Community Onsite Anaerobic Sewage Treatment
In a UASB-Septic Tank System: System behavior during winter period in Palestine

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This thesis was prepared under the main supervision of Dr. Nidal Mahmoud and has been approved by all members of the examination committee.

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

DEDICATION

FOR MY COUNTRY

" PALESTNE "

AND ALL PALESTINIANS

ESPECIALLY

THOSE WHO WORK ON THE

DEVELOPMENT AND FREEDOM OF

PALESTINE

TO MY PARENTS, MY SISTERRS,

MY BROTHERS IN LAW,

AND ALL OF MY FRIENDS

WITH MY LOVE AND RESPECT

Wafa Mohammed Al-Jamal

September, 2005

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ACKNOWLEDGMENTS

Abstract

The amount of water available per person has been declining throughout the world as a result of expanding populations and environmental changes. Industrialization and urbanization in Palestine are polluting groundwater and degrading the quality of surface waters by overloading them with more organic material than can be assimilated naturally. So wastewater management should be viewed as an important component of water resource management. In view of the economical situation existing in Palestine and the necessity for pollution control, wastewater treatment technologies should be sustainable, cost effective and environmentally sound. These technologies should combine a high efficiency with simplicity in construction and operation and maximize the opportunities for efficient removal of pollutants.

Particular attention is given to the up flow anaerobic sludge blanket (UASB) - septic tank technology to be the key point to be the affordable on-site sanitation alternative for household instead of the cesspits which consider as the known and commonly on-site method for wastewater disposal and sewage pre-treatment in Palestine. However, the performance of this technology has not been investigated especially under Palestine winter conditions.

The main objective of this thesis was to formulate design criteria for designing the UASB-septic tank for pre sewage treatment under Palestinian\Middle East conditions namely at law temperature period of the year. Moreover, Attempts were made to evaluate the effect of HRT on performance of UASB-septic tank.

An on-site two pilot scale UASB-septic tank reactors treating domestic sewerage under different HRT (2 days for R1 and 4 days for R2). The two reactors were operated in parallel at Al-Bireh wastewater treatment plant in Palestine. The two reactors were operated for six months at ambient temperature fluctuates between 2 to 27 ℃ with an average value of 14.7 ℃, the average sewage temperature was 17.3 ℃ with 12 ^{0}C and 25 ℃ extreme values. The domestic sewage treated in the research period classified as (medium strength) regarding to the Metcalf and Eddy (1991) and EPA (1999), with average concentration COD_{tot} of 905 mg/l with (COD/BOD$_5$) of 1.97. The COD_{ss} in the raw sewage represented a high fraction of the total COD, viz. about 43.7% from the COD_{tot}.

The performance data obtained During the period of the research showed for R1 with HRT of 2 days that the average removal efficiency for COD_{tot}, COD_{sus}, COD_{col}, COD_{dis}, were 51%, 83%, 20%, 24% respectively also the BOD_5 and TSS average removal efficiency of 45% and 74% respectively. And so for R2 of   HRT of 4 days the average removal efficiency for COD_{tot}, COD_{sus}, COD_{col}, COD_{dis}, were 54% , 87 %, 10% , 28% respectively with BOD_5 and TSS   average removal efficiency of   49% and 78% respectively.   Moreover, Results show that R1 and R2 are not efficient for removing nutrient from wastewater but also it shows an increase in the (NH_4^+ - N), Nkj-N removal efficiency comparing to the summer period.

The evolution of biogas production ($CH_{4 \ (gas \ form \ + \ liquid \ form)}$) was strongly affected by temperature. The average total methane production from both reactors was 0.11 N m^3/kg COD removed and 0.10 N m^3/kg COD removed   for R1 and R2 respectively.

The sludge hold-up time of the system is so long and withdraw of sludge could be done once every 4 years for this system. The (VS/TS) ratio for the sludge was about average ratio of 67.9 and 67.02 for R1 and R2 respectively those values can indicate a well-stabilized sludge and this proofed with the stability tests. Also stability tests show that the retained sludge in R2 was more stable than R1.The results obtained in this research shows that the longer HRT R2 (4 days) gave better efficiency than R1 (2 days) in most the tested parameter during this research, even if most of them not statistically significant. As a general conclusion the anaerobic systems can be easily applied at any scale, and it could be applied to the Palestine and Middle East region.

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List of Abbreviations

A :	aciddification
atm:	atmospheric
AF:	anaerobic filter
AH:	anaerobic hybrid
AVR:	average
BOD:	biological oxygen demand
COD_{col}:	colloidal COD
COD_{dis}:	dissolved COD
COD_{filt}:	filtered COD
COD_{sus}:	suspended COD
COD_{tot}:	total COD
$COD-CH_4$:	COD as CH_4
d:	day
eff:	effluent
EGSB:	expanded granular sludge
g:	gram
GLS:	gas –liquid-solids three phase separation
FB:	fluidized bed
H:	hydrolysis
hr:	hour
HRT:	hydraulic retention time
HUSB:	hydrolysis upflow sludge bed
in:	inch
inf:	influent
L:	liter
M:	methanogenesis
m:	meter
mg:	milligram
ml:	millititer
Nkj:	kjeldhal nitrogen
nm:	nanometer
OLR:	organic loading rate

P:	phosphorous
R:	reactor
rpm:	revolution per minute
S:	sulfate
SRT:	solids retention time
SS:	suspended solids
STD:	standard deviation
SVI:	sludge volume index (ml/g)
T:	temperature (ºC or ºK)
TS:	total solids
TSS:	total suspended solids
UASB:	upflow anaerobic sludge blanket
V:	volume
VFA:	volatile fatty acids (g COD/l)
VS:	volatile solids
VSS:	volatile suspended solids
V_{up}:	upflow velocity (m/hr or m/d)
WWTP:	wastewater treatment plant
Greek	
ρ:	density
μm:	micrometer

Chapter 1

Introduction

1.1 Background

Sewage is the main point-source pollutant on a global scale (Gijzen, 2002). Between 90 and 95% of the sewage production in the world is released into the environment without any treatment (Bartone *et al.,* 1994). In some developing countries about 100% of the wastewater production in households from most cities and towns is discharged untreated in water bodies or infiltrate to the ground water. Waste water generated from Palestinian cities, Villages and Israelis colonies is considers as the primary source of pollution in Palestine, such wastewater is discharged untreated in to open area or through cesspits where approximately 70% of the West Bank is not served with sewage net work (Mahmoud *et al.,* 2003).

In general the existing wastewater treatment plant either inadequate or non-existent in Palestine, about 6% of the total population in Palestine served with wastewater treatment plants, which are not functioning appropriately (Mahmoud *et al.,* 2004-a). The existence situation on Palestine could immediately or later disastrous effects on public health and the quality of environment will take place. According to WHO (1996), as a consequence to this lack of sanitation, 3.3 million people die annually from diar-rhoea diseases, out of 3.5 billion infected. In Africa alone, 80 million people are at risk from cholera, and the 16 million cases of typhoid infections each year are a result of lack of adequate sanitation and clean drinking water.

Appropriate and sustainable sewage treatment technologies will help to preserve and maintain health and freshwater. In many cases, traditional wastewater treatment technologies, such as the aerobic activated sludge process are inappropriate for the physical and economic characteristics of the small communities. The major reason for failure is that the conventional sewerage systems that are normally accompanied with centralized wastewater treatment plants are certainly far too expensive and complex for poor countries (Zeeman *et al.,* 2001). The application of these expensive systems, which are popular in Europe and America, does not

offer a sustainable solution for sewage treatment in less wealthy countries. Hence, non-point pollution, caused by direct discharges from rural communities can be significantly reduced by the promotion of on-site low cost treatment systems. Anaerobic treatment has been proven to be an admirable process and considered by many authors as the core of sustainable waste management (Mahmoud, 2002).

The anaerobic technology has been in existence for a very long time. According to McCarty (1981), it has existed as a technology for over one hundred years. Increasing energy prices and cost of operation and maintenance of aerobic treatment favored the development of anaerobic treatment processes, since these processes do not require energy input and just little maintenance and attention (Schellinkhout *et al.,* 1985). Anaerobic digestion processes occur in many places where organic material is available and oxidation potential is low (zero oxygen) as in the stomach of ruminants, in marshes, sediments of lakes and ditches, municipal landfills and also sewers (Alaerts *et al.,* 1990).

Anaerobic processes can be profitably applied for all types of waste of natural origin. Successful full-scale facilities have been constructed and operated for dilute wastewater such as municipal sewage and for very concentrated effluents such as rum stillage. Anaerobic treatment has been increasing rapidly in popularity worldwide (Lettinga *et al.,* 1988) and as a result of its cost competitiveness has evolved into a mature technology for waste treatment (Pfeiter *et al.,* 1986). However, with the present state of technological development and basic insight into the process, only a few of the presumed drawbacks remain, while all its principal benefits over conventional aerobic methods are still relevant (Lettinga, 1995, Lettinga *et al.,* 1988, Lettinga, 1996). 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). However, little experience is available on the performance and design of these reactors under the environmental conditions and wastewater characteristics of Palestine.

1.2 Aim of research

Wastewater characteristics such as COD concentration and temperature are the determinants of the UASB-septic tank design. Regarding the Palestinian domestic wastewater with high COD and seasonal temperature fluctuation, the design criteria of the UASB-septic tank are still to be formulated. Few investigations and researches had been done during the last years on such system (Al-Juidy, 2001; Ali, 2001) those researches were of short periods and thus did not consider the influence temperature fluctuation over the year. Moreover, the previously researched reactors were mostly fed with wastewater from Birzeit University or septage, and so no research had so far considered real domestic wastewater. Al-Shayah (2005) investigated the UASB-septic tank using high concentration wastewater from Al-Bireh wastewater treatment plant for six month from the first of April 2004 to the first of October 2004 this duration considered as the summer climate of Palestine where the temperature is high.

This research aimed to monitor the UASB-septic tank behavior at different environmental conditions, namely low temperature period of the year at Palestine. As such the influence of temperature fluctuation on the system behavior would have been covered. The inclusion of winter period influence on the system behavior is vital, as biological treatment is very sensitive to low temperature conditions.

In general little effort had been made to optimize the design criteria of the UASB-septic tank such as hydraulic retention time (HRT) under varied operational and environmental conditions, also the comparison of the previous results is in many cases difficult, as too many factors affect the anaerobic degradation and the performance of the reactor and each research work was carried out under different conditions.

1.3 Objective of thesis

The main goal of this research is to formulate design criteria for designing the UASB-septic tank for sewage treatment under Palestinian/ Middle East conditions. 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 reactors, 1 and 2, were operated respectively at 2 and 4

days at ambient temperature for the period from the first of October 2004 to the end of March 2005, so the influence of low temperature period on the reactors performance will be elucidated.

The Sub-goals of this research are:

- Monitoring the performance of the UASB-septic tank pilot plant treating domestic wastewater under Palestinian conditions. The reactor performance will be evaluated in terms of process efficiency (COD total and fractions, Volatile Fatty Acids (VFAs), ammonia, kjeldhal, phosphate) and process stability through monitoring the quantity of biogas produced, sludge bed floatation, sludge wash-out.

- Optimize and propose the applicable hydraulic retention time (HRT) for designing the UASB-septic tank,

- Study the sludge build-up and the filling period of the sludge in the UASB-septic tank.

1.4 Thesis Outline

This thesis contains five chapters, chapter 1 is the research introduction in which background, aim of the research and objectives are represented. Chapter 2 represents the literature review on anaerobic treatment process and alternatives. Chapter 3 reviews the materials methods used in the research. Chapter 4 present and discuss the results of this research. Finally chapter 5 summarizes the conclusion and the recommendation of this research.

Chapter 2

Literature review

2.1 Introduction

The quantity and strength of domestic wastewater depends on the size and the socioeconomic behavior of the population constituting the community. The wastewater generation in a community may constitute wastewater generated by: domestic, industrial, commercial, storm water in case of combined sewers and groundwater infiltration. The major constituents of the wastewater at community level are derived from the domestic sources compared to the other mentioned categories. The domestic sewage is composed of toilet wastewater (black water) and household wastewater, from the kitchen and bathroom (Haandel *et al.,* 1994). Wastewater is characterized in terms of its physical, chemical, and biological composition. The understanding of those characteristic influences the design of the treatment plant, particularly the size and type of the plant.

Conventional mechanical treatment facilities in developing countries had a sparse record of success. They frequently do not function as expected because of a variety of technical, financial and institutional reasons. Alternative treatment technologies emphasize cost reduction, integrated system management, minimal mechanical operations, and energy self-sufficient. As the Palestinian society is facing large economical assignment, the application of conventional aerobic wastewater treatment technologies is not a sustainable solution for treating the wastewater. Anaerobic digestion has been widely recognized as the core of sustainable waste management (Mahmoud, 2002), which has also been recognized by the Palestinian officials (PWA, 1998).

This research aims at increasing the knowledge on the technical applicability of the UASB reactor as a core technology for sustainable sewage treatment in Palestine especially at winter season.

2.2 Wastewater Treatment

Wastewater influent may be weak, medium and strong in terms of the concentration of the major constituents, oxygen consuming substance (COD and BOD_5), suspended solids (SS) and nutrients (N and P). Treatment of wastewater depends on natural processes, such as gravity to remove solids and bacterial action to stabilize biodegradable organic material. Complete wastewater treatment consists of a series of steps, which are commonly defined as follows:

1- Preliminary treatment is defined as the removal of wastewater constituents that may cause maintenance or operational problems (Metcalf and eddy, 1991) so large and heavy solids removes by screening and degritting.
2- Primary treatment entails sedimentation of (45-70) percent of settable solids that contain significant amount of oxygen consuming substance (20-40) percent but little or no removal of colloidal and dissolved organic matter (Scott McNiven, 1996).
3- Secondary treatment removes about 85 percent of suspended solids and BOD_5/COD (Scott McNiven, 1996).
4- Advance or tertiary treatment removes up to 99 percent of residual suspended solids and nutrient from a secondary treatment effluent.

Treatment to advance stage is typically not undertaken except to protect economically important receiving bodies of water against eutrophication, or to meet specific criteria for a particular reuse application. The main reasons are that the infrastructure and operating costs escalate dramatically, and operator with specialist knowledge is needed to manage the process.

2.3 Conventional Wastewater Treatment System

Conventional wastewater treatment systems use various types of mechanical equipment to supply air to aerobic bacteria that remove organic matter found in wastewater. Aerobic treatment system may be designed to support nitrification and denitrification to remove nitrogen and to remove phosphorus through biological action. Conventional treatment systems are used in large, medium and small-scale applications for domestic and municipal wastewater effluents. Conventional treatment systems that have been used in developing countries include

the activated sludge process and more recent variant, including sequencing batch reactors, extended aeration and oxidation ditch (Scott McNiven, 1996).

The disadvantages of conventional treatment that are most prominent in developing countries include high power consumption, high maintenance requirements and need for close supervision by skilled operators.

2.4 Anaerobic Wastewater treatment process

Anaerobic wastewater treatment is the use of biological processes in the absence of oxygen to stabilize organic materials by conversion to methane (CH_4) and inorganic products, including orthophosphate, carbon dioxide, hydrogen sulfate gas, nitrogen gas and ammonia (NH_3) (McCarty, 1986). Treatment in anaerobic reactor removes the major part of the carbonaceous oxygen demand from raw wastewater, but substantial nitrogenous oxygen demand remains.
Anaerobic technology has been in existence for a very long time. According to McCarty (1981). The anaerobic treatment is attracting more and more the attention of sanitary engineering and decision-makers, it is being used successfully in tropical countries, and there are some encouraging results from subtropical temperate regions. However, with the present state of technological development and basic insight into the process, only a few of the presumed drawbacks remain, while all its principal benefits over conventional aerobic methods are still relevant (Lettinga, 1995, Lettinga *et al*., 1988, Lettinga, 1996).

2.5 Differences between aerobic and anaerobic wastewater treatment processes

That aerobic digestion transforms oxygen-consuming substances in the wastewater into a residual sludge. The resulting sludge contains large amounts of volatile solids mostly in the form of bacterial biomass that require further stabilization. Smaller amounts of oxygen consuming substances and solid material remain in the effluent, but the large amounts of unstable sludge create an additional disposal problem. Anaerobic digestion results in a much smaller amount and relatively more stable sludge than aerobic processes. Methane and other gases are produced, but larger amounts of residual solids and oxygen demand substance remain in the effluent than a typical aerobic effluent. The residual sludge does not require additional treatment because it is

more stable, i.e., it is more thoroughly biodegraded than an aerobic sludge. Anaerobic sludge has better settling properties than an aerobic sludge and is easier to dewater. Where a secondary quality treated effluent is required, additional treatment is needed to remove the residual oxygen demand and suspended solids from the anaerobic enhanced primary treated effluent. The following figure. Figure 2.1 will show the COD balance and energy consumption between Aerobic and Anaerobic Treatment Processes.
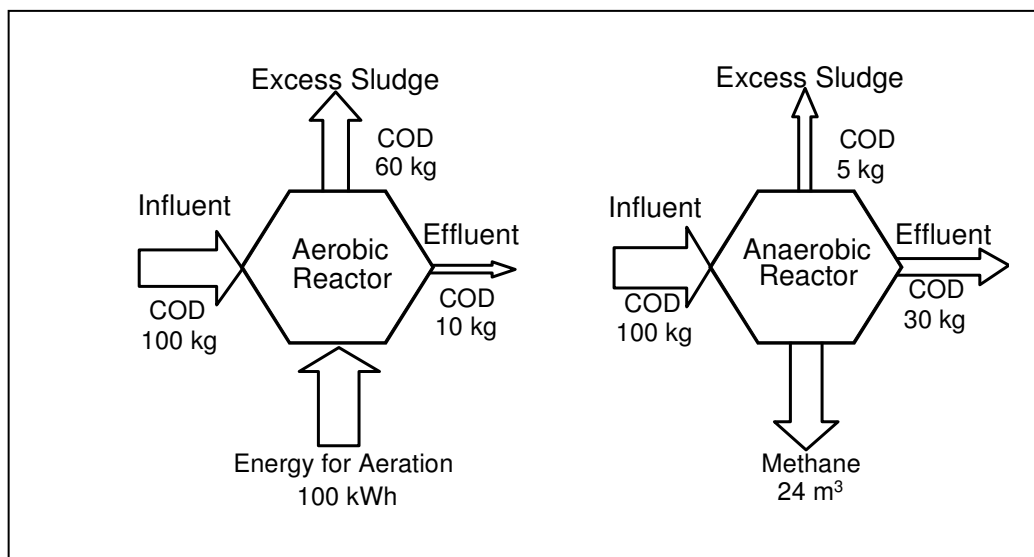
Figure 2.1 COD Balance and energy comparison between Aerobic and Anaerobic treatment Processes. (Jewell, 1994).

2.6 Anaerobic degradation processes in wastewater treatment

Anaerobic degradation of organic matter is a complicated microbial process consisting of several interdependent consecutive and parallel reactions as shown in Figure 2.2. Four different reactions phases can be distinguished in the over all anaerobic conversion process, these are Hydrolysis, Acidogenesis, Acetogenesis and Methanogenesis. In those processes mixed culture of the anaerobic bacteria used remove the organic matter that is present in the wastewater and convert it to by-product in the form of biogas, mostly methane (CH_4) and carbon dioxide (CO_2), in the absent of the oxygen.
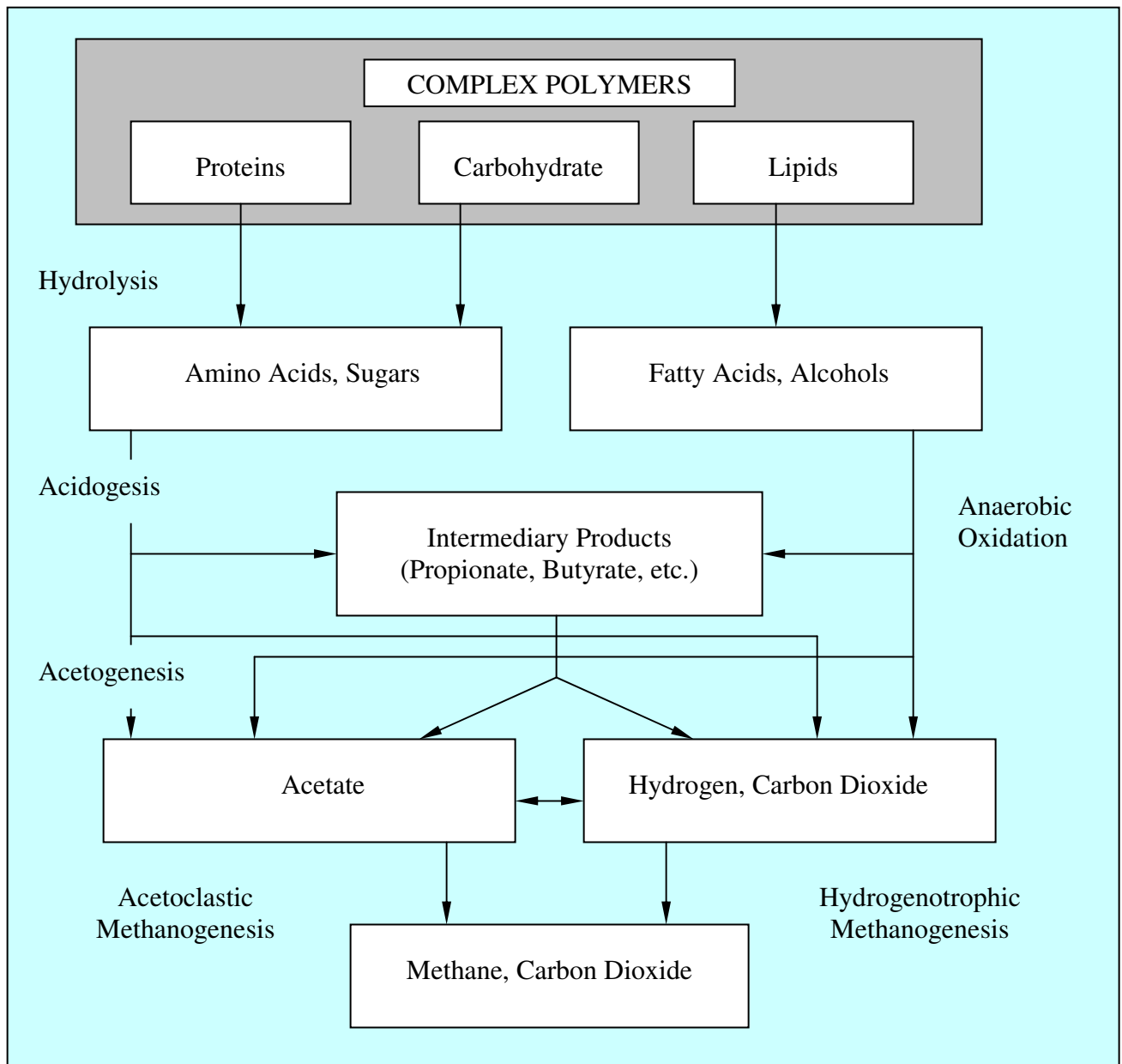
Figure 2.2 Anaerobic digestion reaction and steps of organic polymeric materials (Gujer and Zehnder, 1983).

1. Hydrolysis: it is the first and slowest step in the sequence of anaerobic digestion which is step is the overall reaction rate-limiting step in the case of domestic sewage treatment, especially this becomes notable in low temperature climates (5-20 °C) (Lettinga *et al.,* 1993). This is however not due to a lack of enzymes activity but to the availability and structure of the substrate (Zeeman *et al,.* 1997; Sanders *et al,.* 2001). In this step the complex polymeric materials such as proteins and lipids (fats and grease) are hydrolyses by an enzyme that's

produced by fermentative bacteria to soluble products of a size small enough to allow their transport across its cell membrane such as (log chain fatty acids ,simple sugars and amino acids).

