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# **Birzeit University**

# Enhanced Pretreatment of Black Wastewater from Birzeit University Using UASB Technology

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

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### Abstract

Recent implementation of the UASB pilot plant scale and full scale have produced very encouraging results which show its feasibility as a pre-treatment for black and domestic wastewaters. The economic advantages it offers, further encourages developing countries to become self-sufficient in this respect. In this research, the start-up of the UASB-septic tank system and its monitoring during this period was the main objective. Although the system functioned as batch during the beginning and only briefly was operated as a continuous reactor, the results obtained demonstrated that these systems are quite effective in removing organic pollutants. Thus the removal of the organic pollutants was mainly due to bio-physical process, which is sedimentation of the particles and its microbial degradation. Although, the microbial activity was yet slow it contributed to the total COD removal efficiency. The COD<sub>total</sub> removal attained was approximately 76% in which the removal of COD<sub>ss</sub>, COD<sub>col</sub>, and COD<sub>dis</sub> was 86%, 23% and 52% respectively.

The UASB effluent has a good transparency, thus this will facilitate the successive applied treatment since the photosynthesis and pH increase will instigate carbon dioxide consumption, accelerating the death rate of the pathogens and opening the possibility of reducing nutrients: Nitrogen by desorption of gaseous NH<sub>3</sub> and phosphorous by phosphate precipitation.

The practical advantages the UASB-septic tank system offers, such as its relatively small size, the capture and use of the biogas, and elimination of odour problems make this system neighbour friendly. Furthermore, its ease of operation encourages not only decentralized use but can also be applied in densely populated areas.

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### List of Abbreviations

A: Acidification AF: Anaerobic Filter AVR: Average BOD: Biological Oxygen Demand **BZU:** Birzeit University C: Colloidal Cm: Centimeter COD: Chemical Oxygen Demand COD<sub>col</sub>: Colloidal COD  $\text{COD}_{\text{dis}}$  : dissoved COD COD<sub>f</sub>: Filtrate COD COD<sub>ss</sub>: suspended COD CODt: total COD CSTR: Completely Stirred tank reactor d: day D: dissolved Eff: Effluent EGSB: Expanded Granular Sludge Bed F: Filtrate GLS: Gas Liquid Solids Separator g : gram GSS: Gas Solid Separator h: hour H: Hydrolysis HRT: Hydraulic Retention Time Inf: Influent L: Litre LCFA: Long Chain Fatty Acid M: Methanogenesis Max: Maximum

mg: milligram MIN: Minimum ml: milliliter N: Nitrogen °C: Degree Celsius PM: Packing Material S: Suspended SRT: Solid retention Time SS: Suspended Solids STDEV: Standard Deviation SVI: Sludge Volume Index T: Total TKN: Kjeldal Nitrogen TSS : Total Suspended Solid Twth: Temperature of weather (ambient) UASB: Upflow Anaerobic Sludge Blanket VFA: Volatile Fatty Acid Vol: Volume Vup: up flow velocity WW: Wastewater

### Chapter 1 Introduction

#### 1.1 Background

Palestine, like many countries in the Middle East, suffers from an extremely limited water supply, rapidly increasing population and limited financial resources. The country has made great efforts in provision of water and wastewater services for it's population in the last five years.

The increasing urbanization and industrialization in Palestine have dramatically increased the amount of generated wastewater. With the very limited existing disposal systems throughout the country, Palestine is now threatened by this waste which my infiltrate and pollute its underground water resources unless measures are taken to alleviate this dilemma. Thus the new treatment process that offers a substantial promise may be the anaerobic treatment (Al-Sa'ed *et al.*, 2001).

### 1.2 Urban Sanitation in Palestine

In urban areas in Palestine; about 20% of the total population are served with a centralized sewerage systems, while only a 5% of the collected municipal wastewater is subjected to partial treatment. Lack of effective planning in water and infrastructure, and exploding urbanization in rural areas in Palestine are polluting groundwater and degrading of both water quality and quantity, (Al-Sa'ed *et al.*, 2001), even this impact on environment is small when it is compared with the disposal of pollutants and extensive pumping on ground water by "Israel".

