a need for low cost reliable method for wastewater treatment.

Studies with UASB-Septic tanks treating domestic sewage are scarce, and to our knowledge so far only a one research project had been conducted on the use of a UASB-Septic tank system for the onsite sewage treatment at Dutch and Indonesian ambient conditions by Lettinga and his coworkers (Bogte *et al*., 1993; Lettinga *et al*., 1993). Nonetheless the system has not been applied and demonstrated in other countries of different environments and sewage characteristics nor has it been optimized. The sewage in Palestine is of high solids content, and is of low temperature during wintertime which entails special attention of the reactor technology. That is because the performance of the UASB reactors at low temperature climates (5-20°C) is highly limited by the low degree of hydrolysis (first step in the sequence of anaerobic digestion) of entrapped solids (Mahmoud, 2002).

1.2Wastewater in Palestine-present situation

1.2.1Wastewater management

Wastewater management in Palestine had been neglected for decades (Daibes, 2000). Domestic and industrial wastewater used to be collected mainly in cesspools or, to a much lesser extent, in sewerage networks. In many of the Palestinian villages and refugee camps, black wastewater is collected in cesspools, while grey wastewater is discharged via open channels. The majority of the collected wastewater from the sewered localities is discharged into nearby valleys without being subjected to any kind of treatment. It is estimated that about 30% of the West Bank population is served with sewerage networks, but less than 6% is connected to treatment plants (Mahmoud *et al*., 2003).

As the Palestinian society is facing heavy economical burdens, the application of conventional aerobic wastewater treatment technologies is too expensive and not providing a sustainable solution for environmental protection and resource conservation. Anaerobic digestion has been widely recognized as the core of sustainable waste management (Hammes *et al*., 2000; Zeeman and Lettinga, 1999), which has also been recognized by the Palestinian officials (PWA, 1998). The feasibility of the upflow anaerobic sludge blanket (UASB) reactor for sewage treatment has been successfully demonstrated in many tropical countries. Experience with the application of the UASB in the Middle East countries however is still limited (Zeeman and Lettinga, 1999). The main factors dictating the applicability of anaerobic technologies for domestic wastewater treatment are the sewage temperature and the characteristics and concentration of the pollutants in this sewage (Lettinga *et al*., 1993).

1.2.2 Domestic Wastewater

The quantity of domestic wastewater generated by a community is equal to 80-90% of average per capita water consumption and the total population based on a yearly per capita water consumption of 35 CM, it is estimated that 5.0 MCM of domestic wastewater is generated in the Ramallah district every year (ARIJ, 2006).

The quality of wastewater depends on daily activities and the per capita consumption of the population. The BOD from Palestinian localities is very high compared to other countries.

1.2.3 Wastewater Generation and Collection

A total of approximately 66 MCM of wastewater was generated in the OPT in the year 2005. Of this only about 36.5 MCM (55.3%) is collected by the sewage network (Table1.2). In the West Bank, only 56 communities are connected to the sewage network, whereas 513 communities use cesspits to dispose their sewage. In the Gaza Strip, 19 communities are connected to the sewage network, whereas 11 communities use cesspits. A wastewater collection network is limited to the major cities in the OPT. Many of these networks are poorly designed and suffer from leakage, especially those implemented during the 1970s. Moreover, many sewage collection pipes are of a small diameter (8-12 inches), in sufficient to deal with the input into them, making blockage and flooding frequent phenomena (ARIJ, 2006).

Thus, even existing systems need rehabilitating and upgrading. Wastewater collection networks in most of the Palestinian refugee camps (in both of the West Bank and the Gaza Strip) are either not present or undeveloped and primitive. Most camps use open channels to convey wastewater away from dwellings. From the data shown in (Table 1.2), it is evident that there is a need for development in every governorate in the OPT, and that the sewage collection network in the West Bank is more underdeveloped than that in the Gaza Strip. This is clear in terms of sewage collection systems (Table 1.2). However, in the Gaza Strip, it is primarily the refugee camps that require development. El Nuserrat, El Bureij, El Maghazi and El Zawida are all densely populated camps that do not have any sewage facilities (ARIJ, 2006).

Cesspits have been the traditional mean of disposing of sewage in the OPT. They vary in size, depending on the number of homes they serve, the availability of land and the cost of construction. Their capacity ranges between 5 and 50 m^3. They are deliberately constructed without a concrete liner, in order to encourage seepage into the ground. Hence, they have high potential to cause pollution of groundwater. Periodically cesspits become full and are emptied by vacuum tankers which are owned by municipalities or private businesses. However, in the absence of adequate treatment facilities, the vacuum tankers mostly release the sewage into nearby wadis or onto a piece of disused land, causing further pollution (ARIJ, 2006).

Table (1.2) Annual Volume of Collected Wastewater in the OPT (ARIJ, 2006)

Governorate	Population	Total Wastewater Generation (MCM/yr)	Volume of Wastewater (MCM/yr)		
			Collected by Sewage Network	Collected in Cesspits	Discharged into Channels
Nablus	326,873	2.299	1.236	1.057	0
Ramallah	280,805	0.374	0.104	0.252	0.104
Jericho	43,620	3.182	0.000	3.182	0
Jerusalem	149,150	5.208	0.954	4.248	0
Bethlehem	174,654	2.160	0.990	2.154	0
Jenin	254,218	0.902	0.126	0.168	0.044
Tubas	46,664	1.291	0.004	1.134	0.151
Tulkarm	167,873	5.611	2.508	3.102	0
Qalqiliya	94,210	3.355	1.542	1.806	0
Salfit	62,125	4.358	0.660	3.690	0
Hebron	524,510	7.267	2.322	4.722	0.216
WestBank	2,124,702	36.010	10.447	25.5156	0.425
Deir Al-Balah	201,112	2.760	2.538	0.221	0
Gaza	487,904	16.806	16.602	0.198	0
Khan Yunis	269,601	4.032	2.580	1.446	0
North Gaza	265,932	4.380	2.759	1.620	0
Rafah	165,240	1.980	1.500	0.474	0
Gaza Strip	1,38979	29.940	25.979	3.959	0

1.2.4 Wastewater Treatment

The centralized wastewater treatment plant, existing in Al-Bireh in Ramallah Governorate, which was constructed in 1998, with funding from the German Development Agency (KFW), is the only functioning wastewater treatment plant in the West Bank. Approximately 7% of the total wastewater generated in the West Bank is treated in that plant, meaning the remaining 93% is discharged untreated into the environment. Some of the untreated wastewater flow seas towards in wadis towards the Dead Sea (e.g., Wadi en-Nar, which carries wastewater from Bethlehem, Abu Dis and Jerusalem), and some flows west into Israel (e.g., Wadi Zimar, which carries wastewater from Tulkarm). In several instances, this wastewater is treated in Israeli treatment plants and reused for irrigation purposes (ARIJ, 2006).

In the Gaza Strip, there are 3 centralized wastewater treatment plants, located in Gaza City, north Gaza (Beit Lahia), and Rafah. However, these plants are functioning at moderate efficiency rates, ranging between 40-60 %, and do not have the capacity to treat the volume of wastewater generated by the ever expanding population. Both the partially treated and untreated wastewaters are discharged into open areas, such as Wadi Gaza or into the Sea and sand dunes.

Due to the fact that the West Bank comprises the recharge zone of the West Bank's aquifer system, direct discharge of untreated or partially treated wastewater into open areas endangers the groundwater quality. In the Gaza Strip, the pollution of the aquifer will not directly affect Israel, but it has the potential to irreparably damage the only significant source of drinking water for close to 1.5 million Palestinians living there. Nitrate pollution in the Costal aquifer (in the Gaza Strip) is very common. In addition, pollution has been recorded at different locations in the West Bank (ARIJ, 2006). In addition, springs are more vulnerable to pollution than the aquifers themselves. It is worthy to mention that, these springs often provide the only source of drinking water to Palestinian villages that are not connected to the water network.

1.3 Thesis Objectives

The main goal of this research is to asses the process performance of the UASB-Septic tank system after long period of operation. In addition to the UASB-Septic tank system the potential of the anaerobic hybrid (AH)-Septic tank for onsite sewage treatment will be elucidated. Two pilot scale UASB-Septic tanks, namely Reactor 1 and Reactor 2, were operated in parallel and fed with domestic wastewater from Al-Bireh City. The UASB-septic tank reactors, 1 and 2, operated respectively at 2 and 4 days at ambient temperature to elucidate the reactors performance after being operated for a long period.

The sub-goals of this research are:

- Monitoring and compare the performance of the two UASB-Septic tank reactor and an AH-Septic tank reactor treating domestic wastewater under Palestine conditions. The reactors performance was evaluated in terms of process efficiency (COD total and fractions, Volatile Fatty Acid (VFAs), ammonia, phosphate) and process stability through monitoring the quantity of biogas produced, and sludge stability.
- Assessment of the long term performance of the UASB-Septic tank system.

1.4 Thesis structure

This thesis consists of five chapters. Chapter1 is the research introduction in which background, aim of the research and objectives are introduced. Chapter 2 represents the literature review on anaerobic treatment of domestic wastewater process. Chapter 3 reviews the materials methods used in this research. The research results are presented and discussed in chapter 4 finally chapter 5 summarizes the conclusions and the recommendations of this research.

Chapter 2
Literature review

2.1 Introduction

The term 'sewage' refers to the wastewater produced by a community, which may originate from three different sources, domestic wastewater, generated from bathrooms & toilets, industrial wastewater from industries, and rainwater (Van Haandel & Lettinga, 1994).

In general wastewater is characterized in terms of its physical, chemical, and biological composition. However the most important constituents of these categories of characteristics are those of undesirable properties and usually are the ones liable for removal in a wastewater treatment plant.

Domestic wastewater can be divided into different streams according to their origin. Generally two streams are distinguished: concentrated – black water from toilets (faeces, urine and flushing water) and diluted – grey water from bath, wash and kitchen (Henze and Ledin, 2001). Average characteristics of domestic wastewater, black water and grey water are presented in Table 2.1.

Table 2.1 Average characteristics of domestic wastewater, black water and grey water from conventional flush toilets (Luostarinen *et al*., 2007).

Parameter	Domestic wastewater	Black water	Grey water
BOD	115-400	300-600	100-400
COD	210-740	900-1500	200-700
Total N	20-80	100-300	8-30
Total p	6-23	40-90	2-7

All parameters are in mg/l.

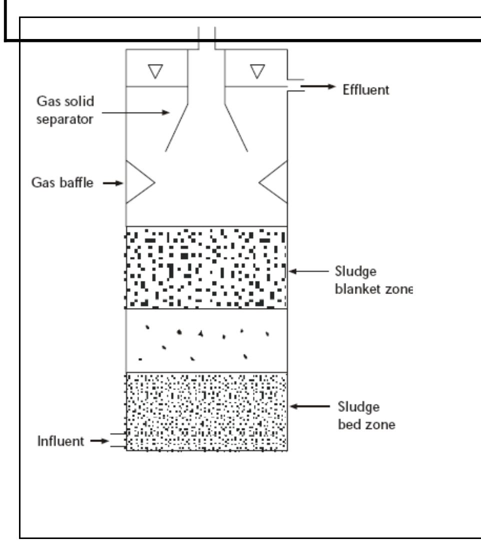
Human societies produce wastes that can represents a useful raw material for the production of energy and recovery of by-products and component water. Several techniques are already available to attain the goals of "Environmental Protection & Resource conservation" (Lettinga *et al*., 2000).

Domestic sewage treatment consists of an item that deserves ample due to the environmental impact caused by such wastewater if directly discharged into receiving waters. In addition, due to an increase in the scarcity of clean water (Aiyuk *et al*., 2006).

Yet several technological options are available today in the field of wastewater treatment, including conventional aerobic treatment in ponds, trickling filters and activated sludge plants (Metcalf & Eddy, 1991). Direct anaerobic treatment, (Leita˜o,2006), and resource-recovery wastewater treatments with biological systems, in which a combination of anaerobic and aerobic processes is applied (Jewell, 1996). The application of anaerobic technologies for sewage treatment dates back over 100 years. Table 2.2 shows the historical developments in anaerobic treatment technology (Khanna, 1989).

Table 2.2 Historical developments in anaerobic treatment technology (Khanna, 1989)

Investigator	Process description
M Louis Mouras (1881)	Mouras-Automatic Scavenger
WD Scot- Moncrieff(1880) England	The application of an anaerobic filter
Donald Cameron (1883) England	Septic tank
At Matunga(1897) Bombay	Waste disposal tanks at leper colony with gas collectors
Harry W Chark (1899) USA	Sludge was formed in a separate tank
William O Travis (1904)	Travis tank with hydrolyzing chamber
Karl Imhoff (1905)	Modification of Travis tank
Germany (1927)	The first sludge heating apparatus in a separate digestion tank was set up. The collected gas was delivered to municipal gas system.
Fair and Moore (1930)	Importance of seeding and Ph control
Morgan and Torpey (1950)	Mixing in digester and development of high rate digestion
Stander (1950)	Development of Clarigester and anaerobic baffled reactor based on biological contacter (RBC) concept
Young and McCarty (1969)	Anaerobic filter
Lettinga (1979)	UASB Figure2.1
Switzenbum and Jewell (1980)	Developed the further concept of anaerobic filters to fixed film reactors (Figure 2.2)



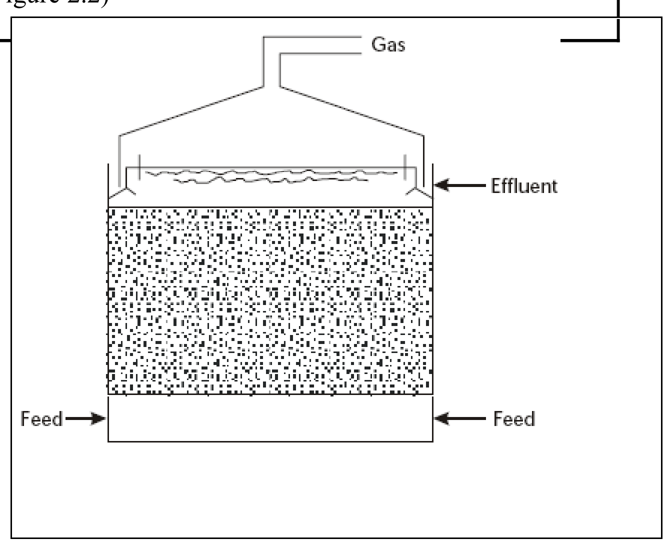


Figure 2.1 Schematic diagram of UASB reactor **Figure 2.2** Schematic diagram of fixed bed reactor

2.2 Treatment of wastewater

The objectives of sewage treatment include the removal of suspended solids and organic material (Van Haandel & Lettinga, 1994). Wastewater treatment systems are designed to digest much of the organic matter before the wastewater is released so that this will not occur. Treatment systems use physical, chemical, and biological processes. Complete wastewater treatment consists of a series of steps:

Preliminary treatment

The first treatment process consists of the removal of substances that may interfere with the downstream processes or be detrimental to the plant equipment. Materials removed may include rags, plastic, lumber, and grit.