The hydrolysis rate is depending of some factors, which could be summarizing as following (Sanders, 2001): -

A- pH

B- Temperature

C- Availability and structure of the substrate

D- Product inhibition

E- Sludge retention time

F- Available of surface area

The sizing of anaerobic reactors for treating complex substrates like sewage should be based on the hydrolysis step (Mahmoud, 2002). The hydrolysis rate can be described by first order kinetics as shown in equation (Eq.2.1) (Eastman and Ferguson, 1981; Pavlostathis and Giraldo-Gomez, 1991). However, the hydrolysis rate should be measured each time for that specific waste and not adopted from literature data. The hydrolysis rate constant (k_h) had been determined in raw sewage by (Halalsheh, 2002) and sewage sludge by (Mahmoud, 2002).

$$\frac{dX_{degr}}{dt} = -K_h . X_{degr} \qquad \text{………...………………...…………………….. (2.1)}$$

Where:

X_{degr} : biodegradable substrate (kg COD/ m^3)

t : time (days)

k_h : first order hydrolysis rate constant (l/day)

2. Acidogenesis: it is the second step in the anaerobic digestion where the product of hydrolysis (sugars, amino acids, and long-chain fatty acid) is converted to organic acids, ammonia, CO_2 and H_2 by large group fermentative bacteria. The product of this stage depends on the type of bacteria, temperature and pH.

3. Acetogenesis: it is the third step in the anaerobic digestion where the products of the fermentative bacteria (short-chain fatty acid converted into acetate, hydrogen and carbon dioxide by the acetogenic bacteria.

4. Methanogenesis: it is the last and most important step in the sequence of anaerobic digestion where the Methanogens (acetotrophic and hydrogenotrophic) utilize the simple and fermentation products such as (acetate, methanol, carbon dioxide and hydrogen) and convert them to methane and carbon dioxide. This will be according to following reactions.

$$CH_3COOH \xrightarrow{acetoclastic} CH_4 + CO_2 \qquad \text{.........................….……} \quad (2.2)$$

$$CO_2 + 4H_2 \xrightarrow{hydrogenotrophic} CH_4 + 2H_2O \qquad \text{.........................….……} \quad (2.3)$$
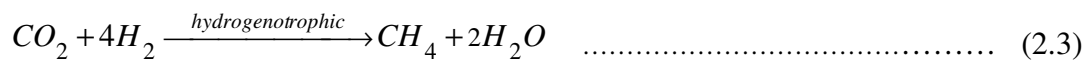
Acetate is the major intermediate in bioconversion of organic matter to methane and CO_2, Where about 70 % of the total methane produced in anaerobic digestion originates from acetate (Guier and Zehner, 1983)

The over all efficiencies for the anaerobic process is limited by the individual efficiencies of each essential group of bacteria in the anaerobic process. So the slowdown at one stage of the anaerobic processes can cause accumulation of biodegradable intermediate products that exit in the last products of the process.

2.7 Environmental factors affecting anaerobic degradation in wastewater treatment

The technical utilization of microbiological process involves a number of environmental factors:-

A- temperature: it is not only influences the metabolic activities of the microbial population but also has a profound effect on such factors as gas-transfer rats and the settling characteristics of biological solids. So the efficiency of the anaerobic process is highly dependent on the temperature (Bogte *et al*., 1993; van Haandel and Lettinga, 1994). At low

temperature, more organic matter will remain undegraded at a given hydraulic retention time HRT (Seghezzo, 2004).

B - pH: the value and stability of pH in the anaerobic process is extremely important because methanogensis only proceeds at a high rate when the pH is maintained in neutral range (6.3-7.8) (Haandel and Lettinga, 1994).

C- Substances contained in wastewater: the anaerobic microorganisms at different rates metabolize different substances. Anaerobic bacteria processing carbohydrates and proteins grow with generation time of less than one day. On the other hand, bacteria degrading fatty acids grow slowly with generation times of about five days, so that the fatty acids degradation is rather slow.

2.8 UASB reactor

The interest on anaerobic system as the main biological step in wastewater treatment was scarce until the development of the UASB reactor at 1970 by Lettinga and his group in the Netherlands (Haandel and Lettinga, 1994). At 1990s the UASB reactor was applied successfully in large scale to the treatment of municipal, mixed industrial and relatively diluted domestic wastewater effluents. The UASB reactor is a high-Rate suspended growth type of reactors in which wastewater is introduce in to the reactor from the bottom where it consist of four zones: the sludge bed, fluidized zone, gas-liquid-solids (GLS) phase separator, and the settling zone. See Figure 2.3.
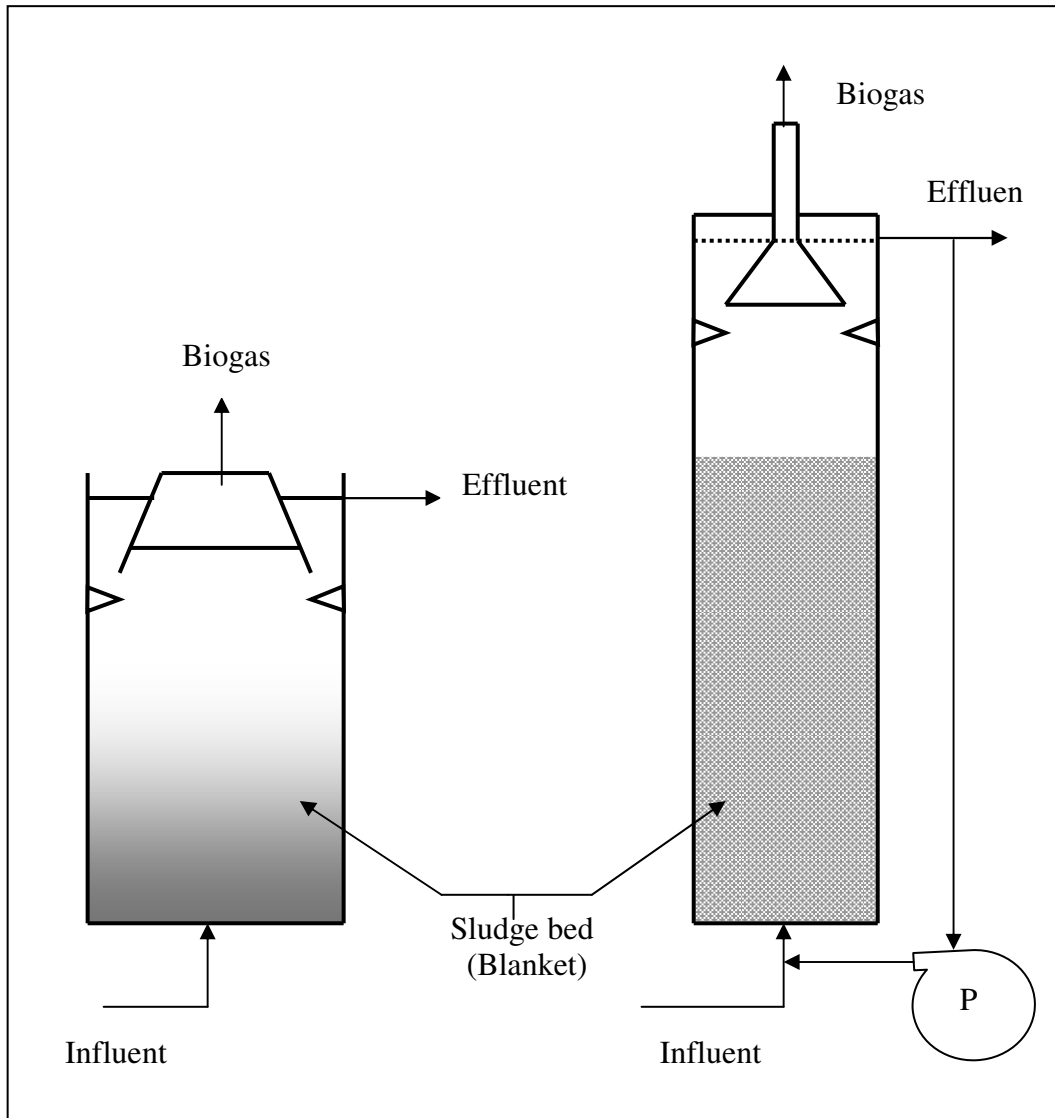
Figure 2.3 Schematic diagram of UASB reactor (left) and EGSB (right) reactors. Modified from Haandel and Lettinga (1994), Wang (1994). P = Pump.

The success of the UASB concept relies on the establishment of a dense sludge bed in the bottom of the reactor in which all biological processes tack place (Seghezzo, 2004). This sludge bed is basically formed by accumulation of incoming suspended solids and bacterial growth in up flow anaerobic systems and under certain conditions. It was observed that the bacteria could naturally aggregate in flocks and granules (Hulshoff Pol *et al*., 1983; Hulshoff Pol, 1989). Where these dense aggregate have good settling properties and are not susceptible to wash out from the system.

Higher organic loads can be applied in the UASB system where retention of active sludge, either granular or flocculent, enable good treatment performance with the help of the good

wastewater biomass contact and mixing that caused by the natural turbulence caused by the influent flow and the biogas production. This feature makes reactor volumes smaller and permits anaerobic treatment at lower temperatures (Mahmoud, 2002).

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).

Anaerobic up flow reactors can be operating at very high up flow liquid velocity, without the loss of biocatalyst from the system under practical reactor conditions (Van lier *et al.,* 2001). Tracer studies demonstrated that internal mixing was not optimal in a pilot-scale UASB reactors treating sewage at temperature range from 4 to 20ºC (de Man *et al.*, 1986). So in order to improve the sludge-wastewater contact a better influent distribution was required for that different feed inlet devices, more feed inlet points per square meter or higher superficial velocity have been proposed as solution. The use of effluent recalculation combined with taller reactors   (high height/ diameter ratio), resulted in the expanded granular sludge bed (EGSB) reactor where a high superficial velocity in the range of 4-10 m/hr is applied (van der Last and Lettinga, 1992), See Figure 2.3. In UASB reactors, the sludge bed behaves more or less as a static bed, but in fully expanded EGSB reactor, it is considered as a completely mixed tank (Seghezzo, 2004).

2.9 Design consideration for UASB reactors

 Wiegant (2001) reported that the design criteria of UASB reactors for domestic wastewater seem still not to have converted to a point that adequate prediction of the effluent quality as a function of design can be made. And also he reported that, most of performance data and results have not yet been published and limited for regions with worm temperature conditions, at middle east countries. With high strength domestic wastewater and seasonal temperature fluctuation, it is very hard to comment on the available operational results because they differ quite widely and therefore, the design criteria of UASB reactor of domestic wastewater treatment in the Middle East are still to be formulated.

In general the main three design consideration for designing UASB are as following, first the volumetric organic load (OLR) secondary the up flow velocity and finally the gas collection system. First the volumetric organic load rate which is the critical factor for the reactor volume. It can be controlled by changing the influent concentration by changing the flow rate, which directly change the hydraulic retention time (HRT) and the up flow velocity (Mahmoud, 2002). The organic loading rate could be expressed according to the following equation.

$$ORL = (Q*COD)/V = COD/HRT \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \quad (2.4)$$

Where:

OLR: organic loading rate $(KgCOD/m^3.d)$.

COD: chemical oxygen demand $(Kg\ COD\ /m^3)$.

Q: flow rate (m^3/d).

V: reactor volume (m^3).

HRT: hydraulic retention time (d).

To obtain satisfactory COD removal efficiency when treating domestic wastewater using conventional UASB reactor a organic loading rate between 0.4-3 kg $COD/m^3.d$. In temperature range of 15ºC to 25ºC and it is preferred to be low for temperature below 15ºC (Kalogo and Verstraete, 1999; Halalsheh, 2002).

Secondary the up flow velocity that is consider as a critical design parameter. Where the wastewater entering the reactor bottom is distributed equally and flows up wards through the bed of sludge. Sufficient upflow velocity are maintained in rectors, in order to facilitate sludge blanket formation offering higher contact area between sludge and wastewater. Upflow velocity in typical UASB reactor rang up to 1-2 m/hr (Droste, 1997). (Haandel and Lettinga, 1994) reported a linear decrease in efficiency with increasing up flow velocity and they recommended that the average daily up flow velocity should not exceed 1 m/hr with a typical value of 0.7 m/hr for treating domestic wastewater water. COD removal efficiency at different up flow velocity has been studied in the past in full – scale reactor but a clear relationship could not be found (Wiegant, 2001).

The upflow velocity can be determined according to the following equation:

V_{up} = (H/ HRT) …………………………………………..…………….. (2.5)

Where

V_{up} : up flow velocity (m/hr).

H: height of reactor (m).

HRT: hydraulic retention time (hr).

Finally the (gas-liquid-solids) separator (GLS). It designed to separate gas, liquid and solids from each other from the effluent of the reactor. Where the gas collected and solids prevented from washout where it is slide back to the sludge blanket zone.

2.10 Effect of solids retention time (SRT)

(Zeeman *et al.,* 2001) reported that removal of suspended solids in sewage occurs by physical processes such as settling, adsorption, and entrapment. Subsequent hydrolysis and methanogenesis of the removed particulate fraction both depend mainly on temperature and solids retention time (SRT), which is the average time that a solid particle stays in the reactor.

Success of the UASB reactors is highly depends on the SRT the key factor for determining the ultimate a mount of hydrolysis, acidification and methanogenesis in a UASB system at certain temperature conditions (Mahmoud *et al.,* 2004). A specific SRT is required for each temperature and each type of sewage. The lower temperature longer the SRT required in one-step UASB reactors to provide enough hydrolysis and methanogensis to degrade entrapped organic particulate fraction (Zeeman *et al.,* 2000).

If the required SRT is known, the needed (HRT) can be calculated with the equation proposed by Zeeman and Lettinga (1999).

SRT = X/X_p ……………………………………………………………..... (2.6)

X : Sludge concentration in the reactor (gCOD/l) ; 1g VSS = 1.4 g COD)

X_p : Sludge production (g COD/ l.d)

$X_p = O * SS * R * (1-X)$ ....……….. $\qquad$ (2.7)

O   : organic loading rate (kg COD/ m^3.d)

SS : (COD_{ss} / COD_{inf})

R   : fraction of COD_{ss} removed

$HRT = C/O$ ....……. $\qquad$ (2.8)

C : COD concentration in influent (g COD/l)

$HRT = (C*SS/X) *R* (1-H)*SRT$ ....…... $\qquad$ (2.9)

SRT: solids retention time (days)

H = fraction of removed solids Hydrolysed

Mahmoud *et al.,* (2003) reported that According to the equation up a minimum HRT of 22 hour is required for application of one-stage UASB reactor in Palestine to overcome the wintertime which is long comparison with normal HRT applied in the tropical countries (6 – 12) hours.

2.11 Effect of Suspended solids on anaerobic treatment

The Suspended solids (SS) content of wastewater is a primary factor that may affect the performance of an anaerobic reactor (Lettinga. *et al,.* 1993; Zeeman and Lettinga, 1999; Kalogo and Verstraete, 1999), (Mahmoud, 2002).

Lettinga and Pol (1991) pointed out that the presence of suspended solid in the wastewater can affect the anaerobic treatment adversely, such as: (1) reducing the specific methanogenic activity of the sludge in the case that the suspended solid is poorly or non-biodegradable and accumulates in the sludge bed, (2) tendency of the formation of scum layers consisting of floating substrate together with entrapped or attached active sludge which may result in washout of active matter and in the production of considerable quantities of poorly stabilized excess scum layer sludge, (3) possibility of slowing down or even counteracting the formation

of granular sludge, and (4) spontaneous and sudden washout of sludge bed if there is a prolonged continuous entrapment of voluminous suspended solid in granular sludge bed.

Mahmoud, 2002 reported that the removal of suspended solids is one of the main objectives of sewage treatment Mahmoud found that the particulate materials   exceeding 0.45 represented the major fraction of domestic sewage about (65-71)% of the total COD.

UASB reactors are very efficient at retaining SS from the swage, especially in tropical region (Haandel and Lettinga, 1994; Cavalcaunti, 2003). SS removal at in UASB reactors depends on the type of sewage and the combined effect of the sludge bed height and liquid up flow velocity in the reactor. Treating complex wastewater containing high a mount of SS is usually limited by accumulation of these compounds in the sludge bed, especially at temperature lower than 18 ℃ due to very slow hydrolysis, forcing a reduction of loading rate (Mahmoud, 2002). The effect of sludge bed height and up flow velocity on the removal of SS needs to be assessed to optimize the design and performance of UASB reactors for the treatment of settled sewage at law temperature (Seghezzo, 2004).

2.12 The UASB-septic tank system

 The design of the UASB-septic tank is almost as simple as that conventional septic tanks but the treatment efficiency is much higher. The septic tank is the most known and commonly applied method for on-site treatment of sewage.

 (Bogte *et al.,* 1993) and (Lettinga *et al.*, 1993) researched the use of UASB-septic tank for on-site treatment of black water and domestic sewage. UASB-septic tank differed from the conventional septic tank system by the up flow mode in which the system operated resulting in both improved physical removal of suspended solids and improved biological conversion of dissolved components (Elmitwalli *et al.,* 2003). It is differs from the traditional UASB system by that the UASB-septic tank system is also designed for accumulation and stabilization of sludge.

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 0.86 m^3 also tested in Bandung (Indonesia) by Lettinga *et al.* (1993). See Table 2.1.

The UASB-septic tank is designed with long HRT implies a low hydraulic load rate and long sludge retention time as typical conventional septic tank (Mgana, 2003). The sludge from this type of reactors discharges once every (1-4) year (Zeaman *et al.,* 2000). On the other hand the sludge of the conventional UASB must be discharged frequently (once or twice a week) and this due to the short HRT which implies high hydraulic loading rate resulting in minimizing the sludge hold-up period in the reactor (Kalogo and verstraete, 1999). Because of the last facts the discharged sludge from the conventional UASB need to be stabilized, but the sludge from the UASB septic tank reactor can be used for solid conditioning and fertilization directly.

Table2.1 Summary of applications of on-site pilot scale UASB-septic reactors to sewage treatment under different conditions

Place	V (m³)	T (ºC)	Influent Type	Influent concentration (mg/L)			HRT (h)	Removal efficiency (%)			Gas production (l/d)	Period (Months)	Reference
				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; GW=Grey wastewater; BW= Black wastewater

2.13 Application of UASB and UASB-septic tank at law temperature and tropical Country climate.

There are clear indications that UASB reactor can cope with sewage temperature a round 18ºC and lower for prolonged periods without substantial reduction in their treatment efficiency (Haskoning, 1996).

The application of UASB reactor to sewage treatment under low temperature conditions had been studied in the Netherlands since 1976 [Lettinga *et al.,* 1981; Grin *et al.,* 1988; de Man *et al.,* 1986; Van velsen and Wildschut, 1988], where they concluded that the UASB concept was a simple, compact, and inexpensive technology for sewage treatment, even at relatively law temperature. Some of their results as summarized in Table 2.2. (Fernandes *et al.,* 1985) confirmed their results using two small UASB reactors to treat settled domestic sewage. (de Man *et al.,* 1986) concluded that anaerobic treatment of raw domestic sewage (COD = 500-700 mg/l) can be accomplished at (12-18) ºC applying HRTs of 7-12 h with total COD and BOD removal efficiencies of 40-60 % and 50-70 % respectively where this performance was not considered attractive to treat sewage under Dutch conditions. But it considered a real challenge for researchers in the field of environmental technology. However, the investigation, which had been carried out, represented a commendable move towards the understanding of the involved complex processes and development of a series of novel technologies (Mahmoud *et al,* 2002).

The resulted of several bench scale and pilot scale systems operated at low temperature have open a new perspectives Table 2.2, but no full-scale application has so far been realized (Zeeman and Lettinga, 1999; Lettinga *et al.,* 2001). Nevertheless, (Mahmoud, 2002) reported that experience with the application of one stage UASB reactor system and low temperature and high influent suspended solids concentration as found in many Middle East countries (Lier and Lettinga, 1999) is still be developed.

Specific alterations in process layout reactor technology or operational techniques are investigated to treat domestic wastewater under Middle East conditions some examples of these technologies are described below.

Elmitwalli (2000) studied a two-step UASB system consisting of an anaerobic filter (AF) plus a (AH) reactor (a UASB reactor with a filter on top) at sewage temperature of 13 ºC. Removal of suspended and dissolved COD was high and this because reactors were fed with settled sewage. The sludge that produced is poorly stabilized so, further stabilization process is still needed. See Table 2.2.

Halalsheh (2002) studied two-stage UASB reactors in Jordan, in which the first one operating at HRTs (8-10) h and the second one on HRTs (5-6) h. Both of the two reactors fed with strong raw sewage and controlled temperature to be 18 ºC at winter and 25 ºC in summer. Halalsheh (2002) reported that in the two–step UASB system most of the COD was removed in the first stage and the average results obtained during winter time with the first stage of the two-stage system and the one stage reactors were the same with no significant effect of temperature. See Table 2.2.

Mahmoud (2002). Studied the use of UASB reactors to treat domestic wastewater at sewage temperature of 15ºC (The average sewage temperature in Palestine during winter time) with HRT of 6h and reactor volume of 0.14m^3 see Table 2.2. Mahmoud (2002) reported that digesting the excess sludge at 35 ºC in anaerobic digester and then recirculating the sludge back into the reactor improved the performance of single-stage UASB reactor.

In Palestine, which considered one of the Middle East countries mostly of relatively low temperature during the wintertime that lasts for three months, one can expect limited performance and some problems such as poor granular sludge formation, accumulation and slow methanogenic activity and low biogas production (Kalogo and Verstraete, 1999). Some researches had been take place In Palestine to study the performance of the UASB, such as studies done by Al-Juaidy (2002), Ali (2001) and Al-Shayah 2005. See Table 2.4. Al-Shayah (2005) studied the performance of two UASB reactors of two different HRTs (2 and 4 days) in

Palestine, both of the reactors fed with domestic wastewater. The two reactors were operated for six months at summer ambient temperature fluctuates between (15 and 34) ºC with an average value of 24.2 ºC. The performances of those two reactors are tabulated at Table 2.2. The performance of the two reactors used by Al-Shayah continued to be investigated in this research at the same conditions but at winter ambient in Palestine.

Table 2.2 Summary of results for anaerobic domestic wastewater treatment in pilot and full scale UASB and UASB-septic tank reactors at law temperature and tropical Country climate.

Place	Influent Type	T (ºC)	V m³	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)

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

Chapter 3

Materials and Methods

3.1 Location

The location of this study is chosen to be in "Ramallah /Al-Bireh district area, which is located at the central part in the West Bank and considered as one of the most important administrative centers in Palestine. Ramallah and Al-Bireh are the main urban centers for commerce and services with small and medium scale industries. According to the last census carried out by the Palestinian Central Bureau of Statistics (PCBS, 1997), the population of Ramallah and Al-Bireh are 18,017 and 27,972 inhabitants, respectively. According to the records of JWU (2000; the water supplying company), the average billed water consumption for the two cities Ramallah and Al-Bireh are 137 l/c.d, where sewage from Ramallah and Al-Bireh is collected in sewer systems, serving about 75% of the population. For this research sewage was taken from Al-Bireh treatment plant from a pilot plant, which was build there (Mahmoud *et al.,* 2003)

3.2 Experimental set-up

Two UASB-septic tank reactors were installed at Al-Bireh treatment plant was the specification for each of them is summarized at the next table. See Photo 1 at Appendix 3.

