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Community On-site Anaerobic Sewage Treatment In a UASB-Septic Tank System

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Birzeit, 2005

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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.

DITICATION

To My Parents, My Brothers,

My Sisters and My Friends

with love and respect,

Mohammad N. Al-Shayah
January, 2005

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لكم جميعاً جزيل الشكر.

Abstract

In Palestine, cesspits are the most known and commonly applied on-site methods for wastewater disposal and sewage pre-treatment. However, the present practical mode of those low-rate anaerobic pre-treatment units can pose a significant risk to public health and to the environment. Therefore, a sanitation intervention is needed and consequently the demand for effective but low-cost wastewater treatment technologies for communities in Palestine, particularly the rural areas, is definitely great. On the basis of already available technical information concerning the UASB-septic tank system performance, the system represents a viable and affordable on-site sanitation alternative for household. However, the performance of these systems in an actual community on-site situation has so far not been investigated especially under Palestine conditions, where the domestic wastewater is characterized by high strength with considerable solids content; and fluctuation in seasonal temperature. Furthermore, the design criteria of the UASB-septic tank system are still to be formulated in Palestine.

The main objective of this thesis was to investigate the performance and feasibility of using the UASB-septic tank reactor for the pre-treatment of domestic wastewater under the conditions that arise at community level in Palestine. Moreover, possibilities to evaluate the influence of HRT on the performance of the UASB-septic tank reactor had also been made, in attempt to optimize the design of the UASB-septic tank system. Community on-site two pilot scale UASB-septic tank reactors treating domestic sewage under two different HRTs (2 days for R1 and 4 days for R2) were operated in parallel at the sewage treatment plant of Al-Bireh City, Palestine. The two reactors were operated for six months at ambient temperature fluctuates between 15 and 34ºC with an average value of 24.2ºC. Mean sewage temperature during the experiment was 24°C with 18.2 and 29ºC extreme values. The wastewater in the study area was characterized by a high concentration of COD_{tot} with an average value of 1189 mg/L, and with a large fraction in the COD_{sus} form around 54% (640 mg/L). Moreover, the raw wastewater was highly

biodegradable with an average value of 65% and COD: BOD_5 ratio of 2.0. The performance data obtained via regular monitoring of the two reactors showed average removal efficiencies for COD_{tot}, COD_{sus}, BOD_5 and TSS of 54, 85, 56 and 79%, respectively for R1. Likewise, the removal efficiencies in R2, for the same parameters were 58, 89, 59 and 80%. R2 was achieved slightly better removal efficiencies compared with R1. The longer HRT imposed to R2 had a significant effect on the COD_{tot}, COD_{sus}, BOD_5 and TSS removals. The results of statistical tests on the removal efficiency data sets of the previous parameters also confirmed the enhanced performance of R2 ($\rho<0.05$). This suggests that the design HRT = 4 days in UASB-septic tank reactors seems more adequate for the anaerobic treatment of domestic sewage under Palestine conditions. The results revealed that the removals of COD_{col} and COD_{dis} correlated well with increases in temperature and microbial adaptation. The average COD_{col} and COD_{dis} removals during the whole period of study were respectively 27 and 12% for R1; and 32 and 14% for R2. The results also revealed that the evolution of biogas production varying and strongly affected by temperature and ecology of both reactors. The average total methane production (gas form + liquid form) from both reactors was 0.1 $Nm^3/kgCOD_{removed}$. The observations made to sludge hold-up in both reactors concluded that, the sludge volume was not increased during the 6 month of operation, however, the sludge concentrations were increased with average values of 46.8 gTS/L and 48.6 gTS/L respectively for R1 and R2 during the whole period, as compared to the first operational period (13.78 gTS/L), indicating the sludge accumulation. Therefore, sludge withdrawal from the reactors is deemed to be after long time of operation. Finally, as a general conclusion, it could be said that the one-step UASB-septic tank reactors configuration is a potential compact and effective community onsite pre-treatment unit for domestic wastewater in Palestine.

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List of abbreviations

A:	acidification
atm.:	atmospheric
AF:	anaerobic filter
AH:	anaerobic hybrid
AVR:	average
BOD:	biological oxygen demand
CFU	colony forming unit
COD:	chemical 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 bed
g:	gram
GLS:	gas-liquid-solid three phase separator
FB:	fluidized bed
H:	hydrolysis
hr:	hour
HRT:	hydraulic retention time (days or hrs)
HUSB:	hydrolysis upflow sludge bed
in.:	inch
inf.:	influent
L:	liter
M:	methanogenesis
m:	meter
mg:	milligram
ml:	milliliter
N:	nitrogen
NGOs:	nongovernmental organizations
NKj:	kjeldhal nitrogen
nm:	nanometer
OLR:	organic loading rate (kg COD/m^3.d)
P:	phosphorous
R:	reactor
RPF:	reticulated polyurethane foam
rpm:	revolutions per minute

S:	sulphate
SRT:	sludge retention time
SS:	suspended solids
STD:	standard deviation
SVI:	sludge volume index (ml/g)
T:	temperature (°C or °K)
TS:	total solids
TSS:	total suspended solids
UASB:	upflow anaerobic sludge blanket
V:	volume (m^3)
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:	significance level
μm:	micrometer

Chapter 1

Introduction

1.1 Background

The need for adequate treatment of domestic wastewater is self evident in Palestine particularly for small rural communities, in which about 60% of the total populations in Palestine are living. The primary mode of wastewater disposal in rural communities is cesspits, which are installed on-site at residential dwellings and often associated with inefficiency, poor maintenance and groundwater pollution (PECDAR, 2001; CDM, 2002).

PECDAR (2001) reported that this present situation for wastewater collection and the lack of adequate treatment profound risks to Palestinians; and the resulting pollution poses public health risks and aquifer damage (ARIJ, 2001). A sanitation intervention is needed. Therefore, setting up an effective wastewater management system is given the highest priority in rural Palestine according to the Palestinian Environment Strategy (PES) and was categorized on top of the PES eleven elements defined by Ministry of Environmental Affairs that need immediate action such introducing of new technologies for small-scale wastewater treatment plants that could be applied in rural areas (MEnA, 1999).

The alternative new treatment systems for such small communities in Palestine essentially should be sustainable, plain, low-cost, and effective for environmental protection and resource conservation. A number of systems can be formulated however, only some of them can be considered as sustainable, complying with the general sustainability criteria as proposed by Lettinga *et al.* (1997) (Table 1.1).

Al-Sa'ed (2000) reported that in many cases sewage treatment through conventional centralized wastewater treatment technologies, such as the aerobic activated sludge process, is inappropriate for the physical and economic characteristics of the small communities. Hence, non-point pollution, caused by direct discharges from rural communities can be significantly reduced by the promotion of small on-site low cost treatment systems. In addition to the high costs of the conventional systems, they may even be technologically inadequate to handle the locally produced sewage. For example, in comparison to the sewage in Europe and United States, domestic wastewater particularly in arid areas of e.g. the Middle East, are more concentrated (up to 5 times) (Mahmoud *et al.*, 2003). The amount of oxygen demand per m^3 of sewage is extremely high and, consequently, the excess sludge production is huge. Therefore, extremely high operational and maintenance costs and high losses of energy are experienced in case conventional aerobic treatment methods, like the activated sludge systems, are applied.

Table 1.1. Criteria for sustainable environmental protection concepts (Lettinga *et al.*, 1997)

- No dilution of high strength residues (wastes) with (clean) water, i.e. for conveying them from the site where they are produced (i.e. installation of expensive sewerage).
- Maximum of recovery and re-use of treated water and by-products obtained from the polluting substances, i.e. for irrigation, fertilisation etc.
- Application of efficient, robust and reliable treatment/conversion technologies, which are low-cost (in construction, operation and maintenance), which have a long life-time and are plain in operation and maintenance.
- Applicable at any scale, very small and very big as well.
- Leading to a high self-sufficiency in all respects.
- Acceptable for the local population.

It maybe concluded that implementation of conventional centralized wastewater treatment systems which depends on the presence of a large and expensive sewage network, is highly questionable, especially for rural areas in Palestine which are still lacking adequate sanitation. Furthermore, the classically applied centralized conventional sanitation concepts completely clashes with the sustainable criteria listed in Table 1.1.

Decentralized sewage treatment is more and more considered to be a sustainable way of wastewater treatment (Zeeman and Lettinga, 1999). In the United States, on-site treatment (mainly septic tank) for domestic sewage serves about 20% of the US population, more than 20 million houses (Scandura and Sobsey, 1997). Therefore, decentralized treatment can represent a sustainable option for the treatment not only for rural areas in developing countries, but also for unserved areas with wastewater collection and treatment facilities in developed countries (Zeeman and Lettinga, 1999; Elmitwalli *et al.*, 2003; Mahmoud *et al.*, 2003).

Anaerobic treatment of sewage represents a low cost and sustainable technology for domestic wastewater treatment (Lettinga *et al.*, 1993; Zeeman *et al.*, 2000). 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). Zeeman *et al.* (2000) argued that anaerobic treatment of domestic wastewater in a UASB-septic tank system could be profitable for household and community on-site. The UASB-septic tank system differs from the conventional septic tank in the modified upward direction of the influent, which enables better substrate and sludge contact and so better conversion and removal efficiency. However, so far little if any experience is available on the performance and design of these reactors under the environmental conditions and wastewater characteristics of Palestine. This research aims at increasing the knowledge on the design and process performance of the UASB-septic tank for domestic wastewater treatment in Palestine.

1.2 Sanitation in Palestine- Existing situation

1.2.1 Wastewater management

The sewage infrastructure and wastewater management in Palestine had been neglected over the past years (Nashashibi, 1995; Mahmoud, 2002). As a result of prolonged neglect and increasing poverty, the rural areas in the West Bank and Gaza Strip suffer from underdevelopment of their physical, economical and social infrastructures especially from a lack of safe and adequate water supply and proper sanitation facilities (CDM, 2002).

It should be noted that the situation for wastewater collection and treatment is extremely critical in both urban centers and rural areas of Palestine. Approximately 70% of the West Bank population is not served with sewage networks, and uses mainly cesspits and occasionally septic tanks. Thus the other 30% is served with sewage networks, but less than 6% of the total population is served with treatment plants (Mahmoud *et al.*, 2003). Al-Sa`ed (2000) reported that the major sanitation problems in Palestine are due to the weak economy and low income, low level of technical operating expertise and very limited access to the existing advance wastewater treatment technologies.

The lack of sufficient wastewater management in both the West Bank and Gaza Strip highly contributes to the water resources depletion and water quality deterioration. It has also a direct impact on problems related to public health, shoreline and marine pollution in Gaza, deterioration of nature and biodiversity as well as landscape and aesthetic distortion (MEnA, 1999; ARIJ, 2004).

1.2.2 Wastewater collection and treatment

Currently, sewage networks serve approximately 28% and 66% of the West Bank and Gaza Strip population, respectively (ARIJ, 2004) (see Table 1.2). They are limited to major cities and refugee camps but most of them are poorly designed and suffer from leakage. The remaining population uses cesspits for wastewater disposal or septic tanks in some cases.

Cesspits (or cesspools) are the traditional method for sewage disposal in Palestine. It has been used for centuries in all the communities before they were slowly replaced in the major cities by the sewage collection networks. However, they are still in the villages and the rural communities. About 73% of the households in the West Bank have cesspit sanitation and almost 3% are left without any sanitation system (MOPIC 1998). Cesspits are essentially covered pits that receive raw sewage. They are dug into pervious soils. Most of the cesspits are left without a cement basement of liner so that sewage infiltrates into the earth layers and the owners avoid using the expensive services of the vacuum tankers to empty them (ARIJ, 2004). Therefore, cesspits themselves constitute a threat to freshwater if they overflow, as frequently happens, they contaminate the soil and groundwater with raw sewage. If they are pumped out, the sewage is usually dumped into the nearest water body without being subjected to any kind of treatment.

A better on-site sanitation method than cesspits is the septic tank. The septic tank is an underground covered watertight settling tank that collects and provides primary treatment of wastewater by holding the wastewater in the tank and allowing settleable solids to settle to the bottom while floatable solids (oil and grease) rise to the top. Up to 50% of the solids retained in the tank decompose, while the remainder accumulate as sludge at the bottom of the tank and must be removed periodically by pumping the tank. The effluent form the septic tank is either disposed of through soil absorption fields, e.g. trenches or beds, provided that site characteristics are appropriate, or subjected to further treatment employing a sand filter (USEPA, 2000).

While the septic system is a simple disposal method and provides primary treatment of the raw sewage, misapplication of the technology is common. Various NGOs with varying degree of success have piloted a version of the septic system in some portions of some of Palestinian villages. Main problems seem to be with the poor quality of construction and villagers' expectations of the system (CDM, 2002). Factors that have hampered its widespread application versus cesspits are: it requires a larger land area and that it is more costly and its operational cost is higher due to the need for periodic desludging (Coelho *et al.*, 2003).

There are eight central treatment plants in the West Bank and Gaza Strip. Five of them are located in the districts of Ramallah, Jenin, Tulkarm and Hebron in the West Bank and the rest are found in Gaza, Rafah and North Gaza districts in Gaza Strip. However, all of the existing treatment plants except Al Bireh plant haven't been well maintained and are presently either not functioning such as Hebron and Jenin plants or functioning at very low efficiency rate such as Tulkarm and Ramallah plants (see Table 1.2).

This present situation of the WWTPs in Palestine can be mainly attributed to the overloading in general, misconception in planning, design and operation; and insufficient capacity of the mechanical and electrical plant in particular (PECDAR, 1994; ARIJ, 2004). Table 1.2 shows some of the data related to wastewater management in the West Bank and Gaza Strip.

In sparsely populated Palestinian poor rural and semi-urban communities, which form about 60% of the total population in the West Bank, few small sewage treatment plants were installed for the protection of aquatic environment (Al-Sa`ed, 2000). Such facts indicate that all the wastewater, whether from treatment plants, sewage networks or cesspits, is discharged raw into open areas including Wadis where water streams flow, agricultural lands, and dumping sites end into the sea and groundwater.

1.2.3 Wastewater characteristics

In general, wastewater in Palestine is characterized as being of "high strength" (ARIJ, 1996; CDM, 2002; Mahmoud *et al.*, 2003). The amounts of Biological Oxygen Demand (BOD), Chemical Oxygen Demand (COD), Kjeldhal Nitrogen (NKj) and Total Suspended Solids (TSS) in the wastewater is relatively high compared to other countries and according to the sewage strength classification proposed by Metcalf and Eddy (1991). The high strength of sewage can be attributed to low water consumption, industrial discharges, and people's habits (Mahmoud *et al.*, 2003). In addition, the generated sewage in rural communities could be more concentrated, because of the lack of water and the extreme frugality with which villagers use water.

Although, light industries are prevailing in some localities in Palestine; domestic wastewater, which is generated from residential, commercial, institutional and public buildings, is expected to be the most significant contributor to the waste streams in most communities. Hence, as in existing conditions, it is expected wastewater from industrial enterprises will continue to be pre-treated on-site in cesspits, as with stone and brick processing by-products or slaughterhouse wastes streams. Table 1.3 shows domestic wastewater characteristics of some cities and rural areas in the West Bank.

Table 1.2. Data related to the existing situation of the sanitation in the West Bank and Gaza Strip (ARIJ, 2004)

District	Wastewater treatment plant (WWTP)	WWTP Capacity (m^3/day)	WWTP Status	WWTP Efficiency (%)	% of population connected to the public sewage network	No. of sites where raw wastewater is discharged directly into the environment
WEST BANK						
Bethlehem					40	25
Hebron	Deir Samit WWTP	15	Operating well	83	22	73
	Hebron WWTP	6,742	Not functioning	0		
Jerusalem					23	11
Ramallah	Al Bireh WWTP	3,600	Operating well	95	24	54
	Ramallah WWTP	1,276	Not functioning well	5		
Salfit					8	19
Tubas					11	9
Nablus	Sarra WWTP	50	Constructed but hasn't been yet operated		51	31
Tulkarm	Tulkarm Cesspools	5,000	Overloaded	15	31	44
Qalqiliya					41	29
Jenin	Jenin WWTP	1,000	Not functioning (heavily overloaded)	0	12	62
Jericho					0	6
Total					**28**	**363**
GAZA STRIP						
Deir Al-Balah					50	1
Gaza	Gaza WWTP	51,000	Operating well	60	85	1
Khan Yunis					2	15
North Gaza	Beit Lahiya WWTP	12,000	Not functioning well	40	71	2
Rafah	Rafah WWTP	5,567	Not functioning well	40	62	1
Total					**66**	**20**

Table 1.3. Sewage characteristics of some cities and villages in the West Bank-Palestine

Location	BOD_5	COD	NKj	NH_4^+	Total P	PO_4^{3-}	SO_4^{2-}	TSS	pH	*Reference*
Al-Bireh City		1586	104	80	13	12.9	138	736	7.26	Mahmoud *et al.*, 2003
	750	1230	37	27		4.3	61			Nashashibi, 1995
Ramallah City		2180	99	58	12.8	12.4	975	729	7.45	Mahmoud *et al.*, 2003
	525	1390	79	51		13.1	132	1290		Nashashibi, 1995
Nablus City	739							1408		ARIJ, 2004
	570							1285		ARIJ, 2004
	1185	2115	120	104		7.5	137	1188		Nashashibi, 1995
Jenin City	1100	1440			46	15.3		1088	7.5	PECDAR, 1994
Tulkarm City	250	540			17.9	5.96		398	6.5	PECDAR, 1994
Bethlehem City		2720						1080		Nashashibi, 1995
	660	2724			141.4	45.6		688	6.5	PECDAR, 1994
Hebron City	1025	3050	255		129	16.5	220	25131		CH2MHILL, 2001
	520	2736			413.8	133.5		1794	6.0	PECDAR, 1994
Al-Jalazoun R.Camp		1489	71	56.2	15	11.9	213	630	7.31	Mahmoud *et al.*, 2003
Surda Village	214							1763		ARIJ, 2004

All units are in mg/l except: pH no unit and NH_4^+ measured as N

1.3 Decentralized wastewater management system

Decentralized sanitation could be a new perspective and sustainable approach for wastewater management in Palestine, particularly for small rural communities where population is sparse, water supplies is intermittent and safe sanitation facilities are absent. In addition, decentralized sanitation seems to be an economically and ecologically sound alternative to the traditional centralized urban wastewater management systems (Wilderer and Fall, 2001). In peri-urban areas in low-income countries, conventional centralized approaches to wastewater management have generally failed to address the needs of communities for the collection and disposal of domestic wastewater from on-site sanitation (Zeeman *et al*., 2001; Parkinson and Tayler, 2003). The major reason for failure is that the conventional sewerage systems, "end-of-the-pipe" technology that are normally accompanied with centralized wastewater treatment plants are certainly far too expensive and complex for poor countries (Zeeman *et al*., 2001).

The decentralized wastewater management system is meant by small, individual or cluster type decentralized wastewater treatment systems implies collecting, treating and re-using the wastewater from individual homes and/or clusters of homes at or near the point of wastewater generation. Therefore, implementing wastewater management systems based on a decentralized approach that may create possibilities for wastewater re-use and resource recovery close to the point of origin; also offer opportunities to separately collect and treat the different wastewater streams (Zeeman and Lettinga, 1999) as well as improvements in local environmental health conditions, reduce energy use and water consumption, prevent water pollution, reduce the tremendous costs associated with the installation of sewers and pumping stations, and stimulate energy production (Lettinga *et al.*, 1997; Van Lier *et al.*, 1999).

1.4 On-site anaerobic sewage treatment

So far, anaerobic treatment of domestic wastewater is mostly applied as an off-site treatment system in, e.g. Colombia, Brazil and India, replacing the more costly activated sludge processes or distinctly diminishing the required pond areas (Vieira and Souza, 1986; Draaijer *et al.*, 1992; Schellinkhout and Osorio, 1994; Lettinga, 1996). On the other hand, various cities in Brasil, e.g. Campina Grande, show interest in applying anaerobic as a decentralised on-site treatment system for “sub-urban”, poor, districts. Application of modified UASB reactors for single households, not connected to the centralized sewerage system, was studied under Dutch (low) and Indonesian (high) ambient temperatures. The UASB process was also applied to treat sewage from small-size communities (235 houses) in Brazil (Vieira *et al.*, 1994), and a pilot-scale single-step community on-site UASB reactor was also operated for a long period in a University in Tanzania (Mgana, 2003). Results form all showed that it is feasible to attain high COD removal efficiencies (Tables 2.4 & 2.5).

1.4.1 Alternative on-site systems for a single house

The septic tank is the most known and commonly applied method for on-site (anaerobic) treatment of sewage. However, the observed poor performance of septic tanks treating domestic wastewater from the literature (Mgana, 2003; Lettinga *et al.*, 1991) show that septic tanks operated in the present practical mode are not suitable as on-site treatment option for wastewater. Mgana (2003) found that the observed poor performance of the community on-site septic tank despite the long HRT is mainly attributed to the inherent design feature of septic tank, viz. the horizontal flow mode of the influent sewage in septic tanks. The horizontal flow mode of the sewage in septic tanks is the predominant design feature responsible for the insufficient contact between the influent and the active biomass available in the settled sludge. Most of the substrate from the horizontal flow mode in septic tanks reaches the active biomass by trickling through the sludge downwards from top. This is a very inefficient mechanism of enhancing contact between substrate and active microorganisms.

This implies that for the septic tank to perform better; improvements need to be made in its design. However the most essential features that need to be incorporated in the common septic tank in order to improve this most likely will lead to application of the Upflow Anaerobic Sludge Bed (UASB) reactor (Bogte *et al.*, 1993). A significant low-cost/ low-tech improvement of the septic tank may be achieved by applying modern reactor technology to the system, i.e. upward flow and gas/solids/liquid separation at the top (Zeeman *et al.*, 2000). This modification will lead to a so called UASB-septic tank system (Bogte, *et al.*, 1993; Zeeman *et al.*, 2000) because the system shares features of both methods. Sludge gradually accumulates in the reactor, as in septic tanks, but it is operated in upflow mode, as UASB reactor.

1.4.2 Alternative on-site system for a cluster of houses

In certain cases, it is more appropriate to employ a wastewater management system for a cluster of houses rather than installing individual ones for each single house. In such cases, there is a need to install a sewage collection system. Small diameter gravity and pressure sewers are appropriate for small communities as they are affordable and less water-intensive alternatives to the conventional sewerage collection systems. The UASB-septic tank could be profitable to be applied in such cases; even though for small communities with densely-populated areas, like a UASB-septic tank for each street.

1.5 Aim of research

This research aims to promote a viable and affordable on-site sanitation alternative for rural communities in Palestine that increases environmental protection and resource conservation by pilot-testing the UASB-septic tank system for anaerobic wastewater treatment of actual domestic sewage under Palestine local conditions. Hence, 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.

On the basis of already available technical information concerning the UASB-septic tank system performance (Lettinga *et al*., 1991; Bogte *et al*., 1993; Lettinga *et al*., 1993), the system represents an effective and low-cost onsite pre-treatment system for both black and total domestic wastewater. In Palestine, few investigations and researches had been done during the last years on such system (Al-Juaidy, 2001; Ali, 2001). However, the previous researches were of short periods and thus did not consider the influence of temperature fluctuations. Moreover, the previously researched reactors were mostly fed with wastewater from Birzeit University or septage, and no research had so far considered real domestic wastewater.

Moreover, 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 in many cases is difficult, as too many factors affect the anaerobic degradation and reactor performance, and each research carried out under different conditions.

1.6 Research objectives

The main objective of this research is to assess the performance of the UASB-septic tank for domestic sewage treatment under Palestinian/ Middle East conditions.

The specific objectives of this research are:

- Assessment of the UASB-septic tank pilot plants performance for treating domestic (municipal) wastewater under Palestinian conditions. The reactors performance will be evaluated in terms of process efficiency (COD total and fractions, Volatile Fatty Acids (VFAs), solids, ammonia, kjeldhal, phosphate and sulphate) and process stability through monitoring the quantity of biogas produced, sludge stability, sludge bed flotation and 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;

- Gaining hands-on experience in the operation and monitoring of anaerobic sewage treatment plants by pilot plant studies.

1.7 Thesis structure

This thesis consists of six chapters. Chapter 1 is the research introduction in which background, aim of the research and objectives are introduced. Chapter 2 provides a comprehensive literature review on anaerobic treatment of domestic wastewater, UASB-septic tank concepts and designs. Chapter 3 deals mainly with materials and methods used in this experimental research. The results of this research are presented and discussed in Chapter 4. Finally, conclusions and recommendations are summarized in Chapter 5.

Chapter 2

Literature Review

2.1 Introduction

Historical evidence indicates that the anaerobic digestion process is one of the oldest technologies applied for the treatment of sewage (McCarty, 1985). Anaerobic processes have been used for the treatment of concentrated domestic and industrial wastewater for well over a century (McCarty and Smith, 1986). The simplest, oldest, and most widely used process is the septic tank (Jewell, 1987). The first full-scale applications of anaerobic treatment was for domestic wastewater in the 1860's, in an air-tight chamber with a configuration more like a septic tank, and was called "Mouras' Automatic Scavenger". This invention was enthusiastically defined at that time as "the most simple, the most beautiful, and perhaps, the grandest of modern inventions" (McCarty, 1985). Afterwards, the application of the anaerobic treatment was studied by various researchers, Scott-Monrief in 1891, Cameron in 1895, Imhoff in 1905, Winslow and Phelps in 1910 and Coulter, Soneda and Ettinger in 1957 (McCarty, 1985). McCarty (2001) provided a summary of the development of anaerobic treatment, with some considerations about its future.