Improved water supply and sanitation services are needed mainly in ruler areas, but the high cost of the infrastructure, scarcity of development funds are major constraints.

The collection and treatment of wastewater in Palestine illustrates problems and constrains that associated with an area under rapid development while facing conditions of water scarcity, not only in Palestine but also in all the Mediterranean

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and Middle East countries. This increases year by year because of the extensive use of water for human consumption, industrial and agricultural production. Low cost sanitation and water reuse is important as a mean to improve water resources against growing demands of fresh water.

Wastewater management in Palestine has been a neglected issue. No comprehensive data on wastewater characteristics and amounts discharged are yet available. The existing treatment plants have been designed upon assumption of characteristics of wastewater and flows. They are poorly operated and maintained and the effluent is not better than influent in most of the cases. Raw or partially treated wastewater is discharged into the wadis where it is used for irrigation purposes (MOPIC, 1998).

Urban wastewater has the characteristics of domestic wastewater since small scale industries are prevailing, but some factories are disposing of toxic materials. Laws and regulations are difficult to enforce. Without permits, illegal connections to the sewer system are made, since municipalities do not have any executive power.

### 1.3 Existing Sewerage Systems

In the areas that are not connected to the sewerage network, wastewater is discharged into percolating pits and, to lesser extent, into septic tanks. The septic tanks are emptied by vacuum trucks and disposed of either in the treatment plant or just in the wadis. Rainwater drains do not exist. Table 1.1 gives an estimate of the urban wastewater production from the Palestinian Central Bureau of Statistics (2000).

In villages, no sewage network exists, and wastewater is discharged into percolating pits. Pit latrines also exist in some rural areas.

City	Population	%Connected to	Water	Wastewater
		the sewerage	Consumption	Production
		system	litre/capita.day	m <sup>3</sup> /d
Jenin	23800	60	40	570
Tulkarem	35145	65	60	1370
Qalqilya	28280	50	90	1270
Nablus	102460	70	110	7890
Ramallah	20560	70	80	1150
Al-Bireh	33540	60	80	1610
Bethlehem	21700	85	100	1845
Hebron	94760	55	35	1825

Table 1.1: Urban Wastewater Production and Sanitary status in the Major Cities

### **1.4 Existing Treatment Facilities**

### **1.4.1 Publicaly Owned Treatment Plants**

The Jenin treatment plant has 3 algal ponds that were heavily overloaded and never desludged. In Tulkarem, a nearby Israeli settlement uses the effluent in irrigation cotton crops. There are plans to rehabilitate the exsiting treatment plant of Ramallah.

In the rest of the cities of the west Bank, where sewerage systems exist but no treatment plants are available, wastewater is flowing into the wadis by natural drains. During winter, it mixes with non-perennial rivers, and during summer, it mixes with natural springs flowing in the wadis. In nearly all the locations where raw wastewater is flowing. It is used for direct irrigation.

### 1.4.2 Privately owned treatment plants

There are five privately owned treatment plants. Two are located in the district of Bethlehem, one in Jericho, another in Birzeit University and the other in Abu Dis. The effluent is used for on-site irrigation purposes. A pilot treatment plant has been already constructed in Nablus. The technologies that tested there are trickling filters and extended aeration.

### **1.5 Wasterwater Characteristics**

Water consumption is very low in Palestine due to the lack of adequate and regular supply and high water rates. Wastewater is mainly of domestic origin. But since water consumption is very low, wastewater is concentrated and its strength, in some locations, is comparable to that of industrial wastewater.

Table 1.2 gives the characteristics of wastewater of some cities in the West Bank. The values have been attained through analyses of composite samples. For each city, the sampling has been carried out for an 8 hour period and it has not been related to flow, because flow measurments were nearly an impossible tasks.