Table 3.1 Specification of the tow UASB Reactors and the Operational condition used during the first six-month by Al-Shayah.

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

The reactors were made of 3 mm thick galvanic steel sheets. Each reactor was provided with nine sampling port along it's height   with separation distance 25 cm from each other for sludge sampling. Polyvinyl chloride (PVC) pipes with internal diameter of 1.15 in where used for

influent and effluent distribution. The gas, liquid and solids (GLS) were installed at the top of each reactor. The treated effluent flowing out of the reactor was collected in a pocket where the sample had been taken.

The influent was distributed in each reactor through a one-inlet pipe with 4 outlets located about 5 cm from the bottom of the reactor. The methane gas that is available at the biogas produced by the reactor was continuously measured with a wet-type gas meter this after passing the biogas through a 16% NaOH solution available at tightly, closed, glass cylinder in order to separate the methane gas from CO_2 gas which will be dissolved at the NaOH solution, the other gases that's available in the biogas such as hydrogen sulfide was neglected. (The detail for the gas collection system where presented at the Appendix3 by Photo 2.)

3.3 Reactors sewage feeding

A preliminary treated provided by screens and grit removal chamber where done for the raw sewage from the main sewage trunk pipe of Al-Bireh WWTP. Before it had been pumped every 5 minutes to feed both of the UASB-septic tank reactors at the pilot scale. An automatic controlled submersible pump, used to pump the wastewater from the girt chamber to a holding plastic tank (200 L) from which the reactors fed and the influent was sampling also it used to reduce the pumping distance to the reactors. The holding tank was worked also as a balance tank where the total outlet wastewater flow form the hold tank to the (two reactor and the outlet valve) equal the flow rate inter to it by the submersible pump which had been pumped every 5 minutes.

The sewage was continuously pumped from the holding tank to the reactors with peristaltic pumps to maintain constant discharge of influent for each reactors and this by using MASTERFLEX® L/S 7520-57 series (flow rate range: 4.8-480 ml/minute) equipped with MasterFlex Tygon L\S® 36 tubing. Flow rate were checked almost continuously and adjusted with (1 to 10)-turn speed control (1-100 rpm, 230v drive). The description of sewage feeding operational system could be presented by the flow diagram FigureA1.1 at the Appendix.

3.4 Pilot plants operation and start-up

Al-Shayah started up the UASB-septic tank reactors in April 2004. The two reactors were operated in parallel at ambient temperature conditions with temperature variation between 15ºC and 34ºC. The two reactors had been designed to be operated at HRTs of 2 and 4 days for R1 and R2, respectively for six-month period. A detailed description of the operation conditions during the first six month that's has been operated by Al-Shayah at the whole experiment was presented in Table 3.1.

This research continued to use the same Operational conditions as Al-Shayah at ambient temperature conditions with temperature variation between 2ºC and 27ºC at winter season which is the critical period at the operation of this pilot plant and this will stared at September 2004 to April 2005. At the end of this research the two reactors will be examined for one year.

3.5 Sampling

Grab sample of raw sewage sample after the preliminary treatment units, R1 and R2 effluent were taken two to three times a week (1 L for each). Sample was kept at 4ºC until they were analyzed. An alcohol thermometer at the Al-Bireh treatment plant measured sewage and ambient temperature daily. The pH measured for the samples by using EC pH meter (HACH). Gas production was monitoring daily and recorded.

Samples were analyzed for CODtot, COD_{sus}, COD_{col}, TSS, VSS, NH_4^+, Nkj, total PO_4, ortho PO_4^{-3} and SO_4^{-2} was all according to standard methods (APHA, 1995). Moreover, sludge sample were analyzed for TS, VS and stability. Biodegradadability test was also done for the effluent samples from the reactors.

3.6 Analytical Methods

The analytical methods for wastewater parameters in General could be distributed in tree field's chemical analysis, physical analysis and Microbiological analysis. In this research only the chemical and the physical analysis were analyzed.

3.6.1 Chemical analysis

The chemical parameters that had been analyzed in this research could be summarized as following: Chemical Oxygen Demand (COD), Biological Oxygen Demand (BOD), Volatile Fatty Acid (VFA), Kjeldhal Nitrogen (NKj-N), Ammonia (NH_4^+-N), Sulfate (SO_4^{-2}), Total Phosphorous (Total P) and Ortho-Phosphate (PO_4^{-3}).

3.6.1.1. Chemical Oxygen Demand (COD)

Samples were used for measuring total COD (COD_{tot}), 4.4 µm folded paper-filtered (Schleicher and Schuell 5951/2, Germany) samples for particulate COD_p and 0.45µm membrane - filtered (Schleicher and Schuell ME 25, Germany) samples for dissolved COD (COD_{dis}). The suspended COD (COD_{ss}) and colloidal COD (COD_{col}) were calculated as the difference between COD_{tot} and COD_p and the difference between COD_p and COD, $_{dis}$ respectively. Where COD test done by using reflux method (acid destruction at 150 C^0 for 120 minutes where the absorbance was then measure by spectrophotometer at 600 nm wavelength according to Standard Methods (APHA, 1995).

3.6.1.2. Biological Oxygen Demand (BOD)

A raw wastewater samples from the influent and effluent of the two reactors were used to determine DOD_5 at 20ºC. This test is done according to Standard Methods (APHA, 1995).

3.6.1.3. Volatile Fatty Acid (VFA)

The volatile fatty acid analysis was carried out using titrimetric method according to (Kapp, 1984; Kapp, 1992) (Quoted by Buchauer, 1998). This method dose not requires high investment in technical equipment like Gas Chromatograph (GC). Where the analysis as it had reported by Buchauer (1998) listed as following.

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

Law 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).

$$\text{VFA} = (131340 * N_2) * (VA_{(pH(5-4))} / VS) - (3.08 * Alk_{meas}) - 25 \quad \text{……….…….} \quad (3.1)$$

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

Where: -

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

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

$VA_{(pH \text{ (Initial -5)})}$: measured volume of acid (ml) required to 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 0.02 N.

3.6.1.4. Kjeldhal Nitrogen (NKj-N)

The Kjeldhal method (digestion, distillation and titration) according to Standard Methods (APHA, 1995) was used to determine the amount of the organic and ammonium nitrogen.

3.6.1.5. Ammonia (NH_4^+-N)

Nesslerization method using spectrophotometer at absorbance of 425 nm wavelength used to determine the Amount of Ammonia (NH_4-N) from paper-filtered samples and this regarding to Standard Methods (APHA, 1995).

3.6.1.6. Sulfate (SO_4^{-2})

Spectrophotometer at absorbance at 420 nm wavelengths was used to measure the amount of sulfate from paper-filtered sample and this was regarding to Standard Methods (APHA, 1995).

3.6.1.7. Total Phosphorous (Total P) and Ortho-Phosphate (PO_4^{-3})

Spectrophotometer at absorbance at 880 nm wavelengths was used to determine the amount of total phosphorous, from digested raw wastewater sample and ortho-phosphate from, membrane-filtered sample this regarding to Standard Methods (APHA, 1995).

3.6.2 Physical analysis

The Physical parameters that had been analyzed in this research could be summarized as following: Total and suspended Solids (TS, TSS), Volatile and Suspended Solids (VS, VSS), Sludge Volume Index, pH, Temperature, Color, Atmospheric pressure.

3.6.2.1. Total and suspended Solids (TS, TSS)

Total and suspended solids were measured related to Standard Methods (APHA, 1995) by oven drying at 105 ºC this by using paper of glass microfiber filters (GF/C 125 mm ϕ, CATNO 1822 122 Whatman®)

3.6.2.2. Volatile and Suspended Solids (VS, VSS)

Volatile and suspended Solids were measured related to Standard Methods (APHA, 1995) by oven burning at 550 ºC.

3.6.2.3. Sludge Volume Index (SVI)

SVI was measured according to Standard Methods (APHA, 1995) by using Imhoff Cone.

3.6.2.4. pH

pH was measured for total samples using EC pH meter (HACH).

3.6.2.5. Temperature

Wastewater and ambient temperature were measured in Al-Bireh treatment plant by using alcohol thermometer.

3.6.2.6. Color

Color was determined by visual appearance.

3.6.2.7. Atmospheric pressure

The atmospheric pressure was measured in site by barometer pocket device.

3.7 Batch experiments

In this research two types of batch experiment had been taken place: first one is the stability test which represent the maximum percentage of COD converted to CH_4 of the digested sludge and the second one is the biodegradability test which was used to determine the percentage of the chemical oxygen demand (COD) presented in organic sample that transformed to methane under anaerobic conditions. The tests are carried out in batch reactors, sealed serum bottles, of 500 ml with a headspace volume of 70 ml incubated at 30 ºC 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). For more detail see Appendix Figure A2.1. Both of the two experiments standard procedure is still lacking and comparison of results reported in literature can be unclear.

3.7.1 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. Anaerobic digestion of sludge is one of the technologies available for 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 gCOD-CH_4/gVSS or gCOD-CH_4 /g COD.

Sludge stability was measured two times in duplicate 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 1.5 g COD-sludge/l, tap water and a mineral solution of macronutrients, trace elements and bicarbonate buffer. The stability batches incubate at 30 ºC. The total sludge stability was calculated as the amount of methane produced 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.

3.7.2 Biodegradability

The anaerobic biodegradability is the anaerobic analogous of the biological oxygen demand (BOD) which in turn, represents the aerobic biodegradability of a sample.

The biodegradability of raw wastewater sample and effluent from R1 and R2 were measured twice in duplicate during the whole experiment period were each experiment lasted for 120 day. The experiment set-up and procedure for determination of anaerobic sludge stability and biodegradability are the same. However each bottle of the biodegradability test was filled with about 450 ml wastewater and a mineral solution of macronutrients, trace elements, and bicarbonate buffer. The biodegradability batches also incubate at 30 ℃. Total COD was measured at the beginning and at the end of the test period. The experimental procedures for determination of anaerobic biodegradability and the composition of macronutrients and trace elements used in experiment are presented in details in Appendix 2.

3.8 Calculations

3.8.1 Removal efficiency

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

Removal Efficiency % = [(Influent - Effluent)*100%] / Influent   ....   (3.3)

Where:

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

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

3.8.2 Hydrolysis, Acidification and Methanogenesis

The Hydrolysis, Acidification and Methanogenesis percentage during the anaerobic digestion process can be calculated regarding to the following equations.

$$H(\%) = \left(\frac{COD_{CH4} + COD_{dis,eff} - COD_{dis,inf}}{COD_{tot,inf} - COD_{des,inf}} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. \quad (3.4)$$

$$A(\%) = \left(\frac{COD_{CH4} + COD_{VFA,eff} - COD_{VFA,inf}}{COD_{tot,inf} - COD_{VFA,inf}} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. \quad (3.5)$$

$$M(\%) = \left(\frac{COD_{CH4}}{COD_{tot,inf}} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6)$$

Where:

H:- Hydrolysis (%);

A:- Acidification (%);

M:- Methanogenesis (%);

COD_{CH4} :- amount of produced $CH_{4(\text{dissolved form + gas form})}$ (mg CH_4 as COD/l);

$COD_{dis,eff}$:- amount of dissolve COD in effluent (mg COD/l);

$COD_{dis,inf}$:- amount of dissolve COD in influent (mg COD/l);

$COD_{VFA,eff}$: - amount of VFA in effluent (mg VFA as COD/l);

$COD_{VFA,inf}$: - amount of VFA in influent (mg VFA as COD/l);

$COD_{tot,inf}$: - amount of total COD in influent (mg VFA as COD/l);

* $CH_{4(\text{dissolved form})}$ depend on the solubility of CH_4 in wastewater .

3.8.3 COD-mass balance

$$COD_{inf} = COD_{accumulated} + COD_{CH4} + COD_{effluent} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.7)$$

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 $CH_{4(\text{dissolved form + gas form})}$ (mg CH_4 as COD/l);

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

3.8.4 Stability and Biodegradability calculations

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

Biodegradability (%) = $100(CH_{4 \ (as \ COD)} / COD_{tot, \ t= 0 \ days})$ ……….….… (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)

$COD_{CH4} = n * 64 * 1000$(mg CH_4 as COD/l) ………………….......... (3.10)

Where:

V = Volume occupied by the gas (L);

n = moles of CH_4 (mole), (1 mole CH_4 = 64g COD);

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

P= absolute pressure (atm), 0.945 atm at Birzeit University;

T= temperature (K), (273.15 + ^{o}C);

Moreover, the following equation could be used also to calculate the biodegradability.

Biodegradability (%) = $100 \ [(COD_{tot, \ t= 0 \ days} - COD_{tot, \ t= \ days}) / COD_{tot, \ t= 0 \ days}]$ ………. (3.11)

3.9 Statistical analysis of data

The variation range and the arithmetic averages and the standard deviations, of different data had been calculated this was done by Microsoft Excel 2003. The SPSS software release 11.0.0 SPSS® Inc.,(2001) was used to compare between the removal performance of the reactors R1 and R2 by the T-test. The series of orders used was as following for using SPSS software:-

(1) "Analyze"➔"Correlate" ➔ "Bivariate"➔ "Pearson correlation coefficient"

➔ Two- tailed tests of significance were assigned.

(2) "Compare Means" ➔ "Paired samples T-tests" ➔ type confidence interval 95%

(3) The output data was read from the output-SPSS viewer Paired Samples Test table, which ended with the Significance (2- tailed) value (ρ).

(4) 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

During the period of the research, that lasted for six months from the first of October 2004 until the end of March 2005. The characteristics of the raw sewage at Al-Bireh wastewater treatment plant had been tested and the results could be shown in Table 4.1.

Table 4.1. Characteristics of the influent sewage at Al-Bireh WWTP-Palestine

Parameter	# samples	Range	Average	STD
COD_{tot}	41	485-1527	905	254
COD_{ss}	41	105-909	396	164
COD_{co}	41	73-262	135	42.8
COD_{dis}	41	140-664	350	124
VFA as COD	41	5-261	99	56
BOD_5	13	300-690	502	133
COD/BOD_5	13	1.3-2.6	1.97	0.41
Nkj as N	14	54-85	70	10
NH_4^+ as N	19	6.7-65.1	39	18
Total PO_4 as P	11	4.5-14	10	3.5
PO_4^{-3} as P	11	3-15	8.4	4
SO_4^{-2} as SO_4^{-2}	15	55-141	95	24
COD_{tot}/ SO_4^{-2}	15	6-15	9.2	2.7
TSS	13	153-581	371	141
VSS	13	122-513	313	128
pH	39	6.9-8	7.6	0.28
T_{ww}	141	12-25	17.3	4.3
T_{amb}	141	2-27	14.7	5.5

All parameters are in mg/l except: [wastewater temperature (T_{ww}) and ambient temperature (T_{amb})] in (ºC); pH no units.

The domestic sewage classified as (medium strength) and this regarding to the Metcalf and Eddy (1991) and EPA (1999). Characteristics of the domestic wastewater at Al-Bireh wastewater treatment plant drop from high strength as classified at the summer period by Al-Shayah (2005) to medium strength wastewater that was a result of the dilution, which happened to the sewage by the rain during the period of this research. The variation of the rain rate and so the dilution factor caused the large variation in the values of COD, BOD_5, and TSS as shown from the standard divination in Table 4.1.

In returned to Table 4.1 the averages of COD_{tot} at the influent of Al-Bireh WWTP was 905(253.5) mg/l, the average concentration values of the COD_{ss}, COD_{dis} and the COD_{col} were 396(163.8) mg/l and 350(124.2) mg/l and 135(42.2) mg/l, respectively.

Related to the results one can see that the COD_{ss} was the raw sewage that represent a high fraction of the COD total about 43.7% which was less than the value that was reported by Mahmoud *et al.,* (2003) which was 58%, moreover it was less than the value reported during the summer period by Al-Shayah, (2005) about 53.8%. The percentages of the other fraction of the COD_{tot} were as follow 14.9% and 38.6%for COD_{col} and COD_{dis}, respectively.

The average VFA of wastewater entered to the treatment plant at Al-Bireh treatment plant as shown in Table 4.1 was about 99(55.8) as an average value, this value was less than the value that had been reported by Mohmoud (2003) and Al-Shayah (2005) 150 mg COD /l and 160 mg COD/l, respectively.

Table 4.2. Show the ratios between the (VFA/COD $_{tot}$) and the hydrolysis percentage, which represented as (COD_{dis}/COD_{tot}), the acidification percentage that could be represented as (VFA/COD_{dis}) and ratio of the (VSS/TSS) and COD_{ss}/VSS).

Table 4.2. Percentage of hydrolysis, acidification and protein of total COD and acidification of dissolved COD and VSS/TSS and COD_{sus}/VSS ratio for the influent of Al-Bireh WWTP and Abu-Nusier WWTP-Jordan

	(VFA/COD_{tot})	(VFA/COD_{dis})	(COD_{dis}/COD_{tot})	(VSS/TSS)	(COD_{sus}/VSS)
Palestine Al-Bireh[1]	10	36	28	84	1.49
Palestine Al-Bireh[2]	12.7	41.1	30.9	83	1.25
Palestine Al-Bireh[3]	10.9	28.28	38.7	84.4	1.27
Jordan Amman[4]	9.4	40	23.5	72	3.21

(1)Mahmoud *et al.,* 2003; (2) Al-Shayah (2005); (3) this research; (4) Halalsheh (2002)

The average TSS and VSS for the influent was tabulated at Table 4.1, about 371(141) mg/l and 313(128) mg/l, respectively. The ratio of the (VSS/TSS) was 84.4% as represented at Table 4.2 this value was very close to the results achieved by Mahmoud *et al.,* 2003 and Al-shayah (2005). This value was higher than the value reported by Halalsheh (2003) for domestic sewage treatment this might be due to the difference in people habits.

During the period of the study, the sewage temperature varies regarding to the variation at the ambient temperature this variation fluctuate from (12-25) ºC with average value of 17.34 ºC. Which increased by 2.5 ºC from the average ambient temperature 14.72 ºC that range from (2-27) ºC. In this research the ambient temperature considered as the temperature of the two reactors R1 and R2 which is placed at ambient and the effluent temperature was very closed in average to the ambient temperatures.

4.2 Performance of the two UASB-septic tank reactors

The performance of the two UASB-septic tanks R1 with HRT of 2 day and R2 with HRT of 4 days which had been studied during the research period are summaries in Tables 4.4 which explains the specification in form of numbers, and percentage.

4.2.1 COD Removal efficiency

The whole results of the COD removal efficiency for the two reactors R1 and R2 are tabulated in Table 4.3 and represented by figures 4.1, 4.2, (4.3 and 4.4), 4.5 and 4.6 for COD $_{tot}$, COD$_{sus}$, COD$_{col}$, and COD$_{dis}$, respectively. During the period of the research the results of R1 with HRT of 2 days shows that the average removal efficiency for COD$_{tot}$, COD$_{sus}$, COD$_{col}$, COD$_{dis}$, were 51%(9), 83%(10), 20%(32), 24%(15), respectively. The results also show for R2 with HRT of 4 days that the average removal efficiency for COD$_{tot}$, COD$_{sus}$, COD$_{col}$, COD$_{dis}$, were 54% (11), 87 % (8), 10% (37), 28% (18), respectively . In general one can see that R2 was more efficient in removing COD total and all fraction except COD$_{col}$ fraction. Regarding to the statistical analysis the difference in removal efficiency between the two reactors were statically significant in just only for COD$_{sus}$ (ρ<0.05), as will be explained later.

4.2.1.1 COD $_{tot}$

The average removal efficiency and the average effluent concentration of COD $_{tot}$ were shown in Table 4.3 for both of the two reactors. The average effluent concentrations of COD $_{tot}$ for R1 and R2 were 433 (109) mg/l and 408 (109) mg/l, respectively with average removal efficiency of 51% (9) and 54% (11) for R1 and R2, respectively. The results from statistical analysis show that the differences of COD$_{tot}$ removal efficiency found between the two reactors were not statistically significant (ρ>0.05). Figure 4.1 shows the variation of the effluent COD $_{tot}$ concentration of R1 and R2 and the removal rate of COD $_{tot}$ to the influent concentration. From the results above one can see that R2 is slightly more efficient in removing COD$_{tot}$. The variations in the effluent at both reactors were large which were proved in the standard divination of effluent concentration large variation in the effluent concentration caused by the

large variation in the influent concentration that was affected by the dilution factor caused by rainwater during the research period.
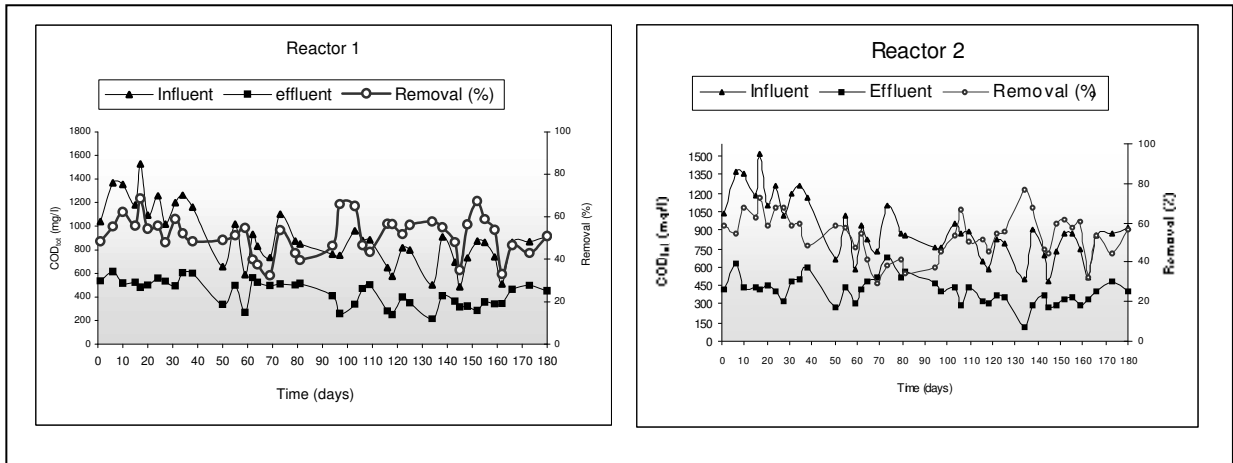
Figure 4.1. COD$_{tot}$ influent and effluent concentrations and removal efficiencies for R1 (left) and R2 (right).

The variety in the efficiencies between two reactors can be explained to a great extend by the difference in hydraulic conditions, reflecting physical phenomena, Rather than the change in biological characteristics of reactors. If the results which were obtained in this research during the winter time, compared with the results that had been reported by Al-Shayah (2005), during the summer period of the same year, one can see that the efficiency for removing COD$_{tot}$ for R2 continued to be more efficient than R1 where it was 54% and 58% for R1 and R2, respectively as Al-Shayah (2005) reported.

In this research the efficiency in removing COD$_{tot}$ had been decreased in both of the reactors with about (3-4) % and this may be regarding to the decrease in the temperature during the winter, where the anaerobic process known for its high affection of the change in temperature regarding to the change in the biological growth, which depend for a great deal on the temperature degree. The removal efficiency for R1 and R2 for removing COD$_{tot}$ were in the range of results obtained with well functioning UASB reactors treating raw domestic sewage in sub-tropical regions, as reported by Halalsheh (2002), COD$_{tot}$ removal efficiency's of 58%, (50-62)%, respectively for pilot and full scale UASB reactors treating raw domestic sewage at 24°C in Jordan which is from a wastewater composition point of view, very close to the Palestinian wastewater characteristics as Al-Shayah (2005) reported. On the same context, Bogte *et al.,* (1993) reached to 33 % removal efficiency of COD$_{tot}$ when raw domestic wastewater was

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tested for 28 months at 13.8C^0 in on-site UASB-septic tank reactor with HRT of 44.2 hours and this in Noordwuk.