Primary Treatment

The second step in the treatment process is primary treatment. The wastewater enters two primary clarifiers (sedimentation basins) which remove suspended and floating materials. The primary clarifiers remove about 61% of the Total Suspended Solids and about 35% of the Biochemical Oxygen Demand in the incoming wastewater.

Secondary Treatment

Secondary treatment usually consists of two steps which remove the dissolved and colloidal organic material not removed by the primary treatment.

The primary and secondary treatment processes generally remove at least 85% of the total suspended solids and biochemical oxygen demand. The Gillette Wastewater Treatment Facility averages removal of about 94% biochemical oxygen demand and 97% total suspended solids. The secondary process is a very sensitive biological process and can be adversely impacted by the discharge of incompatible or toxic wastes into the sewer system. Precise control of this process is necessary to effectively treat the wastewater.

Tertiary treatment

Is used only where it is needed to protect the receiving waters from excess nutrients. In tertiary treatment, the concentrations of phosphorus or nitrogen are reduced through biological or chemical processes.

2.3 Anaerobic wastewater Treatment process

Historical evidence indicates that the anaerobic degradation process is one of the oldest technologies. Classic anaerobic sewage treatment systems are related with the earlier digesters developed by Mouras in France (1872), Cameron & Travis in England (1896 & 1903) & Imhoff in Germany (1906). Anaerobic processes have gained popularity over the past decade, and have already been applied successfully for the treatment of a number of waste streams & geared mainly towards highly concentrated soluble wastewater (Foresti, 2001). Anaerobic digestion presents a high potential in most developing countries for domestic wastewater treatment & thus is a suitable and economical solution (Foresti, 2001).

2.4 Anaerobic degradation process in wastewater Treatment

During anaerobic treatment, a complex microbial community consisting of many interacting microbial species degrades natural polymers such as polysaccharides, proteins, nucleic acids, and lipids, in the absence of oxygen, into methane and carbon dioxide (McInerney, 1999).

$$\text{Organic materials + Nutrients} \xrightarrow[\text{microorganisms}]{\text{Anaerobic microorganism}} CH_4 + CO_2 + NH_3 + \text{Biomass}$$

Anaerobic degradation of organic matter is a balance between the activities of different groups of micro-organisms and occurs as a sequence of four steps: hydrolysis, acidogenesis, acetogenesis,and methanogenesis (Gujer & Zehnder 1983;) Figure 2.3.

During hydrolysis, hydrolytic micro-organisms produce extracellular enzymes which degrade complex organic compounds into their monomeric and dimeric compounds, i.e. proteins into amino acids, carbohydrates into simple sugars, and lipids into long chain fatty acids. Hydrolysis is often considered the rate-limiting step for anaerobic digestion, and it is affected by availability of hydrolytic enzymes, availability and structure of substrate (spherical, flat, or cylindrical; Sanders *et al*., 2000), pH, temperature, as well as short sludge retention time (SRT) and subsequent accumulation of acidic intermediates (Sanders *et al*., 2000).

Acidogenic bacteria then degrade these components further into volatile fatty acids (VFA), such as acetic, propionic, butyric, and valeric acids, and alcohols. During acetogenesis, these intermediary compounds are converted to acetic acid, hydrogen, and carbon dioxide, from which methanogenic bacteria produce methane and carbon dioxide as end products (Mata-Alvarez, 2002; Gerardi, 2003).

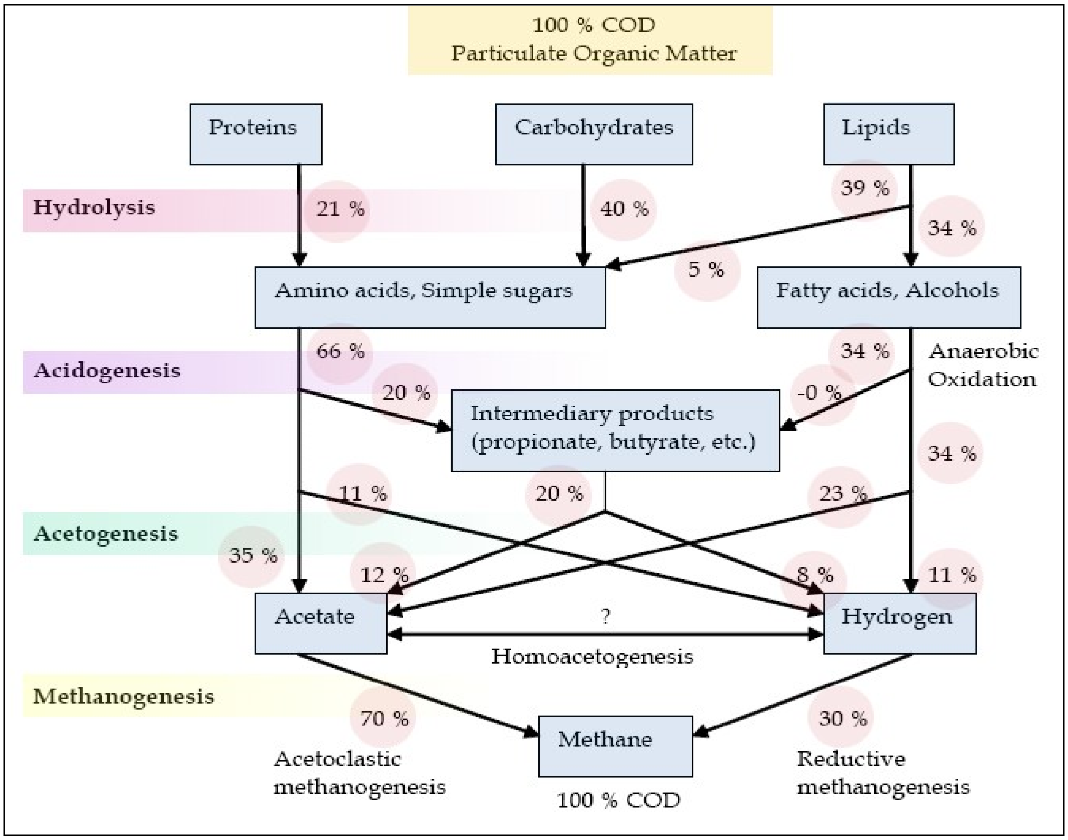


Figure 2.3 Schematic of the different metabolic steps and microbe groups involved in the complete degradation of organic matter to methane and carbon dioxide (Zehnder, 1982)

2.5 Factors affecting anaerobic degradation

There are several conditions and variables that must be applied in order to obtain a proper breakdown of the organic compounds. The operating parameters of the digester must be controlled so as to enhance the microbial activity and thus increase the anaerobic degradation efficiency. Some of these parameters are discussed below.

1. Temperature

The temperature dependence of the biological reaction-rate constants is very important in assessing the overall efficiency of a biological treatment process. Temperature not only influences the metabolic activities of the microbial population but also has a profound effect on such factors as gas-transfer rates and the settling characteristics of the biological solids. The efficiency of the anaerobic process is highly dependent on the reactor temperature (Bogte et al., 1993; van Haandel and Lettinga, 1994). The optimum range for mesophilic digestion is between 30 and 40 °C, and for temperature below the optimum range the digestion rate decreases by about 11% for each °C temperature decrease, according to the Arrhenius expression. Temperature affects not only the rate of the process, but also the final degradation extent. At low temperatures, more organic matter will remain under graded at a given hydraulic retention time (HRT) due to slow hydrolysis of volatile solids .however ,as long as the solids can be retained in the anaerobic reactor, they are removed from the liquid phase .

2. pH

The value and stability of the pH in an anaerobic reactor is extremely important because methanogenesis only proceeds at a high rate when the pH is maintained in the neutral range (6.3 to 7.8) (van Haandel and Lettinga, 1994). When treating a complex wastewater like domestic sewage, pH is usually in the optimum range without the need for chemical addition, due to the buffering capacity of the most important acid-base system in anaerobic digester: the carbonate system (van Haandel and Lettinga, 1994).

3. Particle decomposition

Water and wastewater often contain significant amount of colloidal and particulate matter in addition to soluble substances. Colloidal particles play an important role in the distribution of pollutants in natural aquatic systems because they may adsorb significant quantities of both inorganic and organic substances due to their large surface area relative to their mass.

4. Carbon to Nitrogen Ratio (C/N)

The relationship between the amount of carbon and nitrogen present in organic materials is represented by the C/N ratio. Optimum C/N ratios in anaerobic digesters are between 20–30. A high C/N ratio is an indication of rapid consumption of nitrogen by methanogens and results in lower gas production. On the other hand, a lower C/N ratio causes ammonia accumulation and pH values exceeding 8.5, which is toxic to methanogenic bacteria.

Optimum C/N ratios of the digester materials can be achieved by mixing materials of high and low C/N ratios, such as organic solid waste mixed with sewage or animal manure.

5. Retention (or residence) Time

The required retention time for completion of the anaerobic degradation reactions varies with differing technologies, process temperature, and waste composition. The retention time for wastes treated in mesophilic digester range from 10 to 40 days. Lower retention times are required in digesters operated in the thermophilc range. A high solids reactor operating in the thermophilic range has a retention time of 14 days (Lakos, 2001).

6. Organic Loading Rate (OLR)/ Volatile Solids (VS)

OLR is a measure of the biological conversion capacity of the anaerobic degradation system. Feeding the system above its sustainable OLR, results in low biogas yield due to accumulation of inhibiting substances in the digester slurry (i.e. fatty acids).Under such circumstances, the feeding rate of the system must be reduced. ORL is a particularly important control parameter in continuous systems. Many plants have reported system failure due to overloading .ORL is expressed in kg Chemical Oxygen Demand (COD) or Volatile Solids (VS) per cubic meter of reactor. It is linked with retention time for any particular feedstock and anaerobic reactor volume (Lakos, 2001).

7. Mixing

Mixing, within the digester, improves the contact between the micro-organisms and substrate and improves the bacterial population's ability to obtain nutrients. Mixing also prevents the formation of scum and the development of temperature gradients within the digester. However excessive mixing can disrupt the micro-organisms and therefore slow mixing is preferred (Lakos, 2001).

Syntheses of important ratios that influence the anaerobic digestion process are presented in Table 2.3.

Table 2.3 Focused ratios that influence anaerobic digestion (Aiyuk, 2006).

Ratio	***Threshold and/or significance of increase***	***References***
VSS:TSS	Indicates bacterial enrichment and increased biodegradability; constancy shows sludge bed stability; during start-up values as low as 0.4 can occur; ratios of VSS:SS ranging from 0.7 to 0.85 are likely to cause granulation	Wu (1985), Amatya (1996) and Mahmoud (2002)
COD_s :VSS	Indicates substrate enrichment in readily available COD; for UASB, deceases the need for excess sludge discharge; decreases reactor volume	Kalogo (2001), Aiyuk *et al*. (2004a)
COD_s:SS	• Expresses increased availability of readily biodegradable COD Aiyuk et al. (2004a) • Reduces HRT and increases reactor compactness • Translates in same manner as CODs:CODt	Aiyuk *et al*. (2004a)
CODp:VSS	Indicates high lipids content	Mahmoud (2002)
TSS:COD	Compromises reactor performance and hence granulation	De Smedt *et al*. (2001), Aiyuk *et al*. (2004a)
COD:N:P	Should be at least 300:5:1 for efficient rapid start-up	Amatya (1996), Aiyuk *et al*. (2004a)
C:N:P	Minimum set at 400:5:1, or 100:28:6	Alphenaar *et al*. (1993), Thaveesri (1995)
COD:N	Min 70	Brunetti *et al*. (1983)
COD:P	Min 350	Brunetti *et al*. (1983)
COD:SO4	Min 10:1; if less H2S inhibition arises; low ratio also leads to obnoxious odor, corrosion, deteriorated biogas quantity and quality, and decreased COD removal	Lettinga (1981), Souza (1986), Hulshoff Pol *et al*. (1998)
VFA: alkalinity	Indicates reactor instability, and should be preferably much less than unity	Amatya (1996), Switzembaum *et al*. (1990)
SAA:SM	Indicates enrichment in acidogens in relation to methanogens, usually brought about by RACOD; enhances hydrolysis	Kalogo (2001)
Propionate :acetate	>1.4 signifies reactor imbalance	Hill *et al*. (1987)

CODp = particulate COD, SAA = specific acidogenic activity, SMA = specific methanogenic activity, RACOD = rapidly acidifiable COD

2.6 Differences between aerobic and anaerobic wastewater treatment processes

The anaerobic process can serve as a viable alternative compared to conventional aerobic process (Lettinga, 1995; Schink, 2001) for a variety of reasons.