	Municipal Urban Wastewater				Municipal		ipal Urban Wastewater	ter	Rural I Wast	Domestic ewater
Parameter	Ramallah	Nablus	Hebron	Al-Bireh	Grey	Black				
BOD <sub>5</sub> <sup>20</sup>	525	11850	1008	522	286	282				
CODt	1390	2115	2886	1044	630	560				
Kj-N	79	120	278	73	17	360				
NH4 <sup>+</sup> -N	51	104	113	27	10	370				
NO <sub>3</sub> <sup>-</sup> N	0.6	1.7	0.3	-	1	-				
$SO_4^{-2}$	132	137	267	-	53	36				
PO <sub>4</sub> -3	13.1	7.5	20	44	16	34				
Cl	350	-	1155	1099	200	-				
TSS	1290	-	1188	554	-	-				

Table 1.2 : Characteristics of raw wastewater from some cities in the West Bank

-All data in mg/l, (Al-Sa'ed et al., 2001)

High values of Chemical Oxygen Demand (COD) and Biological Oxygen Demand (BOD) means that wastewater is highly concentrated with organic matter. Hence, the treatment process might be complicated and need advanced technology to reach an effluent that is safe to be discharged in the wadis or reused in agriculture. Another important parameter is the chloride concentration in wastewater. Since the chloride ions are dissolved in the wastewater, the conventional treatment processes do not

remove chloride. Thus, in case treated wastewater will be used in agriculture, then salt-tolerant crops should be considered.

West Bank and Gaza strip still plaguned by the following constraints: (1) rapid population, (2) lack of international and /or local finance for wastewater collection and treatment, (3) lack of knowledge and expertise in alternative sanitation technologies, (4) lack of awareness, legislation and financial incentives for business and landowner to invest in appropriate on-site sanitation and indiscriminate dumping by tankers of untreated sewage.

Problem with existing sewered sanitation and wastewater treatment include:(1)limited household water consumption resulting in low flows and blockages of sewers, (2) lack of appropriate houshold infrastructure to connect with conventional sewerage systems, (3) technical and management problems with some wastewater treatment plants, as a result of various factors including overloading, high BOD content of sewage and (4) untreated industrial waste.

So, low cost anaerobic treatment technologies such as UASB have shown considerable promise recently as advanced pre-treatment option. The UASB technology is feasible in an urban and rural communities in devoloping world and industrialized countries because of its high organic removal efficiency, simplicity, low cost, low capital and maintenanace costs and low land requirment (Lettinga and Hulshoff Pol, 1991). These systems are capable of attain high levels of wastewater treatment, produce minimal sludge that is, itself, high in N-P and are capable for producing biogas energy that can be recovered and reused.

### 1.6 House on-site treatment of total sewage and night soil in a modified UASB-Septic-system.

Conventional septic tank systems are world wide used for the house on site treatment of domestic sewage. The removal efficiency of these systems is however limited, morever have large volumes are generally applied. As the flow through systems is mainly horizontal, the purification is mainly due to settling of solids. The UASBseptic-tank is a promising alternative for the conventional septic-tank (Bogte et al; 1993 and Lettinga et al; 1993). 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. 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. So UASB-septic-tank is a continuous system with respect to the liquid, but a fedbatch or accumulation system, with respect to the solids. Application of these modified septic tank systems, was studied under Dutch (low) and Indonesian (high) ambient temperatures. Zeeman and Lettinga (1999) recommend the use of modified UASB septic tanks for the improvement of the removal efficiencies especially for low temperature regions or regions with a period of short winter period, like Middle East. Therefore, this study will investigate the performance of UASB septic tank system. The black wastewater discharged from the Faculty of Commerce at Birzeit university will be tested.

### 1.7 Objectives

The objectives of this research can be summarized as follows:

1- Identify the change of physical characteristics of sewage during anaerobic digestion of black wastewater.

2- Identify the optimal design parameters for the startup phase of the UASB septic tank.

3- Study the performance of a UASB-septic tank under Palestinian environmental conditions with an temperature fluctuation from 15 °C to 25 °C.

### **1.8 Approach**

In order to achieve the aforementioned objectives, the following steps were carried out.

1) A comprehensive literature review was conducted with respect to wastewater treatment including specialized reading on UASB anaerobic treatment.

2) Design and building up a modified UASB-septic tank reactor at Birzeit University campus that treat black wastewater from the Faculty of Commerce.