4.2.1.2 COD$_{sus}$

The results in this research as shown in Table 4.3 recorded a high average removal efficiency for COD$_{sus}$ in both of the reactors 83 % (10) and 87 % (8) for R1 and R2, respectively with average effluent concentration of COD$_{sus}$ 62 (34) mg/l and 45 (30) mg/l for R1 and R2, respectively.
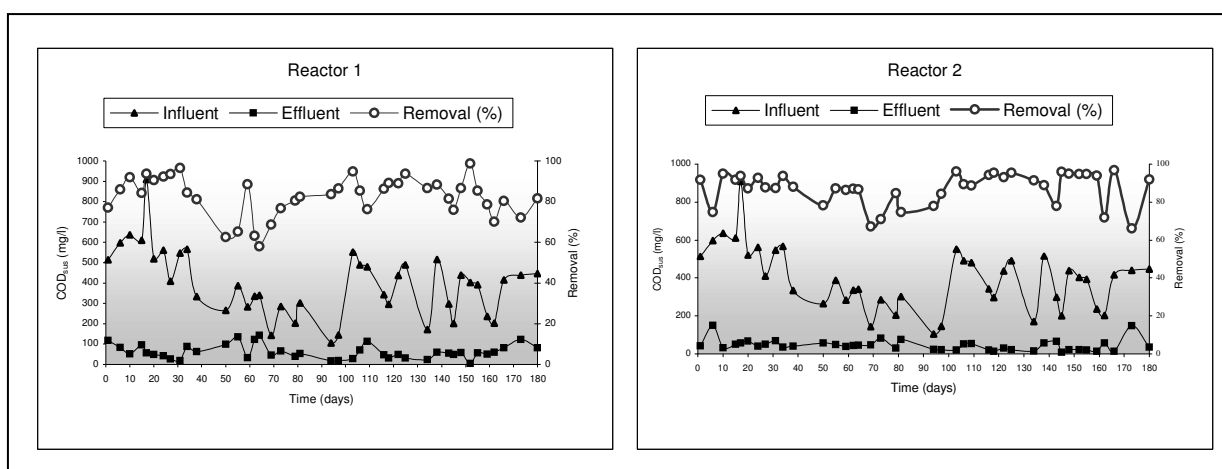
Figure 4.2. COD$_{sus}$ influent and effluent concentrations and removal efficiencies for R1 (left) and R2 (right).

From the tabulated results Table 4.3 and from Figure 4.2 one can see that the effluent concentration of COD$_{ss}$ was very stable if it is compared to the influent COD$_{sus}$ at Figure 4.3 also this could be proved by the standard deviations and this was seen in both of the tow reactors R1 and R2.

If the results obtained in this research are compared to the results that had been recorded by Al-Shayah (2005) at summer period one can see that the COD$_{sus}$ average removal efficiency decreased with the same value 2% at both of the reactors. R2 still has the higher removal efficiency for removing COD$_{sus}$ than R1, also the results from the statistical analysis prove this results where the removal efficiency of COD$_{sus}$, is statistically significant ($\rho<0.05$). These results were expected related to Mahmoud (2002) who pointed out that the effect of HRT could prove as a result of its direct relation to V_{up} and also to solids contact time in the reactor and so the possibility of solids to be entrapped in sludge bed. In this research and during the research

done by Al-Shayah (2005), the previous observation was clearly observed, where R1 was operated at V_{up} faster than R2 0.05 m/h and 0.025 m/h, respectively.

The reduction in the efficiency at the same rate should be related to the change in temperature where Mahmoud (2002) reported that the increase of the V_{up} reduced the removal efficiency of solids by increasing the hydraulic shearing force and solids particles so that the solids particle will move out the reactor. The decrease in the temperature caused an increases in the viscosity of the wastewater and so the hydraulic shearing force on solids particle so solids particles will move out the effluent and this will increase the COD_{sus} concentration at the effluent and so the removal efficiency will decrease.

4.2.1.3 COD_{col}

In this research and as shown in Table 4.3 and Figure 4.3and 4.4 one can see that both of the reactors R1 and R2 are not sufficient for removing COD_{col} from the influent during the research durations. The average removal efficiency was 20% (32) and 10% (37) for R1 and R2, respectively. The difference of COD_{col} removal efficiency between the two reactors were found not statistically significant ($\rho > 0.05$). In addition to the low removal rate one can see from the results that there was a wide range in the removal rate as shown from the standard deviations. Not only this also negative removal efficiency had been observed, where this means that the effluent concentration of COD_{col} some times exceeds the influent concentration. The same results had also observed by Al-Shayah (2005) and Elmitwalli (2002). Where Elmitwalli (2002) justify the results as the increase in the COD_{col} was generated from the COD_{sus} that had been digested. The temperature variations may affect the removal efficiency of the COD_{col} where from Figure 4.3 and 4.4 at the beginning of this research and during the first two month one can see an offset range on the graphs between the influent concentration and effluent concentrations. This offset started to disappear along with on going period of the research and the temperature.
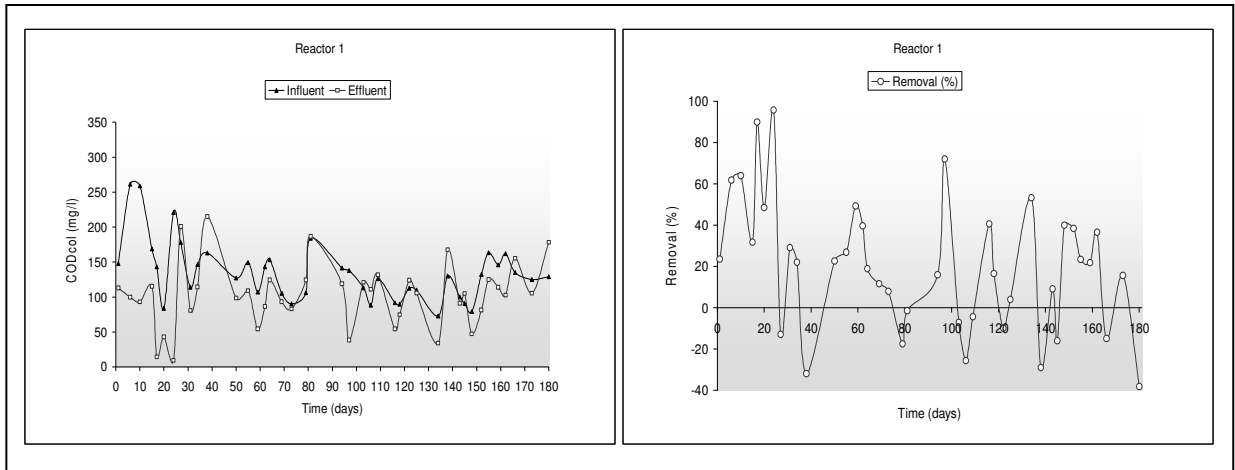
Figure 4.3. COD_{col} influent and effluent concentrations (left) and removal efficiencies (right) for R1.
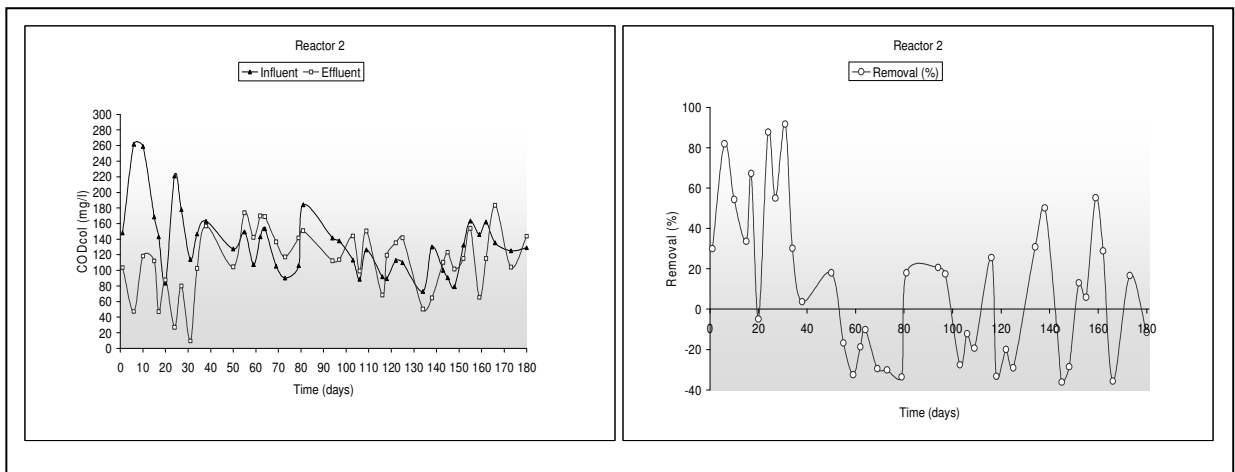
Figure 4.4. COD_{col} influent and effluent concentrations (left) and removal efficiencies (right) for R2.

If the results obtained in this research had been compared to the results obtained by Al-shayah (2005) one can say in general that the UASB reactors were not efficient in removing colloidal matter and this was proved in this research, and by the research done by Al-Shayah (2005) with removal rate of COD_{col} of 27% and 32% for R1 and R2, respectively also this result reported by Emitwalli (2002).

In this research one can see that R1 is more efficient than R2 for removing COD_{col}, this result contrast the result obtained by Al-Shayah (2005) during the summer period of the year not only this but also one can see that the removal efficiency for R2 drop with large rate comparing to results obtained by Al-Shayah (2005) and the drop in removal efficiency in R1.

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The decrease in the removal rate efficiency may be regarding to the hydraulic rate where the COD_{sus} takes more time to degradable and so produce more and more COD_{col} in the reactors. However, this is proved from the high removal rate of COD_{sus} in R2 compared to R1. In R1 the solids leave the reactor faster than R2 without complete degradation relatively to R2 so there will be no more COD_{col} from the degradation of COD_{sus}.

4.2.1.4 COD_{dis}

In returns to the results obtained in research in removing COD_{dis} that are shown in Table 4.3 one can see that the average removal rate was 24% (15) and 28% (18) for R1 and R2, respectively and the pattern at which the removal took place in both of the reactors was the same as shown in Figure 4.5. This may indicate that the biological conditions are nearly the same in the two reactors. Moreover, no significant difference were found in removing COD_{dis} between the two reactors ($\rho > 0.05$).
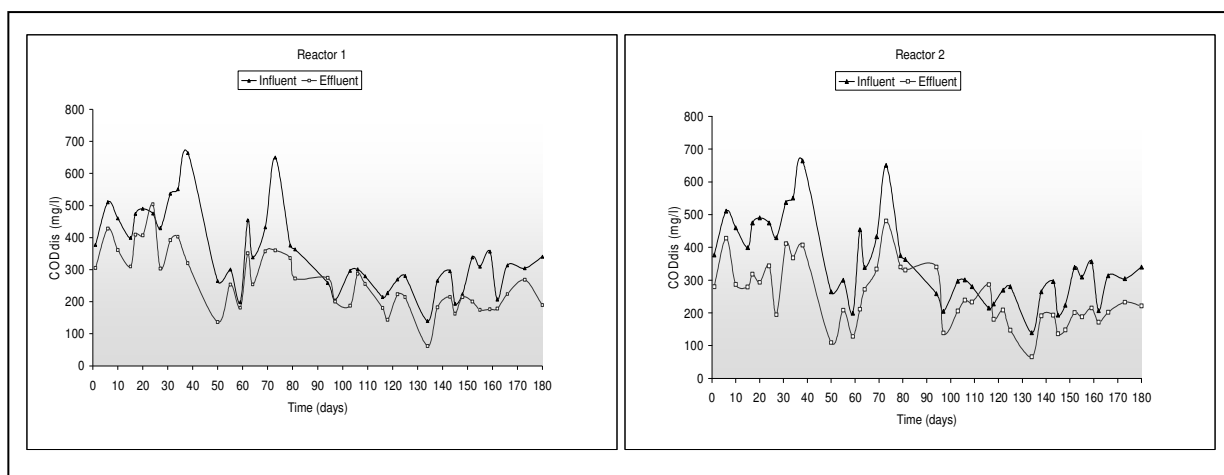
Figure 4.5. COD_{dis} influent and effluent concentrations for R1 (left) and R2 (right).

If the results obtained in this research compared to the results reached by Al-Shayah (2005) it could be seen that the over all removal efficiency in this research increased relatively to the results obtained by Al-Shayah (2005) during the summer period of the year which is 12% and 14% for R1 and R2, respectively.

In this research COD_{dis} in the effluent represents about 75.7% and 70.8% from COD_{tot} for R1 and R2, respectively where it was about 60% for both of the reactors during the results done by Al-Shayah (2005). Those results where in agreement with the results found by (Halasheh,

2002) about 50% from COD $_{tot}$ in the final effluent of UASB. Lucas Seghezzo (2004) also reported about 50% of the COD$_{tot}$ effluent from UASB was as COD$_{dis}$.

4.2.1.5 VFA

The results of the volatile fatty acids (VFA) concentrations for influent and effluent in R1 and R2 where shown in Table 4.3 and Figure 4.6 where the average concentration for VFA at the effluent in both of the reactors where 80 (37) mg/l and 69 (38) mg/l with average removal efficiency of negative removal and 2 % (82) for R1 and R2, respectively. The concentration of the VFA represent about 30% and 27.8% from the COD$_{dis}$ so most of the COD$_{dis}$ was in the form of non-acidified COD$_{dis}$ about 70% or more in both reactors. Wang (1994) found about 46% of the effluent COD$_{tot}$ after anaerobic sewage treatment could be attributing to non-acidified COD$_{dis}$ as proposed by van der last and Lettinga (1992). Limited acidification of soluble COD may reduce the maximum possible removal efficiency of anaerobic treatment process for treating sewage at low temperature (Seghezzo, 2004).
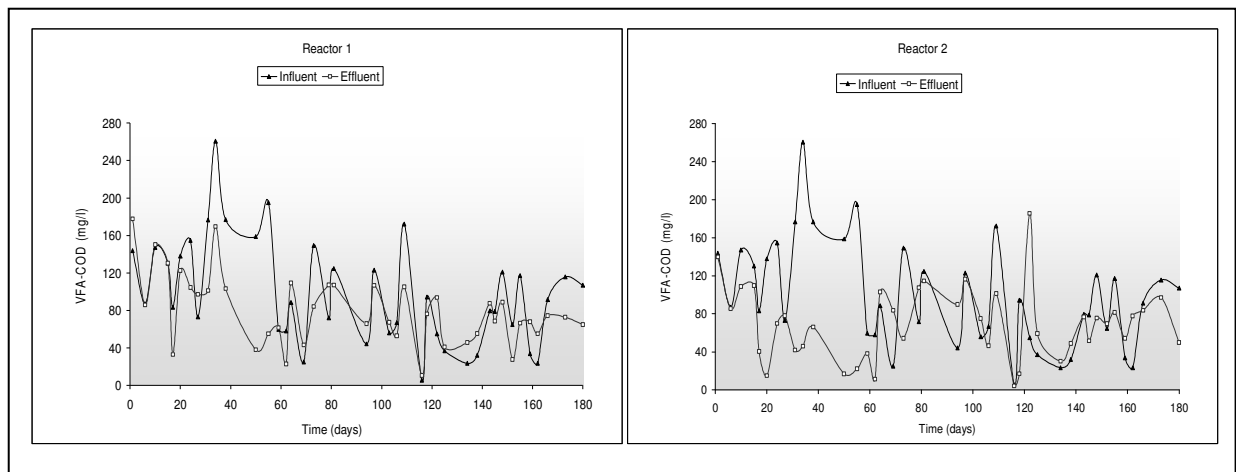
Figure 4.6. VFA influent and effluent concentrations for R1 (left) and R2 (right).

The very law removal efficiency or the negative one represents an increase in the VFA concentration which is mainly as a result of the predominant acidification process occurred in the two reactors as it will be shown later. The VFA concentration in the effluent was affected by temperature and the methanogenises conditions where the production of the VFA decreased during the winter period comparing to the results obtained by Al-Shayah (2005). Bogte *et al.,*

1993 reported that failing temperature resulted in reduced production of VFA accumulate at the reactors and a complete conversion of VFA in to CH_4 was achieved during 3 to 4 month of second year of the UASB-septic tank operation, when the temperature reach above 15 ºC.

4.3   Biodegradability of effluent

The anaerobic biodegradability for the effluent sewage was 47.64% and 41.7% for the reactors R1 and R2 respectively after 120 days at 30 ºC incubator for the two tests that had been took place at day 204 and 250 from the start up). The results obtained showed that the biodegradability increased in both reactors. If this result compared to the results reported by Al-Shayah during the summer period at the same year about 42% and 39% for R1 and R2, respectively. Results were very reasonable regarding to the decrease in the removing rate of the COD_{sus} (between the summer and winter period), the decrease in the removal rate in this parameter indicate that there will be more organic mater that can be degradable anaerobic escaped from the reactors by the effluent. Results showed that effluent biodegradability in R2 less than R1, and this also was justified by observed better efficiency of R2 for removing COD_{sus}.

Table 4.3. Research results for the effluent concentration and removal efficiency (%) during the whole period of experiment in the two UASB-septic tank reactors under the imposed operational conditions. Standard deviations are presented between brackets.

Parameter	Sample #	Influent concentration	UASB-septic tank (R1) (HRT = 2 days)				UASB-septic tank (R2) (HRT = 4 days)			
			Effluent concentration		Removal efficiency (%)		Effluent concentration		Removal efficiency (%)	
			Range	Average	Range	Average	Range	Average	Range	Average
COD_{tot}	41	905	611-213	433 (109)	69 – 32	51(9)	680 -115	408(109)	77-29	54(11)
COD_{ss}	41	396	142-5	62 (34)	99 – 58	83(10)	151-8	45(30)	97-66	87(8)
COD_{col}	41	135	215-9	104 (46)	96 - ˉ38	20(32)	183-9	112(41)	92 - ˉ36)	10(37)
COD_{dis}	41	350	504-62	265 (96)	56 - ˉ6	24(15)	481-66	248(94)	58- ˉ34	28(18)
VFA as COD	41	99	178-10	80 (37)	76 - ˉ135	-1(52)	186-4	69(38)	(89 - ˉ238	2(82)
BOD_5	13	502	132-410	283 (81)	60 - 25	43(12)	331-133	246(64)	65-15	49(16)
NKj as N	14	70	(77-45)	58 (7.5)	29 - 5.2	17(7.2)	74 -45	59(8.4)	33-4.6	15(8.3)
NH_4^+ as N	19	39.2	(75.5-7.2)	35.6 (20.7)	60.7 - ˉ20.2	11.5(20.7)	72.23-3.7	36(21.2)	59.44 - ˉ14.2	13.1(22.6)
Total PO_4as P	11	10.1	(13.4-4.2)	9.8 (3)	24.3 - ˉ45.2	0.43(16.6)	14-4.7	10.25(3.1)	24 - ˉ42.7	-4.34(16.4)
PO_4^{-3}as P	11	8.4	(16.7-3.6)	10.7 (4.47)	-2.6 - ˉ194.5	-37.8(53.5)	18.3- 4.4	11.9(4.5)	-7.8 - ˉ193.1	-57.3(59)
SO_4^{-2}	15	94.7	(49.4-20.9)	38.4 (8.23)	76.32 - 44.68	57.65(8.48)	54.67-18.47	36.47(10.74)	74.73-51.84	61.45(6.16)
TSS	13	371	(130-50)	89 (29)	87 - 50	74(10)	119-20	73(32)	95-52	78(11)
VSS	13	313	(109-40)	75 (26)	89 - 49	74(10)	102-16	59(27)	95-48	78(12)
(VSS/TSS)	13	83	(97-73)	84 (6)	---	---	86-72	81(4)	---	---
pH	39	7.6	7.68-7.14	7.44 (0.13)	---	---	7.79-7.1	7.47(0.16)	---	---
* Biodegradability	2	---	(59.4 -37.3)	47.64 (9.6)	---	---	51.9-34.4	41.7 (7.5)	---	---

All parameter are in mg/l except:- pH no units;VSS/TSS (%) ; Biodegradability (%).* Biodegradability done twice the first test at day 204 and the second experiment at day 250 from the start up of the reactors.

4.4 Hydrolysis, Acidogenesis and Methanogensise

The average value of the Hydrolysis, Acidogenesis and Methanogenise for the total period of the research was summarized in Table 4.4.

Table 4.4 the calculated average values for Hydrolysis (H), Acidogenesis (A) and Methanogenesis (M) in both reactors (R1 and R2) for each month and over all average during total research period. Standard deviations are presented in brackets.

Months	Reactor 1			Reactor 2		
	H%	A %	M%	H%	A%	M%
October,2004	32.51	28.89	26.33	16.79	20.86	23.00
November,2004	21.65	22.09	26.97	10.93	9.32	22.47
December,2004	12.00	20.26	18.87	13.26	21.13	19.06
January, 2005	28.58	25.91	24.09	28.29	23.69	21.80
February, 2005	31.18	32.33	28.71	21.41	28.45	26.52
March, 2005	27.19	31.01	29.87	23.39	26.34	27.07
Average	26(15)	27(9)	26(6.9)	19(14)	22(10.8)	23(5.5)

The research shows that the methanogenesis was apparently the limiting rate step for the over all conversion of organic matter to methane in both reactors as the effluent (soluble and VFA) COD remained relatively high. See Table 4.3. On the other hand there were unexpected results appear in this research where all the anaerobic process rates (Hydrolysis, Acidogenesis and Methanogensise) in R1 were grater than R2 that have the longer HRT where the statistical analysis proved that this result was statistically significant ($\rho<0.05$). This result unexpected because for long HRT anaerobic process. The anaerobic bacteria groups will have more time for metabolism and what was happened is exactly the opposite and disagrees with the results reached by Al-Shayah (2005) during the summer period. This may be regarding to degradable of the organic matter that was accumulated at sludge during the summer period, also the methanogensise may increased because the amount of sludge in R1 which is larger than R2 as will shown at Table 4.7 evolve more methane gas, so the process looks more efficient than it

was at R2. The variation of the Hydrolysis, Acidification and Methanogenesis steps during the whole period of the research are drown in Table 4.5 and Figure 4.7.
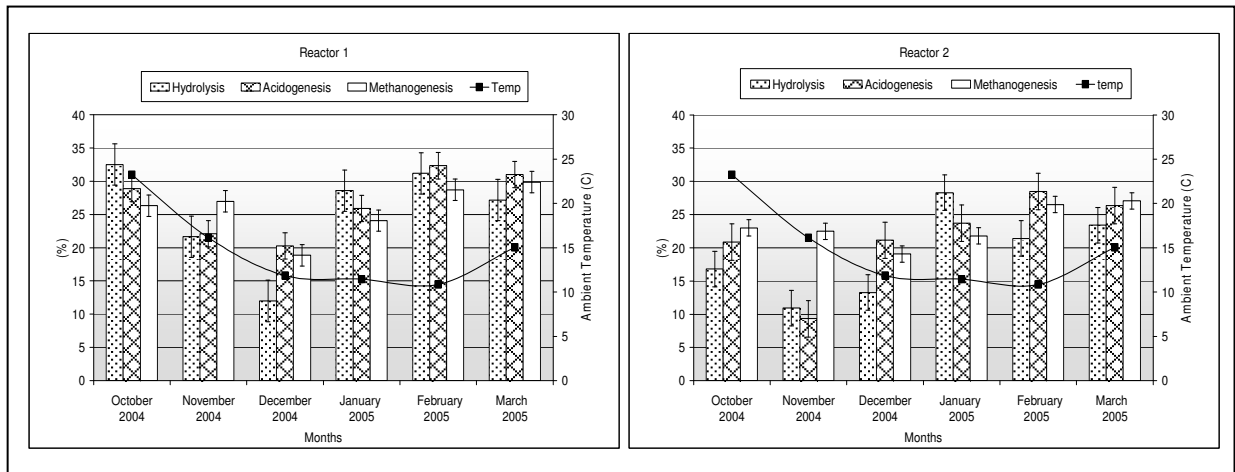

Figure 4.7. Percentage of hydrolysis, acidification and methanogenesis of domestic sewage treatment in UASB-Septic tanks R1 (left) and R2 (right).