Advantages and benefits of anaerobic treatment (Aiyuk, 2006)

-Energy

- Instead of consuming energy, it is a net energy producing process
- Consequently the process does not use electricity or other mineral fuels
- Generates high quality renewable fuel in form of biogas
- Biogas is employed in numerous end-use applications

-Environmental

- Significantly lower sludge production (DS/kg COD removed and m^3/kg) →1/10 aerobic
- Excess sludge generally well stabilized. Also produces a sanitized compost and nutrient-rich liquid fertilizer
- Viable sludge can be preserved unfed for long periods of time (more than 1 year) without activity, settleability, etc., affected significantly
- Reduces run-off
- Can substantially decrease incidence of pathogens (mainly thermophilic)
- Can substantially decrease use of fossil fuels
- Maximizes recycling benefits
- Method easily leads to application of integrated EP (e.g., combined with post- treatment by which useful products like ammonium can be recovered, while in specific cases effluents and excess sludge could be employed for irrigation and fertilization or soil conditioning.

-Economic and others

- Obtainable at very low costs. In fact, anaerobic digestion (AD) is more cost-effective than other treatment options from a life-cycle perspective.
- Generally has much smaller footprint

• It is the core method for integrated environmental protection, because when combined with proper post-treatment products like ammonia and sulphur can be recovered

• Has low nutrient requirement

• Can be applied practically at any place and at any scale. Suitable for on-site application in residential areas and industry, with good potentials for closing water cycles

Limitations of anaerobic processes

- Low pathogen and nutrient removal. Pathogens are only partially removed, except helminthes eggs, which are effectively captured in the sludge bed. The removal of nutrients is not complete and a post treatment is sometimes required.
- Long start –up. Due to the low growth rate of methanogenic organisms, the start-up takes longer as compared to aerobic process, when no good inoculum is available.
- Possible bad odor. Hydrogen sulfide is produced during the anaerobic process, especially when there are high concentrations of sulfate inn the influent. A proper handling of the biogas is required to avoid bad smell. A significant proportion of the total amount of methane produced by the reactor maybe dissolved in the effluent. its recovery may be required to minimize smell nuisances and methan emissions to the atmosphere.
- Necessary of post –treatment. Post treatment of the anaerobic effluent is generally required to reach the discharge standards for organic matter, nutrients and pathogens.

Aerobic versus anaerobic treatment

Most conventional wastewater treatment processes are 'aerobic' — the bacteria used to break down the waste products take in oxygen to perform their function. This results in the high energy requirement (oxygen has to be supplied) and a large volume of waste bacteria ('sludge') is produced. This makes the processes complicated to control, and costly. The bacteria in 'anaerobic' processes do not use oxygen. Excluding oxygen is easy, and the energy requirements and sludge production is much less than for aerobic processes — making the processes cheaper and simpler. Also, the temperature in which the bacteria like to work is easy to maintain in hot climates. However, the main disadvantages of anaerobic processes are that they are much slower than aerobic processes and are only good at removing the organic waste (the ‘simple’ waste, the sugary material) and not any other sort of pollution — such as nutrients, or pathogens Figure 2.4.

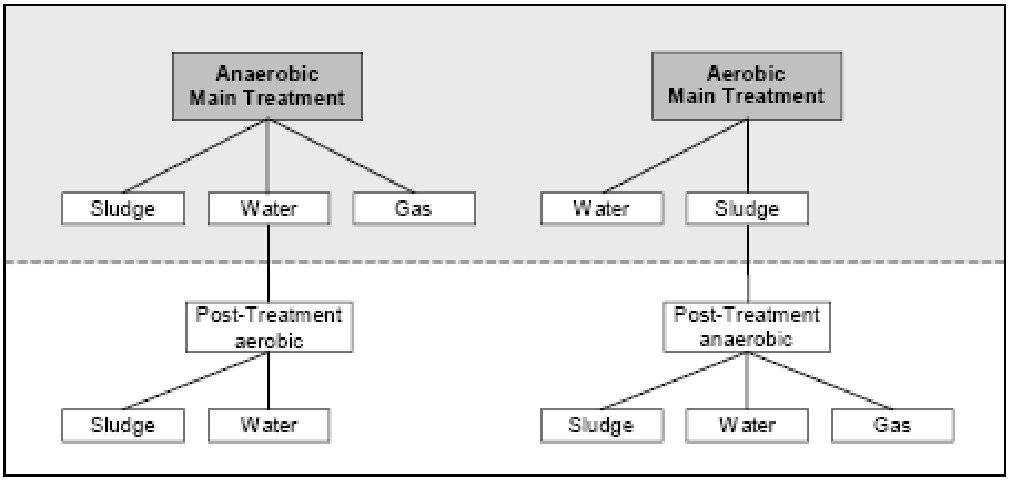


Figure 2.4. Principle difference between anaerobic and aerobic intensive wastewater treatment. (Field, 2007).

In the wastewater engineering field organic pollution is measured by the weight of oxygen it takes to oxidize it chemically. This weight of oxygen is referred to as the "chemical oxygen demand" (COD). COD is basically a measure of organic matter content or concentration. The best way to appreciate anaerobic wastewater treatment is to compare its COD balance with that of aerobic wastewater treatment, as shown in Figure 2.5 below.

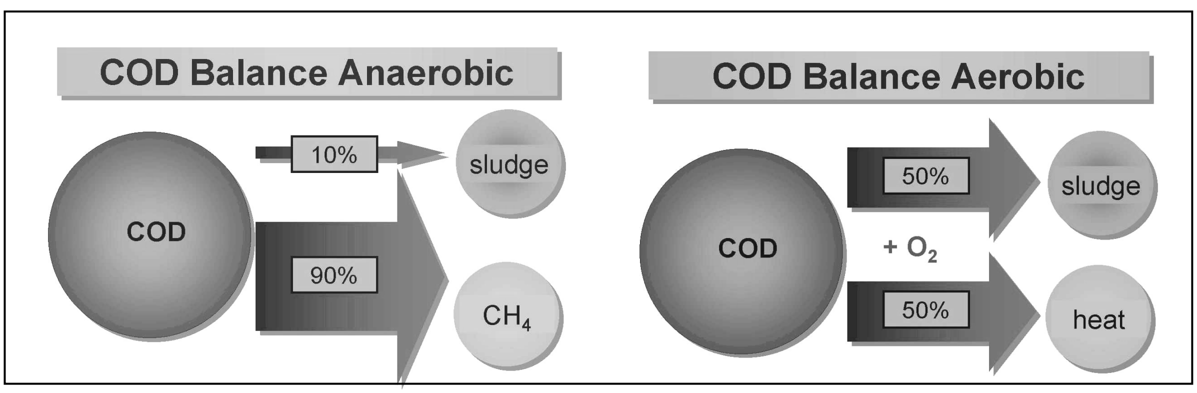


Figure 2.5. Comparison of the COD balance during anaerobic and aerobic treatment of wastewater containing organic pollution (Field, 2007).

Anaerobic Treatment: The COD in wastewater is highly converted to methane, which is a valuable fuel. Very little COD is converted to sludge. No major inputs are required to operate the system.

Aerobic Treatment: The COD in wastewater is highly converted sludge, a bulky waste product, which costs lots of money to get rid of. An aerobic wastewater treatment facility is in essence a "waste sludge factory". Elemental oxygen has to be continuously supplied by aerating the wastewater at a great expense in kilowatt hours to operate the aerators.

2.7 Upflow anaerobic sludge blanket

This type of reactors was studied for the first time in the Netherlands with low ambient temperature and in Indonesia with high ambient temperature .Also this type of reactor was studied by Bogte *et al*. (1993) at Netherlands in different rural locations with varying results by using a reactor of $1.3m^3$ volumes. The same reactor with $0.86m^3$ also tested in Bandung (Indonesia) by Lettinga *et al*. (1993), See Tables 2.4, 2.5, and 2.6.

Since the earlier anaerobic treatment systems, the design concepts were improved from classic reactors like septic tanks and anaerobic ponds, to modern high rate reactor configurations like anaerobic filters, UASB,(Expanded Granual Sludge Bed) EGSB fixed film fluidized bed and expanded bed reactors, and others (van Haandel, 2006).

Bogte *et al*. (1993) and Lettinga *et al*. (1993) researched the use of a UASB-septic tank for on-site treatment of black water and domestic sewage. The UASB-septic tank differs from the conventional septic tank system by the up flow mode in which the system is operated resulting in both improved physical removal of suspended solids and improved biological conversion of dissolved components. The most important difference with the traditional UASB system is that the UASB-septic tank system is also designed for the accumulation and stabilization of sludge. For low temperature conditions, Zeeman and Lettinga (1999) proposed a two-step UASB-septic tank for on-site treatment of domestic sewage (Elmitwalli *et al*., 2003).

However, due to the short liquid retention time in the reactor, the removal of pathogens is only partial. The effluent from the UASB reactor usually needs further treatment in order to remove remnant organic matter, nutrient and pathogens. This post treatment can be accomplished in conventional aerobic system like waste stabilization ponds (WSP).

2.8 UASB technology

The two main conditions for any well performing biological wastewater treatment system are: (1) to ensure good contact between the incoming substrate and the sludge mass in the system and (2) to maintain a large sludge mass in the system. In the UASB reactor the influent is distributed uniformly over the bottom of the reactor and then, following an up flow pathway, rises through a thick layer of anaerobic sludge, where after it is withdrawn at the top of the reactor. Thus the contact between the influent organic material and the sludge mass, in the reactor, is automatically guaranteed. In order to maintain a large sludge mass, the UASB reactor has a built-in phase separator, where the dispersed solids are retained by settling, so that an effluent virtually free from settleable solids can be discharged. The retained sludge particles will end up sliding back from the settler compartment into the digester compartment and accumulate there, thus contributing to the maintenance of a large sludge mass in the reactor.

2.9 Conventional UASB reactor

The UASB reactor is by far the most widely used high rate anaerobic system for anaerobic sewage treatment. Several full-scale plants have been put in operation and many more are presently under construction, especially under tropical or subtropical conditions. Some studies have also been carried out in regions with a moderate climate. Figure 2.6 is a schematic representation of the conventional UASB reactor.

The most characteristic device of the UASB reactor is the phase separator, placed in the upper section and dividing the reactor in a lower part, the digestion zone, and an upper part, the settling zone. The sewage is introduced as uniformly as possible over the reactor bottom, passes through the sludge bed and enters into the settling zone via the apertures between the phases separator elements and is uniformly discharged at the surface. The biogas produced in the digestion section is captured by the separator so that unhindered settling can take place in the upper zone. To avoid blocking of the biogas outlet and allow separation of biogas bubble from sludge particles, a gas chamber is introduced under the separator element. The settled sludge particles on the separator elements eventually slide back into the digestion zone.

Thus, the settler enables the system to maintain a large sludge mass in the reactor, while an effluent essentially free from the suspended solids is discharged (van Haandel, 2006).

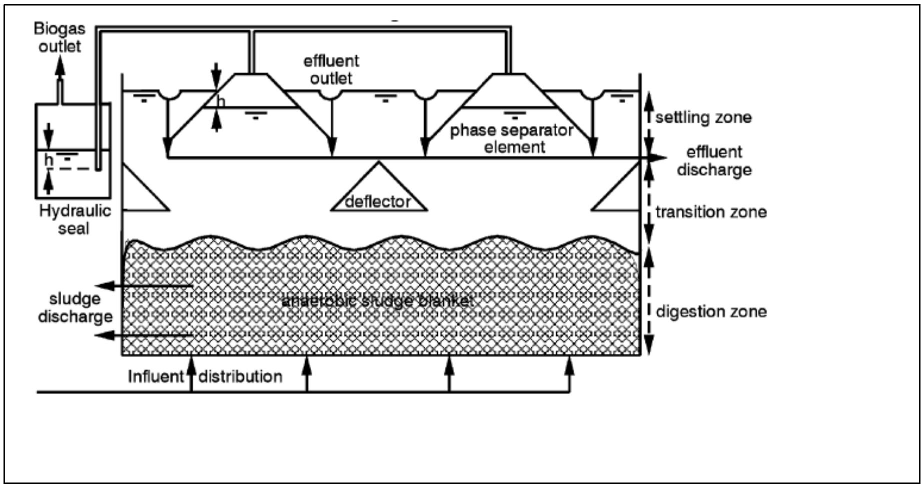


Figure 2.6. Schematic representation of a conventional UASB reactor with external hydraulic seal to maintain the required water level in the biogas chambers.

2.10 Anaerobic filter (AF)

Anaerobic filters were first described in 1968 and have been used as an advanced technology for effective treatment of a variety of industrial wastes. An important concern can be the high price of many carrier materials that may result in costs of the same order as that of the construction costs of the reactor itself.

Although septic tank systems are used predominantly for individual households, they have also been used for urban or rural small communities (200 to 5000 inhabitants) and housing projects in urban areas where there is lack of service by a central sewerage network and treatment plant. In most cases, the carrier material consists of 5-cm construction stones. Recent studies showed the feasibility of using alternatives materials like bamboo rings, river stones, bricks, and pieces of plastic electro ducts. They are relatively easily available in the market, of lower price, lighter and good specific superficial area for bacterial adherence (Andrade Neto 2004). While industrial carrier materials like Pall rings or other modular media tend to improve the performance of AF (Young, 1990). Their price is still very high (van Haandel, 2006).

2.11 Anaerobic hybrid septic tank (AH)

Anaerobic hybrid septic tank reactor, which consists of a sludge bed in the lower part and an anaerobic filter combines advantages of a UASB and AF reactors while minimizing their limitations (Elmitwalli, 2003). Oriented and porous media in the AF reactor provide better performance than random and non-porous media respectively (Young, 1991). Huysman *et al*. (1983) reported that RPF media, porous media, offers an excellent colonisation matrix for the AF reactor. Elimitwalli *et al*. (2000) showed that clean vertical sheets of RPF were efficient in removing suspended COD (>75%) in domestic sewage even at a short HRT as low as 0.5 h and at a high upflow velocity as high as 10 m/h (Elmitwalli *et al*., 2003). The filter zone in the AH reactor, in addition to its physical role for biomass retention, has some biological activity contributing to COD reduction in a zone which is lacking biomass in a classical UASB reactor. Oriented and porous media in the AF reactor provide better performance in comparison with random and non-porous media, respectively (Elmitwalli, 2002). The performance of hybrid upflow anaerobic filters depends on contact of the wastewater with both the suspended growth in the sludge layer and the attached biofilm in the media matrix.