Anaerobic treatment is becoming more widely accepted for the treatment of domestic wastewater after the knowledge gained during the operation of several municipal anaerobic plants all over the world (Schellinkhout, 1993). High organic loading rates and low sludge production are among the many advantages anaerobic processes exhibit over other biological unit operations. But the one feature emerging as a major driver for the increased application of anaerobic processes is the energy production. Not only does this technology have a positive net energy production but the biogas produced can also replace fossil fuel sources (Batstone *et al.*, 2002).

The energy crisis of the seventies greatly stimulated engineering research on anaerobic digestion of domestic and industrial wastewaters, and resulted in the development of a new generation of high rate anaerobic system designs based on biomass recycle or on biomass retention independent of waste flow. This reduced reactor volume requirements and improved process stability and control, counteracting the early feelings of unreliability associated with anaerobic treatment (Wilkie and Colleran, 1988) and led to a world-wide acceptance of anaerobic wastewater treatment (Van Lier *et al.*, 2001).

Among the different treatment systems now available worldwide, the anaerobic process is attracting more and more the attention of sanitary engineers and decision-makers. It is being used successfully in tropical countries, and there are some encouraging results from subtropical and temperate regions (Elmitwalli, 2000; Halalsheh, 2002; Mahmoud, 2002; Mgana, 2003; Seghezzo, 2004). Consequently, anaerobic treatment is increasingly recognized as a core method technology for environmental protection and resource conservation (Lettinga, 1996; Lettinga, 2001). Furthermore, application of anaerobic treatment creates the possibility for implementation of economically attractive sanitation concepts, which is of particular importance for developing countries. Advantages and drawbacks of anaerobic sewage treatment, with special emphasis on high rate reactors, are summarized in Table 2.1 and Table 2.2, respectively.

Table 2.1. Advantages of anaerobic wastewater treatment

- *High efficiency*. Good removal efficiency can be achieved in the system, even at high loading rates and low temperatures.

- *Flexibility*. Anaerobic systems can easily be applied at any scale, enabling a decentralized application; and are being able to treat wide range of waste streams.

- *Simplicity*. Anaerobic reactors are relatively simple in construction and operation as little equipment is needed.

- *Stability*. Better process stability to handle shock loads and toxic substances, due to long SRT and larger biomass inventory.

- *Low energy cost*. Low operational and maintenance costs compared to aerobic conventional systems, as no energy is required for aeration, mixing and moving parts; on the contrary energy is produced in the form of methane gas.

- *Low space requirement*. The anaerobic systems can handle high hydraulic and organic loading rate. Thus, those systems are rather compact and reduce the facilities required for sludge handling and post treating stages. Consequently, reduce the investment costs.

- *Low sludge production*. The sludge production is low, when compared to aerobic methods. The sludge is well stabilized for final disposal and has good dewatering characteristics. Consequently, lower sludge disposal costs due to longer storage and greater digestion.

- *Low nutrients requirement*. Due to low growth yield of methanogenic and acetogenic organisms, the nutrients (N, P and such like) requirements are low compared to aerobic methods. Moreover, in anaerobic treatment nutrients are conserved which give potential for crop irrigation.

Table 2.2. Drawbacks of anaerobic wastewater treatment

- *Long start-up*. Longer start-up period is required compared to aerobic processes, due to the low growth of methanogenic organisms, when adequate inoculum is not available.

- *Low pathogen and nutrients removal*. Pathogens are only partially removed and the removal of nutrients is not complete.

- *Necessity of post-treatment*. Post-treatment of the anaerobic effluent is generally required to reach the discharge standards for organic matter, nutrients and pathogens.

- *Possible bad odors*. When treating S-rich wastewaters, the anaerobic treatment process might be accompanied with some odour nuisance due to H_2S formation. A proper handling of the biogas produced is required to avoid bad smell.

2.2 Anaerobic digestion processes and bioconversions

Anaerobic digestion is a biological process that utilizes a mixed culture of bacteria in the absence of free oxygen to remove organic matter that is present in the wastewater. The overall process yields a useful by-product in the form of biogas, primarily methane (CH_4) and carbon dioxide (CO_2). Anaerobic degradation of organic matter is a complicated microbial process consisting of several interdependent consecutive and parallel reactions (Fig. 2.1), and anaerobic digestion encompasses a complex consortium of micro-organisms. The microbial species involved in anaerobic digestion process could be classified into four main groups: (1) fermentative bacteria, (2) hydrogen-producing acetogenic bacteria, (3) hydrogen and carbon dioxide-consuming methanogens, and (4) acetoclastic methanogens.

The science underlying anaerobic digestion can be complex and the process is best understood if split into the four main steps according to Sanders (2001): hydrolysis, acidogenesis, acetogenesis and methanogenesis

1. Hydrolysis

Hydrolysis is the first step in the anaerobic digestion in which complex polymeric substances, particulate or undissolved, are converted by enzymes which are excreted by fermentative bacteria into less complex, dissolved compounds (such as simple sugars, amino acids, and long chain fatty acids) which can pass through the cell walls and membranes of the fermentative bacteria. This step is known to be complex and likely to be as diverse as the particles and organisms that are involved in the process (Morgenroth *et al.*, 2001). Generally hydrolysis of particulate matter, suspended and colloidal, is considered to be the rate-limiting step (the slowest step in a sequence of reactions) in the whole digestion process (Eastman and Ferguson, 1981; Zeeman *et al.*, 1997; Sanders, 2001). The results of Mahmoud (2002) reveal that sizing of anaerobic reactors for treating complex substrates like sewage should be based on the hydrolysis step, which is limiting the digestion rate.

The hydrolysis rate is affected by several factors like: pH, temperature, availability and structure of the substrate, sludge retention time, product inhibition, particle size distribution and particle size, and available surface area (Sanders, 2001).

There are different mathematical relationships to estimate the hydrolysis rate. First order kinetics (Eq. 2.1) are most commonly used to describe the hydrolysis of particulate substrates during anaerobic digestion (Eastman and Ferguson, 1981; Pavlostathis and Giraldo-Gomez, 1991).

$$dX_{degr.}/dt = -k_h \cdot X_{degr.} \qquad (2.1)$$

where:
$X_{degr.}$: concentration biodegradable substrate (kg/m^3),
t: time (days),
k_h: first order hydrolysis constant (1/day).

Hydrolysis rate constants have been determined in sewage sludge (Mahmoud, 2002) and raw sewage (Halalsheh, 2002). However, the hydrolysis rate constants should be measured each time for that specific waste and not adopted from literature data (Mahmoud, 2002).

2. Acidogenesis

In acidogenesis step, the products of hydrolysis are converted to organic acids by large group of fermentative bacteria. They convert sugars, amino acids, and long chain fatty acids into short-chain fatty acids like acetic, propionic, formic, lactic, and butyric; and alcohols, ammonia, CO_2 and H_2. The products of this stage vary with the type of bacteria and environmental conditions (i.e. temperature and pH).

3. Acetogenesis

The acetogenic bacteria convert the products of the fermentative bacteria (short-chain fatty acids) into acetate, hydrogen gas and carbon dioxide; which are the substrate for methanogens.

4. Methanogenesis

Methanogenesis, which carry out the terminal reaction in the anaerobic food chain, are most important in anaerobic treatment systems. This step comprises the production of methane (CH_4) from acetate or from the reduction of CO_2 by acetotrophic and hydrogenotrophic.

The acetotrophic (acetoclastic) methanogens convert acetate into CH_4 and CO_2 according to the following reaction.

$$CH_3COOH \rightarrow CH_4 + CO_2$$

Meanwhile, the hydrogen-utilizing methanogens (hydrogenotrophic) convert hydrogen and carbon dioxide into methane according to the following reaction.

$CO_2 + 4H_2 \rightarrow CH_4 + 2H_2O$

The acetate reaction is the primary producer of methane (about 70%) because of the limited amount of hydrogen available (Guijer and Zehnder, 1983). Methane and carbon dioxide are the chief gaseous products of the process. These gases constitute approximately 75 to 80% of the gas collected and the remaining volume is composed of hydrogen sulfide, nitrogen and hydrogen.

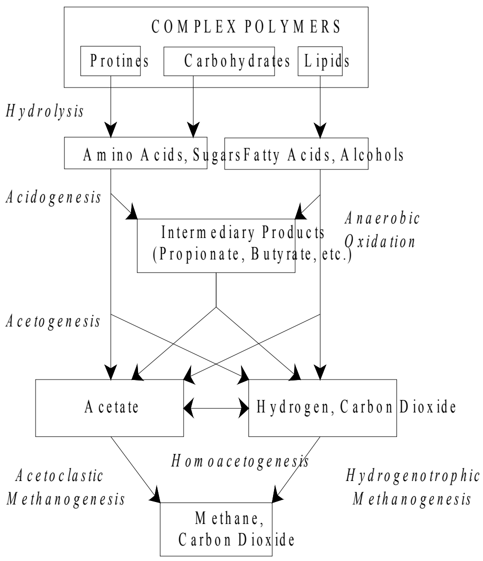

Figure 2.1. Anaerobic digestion reactions and steps of organic polymeric materials (Guijer and Zehnder, 1983)

2.3 High rate anaerobic systems

Advances in the understanding of how anaerobic system functions improved understanding of mixing and mass transfer, and anaerobic reactor design, has led to the evolution of a new generation of high-rate anaerobic processes, i.e. anaerobic filters (AF), anaerobic expanded/ fluidized bed reactors, upflow anaerobic sludge blanket (UASB) reactor, etc. The key feature offered by the high-rate processes is their ability to maintain high biomass concentration under high loading conditions at a relatively short hydraulic retention time (HRT) (Kobayashi *et al.*, 1983; Frankin, 2001; Mulder *et al.*, 2001). This feature makes reactor volumes smaller, and permits anaerobic treatment at lower temperatures than previously thought possible or economical (Kobayashi *et al.*, 1983).

Anaerobic treatment in high-rate reactors is increasingly recognized as a core method technology for environmental protection and resource preservation (Lettinga, 1996; Lettinga, 2001). Among the many various systems of high rate anaerobic reactors, the "Up-Flow Anaerobic Sludge Blanket" (UASB) is by far the most convenient, economical, and easily operated and controlled system that can used for the anaerobic treatment of wastewater (Sayed and Fergala, 1995).

2.3.1 The conventional UASB reactor

The Upflow Anaerobic Sludge Bed (UASB) reactor was developed in the 1970s by Lettinga and his group in the Netherlands (Haandel and Lettinga, 1994). The UASB reactor presently is the most widely and successfully used high-rate system for sewage pre-treatment of several types of wastewater (Lettinga, 1996; McCarty, 2001). The UASB reactor is a high-rate suspended growth type of reactor in which wastewater is introduced into the reactor from the bottom and distributed evenly. The UASB reactor essentially consists of four zones (from bottom to the top): the sludge bed, the fluidized zone, the gas-liquid-solids (GLS) phase separator, and the settling zone (Fig. 2.2).

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 take place (Seghezzo, 2004). This sludge bed is basically formed by accumulation of incoming suspended solids and bacterial growth. In the UASB process, influent passes upward through a sludge bed (granular or flocculent), where different physical and biochemical mechanisms act in order to retain and biodegrade organic substances. Retention of active sludge within the UASB reactor enables good treatment performance at high organic loading rates. Digestion of the particulate matter retained in the sludge blanket and breakdown of soluble organic matter generates gas and relatively small amounts of new sludge. Natural turbulence caused by the influent flow and the biogas production provides good wastewater-biomass contact and mixing in UASB systems. Consequently, a properly designed UASB reactor eliminates the need for mechanical mixing.

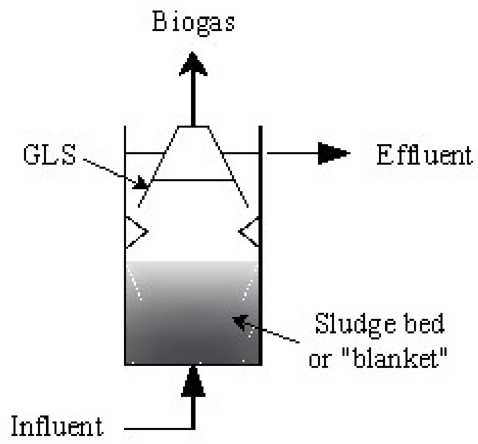

Figure 2.2. Schematic diagram of UASB reactor

The biogas, the liquid fraction and the sludge are separated in the gas/liquid/solids (GLS) phase separator, consisting of the gas collector dome and a separate quiescent settling zone. The settling zone is relatively free of the mixing effect of the gas, allowing the solids particles to fall back into the reactor. The clarified effluent is collected in gutters at the top of the reactor and removed. The biogas has methane content typically around 75 percent and may be collected and used as a fuel or flared.

An important observation made in studies carried out with the UASB reactor is the presence of anaerobic granular sludge under certain conditions (Hulshoff Pol *et al.*, 1983), which has the advantage of possessing higher settling properties than flocculent sludge (Elmitwalli, 2000; Seghezzo, 2004). Because of its dense structure and high settle-ability, anaerobic upflow reactors can be operated at very high upflow liquid velocities, without the loss of biocatalyst from the system under practical reactor conditions (Van lier *et al*., 2001). Van lier *et al*. (2001) reported that though always desired, the formation of anaerobic sludge granules cannot be guaranteed on each type of wastewater. Sludge granulation is possible when the SRT reaches a time period of several months. Particularly UASB-type reactors treating wastewaters with a high concentration of SS, such as domestic wastewater, are generally operated with a flocculent "fluffy" type of biomass. Several factors, such as sludge flotation or the adsorption of finely dispersed colloidal matter on the surface of the sludge also may cause the granulation process to be difficult or the granular sludge to deteriorate (Sayed, 1987). According to Haandel and Lettinga (1994), it had not been observed in any of the existing full-scale UASB reactors treating sewage. In fact, granulation was observed in reactors treating settled sewage (van der Last and Lettinga, 1992; Seghezzo, 2004).

Experiments aiming at optimizing the contact between the wastewater and the sludge in the UASB reactor led to the development of more advanced reactor design, viz. the expanded granular sludge bed (EGSB) reactor (Men *et al.*, 1988), into which higher upflow velocities in the range of 4-10 m/hr are applied (van der Last and Lettinga, 1992). Compared to UASB reactors, higher organic loading rates (as kgCOD/m^3.d) can be accommodated in EGSB systems. Soluble pollutants are efficiently treated in EGSB reactors but suspended solids are not substantially removed from the wastewater stream due to the high upflow velocities applied (Mahmoud 2002; Seghezzo, 2004). Process, which apply a high upflow domestic sewage, such as EGSB and the Fluidized Bed (FB) reactors are unsuitable for domestic sewage treatment, unless they are combined with an adequate pre-settling/ treatment (Lettinga and Hulshoff Pol, 1991).

2.4 Application of the conventional UASB reactor in Palestine

The full-scale application of UASB reactors to domestic wastewater has been a success in tropical areas, where mean sewage temperature can go up to 30ºC (Mahmoud, 2002). However, in Middle East countries, like Palestine, where domestic wastewater is characterized by a high fraction of suspended solids (Mahmoud *et al.*, 2003) and mostly of relatively low temperatures during the wintertime which lasts for three months, the reactor has limited performance and could be confronted with some problems such as poor granular sludge formation, accumulation and slow methanogenic activity and low biogas production (Kalogo and Verstraete, 1999).

Mahmoud (2002) reported that, the design and performance of anaerobic reactor strongly depends on the solids retention time, operational temperature, and the biodegradability and concentration of the entrapped solids, which are interrelated parameters. These factors are discussed below.

2.4.1 Effect of solids retention time (SRT) and temperature

The solids retention time is a fundamental design and operating parameter for all anaerobic processes (Zeeman and Lettinga, 1999). The SRT is the average time that a solid particle stays in the reactor. The success of UASB reactors is highly dependent on the SRT, which is a key factor determining the ultimate amount of hydrolysis, acidification, and methanogenesis in a UASB system at certain temperature conditions (Mahmoud *et al.*, 2004). The SRT should be long enough to provide sufficient methanogenic activity at the prevailing conditions. In general, SRT values greater than 20 days are needed for anaerobic processes at 30ºC for effective treatment performance, with much higher SRT values at lower temperatures (Metcalf and Eddy, 2003). At low temperatures (5-20ºC) during the winter period, hydrolysis, the rate-limiting step of the process of anaerobic digestion of particulate organic matter, may become too slow, leading to accumulation of undegraded SS in the reactor's sludge bed, resulting in a decrease of the methanogenic activity of the sludge, unless long HRTs are applied (Man, 1990).

The lower the temperature the longer the SRT required in one-step UASB reactors to provide enough hydrolysis and methanogenesis to degrade the previously entrapped organic particulate fraction organic particulate fraction (Zeeman and Lettinga, 1999). A specific SRT is then required for each temperature and for each type of sewage. If the required SRT is known, based on literature or former experiences, the needed hydraulic retention time (HRT) can be calculated with model proposed by Zeeman and Lettinga (1999):

$$SRT = X/Xp \quad (1)$$

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

Xp: sludge production (g COD/L.d)

$$Xp = O*SS*R*(1-H)$$

(2)

O: organic loading rate (kg COD/m^3.d); SS = COD_{sus} / COD_{inf}

R: fraction of COD_{sus} removed

$$HRT = C/O \text{ (days)} \quad (3)$$

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

$$HRT = (C *SS/X)*R*(1-H)*SRT \quad (4)$$

SRT: sludge retention time (days)

H: fraction of removed solids that are hydrolysed

Mahmoud *et al.* (2003) pointed out, according to the model calculations, that a minimum HRT of 22 hour is required for the application of the one-stage UASB reactor for domestic wastewater treatment in Palestine to overcome the wintertime; considering a minimum SRT of 75 days at 15ºC (the average temperature in winter) is required. For temperatures below 15ºC, a SRT >100 days is necessary to retain sufficient methanogenic activity in the reactor (Zeeman and Lettinga 1999).

Temperature affects the activity and the growth of microorganism. A decrease in the operational temperature generally leads to a drop in the maximum specific growth rate and specific substrate utilization rate of anaerobic biomass (Lettinga *et al.*, 2001). Temperature not only influences the metabolic activities of the microbial population but also has a profound effect on such factors as gas-transfer rates and the settling characteristics of the biological solids (Seghezzo, 2004). Moreover, Temperature affects the final degradation extent.

2.4.2 Effect of suspended solids on anaerobic treatment

The main technical obstacle for the application of the UASB reactors for domestic wastewater treatment was allegedly the presence of suspended solids (SS) in the wastewater (Kalogo and Verstraete, 1999; Seghezzo, 2004). In Palestine, Mahmoud *et al.* (2003) found that particulate materials, exceeding 0.45 µm, represent the major fraction of domestic sewage, about 65-71% of total COD. Several authors pointed out that the presence of SS in the wastewater can affect the anaerobic treatment adversely, such as: accumulation of undegraded SS may induce a reduction in the methanogenic activity of the sludge, a deterioration of bacterial aggregates (granules) and possibility of slowing down or even counteracting the formation of granular sludge in the case where flocculant seed sludge is used, a reduction in COD conversion efficiency, and the formation of scum layers, leading to overloading of the reactor (Lettinga and Hulshoff Pol 1991; Kalogo and Verstraete, 1999). Accumulation of SS may become significant at temperatures lower than 18°C due to very slow hydrolysis, forcing a reduction of the loading rate (Mahmoud, 2002).

At low temperatures, more organic matter will remain undegraded. Bogte *et al.* (1993) found evidence of accumulation of biodegradable solids during wintertime and degradation during summer time when operating small-scale UASB-septic tank reactors for on-site sewage treatment in the Netherlands. The entrapped solids have been successfully degraded in a separate heated digester (Mahmoud, 2002).

2.5 Examples of domestic sewage treatment in upflow reactors

In tropical countries, UASB reactors treating sewage showed chemical oxygen demand (COD) removal efficiencies around 65%, with some reports of up to 80% in low loaded reactors (Wiegant, 2001). The hydraulic retention time (HRT) applied fluctuates around 6 hrs, aiming at an upflow velocity (V_{up}) of about 0.75 m/hr in standard 4-m tall reactors (Wiegant, 2001). Kalogo and Verstraete (1999) reported that under temperature conditions >20ºC, the COD removal efficiency of the UASB reactors was directly related to the HRT. The higher the HRT, the better was the removal efficiency.

At lower temperatures, reported results differ widely, depending on factors such as sewage temperature and composition, operational parameters, type and dimensions of the reactor, and the amount and quality of the inoculum (see Table 2.4). Removal efficiency decreases at lower temperatures (Haandel and Lettinga, 1994). Analysis of data from several works reviewed by Seghezzo (2004) indicates that average COD removal efficiencies of 41.7, 52.8, and 69.1% have been observed at temperatures below 15ºC, between 15 and 22ºC, and above 22ºC, respectively.

Two-stage anaerobic systems have been proposed as one of the ways to retain and degrade suspended solids (SS) from raw sewage at low temperatures (Haandel and Lettinga, 1994; Wang, 1994; Elmitwalli, 2000). Table 2.4 summaries some of the recent results for anaerobic treatment of domestic wastewater in pilot and full scale UASB reactors under different conditions.

In order to enable anaerobic treatment of domestic wastewater under conditions prevailing in the Middle East (low sewage temperatures in winter and SS-rich wastewaters), specific alteration in process layout, reactor technology or operational techniques are emerged (Kalogo and Verstraete, 1999; Elmitwalli, 2000; Mahmoud, 2002). Some examples of these technologies are described below.

Wang (1994) proposed two-stage anaerobic processes to retain and degrade suspended solids from sewage at lower temperatures. A process consisting of a sequential HUSB reactor followed by an EGSB reactor, combined with an additional sludge stabilization tank. In the first stage, the particulate organic matter is entrapped and partially hydrolyzed into soluble compounds, which are then digested in the second stage. The HUSB reactor differs from the UASB reactor by the absence of a three- phase separator, which is an important aspect of the design of the latter. The removal efficiency of suspended solids in the first reactor will be higher than that of organic matter and excess sludge needs to be discharged regularly. HRTs applied were 3 and 2 hrs for the HUSB and EGSB reactors respectively, and two days for the sludge stabilization tank. The total process provided 71% COD and 83% SS removal efficiencies at temperatures above 15°C, and 51% COD and 77% SS removal at 12°C.

Sayed and Fergala (1995) also studied the feasibility of a two-stage anaerobic system for domestic sewage treatment. The first stage consisted of two flocculent UASB reactors operated intermittently while the second stage was a UASB reactor seeded with granular sludge. The first stage was intended to remove and partially hydrolyze SS and the second was devoted to the removal of soluble organic material. It was claimed that intermittent operation of the first stage provides further stabilization of the removed solids. The experiments were carried out at an ambient temperature of 18- 20°C and average HRTs of 8-16 hrs for the first stage and 2 hrs for the second stage. COD and BOD removal efficiencies up to 80 and 90%, respectively, were achieved. Most of the removal took place in the first stage.

Elmitwalli (2000) investigated the treatment of pre-settled sewage at 13°C in anaerobic hybrid (AH) reactor with small sludge granules. The AH reactor used was basically an upflow reactor in which a sludge bed was at the bottom and a synthetic filter medium replaced the gas-solid-liquid separator, typical of UASB reactors, at the upper part. The medium consisted of vertically oriented reticulated polyurethane foam (RPF) sheets with knobs at one side. Elmitwalli (2000) showed that clean vertical sheets of RPF were efficient in removing suspended COD (>75%) in domestic sewage. The removal of colloidal and dissolved COD was significantly higher when the reactors were fed with settled sewage. Drawback of this system is the production of poorly stabilized sludge; therefore further stabilization process is still needed. Some improvements in such system appear to be necessary to avoid the formation of channels and gas pockets in the sludge bed (Kalogo and Verstraete, 1999).

Mahmoud (2002) studied the application of UASB reactors for domestic wastewater treatment at a sewage temperature of 15ºC, the average sewage temperature in Palestine during wintertime (Table 2.4). The performance of a single-stage UASB reactor was improved by digesting the excess sludge in an anaerobic digester at 35ºC, and recirculating the sludge back into the reactor. The performance of the UASB-Digester system was as good as that achieved in tropical countries with single-stage UASB reactors, and the wasted sludge was much more stabilized.

Halalsheh (2002) studied the performance of UASB reactors treating strong raw sewage in Jordan for a long time (2.5 years) at a temperature of 18ºC in winter and 25ºC in summer. A comparison was made between one and two stage systems. The average results obtained during winter time with the first stage of the two-stage system, and the one-stage reactor, were the same with no significant effect of temperature (see Table 2.4). Moreover, higher degree of sludge stabilization was observed in the one-stage reactor, compared to the first stage of the two-stage system and the second stage had poor performance. The author reported that most of the COD_{tot} in a two-step UASB system for sewage treatment in Jordan was retained in the first step, indicating that a second anaerobic step may not be indispensable under these conditions.