The figure shows large variation between the steps during the period of the research, this also was shown from the standards division in Table 4.4.

4.5 COD mass balance

The amounts of the COD that inter the UASB-septic tank reactors has only three choices either to leave the reactor as CH_4 or to leave it through the effluent, or to stuck at the reactors and accumulate in the sludge bed. On other word the daily mass of the influent COD is equal to the sum of the daily mass of COD leaving the reactor as (methane and effluent) and accumulated COD in the sludge bed.

Some researchers have provided information about their systems that could lead to formulation of COD balance (Bogte *et al*., 1993; Mahmoud, 2002; Segezzo, 2004; Al-Shayah, 2005). In his research the monthly mass balance over the two UASB-septic tanks during the period of the research are summarized in Figure 4.8 below for R1 and R2.

Figure 4.8. The monthly COD mass balance of R1 (left) and R2 (Right) over the total test period as a percentage of average influent COD_{tot} and divided over COD accumulated, COD effluent and CH_4 as COD.

Each column represents the effluent total COD, total methane produced as COD (gas formed and dissolved methane) and accumulated COD. Through analyzing Figure 4.8 for both R1 and R2 one can see that in the second month of the experiment, which took place when winter started, the accumulated COD started to decrease and move out in the direction of the effluent COD without any change in the amount of the production methane. Later on when the temperature started to decrease gradually, a reduction in the amount of COD accumulated and

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methane production was observed and so an increase in the amount of COD left the reactor with the effluent. This observation during that month could be also proved by the lowest monthly COD removal efficiency during the period of the experiment about 41% and 40% for R1 and R2 respectively. Later on in the forth month of the research the removal efficiency of COD increased regarding to the reduction in the COD escaped from the reactor by the effluent, this reduction observed as an increase in the methane production and the amount of COD accumulated. Moreover, later in the fifth and the sixth month of the research period the amount of methane production stayed approximately constant while the accumulated COD started to decrease with an increase again in the COD which left with the effluent. During the six month of the research the COD mass balance could be represented in Figure 4.9 where the figure shows that about 25.25% and 30% of the COD accumulated in the reactor R1 and R2 respectively, and about 25.6% and 23.32% from the total COD entered the reactors R1 and R2 respectively converted to methane. The proportion of COD accumulated and found in R2 which is relatively higher than R1 justified the slightly better removal efficiency that detected in R2.
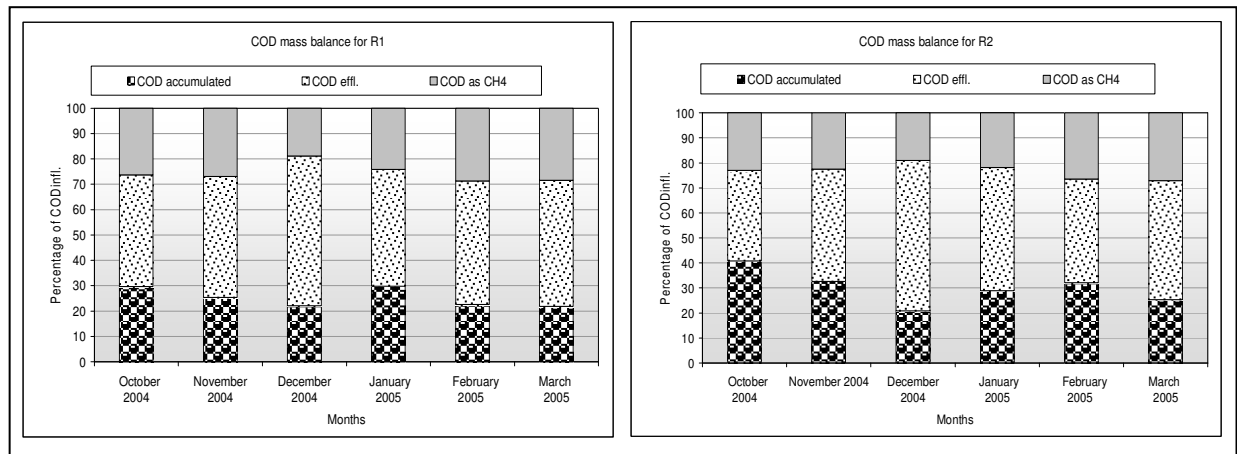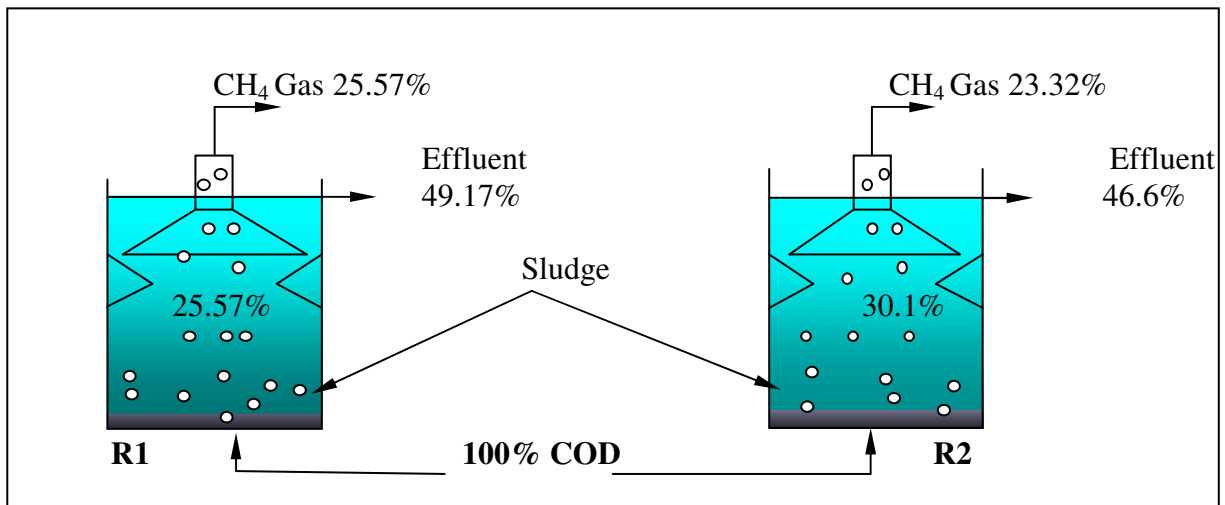

Figure 4.9. COD mass balance of R1 (left) and R2 (Right) over the total test period as a percentage of average influent COD $_{tot}$ and divided over COD accumulated, COD effluent and CH4 as COD.

4.6 Biogas production

The average CH_4 gas measured at Al-Bireh wastewater treatment plant for R1 and R2 respectively was 12 (10.55) l/d and 3.88 (3.7) l/d. Figure 4.10 shows the rate of gas production in R1 and R2 with the ambient temperature variation during the study period.
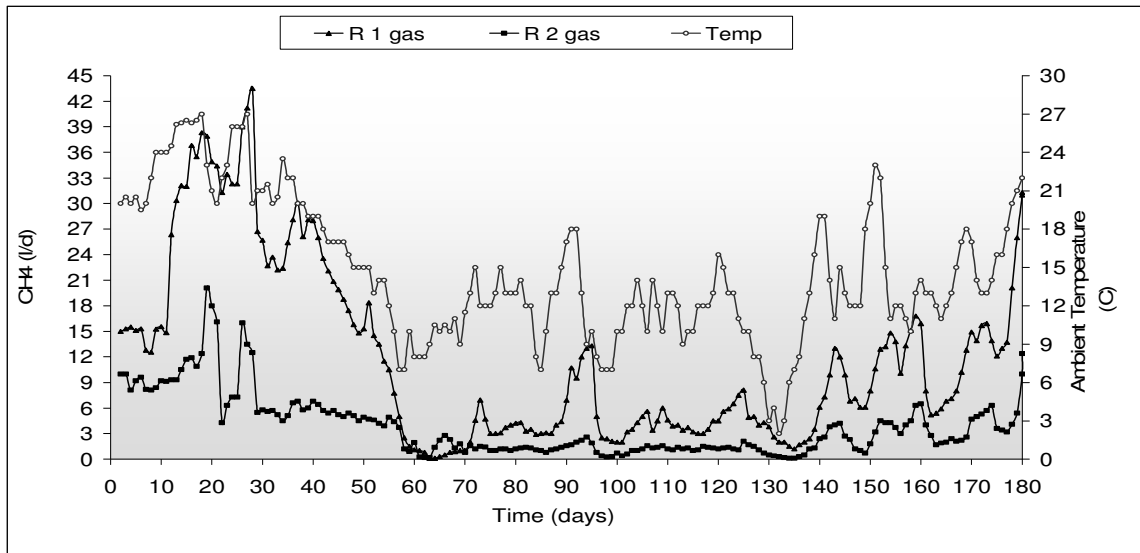
Figure 4.10. Gas productions in R1 and R2 with the ambient temperature variation during the study period.

From the figure one can see how much the variation in temperature affected the amount of gas produced. The figure shows that the gas production decreased continuously during the beginning of winter and continued to decrease until it reached its lowest value about (0.1 CH_4 l/d) at the coldest period in the year at Palestine climate which is located between the end of November 2004 and the beginning of January 2005 then the gas curve started to increase, as the temperature increased gradually until the end of the research period, the same phenomena took place for R1 and R2 but it was clearly shown clearly in R1.

However, the average "total" CH_4 production during research period was 16.57 l/d and 6.17 l/d from R1 and R2 respectively. The dissolved CH_4 represented about 42.5% and 46% from the total gas produced, comparison to 33.5% and 29.5% at Al-Shayah (2005). In general the total CH_4 production referred to the sum of the collected CH_4 and the dissolved amount which calculated according to Yamamoto *et al.,* 1979 where the following assumption was according to Yamamoto (1979) used to calculate dissolved methane.

1- Assumed Distilled water.

2- Pressure of 1 atm where it was 0.923 atm at the wastewater treatment plant.

3- The following curve will use to represent the dissolved methane at different temperature.
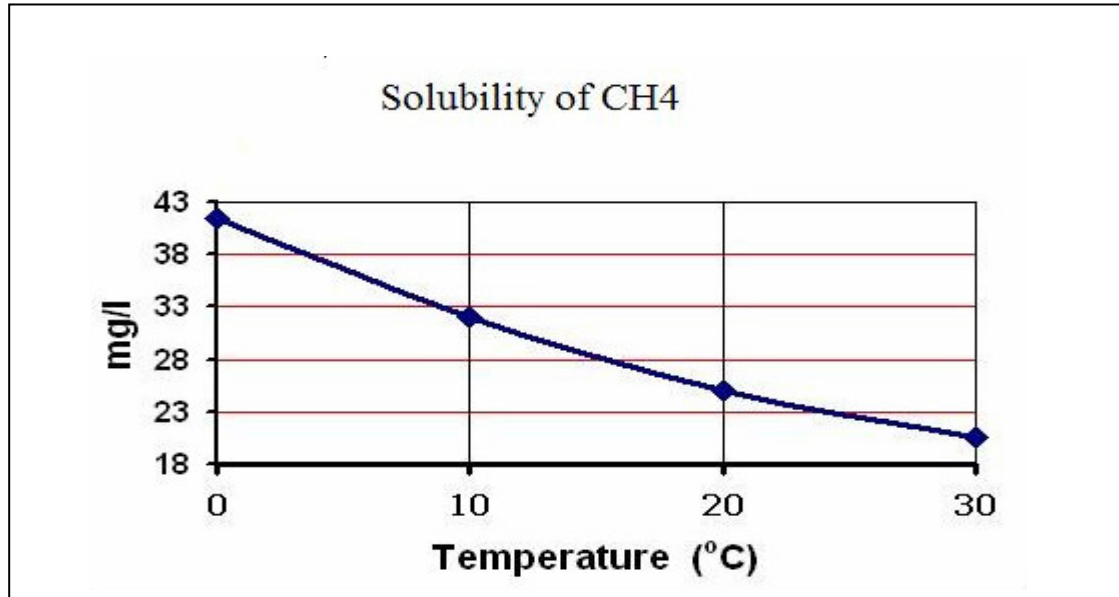
Figure 4.11 Solubility of Methane in Distilled water at 1atm. (Yamamoto *et al.,* 1976).

The average total methane production from both reactors was 0.11 N m³/kg COD removed and 0.10 N m³/kg COD removed for R1 and R2 respectively. (N indicates the volume is expressed at (STP) conditions).

The results obtained here confirm the results that had been obtained by Al-Shayah (2005) and there was no difference found between the two reactors regarding to the ratio of the total gas production to kg COD removal, Al-Shayah (2005) reported 0.1 N m³/kg COD removed in both of the reactors. The results of the research and the results reported by Al-Shayah(2005) close to the results reported by Harada (2000)(0.16 N m³/kg COD removed) and by Mahmoud (2002) (0.15 N m³/kg COD removed) and 0.1 N m³/kg COD removed by Seghezzo (2004). Considering that the theoretical ration for the maximum possible methane production from organic matter is 0.35 N m³/kg COD removed COD (Haandel and Lettinga, 1994).

4.7 Characteristics of the retained sludge in the UASB- septic tank reactors

The characteristics of the retained sludge of both reactors used in this research R1 and R2 are tabulated in Table 4.5 were the sludge sample during the period of the research taken from (port No1) of both reactors which is about 15 cm from the bottom of the reactor, the sludge sample analyzed for total solids (TS), Volatile solids (VS), COD and Stability.

Table 4.5. Characteristics of the retained sludge in UASB-septic tank reactors from the first port (0.15 m from reactors bottom). Standard deviations are present between brackets.

Parameter	# Sample	R1	R2
COD_{tot}	9	48.56 (3.66)	47.56 (2.87)
Total Solids (TS)	9	66,67 (9.45)	52.90 (5.72)
Volatile Solids (VS)	9	45.14 (5.49)	35.35 (3.09)
(VS/TS)	9	67.90 (2.59)	67.02 (3.51)
(COD/VS)	9	1.09 (0.17)	1.36 (0.17)
*Stability at day 204	2	68.4	65.68
*Stability at day 250	2	62.93	61.40

All parameter are in g/l except stability (%)(g CH_4- COD); (VS/TS) ratio (%);(COD/VS) ratio; *The bottles of the stability tests incubated at 30°C for a period of 120 days.

On day 160 of the research the sludge reached the height of 0.4 m (port 2) in R1 and at day 175 the sludge reached the same point in R2. In general the height of the sludge at the end of the research (after one year) reach to 50 cm at R1 and 40 cm at R2 see Table 4.7. The characteristics of the sludge from (port 2) which was analyzed only one time at the end of the research period on day 186 and the following results obtained and written in Table 4.6.

Table 4.6. Characteristics of the retained sludge in UASB-septic tank reactors from the second port (0.4 m from reactors bottom).

Parameter	# Sample	R1	R2
COD $_{tot}$	1	38.07	10.9
Total Solids (TS)	1	21.11	14.2
Volatile Solids (VS)	1	9.5	9.31
(VS/TS)	1	45	65

All parameter are in g/l ; (VS/TS) ratio(%);(COD/VS) ratio

In general and as reported in the literature review the sludge hold-up time of the system is so long and withdraw of sludge could be done once every 4 years for this system. The sludge height growth inside the reactors during the research period was clearly observed in both reactors, from continues increase in the total solids consideration as shown in Figure 4.12.
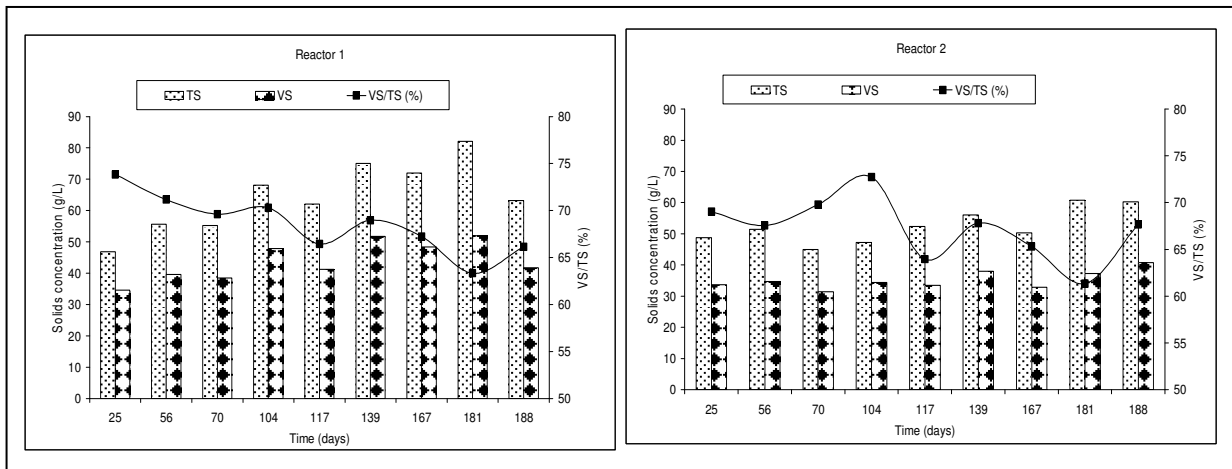
Figure 4.12. The time course for the sludge concentration in R1 (left) and R2 (Right) as TS, VS ratio at 0.15 m height stands from the bottom of the reactors.

The average of the total solids concentration (TS) in this research was about 66.65 (9.45) g/l and 52.9 (5.72) g/l for R1 and R2 respectively, with a comparison to 46.8 g/l and 48.6 g/l as reported by Al-Shayah (2005) one can see that the concentration increased and as an agreement with the results that obtained by Al-Shayah (2005), the development of granules was not detected in the UASB-septic tank reactors. The increase in the sludge concentration in R1 rather than R2 could be regarding to the increase in the HRT which directly increased the OLR. Figure 4.12 shows a decline trend in (VS/TS) ratio at both reactors were the average

ratio was 67.9 (2.59) and 67.02 (3.51) for R1 and R2 respectively which was approximately the same but, lower than the values obtained by Al-Shayah (2005) about 73% and 71% for R1 and R2 respectively. Regarding to Wang (1994) a (VS/TS) ratio of 63% can be considered a well-stabilized sludge. The decline trends in (VS/TS) ratio during the research period indicate a more stable sludge is achieved as reported by Al-Shayah (2005).

The results that are found and obtained in Figure 4.12 above agree with the stability tests, which show a good stability as shown in Table 4.5. The stability tests shows that the retained sludge in R2 was more stable than R1 during the stability tests that took place at the days 204 and 250 from the start up of the reactors. These results were reasonable regarding to the variation in the HRT of the two reactors that lead to expect high stability for the returned sludge in the reactor that had lowest HRT. Moreover, the two stability experiments agree with the results expected from the decay ratio of (VS/TS) and this could be seen from the decrease in stability percentage in R1 and R2 during the research period. Finally there were no significant difference ($\rho>0.05$) between the sludge elements (COD $_{tot}$, TS, VS, (VS/TS), Stability) in both reactors.

4.8 Scum layer and sludge washout phenomenon

There was a relationship that constitutes scum layer, washout of sludge and effluent quality in UASB reactors Mgana (2003). In this research a very thin green to brown layer of (1-2) mm in thick present at the top of the reactors during the whole research period (See Appendix 3 photo A3.3), during this research some intermittent washout sludge were observed during the research period, the amount of sludge was very small which accurately observed during the first and last month of the research when the gas productivity was high and its amount during one year will be shown later in Table 4.7.

Regarding to the scum layer, different researchers reported several reason that cause the formation of scum layer such as insufficient mixing high grease continent in the influent, severe temperature fluctuation, high concentration of fatty acid, and accumulation of

undegraded SS (Pagilla *et al.,* 1997; Yoda and Nishimura, 1997; Kalogo and Verstraete, 1999).

4.9 BOD Removal Efficiency

BOD_5 considered as a measure for the biodegradable organic matter in the wastewater. In this research the BOD_5 mean value of the influent and the effluent for the two reactors and removal efficiency for each of them are tabulated in Table 4.3. From the table one can see that the average BOD_5 for the Influent is about 502 (132.8) mg/l where the largest value of the standard deviation is related to the rain variation during the winter season. The average BOD_5 effluent from the two reactors R1 and R2 are 283 (81) mg/l and 246 (64) mg/l, respectively with average removal efficiency during the period of the experiment for R1 and R2 43 % (12) and 49 % (16). Figure 4.13 shows the relation between the influent and the effluent of the BOD_5 concentration and the removal efficiency for both of the reactors.

Figure 4.13. BOD_5 influent and effluent concentrations and removal efficiency for R1 (left) and R2 (right) along with study period.

From Figure 4.13 the BOD_5 effluent quality for R1 and R2 relatively stable if compared with the BOD_5 of the influent and the value of the standard deviation can also confirm this result. R2 give more stability than R1 for the BOD_5 effluent concentration also R2 is more efficient than R1 for removing DOD_5. The results of DOD_5 removal were not statistically significant ($\rho > 0.05$).

In this research the removal efficiency in R1 and R2 decreased in comparison with the removal efficiency for both reactors at summer period where the removal efficiency were 56% and 59% for R1 and R2 respectively as reported by (Al-Shayah, 2005).

4.10 TSS and VSS removal efficiency

The removal of the suspended solids is one of the main objectives of sewage treatment. UASB reactors are very efficient at retaining suspended solids from sewage, especially in tropical regions (Haandel and Lettinga, 1994, Cavalcanti, 2003). In this research and during its period the average TSS and VSS of the influent and effluent of the two reactors R1 and R2 are tabulated in Table 4.3.