Table 2.4 summary of applications of on-site pilot scale UASB-septic reactors to sewage treatment under different conditions.

Place	V (m^3)	T (°C)	Influent Type	Influent concentration (mg/l)			HRT(h)	Removal efficiency (%)			Gas production (l/d)	Period (Months)	References
				COD_{tot}	BOD	TSS		COD_{tot}	BOD	TSS			
Netherlands	1.2	13.8	GW+BW	976	454	641	44.3	33	50	47	66.5	28	Bogte *et al*. (1993)
Netherlands	1.2	12.9	GW+BW	821	467	468	57.2	3.8	14.5	5.8	16.1	24	Bogte *et al*. (1993)
Netherlands	1.2	11.7	BW	1716	640	1201	102.5	60	50	77.1	16.7	13	Bogte *et al*. (1993)
Indonesia	0.86	>20	BW	5988	2381	2678	360	90-93	92-95	93-97	118	40	Lettinga *et al*. (1991)
Indonesia	0.86	>20	GW+BW	1359	542	568	34	67-77	78-82	74-81	168	30	Lettinga *et al*. (1991)
Netherlands	1.2	14-19	BW	2751	------	2482	160	69	------	71	52	3	Luostarinen *et al*. (2003)

V= volume; T=Temperature; W=Gray wastewater; W=Black wastewater.

Table 2.5 Summary of results for anaerobic domestic wastewater treatment in pilot and full scale UASB-septic tank reactors at low temperature and tropical country climate.

Place	Influent type	T (°C)	V(m^3)	Influent concentration (mg/l)				HRT (h)	Removal efficiency (%)				Period months	Reference
				COD_{tot}	COD_{dis}	COD_{ss}	TSS		COD_{tot}	COD_{dis}	COD_{ss}	TSS		
Netherlands	R	13	0.004	456	112	82	NP	8	67	30	90	NP	2	Elmitwalli(2000)
Netherlands	S	13	0.004	339	124	229	NP	8	60	49	79	NP	3	Elmitwalli(2000)
Netherlands	R	15	0.140	721	172	398	NP	6	44	5	73	NP	3	Mahmoud(2002)
Jordan	R	24	1.2	1412	-----	830	451	23	58	-----	65	62	12	Halalsheh(2002)
Jordan	R	18-25	60	1531	277	1122	396	8-10	50	-7	53	41	12	*Halalsheh,(2002)
Jordan	R	18-25	60	1531	277	1122	396	23-27	51	23	60	55	12	**Halalsheh(2002)
Palestine	PBW	16-35	0.35	1013	-----	-----	715	11.6	76	-----	-----	58	1.4	Al-juaidy(2001)
Palestine	PDW	16-35	0.35	566	-----	-----	560	14	79	-----	-----	46	1.4	Ali,(2001)
Palestine	R	15-34	0.8	1189	361	643	614	48	54	12	85	79	6	Al-Shayah(2005)
Palestine	R	15-34	0.8	1189	361	643	614	96	58	14	89	80	6	Al-Shayah(2005)
Palestine	R		0.8	905	350	396	371	48	51	24	83	74	6	Al-Jamal(2005)

V=Volume; T=Temperature; S=Settled wastewater; R=Raw wastewater; GS = Granular Sludge; FS = Flocculent Sludge; PDS = Partially Digested Sludge; PDW = Pre-settled Domestic Wastewater; PBW = Pre- settled Black Wastewater; *First stage of a two-stage of UASB, ** one stage UASB reactor.

Table 2.6. Comparisons between present and earlier studies with UASB-septic tanks or anaerobic hybrid (AH) tanks (Luostarinen, 2007).

Reactor	wastewater	Temp (°C)	HRT(d)	OLR ($kgCOD/m^3d$)	Removal COD_t	COD_{ss}	COD_{col}	CODdis	Ref.
UASB-septic tank[a]	Black water	10	4.4(4.2)+1.4(0.36)	0.301(0.155)	94(3.3)	98(2.6)	50(32)	71 (19)	Luostarinen *et al*. (2007)
UASB-septic tank[a]	Dairy parlour wastewater	10	3.5+(0.8)+1.5(0.43)	0.191(0.074)	82(6.3)	86(15)	62(24)	70 (20)	Luostarinen *et al*. (2007)
UASB-septic tank	Black water	>20	15	0.37	90-93	ND	ND	ND	Lettinga *et al* (1993)
UASB-septic tank	Domestic wastewater	>20	1.4	0.96	67-77	ND	ND	ND	Lettinga *et al*. (1993)
UASB-septic tank	Black water	11.7 (4.0)	4.3	0.40	60	77	ND	6b	Bogte *et al*.(1993)
UASB-septic tank	Grey water	13.8(3.7)	1.8	0.53	31	9	ND	47b	Bogte *et al*.(1993)
UASB-septic tank	Grey water	12.9(4.9)	2.4	0.34	4	6	ND	_1b	Bogte *et al*.(1993)
UASB-septic tank	Black water	14-18	7.2	0.741-0.968	71	75	ND	44b	Luostarinen (2007) *et al*.
AH-septic tank[a]	Concentrated sewage	13	2.5	1.44	94(1.7)	98(2.3)	74(10.3)	78 (1.7)	Elmitwalli *et al*.(2003)

SD in parenthesis where available, ND = not detected, [a]Two phase, [b]CODcol+di

Chapter 3

Materials and methods

3.1 Experimental set-up

Two pilot scales UASB –septic tank reactors, namely R1 and R2, were operated in parallel at the main wastewater treatment plant (WWTP) of Al-Bireh city, Palestine. The reactors will be made of galvanized steel with working volumes at 0.8 m^3 (height 2.5m; diameter 0.638m) sampling ports were installed along the reactor at 0.25m for sludge sampling. The influent was distributed in the reactor through polyvinyl chloride (PVC) tube with four outlets located 5 cm from the bottom. Biogas will be led through a 16% NaOH solution for scrubbing the CO_2, and then methane quantity was continuously measured by wet gas meters. Schematic diagram of the experimental set-up is presented in Figure 3.1.

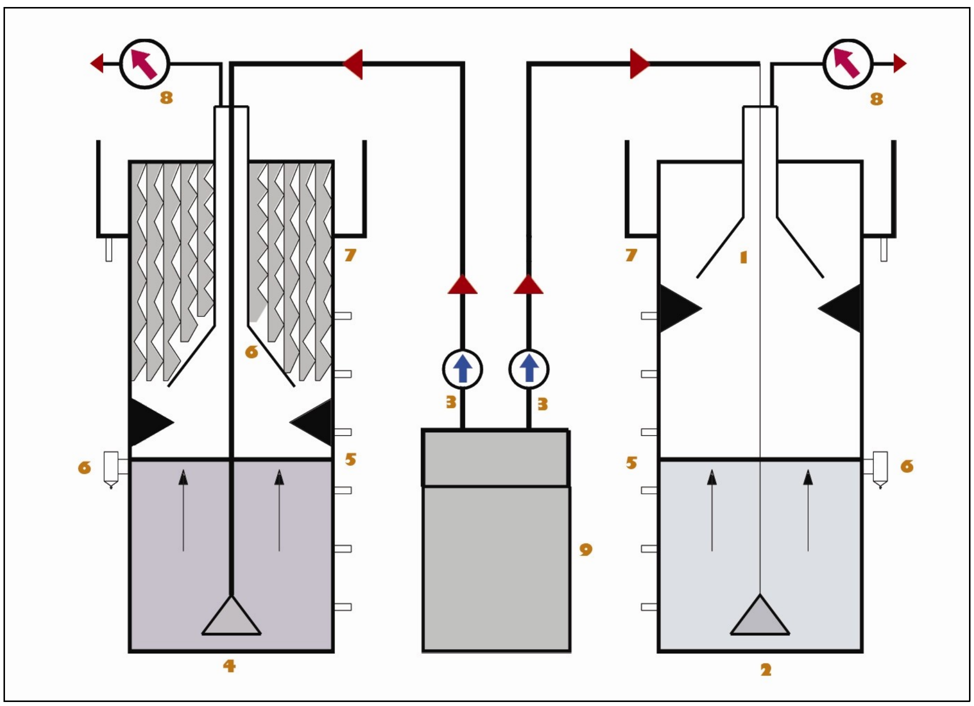


Figure 3.1 Schematic diagram of the experimental set-up (not to scale)

1, influent wastewater; 2, influent tank; 3, peristaltic pump; 4, AH-septic tank; 5,sludge bed; 6, gas-solids-liquid separator; 7, media of the AH-septic tank (vertical sheets of RPF, type TM10); 8, gas meter.

3.2 Pilot plants start-up, operation and monitoring

The UASB–septic tank and AH reactors have been in operation in parallel at ambient temperature conditions with temperature between 15°C and 34°C, since April 2004 treating domestic sewage from the main sewage trunk at Al-Bireh WWTP. The sewage will be pumped every five minutes to a holding tank (200L plastic container) with a resident time of about 5 minutes, and from there the reactors will be fed and the influent will be sampled. Sewage temperature will be measured in situ for each grab sample. The biogas production and ambient temperature will be monitored on daily basis.

Al-Shayah started up the two UASB reactors in April 2004 for six months and Al-Jamal continued to operate them under the same conditions for the next six months. Characteristics and operational conditions of the two UASB reactors are presented in Table 3.1.

Table 3.1 Characteristics and operational conditions of the UASB during the whole one year

Reactor	Total Volume	Total Height	Diameter	HRT	Inflow	Up flow velocity (V_{up})
R1	800L	2.5m	0.638m	2 days	0.4 m^3/d	0.052 m/hr
R2	800L	2.5m	0.638m	4 days	0.2 m^3/d	0.026 m/hr

After a round a year of continuous operation, the sludge of the both reactors was drawn out, mixed together. Then equally re-added to both reactors, 50 L for each, the sludge was characterized for its VS, TS, one reactor was modified to an anaerobic hybrid –septic tank by adding vertical sheets of (RPF) Figure 3.2. The media used in the reactor was vertical sheets of RPF (Reticulated Polyurethane foam). The characteristics of the RPF sheets used are presented in Table 3.2. The media were oriented vertically, back to back with a 3mm steel sheet without spacing the dimensions of sheets presented in Table 3.3,and the other reactor was operated in parallel as a UASB-septic tank as a control. The reactors were operated at 2days HRT.

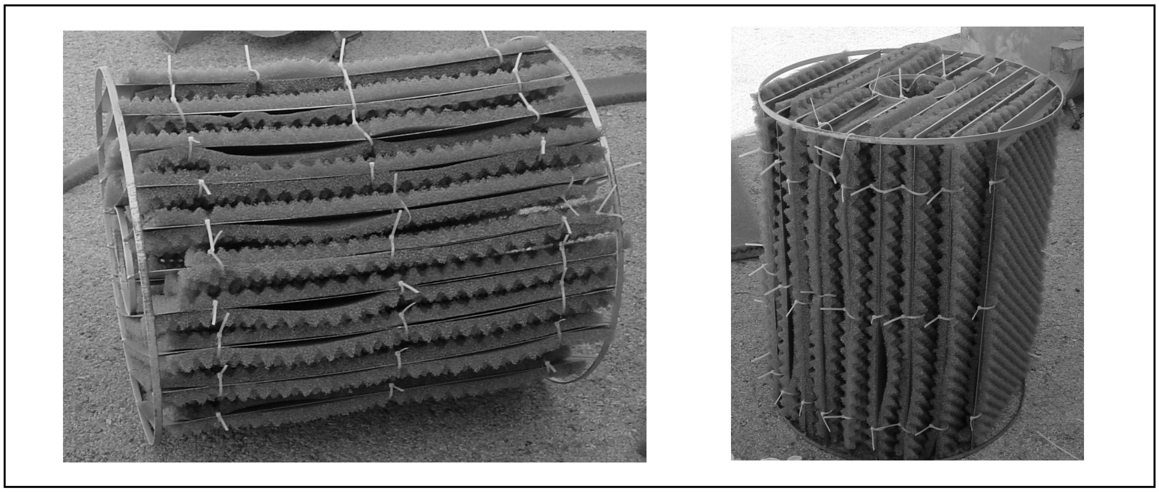


Figure 3.2 Reticulated Polyurethane foam (RPF)

Table 3.2 Characteristics of the RPF sheets used in the experiment

Parameter	***Unit***	***Value***
Total sheet thickness	mm	25
Specific surface area	m^2/mm^3	500
Pore size	mm	2.5

Table 3.3 Dimensions of RPF sheets

Width	***Length***	***Unit***	***# of sheets***
32	72	cm	2
46	72	cm	2
53	72	cm	2
58	72	cm	2
22	72	cm	4

Figure 3.1 shows a schematic diagram of the experimental set-up, consisting of Anaerobic Hybrid-septic tank reactor and a UASB- septic tank each (0.8 m^3). The height of each reactor was 2.5 m. The media used in the (AH) were vertical sheets of RPF with Knobs at one side. Both reactors were inoculated with equal volume of sludge, the UASB-septic tank reactor was operated for 6 months (HRT=2 days), the first 45 days to assess the long term performance of the UASB-Septic tank system and to compare it with the previous removal efficiency done by Al-Shayah (2004) and Al-Jamal (2005) for about one year. AH-septic tank was operated for 4 months started after two months of the UASB-septic tank was operated.

This research consists of two parts, first one was continue sewage analysis the same as Al-Shayah and Al-Jamal under the same conditions for the UASB-septic tank with HRT equals 2 days. The second part was modifying one of the reactors to hybrid-UASB-septic tank by adding vertical sheets of RPF and compare its performance with the UASB-septic tank with HRT equals 2 day Table 3.4 summarized the characteristics of the UASB and the anaerobic hybrid reactors.