Elmitwalli *et al.* (2003) also investigated the treatment of concentrated sewage (about 3600 mg COD/l) at low temperature of 13ºC in a two-step anaerobic hybrid (AH)-septic tanks with reticulated polyurethane foam (RPF) sheets. The presence of RPF sheets in the AH reactor prevented sludge bed flotation. The used HRT was 2.5 days for each reactor.

Mean removal efficiencies in the two-step AH-septic tank at 5 days HRT and 13°C were 94, 98, 74 and 78% for COD_{tot}, COD_{sus}, COD_{col} and COD_{dis} respectively. The first AH-septic tank was full of sludge after 4 months of operation due to the high removal of particulate COD and the limited hydrolysis at low temperature conditions. Based on the experimental results and the mathematical model carried out by Elmitwalli *et al.* (2003), only a one-step AH septic tank is required and; an HRT of 5.5-7.5 days is needed for treatment of concentrated sewage at a low temperature of 13°C, when one-step AH-septic tank is used.

Based on the above discussion, the two-staged reactor concept seems particularly attractive. However, there is a real need for regular discharge of the excess sludge from the first reactor. Moreover, a digester should be combined to the system. The necessity of introducing a second reactor can increase the investment and operational costs of the treatment plant. It can also make it more complicated technology (Kalogo and Verstraete, 1999), which clashes with the local conditions in the rural areas of the developing countries.

Among the previous technologies and process layouts taking into account the prevailing conditions in the rural communities of Palestine, the UASB-septic tank system is the most cost-effective and attractive option that can best be employed in the treatment of wastewater on-site both at individual household and at community level (Zeeman and Lettinga, 1999; Zeeman *et al.*, 2000). Hence, the system is simple in operation and maintenance and needs less attention compared to other anaerobic systems. Furthermore, the system is also designed for sludge accumulation and stabilization; and therefore no need for additional stabilization process and the accumulated sludge needs to be wasted once a year or more. The choice between a UASB or UASB-septic tank system will mainly be based on the scale (Zeeman and Lettinga, 1999).

2.6 The UASB-septic tank system

The UASB-septic tank system is a promising alternative for the conventional septic tank (Bogte *et al.*, 1993; Lettinga *et al.*, 1993). It differs from the conventional septic tank system by the upflow mode in which the system is operated, resulting in both improved physical removal of suspended solids and improved biological conversion. The most important difference with the traditional UASB system is that the UASB-septic tank system is also designed for the accumulation and stabilization of sludge. So an UASB-septic tank system is a continuous system with respect to the liquid, but a fed-batch or accumulation system, with respect to the solids.

First applications of this reactor concept for the treatment of domestic wastewater for on-site single households in isolated locations, like farms and recreational facilities not connected to the centralized sewerage system, was studied under Dutch (low) and Indonesian (high) ambient temperatures. In The Netherlands, Bogte *et al.* (1993) tested three 1.2 m^3 UASB-septic tank reactors in different rural locations with varying results (Table 2.5). A similar configuration was tested in a 0.86 m^3 reactor in Bandung (Indonesia) by Lettinga *et al.* (1993). Treatment efficiencies in Indonesia were more interesting with very high removal efficiencies (see Table 2.5); while good sludge stabilization and high sludge hold-up were achieved. Below 12°C (Dutch winter conditions) the conversion of produced VFA to methane gas was too low, although the research period was too short to draw definite conclusions.

For low temperature conditions the application of a two step UASB-septic tank system could be profitable (Zeeman and Lettinga, 1999; Zeeman *et al.*, 2000). The first reactor will in winter mainly retain solids, while just a limited amount of hydrolysis, acidification and methanogenesis will occur. In the second reactor mainly methanogenesis will occur at the low temperature conditions. In summer hydrolysis and acidification of both fresh and accumulated solids will take place in the first reactor together with methanogenesis, while the second reactor acts as a polishing step for removing and converting remaining

VFA and suspended COD, washed from the first reactor- as a result of the increased gas production (Zeeman and Lettinga, 1999).

The design of the UASB-septic tank is almost as simple as that of conventional septic tanks but the treatment efficiency is much higher (Lettinga *et al.*, 1991; Zeeman *et al.*, 2000). These reactors should startup in summer with an inoculum of at least 15% of the volume, according to the results of Zeeman (1991) that worked on manure digestion at low temperatures in accumulation systems. The UASB-septic tank is designed with the same long HRT typical of conventional septic tank and long sludge retention time (Mgana, 2003). The long HRT generally applied for the UASB-septic tank implies a low hydraulic load. But the sludge hold-up time of the system is so long that sludge discharge is only required once every 1 to 4 years (Kalogo and Verstraete, 1999; Zeeman *et al.*, 2000), and can be used for soil conditioning and fertilisation. In a conventional UASB, due to the short HRT, the hydraulic loading rate is higher. Thus, the high hydraulic load considerably shortens the sludge hold-up period of the reactor. This requires the conventional UASB reactor to discharge frequently (once or twice a week) the excess sludge produced (Kalogo and Verstraete, 1999). The discharged sludge needs to be further stabilized in a separate reactor.

On the basis of already available technical information concerning the UASB-septic tank system performance (Lettinga *et al*., 1991; Bogte *et al*., 1993; Lettinga *et al*., 1993) (Table 2.5), the system does appear to be potentially useful on-site treatment system for both black and total domestic wastewater in rural areas where it is uneconomic to build sewers and conventional treatment plants.

2.7 Design considerations for UASB reactors

Although substantial experience on the design and operation of UASB reactors for treatment of domestic wastewater (Draaijer *et al.*, 1992; Haandel and Lettinga 1994) has been gathered lately, most of the performance data and results have not yet been published (Wiegant, 2001) and limited so far to regions with constant and relatively warm temperature conditions. However, regarding to the Middle East countries, with high strength domestic wastewater and seasonal temperature fluctuation, it is very hard to comment on the available operational results. They differ quite widely and therefore, the design criteria of the UASB reactor for domestic wastewater treatment in the Middle East are still to be formulated.

Wiegant (2001) reported that the design criteria of UASB reactors, for domestic wastewater, seem still not to have converged to a point that adequate predictions of the effluent quality as a function of the design criteria can be made.

A comprehensive review of design considerations for UASB reactors has been provided by Lettinga and Hulshoff Pol (1991). Important design considerations are: (1) volumetric organic load, (2) upflow velocity, (3) gas collection system.

1. Applicable organic loading rate

The OLR can be varied by changing the influent concentration and by changing the flow rate. Changing the flow rate implies changing the HRT and the upflow velocity (Mahmoud, 2002). The OLR can be determined according to the following equation:

$$\text{OLR} = \frac{Q * COD}{V} = \frac{COD}{HRT}$$

where:
OLR: organic loading rate (kg COD/ 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)

Organic loading rates for UASB reactors range on a COD basis from 0.5 to 40 kg/m^3.d (Droste, 1997). However, according to literature, the conventional UASB reactor for the treatment of domestic sewage was reported to obtain satisfactory COD removal efficiencies at organic loadings between 0.4-3 kg COD/m^3.d in the temperature range of 15°C to 25°C (Kalogo and Verstraete, 1999; Halalsheh, 2002). At low temperatures low OLR is preferred.

2. Applicable upflow velocity

The upflow velocity is a critical design parameter in upflow reactors (Metcalf and Eddy, 2003). Upflow velocities in typical UASB reactors range up to 1-2 m/hr (Droste, 1997) although Lettinga and Hulshoff Pol (1991) and Haandel and Lettinga (1994) recommended that the average daily upflow velocity should not exceed 1 m/hr with a typical value of 0.7 m/hr for domestic wastewater. Haandel and Lettinga (1994) also reported a nearly linear decrease in efficiency with increasing upflow velocity.

However, an optimum design upflow velocity is not fully determined. Full scale reactors, for domestic wastewater with reactor height range of 4-5 m, are generally designed at upflow velocities between 0.15-0.75 m/hr (Wiegant, 2001). Vieira *et al.* (1994) showed that high removal efficiencies for COD and TSS of 80 and 87%, respectively, at an upflow velocity below 0.15 m/h in a full-scale 67.5 m^3 UASB reactor treating domestic wastewater at temperature between 16 and 23°C. The upflow velocity can be determined according to the following equation:

$$V_{up} = \frac{H_{reactor}}{HRT}$$

where:

V_{up}: upflow velocity (m/h)

H: height of reactor (m)

HRT: hydraulic retention time (h)

3. Gas collection and solid separation

The gas-liquid-solids separator (GLS) is an important aspect of the design of the UASB reactor (Kalogo and Verstraete, 1999). The GLS is designed to collect the biogas, prevent washout of solids, encourage separation of gas and solid particles, allow for solids to slide back into the sludge blanket zone, and help improve effluent solids removal. Guidelines for the GLS design are summarized in Table 2.3.

Table 2.3. Recommended design considerations for the gas-liquid-solids separator for UASB reactors (Metcalf and Eddy, 2003)†

- The slope of the settler bottom, i.e., the inclined wall of the gas collector, should be between 45 and 60°.
- The surface area of the apertures between the gas collectors should not be smaller than 15 to 20 percent of the total reactor surface area.
- The height of the gas collector should be between 1.5 and 2 m at reactor heights of 5-7 m.
- A liquid-gas interface should be maintained in the gas collector to facilitate the release and collection of gas bubbles and to control scum layer formation.
- The overlap of the baffles installed beneath the apertures should be 10 to 20 cm to avoid upward-flowing gas bubbles entering the settler compartment.
- Generally scum layer baffles should be installed in front of the effluent weirs.
- The diameter of the gas exhaust pipes should be sufficient to guarantee the easy removal of the biogas from the gas collection cap, particularly in the case where foaming occurs.
- In the upper part of the gas cap, antifoam spray nozzles should be installed in the case where the treatment of the wastewater is accompanied by heavy foaming.

† Adapted from Malina and Pohland (1992)

Table 2.4. Summary of results for anaerobic treatment of domestic wastewater in pilot and full scale UASB reactors under different conditions

Place	V (m^3)	T (°C)	Influent type	Influent (mg/L) COD$_{tot}$	Influent (mg/L) SS (COD$_{sus}$)	HRT (h)	Removal efficiency (%) COD$_{tot}$	Removal efficiency (%) SS (COD$_{sus}$)	Inoculum type	Reference
Netherlands	0.12	18-20	R	581	-	12	72	-	GS	Lettinga *et al*. (1983b)
Netherlands	0.12	12-20	R	190-1180	-	7-8	30-75	(60)	GS	Man *et al*. (1988)
India	1200	20-30	R	563	418	6	74	75	None	Draaijer *et al.* (1992)
Colombia	35	-	R	-	-	5-19	66-72	69-70	-	Schellinkout & Collazos (1992)
Colombia	3350	24	R	380	240	5.2	45-60	60	None	Schellinkout & Osorio (1994)
Netherlands	0.004	13	S	339	(82)	8	59	(79)	GS	Elmitwalli (2000)
Netherlands	0.004	13	R	456	(229)	8	65	(90)	GS	Elmitwalli (2000)
Japan	0.021	13-25	R	312	(187)	4.7	69	(80)	GS	Uemura and Harada (2000)
Netherlands	0.14	15	R	721	(398)	6	44	73	FS	Mahmoud (2002)
Jordan	1.2	24	R	1412	451 (830)	23	58	62 (65)	FS	Halalsheh (2002)
Jordan	60	18-25	R	1531	396 (1122)	23-27	51-62	59 (53)	None	Halalsheh (2002)
Jordan*	60	18-25	R	1531	396 (1122)	8-10	50-62	53 (60)	None	Halalsheh (2002)
Tanzania	1.5	25-34	R	529	(264)	1.7-40	64	(57)	STS	Mgana (2003)
Argentina	0.5	16.5	S	147	(69)	6.1	50-55	(66)	PDS	Seghezzo (2004)

V = Volume; T = Temperature; S = Settled wastewater; R = Raw wastewater; GS = Granular sludge; FS = Flocculent sludge; STS = Septic tank sludge; PDS = Partially digested sludge; *: First stage of a two staged UASB system

Table 2.5. Summary of applications of on-site pilot scale UASB-septic tank reactors to sewage treatment under different conditions

Place	V (m^3)	T (^oC)	Influent type	Influent concentration (mg/L) COD$_{tot}$	BOD	TSS	HRT (h)	Removal efficiency (%) COD$_{tot}$	BOD	TSS	Gas production (l/d)	Inoculum type	Period (months)	Reference
Netherlands	1.2	13.8	GW+BW	976	454	641*	44.3	33	50	47*	66.5	DSS	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	DSS	24	Bogte *et al.* (1993)
Netherlands	1.2	11.7	BW	1716	640	1201 *	102.5	60	50	77.1*	16.7	GS	13	Bogte *et al.* (1993)
Indonesia	0.86	>20	BW	5988	2381	2678	360	90-93	92-95	93-97	118	STS	40	Lettinga *et al*. (1991)
Indonesia	0.86	>20	GW+BW	1359	542	568	34	67-77	78-82	74-81	168	CS	30	Lettinga *et al*. (1991)
Palestine	0.35	16-35	PBW	1013	458	715	11.6	76	59	58	None	APS	1.4	Al-Juaidy (2001)
Palestine	0.35	16-35	PDW	566	200	560	14	79	70	46	None	APS	1.4	Ali (2001)
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; PB = Pre-settled black wastewater; PDW = Pre-settled domestic wastewater; DSS = Digested sewage sludge; GS = Granular sludge; STS = Septic tank sludge; CS = Cesspool sludge; APS = Anaerobic pond sludge; *: expressed as COD (suspended + colloidal)

Chapter 3

Materials and Methods

3.1 Experimental set-up

To evaluate suitability of the UASB-septic tank process for real domestic sewage treatment, two pilot scale UASB-septic tank reactors R1 and R2, were installed at the city's main sewage treatment plant of Al-Bireh. Each pilot scale UASB-septic tank reactor employed in this research essentially had vertical cylindrical shape and was made of 3 mm thick galvanized steel plate with internal working volume of 0.8 m^3 (working height = 2.50 m; diameter = 0.638 m). Nine sampling ports (diameter = 3/4 in.) separated 0.25 m from each other were installed along the reactor for sludge sampling. Polyvinyl chloride (PVC) tubes and hoses (internal diameter = 1.25 in.) were used for influent and effluent distribution. The gas/liquid/solids (GLS) separator was of inverted galvanized steel cone and installed at the top of the reactor. The treated effluent flowing out of the reactor was collected in a settling pocket where washed out sludge settled at the bottom and the supernatant was partly discarded back to the grit removal chamber.

The influent was distributed in the reactor through a one inlet pipe with 4 outlets located 5 cm from the bottom. Biogas generated from the reactors was continuously measured in wet-type gas meters. Methane content in the biogas was determined by displacing a 16% NaOH solution from a tightly, closed, glass cylinder. CO_2 was retained in the solution. The content of other gases in the biogas, like hydrogen sulfide, was neglected. The reactors and the biogas traps were fabricated locally. The details of the reactors and gas collecting assembly are presented in Appendix (1) (Photos from 1 to 10). A schematic diagram of the experimental set-up is shown in Fig. 3.1.

3.2 Sewage

The two UASB-septic tank reactors were fed with domestic sewage from the main sewage trunk at the Al-Bireh WWTP. Preliminary treatment of the raw sewage was provided by screens (retention of coarse materials) and grit removal chamber. The wastewater from the grit chamber was pumped every five minutes, using automatic controlled submersible pump, to a holding tank (200 L plastic container) from which the reactors were fed and the influent was sampled. The wastewater in the holding tank had a resident time of about 5 minutes, controlled by water level device and returned drain pipeline to grit chamber. The holding tank was however emptied and cleaned frequently to prevent the accumulation of solids. From there sewage was continuously pumped to reactors with peristaltic pumps to maintain constant discharge of influent for each reactor using MASTERFLEX® L/S 7520-57 series (flow rate range: 4.8-480 ml/minute) equipped with MasterFlex Tygon L/S® 36 tubing. Flow rates were checked almost everyday and adjusted with 1 to 10 turn speed control (1-100 rpm, 230v drive). Therefore, the holding tank was used, in attempt to reduce the pumping distance of the peristaltic pumps, moreover, to equalize the influent sewage to the reactors. A description of the operation is presented in Appendix (1) (Photos from 1 to 9)

3.3 Pilot plants operation and start-up

The UASB-septic tank reactors were started up in April 2004. The two pilot plants (Reactor 1 and Reactor 2) were operated in parallel at ambient temperature conditions with temperature variation between 15ºC and 34ºC.

The reactors were inoculated with anaerobic fresh sludge (flocculent type). This inoculum was obtained from cesspit serving a small residential house in Al Bireh City. Hence, the seed sludge was well acclimatized with the wastewater constituents. The seed sludge was characterized for its stability, VSS and TSS.

Initially, R1 was seeded with 160 L of sludge which constitute about 10% of reactor volume. However, R2 was seeded with 80 L (10% of volume) hence; R2 designed to operate with half the OLR of R1. By this, sludge accumulation in the two reactors also can evenly be observed. The two UASB-septic tank reactors were operated by feeding the sewage influent for a period of six months. The two reactors were designed to operate at HRTs of 2 and 4 days for R1 and R2, respectively. A detailed description of the operation during the whole experiment is presented in Table 3.1.

Table 3.1. Operational conditions of the pilot scale reactors during the whole experiment

	Reactor 1 (R1)	Reactor 2 (R2)
HRT [days]	2	4
Influent flow [l/d]	400	200
Upflow velocity [m/h]	0.052	0.026

3.4 Sampling

Daily monitoring was started since the onset of the experiment. Grab samples of raw sewage, R1 and R2 effluents were taken two to three times a week (1 L for each). Raw sewage samples were taken after preliminary treatment units. Samples were kept at 4ºC until they were analyzed. Samples were analyzed for COD_{tot}, COD_{sus}, COD_{col}, COD_{dis}, VFA, BOD_5, pH, TSS, VSS, SVI, ammonia, N-kjeldhal, phosphate and sulphate. Furthermore, sludge samples were analyzed for TS, VS and stability. Sludge samples from the reactors were obtained from sampling port no.1 at 0.15 m from the bottom of the reactor. Biodegradability test was performed on influent and effluent samples of the UASB-septic tank reactors. All measurements were determined in duplicate except, VFA and SVI were done in single. The biogas production and ambient temperature were monitored on daily basis.

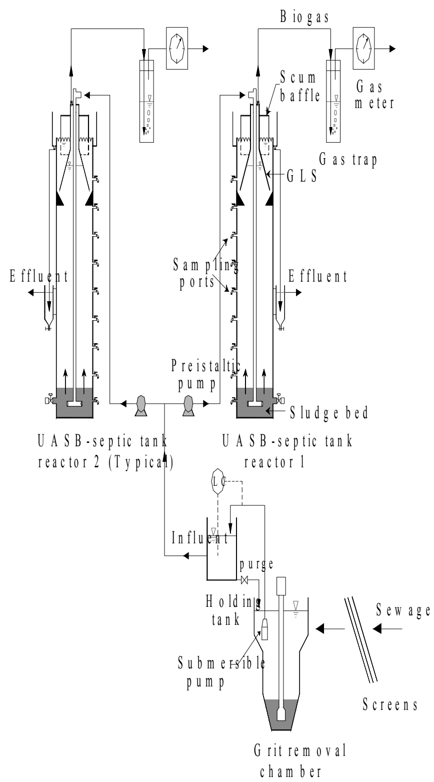

Figure 3.1. Schematic diagram of the experimental set-up (not to scale). Description of the equipments is provided in the text. GLS = Gas-liquid-solids separator; LC = Level controller

3.5 Analytical Methods

3.5.1 Chemical analysis

1. Chemical Oxygen Demand (COD)

COD analysis was carried out using reflux method (acid destruction at 150 ºC for 120 minutes). The absorbance was then measured by spectrophotometer at 600 nm wave length according to Standard Methods (APHA, 1995). Total COD (COD_{tot}), paper-filtered COD (COD_{filt}) (Schleicher & Schuell 595½ 4.4-µm paper filters), and membrane-filtered (dissolved) COD (COD_{dis}) (Schleicher & Schuell ME 25 0.45-µm membrane) were determined in the samples. Suspended and colloidal COD (COD_{sus} and COD_{col}) were calculated as (COD_{tot} - COD_{filt}) and (COD_{filt} - COD_{dis}), respectively. The sludge samples analysed for total COD were firstly diluted 50 times with demi water.

2. Volatile Fatty Acid (VFA) and alkalinity

The volatile fatty acid analysis was carried out using titrimetric method according to (Kapp, 1984; kapp, 1992) (quoted by Buchauer, 1998). This method is mostly very simple procedures, which can be conducted with minimum effort, and does not require high investment in technical equipment which is commonly not available in laboratory and WWTP like Gas Chromatograph (GC) (Buchauer, 1998). Analysis description was reported by Buchauer (1998) as follows:

- Before analysis the sample is filtered through a 0.45-µm membrane filter.
- Filtered sample (20 ml) is put into a titration vessel, the size of which is determined by the basic requirement to guarantee that the tip of the pH electrode is always immersed below the liquid surface.
- Initial pH is recorded.
- The sample is titrated slowly with 0.1 N sulphuric acid until pH 5.0 is reached. The added volume of the titrant is recorded.

- More sulfuric acid with 0.02 N is slowly added until pH 4.3 is reached. The total volume of the added titrant is again recorded.
- The latter step is repeated until pH 4.0 is reached, and the volume of added titrant recorded once more.
- A constant mixing of sample and added titrant is required right from the start to minimize exchanging of CO_2 with the atmospheric during titration.

Finally, VFA (as acetic acid) can be calculated from the following empirical equations (Eq. 3.1 & 3.2) for variable acid normality N and variable sample volume as follows (Buchauer 1998):

$$VFA = (131340*N) * \left(\frac{VA_{(5\ -\ 4,\ meas)}}{VS}\right) - (3.08*Alk_{meas}) - 25 \qquad (3.1)$$

$$Alk_{meas} = (VA_{(4.3,\ meas)}*N*1000)/\ VS \qquad (3.2)$$

where:

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

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

$VA_{(4.3,\ meas)}$: measured volume of acid (ml) required to titrate a sample from initial pH to pH 4.3;

VS: volume of a titrated sample (ml);

Alk_{meas}: measured alkalinity (mmol/L);

N: normality (mmol/L).

3. Biological Oxygen Demand (BOD)

BOD_5 was determined in raw samples (before filtration), by placing diluted wastewater in BOD_5 bottles then, initial dissolved oxygen was measured. After five days of incubation at 20ºC temperature, final dissolved oxygen was measured. Measurement was according to Standard Methods (APHA, 1995).

4. Kjeldhal Nitrogen (NKj-N)

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

5. Ammonia (NH_4-N)

The amount of NH_4-N was determined from paper-filtered samples by Nesslerization using spectrophotometer according to Standard Methods (APHA, 1995). Sample absorbance was measured at 425 nm wavelength.

6. Total Phosphorous (Total P)

To determine the amount of total phosphorous, raw wastewater sample was digested by auto-calving at 120ºC for 30 minutes to achieve one bar pressure, according to Standard Methods (APHA, 1995). Sample Absorbance was measured using spectrophotometer at 880 nm wavelength.

7. Ortho- Phosphate (PO_4^{3-})

The amount of ortho-phosphate was determined from membrane-filtered samples according to Standard Methods (APHA, 1995). Sample Absorbance was measured using spectrophotometer at 880 nm wavelength.

8. Sulfate (SO_4^{2-})

The amount of sulfate was determined from paper-filtered samples according to Standard Methods (APHA, 1995). Sample Absorbance was measured using spectrophotometer at 420 nm wavelength.

3.5.2 Physical analysis

1. Total Solids and Suspended (TS, TSS)

Total solids and suspended were determined according to Standard Methods (APHA, 1995) by oven drying at 105ºC.

2. Volatile Solids and Suspended (VS, VSS)

The volatile solids and suspended were determined according to Standard Methods (APHA, 1995) by oven burning at 550ºC.

3. Sludge Volume Index (SVI)

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

4. pH

pH was determined for the total sample by pH meter (HACH).

5. Temperature

The ambient and wastewater temperatures were measured *in situ* by alcohol thermometer.

6. Color

Color was determined by visual appearance.

7. Atmospheric pressure

The atmospheric pressure was measured *in situ* by barometer pocket device.

3.5.3 Microbiological research

Fecal coliform, helminth eggs and other microbial detection and quantification were carried out in a separate M.Sc research (Samhan, 2005) in which, the focus was on the microbial diversity. Detailed information of the used method, theoretical background and used probes can be found in Samhan (2005).

3.6 Batch experiments

3.6.1 Biodegradability

The anaerobic biodegradability can be defined as the percentage of the chemical oxygen demand (COD) present in an organic sample that is transformed into methane under anaerobic conditions. The anaerobic biodegradability is the anaerobic analogous of the biological oxygen demand (BOD) which in turn, represents the aerobic biodegradability of a sample. Within a certain range of temperature, the final anaerobic biodegradability is pretty constant, yet the degradation rate can vary considerably (Mahmoud, 2002). Results reported in literature should be compared with care because a standard biodegradability test is lacking.