In this research some results were encouraging as the TSS removal efficiency that is 74 % (10) and 78% (11) removal efficiency for R1 and R2 respectively but with no statistical significant differences ($\rho > 0.05$) between the two reactors. These results and if are compared with the reactor efficiency during the summer period which is 79% and 80% for R1 and R2 respectively as reported by (Al-Shayah, 2005) one indicate that the removal efficiency for both of the reactors decrease but the removal efficiency of R2 did not affected as R1 did during the winter period (low temperature).
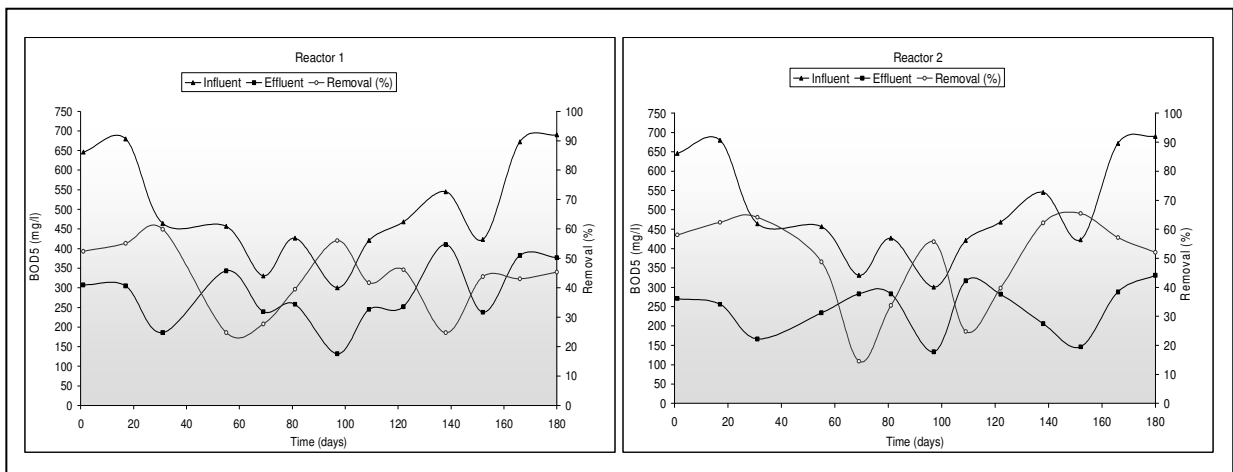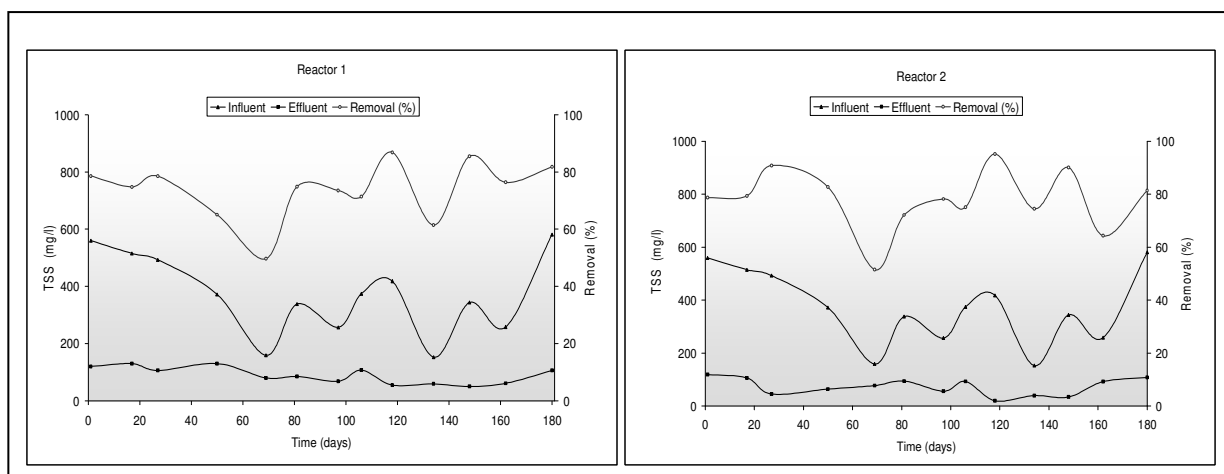

Figure 4.14. TSS influent and effluent concentration and removal efficiency for R1 (Left) and R2 (Right) along with the study period.

Figure 4.14 shows the average values of the TSS concentrations and removal efficiency for R1 and R2. From this figure one can see how much the two reactors are stable regarding to the TSS concentrations measured at the effluent throughout the period of the research. The results reported for TSS in this research are better than the results reported in literature review for conventional UASB reactors that have treated domestic wastewater.

The removal efficiencies averages for VSS for this research were 74% (10) and 78% (12) for R1 and R2, respectively. However, R2 is significantly better than R1 with respect to VSS removal efficiency ($\rho<0.05$). If those results are compared to the results that had been obtained by Al-Shayah, (2005) 79% and 80% VSS removal efficiency for R1 and R2, respectively one can conclude that the VSS removal efficiency also decreased , R2 was not affected as much as R1 did during the winter (law temperature) period.
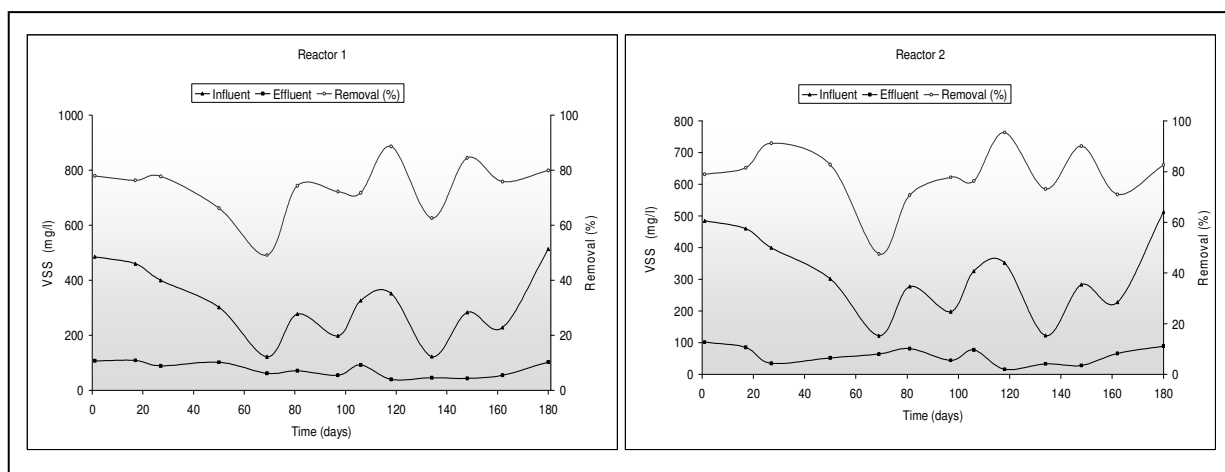
Figure 4.15. VSS influent and effluent concentration and removal efficiency for R1 (Left) and R2 (Right) along with the study period.

Figure 4.15 shows the average value of the VSS concentration and the average removal efficiency for R1 and R2 also it shows how much the effluent VSS concentration was stable for both of the reactors during the whole research period. The removal efficiency of the TSS and VSS in UASB depend on the type of sewage, temperature (Elmitwalli, 2000; Mahmoud, 2002; Seghezzo, 2004). The decrease in the efficiency in this research was due to the decrease in the temperature which is directly increased the viscosity of the wastewater and so increase the hydraulic shearing force on solid particles (Mahmoud, 2002) so that solids particle will

move out the reactor then the concentration of the solids will increase leading in the increase of the TSS and VSS concentration which directly reduce the efficiency removal in the two reactors.

The average VSS/TSS ratio for both of the reactor R1 and R2 were 0.84 (0.06) and 0.81(0.04) respectively, which is closed to the results that are reported by (Al-Shayah, 2005). Also the sludge volume index (SVI) did not record any value in the effluent of both the UASB reactors and this observation agrees with the observation remarked by (Al-Shayah, 2005)

4.11 Nutrient removal efficiency

4.11.1 Nitrogen removal

4.11.1.1 (NH_4^+) removal

The results during the whole period of the research show that the average removal of the NH_4^+ was very low for both of the reactors where the average (NH_4^+-N) concentration for the UASB reactors R1 was 35.61 (20.71) mg/l with average removal efficiency 11.47% (20.66) and so for R2 35.99 (21.21) mg/l with average removal efficiency of 13.06 % (22.6). However, the difference in removal efficiency of (NH_4^+-N) were not statistically significant ($\rho>0.05$). Comparing those results with the results that had been obtained by (Al-Shayah, 2005) indicate that there is an increase in the removal efficiency in both of the reactors especially at R2. This was regarding to the low hydrolysis rate in the part of the organic matter which contain organic nitrogen i.e (the organic nitrogen and protein did not hydrolyses completely) this result opposite the results that was obtained by (Al-Shayah, 2005) that reported an increase in the concentration value of the effluent NH_4^+-N than the concentration of the influent specially at R2. (Al-Shayah, 2005) justifies this increase to the mineralization of the compounds containing organic nitrogen as result of protein hydrolysis. In general the hydrolysis process rate affected by several factors such as temperature, which affected the hydrolysis rate, in this research during the winter period. Figure 4.16 shows the variation of the NH_4^+-N concentration and the removal efficiency of the two UASB reactors during the period of the study.
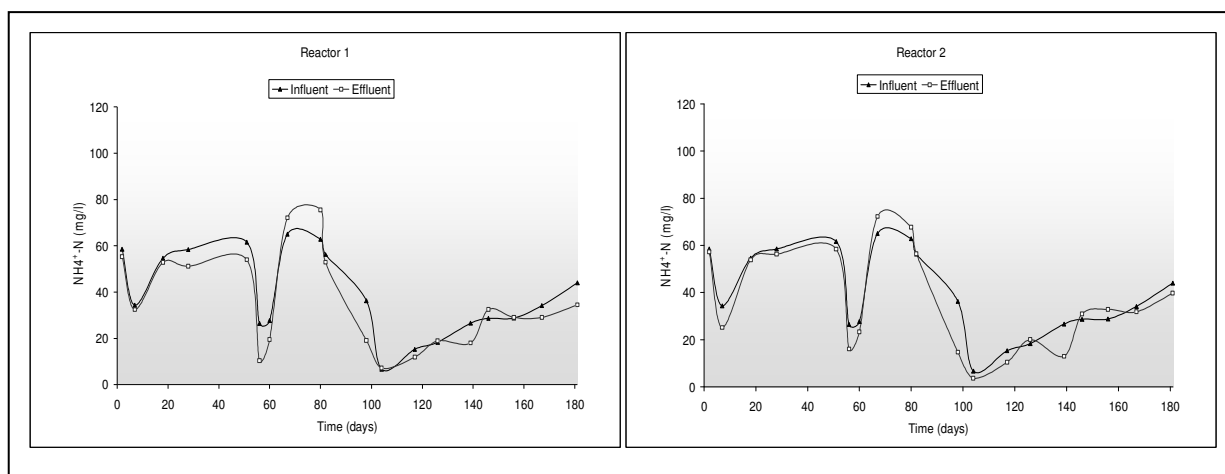
Figure 4.16. NH_4^+-N concentration for influent and effluent for R1 (Left) and R2 (Right) along with the study period.

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4.11.1.2 (Nk$_j$-N)

The Nkj-N was partially removed in the USAB reactors due to particulate N removal see Table 4.3 and Figure 4.17 the average removal efficiencies of Nkj-N were 17 % (7) and 15 % (8.3) for R1 and R2, respectively. Moreover, the difference in removal efficiency of (Nkj-N) were not statistically significant ($\rho > 0.05$).

If those results are compared again to the results during the summer period that had been obtained by (Al-Shayah, 2005) one can see that the efficiency of removing Nkj-N was also increased but in a form of small change 16 % and 12 % for R1and R2, respectively. The same trends of Nkj-N also reported by Bogte *et al.,* 1993 and (Mahmand, 2002) when treating domestic wastewater in UASB reactors.
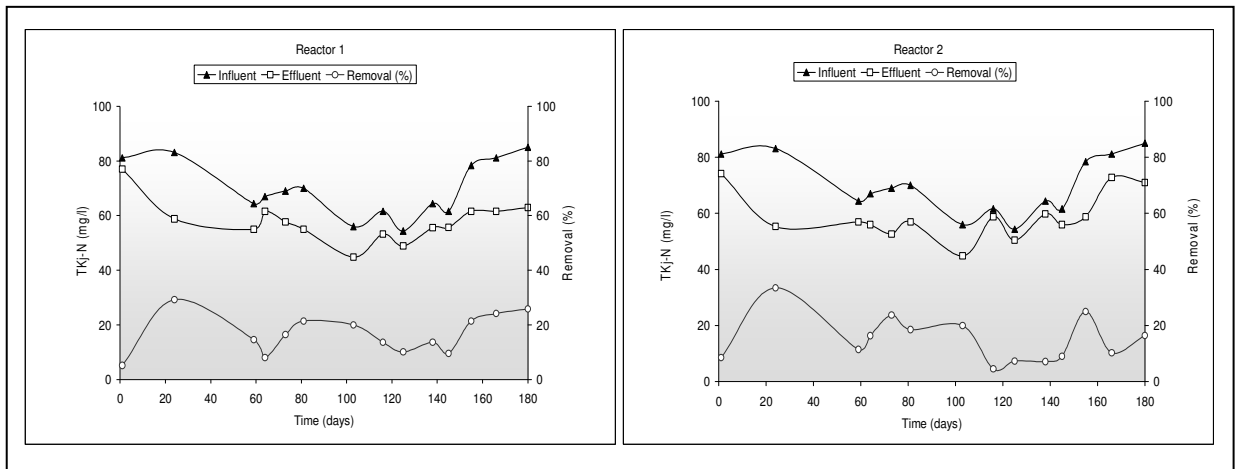
Figure 4.17.Nkj-N influent and effluent concentration and removal rate efficiency for R1 (left) and R2 (right).

4.11.2 Phosphorus removal

4.11.2.1 (Total – P)

The results show that the difference in Total - P concentration between influent and effluent in the two reactors was very low and within the marginal error of the used measuring instrument Nevertheless, the average Total - P concentration before and after the UASB-septic tank treatment decreased from 10.09 (3.5) to 9.8 (3) mg/l in R1 with removal efficiency of 0.43% (16.6), while slightly increased from 10.09 (3.5) mg/l to 10.25 (3.1) mg/l in R2, the statistical results shows that the different in removal efficiency between R1 and R2 of total – P is statistically significant ($\rho<0.05$). The results observed could be clearly shown in Figure 4.18 and Table 4.3.

Figure 4.18. The concentration of total phosphorous in the influent and effluent for R1 (Left) and R2 (Right) along with the study period.

Regarding to the results obtained by Al-Shayah (2005) during the summer period 2% removal efficiency and increase in the concentration of the effluent in R1 and R2, respectively. One can figure out that there is no change in the pattern that the two reactors acted in during the summer and winter periods in removing the Total-P.

4.11.2.2 Ortho phosphorous

The research shows that there is no removal take place for Ortho phosphorous, on the opposite the effluent concentration an increase in both of the reactors from average concentration at the influent 8.4 (4.07) mg/l to 10.7 (4.42) and 11.9 (4.5) mg/l for R1 and R2, respectively.

The same results also had been obtained by Al-Shayah (2005) during the summer period so no change in the reactors functions between the summer and winter period.
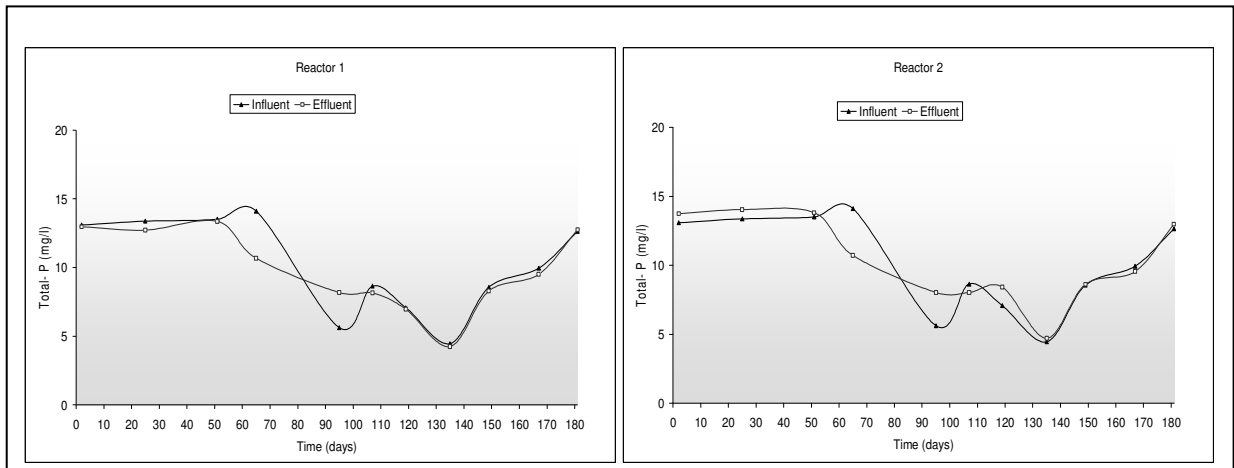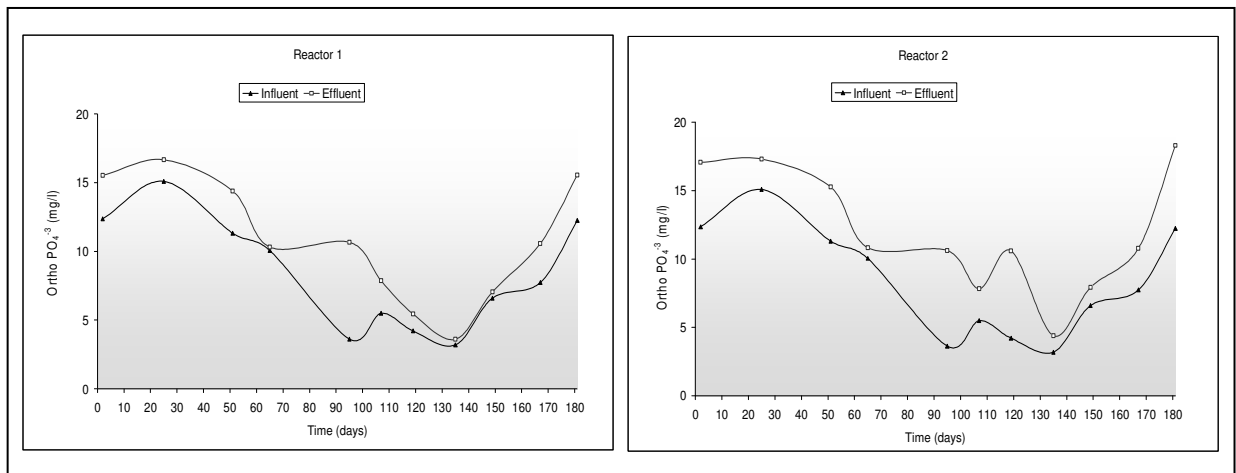

Figure 4.19. The concentration of Ortho-phosphorous (PO_4^{-3}) in the influent and effluent for R1 (Left) and R2 (Right) along with the study period.

Sedimentation and further degradation of particulate organic compound containing organic phosphorus as well as biological degradation of the soluble organic matter inside the reactors seemed to be the key mechanisms involved and stand behind this increase of Ortho-phosphorus concentration as reported by Al-Shayah (2005), where the phosphate concentration increase as result of the release of phosphorus from the polyphosphate pool under anaerobic concentration.

From Figure 4.19 one can see that the effluent concentration of PO_4^{-3} Ortho is always grater than the concentration of the influent. This observation also takes place as pointed out by Haandel and Lettinga (1994).

As a conclusion of the results that obtained through nutrient removal, one can say that the UASB- septic tank reactors are not efficient for removing nutrient from wastewater and only a change in the chemical forms of nitrogen and phosphorus take place as reported by Bogte *et al.,* 1993. Therefore, a nutrient removal can only be achieved in separate post-treatment step after the UASB septic tank Haandel and Lettinga (1994).

4.12 Sulfate removal efficiency

The major problem associated with anaerobic treatment of sulfate rich wastewater is the production of sulfide. Since sulfide can lead to several problems such as toxicity, bad smell, corrosion, deteriorated quality and quantity of the biogas and reduction of COD removal efficiency Mahmoud (2002).

In this research the average concentration for sulfate SO_4^{-2} in the effluent of R1 and R2 was 38.37 (8.23) mg/l and 36.47 (10.74) mg/l, respectively. No significant difference were found of SO_4^{-2} removal efficiency between both reactors ($\rho > 0.05$). The influent concentration as shown at Table 4.3 was about 94.67 (23.76) mg/l and so the removal efficiency for removing SO_4^{-2} for reactors R1 and R2 are 57.65% (8,48) and 61.45 % (6.16), respectively. In practice, anaerobic treatment always proceeds successfully for wastewater with COD to sulfate ratios exceeding 10. At (COD / sulfate) ration lower than 10 and very high concentration sulfate in the influent, process failures of anaerobic reactors as reported by (Halshoff Pol, 1998). In this research the (COD/sulfate) ratio for the treated domestic wastewater was 9.2 which is less than 10 but the concentration of the sulfate was not high so this observation did not effect the COD removal in the reactors since the high concentration of sulfide is inhibiting compound for anaerobic bacteria, including Methanogenic, Acetogenic and even sulfate reduction bacteria (SRB) such as Desulfovibrio (Visser, 1995).

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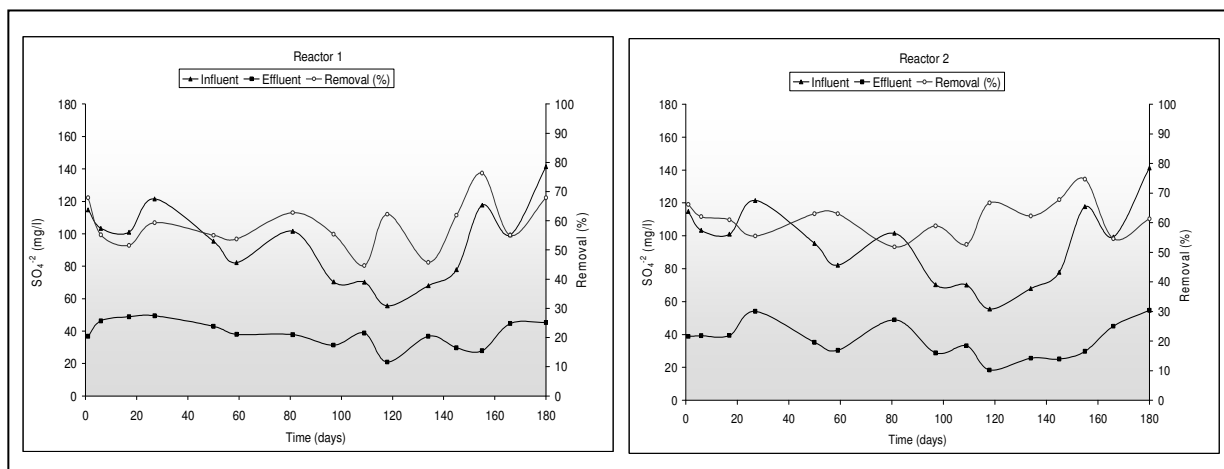
Figure 4.20. Sulfate (SO_4^{-2}) influent and effluent concentration and removal efficiency for R1 (Left) and R2 (Right) along with the study period.

From Figure 4.20 one can see that the effluent qualities for R1 and R2 for SO_4^{-2} were stable throughout the research period and it seemed to be not affected by the fluctuation in influent concentration. If the results obtained in this research compared with the results that had been obtained by Al-Shayah, (2005) one can see that the removal efficiency decreased where it was 72% and 71% for R1 and R2, respectively. Also one can see that R2 in this research was more efficient than R1 for removing SO_4^{-2} which is exactly the opposite the results obtained by Al-Shayah (2005) during the summer period.

4.13 pH in the UASB- septic tank reactors

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

In this research the pH mean value for the raw sewage Influent was 7.6 (0.28) and 7.44(0.13) and 7.47 (0.16) for the effluent of R1 and R2, respectively. The slightly lower pH values which was observed in the UASB effluent is expected in the anaerobic treatment where the buffering capacity in the raw domestic wastewater is enough to neutralize the production of volatile acids and carbon dioxide, which dissolved at the operating pressure (Drost, 1997).