3) Measurement of the COD fractions (namely  $COD_t$ ,  $COD_{ss}$ ,  $COD_{col}$ ,  $COD_{dis}$ ) and the biochemical transformation of the COD (% hydrolyses, % Acidification, % Methanogenesis) in order to assess the efficiency of this system.

4) Determination the degree of sludge stabilization by analyzing and recording the accumulation of the VSS and TSS in the sludge bed.

5) Calculation the specific sludge activity by studying the gas production.

This thesis consist of five chapters, chapter one introduces the subject matter, provide the background information into the aim of the study and explores the scope of the problem through a series of objectives. Chapter two reviews literature on the subject matter and investigate previous studies. Chapter three documents the general approach and methods used in this work. Chapter four addresses and discusses the main results of the reaearch study. Chapter five lights high the main findings of the various aspects of this research and outlines the recommendations.

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### Chapter 2 Literature Review

### 2.1 General

Anaerobic digestion is one of the oldest processes applied for the treatment of domestic sewage (McCarty, 1985). The oldest and simplest process is the septic tank. which combines settling and digestion of settled solids from domestic sewage. The first reported installation for the anaerobic treatment of domestic sewage was developed by Louis Mouras in about 1860 (McCarty, 1985). From then onwards, the application of the anaerobic treatment of domestic sewage was studied by various researchers (Scott-Monrief, 1891 and Cameron, 1895). However, with the development and large scale implementation of aerobic processes, application of anaerobic digestion remained mainly limited to the anaerobic digestion of sludge. At present, domestic sewage is treated mainly aerobically. However, due to the oil crises in the seventies and considerable construction, operation, maintenance costs of the aerobic system, many researches were encouraged to investigate the application of high rate anaerobic systems for the treatment of domestic sewage. Anaerobic systems have been developed in the last three decades mainly for the treatment of high strength wastewater. In these systems, the wastewater passes through the anaerobic biomass, where dissolved substrate then is digested and the particulate matter firstly is captured and next digested.

Anaerobic treatment of raw domestic sewage is however new. Pilot-plant studies in Colombia and Brazil showed about 10 years ago that it can be an alternative achieving removal efficiencies of 65%, 80% and 70% for  $COD_t$ , BOD and TSS respectively (Vieira, 1998).

Anaerobic treatment by itself will usually be insufficient to meet the effluent discharge standards and should therefore be combined with a post treatment. In most cases this post-treatment is an aerobic process organic material at low temperatures (Vieira, 1998).

Since the 80s, an increasing number of anaerobic sewage treatment plants have been installed. To meet the discharge standards, the anaerobic treatment is usually combined with an aerobic post-treatment, e.g. a small facultative pond for further polishing of the effluent organic material at low temperatures (Vieira, 1998).

The applicability of anaerobic treatment for domestic sewage depends strongly on the temperature of the sewage. The activity of the anaerobic bacteria is optimal at 35 °C, if the temperature becomes too cold, very little activity is left. This results in poor treatment performances. At present it has been established that with sewage temperature of above 20 °C, anaerobic sewage treatment is feasible. In the range 12 to 20 °C is also feasible, but more research and development work is necessary to assess the optimal treatment conditions (Vieira, 1998).

As a result, anaerobic sewage treatment is primarily of interest for countries with a tropical or sub tropical climate, which are mostly developing countries. In most developed countries with a more moderate climate anaerobic sewage treatment is considered to be not feasible. However, research is still going on to develop anaerobic systems that can also be used in colder climates (Vieira, 1998).

In comparison with most industrial wastewater, sewage is very dilute. As a result the production of biogas is of minor importance. Collection and treatment of the gas, whether it is utilized or not, is necessary in order to prevent the emission of bad odour to the neighbourhood and to avoid the release of methane into the atmosphere (Vieira, 1998).

Anaerobic systems can well be applied on a small scale. This is of importance for developing countries where there is a need for decentralized sewerage systems, since large-scale centralized treatment is very costly. In this respect a distinction can be made between on-site, community on-site and off-site treatment (Vieira, 1998).