The biodegradability of raw wastewater samples and effluents from R_1 and R_2 were measured once in triplicates during the whole period of experiment. The tests are carried out in batch reactors, sealed serum bottles, of 500 ml with a headspace volume of 70 ml incubated at 30ºC for a period of 120 days. Anaerobic sludge was not added to the bottles as inoculum. Each bottle of the biodegradability test was filled with about 450 ml wastewater and a mineral solution of macro nutrients, trace elements, and bicarbonate buffer. The composition and concentrations of the mineral solution and the experimental procedure are as described by (Elmitwalli, 2000). COD total was measured at the beginning and at the end of the batch period. The experimental procedures for determination of anaerobic biodegradability and the compositions of the macro nutrients and trace elements used in the experiment are presented in details in Appendix 2.

3.6.2 Stability

Stability is defined as the maximum percentage of COD converted to CH_4 of the digested sludge. Stability tests allow the determination of hydrolytic parameters and give a clear idea of the course of the digestion process (Seghezzo, 2004). A standard procedure for stability tests is still lacking and comparison of results reported in literature can be equivocal (Mgana, 2003). Misleading conclusions could be drawn if anaerobic biodegradability is expressed in different units. A sludge stability standard, preferably expressed in gCOD-CH_4/gVSS, or gCOD-CH_4/gCOD, should be established (Seghezzo, 2004).

Sludge stability was measured three times in duplicate during the period of experiment. The experimental set-up and procedure for determination of anaerobic biodegradability and sludge stability are the same according to Mahmoud (2002). However, each bottle of the stability test was filled with about 1.5 g COD-sludge/L, tap water and a mineral solution of macro nutrients, trace elements, and bicarbonate buffer. The stability batches also incubated at 30ºC. The collected methane gas in the headspace was regularly measured using a Mariotte displacement set-up filled with a 5% NaOH solution. 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 sludge stability are also presented in details in Appendix 2.

3.6.3 Methane gas measurement

The collected methane gas in the headspace of the sample serum bottles was regularly measured using the liquid displacement method (Mariotte displacement set-up) as described by Lettinga *et al.* (1991). A serum bottle filled with 5% NaOH solution was hanged upside down. A connection between the headspace of the tested sample serum bottle and the NaOH-bottle is made via a tube with syringes attached to both sides. The biogas (methane and carbon dioxide) collected in the headspace of the sample serum bottles escapes through the tube to the NaOH bottle. The carbon dioxide in the biogas dissolves in the NaOH and kept in the solution while, the remaining CH_4 gas increases the internal pressure of the NaOH serum bottle. The volume of the methane that accumulates at the top of the bottle is equal to the displaced volume of NaOH moved out via another tube. The displaced NaOH solution is collected and measured in a graduated cylinder. The measurements were done inside the incubator at 30ºC. The set-up and arrangement of measurement is shown in Fig. 3.2 and (Photo 12, Appendix 1).

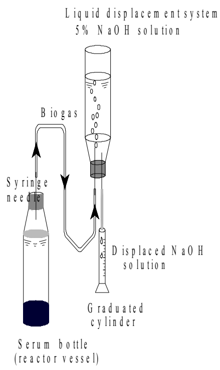

Figure 3.2. Schematic diagram of the liquid displacement setup for methane gas measurement

3.7 Calculations

3.7.1 Removal

The removal of different component can be calculated with equation 3.3.

$$\text{Removal (\%)} = \frac{\text{influent} - \text{effluent}}{\text{influent}} * 100$$

(3.3)

where:
Removal: removal efficiency (%);

influent: concentration of component in influent (mg/l);

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

3.7.2 Biodegradability and Stability

The anaerobic percentages of wastewater biodegradability and sludge stability can be calculated according to equation 3.4 or 3.5. The total CH_4 production in each serum bottle was the summation of the collected CH_4 in the headspace and the dissolved CH_4 in the tested sample. The dissolved CH_4 was calculated by Henry's law (Appendix 3).

$$\text{Biodegradability (\%)} = 100\ (COD_{CH4} / COD_{tot,\ t=0\ days}) \qquad (3.4)$$

or

$$\text{Biodegradability (\%)} = 100\ (COD_{tot,\ t=0\ days} - COD_{tot,\ t=t\ days}) / COD_{tot,\ t=0\ days} \qquad (3.5)$$

where:
COD_{CH4}: amount of produced CH_4 $_{(liquid\ form\ +\ gas\ form)}$ (mg CH_4 as COD/l);

COD_{tot}: amount of total COD in the tested sample (mg COD/l).

The sample calculations of the amount of produced CH_4 as COD in the liquid form is shown in Appendix 3.

The amount of produced CH_4 from the batch bottles, equal to volume of the displaced NaOH solution, and the COD equivalence of CH4 gas (COD_{CH4}) were calculated using the Ideal gas law (Metcalf and Eddy, 2003), equations 3.6 and 3.7 respectively.

$$V = \frac{nRT}{P}$$

(3.6)

where:
V: volume occupied by the gas (L);

n: moles of CH_4 (mole), (1 mol CH_4 = 64 g COD);
R: ideal gas law constant, 0.082057 atm.L/ mol.K;
P: absolute pressure (atm), 0.945 atm at BZU (measured at the Faculty of Chemistry);
T: temperature (k), (273.15 + °C).

COD_{CH4} = n * 64 * 1000 (mg CH_4 as COD/l) (3.7)

3.7.3 Hydrolysis, Acidification and Methanogenesis

Percentage of hydrolysis, acidification and methanogenesis were calculated according to equations 3.8, 3.9 and 3.10 respectively.

$$H(\%) = 100\left(\frac{COD_{CH4} + COD_{dis,\ eff} - COD_{dis,\ inf}}{COD_{tot,\ inf} - COD_{dis,\ inf}}\right)$$

(3.8)

$$A(\%) = 100\left(\frac{COD_{CH4} + COD_{VFA,\ eff} - COD_{VFA,\ inf}}{COD_{tot,\ inf} - COD_{VFA,\ inf}}\right)$$

(3.9)

$$M(\%) = 100\left(\frac{COD_{CH4}}{COD_{tot,\ inf}}\right)$$

(3.10)

where:
H: hydrolysis (%); A: acidification (%); M: methanogenesis (%)
COD_{CH4}: amount of produced CH_4 $_{\text{(liquid form + gas form)}}$ (mg CH_4 as COD/l); $COD_{dis,\ eff}$: amount of dissolved COD in effluent (mg COD/l); $COD_{dis,\ inf}$: amount of dissolved 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 COD/l). The sample calculations of the amount of produced CH_4 as COD in the liquid form is shown in Appendix 3.

3.7.4 COD - mass balance

$$COD_{influent} = COD_{accumulated} + COD_{CH4} + COD_{effluent} \quad (3.11)$$

where:

$COD_{influent}$: amount of total COD in influent (mg/l)

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

COD_{CH4}: amount of produced CH_4 $_{(liquid\ form\ +\ gas\ form)}$ (mg CH_4 as COD/l)

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

3.7.5 COD conversion factors

- 1 g protein, assumed as $(C_4H_{6.1}O_{1.2}N)_X$ is equivalent to 1 g amino acids, 0.16 g NKj-N, 0.16 g NH_4-N and 1.5 g COD (Mahmoud *et al.*, 2004).

3.8 Statistical analysis of data

Process monitoring data were analyzed by conventional descriptive statistics, i.e. variation ranges, arithmetic averages and standard deviations. Correlations between different variables were performed and behaviour of different parameters with time was plotted. The Excel 2002 (Microsoft Corporation) package was used to carry out most of the statistical analyses of data and graphs. Moreover, since the two UASB-septic tank reactors were operated in parallel with the same domestic sewage, a good comparison between the two reactors can be made. Thus, statistical comparisons "t-test" for the performance data of the two reactors were built at a level of significance (ρ) of 0.05 (5%) using the SPSS program for windows. Release 11.0.0, SPSS® Inc., (2001).

The series of orders as follow: (1) "Analyze", "Correlate" and "Bivariate", then from there the Pearson correlation coefficient and the two-tailed test of significance were assigned. (2) "Compare Means" followed by "Paired samples T-Test", from which the confidence interval 95% was also typed. Finally, the output data was read from the Output-SPSS Viewer, Paired Samples Test Table, ended with the Significance (2-tailed) value (ρ). If the resulted value of (ρ) < 0.05, we confidently state there was a difference between the means of the two tested groups.

Chapter 4

Results and Discussion

4.1 Influent sewage characteristics

The main characteristics of the raw sewage used in this research are presented in Table 4.1 during the period between 4th of May and 23rd of October. The results presented in Table 4.1 revealed that the sewage from Al-Bireh City is of domestic type and can be classified as "high strength" according to the sewage strength classification proposed by Metcalf and Eddy (1991). This also can be seen from the values of COD_{tot}, BOD_5, NKj, phosphorous, sulphate, ammonia and solids, which are being higher than that of an average domestic wastewater in other countries. The high strength character of the sewage can be attributed to low water consumption, people's habits, industrial discharges, and to the local food commerce (restaurants) in the City.

It is worth noting that the composition of the raw wastewater presented a considerable variation for COD, BOD and TSS parameters during the period of study as shown by the high standard deviation values, however, small variation for others. The environmental conditions and the actual domestic wastewater features at the site, gave good reasons to carry out this research work at Al-Bireh WWTP. Furthermore, an adequate start-up phase for the UASB-septic tank was a prerequisite in order to study its performance under real operating conditions.

The results of COD fractions for the influent to Al-Bireh WWTP presented in Table 4.1 and Fig. 4.1, show that the COD_{sus} in the raw sewage constitutes a high fraction of the COD_{tot} about 53.8% (640 mg/L). This percentage is close to the values reported in literature for domestic sewage which were found to be in the range of 45-55% (Kalogo and Verstraete, 1999; Elmitwalli, 2000) and slightly lower than the 58% proportion found by Mahmoud *et al.* (2003) also for the sewage from Al-Bireh City.
COD_{col} represents 15.3% of the COD_{tot} in raw sewage, lower than the 20-30% proportion cited by Elmitwalli (2000) for the sewage from Bennekom-The Netherlands and higher than the 10% proportion reported by Halalsheh (2002) for the sewage from Amman City, Jordan. The results also reveal that main fraction of COD in the raw sewage is particulate (suspended and colloidal), which is represented 69.1% of the total COD and close to the value -about 70%- that was found by Wang (1994) in domestic sewage.

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

Parameter	# of Samples	Range	AVR	STD
COD Total	56	888-1718	1189	165.9
Suspended	56	404-954	640	114.4
Colloidal	56	122-321	182	41.6
Dissolved	56	189-598	367	103.6
VFA as COD	56	115-208	151	20.1
BOD_5	28	468-744	616	81.3
COD/ BOD_5	28	1.52-2.6	2.0	0.28
NKj as N	21	66-87	78	6.3
NH_4^+ as N	28	51-71	58.9	3.8
†Proteins			178	
Total PO_4 as P	19	11-17.4	14.0	1.5
PO_4^{3-} as P	19	10-15	12.6	1.14
SO_4^{2-}	12	95-159	124	16

TSS	24	490-951	614	118.9
VSS	24	394-720	512	91.6
SVI	5	16-29	21.5	4.7
pH	40	7.0-7.91	7.5	0.22
T_{ww}	104	18.2-29	24	1.96
$T_{amb.}$	179	15.5-34	24.1	3.16
$P_{atm.}$	2	0.917-0.929	0.923	0.008
Biodegradability	3	62-69	65	3.43
Fecal coliform*	23	-	$2.1x10^7$	0.61
Color		Medium brown		

All parameters are in mg/l except: sludge volume index (SVI) in ml/g; wastewater temperature (T_{ww}) and ambient temperature ($T_{amb.}$) (^oC); pH no unit; Atmospheric pressure ($P_{atm.}$) atom; Biodegradability (%); Proteins mg COD/l; Fecal coliform: CFU/100 ml; †: Calculated; *: from Samhan (2005)

It can also be seen from Table 4.1 that the ratio COD_{dis}/COD_{tot} is 30.9% in raw sewage. Part of the original COD_{col} and COD_{dis} fractions may has been degraded in the sewerage system before reaching the treatment plant, located in the outskirts of the city (about 1.5 km from the center), explaining low concentrations observed for these fractions. Figure 4.1 shows the evolutions of the concentrations of COD fractions (COD_{sus}, COD_{col} and COD_{dis}) of the influent sewage to Al-Bireh wastewater treatment plant during the whole period of experiment.

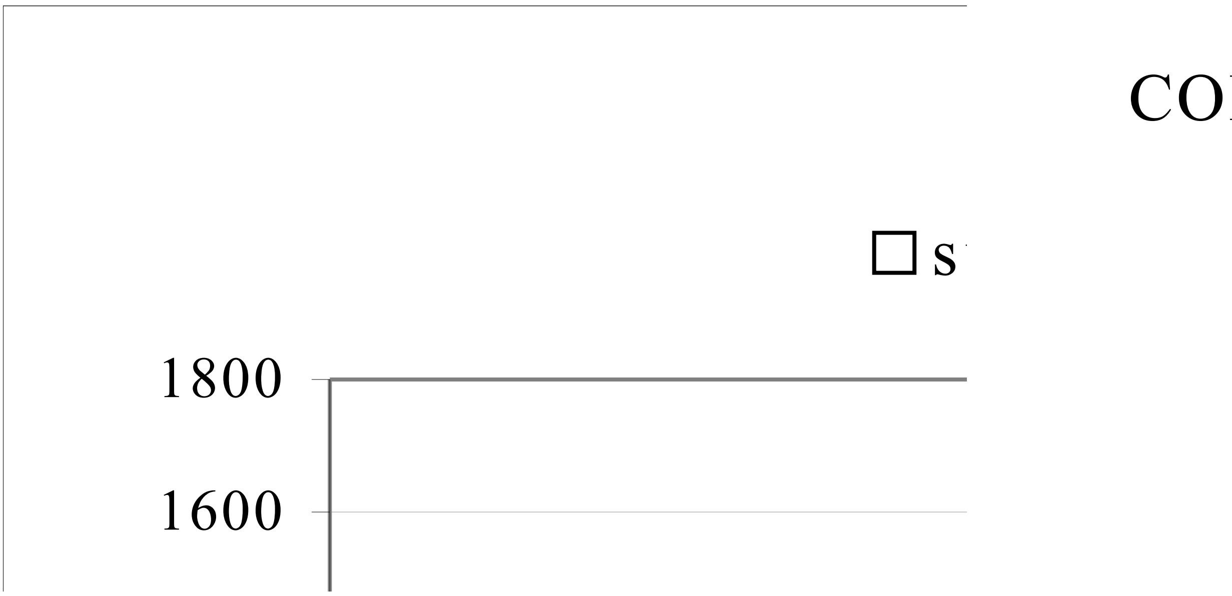

Figure 4.1. The evolutions of the concentrations of COD fractions (COD_{sus}, COD_{col} and COD_{dis}) of the influent sewage to Al-Bireh wastewater treatment plant during the whole period of experiment

The average concentration of the VFA as COD of the influent to Al-Bireh WWTP presented in Table 4.1 is relatively high (151 mg/L), probably due to some hydrolysis and acidification taking place during the transportation time before the wastewater reaches the treatment plant. Likewise, Halalsheh (2002) showed high concentrations of VFA as COD around 150 mg/l in the influent sewage to the Abu-Nusier WWTP in Jordan. Mahmoud *et al.* (2003) found that the average value of the VFA as COD in the raw swage enters Al-Bireh WWTP was about 160 mg/l. The results of VFA as COD and COD ratios are presented in Table 4.2.

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

	Parameter	Palestine(1) Al-Bireh	Palestine(2) Al-Bireh	Jordan(3) Amman
Acidified fraction	VFA/COD_{tot}	12.7	10	9.4
Acidified of dissolved	VFA/COD_{dis}	41.1	36	40
Hydrolysed fraction	COD_{dis}/COD_{tot}	30.9	28	23.5
	Protein-COD/COD_{tot}	15	14	48
	VSS/TSS	83	84	72
	COD_{sus}/VSS	1.25	1.49	3.21

(1), this study; (2), Mahmoud *et al.* (2003); (3), Halalsheh (2002)

The mean total and volatile suspended solids (TSS, VSS), and the VSS/TSS ratio that found in this study area for raw sewage were 614 mg/L, 512 mg/L and 83%, respectively. These values are significantly higher than the values reported by Elmitwalli (2000) and Halalsheh (2002) for domestic sewage. This might be due to the difference in people habits. In general, the obtained results in this research with respect to the raw sewage strength of Al-Bireh City, is relatively lower than the average values reported by Mahmoud *et al*. (2003) for most of the parameters as shown in Table 1.3, Chapter 1.

Sewage temperature. Mean temperature of raw sewage during the period of experiment was 24ºC. Extreme values observed were 18.2ºC and 29ºC. Sewage from the city of Al-Bireh seems to be, on average, warm enough to be treated anaerobically. However, the situation in winter has to be taken into account because sewage temperature can drop below 15ºC for some months. Sewage temperature was on average around the ambient temperature.

Biodegradability of the wastewater. In raw sewage, total anaerobic biodegradability was in the range of 62-69% with an average of 65%, after 120 days at 30ºC incubation based on equations 3.4 and 3.5 (Chapter 3). This wastewater is considered to be biodegradable under anaerobic conditions. This also can be confirmed by the results obtained for BOD/COD percentages that show an average value of about 50% with a maximum of 65% for the influent to Al-Bireh WWTP, which indicates that the COD in raw sewage is potentially biodegradable. The obtained results were relatively close to values reported in literature. Elmitwalli (2000) reported that the total anaerobic biodegradability of raw sewage from Bennekom village in The Netherlands was 74% either at 20 and 30ºC, after 135 and 80 days of digestion, respectively. Halalsheh (2002) reported biodegradability of strong domestic sewage ranging from 76 to 79% at 25ºC for different sewage sources in Jordan, in tests lasting from 130 to 224 days. Likewise, Seghezzo (2004) reported that the total anaerobic biodegradability of raw sewage in Salta City, Argentina, was approximately 70 and 65% at 30 and 20ºC, respectively.

4.2 Reactors inoculation

The long periods and some degree of uncertainty during the start-up phase are probably the main drawbacks of full-scale UASB reactors used in domestic wastewater treatment. In this sense, Haandel and Lettinga (1994) point out that operational conditions as well as quality and quantity of seed sludge are key factors that have a strong influence on the duration of start-up. These authors quote the experiences of Kampur (India) where a

UASB reactor was started-up in three months. In another experience, in Sao Paulo (Brazil) a 120 m^3 UASB was started-up in four weeks at an initial HRT of 16 hours but using a granular sludge inoculum. Thus, it seems that quality of the seed sludge, organic loading rates and operational conditions together determine the duration of the start-up phase of UASBs treating domestic wastewaters. The characteristics of the sludge used as inoculum in this experiment are shown in Table 4.3. As already mentioned, the seed sludge used in this research was taken from an old anaerobic cesspit serving a small residential house in Al-Bireh City in order to shorten the start-up period of the reactors. The seed sludge used in this research considered as anaerobic sludge with poor quality, flocculent in type and not well stabilized as shown from the obtained results of VS/TS and stability values (Table 4.3). Nevertheless, it is well acclimatized with the wastewater constituents, this was observed from the release of methane gas immediately after feeding the reactors with wastewater. From the results obtained, it is worth mentioning that the amount of sludge equals to10% of reactor volume, such the case in R2, is adequate to seed and start-up a new UASB-septic tank reactor. This is agreed with what revealed by Lettinga *et al*. (1991) that the minimum amount of seed sludge required for proper operation of the system only amounts to approximately 8-10% of the reactor volume, both for black and grey wastewater.

Table 4.3. Characteristics of the seed sludge used in the experiment

Parameter	Value
COD_{tot}	18.30
TS	13.78
VS	9.58
VS/TS	0.7
TSS	11.15
VSS	8.59
Stability^{+}	60 %

All parameters are in g/l except: stability (%); VS/TS ratio
$^{+}$ After 100 days

4.3 Performance of the two UASB-septic tank reactors

The performance of the two pilot-scale UASB-septic tank reactors was monitored for half a year, starting at the end of April 2004. After six months of continuous operation of the two UASB-septic tank reactors treating real domestic wastewater at ambient temperature and under different HRTs (2 & 4 days for R1 and R2, respectively), the obtained results and the calculated removal efficiencies for the two reactors over the whole period of operation are depicted in Table 4.4. Mean organic loading rates (OLR) applied during the whole period of operation were 0.6 $kgCOD_{tot}/m^3_{reactor}.d$ (range 0.44-0.86) and 0.3 $kgCOD_{tot}/m^3_{reactor}.d$ (range 0.22-0.43) in R1 and R2, respectively.

4.3.1 COD removal efficiency

The mean values of effluent COD_{tot} and fractions and the calculated removal efficiencies of the two UASB-septic tank reactors (R1 & R2) are depicted in Table 4.4 and Figures 4.2, 4.3, (4.4; 4.5) and (4.6; 4.7) for COD_{tot}, COD_{sus}, COD_{col} and COD_{dis}, respectively.

The results of R1 (2 days HRT) over a period of six months, showed average removal efficiencies (with standard deviation in brackets) of 54% (6), 85% (6), 27% (19) and 12% (20) for COD_{tot}, COD_{sus}, COD_{col}, COD_{dis}, respectively. Likewise, the average removal efficiencies in R2 during the whole experimental period were 58% (7), 89% (4), 32% (17) and 14% (25) for the same parameters.

As shown from Table 4.4, R1 observed to have a slightly lower removal compared to R2 for COD_{tot} and the separate distinguished COD fractions. This variety in efficiencies between the two reactors can be explained, to a great extent, by the difference in hydraulic conditions, reflecting physical phenomena rather than changes in the biological characteristics of the reactors. This can be confirmed by the statistical analysis conducted on the total COD and fractions, which revealed that the differences in removal efficiency and effluent concentration between the two reactors were statistically significant just only for COD_{tot} and COD_{sus} ($\rho<0.05$), as will be discussed later.

Table 4.4 shows a stable performance with respect to COD removals for the two UASB-septic tank reactors along the period of study. This can be seen from the standard deviation (STD) figures, which varied in a small range within each reactor. On the other hand, variations of COD across the experiments showed that COD_{sus} was the more stable parameter followed by COD_{tot}, COD_{col} and COD_{dis} in the two reactors. Figures 4.2, 4.3, (4.4; 4.5) and (4.6; 4.7) depict the average variation of COD_{tot}, COD_{sus}, COD_{col} and COD_{dis}, respectively.

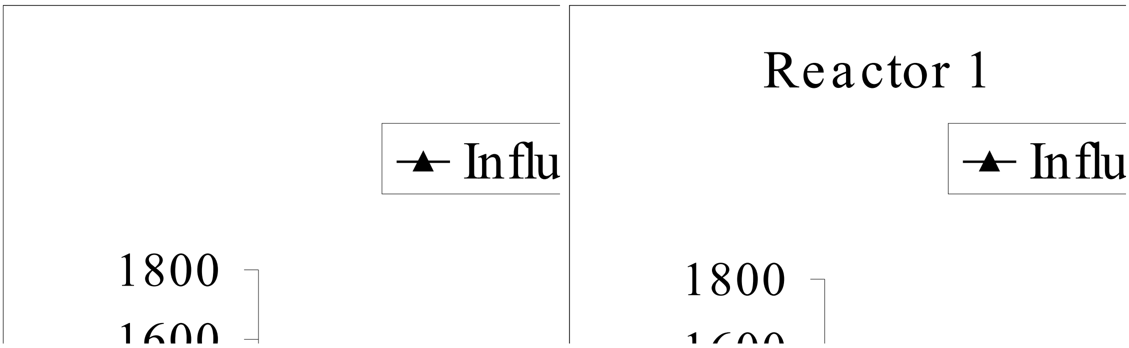

Figure 4.2. COD_{tot} influent and effluent concentrations and removal efficiencies for R1 (left) and R2 (right)

The two reactors deal successfully with simultaneous shock load of COD_{tot} (from day 85 to day 92) as shown in Fig. 4.2, since the overall removal efficiency of COD_{tot} for the two reactors was not affected, while the effluent COD concentration increased. The effluent COD_{tot} was in the range of 366-685 mg/L (AVR: 537 mg/L; STD: 60) for R1 and in the range of 266-810 mg/L (AVR: 493 mg/L; STD: 95) for R2. In general, the effluent qualities for R1 and R2 in terms of COD_{tot} were relatively stable throughout the experiment, maintaining a rather constant effluent concentration and seemed to be not significantly affected by the fluctuation of influent concentration (Fig. 4.2). Although acceptable removal efficiencies were achieved, the final effluent in terms of COD_{tot} from both reactors was not in compliance with discharge standards of 150-200 mgCOD_{tot}/L established by Ministry of Environmental Affairs (MEnA) on treated wastewater (MEnA, 2000).

In this study, removal efficiencies attained in R1 and R2 for COD_{tot} were in the range of results obtained with well functioning UASB reactors treating raw domestic sewage in sub-tropical regions. Recently, Halasheh (2002) reported COD_{tot} removal efficiencies of 58% and (50-62%), respectively for pilot and full scale UASB reactors treating raw domestic sewage at 24ºC in Jordan which is, from a wastewater composition point of view, very close to the Palestinian wastewater characteristics (see Table 2.4, Chapter 2). In the same context, Bogte *et al.* (1993) achieved 33% removal of COD_{tot} when raw domestic wastewater treatment was tested for 28 months at 13.8ºC in on-site UASB-septic tank reactor with 44.2 hrs (HRT) in Noordwuk, The Netherlands. However, the latter found that at summer temperatures (14-20ºC) the removal efficiency was 60%. The performance of on-site pilot scale UASB-septic tank reactors under different conditions was summarized in Table 2.5, Chapter 2.