Table 3.4 Characteristics and operational conditions of the UASB-septic tank and the anaerobic hybrid –septic tank reactors

Reactor	Total Volume	Total Height	Diameter	HRT days	Inflow m^3/d	Up flow velocity (V_{up})	Special characteristics
UASB-septic tank	800L	2.5m	0.638m	2 days	0.4	0.052 m/hr	Without RPF sheets
Anaerobic hybrid septic tank	800L	2.5m	0.638m	2 days	0.4	0.052 m/hr	With RPF sheets

3.3 Sampling

Daily monitoring was started since the onset of the experiment including wastewater and ambient temperature and biogas production measurements. Grab samples of raw sewage and reactors an effluent was collected two or three times a week, (1 L for each). Preliminary treatment of the raw sewage was provided by screens and grit removal chamber. Samples were kept at 4°C until they was analyzed. For the first part of the research grab samples were tested for BOD_5, COD_{tot} and fractions (COD_{col}, COD_{dis}, COD_{sus} , Volatile Fatty Acids (VFA), Ammonia , Phosphate, TS,VS and process stability through monitoring the quantity of biogas produced and sludge samples were analyzed for TS, VS and stability. For the second part of the research grab samples were tested for BOD_5, $COD_{tot,}$ ammonia, TS, VS for sludge samples and process stability. All measurements were determined in duplicate except VFA in single.

3.4 Analytical Methods

The analytical methods for wastewater parameters could be distributed in three fields' chemical analysis, physical analysis and microbiological analysis. In this research only the chemical and physical analysis were analyzed

3.4.1Chemical analysis

3.4.1.1Biological Oxygen Demand (BOD)

BOD test aims to determine the concentration of the organic matter in the wastewater by measuring the oxygen consumed by the microorganisms in biodegrading organic compounds of wastewater.

BOD was determined by obtaining a specific volume of sample (3ml,5ml) this volume will be added to the 300ml BOD bottle , which is about half full of aerated water then, initial dissolved oxygen was measured. After five days of incubation at 20°C temperature final dissolved oxygen was measured according to (standard methods APHA, 1995).

3.4.1.2 Chemical Oxygen Demand

COD test was carried out using reflux method, acid destruction at 150°C for two hours. Absorbance was them measured by spectrophotometer at 600nm wave length. COD_{tot}, COD_{filt} (4.4µm filters paper) and COD_{dis} (0.45µm membrane) were measured for the wastewater samples. COD_{sus} can be calculated from COD_{tot} and COD_{filt}, (COD_{tot} - COD_{filt}). COD for sludge samples is measured after dilution with distilled water.

3.4.1.3 Ammonia (NH_4-N)

Nesslerization method is used to measure (NH_4-N) using filtered samples, and then absorbance was measured at 425nm wave length, (APHA, 1995).

3.4.1.4 Ortho-Phosphate (PO_4^{3-})

Filtered samples from membrane filter were used to determine the amount of ortho-phosphate according to standard methods (APHA, 1995); absorbance is measured at 880nm wavelength.

3.4.1.5 Volatile Fatty Acid (VFA)

The volatile fatty analysis was carried out using titrimetric method according to. This method does not requires high investment in technical equipment like Gas Chromatograph (GC).where the analysis as it had reported by listed as following .

1. 20 ml filtered sample which filtered through a 0.45 μm membrane filter used.
2. The sample is titrated slowly with 0.1N sulfuric acid until pH 5.0 is reached, the initial pH of the sample and the volume of the acid consumed are recorded.
3. More sulfuric acid with 0.02N is added until pH 4.3 is reached; the volume of the acid consumed is again recorded. Another amount of 0.02N sulfuric acid added until pH 4.0 is reached, the volume of the consumed acid recorded.

Low manual mixing needed to minimize exchanging of CO_2 with the atmospheric during titration. Finally, VFA (as acetate acid) can be calculated from the following empirical equations (Eq.3.1 and Eq.3.2).

$$VFA = (131340 \times N_2) \times \frac{(VA_{(pH(4-5))}}{VS} - (3.08 \times Alk_{meas}) - 25 \text{ .... } (3.1)$$

$$Alk_{meas} = \frac{(VA_{(pH(Initial-5))} \times N_1 \times 1000)}{VS} + \frac{(VA_{(pH(5-4.3))} \times N_2 \times 1000)}{VS} \text{ ...... } (3.2)$$

Where:

VFA: Volatile fatty acid (mg/l), considered to be acetic acid (1mg/l VFA (acetic acid) =1.07 mg/l VFA_{COD}.

VA $_{(pH(4-5))}$: measured volume of acid (ml) required to titrate a sample from PH 5.0 to PH 4.0.

$VA_{(pH\ (initial-5))}$: measured volume of acid (ml) required titrate a sample from initial pH to pH 5.0.
$VA_{(pH\ (5-4.3))}$: measured volume of acid (ml) required to titrate a sample from pH 5.0 to pH 4.3.
VS: volume sample (ml).
Alk_{meas}: measured alkalinity (mmol/l).
N_1: Sulfuric acid normality 0.1 N.
N_2: Sulfuric acid normality 0.02N.

3.4.2 Physical analysis

The Physical parameters that had been analyzed in this research could be summarized as following.Total and suspended Solids (TS.), Volatile and Suspended Solids (VS), Temperature.

3.4.2.1 Total Solids (TS)
Total and solids were measured related to Standard Methods (APHA,1995) by oven drying at 105°C this by using paper of glass microfiber filters (GF/C 125 mmΦ, CATNO 1822 Whatman®.

3.4.2.2 Volatile Solids (VS)
Volatile solids were measured related to standard methods (APHA, 1995) oven burning at 550°C.

3.4.2.3 Temperature
The ambient and wastewater temperature were measured in situ by alcohol thermometer.

3.5 Batch experiments

In this research one type of batch experiment had been taken place which is the stability test which represents the maximum percentage of COD converted to CH_4 of the digested sludge. The tests are carried out in batch reactors, sealed serum bottles, of 500 ml with a headspace volume of 70ml incubated at 30°C for a period of 120 days. The collected methane gas in the headspace was regularly measured using a Mariotte displacement set-up filled with a 5% NaOH solution as described by (Lettinga *et al*., 1991).

3.5.1 Stability

All wastewater treatment plant processes quantities of wastewater material in the form of diluted solids mixtures known as sludge. The stability of the sludge is a function of the characteristics of the raw wastewater flow and the treatment process that generated the sludge stabilization where the objectives of the sludge stabilization are to reduce pathogens and liquid volume, eliminate offensive odors and reduce or eliminate potential for putrefaction .a sludge stability standard, expresses in g COD-CH_4 /gVSS or gCOD-CH_4/g COD.

Sludge stability was measured two times induplicate during the period of experiment where samples incubated at 30°C for a period of 120 days. The experimental set-up and procedure for determine sludge stability was according to Mahmoud (2002). Each bottle in the test was filled with about 1.5g COD -sludge /l, tap water and a mineral solution of macronutrients, trace elements and bicarbonate buffer. The stability batches incubate at 30°C. The total sludge stability was calculated as the amount of methane procedure during the test (as COD) divided by the initial COD of the sample. The experimental procedures for determination of stability and the composition of macronutrients and trace elements used in experiment are presented in details in Appendix 2.

3.6 Calculations

3.6.1 Removal efficiency

The removal efficiency of the different parameters will be calculated regarding to equation (3.3)

Removal efficiency % $=[(Influent - Effluent) \times 100\%] / Influent$ ....(3.3)

Where:

Influent: concentration of component in influent (mg/l).

Effluent: concentration of component in effluent (mg/l).

3.6.2 COD-mass balance

$COD_{inf} = COD_{accumulated} + COD_{CH4} + COD_{effluent}$…………………..……… (3.7)

Where:

COD_{inf} : amount of total COD in the influent (mg/l);

$COD_{accumulated}$: amount of accumulated COD in the reactor (mh/l);

COD_{CH4}: amount of produced CH4 (dissolved form + gas form) (mg CH4 as COD/L);

$COD_{effluent}$: amount of total COD in the effluent (mg/l);

3.6.3 Stability calculations

The sludge stability percentage could be calculated after 120 days regarding to the following equations:

Stability (%) $=100(CH4_{(asCOD)}/COD_{tot,t=0days})$…………………….…..(3.8)

COD_{tot} is the amount of initial total COD in tested sample (mg COD/l), CH_4 is the total amount of methane produced at the end of the test (mg CH_4 as COD/l) where the amount of produced CH_4 from the batch bottles could be converted to the equivalent COD using the following equation (3.9) (3.10) (Metcalf and Eddy, 2003).

$V = \frac{nRT}{P}$ ……………….……………………………………………. (3.9)

COD_{CH4}: $n \times 64 \times 1000\ (mg.CH_4 asCOD\ /\ l)$ ……………………….. (3.10)

Where:

V=volume occupied by the gas (L);

n= moles of CH_4 (mole), (1moleCH4=64gCOD);

R=ideal gas law constant, 0.082057atm.L/mole.K;

T=Temperature (K), (273.15+°C);

3.7 Statistical analysis of data

The variation range and the arithmetic averages and standard deviations of different data had been calculated using Microsoft Excel 2003.The SPSS software releases 11.0.0 SPSS® Inc., (2001) was used to compare between the removal performance of the reactors R1 and R2 by the T-test. If the resulted value of ($\rho<0.05$), then there was a difference between the means of the two tested groups and the data between the tested groups were considered statistically significant.

Chapter 4

Results and Discussion

4.1 Influent sewage characteristics

Main characteristics of the raw wastewater tested in this research between 8th of January 2006 and 7th of May 2006 are shown in Table 4.1.

Table 4.1 Characteristics of the influent sewage at Al-Bireh Wastewater Treatment Plant – Palestine during the whole period of the experiment

Parameter	*# of Samples*	*Range*	*Average*	*STD*
COD_{tot}	25	616-1301	1062	179
COD_{ss}	6	548-780	694	91
COD_{dis}	6	277-464	388	74
CODcol	6	161-429	245	80
VFA as COD	6	53-187	125	45
BOD_5	10	301-681	512	122
COD/BOD_5	10	0.03-3.6	2.13	0.94
PO_4^{-3} as P	8	8-24	12	5
NH_4^+ as N	12	47-201	99	59
T ambient	51	8-27	17	5.70
pH	6	7.4-8.0	7.7	0.19
TS	5	1364-3138	1884	732
VS	5	600-2036	980	596

All parameters are in mg/l except :(ambient temperature (T_{amb}) in °C); pH no units.

The sewage of the tested location is classified as high strength to Metcalf and Eddy (1991).

4.2 Performance of the UASB–septic tank (Part I)

The first part of this research is to assess the long term performance of the UASB-Septic tank system.

4.2.1COD Removal efficiency

The results of the COD removal efficiencies for the UASB-septic tank are tabulated in Table 4.4a and represented by Figure 4.1, 4.2, 4.3, 4.4, for $COD_{tot,}$ COD_{ss}, COD_{col} and COD_{dis}, respectively. During the period of the research the results of the UASB-septic tank with HRT of 2 days show that the average removal efficiencies for COD_{tot}, COD_{ss}, COD_{col}, COD_{dis} were 72%(6),81%(19),58%(21),55%(14), respectively. From Table 4.2 one can see when comparing these results with Al-Shayah (2005) and Al-Jamal (2005) at the same period that the UASB-septic tank is more efficient in the long term for removing COD $_{tot.}$ These results show that the UASB-septic tank become more efficient in removing COD_{tot} in the long run.

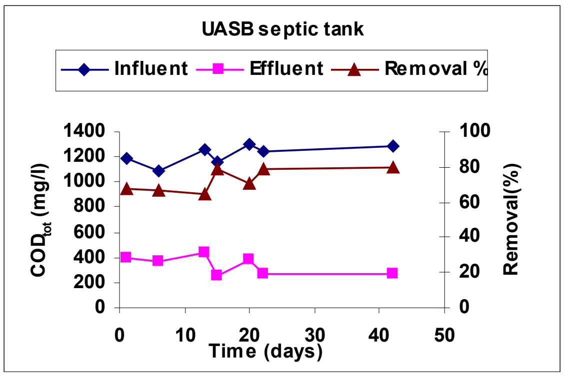


Figure 4.1. COD_{tot} influent and effluent concentrations and removal efficiencies for UASB-septic tank.

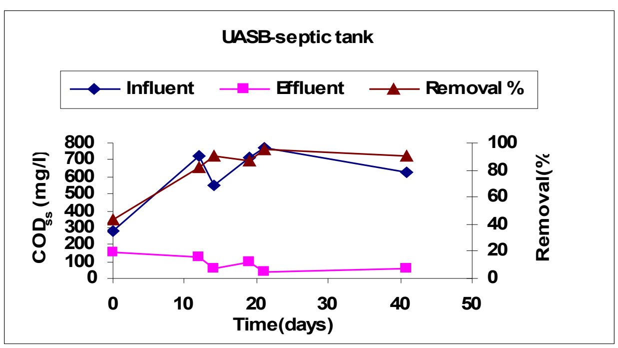


Figure 4.2. COD_{ss} influent and effluent concentrations and removal efficiencies for UASB-septic tank.