At present and also in the near future, nutrient removal will not be required in developing countries. The priority is to remove pathogens and the organic pollutants. In the long run however, it can be expected that N and P-removal will become necessary. Possibilities of biological nutrient removal with the present

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anaerobic/aerobic systems are not sufficiently assessed. Facultative ponds have shown promising results in N and also P-removal if designed for this purpose (Vieira, 1998). This technology has been combined with the conventional septic tank. The upward mode of operation adopted by this system will not only assist in the physical removal of the suspended solids but will also improve the biological conversion of the dissolved components. Thus the UASB-septic tank system is designed for the accumulation and stabilization of the sludge. Therefore, 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 researched (Zeeman *et al.*, 2000; Lettinga *et al.*, 1993; Bogte *et al.*, 1993).

The entrapped suspended solids within the reactor would require longer time for its hydrolysis, and thus during wintertime the entrapped solids is expected to accumulate and further degraded during summertime. Consequently, the desludging frequency of the system will increase since it is expected to be a function of the operational hydraulic retention time HRT.

### 2.2 Advantages and Disadvantages of anaerobic treatment

Anaerobic digestion for wastewater treatment offers a number of significant benefits over conventional aerobic treatment

Advantages	Disadvantages
Advantages	Disadvantages
1-Efficient in removal of organic material	1-Long start-up period when seed sludge
especially for tropical regions >20-35 $^{\circ}$ C.	is not available, as the growth rate of
	methanogenic microorganisms is low.
2- Low construction cost and small land	2-Low pathogenic removal and not
requirements as generally at temperatures	significant in removal of nutrients (N, P).
>20 <sup>O</sup> C high loading rates can be applied.	
3-Low operation and maintenance costs,	3-Requirement for post treatment to reach
as energy consumption is low and little	the effluent standards, depending on the
equipment is needed.	requirements for effluent standards.
4-Lower sludge production as compared	4-Low removal efficiency of particulate

to aerobic and physical-chemical	organic material at low temperatures.
treatment processes.	
5-Biogas production, which can be used	5-Risk for odor nuisance from the
for energy production.	reduction of sulfate to sulfide.
(F1 ', 11' T + 0000)	

(Elmitawalli T. A., 2000)

### 2.3 Treatment systems for domestic wastewater

### 2.3.1 Conventional septic tank system

A conventional septic tank generally consists of one or two chambers with a horizontal inlet flow at the top of the reactor as shown in Figure 2.1. The horizontal influent flow provides that only a part of the suspended solids will be removed by settling. The settled solids will accumulate at the bottom of the reactor. The retention time of the solids depends on the size of the reactor and moreover on the solid concentration that can be provided in the sludge bed. When the reactor is 'full' with sludge, it should be emptied, apart of a certain fraction that is necessary for inoculation. An important disadvantage of the conventional septic tank system is that no contact between sludge and wastewater is provided, so that no conversion of dissolved components will occur. Moreover the removal of SS is limited to settling.



Figure 2.1: Cross sectional view of a conventional septic tank.

### 2.3.2 UASB-septic tank system

The UASB –septic-tank system is a promising altrnative 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 of dissolved components. The most important difference with the traditional UASB system is that the UASB-septic-tank system is also designed for the accumulation and stabilization of sludge. So, a 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. Bogte *et al.*, (1993) and Lettinga *et al.*, (1993) researched the use of a UASB-septic system for the on-site treatment of black water and total domestic sewage at both Dutch and Indonesian ambient conditions. For low temperature conditions the applications of a two step UASB-septic-tank system could be profitable. Zeeman and Lettinga (1999) present expected removal efficiencies at treatment of black water and grey plus black water at low and tropical temperature conditions.

### 2.3.3 Upflow Anaerobic Sludge Blanket (UASB)

In the early seventies new, high rate, mainly upflow systems, were devoloped for the treatment of wastewaters, characterised by the achievement of a long retention of biomass at short liquid retention times and good contact between the biomass and the substrate. The most frequently applied system is the UASB system, developed by lettinga (1979). Ascheme of a UASB reactor is given in Figure 2.2. The three-phase separator provides a high retention of sludge in the reactor. The sludge bed in a UASB reactor can either consists of granular or flocculent sludge. For treatment of domestic sewage with a high fraction of suspended solids no granules will develop. Only when two-step systems are applied, where the main part of the suspended solids (SS) is removed in the first step methanogenic granules can develop in the second step (Zeeman *et al.*, 1997; Elmitwally *et al.*, 1999).