In this research, very high COD_{sus} removal efficiencies were consistently recorded in R1 and R2, respectively 85% and 89% as shown from Table 4.4 and Fig. 4.3. COD_{sus} removal efficiency in both reactors was highly stable throughout the experiment and achieved the highest removal efficiency among the other COD fractions (Table 4.4). The results from statistical analysis show that the difference among R1 and R2 for COD_{sus} removal efficiencies, is statistically significant ($\rho<0.05$). This significance provided the evidence of the strong effects of HRT and the liquid upflow velocity (V_{up}) that related to the HRT, on the removal efficiency of suspended matters in UASB reactors treating domestic sewage.

In this sense, Mahmoud (2002) pointed out that the effect of HRT could manifest as a result of its direct relation to the V_{up} and also to the solids contact time in the reactor and so the possibility of solids to coalesce or to be entrapped in the sludge bed. This was observed clearly in our study, hence R1 operated at V_{up} of 0.05 m/hr, corresponding to HRT of 2 days, which is twice the V_{up} (0.025 m/hr) value applied for R2. Therefore, the removal efficiency for COD_{sus} in R1 is expected to be less than R2. However, it is worth to mention that the removal efficiencies for COD_{sus}, obtained in this study, are much better than that reported in literatures for conventional UASB reactors which almost operated with less HRT and more upflow velocities (see Table 2.4, Chapter2). This is also in agreement with that reported by Mahmoud (2002), increasing the V_{up} could reduce the removal efficiency of solids by increasing the hydraulic shearing force, which counteracts the removal mechanism though exceeding the settling velocity of more particles and detachment of the captured solids.

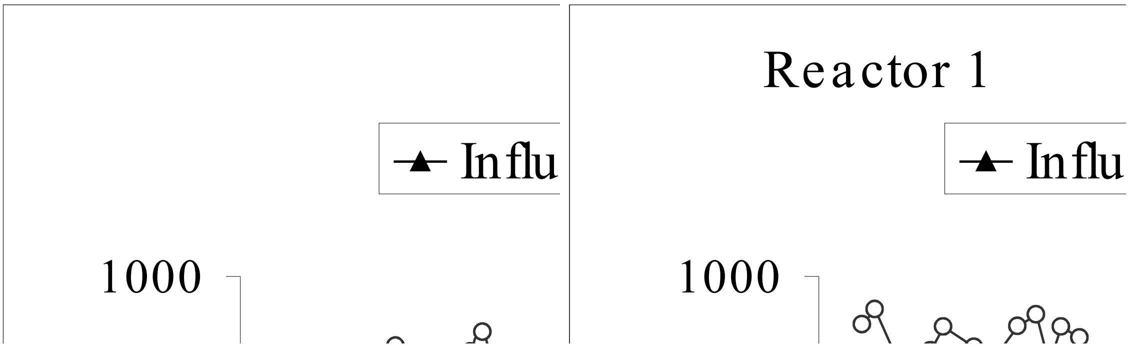

Figure 4.3. COD_{sus} influent and effluent concentrations and removal efficiencies for R1 (left) and R2 (right)

In this research, a clear trend in COD_{col} removal efficiency was not observed for both reactors (R1 and R2). This can be seen from Fig. 4.4 and 4.5, in which removal efficiencies are varied in a wide range. The wide range of removal efficiency and the negative removal of COD_{col} which observed in some cases in R1 and R2, probably can be attributed to both: the variable pattern of COD_{col} in the influent, and the improvement of COD_{sus} removal might result from the better digestion conditions, which also justify the improvement of COD_{col} removal, since colloids may be generated from the COD_{sus} as argued by Elmitwalli (2000) when he observed negative removal of COD_{col} in anaerobic reactor treating domestic sewage.

As mentioned previously, the average COD_{col} removal efficiencies for R1 and R2 were respectively 27% and 32%. The differences of COD_{col} removal efficiency found between the two reactors were not statistically significant ($\rho>0.05$). This is in agreement with that reported by Elmitwalli (2000) that UASB reactors are, in general, not very effective at removing colloidal matter, no matter what the hydraulic conditions are, and depends on biological processes and bioconversion. In the same context, Sayed and Fergala (1995) considered the 'entrapment' mechanism involved in removing solids by the sludge bed in the UASB reactors, not sufficient to remove colloidal particles. Elmitwalli (2000) reported that improvement of the colloidal fraction and therewith conversion to CH_4, could be imposed by addition of coagulants, for destabilization of the colloids and/or by pre-removal of the SS in a separate process. In this study, the higher 5% removal achieved by R2 mainly caused by the significantly better removal of COD_{sus} in R2 which also justify the improvement of COD_{col} removal caused by production of COD_{col} from COD_{sus}.

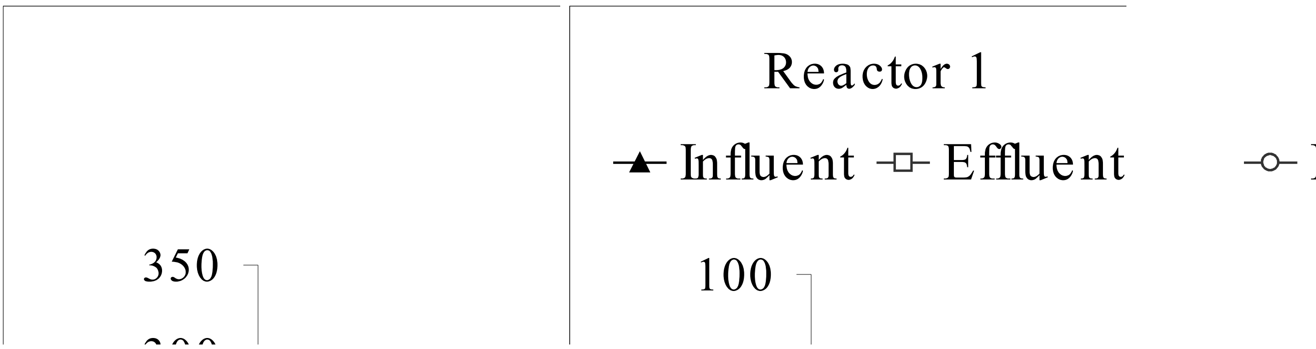

Figure 4.4. COD_{col} influent and effluent concentrations (left) and removal efficiencies (right) for R1

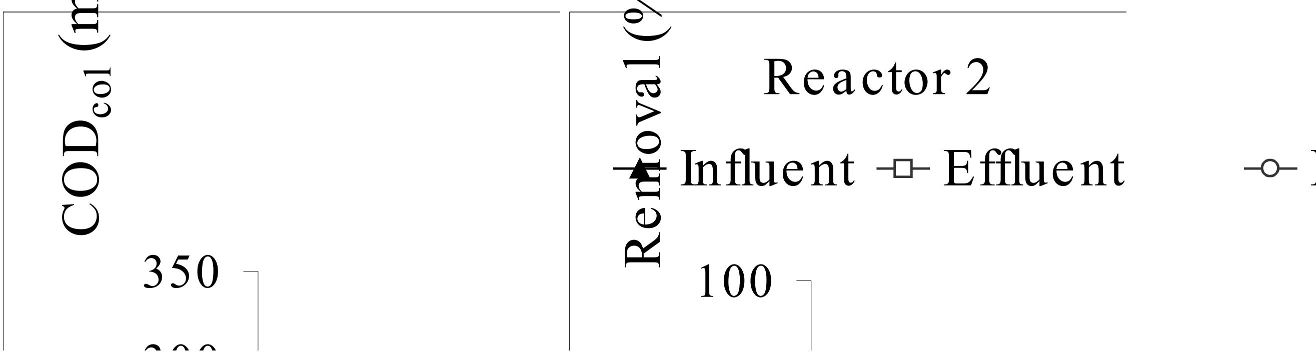

Figure 4.5 COD_{col} influent and effluent concentrations (left) and removal efficiencies (right) for R2

COD_{dis} removal efficiencies increased gradually in R1 and R2 since the beginning of the experiment as shown from Fig. 4.6 and 4.7. The existence of higher concentrations of soluble COD at the beginning of startup can be attributed to the hydrolysis of solid organic substances accumulated at the lower portion of the reactors, which results in liquefaction of the entrapped solids. During the period from day 1 to day 60, negative COD_{dis} removal efficiencies were observed in the two reactors indicating that the biological conditions during this period were close in both reactors as shown clearly in Figures 4.6 and 4.7 respectively for R1 and R2. This period (from day 1 to day 60) could be considered as an acclimatization period, in which the methanogenic activity was very low and seemed to be enhanced steadily. This can be also confirmed by the gradual increase of CH_4 gas production as displayed in Figures (4.6 and 4.7, left).

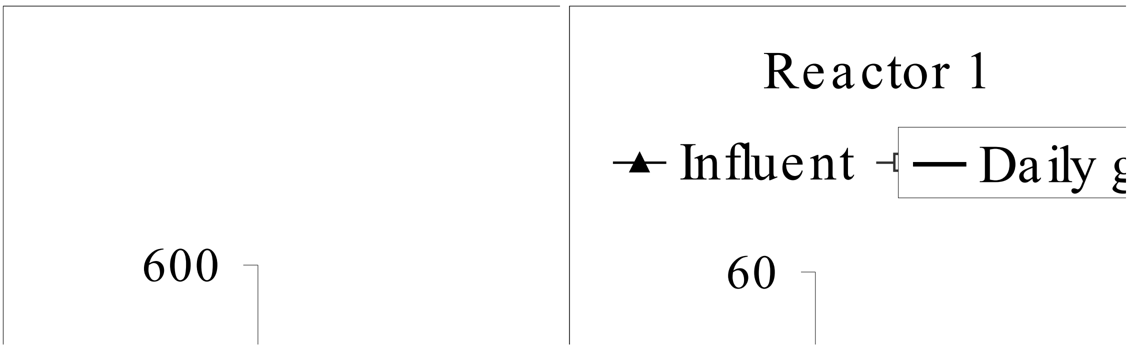

Figure 4.6. COD_{dis} influent and effluent concentrations (left) and removal efficiencies with relation to daily CH_4 gas production (right) for R1

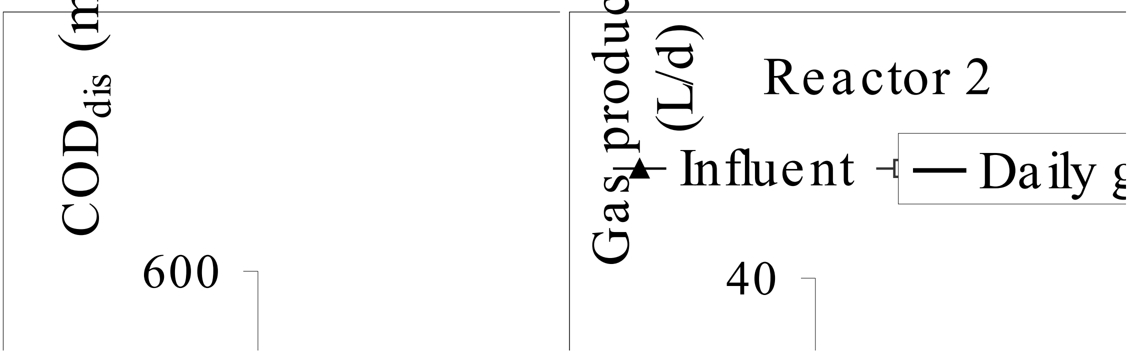

Figure 4.7. COD_{dis} influent and effluent concentrations (left) and removal efficiencies with relation to daily CH_4 gas production (right) for R2

The average COD_{dis} removal efficiencies for R1 and R2 were limited to 12% and 14% during the whole period of experiment (Table 4.4). However, the results of COD_{dis} removal were increased obviously in the period after the acclimatization period with average removals of 21% and 26% for R1 and R2, respectively. No significant differences were found for COD_{dis} removal efficiency between the two reactors ($\rho>0.05$). Removal efficiencies up to 50% and 63% in COD_{dis} have been observed respectively for R1 and R2 at day 81 of operation. After which a steep drop in COD_{dis} removal efficiencies were recorded in both reactors. This can be mainly attributed to the shock load of industrial sewage (with high grease content) that observed at day 85 and confirmed by the unstable performance of Al-Bireh WWTP at that moment. Consequently, the methanogenic activity in the both UASB-septic tank reactors was adversely affected and the COD_{dis} removals decreased (Figures 4.6 and 4.7). However, there is a shock load; the reactors are rapidly restored to normal.

The average COD_{dis} removal efficiencies that revealed in this study during the whole period of experiment are close to those reported by Bogte *et al.* (1993) (around 10%) in on-site UASB-septic tank reactor treating raw domestic wastewater treatment for 28 months at 13.8ºC with 44.2 hrs (HRT) in Noordwuk, The Netherlands. In this study, COD_{dis} represented about 60% of COD_{tot} in the final UASB effluents, in agreement with findings by Halalsheh (2002) and Seghezzo (2004). Wang (1994) found that 46% of effluent COD_{tot} after anaerobic sewage treatment could be attributed to non-acidified COD_{dis}. Volatile fatty acids (VFA) in the final effluent were always about 160 mg/L, an amount that could well account for most of the measured COD_{dis}. Moreover, soluble microbial products (SMP), which are resistant to anaerobic degradation, could also be responsible for part of the effluent COD_{dis}, as suggested by Elmitwalli (2000) and Mahmoud (2002). Halalsheh (2002) reported that 81% of the COD_{dis} in the effluent of a UASB reactor was not anaerobically biodegradable.

The results of COD_{dis} removals obtained in this study and the discussion above, provided strong reasons to believe that the removal of dissolved COD (COD_{dis}) is mainly a biological process in addition to some physical aspects, such as a good contact between wastewater and biomass, mixing, wastewater viscosity, temperature, solubility of gas, which all affect the biological conditions in the UASB reactors.

The evolutions of VFA as COD concentrations and removal efficiencies in R1 and R2 (Fig. 4.8 and 4.9) were clearly observed to have the same trend in behaviour as COD_{dis} (Fig. 4.6 and 4.7). This can be explained by the same reasons discussed above for COD_{dis} and by the considerable amounts of VFA in the final effluents of R1 and R2, were always could well account for most of the measured COD_{dis}, as mentioned before.
Considering the whole period of study, the mean VFA as COD concentrations in the effluents of both reactors were slightly higher than influent values. The average concentrations of VFA as COD increased from 151 mg/L in the influent to 163 and 160 mg/L respectively in the effluents of R1 and R2 (Table 4.4). This increase in VFA concentration is mainly as a result of the predominant acidification process occurred in the two reactors, as it will be shown later. Moreover, the reasons behind the limited VFA removal could be attributed to mass transfer limitations in the reactors due to the low applied upflow velocities, and probably because their anaerobic biodegradability was limited. According to Bogte *et al*. (1993), results from determinations of VFA concentrations before and after on-site UASB-septic tank treatment showed that VFA increased from 118.6 to 119.7 mg/L.

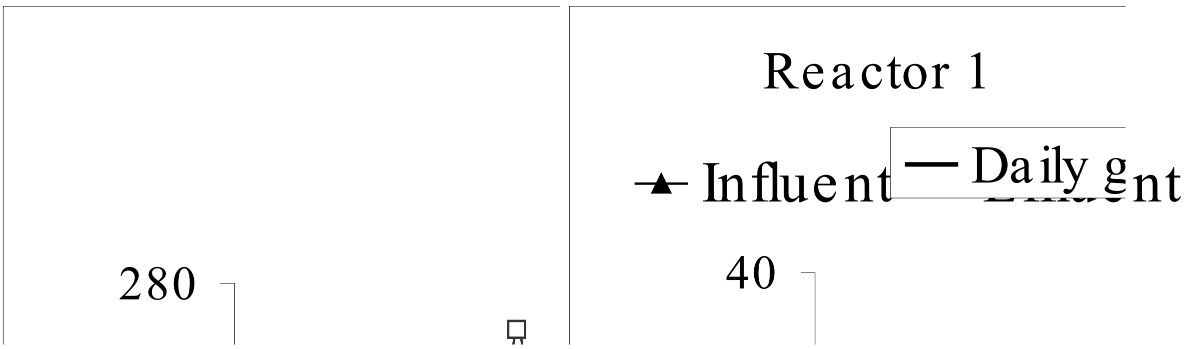

Figure 4.8. VFA as COD influent and effluent concentrations (left) and removal efficiencies with relation to daily gas production (right) for R1

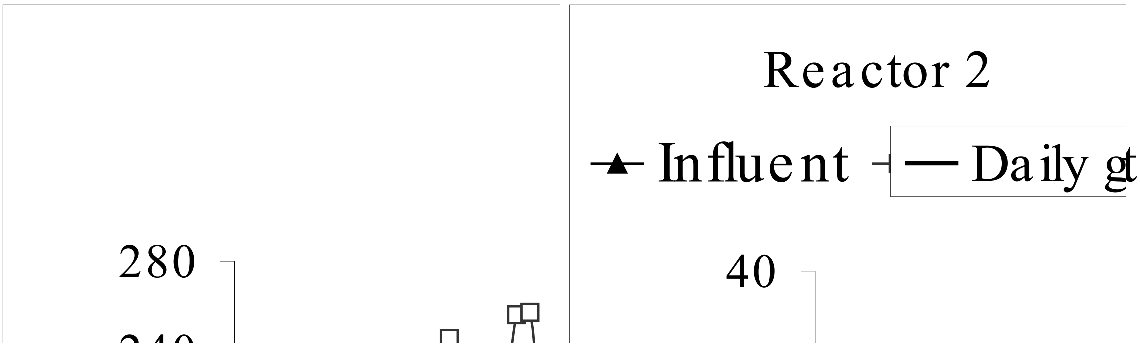

Figure 4.9. VFA as COD influent and effluent concentrations (left) and removal efficiencies with relation to daily gas production (right) for R2

Figures 4.8 and 4.9 reveal that volatile fatty acids (VFA) remained always very high in the effluents since the very beginning of the operation. The VFA concentrations in effluents were observed to be greatly affected by temperature and by the available methanogenic conditions. Such the case in the period between day of 69 and of 78, high amounts of gas productions were detected, indicating that methanogenesis was exceeded and consequently the accumulated VFA was converted to CH_4. In this sense, Bogte *et al.* (1993) reported that falling temperatures resulted in reduced production of VFA; however, prolonged low temperatures reduced the methanogenic activity so severely that complete conversion of VFA to CH_4 ceased to take place, causing the VFA concentration to increase again. Bogte *et al.* (1993) also reported that the production of volatile fatty acids increased when the temperature rose above 8ºC, and complete conversion of VFA into CH_4 was achieved during 3 to 4 months of the second year of UASB-septic tank operation, when temperatures above 15ºC.

Biodegradability of the effluent wastewater. The total anaerobic biodegradability of the effluent sewage from the UASB-septic tanks was approximately 42% for (R1) and 39% for (R2) after 120 days at 30ºC incubation. Effluent sewage is likely to be less biodegradable than raw sewage due to its lower amount of highly biodegradable suspended solids. Moreover, the high COD_{dis} proportions detected in the effluents also could contribute to less biodegradability. As mentioned before, Halalsheh (2002) found that 81% of the COD_{dis} in the effluent of a UASB reactor was not anaerobically biodegradable. The results also show that the effluent biodegradability in R2 is less than R1. This can be justified by the observed better efficiency of R2 in suspended solids removals. In this respect, it is interesting to note that the effluent from UASB-septic tank reactors can also be directly post treated aerobically.

Table 4.4. Research results for the effluent concentrations and removal efficiencies (%) 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	Samples #	Influent concentration	UASB-septic tank 1 (R1) (HRT = 2 days)				UASB-septic tank 2 (R2) (HRT = 4 days)			
			Effluent concentration		Removal efficiency (%)		Effluent concentration		Removal efficiency (%)	
			Range	AVR	Range	AVR	Range	AVR	Range	AVR
COD Total	51	1185	366-685	537 (60)	42-73	54 (6)	266-810	493 (95)	45-78	58 (7)
Suspended	51	643	21-206	97 (43)	72-96	85 (6)	17-143	69 (29)	75-97	89 (4)
Colloidal	51	180	65-197	129 (30)	-26-50	27 (19)	71-225	121 (31)	-19-64	32 (17)
Dissolved	51	361	146-419	311 (64)	-37-50	12 (20)	108-491	304 (80)	-42-63	14 (25)
VFA as COD	51	151	85-259	163 (37)	-69-42	-9 (27)	72-257	160 (45)	-76-58	-7 (33)
BOD_5	28	616	190-314	264 (38)	33-72	56 (10)	195-321	248 (36)	46-71	59 (7)
NKj as N	21	78	53-77	65 (6.1)	-1-37	16 (9)	55-78	68 (6.7)	-6-30	12 (10)
NH_4^+ as N	28	58.9	50-63	56 (3.5)	-3-17	5 (6)	51-67	59 (4.4)	-16-17	-0.4 (8)
Total PO_4 as P	19	14.0	11.9-16.8	13.7 (1.2)	-11-16	2 (7.2)	12.5-16.8	14.2 (1.1)	-33-11	-2 (9.8)
PO_4^{3-} as P	19	12.6	13.7-17.8	16 (1.1)	-49-(-3)	-28 (11)	14.8-18.3	16.7 (1)	-61-(-15)	-33 (12)
SO_4^{2-}	12	124	28-40	34 (3)	65-79	72 (4)	27-45	36 (5)	62-78	71 (5)
TSS	24	614	100-165	123 (17)	69-87	79 (5)	84-160	117 (19)	70-87	80 (4.6)
VSS	24	512	76-132	104 (15)	69-88	79 (4.9)	74-145	101 (17)	69-87	80 (4.9)
VSS/TSS	24	83	76-90	84 (4.6)	-	-	72-92	86 (4.6)	-	-
SVI	5	21.5	None	None	-	-	None	None	-	-
pH	40	7.5	7.12-7.64	7.35 (0.13)	-	-	7.12-7.7	7.4 (0.14)	-	-
Biodegradability	3	65	40-45	42 (2.7)	-	-	37-40	39 (1.4)	-	-
Fecal coliform*	18	2.1×10^7	-	1.55×10^6 (0.93)	-	16 (11)	-	1.26×10^6 (0.6)	-	17 (11)

All parameters are in mg/l except: sludge volume index (SVI) in ml/g.SS; pH no unit; VSS/TSS (%); Biodegradability (%); Fecal coliform: CFU/100ml.
*: from Samhan (2005)

4.3.2 Hydrolysis, Acidification, and Methanogenesis

The calculated average values for hydrolysis, acidification, and methanogenesis in R1 and R2 during the whole period of experiment are depicted in Table 4.5. The considerable fluctuations in the domestic sewage concentration and composition led to high standards deviations in the mean value of hydrolysis, acidification, and methanogenesis (Table 4.5). The results depicted in Table 4.5 demonstrate that hydrolysis, acidification, and methanogenesis remain low in both reactors probably due to short SRT. Zeeman (1991) found during 125 days of a batch digestion of cow manure 18, 27 and 45% hydrolysis at temperatures of 15, 25 and 30°C respectively. Moreover, results clearly reveal that the methanogenesis was limiting the overall conversion of organic matter to methane in R1. However, hydrolysis and/or methanogenesis were the limiting steps in R2. These findings are in agreement with what reported in literature that hydrolysis of particulate matter, suspended and colloidal; is generally considered to be the rate-limiting step in the whole digestion process (Eastman and Ferguson, 1981; Zeeman *et al.*, 1997; Sanders, 2001).

Moreover, acidification in our reactors was appeared to be the predominant step among the other digestion steps. Interestingly, the effluents from both reactors contained a high amount of soluble COD around 310 mg COD/L of which 160 mg COD/L was in the form of VFA. Acidification occurred in the UASB-septic tank reactors resulting in an increase of the VFA/COD_{dis} from 41% in the influent to 52% in the effluent, which is an advantageous to a subsequent post-treatment.

Table 4.5. The calculated average values for hydrolysis, acidification, and methanogenesis in both reactors (R1 and R2) during the whole period of experiment. Standard deviations are presented in brackets

Parameter	Reactor 1 (R1)	Reactor 2 (R2)
H (%)	16 (9.5)	17 (8.5)
A (%)	19 (7.0)	20 (8.4)
M (%)	15 (6.9)	17 (7.0)

Fig. 4.10 shows the course of the hydrolysis, acidification, and methanogenesis in the R1 and R2 during the whole period of experiment. Moreover, since the two UASB-septic tank reactors were operated in parallel with the same domestic sewage, a good comparison between the two reactors can be made from Fig. 4.10 in addition to the statistical analysis. Statistically, significant differences were ($\rho<0.05$) only found for methanogenesis percentages between the two reactors. The reason behind the slightly better conversion of organic matter in R2 could be attributed to the lower OLR in R2 (0.3 $kgCOD_{tot}/m^3.d$) than R1 value (0.6 $kgCOD_{tot}/m^3.d$), since higher OLR will reduce the contact between substrate and biomass resulting in change of the sludge bed composition (microbial, physical and chemical) and cause accumulation of undigested ingredients (Mahmoud, 2002).