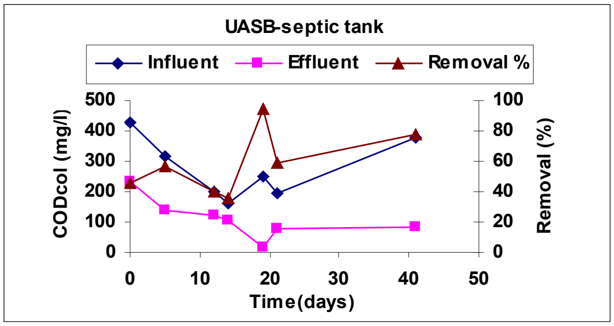


Figure 4.3. COD_{col} influent and effluent concentrations and removal efficiencies for UASB-septic tank

Table 4.2 Results comparison between reactor removal efficiencies after one year operation and 4 month

Parameter	Al-Shayah 2005	Al-Jamal 2005	This Research
COD_{tot}	54(6)	51(9)	72(6)
COD_{ss}	85(6)	83(10)	81(19)
COD_{col}	27(19)	20(32)	58(21)
COD_{dis}	12(20)	24(15)	55(14)
VFA	-9(27)	-1(52)	37(49)
BOD_5	56(10)	43(12)	68(10)
TS	TSS=79(5)	TSS=74(10)	34(3)
VS	VSS=79(4.9)	VSS=74(10)	63(3)
PO_4^{3-}-P	-21(9)	-38(54)	-28(11)
NH_4^+-N	16(8)	12(21)	5(6)

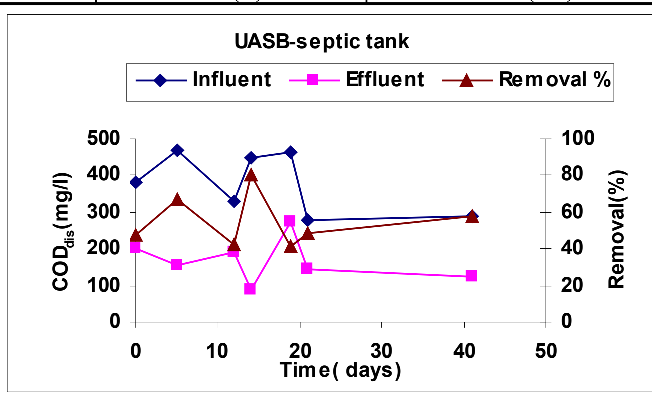


Figure 4.4. COD_{dis} influent and effluent concentrations and removal efficiencies for UASB-septic tank

4.2.2 VFA

The results of the volatile fatty acids (VFA) concentrations for influent and effluent in UASB-septic tank were shown in Table 4.4a and Figure 4.5 were the average concentration for VFA at the UASB-septic tank were 68(47) mg/l with average removal efficiency 37%(49). Comparing with Al-Shayah (2005) and Al-Jamal (2005) one can see that the VFA removal efficiency increased by a good rate when operating the UASB-septic tank for along time. In Al-Shayah research the UASB-septic tank shows a negative removal of VFA at the same period see Table 4.2

The VFA concentration in the effluent was affected by temperature and the methanogenises conditions were the production of the VFA increased during the summer period.

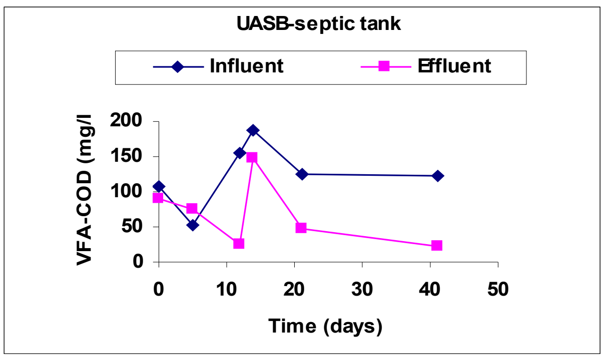


Figure 4.5. COD_{dis} influent and effluent concentrations for UASB-septic tank

4.2.3 COD mass balance

COD mass balance is based on the fact that the daily mass of influent COD is equal to the sum of the daily mass of COD leaving the system (reactors) as methane, effluent and accumulated COD in the sludge bed. In this research the mass balance over the UASB-septic tanks during the period of the research are summarized in Figure 4.6.

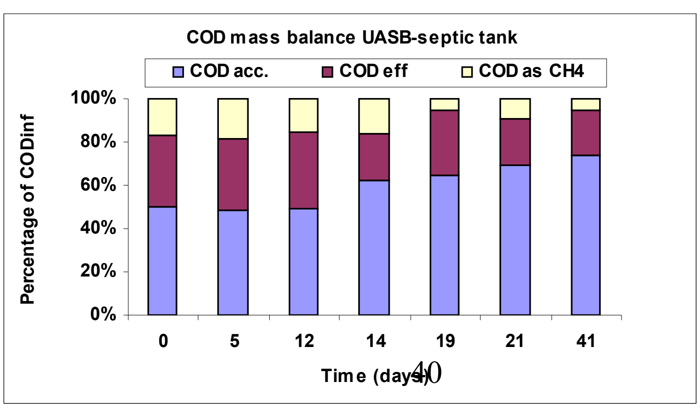


Figure 4.6. COD mass balance for UASB-septic tank over the test period as a percentage of average influent COD_{tot} and divided to $COD_{acc.}$, COD_{eff} and CH4 as COD. UASB reactor was at HRT=2days

Each column represents the acumulated COD in the reactor,effluent COD, total methane produced as COD gas form and dissolved form

From Figure 4.7 one can see that around 60% of the incoming COD was retained and accumulated in the reactor.

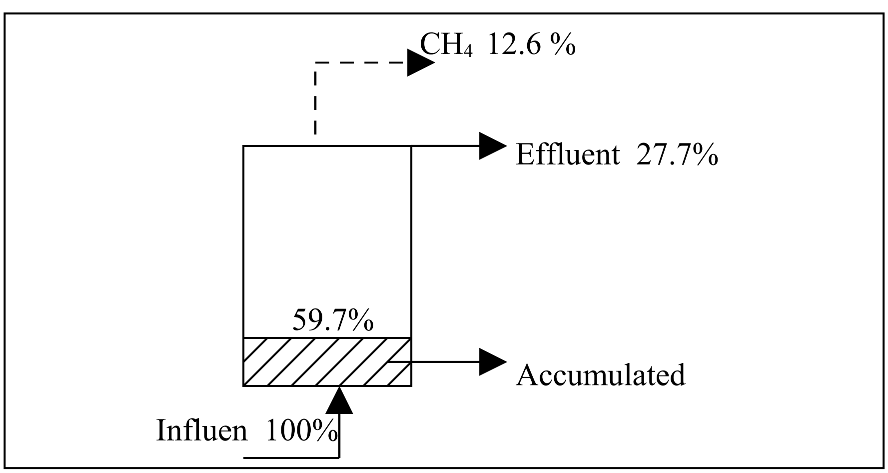


Figure 4.7. COD mass balance of UASB-septic tank over the 45 days. UASB-septic tank reactor was at HRT=2days. # of sample = 7, test period from 24 Augest to 4 October 2005

4.2.4 Characteristics of the retained sludge in the UASB-septic tank

The characteristics of the retained sludge of both reactors used in this research (UASB-septic tank and AH septic tank) are tabulated in Table 4.3a and Table 4.3b.

Table 4.3a. Characteristics of the retained sludge in the UASB-septic tank from the first port 0.15m from the bottom of the reactor

Parameter	# Samples	UASB-septic tank
Total Solids (TS)	2	55.20(0.75)
Volatile Solids (VS)	2	38.99(0.47)
VS/TS	2	70.64(1.81)
Stability at day 120	1	81%

All parameters in g/l except (VS/TS) ratio %; stability %(g CH4-COD). SD presented between parentheses

Table 4.3b. Characteristics of the retained sludge in the UASB-septic tank from the second port 0.4m from the bottom of the reactor

Parameter	# Samples	UASB-septic tank
Total Solids (TS)	2	24.63(1.29)
Volatile Solids (VS)	2	17.51(1.17)
VS/TS	2	71.08(1.02)

All parameters in g/l except (VS/TS) ratio %. SD presented between parentheses

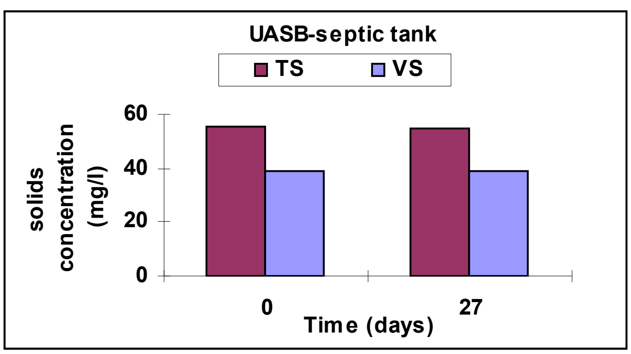


Figure 4.8. The time course for the sludge concentration in UASB-septic tank and anaerobic hybrid (AH) as TS, VS.

4.2.5 BOD removal efficiency

The mean values of the effluent BOD_5 concentration and the calculated removal efficiencies of the UASB-septic tank are shown in Table 4.4a and presented in Figure 4.9 .The average removal efficiency during the 45 days were 68%(10) comparing with 56%(10) obtained by Al-Shayah (2005) and Al-Jamal (2005) at the same time of the year see Table 4.2. One can see that removal efficiency increased by about 12%, these results show that the long run operating of the UASB-septic tank increase BOD_5 removal efficiency.

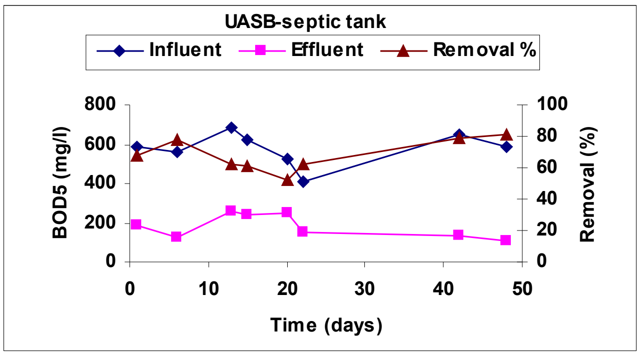


Figure 4.9. BOD_5 influent and effluent concentrations and removal efficiency for UASB-septic tank.

4.2.6 TS and VS removal efficiency

Table 4.4a shows the average TS and VS effluent concentrations and calculated removal efficiencies for the UASB-septic tank. The average removal Efficiencies for TS and VS over the study period for the UASB-septic tank were 34 %(4) and 63 %(3) respectively. These values relatively lower than results obtained by Al-Shayah (2005) and Al-Jamal (2005) see Table 4.2 . From Figure 4.10 and Figure 4.11 one can see that the removal efficiencies in the UASB-septic tank nearly remained constant during 45 days.

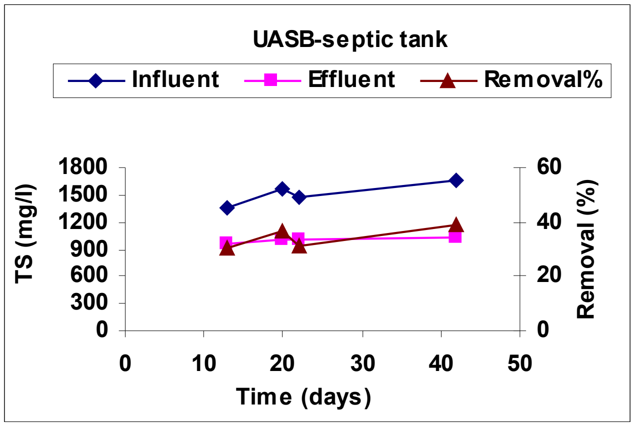


Figure 4.10. TS influent and effluent concentrations and removal efficiency for UASB-septic tank.

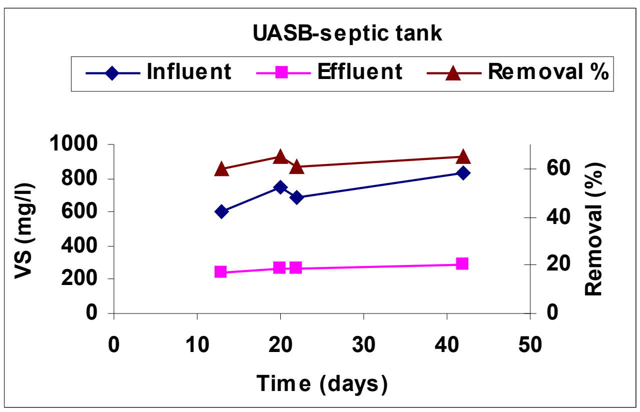


Figure 4.11 VS influent and effluent concentrations and removal efficiency for UASB-septic tank.

4.2.7 Nutrients removal efficiency

Nitrogen removal. Figure 4.12 and Table 4.4a show the variation of NH_4^+-N concentrations and average removal efficiency of the UASB-septic tank during the study period. The average NH4+-N removal efficiency for the UASB-septic tank was 16 %(8). This result is slightly more than the result obtained by Al-Shayah (2005) and Al-Jamal (2005) see table 4.2.

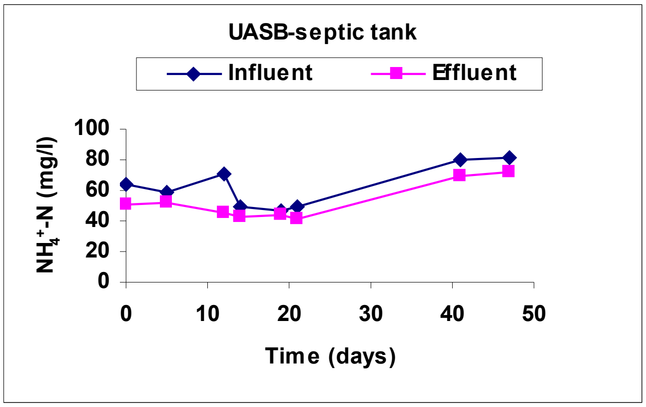


Figure 4.12 The evolution of NH4$^+$-N concentration for the UASB-septic tank.

Phosphorous removal. Table 4.4a and Figure 4.13 show the concentration of ortho-phosphate (PO_4^{3-}) along the 45 days. The average concentration of ortho-phosphate in the influent increased from 13(5) mg/l to 15(4) mg/l in the UASB-septic tank. These results are nearly the same as results obtained by Al-Shayah (2005) see Table 4.2.