The UASB reactor consists of a circular or rectangular tank in which waste (water or sludge) flows in upward direction through an activated anaerobic sludge bed, which occupies about half the volume of the reactor and consists of highly settable granules

or flocs. During the passage of this blanket the purification takes place through complex mechanisms of bio-physical-chemical interrelated processes. Organic matter is converted into biogas (mainly methane which is a useful end product and carbon dioxide) and sludge. The produced biogas bubbles transfer to the top of the reactor, carrying water and solid particles (i.e. biological sludge and residual solids). These bubbles strike the degassing baffles at the upper part of the reactor, leading to an efficient gas-liquid-solid three-phase separation (GLS). The solid particles drop back to the top of sludge blanket, while the released gases are captured in an inverted cone (GLS)located at the top of the reactor. Water passes through the apertures between the degassing baffles carrying some solids particles which settle there due to increase of the cross sectional area and return back to the sludge blanket, while water leaves the settler over overflow weirs. A one step UASB reactor can achieve COD removal efficiency above 75% at HRT as low as 4-6 hours for raw sewage at 25 °C (Alaerts *et al.*, 1990).

The application of the conventional UASB systems is highly limited under suboptimal conditions of temperature (psychrophilic conditions) and high concentration of suspended solids.

### 2.3.3.1 Technical Description and Design Parameter of UASB Reactors

The first step in the design of a UASB reactor is to determine the volume of the reactor, upon which the details are based. The most important details are influent distribution, gas/liquid/solids (GLS) separator and effluent collection. Figure 2.2 is a schematic diagram illustrating the main elements of a UASB reactor which are the influent distribution, sludge blanket, gas dome and effluent collection gutter.



Fig. 2.2: Schematic diagram of a UASB reactor.

#### Volume

The volume of the reactor is determined by the hydraulic loading, organic loading or gas loading. If COD is below about 1000 mg/l hydraulic loading are limiting and determine the volume. Above 1000 mg/l COD, the organic or gas loading should be applied as the design parameters. Depending on the wastewater temperature and the hydraulic regime, a hydraulic retention time (HRT) is determined, usually between 3 and 6 hours for peak loading and 18 hours for average loading (Journey and Scott McNiven, 1996).

### Depth

The typical water depth for a UASB reactor for municipal wastewater is 4m. Above that height a freeboard of 0.3-0.8 m is required. A sump 0.3-0.5m deep required at the bottom of the reactor to allow complete drainage. Four meters is considered the optimal water depth at the current state of the art. Deeper, more compact designs will be the subject of further research. (Journey and Scott McNiven, 1996).

### **Influent Distribution**

Even distribution of the influent over the bottom surface of the reactor is one of the essential points for good performance. For municipal wastewater with high

concentrations of suspended solids (SS) and fibrous material, a series of splitter boxes has given the best results so far. From each of the several outlets of the final splitter box, usually located on top or inside the upper part of the reactor, a pipe leads to afixed point at the bottom of the reactor. The density of distribution points should ideally be as high as possible, but for practical reasons, a density of one inlet point per  $2.5-4 \text{ m}^2$  is regarded as optimal. At the recommended densities, the inlet pipe can be 50-80 mm in diameter and the water velocity 0.3-0.8 m/s. For example, a reactor with an effective volume of 1000 m<sup>3</sup> (and a bottom surface area of 250 m<sup>2</sup>) would have 40-62 inlet points (Journey and Scott McNiven, 1996).

### Gas/Liquid/Solids (GLS) Separator

The GLS separator is the second essential detail. The inclination of the walls of the gas dome should be between  $50^{\circ}-55^{\circ}$ . A flatter angle will lead to accumulation of sludge in the settling compartment is determined by the aperture velocity. The aperture velocity should be less than the initial settling velocity of the sludge. Generally, less than 10m/hour should be designed for at peak loading conditions and five m/h at average loading conditions. The overlap between the lower extremes of the GLS separator and the deflectors should be 10-20 cm. The surface area of the settler should be 60-80 percent of the total surface area of the reactor (Journey and Scott McNiven, 1996).