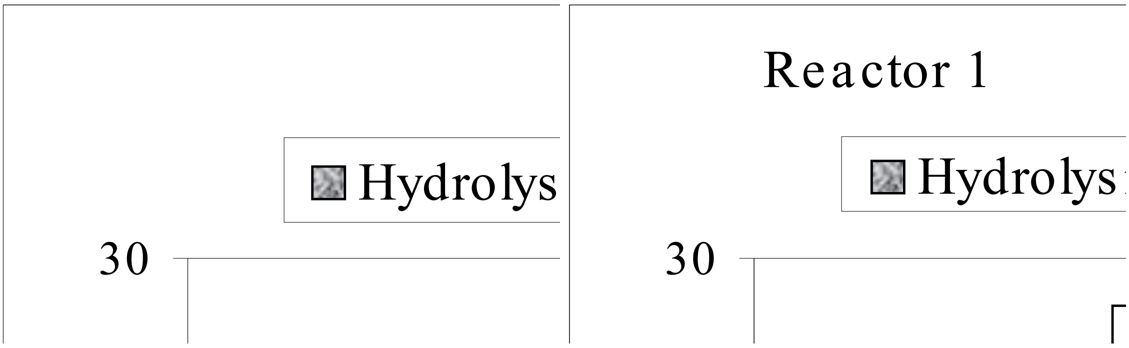

Figure 4.10. Percentages of hydrolysis, acidification, and methanogenesis of domestic sewage in R1 (left) and R2 (right)

4.3.3 Biogas production

The on-site measurements of the recovered CH_4 as biogas were, on average, 19.7 (14.2) and 10.9 (7.1) L/d respectively for R1 and R2 during the whole period of operation. Fig. 4.11 depicts the time course for the biogas CH_4 production in R1 and R2 and the ambient temperature along the study period. Methane content in the biogas was determined, as already mentioned, by stripping the CO_2 in a tightly, closed, glass cylinder with a 16% NaOH solution. CO_2 was retained in the solution. The content of other gases in the biogas, like hydrogen sulfide, was neglected. From literature, the methane content amounted to about 70-90% of the total biogas produced in UASB reactors treating domestic wastewaters (Bogte *et al.*, 1993; Halalsheh, 2002; Mahmoud, 2002; Seghezzo, 2004).

However, the average "total" CH_4 production during the entire period was 29.6 L/d for R1 and 16.9 L/d for R2. Hereafter, total CH_4 production refers to the sum of the amount of CH_4 collected in gas form and that escaped in dissolved form in the effluent. Dissolved methane in the effluents was calculated according to Henry's law (Metcalf and Eddy, 1991). The following assumptions, according to Seghezzo (2004), were considered when the dissolved methane was calculated: (1) the effluents of R1 and R2 were always exposed to an atmosphere of biogas in the space around the GLS separator device on top of the reactor, and in the effluent bucket; and equilibrium concentrations were reached; (2) temperature, pressure were constant; (3) dissolved methane was not found as COD in laboratory analyses, because it left the liquid phase during sampling, storage, and measuring. The partial pressure of CH_4 gas was assumed to be 0.7 atm. Atmospheric pressure at Al-Bireh WWTP was 0.923 atm, measured at the site.

After calculations, around 33.5% and 29.5% of the total produced methane gas in R1 and R2 respectively were lost as dissolved in the effluents. The average total methane production from both reactors was 0.1 $Nm^3/kgCOD_{removed}$ (letter N indicates that volume is expressed at STP conditions). The results obtained here reveal that, no difference was found between the two reactors, regarding the ratio of total methane production to kg COD removed. However, this ratio was in the range of (0.04-0.25) and (0.03-0.2) $Nm^3/kgCOD_{removed}$ for R1 and R2, respectively. These values are close to those reported by Uemura and Harada (2000) (0.16 $Nm^3/kgCOD_{removed}$), by Mahmoud (2002) (0.15 $Nm^3/kgCOD_{removed}$) or by Seghezzo (2004) (0.1 $Nm^3/kgCOD_{removed}$). Considering that the theoretical ratio, maximum possible methane production from organic matter, is 0.35 $Nm^3/kgCOD_{removed}$ (Haandel and Lettinga, 1994). This suggested that a part of the influent solid COD remained within the reactors without being liquefied.

Fig 4.11 demonstrates that the methane gas production in the UASB-septic tank reactors is strongly influenced by the ambient temperatures. The patterns of gas production in Fig. 4.11 clearly display the increased adaptation of the microbial population and increase of temperature, resulting in such a gradual enhancement in the methanogenic activity since the beginning of the experiment. During the first month of operation, the methanogenesis seemed to be very low, resulting in a gradual accumulation of COD in the reactors. Just after that, the biogas production increased. A steep increase during the period between day of 69 and of 78 was detected indicating that methanogenesis was exceeded and consequently the accumulated COD was converted to CH_4. This steep increase of gas production in both reactors accomplished significantly better efficiencies, especially for dissolved COD removals.

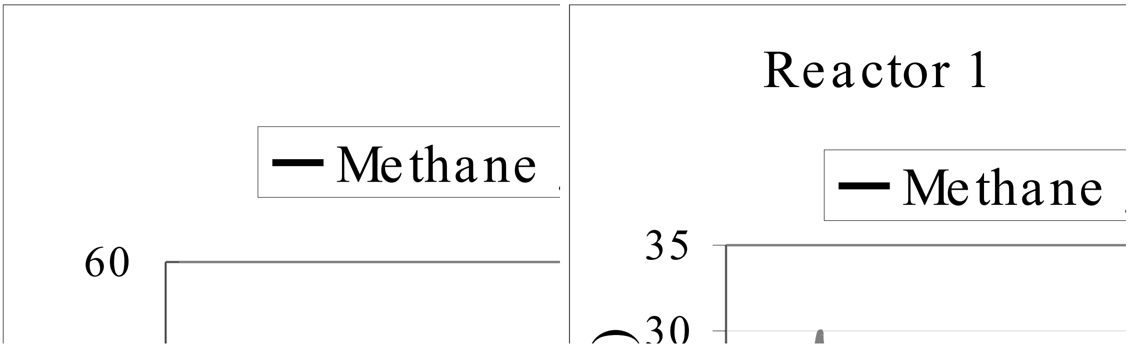

Figure 4.11. The time course for the methane production as biogas in R1 (left) and R2 (right) and the ambient temperature during the whole operational period

4.3.4 COD mass balance

COD balances over UASB reactors might be a useful tool to get insight into the flow of organic matter through the reactor, assess the performance of the process, validate methods and assumptions, and predict outputs (Seghezzo, 2004). A COD balance is based on the fact that the daily mass of influent COD is equal to the sum of the daily mass of COD leaving the system in one of several possible forms (methane, excess sludge, effluent COD, among others). Some researchers have provided information about their systems that could lead to the formulation of COD balances (Bogte *et al.*, 1993; Mahmoud, 2002; Seghezzo, 2004).

For this study, the monthly COD mass balances over the two UASB-septic tank reactors (R1) and (R2) were presented in Fig. 4.12. The COD balances were built among the following: influent and effluent total COD, total produced CH_4 as COD (gas form and dissolved), and accumulated COD. The term COD as excess sludge was not included in these COD balances, since no excess sludge was discharged during the study. In Fig. 4.12, where monthly COD balances are given, it can be seen that the reactors initially worked as a septic tank in the first month; hence the removal of COD was the result of accumulation. After the first month of operation, however, when the temperature rose (see also Fig. 4.11) microbial conversion started up, resulting in an increase of CH_4 production and, partly because of the turbulence in the reactors, in a slight decrease of accumulation. Dissolved COD was responsible for most of the methane production, but from COD balances it was apparent that some methane was also produced from the hydrolysis and fermentation of entrapped particulate organic matter.

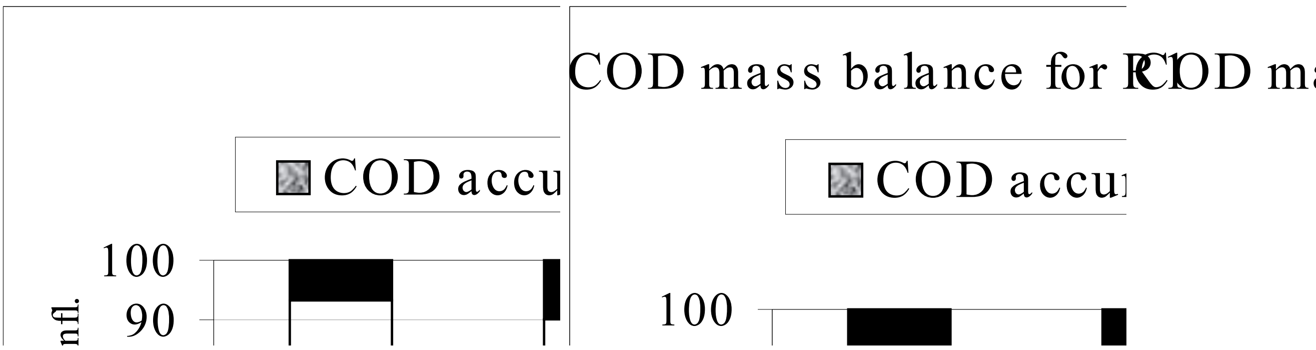

Figure 4.12. COD mass balances per month for R1 (left) and R2 (right) as a percentage of the average influent COD_{tot} and divided over COD accumulated, COD_{effl} and COD as CH_4

COD mass balances of the two reactors during the total test period (6 months) were displayed in Fig. 4.13. The figure reveals that around 40% of the incoming COD was retained and accumulated in the reactors while a relatively lower proportion was converted to methane. The relatively higher proportions of COD accumulated and CH_4 found in R2, also justify the slightly better removal efficiencies that detected in R2 than in R1.

Months

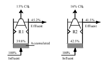

Figure 4.13. COD mass balance of R1 (left) and R2 (right) over the total test period (6 months) as a percentage of the average influent COD_{tot} and divided over COD accumulated, COD effluent and total CH_4 as COD

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

It should be noted here, that the sludge retained in the both UASB-septic tank reactors was only detected at port no.1 at 0.15 m from the bottom of the reactors and not exceeded the 0.4 m (port no.2) all over the study period. From there, the sludge samples were collected and analysed for the total solids (TS), volatile solids (VS) and stability. It seems that the sludge production was so low, due to the slow growth rates of anaerobic bacteria growth in the bottom of the reactors. Therefore, the reactors were not filled with sludge during the period of experiment and desludging of the reactors was deemed to be after long time of operation. This interesting observation is consistent with that reported in literature about the UASB-septic tank reactor, that the sludge hold-up time of the system is so long and the withdrawal of the sludge could be done once every 1 to 4 years (Kalogo and Verstraete, 1999; Zeeman *et al.*, 2000). This implied that the costs for sludge handling associated with sewage treatment, would be reduced dramatically by using UASB-septic tank reactors.

The characteristics of the retained sludge in the UASB-septic tank reactors (R1 and R2) are presented in Table 4.6 and Fig. 4.14. In general, little differences are observed between R1 and R2 with respect to sludge characteristics. However, the difference between R1 and

R2, for all sludge parameters, are not significant ($\rho>0.05$). Sludge hold-up was clearly observed in the two reactors as shown from the gradual increase in the total solids (TS) of the sludge (Fig. 4.14). The average sludge concentrations were 46.8 gTS/L and 48.6 gTS/L respectively for R1 and R2 during the whole period, indicating a higher sludge accumulation as compared to the first operational period (13.78 gTS/L).

Moreover, Fig. 4.14 shows a decline trend in VS/TS ratio of the retained sludge, which indicates a more stable sludge. According to Wang (1994), a VS/TS ratio of 63% can be considered a well stabilized sludge. In this study, the average values 73% and 71% for R1 and R2, respectively, indicates that the sludge is not being well stabilized. However, this is in contraction with the stability test (Table 4.6), which indicates good stability. On the other hand, Table 4.6 displays a slightly higher VS/TS ratio of the retained sludge in R1 in comparison with that from R2, which indicates better stability of the latter. This was also confirmed by the results of the stability test. The sludge retained in R2 has higher stability than the sludge retained in R1. This can be explained by the lower OLR that R2 subjected to, which consequently made the solids retention time (SRT) in R2, higher than that found in R1.

Table 4.6. Characteristics of the retained sludge in the UASB-septic tank reactors. Standard deviations are presented between brackets

Parameter	Reactor 1 (R1)	Reactor 2 (R2)
COD_{tot}	52.4 (7)	55.1(5.5)
TS	46.8 (8)	48.6 (5)
VS	34 (5)	34.6 (4)
VS/TS	73 (3)	71 (1.7)
COD/VS	1.55 (0.14)	1.6 (0.17)
Stability at day = 63	62 (2.4)	60.7 (2.1)
Stability at day = 102	56.2 (2.8)	51.8 (1.4)

All parameters are in g/l except: stability (%) (g CH_4-COD/g COD); VS/TS ratio; COD/VS ratio
$^{+}$ After 100 days

Likewise, it should be mentioned here that the development of granules was not detected in the UASB-septic tank reactors. This can be explained by a high concentration of suspended solids and COD in the influent (Kalogo and Verstraete, 1999).

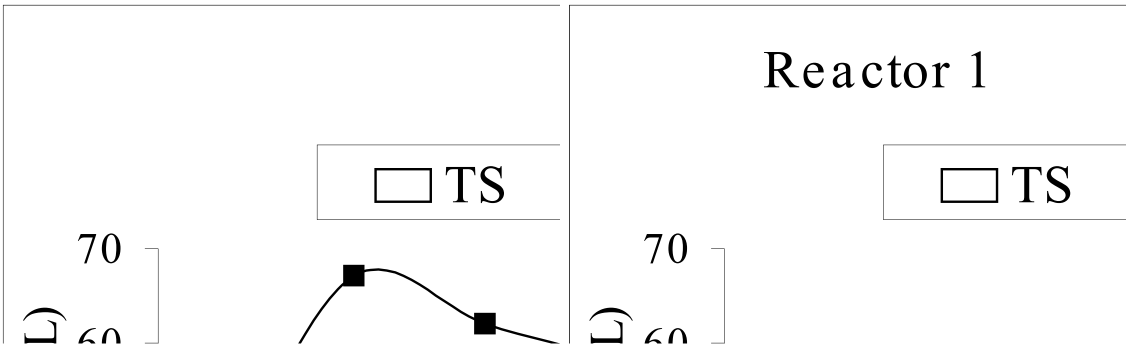

Figure 4.14. The time course for the sludge concentration in R1 (left) and R2 (right) as TS, VS and VS/TS ratio at 0.15 m height stands from the bottom of the reactors

The results obtained in this study, with respect to the retained sludge in the UASB-septic tank reactor, are in agreement with that reported in literatures for UASB reactors treating domestic wastewater (Table 4.7).

Table 4.7. Sludge characteristics in one stage UASB reactors treating domestic wastewater in the countries of Jordan and The-Netherlands.

Parameter	Jordan(1)	The Netherlands(2)
COD_{tot}	23-45	-
VS/TS	66	71
COD/VS	1.42-1.95	1.87
Stability	6-11*	45.6

(1) Halalsheh (2002); (2) Mahmoud (2002). All parameters are in g/l except: stability (%) (g CH_4-COD/g COD); VS/TS ratio; COD/VS ratio; *: (g $VS_{degraded}$/g VS)

4.3.6 Sludge wash-out and scum layer phenomena

Sludge wash-out phenomenon. Many researchers have reported about the problem of sludge washouts from UASB reactors treating a complex wastewater like domestic sewage. So for instance lately Halalsheh (2002) reported significant washouts in the big UASB reactor treating strong domestic wastewater of the city of Amman in Jordan. These wash-outs, although they mainly consist of poorly biodegradable (well stabilized matter), are responsible for the deterioration of the effluent quality.

In our study, sludge washout from the UASB-septic tank reactors was prevented to a large extent by the presence of baffles under the GLS and/or by the very low upflow velocities imposed to the reactors. However, some intermittent washout events of sludge were observed during the study period, but with not significant amounts (2-3 g/L). This accidentally washouts can be likely attributed to the increasing rate at which biogas was produced, especially at the period from day 63 to day 78 (Fig. 4.15). It is possible that some sludge was accidentally washed out without notice during the experimental period. Therefore, fluctuations in TS concentrations of the sludge bed could be reflecting the sludge washout phenomenon (Fig. 4.14). Observations of the occurrence of sludge floating and associated problems like the wash-outs of sludge were also observed by Mgana (2003) while operating the 1.5 m^3 pilot single-step community on-site UASB reactor in Dar Es-salam, Tanzania.

Scum layer phenomenon. The literature available on this subject -scum formation- is scarce, although it is of prime importance as the scum is usually described as one of the main operational problems of anaerobic digesters (Pagilla *et al*., 1997). Moreover, during the treatment of strong domestic sewage, the production of a scum layer has been also experienced in full scale UASB reactors at Kanpur (India) and Amman (Jordan) (Haandel and Lettinga, 1994; Halalsheh, 2002). Different researchers reported several reasons behind the formation of scum, including: insufficient mixing, high grease content in the influent, severe temperature fluctuations, high concentrations of fatty acids, and accumulation of undegraded SS (Pagilla *et al*., 1997; Yoda and Nishimura, 1997; Kalogo and Verstraete, 1999).

In this research, and according to our investigations to the scum baffles placed on the top of the reactors, a thin scum layer was developed in the both reactors at the water-air interface inside the scum baffles (Photo 11, Appendix 2). The scum layer observed twice during the whole period off study and disappeared after few days of its formation (Fig. 4.15). This thin thickness of the scum layer formed in each reactor made the determinations of the scum layer volume and characteristics very difficult to be quantified or identified.

In our study, the formation of scum can be mainly attributed to high doses of grease and lipids come from industries in the influent. As pointed out by Halalsheh (2002) that the latter compounds tend to adsorb on sludge particles and have a strong tendency for flotation. Moreover, scum formation observed to be found after severe fluctuations in temperature and gas production (Fig. 4.15), which is in agreement with that reported above about the causes behind this phenomenon.

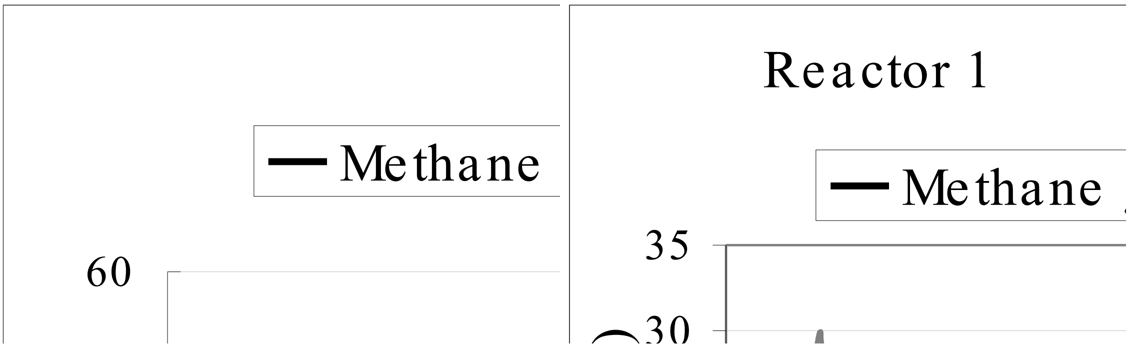

Figure 4.15. The scum layer periods (hatched circles) during the study period for R1 (left) and R2 (right) with relation to ambient temperature and gas production fluctuations

4.3.7 BOD removal efficiency

The mean values of the effluent BOD_5 concentration and the calculated removal efficiencies of the two UASB-septic tank reactors are depicted in Table 4.4. The average removal efficiencies during the whole period of study were 56% (10) and 59% (7) for R1 and R2, respectively. As shown, R2 achieved a slightly higher BOD_5 removal efficiency than R1, however, it is statistically significant ($\rho<0.05$). The average BOD_5 effluent was 264 (38) mg/L for R1 and 248 (36) mg/L for R2. Fig. 4.16 shows the average values of BOD_5 concentrations and removals for R1 and R2. The results presented in Fig. 4.16 reveal that the BOD_5 effluent qualities from both rectors were relatively stable throughout the experiment; however, the removal efficiencies were greatly affected by the influent concentration. These figures confirmed that removal of biodegradable organic matter took place from the very beginning of the start-up and was not greatly affected with time.

According to Bogte *et al*. (1993), 38% removal efficiency in terms of BOD_5 was achieved in on-site UASB-septic tank treating domestic wastewater in Noordwuk, The Netherlands.

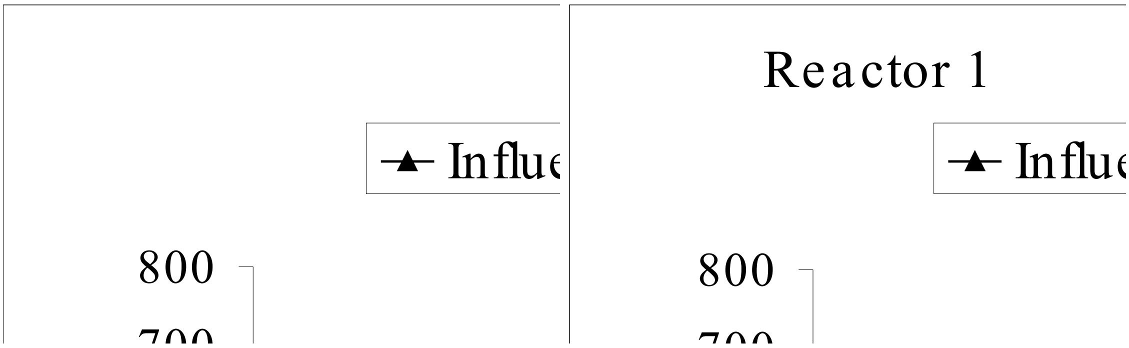

Figure 4.16. BOD_5 influent and effluent concentrations and removal efficiencies for R1 (left) and R2 (right) along the study period

4.3.8 TSS and VSS removal efficiencies

Removal of suspended solids in UASB reactors occurs by physical processes such as settling, adsorption, and entrapment. SS removal in UASB reactors depends on the type of sewage, temperature, and the combined effect of the sludge bed height and the liquid upflow velocity (V_{up}) in the reactor, the latter parameter related to the hydraulic retention time (HRT) and the reactor height (Elmitwalli, 2000; Mahmoud, 2002; Seghezzo, 2004).

Table 4.4 shows the average TSS and VSS effluent concentrations and the calculated removal efficiencies for the both UASB-septic tank reactors. Despite the high levels of solids in the influent, the removal of solids in the two reactors was highly satisfactory. The average removal efficiencies for TSS over the entire study period were 79% (5) and 80% (4.6) for R1 and R2, respectively. However, R2 is significantly better than R1 with respect to TSS removals ($\rho<0.05$). These efficiencies were more comparable to values reported by Lettinga *et al*. (1991) for domestic sewage treatment in a 0.86 m^3 UASB-septic tank reactor in Indonesia (Table 2.5, Chapter 2), however, those are more better than that reported in literature for conventional UASB reactors. Fig. 4.17 shows the average values of TSS concentrations and removals for R1 and R2.

The average removal efficiencies for VSS over the entire study period were also 79% (4.9) and 80% (4.9) for R1 and R2, respectively. However, statistically no significant differences in R1 and R2 have been detected for VSS removals (ρ>0.05). Fig. 4.18 shows the average values of TSS concentrations and removals for R1 and R2.

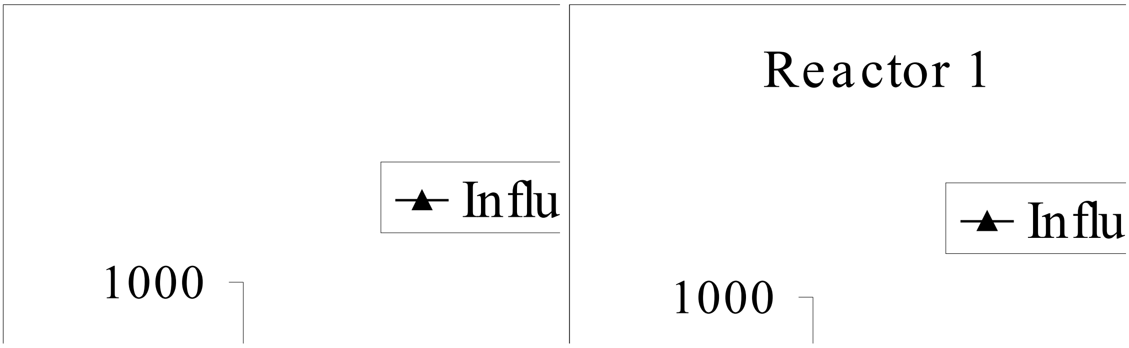

Figure 4.17. TSS influent and effluent concentrations and removal efficiencies for R1 (left) and R2 (right) along the study period

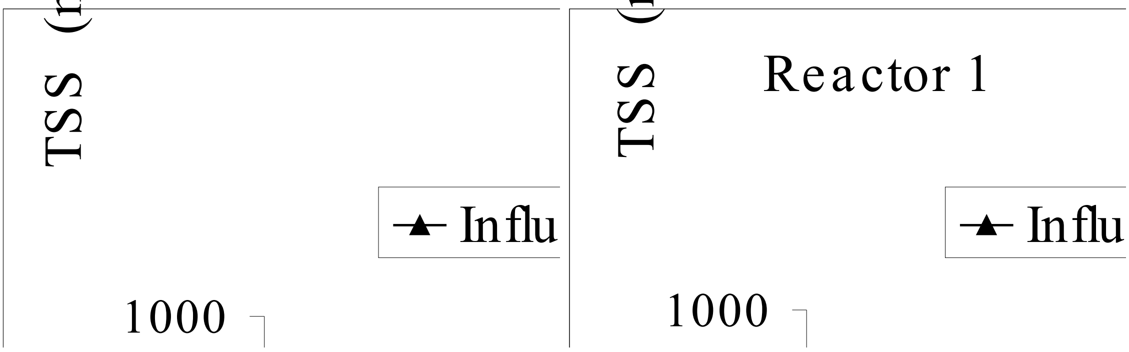

Figure 4.18. VSS influent and effluent concentrations and removal efficiencies for R1 (left) and R2 (right)

As shown from Fig. 4.17 and Fig. 4.18, the TSS and VSS removal efficiencies and the effluent concentrations in both reactors were highly stable throughout the experiment. This also can be seen from the low values of the standard deviations. The average VSS/TSS ratio in the effluents over the entire study period were 84 (4.8) and 86 (4.6) for R1 and R2, respectively. These ratios are relatively high compared to the findings reported by Halalsheh (2002) (VSS/TSS = 0.5) for effluent from a 96 m^3 one stage

UASB reactor treating strong domestic wastewater at 0.22 m/hr V_{up} in Amman, Jordan. The reason could be attributed to the low V_{up} applied for our reactors. Hence, Seghezzo (2004) found that VSS increased steadily in the UASB when a low V_{up} was applied for a relatively long time, probably due to the accumulation of undegraded VSS originated in poor sludge expansion at that low V_{up}. Moreover, the smaller VSS/TSS fraction found at Amman can probably be attributed to a higher inert material fraction in the raw wastewater.