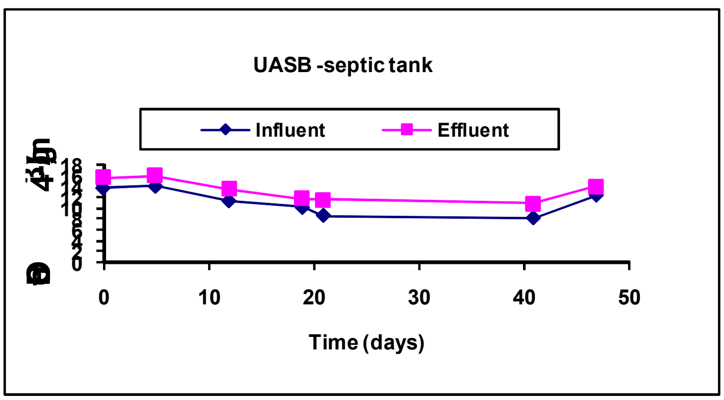


Figure 4.13 the concentration of Ortho-phosphate (PO_4^{3-}) in the influent and effluent for UASB-septic tank

Table (4.4a*) Research results for the effluent concentration and removal efficiency (%) during the whole period of experiment in the UASB-septic tank under the imposed operational conditions. Standard deviations are presented between brackets

Parameter	***# of samples***	***Influent concentration***	***UASB-septic tank (long run) From day 0 to day 42** HRT=2 days***			
			Effluent concentration		Removal efficiency (%)	
			Rang	AVG	Rang	AVG
COD_{tot}	7	1219	248-439	337(75)	65-79	72(6)

COD_{ss}	7	611	279-775	90(46)	44-95	81(19)
COD_{col}	7	253	15-234	111(67)	35-94	58(21)
COD_{dis}	7	296	277-469	168(60)	41-80	55(14)
COD-VFA	7	125	53-187	68(47)	-42-84	37(49)
BOD_5	7	576	405-681	182(60)	53-81	68(10)
PO_4^{3-} as P	8	13	11-24	15(4)	-34-(-13)	-21(9)
NH_4^+ as N	8	62	41-72	52(12)	7-35	`16(8)
pH	6	7.8	7.4-8.2	7.92(0.29)	-	-
TS	5	1520	950-1020	996(32)	30-39	34(4)
VS	5	717	242-290	264(20)	60-65	63(3)

All parameters are in mg/l, PH no unit
*Results for UASB-septic tank in the long run
**Day zero in this table equal the day 444 from the start day of the reactor which is the 4th of May 2004.

4.3 Performance of the UASB–septic tank and the anaerobic hybrid-septic tank reactors (part Π)

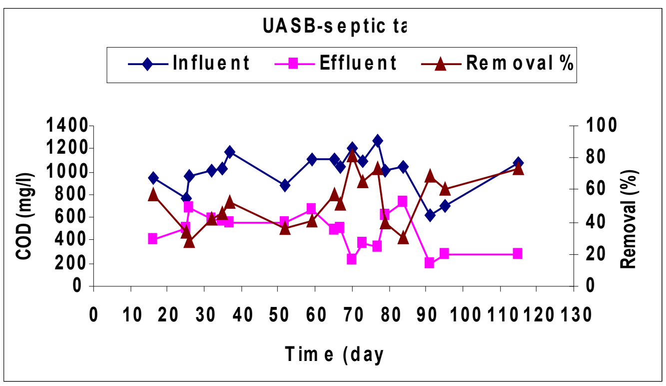
The performance of the UASB-septic tank before adding (RPF) sheets which had been studied for about 45 days and the performance of the anaerobic hybrid septic tank are summarized in table 4.4b.

4.3.1 COD removal efficiency

The whole results of the COD removal efficiency for the UASB-septic tank and the anaerobic hybrid-septic tank during the 120 days (from 23rd of January 2006 to 2nd of May 2006) are tabulated in Table 4.4b and represented by Figure 4.14 for COD_{tot}.
During the period of the research the results of anaerobic hybrid show that the average removal efficiency for COD_{tot} was 49% (17). The results also show for UASB-septic tank that the average removal efficiency for COD_{tot} was 52 %(16). In general one can see that UASB-septic tank was more efficient in removing CODtot in spite of using (RPF) filter in (AH).

The average removal efficiency and the average effluent concentration of COD_{tot} were shown in Table 4.4b for both of the two reactors. the average effluent concentrations of COD_{tot} for both reactors were 503 (153) mg/l and 476(162) mg/l, respectively with average removal efficiency of 49 %(17) and 52 %(16) for (AH) and UASB-septic tank, respectively.

The result from statistical analysis show that the difference of CODtot removal efficiency found between the two reactors were not statistically significant ($\rho>0.05$). Figure 4.14 shows the variation of the effluent CODtot concentration of both reactors and the removal rate of CODtot to the influent concentration. From the results above one can see that UASB-septic tank is slightly more efficient in removing CODtot.



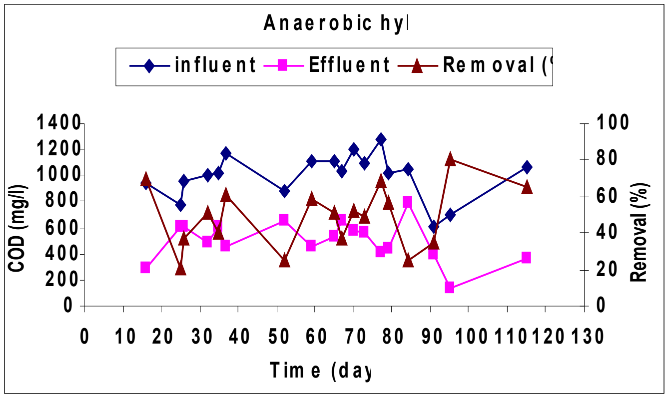


Figure 4.14. COD_{tot} influent and effluent concentrations and removal efficiencies for UASB-septic tank & the anaerobic hybrid-septic tank.

In this part the efficiencies in removing COD_{tot} for UASB-septic tank had been increased with about (1%) and this may be regarding to the slightly increase in the temperature at the beginning of spring (April & May). Temperature is an important factor in anaerobic treatment of domestic wastewater: the higher the temperature, the higher the conversion rates (Luostarinen, 2007).

4.3.2 BOD removal Efficiency

The mean values of the effluent BOD_5 concentration and the calculated removal efficiencies of the two UASB-septic tank reactors shown in table 4.4b the average removal efficiencies during the whole period of study were 56%(3) and 33%(14) for UASB-septic tank and AH-septic tank reactor, respectively. As shown UASB-septic tank achieved a higher BOD_5 removal efficiency than AH reactor, the average BOD_5 effluent was 254(65) mg/l for AH reactor and 169 (33) mg/l for UASB-septic tank. Figure 4.15 shows the average values of BOD_5 concentrations and removals for both reactors. The result from statistical analysis show that the difference of BOD_5 removal efficiency found between the two reactors were statistically significant ($\rho>0.05$); where $\rho = 0.022$.

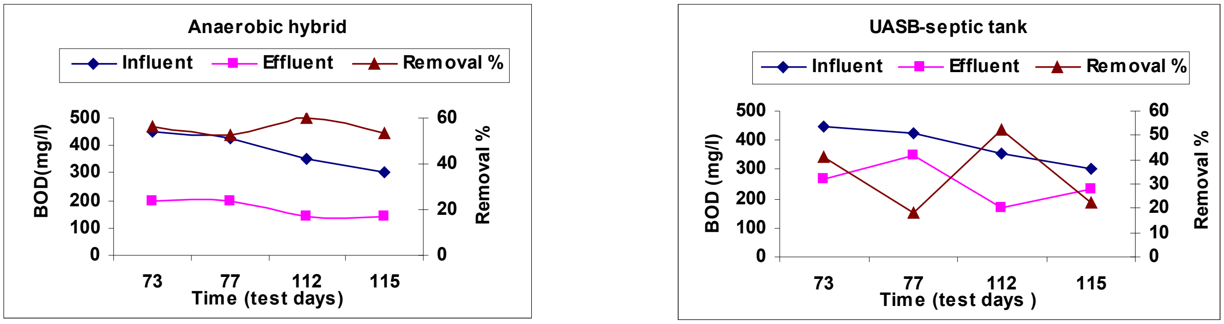


Figure 4.15. BOD_5 influent and effluent concentrations and removal efficiencies for both reactors along the study period

Table (4.4b*) Research results for the effluent concentration and removal efficiency (%) during the whole period of experiment in the UASB-septic tank and Anaerobic Hybrid reactors under the imposed operational conditions. Standard deviations are presented between brackets.

Parameter	***# of samples***	***Influent concentration***	***Anaerobic Hybrid-septic tank HRT=2days From day 0 to day 115****				***UASB-septic tank HRT=2days From day 0 to day 115****			
			Effluent Concentration		**Removal efficiency (%)**		**Effluent Concentration**		**Removal efficiency (%)**	
			Rang	**AVG**	**Rang**	**AVG**	**Rang**	**AVG**	**Rang**	**AVG**
COD_{tot}	18	1001(171)	138-788	503(153)	21-80	49(17)	192-726	476(162)	28-82	52(16)
BOD_5	4	382(68)	168-348	254(75)	18-52	33(16)	140-201	169(33)	53-60	56(3)
NH_4^+ as N	4	171(41)	88-186	151(44)	7-21	13(6)	82-196	147(48)	2-27	15(11)

All parameters are in mg/l,
*Results for UASB-septic tank and Anaerobic Hybrid (AH)
**Day zero in this table equal the start day of the second part of the research.

4.3.3 Characteristics of the retained sludge in the UASB-septic tank and AH-septic tank reactors

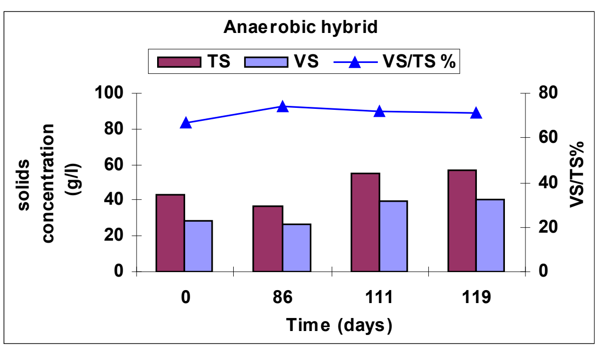
The characteristics of the retained sludge of both reactors used in this research UASB-septic tank and AH-septic tank are tabulated in Table 4.5. The samples during the period of the research taken from tap1 for reactors, the sludge analyzed for total solids (TS), Volatile solids (VS), and stability results shown in Figure 4.16.

Table 4.5. Characteristics of the retained sludge in the UASB-septic tank and AH reactors from the first port 0.15m from the bottom of the reactor

Parameter	*# Samples*	*UASB-septic tank*	*Anaerobic Hybrid*
Total Solids (TS)	5	82.20(30.55)	47.93(9.75)
Volatile Solids (VS)	5	58.94(27.29)	33.99(7.02)
VS/TS	5	71.04(6.58)	70.94 (2.85)
Stability	1	92%	84%

All parameters in g/l except (VS/TS) ratio %; stability %(g CH_4-COD). SD presented between parentheses

At day 120 (i.e. end of the research) the sludge reached height 0.25 m in AH and 0.35m in UASB septic tank.



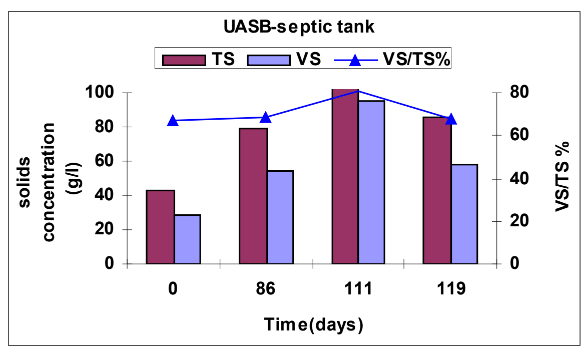


Figure 4.16 The time course for the sludge concentration in UASB-septic tank and anaerobic hybrid (AH) as TS, VS.

The average of the total solids concentration (TS) for both reactors was 47.93(9.75) g/l for Anaerobic hybrid and 81.20(30.55) g/l for UASB-septic tank. Comparing results of UASB-septic tank with 52.9(5.72) g/l reported by Al-Jamal (2005).

4.3.4 Nitrogen removal

The results during the test period show that the average removal of NH_4^+ was very low for both AH an UASB-septic tank reactors where the average (NH_4^+-N) concentration for the AH was 151(44) mg/l and 147 (48) mg/l for the UASB-septic tank. This was regarding to the low hydrolysis rate in the part of organic matter which contain organic nitrogen i.e (the organic nitrogen and protein did not hydrolyses completely) Table 4.4b and figure 4.17 describe the variation of NH_4^+-N concentration and the removal efficiencies of the reactors during the study period the average removal efficiencies during the whole period of study was 13% (6) for AH and 15%(11) for UASB-septic tank. The result from statistical analysis show that the difference of NH_4^+-N removal efficiency found between the two reactors were not statistically significant ($\rho<0.05$) ;where $\rho = 0.28$.

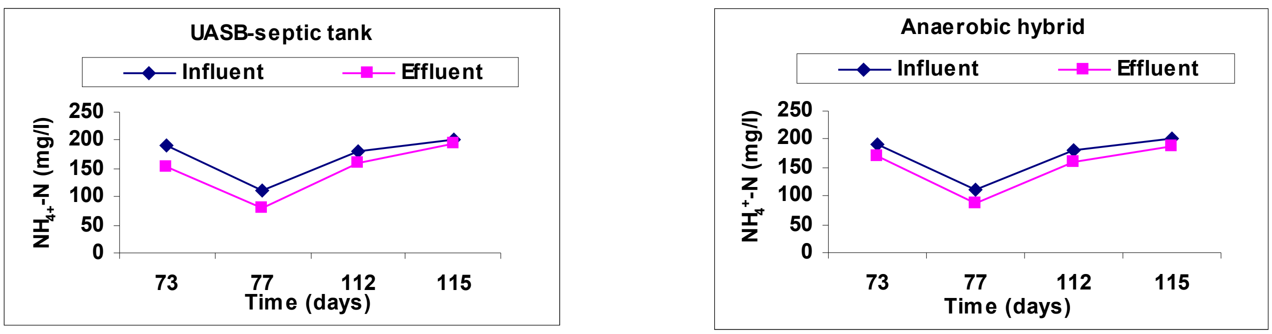


Figure 4.17 NH_4^+-N concentration and removal efficiencies for both influent and effluent for UASB-septic tank and Anaerobic Hybrid (AH).