During the whole of the experiment was no observation for pH value out of the normal and optimum range where for R1 the pH ranged from pH (7.14-7.68) and for R2 pH ranged from (7.1-7.79) and this could be clear from Figure 4.21.
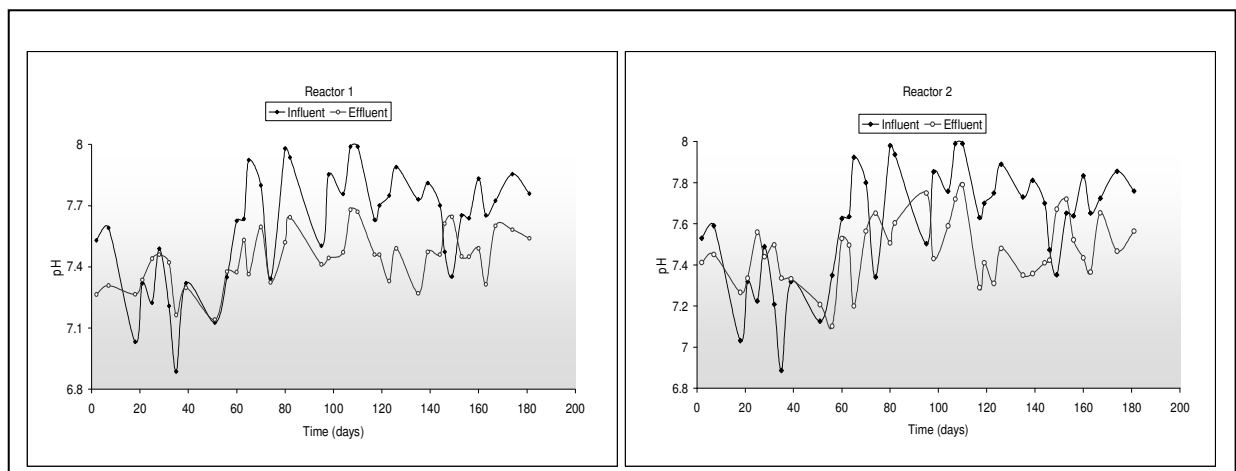
Figure 4.21. The evolution of pH value for R1 (left) and R2 (Right) along the research period.

If the results obtained in this research compared to the results reached at the summer period by (Al-Shayah, 2005) one can see that the pH mean value for R1 and R2 was around 7.4 with range of (7.12-7.7) during the summer period in both reactors.

4.14 General results

Form the average influent concentration of COD_{tot} (1045 mg/l) during one year which was obtained by this research and the research don by Al-Shayah (2005) one can calculate the number capita equivalent to the COD_{tot} entered each reactor which is 3 capita and 2 capita for

R1 and R2, respectively see Table 4.7. About 1892.5 l/c.year and 1321.85 l/c.year of CH_4 in gaseous form were produced from R1 and R2, respectively as measured from the gas meter see Table 4.7. Moreover, the annual specific sludge production for each capita was 2.1 kg TSS/c.year and 2.7 kg TSS/c.year for R1 and R2, respectively see Table 4.7.

The accumulated COD in the sludge after 1 year per person in the reactors equal 2.29 kg COD/c.year and 1.86 kg COD/c.year in R1 and R2, respectively see Table 4.7. These values were reasonable related to (Jewell, 1994) who reported that for each 100 kg COD soluble treated there will be 5 kg COD converted to sludge as mentioned at Figure 2.1. Theoretically and regarding to (Jewell, 1994), the amount of COD that present at the sludge equals 5% of the soluble COD. Regarding to this research the percentage was taken related to the total COD, relatively it is less than the results reached by (Jewell, 1994) it was 4.5% and 4.8% from the total COD for R1 and R2, respectively. In general the design criteria of community on site UASB-septic in Palestine are presented in Table 4.7 and 4.8. At Table 4.8 the OLR for R1 with HRT of 2 days was 0.45(0.12) and for R2 with HRT of 4 days was 0.23(0.06).

Table 4.7 General specification and results that Reached after one year of Full monitoring and operation.

Parameters	R1	R2
*Average COD_{tot} during one year	418 g/d	209g/d
**Equivalent capita	2.34 capita $\simeq$ 3 capita	1.17 capita $\simeq$ 2 capita
Gas produced from reactors in 1 year per person.	1892.5 l/c.year	1321.85 l/c.year
Height of the sludge at reactors	0.5 m	0.4 m
Volume of the sludge	0.16 m^3	0.128 m^3
***Available Sludge at reactors	7.01 kgTSS	4.23 kgTSS
***Accumulated sludge during 1 year	4.94 kg TSS	3.23 kg TSS
*** Annual Specific sludge produces per person.	2.1 kg TSS/c.year	2.7 kg TSS/c.year
Average Sludge COD during one year.	43.32 gCOD /l	29.23 gCOD/l
+The amount of sludge available at the Reactors as COD after 1 year per person.	2.29 kg COD/c.year	1.86 kg COD/c.year

* The calculation done after one year of continues operation and monitoring during this research and the research done by Al-Shayah, 2005. Where the average COD of influent for one year in AWWT = [(this research/Number of samples) + (Al-Shayah 2005/number of samples)]/total number of samples at one year ➔Average total COD inter to the reactor each day   = flow rate (L\d)* Average COD (mg\ L)

** The specific production of COD_{tot} (g/c.d) range from (155-202) with average value of 179 g/c.d as reported by Mahmoud, 2002 , ➔ Equivalent population for the Treatment plant reactors in Capita =   Average total COD inter to the reactor each day (g\d) / Specific COD Production (g\c.d))

*** Total Mass of sludge at the reactor kg TSS/year = [The average sludge TSS of in the reactor (g\m^3) * volume of sludge at the reactor (m^3)] ➔ Accumulated mass of sludge at the reactor (kg TSS/year) = [(Total Mass of sludge at the reactor kg TSS/year) – (Mass of sludge added at start up)] , where at start up the amount of sludge added to the reactor are 160L at R1 and 80 L in R2. With TSS = 13.78(g\l) so mass of the start up sludge added to R1 = 160 l x13.78 (g\l) = 2.2 kg TSS and 80 l x 13.78 (g\l) = 1.1 kg TSS for R2. (Al-Shayah, 2005),

➔ The specific sludge produced per person in one year = [Accumulated mass of sludge at the reactor (kg TSS/year) / Equivalent Population (Capita)]

+ COD at Sludge (kg COD\c.year) =[(Average COD concentration of sludge (g\l) * volume of sludge)/ capita]

Table 4.8. Design criteria of community on site UASB-septic tank in Palestine during the research period.

HRT	OLR (kgCOD/m^3.d)	Removal efficiency (%)			
		COD_{tot}	COD_{ss}	COD_{col}	COD_{dis}
2	0.45(0.12)	51(9)	83(10)	20(32)	24(15)
4	0.23(0.06)	54(11)	87(8)	10(37)	28(18)

Chapter 5
Conclusion and Recommendations

5.1 Conclusion

Based on the results of the study, the following conclusions were reached:

1- The performance of the UASB-septic tank during winter time represented by the removal efficiency of parameters decreased comparing to the summer period, even though; it is considered effective for anaerobic sewage (pre) treatment under Palestine conditions. Where it gave an average removal efficiency of COD_{tot}, COD_{sus}, DOD_5 and TSS of about "51, 83, 43 and 74 %" respectively for R1 with HRT of 2 days and "54, 87, 49 and 78%", respectively for R2 with 4 days HRT. Results obtained in this research continue to show that the longer HRT R2 (4 days) relatively give better efficiency than R1 (2 days) in most of the tested parameter during this research.

2- Both reactors R1 and R2 were not sufficient for removing COD_{col} from the influent during the research duration. The average removal efficiency was 20% and 10% for R1 and R2 respectively.

3- The COD_{dis} in the effluent of the UASB-septic tank relatively represented about 75.7% and 70.8% from the COD_{tot} for R1 and R2 respectively, with average removal rate of 24% and 28% for R1 and R2 respectively.

4- The anaerobic biodegradability of the effluent sewage was 47.64% and 41.7% for the two UASB-septic tank reactors R1 and R2, respectively after an incubation period of 120 days at 30 ºC. The results obtained showed that the biodegradability during winter time increased in both reactors comparing to the summer period.

5- The evolution of biogas production was strongly affected by temperature. The total average of the methane production was 0.11 N m^3/kg COD removed and 0.10 N m^3/kg COD removed for R1 and R2, respectively.

6- The research shows that the methanogenesis was the rate-limiting steps for over all digestion process in R1 and R2.

7- The results shows that the UASB-septic tank is not efficient for removing nutrients but the results show an increase in the (NH_4^+-N) and Nkj-N removal efficiency comparing to the summer period, this was regarding to the low hydrolysis rate in the part of the organic matter which contain organic nitrogen, i.e. (the organic nitrogen and protein did not hydrolyses completely).

8- The results showed a decrease in SO_4^{-2} removal efficiency comparing to the summer period where it showed an average removal efficiency of 57.65% and 61.45 % for R1 and R2, respectively for removing SO_4^{-2}.

9- The sludge hold-up time of the system is so long and withdrawal of sludge could be done once every 4 years for this system. The sludge height growth inside the reactors during the research period was clearly observed in both reactors. The (VS/TS) ratio for the sludge shows a decline trend with time at both reactors were the average ratio was 67.9 and 67.02 for R1 and R2, respectively which was approximately the same. Those values indicate a well-stabilized sludge; the stability tests showed that the retained sludge in R2 was more stable than R1 during the stability tests.

10- The annual specific sludge production per capita was 2.1 kg TSS/c.year for R1 and 2.7 kg TSS/c.year for R2. The accumulated COD in the sludge after 1 year per person in the reactors equal 2.29 kg COD and 1.86 kg COD in R1 and R2 respectively.

5.2 Recommendations

1- Regarding to the results reached in this thesis it is recommended to use the technology of the UASB-septic tank especially the reactor with a design of 4 days HRT to work as a pre-treatment system of wastewater in Palestine and where it could replace the cesspits.

2- It is recommended to investigate the proper method of Post-treatment to be applied after the UASB-septic tank to remove the nutrient and to reach   the needed quality of the final effluent depending on the type reused field.

3- It is recommended to keep monitoring the pilot plants for other months of the second year so as to examine what will happen to the accumulated sludge in the reactors when the temperatures increase after longest period from start up period.

4- It is recommended that the pilot plant researched should be moved and placed inside a green house where the temperature mostly higher than the ambient temperature. Where I expect the green house will work as an incubator.

References

Alaerts G. J., Veenstra S., Bentvelsen M., van Duijl L. A. Lindfield M., Specker H., van Velsen L., Wildschut L., Lettinga G., Halshoff Pol L., and Zuidema M., (1990). *Feasibility of anaerobic sewage treatment in sanitation strategies in developing countries* International Institute for Hydraulic and Environmental Engineering (IHE), Report Series 20, Delft, The Netherlands.

Al-Juaidy A. (2001) Enhanced pretreatment of black waste from Birzeit University using UASB technology. M.Sc. thesis in Water Engineering, Faculty of Graduate Studies, Birzeit University, Birzeit, West Bank, Palestine.

Ali M. (2001) Assessment of UASB technology as a pre-treatment stage for domestic wastewater from Birzeit University using UASB technology. M.Sc. thesis in Water Engineering, Faculty of Graduate Studies, Birzeit University, Birzeit, West Bank, Palestine.

Al-Shayah M. (2005) Community On-site anaerobic sewage treatment in a UASB- septic tank systems M.Sc. thesis in Water Engineering, Faculty of Graduate Studies, Birzeit University, Birzeit, West Bank, Palestine.

AI-Sa'ed R. M. (2000) Wastewater management for small communities in Palestine. In: *Proceedings of the Technical Expert consultation on appropriate and innovative wastewater management for small communities in EMR countries*. WHO/CEHA Amman, Jordan, November 6-9, 2000.

APHA (1995) Standard methods for the examination of water and wastewater. 18th edition, New York.

Buchauer K. (1998) A comparison of two simple titration procedures to determine volatile fatty acids in influents to wastewater and sludge treatment processes. *Water SA*, **24**(1)49-56.

Bartone C., Bernstein J., Leitmann J., and Eigen J. (1994), *Towards environmental strategies for cities*, Urban Environmental Management Paper, 18, World Bank, Washington USA.

Batstone D. J., Keller J., Angelidaki I., Kalyuzhyni S. V., PavJostathis S. G., Rozzi A., Sanders W. T. M., Siegrist H. and Vavilin V. A. (2002) *The IWA Anaerobic digestion Model No.l (ADMI). Water Sci. Technol.,* **45**(10), 65-73.

Bogte J. J., Breure A. M., van Andel J. G. and Lettinga G. (1993) Anaerobic treatment of domestic wastewater in small scale UASB reactors. *Water Sci. Technol.,* **27**(9), 75-82.

Cavalcanti,P. F. F. (2003) Integrated application of UASB reactor and ponds for domestic sewage treatment in tropical regions. Ph.D thesis, Wageningen University, Wageningen, The Netherlands.

de Man A. W. A., Grin P. C., Roersma R. E., Grolle K. C. F., and Lettinga G. (1986) Anaerobic treatment of municipal wastewater at low temperature, in Anaerobic Treatment . *A grown-up technology*, Papers of **IAWQ-NVA** Conference on Advance Wastewater Treatment (Aquatech 1996), Amsterdam, The Netherlands, 451-466.

Droste R. L. (1997) Theory and practice of water and wastewater treatment. John Wiley & Sons lnc, Canada.

Eastman J. A. and Ferguson J. F. (1981) Solubilization of particulate organic carbon during the acid phase of anaerobic digestion. Journal Water Pollution Control Federation **53**, 352-366.

E1mitwalli T. (2000) Anaerobic treatment of domestic sewage at low temperature. Ph.D thesis, Department of Environmental Technology, Wageningen University, Wageningen, The Netherlands.

Elmitwalli R. A., Sklyar V., Zeeman G. And Lettinga G. (2002) low temperature pre-treatment of domestic sewage in an anaerobic hybrid or an anaerobic filter reactor *Biores. Technol.,* **82**, 233-239.

Elmitwlli T. A., Sayed S., Groendijk L., van Lier J., Zeeman G. and Lettinga G. (2003) Decentralised treatment of concentrated sewage at low temperature in a two-step anaerobic system: two upflow-hybrid septic tanks. Water Sci. Technol., **48**(6), 219-226.

Fernandes X. A., Cantwell A. D., and Mosey F. E. (1985), Anaerobic biological treatment of sewage, *Water Pollution Control* **84**, 99-110.

Foresti E. (2001) Perspectives on anaerobic treatment in developing countries. *Water Sci. Technol.,* **44**(8), 141-148.

Gujer W. and Zehnder A. J. B. (1983) Conversion processes in anaerobic digestion. Water Sci. Technol., **15**, 127-107.

Haandel AC. van and Lettinga G. (1994) *Anaerobic Sewage Treatment. A Practical Guide for Regions with a Hot Climate.* John Wiley and Sons, New York.

Halalsheh M, M. (2002) Anaerobic pre-treatment of strong sewage A proper solution for Jordan. Ph.D. thesis, Wageningen University, Wagcningen, The Netherlands.

Haskoning (1996), 36 MLD UASB *treatment plant in Kanpur, India Evaluation report on process performance,* Haskoning Consulting Engineering and Architects.

Hulshoff Pol, L. W., de Zeeuw W. J., Velzeboer C. T. M. and Lettinga G. (1983) Granulation in UASB-reactors. *Water Sci. Technol,* **15**, 291-304.

Hulshoff Pol, L. W. (1989) *The phenomenon of granulation of anaerobic sludge*, Ph.D. thesis, Wageningen University, Wageningen, The Netherlands.

Jewell W. J. (1994) Resorce Recovery Wastewater treatment. *American Sci*. Vol. 82. **pp** 366-375.

Jewell W. J. (1987) Anaerobic sewage treatment. *Environ. Sci. and Tech* **21**(1), 14-21.

Gijzen H. J. (2002) Anaerobic digestion for sustainable development: a natural approach, *Water Sci. Technol.,* **45(10),**321-328.

Kalogo Y, and Verstraete W. (1999) Development of anaerobic sludge bed (ASB) reactor technologies for domestic wastewater treatment: motives and perspectives. *World J. of Microbiol. Biotechnol.,* **15**, 523-534.

Kapp H. (1984) Schlammfaulung mit hohem Feststoffgehalt. Stuttgarter Berichtezur Siedlungswasserwirtschaft, Band 86, Oldenbourg Verlag, Munchen, 300 pp.

Kapp H. (1992) On-line messung der organischen sauren. Korrespondenz Alwasser **39**(7), 999-10ia04.

Lettinga G. (1996) Sustainable integrated biological wastewater treatment. *Water Sci. Technol*., **33(3)**, 85-98.

Lettinga G. (2001) Digestion and degradation, air for life. *Water Sci. Technol.,* **44**(8), 157-176.

Lettinga G. and Hulshoff Pol L. W. (1991) UASB process design for various types of wastewater. *Water Sci. Technol.,* **24**(8), 87 -107.

Lettinga G. Knippenberg van, Veenstra S. and Wiegant W. (1991) UASB law-cost sanitation research project in Bandung, Indonesia.

Lettinga G., Lier J. B. van, Shatanawi M., EI-Gohary F., Khatib A. and Polanco F. (1997) Environmental protection and resource conservation using integrated decentralized urban sanitation concepts. In: *Proceeding of Int. Conf. Water in the Mediterranean,* November 25-29, 1997, Istanbul, Turkey.

Lettinga G., Man A. W. A. de. Last A. R. M. van der, Wiegant W., Knippenberg K., Frijns J. and Buuren J. C. L. van (1993) Anaerobic treatment of domestic sewage and wastewater. *Water. Sci. Technol.*, **27**(9), 67-73.

Lettinga G., Rebac S. and Zeeman G. (2001) Challenge of psychrophilic anaerobic wastewater treatment. *Trends in Biotech.* **19**(9), 363-370

Lettinga G., Roersma R. and Grin P. (l983b) Anaerobic treatment of raw domestic sewage at ambient temperatures using a granular bed UASB reactor. *Biotech. and Bioeng*. **25**, 1701-1723.

Lier J. B. Van, and Lettinga G.(1999) Appropriate technologies for effective management of industrial and domestic wastewater: the decentralized approach *Water Sci. Technol*., **44**(8),157-176.

Mahmoud N. (2002) Anaerobic pre-treatment of sewage under low temperature (15 ºC) conditions in an integrated UASB-digester system. Ph.D. thesis, Wageningen University, Wageningen, The Netherlands.

Mahmoud N., Amarneh M. N., AI-Sa'ed R., Zeeman G., Gijzen H. and Lettinga G. (2003) Sewage characteristics as a tool for the application of anaerobic treatment in Palestine. Environmental Pollution, **126**, 115-122.

Mahmoud N., Zeeman G., Gijzen H. and Lettinga G. (2004) Anaerobic stabilization and conversion of biopolymers in primary sludge: effect of temperature and sludge retention time. *Water Res.,* **38**(4), 983-991.

Mahmoud N., Zimmo O., Zeeman G., Lettinga G. and Gijzen H. (2004) Perspectives for Integrated Sewage Management in Palestine and Middle East, *Water 21 (April 2004).*

McCarty P. L. (1981) On hundred years of Anaerobic treatment. *American Scientist.*(**82)**, 366-375.

McCarty P. L. (1985) Historical trends in the treatment of dilute wastewaters in Proceedings of the Seminar- *Workshop on Anaerobic Treatment of Sewage,* Switzenbaum, M.S., ed., Amherst, USA, 3-16.

McCarty P. L. and Smith D. P. (1986), Anaerobic wastewater treatment, *Environmental Sic. and Technol*., **20(12)**, 1200-1206

Metcalf & Eddy (1991) Wastewater Engineering-treatment, disposal, reuse. 3rd edition, McGraw HiIl Inc., New York, USA.

Metcalf & Eddy (2003) Wastewater Engineering-treatment and reuse, 4th edition, McGraw Hill Inc., New York, USA.

Mgana S. M. (2003) Towards sustainable and robust on-site domestic wastewater treatment for all citizens. Ph.D. thesis, Wageningen University, Wageningen, The Netherlands.

Nashashibi M. and van Duijl L. A. (1995) Wastewater characteristics in Palestine, *Water Sci. Technol.,* **32**(11), 65-75.

Pavlostathis S. G. and Giraldo-Gomez E. (1991) Kinetic of anaerobic treatment .*Water. Sci. Technol.,* **24**(8),35-59.

Pagilla K. R., Craney K. C. and Kido, W. H. (1997) Causes and effects of foaming in anaerobic sludge digesters. *Water Sci. Technol.*, **36**(6-7), 463-470.

Sanders W. T. M. (2001) Anaerobic hydrolysis during digestion of complex substrates. Ph.D. thesis: Department of Environmental Technology, Wageningen University, Wageningen, The Netherlands.

Sayed S. K. L (1987) Anaerobic treatment of slaughterhouse wastewater using the UASB process. Ph.D. thesis, Department of Environmental Technology, Wageningen University, Wageningen, The Netherlands.

Sayed S. K. J. and Fergala M. A. A. (1995) Two-stage UASB concept for treatment of domestic sewage including sludge stabilization process. *Water Sci. Technol.,* **32**(11), 55-63.

Schellinkhout A., lettinga G., van Velsen L., Louwe Kooijmans J. (1985). The application of UASB-reactor for the direct treatment of domestic wastewater under tropical conditions, *in Proceeding of the Seminar/Workshop on Anaerobic Treatment of Sewage*, Switzenbaum. M.S., ed., Amherst, UAS, 259-276.

Schellinkhout A. (1993) UASB technology for sewage treatment: experience with a full scale plant and its application in Egypt. *Water Sci. Technol.*, **27**(9), 173-180.

Seghezzo L. (2004) Anaerobic treatment of domestic wastewater in subtropical regions. Ph.D. thesis, Wageningen University" Wageningen, The Netherlands.

Scott McNiven W. K. Journey (1996), Anaerobic Enhanced Treatment of Wastewater and Options for Future Treatment. Washington, DC 20001.

Van der Last A.RM. and Lettinga G. (1992) Anaerobic treatment of domestic sewage under moderate climatic (Dutch) conditions using upflow reactors at increased superficial velocities. *Water Sci. Technol.*, **25**(7),167-178.

Van Lier J. B., and Lettinga G. (1999) Appropriate technologies for effective management of industrial and domestic waste waters: the decentralized approach. *Water Sci. Technol.,* **44**(8), 157-176.

Van Lier J. B., Tilche A., Ahring B. K., Macarie H., Moletta R., Dohanyos M., Hulshoff Pol L. W., Lens P. and Verstraete W. (2001) New perspectives in anaerobic digestion. *Water Sci. Technol.,* **43**(1), 1-18.

Visser A. (1995) The anaerobic treatment of sulphate containing wastewater. Ph.D. thesis, Wageningen Agricultural University, Wageningen, The Netherlands.

Wang Kaijun (1994) Integrated anaerobic and aerobic treatment of sewage. Ph.D. thesis, Department of Environmental Technology, Wageningen University, Wageningen, The Netherlands.

Wiegant W. M. (200l) Experiences and potential of anaerobic wastewater treatment in tropical regions. *Water Sci. Technol.,* **44**(8), 107-113.

Yamamoto S. , A lcauskas, J. B. and Crozier T. E. (1976). Solubility of methane in distilled water and seawater. *Journal of Chem. And Eng. Date*, **21**(1),79-80.

Yoda M. and Nishimura S. (1997) Controlling granular sludge formation in UASB reactors. *Water Sci. Tech*nol., **46**(6-7), 165-173.