In this study, it should be mentioned here that the sludge volume index (SVI) was not recorded any value in the effluents from the both UASB-septic tank reactors (Table 4.4). This interesting result indicates that the UASB-septic tank reactor is very effective for removing the suspended solids from the domestic sewage. Moreover, the relatively low SVI of the influent to the UASB-septic tank reactors reveals high settleability.

4.3.9 Nutrients removal efficiency

Nitrogen removal. Figures 4.19 and Table 4.4 depict the variation of NH_4^+-N concentrations and average removal efficiencies of the two UASB-septic tank reactors during the study period. The results show that the difference of NH_4^+-N concentrations between influent and effluents in the two reactors was very low and within the marginal error of the used measuring instrument. Nevertheless, the average NH_4^+-N concentrations, before and after the UASB-septic tanks treatment, decreased from 58.9 (3.8) to 56 (3.5) mg/L in R1 with 5% (6) removal efficiency, while slightly increased from 58.9 (3.8) to 59 (4.4) mg/L in R2, however, the difference in removal efficiency between R1 and R2 is statistically significant ($\rho<0.05$). In any case, the increase of NH_4^+-N concentrations was obviously detected in both reactors especially at the startup period (Fig. 4.19). The likely mechanism for such increases may be the mineralization of organic compounds containing organic nitrogen or as a result of protein hydrolysis.

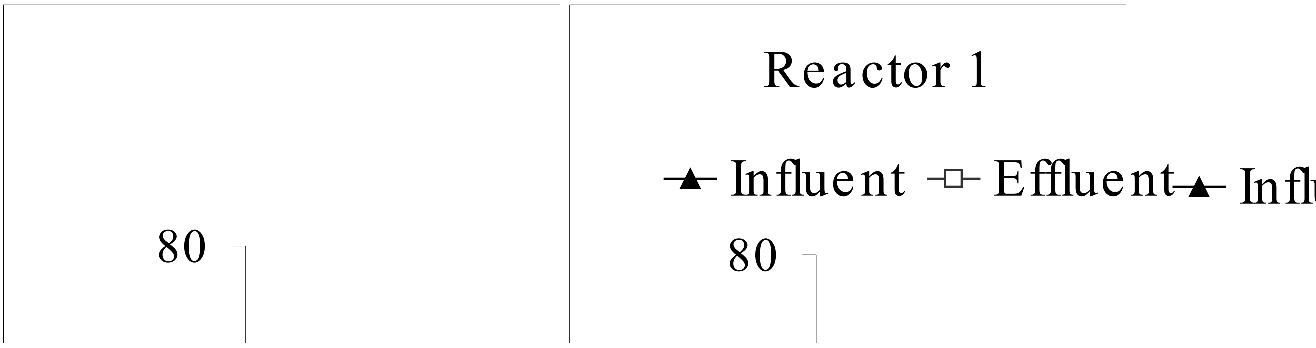

Figure 4.19. The evolution of NH_4^+-N concentrations for R1 (left) and R2 (right) along the study period

The NKj-N was partly removed in the UASB-septic tank reactors due to particulate N removal (Table 4.4). The average removal efficiency of NKj-N was 16% (9) for R1 and 12% (10) for R2. As shown, R1 achieved a slightly higher NKj-N removal efficiency than R2, however, it is not statistically significant ($\rho > 0.05$). Fig. 4.20 shows the evolution of NKj-N and removal efficiency during the study period in the both reactors. The same trends of the TKj-N removals were also reported by Bogte *et al*. (1993) and Mahmoud (2002) when treating domestic wastewater in UASB reactors.

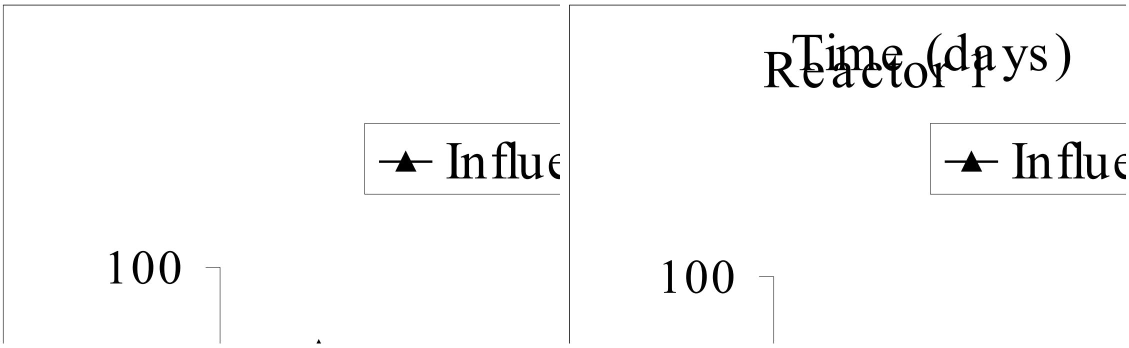

Figure 4.20. NKj-N influent and effluent concentrations and removal efficiencies for R1 (left) and R2 (right)

Phosphorous removal. Total-P removal followed the same trend of NH_4^+-N. Mean concentrations of total-P dropped a little in R1 from 14.0 in the influent to 13.7 mg/L in the effluent with 2% removal efficiency, while a slightly increased from 14.0 to 14.2 mg/L was detected in R2 (Table 4.4). The difference in removal efficiency between R1 and R2 with regard to total-P is statistically significant (ρ<0.05). Figure 4.21 depicts the evolution of total-P concentrations for R1 (left) and R2 (right) along the study period.

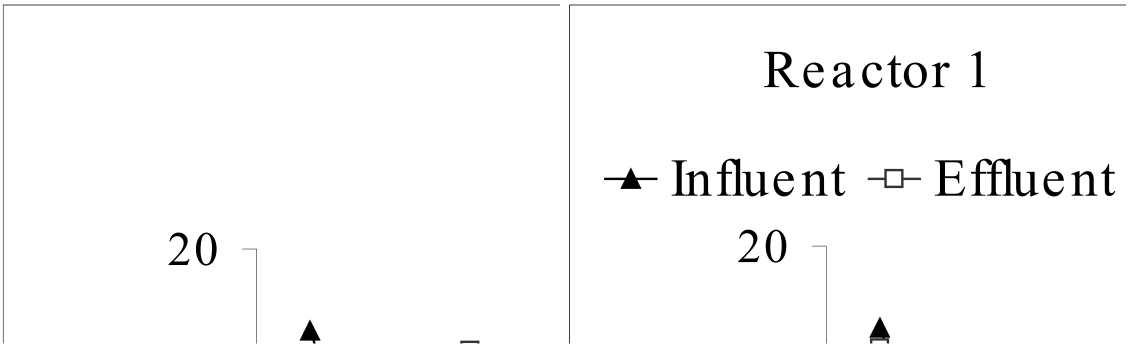

Figure 4.21. The evolution of total phosphorous concentrations for R1 (left) and R2 (right) along the study period

Table 4.4 and Fig. 4.22 show the evolution of ortho-phosphate (PO_4^{3-}) concentrations along the period of study. The results reveal that the influent ortho-phosphate (PO_4^{3-}) concentrations were always increased through the treatment in the UASB-septic tank reactors. The average concentrations of ortho-phosphate increased from 12.6 mg/L in the influent to 16.0 and 16.7 mg/L respectively in the effluents of R1 and R2. The difference in ortho-phosphate concentrations between R1 and R2 is statistically significant (ρ<0.05).

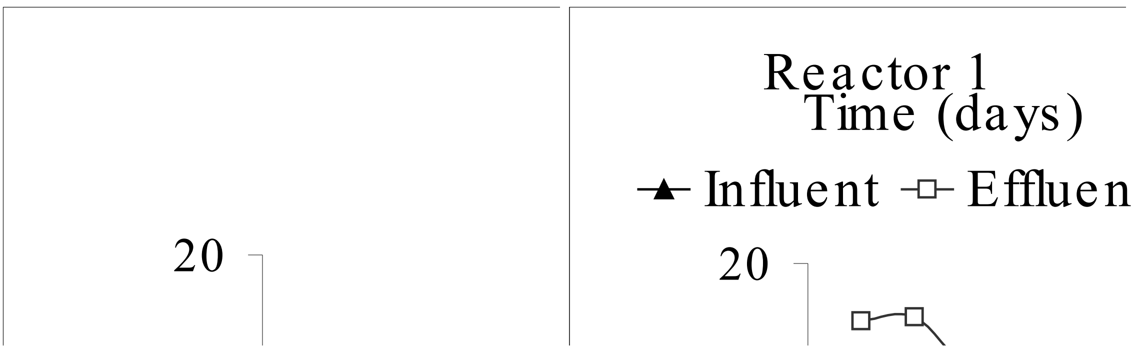

Figure 4.22. The evolution of ortho-phosphate (PO_4^{3-}) concentrations for R1 (left) and R2 (right) along the study period

Sedimentation and further degradation of the particulate organic material 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-phosphate. These mechanisms are however most likely to take place in UASB reactors. As pointed out by Haandel and Lettinga (1994), results from determinations of phosphorus concentrations before and after UASB treatment showed that orthophosphate increased from 5.5 to 9.9 mg/L.

These results suggest that the UASB-septic tank reactor as primary anaerobic treatment of sewage does not effectively remove nutrients. Moreover, the results clearly show a change in the chemical forms of nitrogen and phosphorous present in the UASB liquid rather than an effective removal of it. As pointed out by Bogte *et al*. (1993), results from determinations of nitrogen and phosphorus concentrations before and after on-site UASB-septic tank treatment showed that PO_4 total increased from 18.7 to 19.6 mg/L and orthophosphate from 13.7 to 14.5 mg/L. According to Haandel and Lettinga (1994), in UASB reactor organic nitrogen is hydrolysed to ammonia by hydrolytic bacteria resulted in an increase in ammonia concentration from 35 to 53 mg N/L. Therefore, nutrient removal can only be achieved in a separate post-treatment step.

4.3.10 Sulphate removal efficiency

The mean values of the effluent sulphate (SO_4^{2-}) concentration and the calculated removal efficiencies of the two UASB-septic tank reactors are depicted in Table 4.4. The average removal efficiencies were 72% (4) and 71% (5) for R1 and R2, respectively. No significant differences were found for SO_4^{2-} removal efficiency between the two reactors ($\rho>0.05$). Fig. 4.23 shows the evolution of SO_4^{2-} concentrations and removal efficiencies for R1 and R2. These interesting SO_4^{2-} removals obtained in this study may be due to S accumulation in the sludge, organic matter oxidization and evaporation of H_2S which is formed mainly by the anaerobic reduction of sulphate by sulphate-reducing bacteria (SRB) such as *Desulfovibrio*.

The results in Table 4.4 and Fig. 4.23 reveal that the effluent qualities for R1 and R2 in terms of SO_4^{2-} were stable throughout the experiment, maintaining a rather constant effluent concentration and seemed to be not significantly affected by the fluctuation of influent concentration.

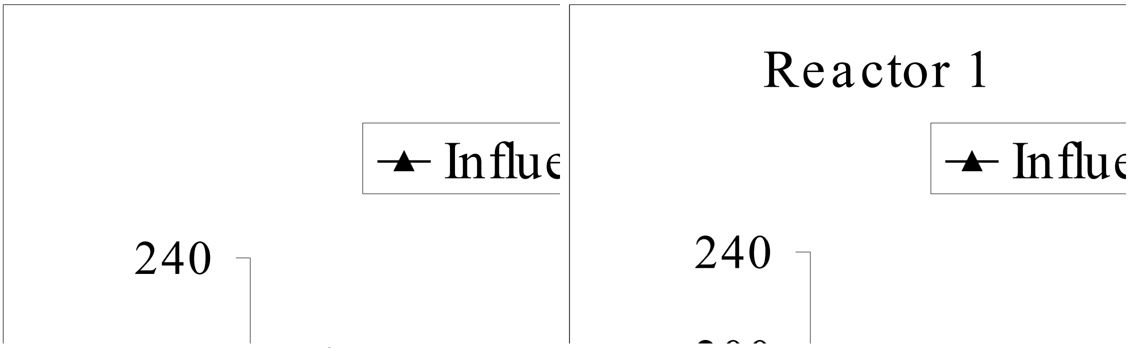

Figure 4.23. Sulphate (SO_4^{2-}) influent and effluent concentrations and removal efficiencies for R1 (left) and R2 (right)

Regarding sulphate removal in UASB reactors, there are few data available in the literature. Nevertheless, a more comprehensive research carried out by Visser (1995) showed that the occurrence of sulphate reduction processes in anaerobic wastewater treatment systems has advantages and disadvantages. The main disadvantages are:

- Since a fraction of the organic compounds in the wastewater is used for the reduction of sulphate, this results in a lower methane yield and therefore negatively affects the overall energy balance of the process.

- Sulphide is an inhibiting compound for anaerobic bacteria, including methanogenic, acetogenic and even sulphate reducing bacteria (SRB). Sulphide accumulation may cause a severe inhibition of the treatment process resulting in its total failure.

- The produced sulphide has a bad smell and cause corrosion problems to pipes and engines. Thus, extra investment costs are necessary to avoid these problems.

On the other hand, Visser (1995) pointed out the following advantages of this process:

- Heavy metals present in wastewaters can be removed by the formation and precipitation of metalsulphides. This will also reduce potential toxicity problems to the anaerobic digestion process.

- In wastewaters containing sulphites, the reduction of this very toxic compound to the less toxic sulphide will increase the potential of anaerobic treatment implementation.

4.3.11 pH in the UASB-septic tank reactors

pH is the most important process control parameter in anaerobic reactors (Droste, 1997). According to Zehnder *et al.* (1982), the optimum pH range for all methanogenic bacteria is between 6.0 and 8.0, but the optimum value for the group as a whole is close to 7.0. In this study, the raw wastewater had pH values around 7.5 (0.22). Mean pH values around neutrality were detected in R1 and R2. The pH value was normally found around 7.4 (0.14) with a range of 7.12-7.7 in both reactors (Table 4.4). These values demonstrated a stable performance along the study as pH values were kept within an optimum range. Moreover, these favorable environmental conditions allowed a healthy development of the bacterial groups responsible for hydrolysis, acidogenesis and methanogenesis. Likewise, there was no risk of reactor acidification in any of the UASB-septic tank reactors throughout the monitoring period, since effluent pH values were above 7.1 in all cases.

Figure 4.24. The evolution of pH values for R1 (left) and R2 (right) along the study period

The slightly lower pH values recorded in the UASB effluents is expected in the anaerobic treatment of most domestic wastewaters given their buffer capacity (Zehnder *et al.*, 1982; Haandel and Lettinga, 1994). Hence, the buffering capacity found in the raw domestic wastewaters is enough to neutralize the production of volatile acids and carbon dioxide, which dissolves at the operating pressure (Droste, 1997). Fig. 4.24 shows the evolution of pH values in both reactors during the study period.

Chapter 5

Conclusions and Recommendations

5.1 Conclusions

The following conclusions are drawn from the results of the research carried out in this work:

1. Raw sewage from the city of Al-Bireh can be considered as high strength domestic sewage, with a COD_{tot} concentration of 1189 mg/L, and a high percentage of COD_{sus} around 54% (640 mg/L). Moreover, the average BOD_5 and TSS concentrations were 616 and 614 mg/L, respectively.

2. Judging by sewage temperature (24ºC), composition, and anaerobic biodegradability of 65%, it can be concluded that raw sewage in the region are well-suited for anaerobic treatment in UASB-septic tank reactors.

3. The here presented UASB-septic tank reactors were effective for anaerobic sewage (pre) treatment under Palestine conditions. Since, the reactors showed a stable performance during the 6 months of operation, i.e. they provided average removal efficiencies for COD_{tot}, COD_{sus}, BOD_5 and TSS of 54, 85, 56 and 79%, respectively for R1 operated at 2 days HRT. Likewise, 58, 89, 59 and 80% for the same parameters were observed in R2 operated at 4 days HRT.

4. The results obtained in this study showed that the longer HRT, such the case in R2, seems to contribute slightly to better reactor performance. The latter, had a significant effect on the COD_{tot}, COD_{sus}, BOD_5 and TSS removals. The results of statistical tests on the removal efficiency data sets of the previous parameters also confirmed the enhanced performance of R2 ($\rho<0.05$). This suggests that the design HRT = 4 days in UASB-septic tank reactors seems more adequate for the anaerobic treatment of domestic sewage under Palestine conditions.

5. The removals of COD_{col} and COD_{dis} correlated well with increases in temperature and microbial adaptation. The latter showed a gradual enhancement of the COD_{col} and COD_{dis} removal efficiencies since the beginning of the experiment. The average COD_{col} and COD_{dis} removals during the whole period of study were respectively 27 and 12% for R1; and 32 and 14% for R2.

6. The final effluents from both reactors contained a high amount of soluble COD about 60% (308 mg/L) of COD_{tot}, of which 52% (160 mg/L) was in the form of VFA. This suggests that the effluents of the tested UASB-septic tank reactors can be easily post treated.

7. According to the results, it can be concluded that the UASB reactor treating domestic wastewater can be started with a poor quality anaerobic seed, such as sludge from cesspits or septic tanks. Moreover, a minimum mount of seed sludge equals to 10% of reactor volume, such the case in R1, is adequate to start-up and operate properly a new reactor.

8. The total anaerobic biodegradability of the effluent sewage from the UASB-septic tank reactors was 42% for R1 and 39% for R2. Effluent sewage is likely to be less biodegradable than raw sewage (65%) due to its lower amount of highly biodegradable suspended solids.

9. The evolution of biogas production varying and strongly affected by temperature and ecology of the UASB-septic tank reactors. The average total methane production (gas form + liquid form) from both reactors was 0.1 $Nm^3/kgCOD$ removed.

10. The scum layer forming was affected by the digestion process, consequently by temperature and biogas production as well. Moreover, it was affected to some extent by the illegal industrial discharges that reach the reactors. However, it was disappeared after few days of its formation.

11. Hydrolysis is the rate-limiting step of the overall digestion process in R1, while methanogenesis and/or hydrolysis are the rate-limiting steps in R2.

12. The results suggest that the UASB-septic tank reactor as primary anaerobic treatment of sewage does not effectively remove nutrients. Moreover, the results clearly show a change in the chemical forms of nitrogen and phosphorous present in the UASB liquid rather than an effective removal of it.

13. The operation of the on-site two UASB-septic tank reactors at Al-Bireh WWTP was developed successfully and the methodology applied confirmed its advantages by reducing the costs associated with periodic desludging, since the sludge retained in the reactors was not exceeded the 0.4 m of the 2.5 m reactor height during the 6 months of operation. However, the sludge concentrations were increased with average values of 46.8 gTS/L and 48.6 gTS/L respectively for R1 and R2 during the whole period indicating the sludge accumulation, as compared to the first operational period (13.78 gTS/L). Therefore, the sludge withdrawal from the reactors is deemed to be after long time of operation.

14. The VS/TS ratios of the retained sludge in both reactors showed a decline trend since the beginning of the experiment indicating more stable sludge. Nevertheless, the retained sludge was not fully stabilized with VS/TS ratio in the range of 69-74% depending on the temperature, likewise, stability values in the range of 52-62%. The sludge retained in R2 has higher stability than the sludge retained in R1.

15. Finally, as a general conclusion, it could be said that the one-step UASB-septic tank reactors configuration is a potential compact and effective community onsite pre-treatment unit for domestic wastewater. The system is more economical and affordable for local relatively poor communities since it can operate successfully without high expertise and does not require any external supply of energy particularly when gravity flow mode can be achieved.

5.2 Recommendations

1. On the basis of the results presented in this research and concerning the reactors performance, the design HRT = 4 days in UASB-septic tank reactors is recommended for the anaerobic treatment of domestic sewage under Palestine conditions.

2. The application of decentralized "community onsite and/or one house or cluster onsite" in Palestine is recommended for the following major reasons: (a) enabling the urban agricultural reuse of treated effluent as the majority of the agricultural land in Palestine is scattered as small agricultural lots; (b) reducing the sewerage work cost and consequently proper environmental protection.

3. A post-treatment step is recommended in most cases after UASB-septic tank systems, not only to remove remnant COD, but also to remove nitrogen and phosphorus (when reuse is not possible), and fecal coliforms, the most commonly used indicator of pathogenic microorganisms.

Recommendations for further research:

- The pilot plants researched herein should continue to be operated for at least another six months including the winter period of the year in order to establish the stability of the system performance. Moreover, this would provide a more concrete conclusion about the proposed HRTs for design purposes of the UASB-septic tank system, in attempt to establish maximum point of operation for the system under Palestine conditions.

- Since the UASB-septic tank reactors would take a long time to be filled with sludge, it is important to analysis and modelling the information gathered in the research mentioned above. This will allow knowing the time needed to achieve the maximum sludge build-up or periodic sludge withdrawals. Moreover, this will help to establish reliable criteria for the UASB-septic tank design, treating domestic wastewater under the prevailing conditions of Palestine.