4.4 General results

As a general result one can see that the performance of the UASB-septic tank is better than Anaerobic Hybrid (AH). Table 4.6 show the results of two step AH-septic tank system at an HRT of 2.5 days for each step at a temperature of 18°C and 13°C . Comparing these results with this study one can see that the Anaerobic Hybrid reactor can perform better if special modification done.

Table 4.6 Average COD removal efficiencies (%) for different fractions in the treatment of concentrated domestic sewage in the two –step AH septic tank system at HRT of 2.5 days for each step and temperature of 18°C and 13°C. Standard deviation is presented in brackets (Elmitwalli *et al*., 2003)

COD	First step		Second step	Two steps
	18°C	13°C		
COD_{tot}	93.9(0.1)	79.7(5.9)	69.5(0.4)	93.8(1.7)
COD_{ss}	97.7(1.4)	80.5(7.3)	90.8(8.4)	97.9(2.3)
COD_{col}	69.3(13.1)	65.6(22.1)	16.7(23.6)	74.0(10.3)
COD_{dis}	70.1(12)	72.7(0.5)	19.6(7.6)	78.1(1.7)

Chapter 5

Conclusions and Recommendations

5.1 Conclusions

According to the results obtained from this study we can conclude that

1. Operating the UASB-septic for a long time results in high removal efficiency for COD_{tot} COD_{col} and COD_{dis} comparing it with previous results. The average removal efficiencies for COD_{tot} COD_{col} and COD_{dis} are 72%, 65% and 55% respectively.
2. Operating the system for along time didn't show an increase in nutrients removal. The average effluent concentration obtained from this research is nearly the same as the results obtained over the first year, were the average effluent concentration in this research for (NH_4^+-N, PO_4^{3-}-P)are 52 (12) mg/l and 15(4) mg/l respectively.

3. For the second part of this study, the results obtained show that in spite of adding RPF media the performance of the UASB-septic tank is slightly better than Anaerobic Hybrid were the removal efficiency for COD_{tot} 49%(19) and 52%(16) for Anaerobic Hybrid and UASB-septic tank respectively.

4. In addition to these results both of the reactors are not efficient for removing nutrients with removal efficiency 15 %(11) for UASB-septic tank and 13 %(6) for Anaerobic Hybrid.

5.2 Recommendations

1. Regarding to the results reached in this study it is recommended to use (RPF) in the Anaerobic Hybrid (AH) but with some modification like decrease the number of sheets used in the filter. In this research 10 RPF sheets were used, this high number of sheets increased the upflow velocity of wastewater between channels created by sheets. If we

decrease sheets number this will decrease the upflow velocity, decrease upflow velocity will reduce solids amount hold up and drawn out from the system.

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Web sites:

www.uasb.org

Appendix

Appendix 1

Photos of the Experimental Set-up

PhotoA2.1: Front view of the UASB-septic tank reactors (R1&R2) And the holding tank which the reactors were fed.

Photo A2.2: Top view of the UASB –septic tank reactors before adding the filters

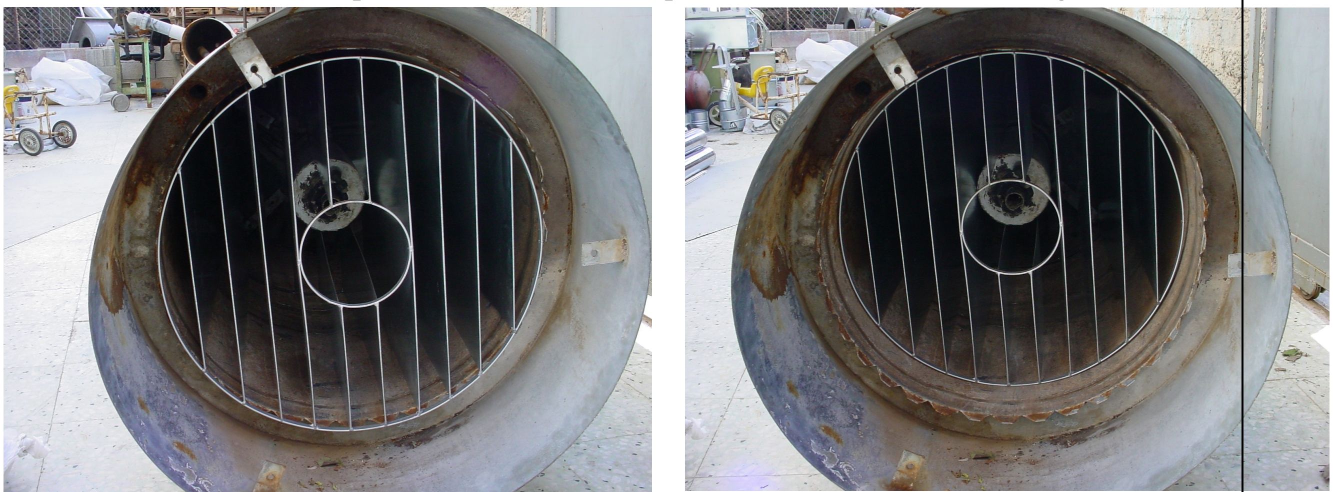

Photo A2.3: Top view of the UASB –septic tank reactors after adding the filters.

Photo A2.4: Side and top view of the filter before adding RPF (Reticulated Polyurethane foam)

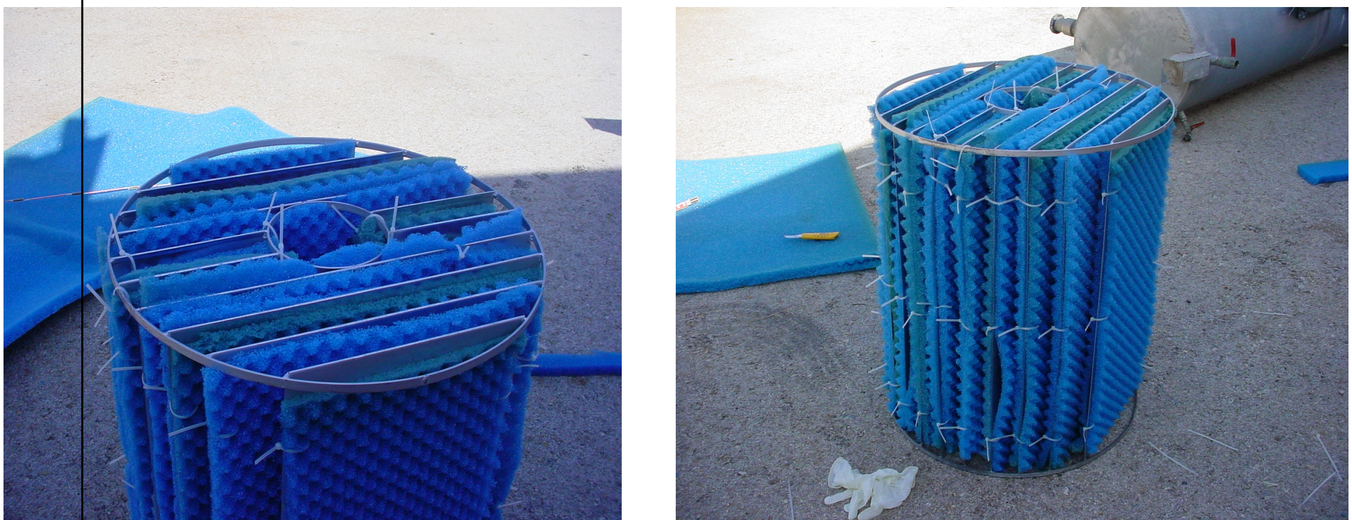

Photo A2.5: Side and top view of the filter after adding RPF (Reticulated Polyurethane foam)

Photo A2.6: Reticulated Polyurethane foam with high surface area

Appendix 2

Preparation of Stability Bottles

Stability Bottles

The stability test was carried out in batch reactors, serum bottle of 500 ml with a headspace volume of 70 ml. The procedure for preparation of the sludge stability bottles were as follow: Each bottle of the stability test was filled with about 1.5 g COD-sludge/L, in addition to 50 ml of specific media and completed to the 500 ml mark with tap water. The is mineral solution of macro nutrients, trace element, bicarbonate buffer and yeast extract as described below. After that the Ph of the content was adjusted to 7 using diluted HCl or NaOH solutions, thereafter, the bottles were sealed with septa and aluminum crimps, and the head space of the bottles were flushed with nitrogen gas for 3-4 minutes to achieve anaerobic conditions. Anaerobic conditions were also assured by syringing of sodium sulfide solution through the septum of each bottle. The bottles then incubated at 30°C for a period of 120 days. COD total was measured at the beginning and at the end of the batch period. All measured were determined in triplicate. The sludge stability was calculated as the amount of methane produced during the test (as COD) divided by the initial COD of the sludge sample. Methane production was monitored in time through the displacement of a 5% NaOH solution.

Media solution preparation

The media used in this research were prepared by the addition of the following contents to 1000 ml flask and stirred using a magnetic bar:

- 20 ml macro nutrients stock solution, as prepared below in table (A2.1.)
- 10 ml micro nutrients (trace elements), as prepared below in table (A2.2)
- 25 g $NaHCO_3$ (buffer solution).
- 0.5 gm yeast extract.
- Demineralized water: fill up the flask to 1000 ml mark.

Sodium Sulphid (Na_2S) solution preparation

The Na_2S solution was prepared fresh by dissolving 1.25 g Na_2S in 50 ml demi water. When the chemical compound available in the form of $Na_2S.XH_2O$(X: 7-9); add 3.57g/50ml.1ml of the prepared Na_2S solution was added to each batch bottle.

Table A2.1 Macronutrients stock solution

Chemical substance	Concentration in 500 ml serum bottle (g/l)	Weight to be added to 250 ml flask as stock solution (500 times concentrated)*(g)
NH_4Cl	0.28	35
KH_2PO_4	0.25	31.25
$CaCl_2.2H_2O$	0.01	1.25
$MgSO_4.7H_2O$	0.1	12.5

*: use demineralized water to fill the flask and shake the solution well.

Table A2.2 Micronutrients (trace elements) stock solution

Chemical substance	Concentration in 500 ml serum bottle (mg/l)	Weight to be added to 1000 ml flask as stock solution*(mg)
FeCl2.H2O	2	2000
H3BO3	0.05	50
ZnCl2	0.05	50
CuCl2.2H2O	0.038	38
MnCl2.4H2O	0.5	500
(NH4)6MO7O24.4H2O	0.05	50
AlCl3.6H2O	0.09	90
CoCl2.6H2O	2.0	2000
NiCl2.6H2O	0.092	92
Na2S2O3.5H2O	0.164	164
EDTA(C10H16N2O8)	1.0	1000
Resazurine	0.2	200
HCl(36%)	0.001(ml/l)	1.0(ml)

*: use demineralized water to fill the flask and shake the solution well.

ملخص

في بلدان مثل فلسطين حيث مصادر المياه النقية شحيحة ووسائل التخلص من المياه العادمة تتم بشكل غير صحي حيث يتم تصريفها في البيئة بدون اي نوع من المعالجة.

من هنا كان لابد من ظهور فكرة ضرورة المحافظة على المصادر المائية ومحاولة استخدامها الاستخدام الامثل رغم الظروف الاقتصادية السيئة التي تعاني منها بعض دول العالم وبحيث يجب العمل على تقليل مصادر تلوث المياه العادمة والعمل على معالجة المياه الملوثة والتي تحتاج إلى تقنيات حديثة ومتطورة وعالية التكلفة وفي فلسطين ونتيجة للظروف القاهرة الناتجة عن الاحتلال كان لابد من اختيار الطرق المناسبة للحد من التلوث ومعالجة المياه العادمة بحيث تكون هذه الطريقة ذات كفاءة عالية وسهلة التطبيق وغير مكلفة لذلك جاءت فكرة استخدام نظام المعالجة الاهوائية (UASB-septic tank)) بدلا من الوسائل التقليدية مثل الحفر الامتصاصية المستخدمة بشكل كبير لتصريف المياه العادمة في فلسطين

إن الهدف الأساسي من وراء هذه الرسالة هو أولا البحث في مدى أداء وجدوى استخدام التقنية الاهوائية في معالجو المياه العادمة على المدى الطويل أي بعد ما تم تشغيله لمدة عام كامل بواسطة الشياح (2005) والجمل(2005) ضمن الظروف الموجودة في فلسطين تحت درجة حرارة تتراوح بين (8-27), بحيث تم تشغيل المفاعل بزمن مكوث للمياه العادمة((HRT لمدة يومين. بالنسبة للمياه العادمة كان معدل تركيز الأكسجين الكلي المستهلك كيميائيا(CODtot) يساوي 1062ملغم/لتر وبنسبة مقدارها 2.13بين ((CODtot/BOD5

لقد تبين أثناء الدراسة ان المفاعل اثبت كفاءة اعلى على المدى الطويل حيث كانت معدلات إزالة الملوثات من المياه العادمة على النحو التالي (CODtot.CODsus,CODcol,CODdis) هي (72%و 82%و 58%و 55%)على التوالي واضف إلى ذلك كانت نسبة إزالة الملوثات ل((BOD5 685ول ((TS 34%. أما بالنسبة للجزء الثاني من البحث فقد كانت نسبة ((CODtot49% و 33% بالنسبة ل (BOD5) وذلك ل(Anaerobic Hybrid).