Zeeman G. (1991) Mesophilic and psychrophilic digestion of liquid manure. Ph.D. thesis, Wageningen Agricultural University, Wageningen, The Netherlands.

Zeeman G. (1997), Decentralized sanitation systems. The role of anaerobic treatment in UASB- septic tank systems, *Workshop in integrated water quality Management,* The Netherlands, May 26-30, 1997,6 p.

Zeeman G. and Lettinga G. (1999) The role of anaerobic digestion of domestic sewage in closing the water and nutrients cycle at community level. *Water Sci. Technol*., **39**(5), 187-194.

Zeeman G., Kujawa R. K. and Lettinga G. (2001) Anaerobic treatment systems for high strength wastewater. In: DESAR, IWA, eds. Lens, P; Zeeman G. & Lettinga G.

Zeeman G., Kujawa R. K. and Lettinga G. (2001) Anaerobic treatment systems for high strength wastewater .In: DESAR, IWA, eds. Lens, P; Zeeman G. & Lettinga G.

Zeeman G., van Lier J. B. and Lettinga G. (2000) The role of anaerobic treatment at community and house on-site treatment of domestic wastewater. Effect of collection and transport systems. In: *Proceedings of the Technical Expert consultation Oil appropriate and innovative wastewater management for small communities in EMR countries.* WHO/CWEHA, Amman, Jordan, November 6-9,2.000.

Zehnder A. J., Ingvorsen K. and Mlarti T. (1982). Microbiology of methanogenic bacteria. In: Anaerobic Digestion. Elsesvier,Amsterdam, The Netherlands.
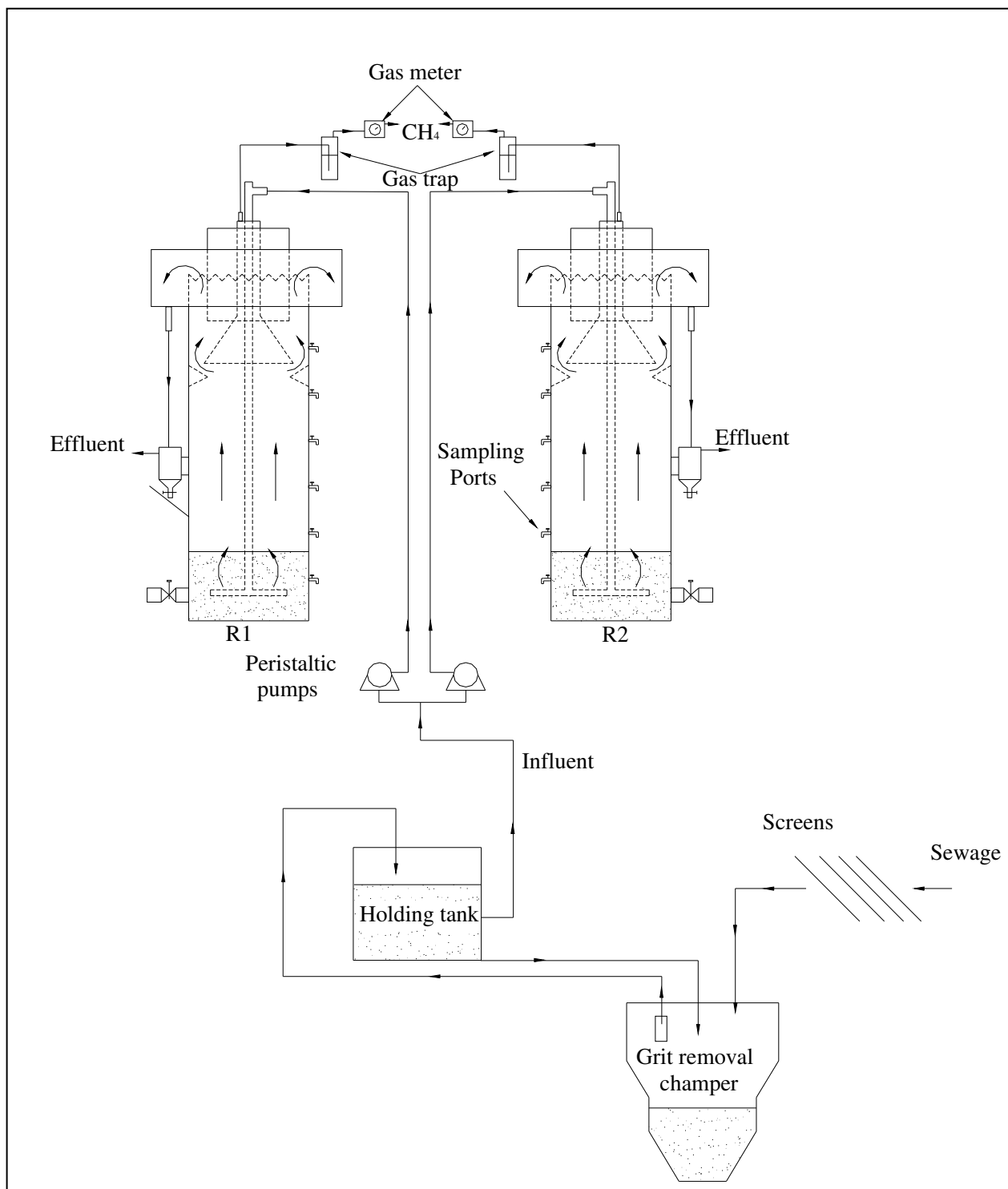
Appendixes

Figure A1. 1. Schematic diagram of the experimental set-up (not to scale).

Appendix 2.

Preparation of Biodegradability and Stability Bottles

The biodegradability and stability tests were carried out in batch reactors, serum bottles, of 500 ml with a headspace volume of 70 ml. The procedures for preparation of biodegradability and sludge stability bottles were as follow:-

1. Biodegradability Bottles

Each bottle of the biodegradability bottle was filled with 450 ml wastewater and 50 ml of specific media. The media is a mineral solution of macro nutrients, trace elements, 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

where 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 measurements were determined in triplicate.

1. Stability Bottles

The procedure for preparation of the sludge stability bottles was similar to the biodegradability bottles. However, each bottle of the stability test was filled with about 1.5 g COD-sludge/l instead of the wastewater, in addition to 50 ml of the same media prepared for biodegradability and completed to the 500 ml mark with tap water. The stability batches also incubated at 30°C for a period of 120 days. 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 (As described previously in Chapter 3).

Media solution preparation

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

- 20 ml macro nutrients stock solution, as prepared below in Table A2.l.

- 10 ml micro nutrients (trace elements), as prepared below in Table A2.2.

- 25g $NaHCO_3$ (buffer solution)

- 0.5 gm yeast extract.

 -Demineralized water: fill up the flask to 1000 ml mark.

Table A2.l. 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)*
NH_4Cl	0.28	35
KH_2PO_4	0.25	31.25
$CaCl_2.H_2O$	0.01	1.25
$MgSO_4.H_2O$	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 (g/l)	Weight to be added to 1000 ml flask as stock solution *
$FeCl_2.4H_2O$	2	2000
H_3BO_3	0.05	50
$ZnCl_2$	0.05	50
$CuCl_2.2H_2O$	0.038	38
$MnCl_2.4H_2O$	0.5	500
$(NH_4)_6MO_7O_{24}.4H_2O$	0.05	50
$AlCl_3.6H_2O$	0.09	90
$CoCl_2.6H_2O$	2.0	2000
$NiCl_2.6H_2O$	0.092	92
$Na_2S_2O_3.5H_2O$	0.164	164
EDTA $(C_{10}H_{16}N_2O_8)$	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.

Sodium Su1phid (Na$_2$S) solution preparation

The Na$_2$S solution was prepared fresh by dissolving 1.25 g Na$_2$S in 50 m1 demi water.
When the chemical compound available in the fom1 of Na2S.XH20 (X: 7-9); add 3.57g/ 50
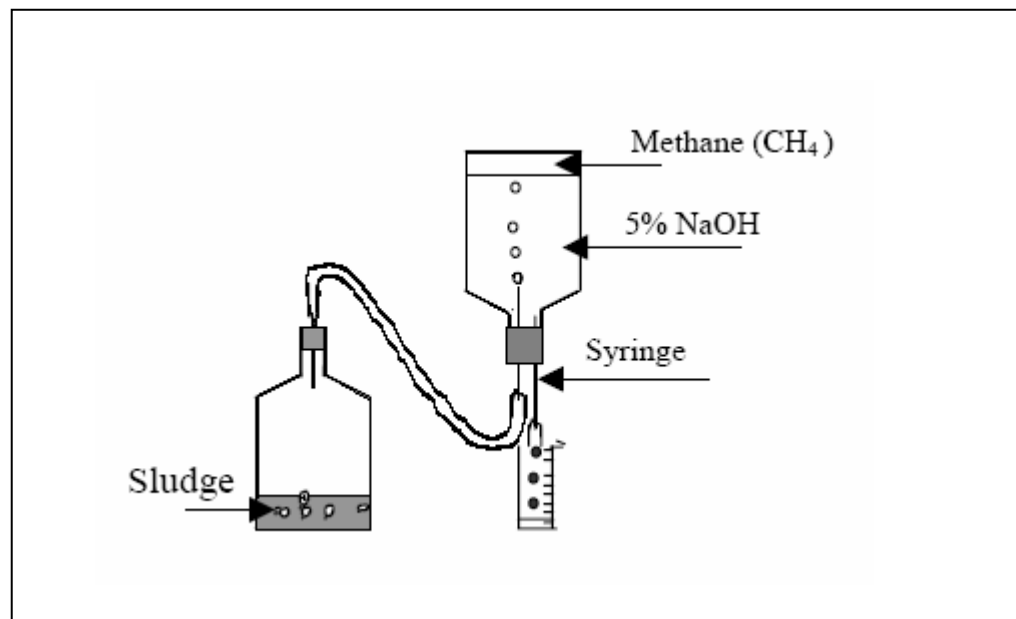ml. 1 ml of the prepared Na$_2$S solution was added to each batch bottle.

Gas measurement

Figure A2.1. A schematic diagram of CH$_4$ gas measurement of batch reactors used in determining sludge Stability and biodegradability by "serum bottle liquid displacement system.
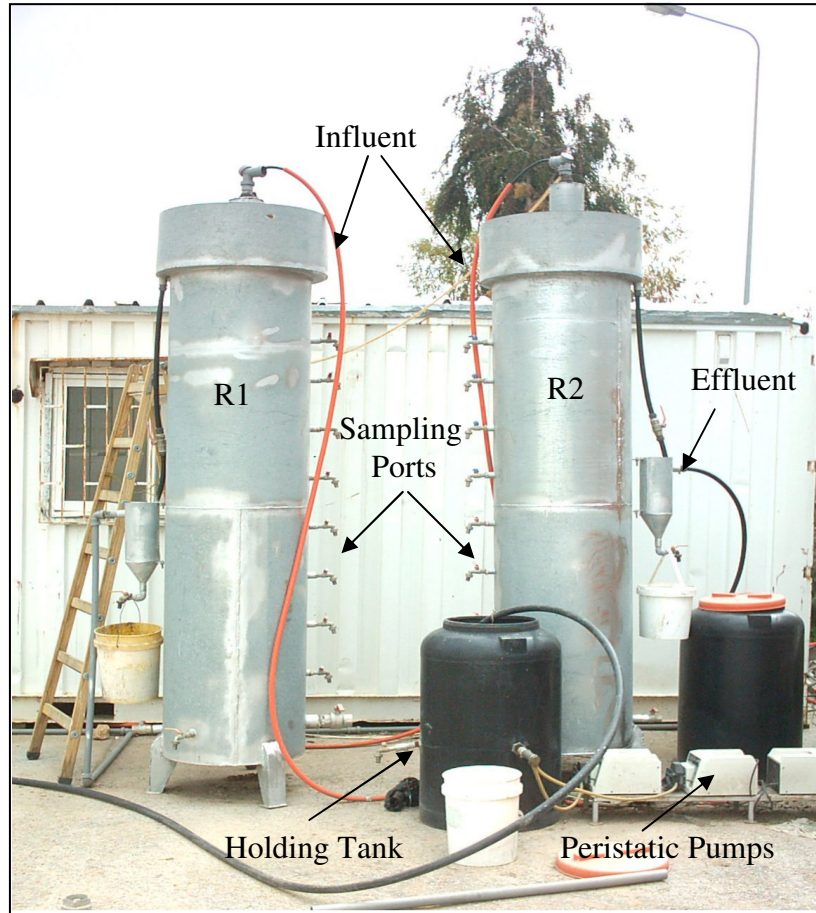
Appendix 3

Photo A3. 1. The UASB-septic tank pilot plants at Al-Bireh WWTP.
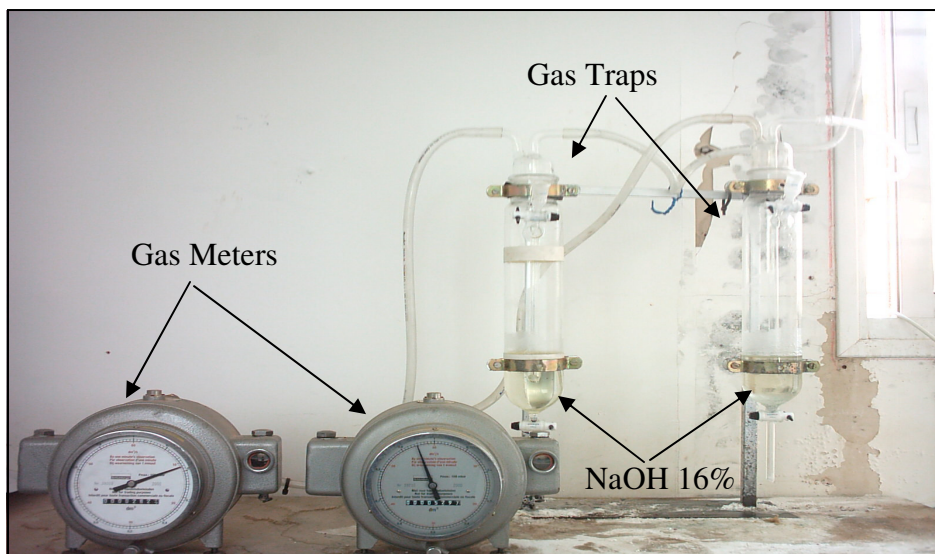
Photo A3.2. The gas meters and the gas traps with 16% NaOH inside. System in the photo. used to measure the methane gas produced from the UASB- septic tank pilot plant reactors.
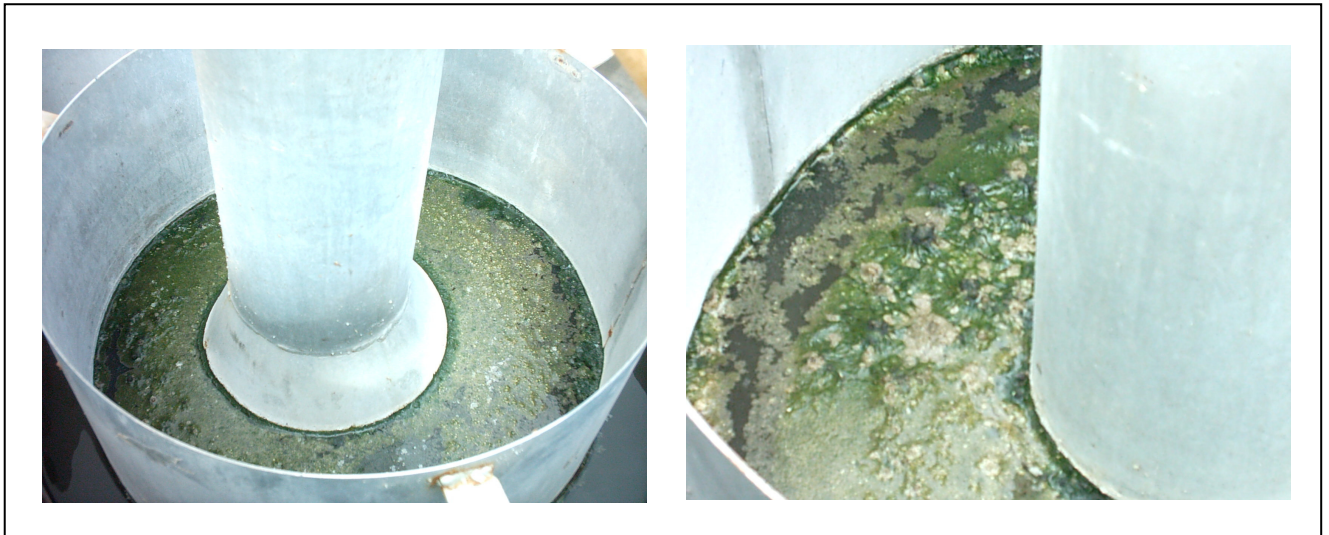

Photo A3.3 The scum layer phenomena inside the scum baffle in both reactors R1 (left) and R2 (right).

ملخص

نتيجة للتغيرات المناخية وازدياد عدد السكان والتقدم الصناعي في هذا العالم إنخفضت كمية الماء المخصصة لكل فرد من المياه الموجودة في الكرة الأرضية , وهذه الحالة تنطبق ايضا في فلسطين حيث بدأ الانسان بتلويث المياه الجوفية و السطحية و انقاص مواصفتها العذبة عن طريق تحميلها الكثير من المواد العضوية التي لا يمكن تحليلها بشكل طبيعي.

من هنا كان لابد من ظهور فكرة ضرورة المحافظة على المصادر المائية ومحاولة استخدامها الاستخدام الأمثل رغم الظروف الاقتصادية السيئة التي قد تعاني منها بعض الدول في هذا العالم , بحيث يجب العمل على تقليل مصادر تلوث المياه و العمل على معالجة المياه الملوثة و التي تحتاج الى تقنيات حديثة ومتطورة وعالية التكلفة. فلسطينيا و نتيجة للظروف الاقتصادية القاهرة الناتجة عن الاحتلال كان لابد من اختيار الطرق المثلى والمناسبة للحد من مصادر التلوث, و معالجة المياه العادمة . حيث لابد ان تكون الطرق المختارة ذات كفاءة عالية وسهلة التطبيق و قليلة التكلفة . وهنا تبرز فكرة استعمال نظام المعالجة الاهوائية "UASB-septic tank" لتكون بديلا للحفر الإمتصاصية وخزانات جمع القاذورات "septic tanks" المستخدمة بشكل كبير لتصريف المياه العادمة ومعالجتها بشكل أولي في فلسطين. حيث تبين أن هذا النظام الاهوائي يمثل بديلا ذي كفاءة جيدة قليلة التكاليف لتصريف و معالجة المياه العادمة بشكل صحي , إلا أن أداء هذا النظام لم يتم بحثه بشكل شمولي لمعالجة المياه العادمة القادمة من حي أو منطقة بأكملها وخصوصا في فلسطين وتحت الظروف المناخية السائدة في فترة شتائها .

إن الهدف الأساسي من وراء هذه الرسالة هو البحث في مدى اداء و جدوى استخدام التقنية الاهوائية " UASB- septic tanks " في معالجة المياه العادمة المنزلية القادمة من حي أو منطقة باكملها ضمن الظروف السائدة في فلسطين خصوصا خلال فصل الشتاء, أضف الى ذلك أن هذا البحث يبحث أيضا في تأثير مكوث المياه العادمة (HRT) داخل المفاعل الاهوائي على أداءه. ولهذا الغرض تم بناء مفاعلين (R1) و (R2) لمعالجة المياه العادمة المنزلية في المحطة الرئيسية لمعالجة المياه العادمة الخاصة بمدينة البيرة , حيث تم تشغيل المفاعلين بصورة متوازية لمدة ستة أشهر إمتدت خلال فترة الشتاء بحيث تم تشغيل المفاعل R1 بزمن مكوث للمياه العادمة (HRT) لمدة يومين بينما الأخر R2 فقد تم تشغيله لزمن مكوث للمياه العادمة (HRT) لأربعة ايام . حيث كان ذلك في درجات حرارة الجو العادية التي تراوحة بين (2 الى 27)^{0}م بمعدل 14.7 ^{0}م , بالاضافة الى ذلك استعملت مياه عادمة تراوحة درجة حرارتها خلال فترة الدراسة مابين (12 الى 25) ^{0}م بمعدل 17.3 ^{0}م و بمعدل تركيز من الأكسجين الكلي المستهلك كيميائيا (COD_{tot}) 905 ملغم/لتر وبنسبة مقدارها 1.97 بين (COD_{tot}/BOD_5) , أضف الى ذلك تبين ان نسبة المواد العالقة (TSS) في المياه العادمة المستخدمة هي نسبة عالية تصل الى حوالي 43.7 % من (COD_{tot}).

لقد تبين اثناء فترة الدراسة أن المفاعين R1 و R2 اثبتا كفاءة جيدة مستقرة نسبيا طوال فترة الدراسة حيث كانت معدلات ازالة الملوثات من المياه العادمة على النحو التالي في R1 لـ (COD_{dis} $COD_{tot},COD_{sus},COD_{col}$) هي (24%,20%,83%, 51%) على التوالي. أضف الى ذلك فإن نسبة الازالة للملوثات لـ (BOD_5) و (TSS) هي 45% و 74 %

على التوالي. أما النسبة للمفاعل الثاني (R2) فقد كانت معدلات إزالة الملوثات من المياه العادمة لـ (COD_{dis} $COD_{col}, COD_{sus}, COD_{tot}$) هي (28%, 10%, 87%, 54%) على التوالي , أما معدل إزالة الملوثات لـ (BOD_5) و(TSS) هي 49% و 78% على التوالي.

وكما هو متوقع فقد تبين من النتائج لكلا المفاعلين أنهما غير فعالان لإزالة المواد العضوية (nutrient) مثل (NH_4^+-N) و (Nk_j-N) الذان أبرزا إرتفاعا في معدل إزالة هذه الملوثات من المواد العضوية مقارنة بفترة الصيف.

لقد بينت الدراسة ونتائجها أن التغير في إنتاج كمية غاز الميثان من المفاعلين كان يعتمد وبشكل كبير على درجة حرارة الجو و الوضع البكتيري في كل مفاعل , حيث كان معدل إنتاج غاز الميثان الكلي خلال فترة الدراسة و تحت الضروف المعيارية حوالي 0.11 م^3 /كجم COD تم ازالته للمفاعل R1 و 0.10 م^3 /كجم COD تم ازالته للمفاعل R2 .

من خلال مراقبة نمو الحمأة "sludge " داخل كل مفاعل تبين أن المفاعلين سوف يصلا الى حالة الإمتلاء من الحمأة خلال اربع سنوات و بالتالي يجب إفراغهما , أضف الى ذلك فقد سجلت الدراسة نسبة (VS/TS) للحمأة حوالي 67.2 للمفاعل R1 و67.9 للمفاعل R2 , حيث أن مثل هذه النسبة تعطي إشارة الى وصول الحمأة الى حالة السكون أو الإستقرار جيدة .

وأخيرا بينت النتائج المستخلصة من الدراسة ان المفاعل R2 ذي الأطول زمن مكوث للمياه العادمة (HRT) كان يعطي معدلات ازالة للملوثات أفضل من R1 في أغلب الإختبارات التي تمت عليهما , إلا أن هذه المعدلات في أغلبها لم تثبت بشكل قطعي من خلال التحاليل الإحصائية.

ختاما يمكن القول أن نظام المعالجة اللاهوائي (UASB- septic tanks) هو نظام جيد لمعالجة المياه العادمة المنزلية ويمكن تطبيقه بمقاييس مختلفة ضمن الظروف البيئية السائدة خلال فترة الشتاء في فلسطين