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Appendixes

Appendix 1. Photos of the Experimental Set-ups

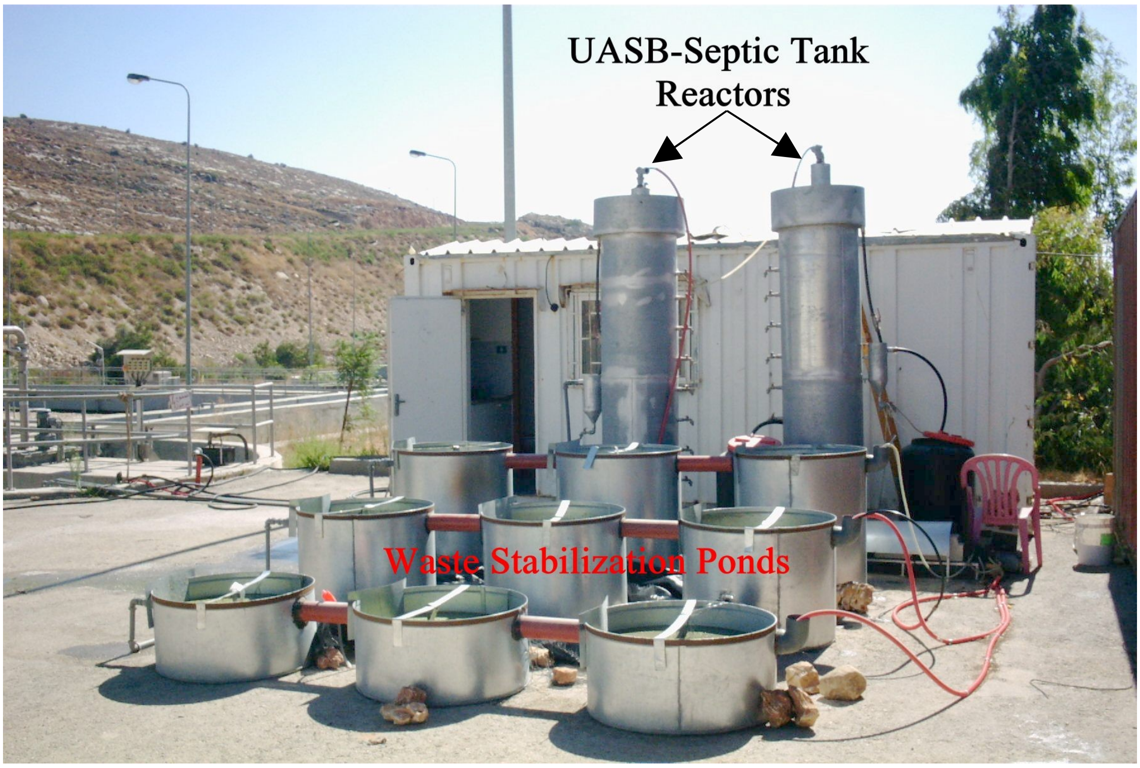
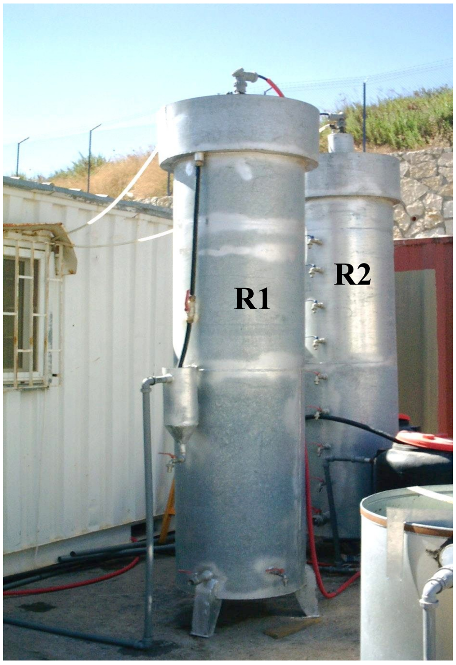


Photo 1. The UASB-septic tank pilot plants with waste stabilization ponds at Al-Bireh WWTP-Palestine, as an integrated treatment of raw sewage



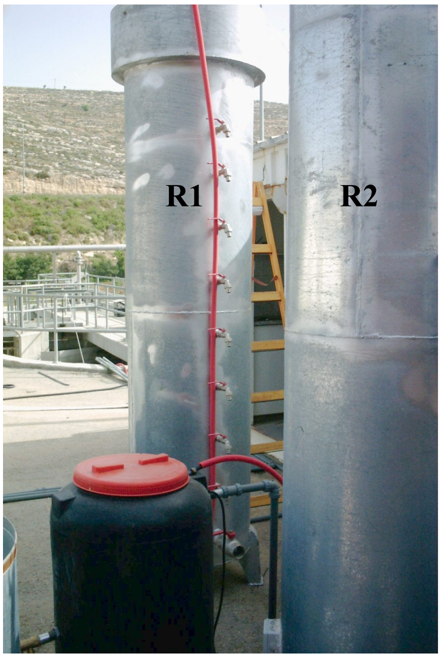


Photo 2. Side view of the UASB-septic tank reactors (R1&R2) and the holding tank from which the reactors were fed

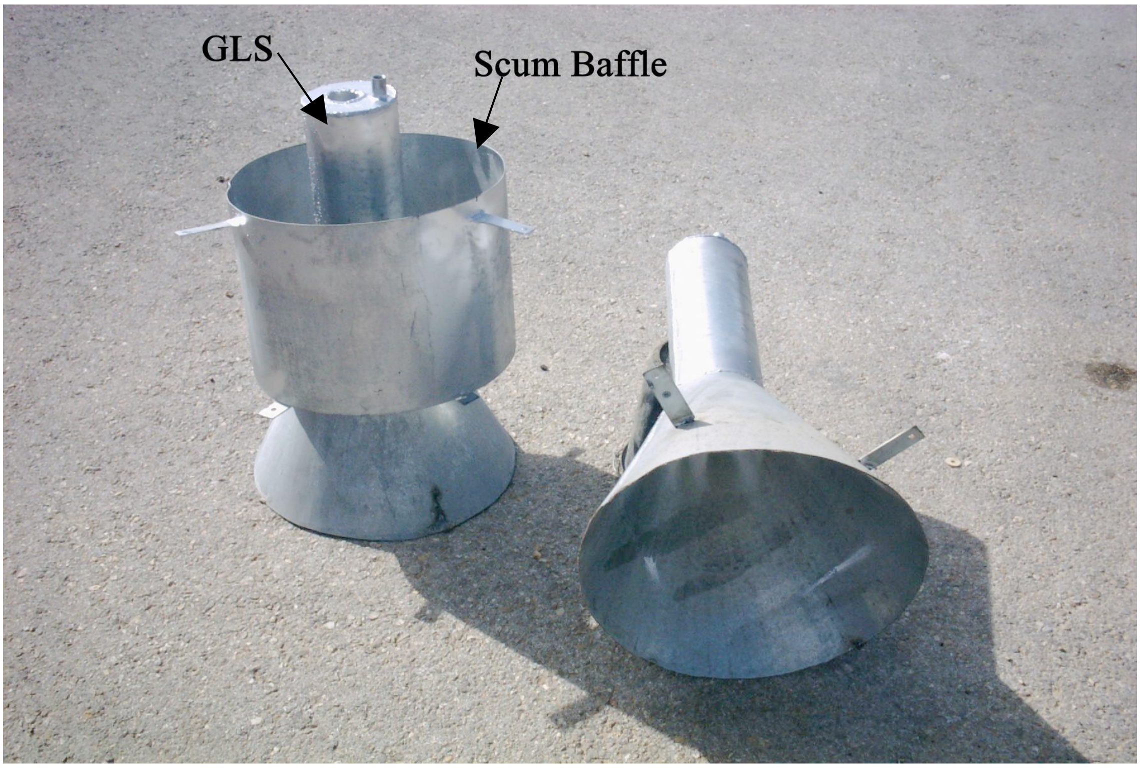


Photo 3. Configurations of the gas-liquid-solids phase separators (GLS) and scum baffles used inside the UASB-septic tank reactors


Photo 4. Top view of the pilot scale UASB-septic tank reactor before installation, showing the arrangement of the GLS and the scum baffle inside the reactors

Photo 5. The holding tank that was used for feeding the two pilot UASB-septic tank reactors. The photo also showing the overflow and the recirculation pipes of the holding tank.

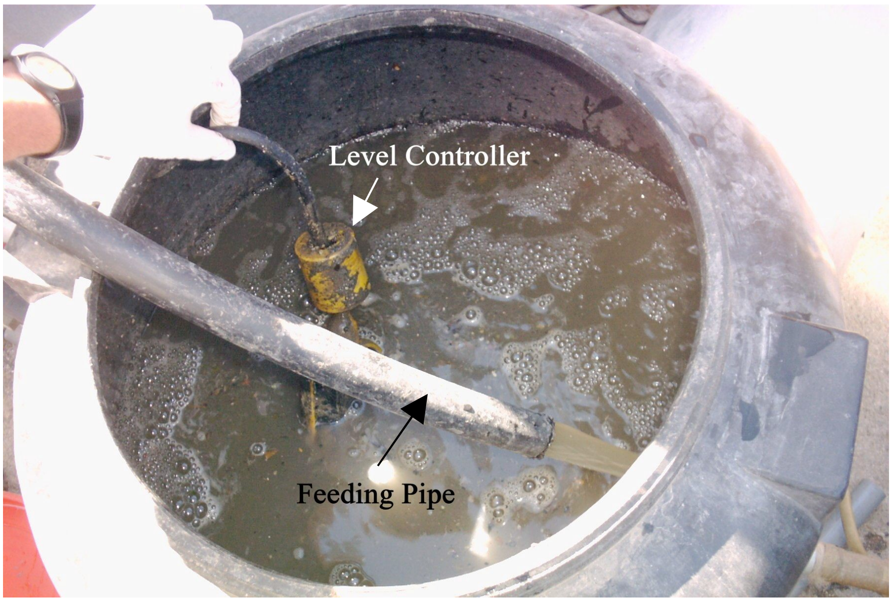


Photo 6. The photo shows the level controller device that was used to control the wastewater level inside the holding tank. In addition to the raw sewage feeding pipe that comes from the grit removal chamber at Al-Bireh WWTP

Photo 7. The submersible pump that was used to feed the holding tank with raw sewage from the grit removal chamber at Al-Bireh WWTP. The submersible pump was placed inside plastic basket to protect it from clogging with course materials.

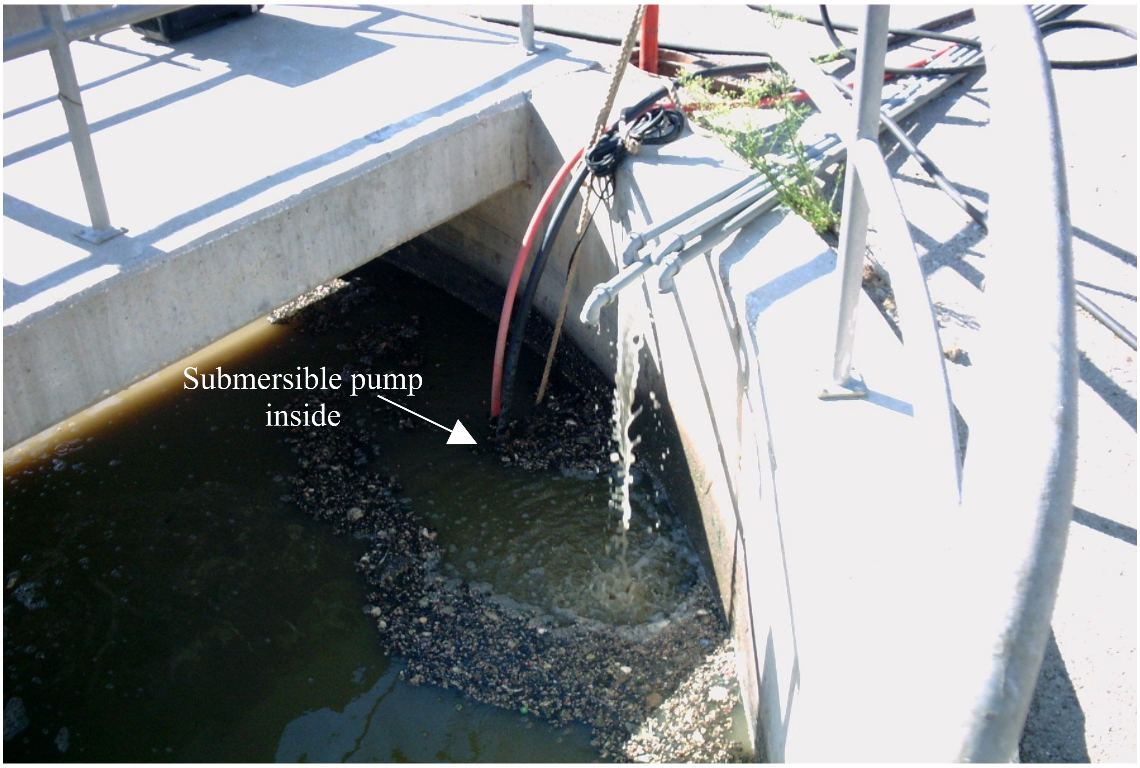


Photo 8. The grit removal chamber at Al-Bireh WWTP that from which the raw sewage was fed to the pilot plants. The photo also shows the pipes used to return sewage from the holding tank and the effluents of the pilot plants

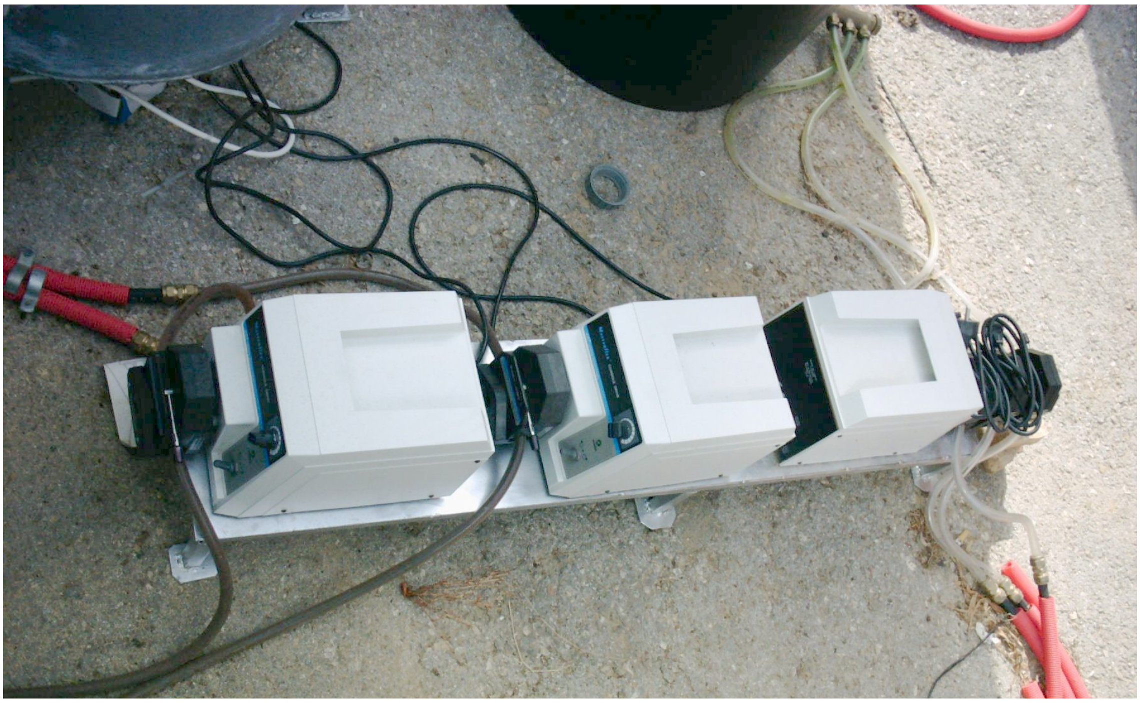


Photo 9. The Masterflex peristaltic pumps that were used to feed the pilot plants with raw sewage from the holding tank.

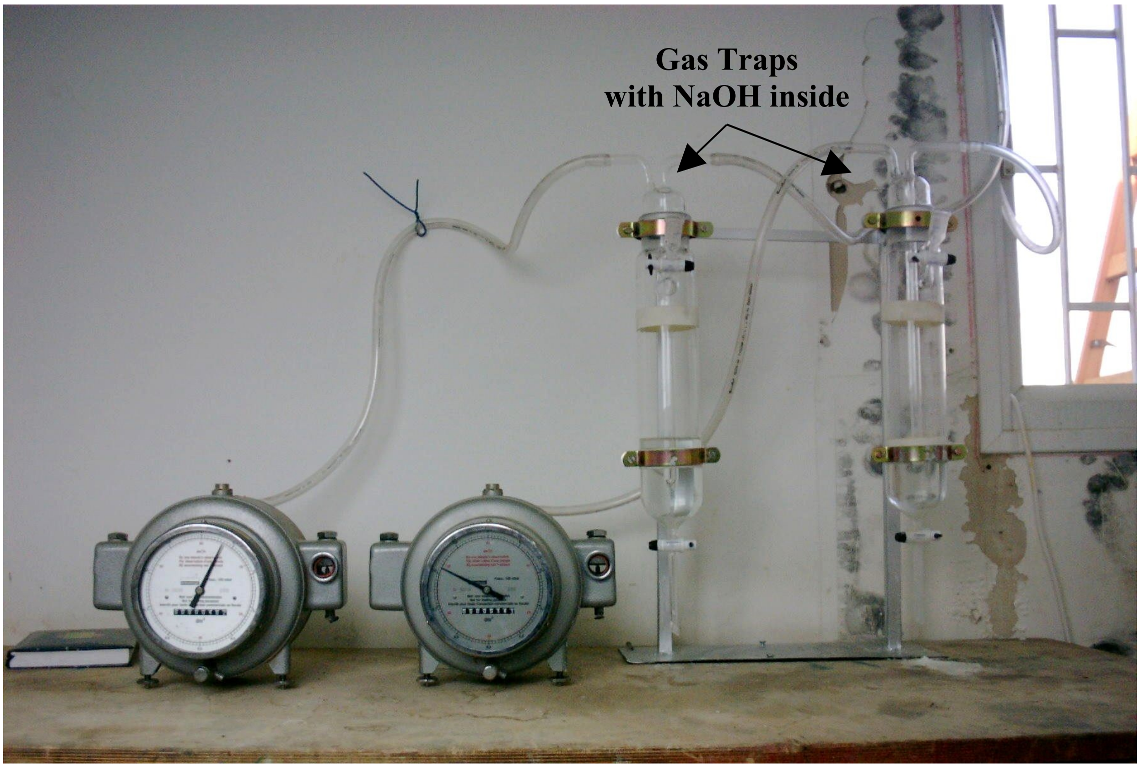


Photo 10. The gas meters (left) and the gas traps with 16% NaOH inside (right) that used to measure the methane gas production from the pilot scale UASB-septic tank reactors

Photo 11. The scum layer phenomena inside the scum baffle in R1 (left) and R2 (right) at day 85 of reactors operation

Photo 12. The liquid displacement set-up that used for methane gas measurements from the stability bottles inside the incubator.

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 were also assured by syringing of sodium sulfide solution through the septum of each bottle. The bottles then incubated at 30ºC 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.

Biodegradability (%) = 100 (COD_{CH4}/ $COD_{tot,\ t=0\ days}$) or
Biodegradability (%) = 100 ($COD_{tot,\ t=0\ days}$ – $COD_{tot,\ t=t\ days}$)/ $COD_{tot,\ t=0\ days}$
where:
COD_{CH4}: amount of produced CH_4 $_{(liquid\ form\ +\ gas\ form)}$ (mg CH_4 as COD/l);
COD_{tot}: amount of total COD in the tested sample (mg COD/l).

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 30ºC for a period of 100 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 ml flask and stirred using a magnetic bar:
- 20 ml macro nutrients stock solution, as prepared below in Table A2.1.
- 10 ml micro nutrients (trace elements), as prepared below in Table A2.2.
- 25g $NaHCO_3$ (buffer solution).
- 0.5 gm yeast extract.
- Demineralized water: fill up the flask to1000 ml mark.

Sodium Sulphid (Na_2S) solution preparation

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

Table A2.1. Macronutrients stock solution

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

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

Table A2.2 Micronutrients (Trace elements) stock solution

Chemical Substance	**Concentration in 500 ml Serum Bottle (mg/l)**	**Weight to be added to 1000 ml flask as stock solution* (mg)**
$FeCl_2.4H_2O$	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.

Appendix 3. Sample calculations of the dissolved CH_4 gas as COD

The dissolved CH_4 as COD (liquid form) is calculated according to as Henry's law:

$$[CH_{4(dis.)}] = K * P_{(CH4)} \tag{1}$$

where:

$[CH_{4(dis.)}]$: concentration of CH_4 in the liquid form (mol/L).

K: Henry's coefficient for methane (mol/L.atm) = $1.34*10^{-3}$ mol/L.atm at T = 25 ^{O}C.

$P_{(CH4)}$: partial pressure of gas (atm); $P_{(CH4)}$ = 0.7 atm (assumed).

From equation 1, $[CH_{4(dis.)}] = 9.086*10^{-4}$ mol/L

Since, 1 mol CH_4 = $64*10^3$ mg COD; CH_4 dissolved as COD = 58 mg CH_4 as COD/L

Arabic Summary

حول أداء المفاعلين أنهم في حالة أداء ثابتة و مستقرة طوال فترة الدراسة، حيث كانت معدلات ازالة الملوثات من المياه العادمة على النحو التالي: 54، 85، 56، 79% لكل من COD_{tot}، COD_{sus}، BOD_5، TSS على التوالي في المفاعل (R1) و كانت 58، 89، 59، 80% لنفس العوامل في المفاعل (R2). النتائج تشير أيضًا إلى أن(R2) حقق نتائج أفضل في ازالة الملوثات من (R1). لقد كان لزمن المكوث الهيدروليكي الأطول والمفروض على R2 الأثر الواضح في تحقيق كفاءة ازالة أفضل للملوثات COD_{tot}، COD_{sus}، BOD_5، TSS كما أكدت أيضًا التحاليل الاحصائية، لهذة المجموعة من الملوثات، الأداء الأفضل للمفاعل R2 حيث كانت النتائج دالة احصائيا ($\rho > 0.05$)، لذلك و بناء على نتائج البحث، فإن زمن المكوث الهيدروليكي (HRT) المساوي لأربعة أيام ينصح به لتشغيل نظام UASB-septic tank لمعالجة المياه العادمة المنزلية في فلسطين. كما و بينت النتائج أن ازالة المواد العالقة و المذابة من COD مرتبطة بشكل جيد بزيادة درجة الحرارة و زيادة تكّيف البكتيريا داخل المفاعل، حيث بلغت نسب ازالة COD العالق (COD_{col}) 27 و 32% لكل من R1 و R2 على التوالي، و كذلك نسب ازالة COD المذاب (COD_{dis}) كانت 12 و 14% لكل من R1 و R2 على التوالي. كما و أوضحت النتائج أن تطور انتاج غاز الميثان من المفاعلين كان متغير و يعتمد بشكل قوي على درجة حرارة الجو و الوضع البكتيري في كل مفاعل، حيث كان معدل انتاج الميثان الكلي لكل من المفاعلين طوال مدة الدراسة و تحت الظروف المعيارية 0.1م^3/ كجم COD تم ازالته. و من خلال ملاحظتنا لنمو الحمأة "sludge" داخل كل مفاعل، لقد تبين أن الحمأة لم تزداد حجما خلال 6 أشهر من التشغيل و انما ازدادت تركيزاً حيث ازداد معدل تركيز المواد الصلبة الكلي للحمأة من 13.78 غم/ لتر في بداية التشغيل الي 46.8 غم /لترفي R1 و 48.6 غم/ لتر في R2 بعد 6 أشهر مما يشير الي تراكم في تركيز الحمأة، لذلك تفريغ المفاعلين من الحمأة قد يكون بعد فترة طويلة من التشغيل. أخيراً يمكن القول بأن نظام المعالجة اللاهوائي -UASB septic tank نظاما محكماً و فعالاً لمعالجة المياه العادمة المنزلية بشكل أولي موقعيا لمنطقة بأكملها و تحت الظروف البئية السائدة في فلسطين.

ملخص

تعتبر الحفر الامتصاصية و خزانات جمع القاذورات"septic tanks" الأكثرمعرفة و شيوعًا للطرق الموقعية المطبقة لتصريف المياه العادمة و معالجتها بشكل أولي. إلا أن الوضع الحالي الممارَس لهذا النمط من وحدات المعالجة الأولية اللاهوائية باستطاعته أن يؤدي الي أخطار ذات أهمية على الصحة العامة و البيئة. من أجل ذلك، التدخل الصحي لتصريف المياه العادمة بات أمرا ضروريا، و بناءً عليه فإن الحاجة الي تطبيقات فعّالة لمعالجة المياه العادمة و لكن بتكاليف قليلة للمناطق الفلسطينية، و خاصة الريفية منها، بدى مما لا شك فيه شيء عظيم و ضروري. بناءً على المعلومات التقنية المتوفرة و التي تهتم بأداء نظام المعالجة اللاهوائي "UASB-septic tank" لقد تبين أن هذا النظام يمثل بديل ذو كفاءة جيدة و قليل التكاليف لتصريف و معالجة المياه العادمة بشكل صحي للبيت المنزلي، إلا أن أداء هذا النظام لم يتم بحثه لحتى الآن لمعالجة المياه العادمة القادمة من حي أو منطقة بأكملها موقعيا و خاصةً في فلسطين، آخذًا بعين الاعتبار الظروف

السائدة بالمنطقة من حيث التراكيز العالية للمواد العضوية الملوثة في المياه العادمة المنزلية و التي تحتوي أيضا على نسب عالية من المواد الصلبة، والتغيرات الموسمية لدرجات الحرارة.
بالإضافة لما تم ذكره، إن المعايير التصميمية لنظام UASB-septic tank ما زالت تحت التطوير في فلسطين.

لقد كان الهدف الرئيسي لهذه الرسالة هو بحث مدى أداء و جدوى استخدام المفاعل اللاهوائي UASB-septic tank في معالجة المياه العادمة المنزلية والقادمة من حي أو منطقة بأكملها و تحت الظروف السائدة في فلسطين، و تقييم تأثير زمن مكوث المياه العادمة (HRT) داخل المفاعل على أداء هذا المفاعل، كمحاولة لتحسين تصميمم هذا النظام. لهذا الغرض لقد تم بناء مفاعلين ((R1 و(R2) لمعالجة المياه العادمة المنزلية في المحطة الرئيسية لمعالجة المياة العادمة الخاصة بمدينة البيرة. لقد تم تشغيل المفاعلين بصورة متوازية و تحت زمن مكوث مختلفين (يومين للمفاعل R1، و أربعة أيام للمفاعل R2) لمدة ستة أشهر و في درجات حرارة محيطة تتراوح ما بين 15 و 34 درجة مئوية و بمعدل = 24.2^{o} م. أما درجة حرارة المياه العادمة خلال فترة الدراسة كانت تتراوح ما بين 18.2 و 29 درجة مئوية و بمعدل =24^{o} م.

لقد بينت نتائج الدراسة أن المياة العادمة القادمة من منطقة الدراسة تتميز بتركيز عالي من الأكسجين الكلي المستهلك كميائيا (COD_{tot}) لقد كان بمعدل 1189 ملغم/لتر و يحتوي أيضا على نسبة عالية من المواد العالقة (COD_{sus}) حوالي 54% أي ما يقارب 640 ملغم/لتر. كما و أشارت النتائج أن المياه العادمة القادمة و الغير معالجة قابلة للتحلل اللاهوائي بنسبة 65% و كانت النسبة ما بين COD و BOD_5 تساوي 2 أي أنها قابلة أيضًا للتحلل هوائيًا. لقد تم تشغيل المفاعلين بطريقة ناجحة و قد بينت النتائج التي تم الحصول عليها في هذا البحث