#### **Effluent Collection**

Equal collection of the effluent is the third crucial part of the design of a UASB reactor. Overflow weirs, or gutters, on both sides of the settling compartment provide good results. The weirs should have 45 v-notches, rounded at the apex that provide an overflow table of 10-20mm under all loading conditions (Journey and Scott McNiven, 1996).

### Sludge Sampling and Withdrawal

The dynamics of the sludge blanket can be controlled only by maintaining optimal blanket height. To determine the height of the blanket and the distribution of the

solids concentration over the height of the reactor, or the sludge profile, samples need to be taken at different heights. Sampling ports of sampling ports are recommended for larger reactors. Sludge withdrawal ports 100-150 mm diameter are placed at different heights and at different locations around the reactor. Sludge should be withdrawn at 0.2, 0.5 and 1.5 m from the bottom, depending on the dynamics of the sludge blanket. One set of sludge withdrawal pipes should be specified for each 100-150 m<sup>2</sup> of bottom surface area. A sump with a 150-250 mm outlet is needed to completely empty the reactor (Journey and Scott McNiven, 1996).

### Performance

Removal efficiencies depend on water temperature, hydraulic loading, reactor design, the quality of the maintenance status of the reactor. Depending on the composition of the wastewater, the removal efficiency of the UASB process may vary between 60-70 percent for COD and 75-85 percent for BOD<sub>5</sub>, at influent temperature between  $20-35^{\circ}$ . At 24° C a properly designed and built reactor treating a typical municipal wastewater, when operated within the design parameters, may be expected to average removal efficiencies of 75 percent of TSS. Only negligible amounts of nitrogen and phosphorus are removed; 75-90 percent of N will be converted to anytonium ion (NH<sub>4</sub><sup>+</sup>). Sulfur compounds are almost completely converted to hydrogen sulfide (H<sub>2</sub>S). Removal of low concentrations, 80-90 percent removal may be expected. Removal of pathogenic bacteria and viruses is about 50 percent.

The composition of biogas generated in the reactor depends on the characteristics of the wastewater and on the loading applied. Gas production is typically 220-250 l/kg of influent COD, excluding gas dissolved in the effluent. For an influent COD concentration of 300 mg/l, gas production will be about 60-75 l/m<sup>3</sup> of treated wastewater. The measured gas production is the primary control parameter of the reactor, e.g., the parameter that indicates whether the reactor is functioning properly. Lower production indicates inhibition of the biological process, sludge loss or some other problem. Sludge production depends mainly on the concentration and organic content of suspended solids in the wastewater and SRT and is adversely affected by sludge washout (Journey and Scott McNiven, 1996).

### **Construction materials**

For reasons of price and availability, concrete is the most utilized material for the tank and internal components of the reactor, such as columns, beams and GLS separator. Steel or plastics for the tank are not price competitive, but internal components could be made of polyester or polyethylene if these materials are available at a competitive price. The concrete must be of superior quality, well compacted, and cast in smooth forms. Concrete exposed to a corrosive atmosphere, such as outlet structures, could be protected with a lining or coating of a corrosion resistant material, such as epoxy. Larger splitter boxes are generally made of concrete and smaller ones of polyester. Inlet pipes are preferably made of concrete and smaller ones of polyester. Inlet pipes are preferably made of PE or PVC. Other piping can be of PVC or polyethylene gas pipe (hostalene). The use of metal should be minimized, since the gas and the treated effluent are highly corrosive. Where metal is essential, stainless steel should be used. Galvanized surfaces may be used in non-submerged or open spaces, such as ladders and railings (Journey and Scott McNiven, 1996).

### **Odor Abatement**

Gases from anaerobic treatment plants contains many offensive compounds. If odors cause a problem they may be reduced by ensuring a gas-tight gas collection system. Concrete gas collectors may be lined. Settlers and effluent gutters be covered and the air a above the water surface extracted and treated by compost filtration or a similar process (Journey and Scott McNiven, 1996).

#### 2.3.4 Modified one stage UASB reactor

The one stage UASB is recommended to be modified to overcome the prevailing conditions in the Middle East (Mahmoud, 1998). Consequently, it is suggested to modify the one stage UASB system by incorporating a supplementary digester (Figure 2.3). In the proposed system, the raw suspended solids will be captured in the UASB reactor and conveyed to the digester, which should be operated under optimal conditions of T, SRT, and others. While the methanogenic capacity of the UASB sludge. Morever, the finely dispersed biodegradable solids which are attached to the

sludge flocs in the UASB could be sludge flocs. Consequently the influent pre(acidified) COD will be partially/totally removed. Therefore, the UASB has three main functions: modified primary sedimentation tank, sludge thickener, and a methanogenic reactor. While, the digester has two main functions: sludge stabilisation (energy production 'CH<sub>4</sub> gas', improves dewatering characteristics) and providing methanogenic bacteria.

The design of the UASB is controlled by hydraulic loading rate or organic loading rate. The hydraulically controlled case is controlled mainly by the SS settling characteristics. While, the organically controlled case is controlled by the influent suspended solids content or VFA. The rate-limiting step of the overall degradation step (either hydrolysis or methanogenesis) is controlled by wastewater characteristics. For complex type of wastewater (high content of suspended solids) hydroysis is the rate-limiting factor.

The design of the digester in the UASB-digester system proposed by Mahmoud *et al.*, (2000) is controlled mainly by the sludge flow rate from the UASB. The minimum flow rate is controlled by the total amount of SS entering the UASB daily and sludge concentration in the sludge bed. While the produced biogas is controlled mainly by the biodegradability of the SS which is controlled by the chemical composition of the solids.



Fig. 2.3: Modified one stage UASB reactor

The wastewater characterictics, which control the design of the UASB- Digester system, are:

-VFA

-Amount of SS

-Settling characteristics of the SS

-Biodegradability of the SS: is mainly controlled by the chemical composition of the SS and particle size distribution

### 2.3.5 Hybrid Reactors

### 2.3.5.1 Hybrid UASB/ Attached Growth Reactor

An EPA-sponsored domonstration project located in Maryland evaluated the performance of an anaerobic reactor that provided enhanced primary treatment in a system treating pre-treated waste water from the Frederick County tratment plant. The treatment objective, advanced quality final effluent, was met for all criteria except for phosphorus removal. The hybrid reactor, a schematic diagram of which is presented in Figure 2.4, incorporates both basic types of anaerobic processes in one unit: suspended growth and fixed film, attached growth.

The first compartment contains a UASB reactor (without the three-phase /liquid/solids separator) that digests primarily particulate organic material, which is retained and digested over a retention time of up to one year. Gases are collected under the floating cover.

The final compartment contains a solid medium on which a bacterial film forms. The attached growth process is capable of digesting soluble and colloidal organic matter is less readily retained and degraded in a suspended growth reactor (Journey and Scott McNiven, 1996).

### 2.3.5.2 Hybrid Baffled Plug Flow/ UASB reactor

Reasearches at the University of the Andes in Bogota developed and demonestrated a hybrid anaerobic reactor that has a relatively high COD removal efficiency ( around 70 percent) in the-optimal temperature range for methanogenic bacteria below 20<sup>o</sup>C. The purpose of developing the reactor was to make anaerobic treatment of municipal effluents feasible in the range from 13-20 <sup>o</sup>C that prevails on the plateau on which Bogota is located. The reactor geometry imposes a plug flow hydrodynamic regime on the wastewater. Alternating vertical baffles force the fluid altenately upward and downward before it enters a final UASB compartment. Figure 2.5 is a schematic diagram of the hybrid reactor (Journey and Scott .McNiven, 1996).






# 2.4 Anaerobic removal and conversion of organic matter in domestic sewage

# 2.4.1 Physical removal of organic matter in domestic sewage

The removal of domestic-sewage particulate matter in an anaerobic reactor originates mainly from physical processes proceeding in the anaerobic biogass present in the system.

#### 2.4.1.1 Removal of Suspended Solids (SS) (Diameter >1 µm)

As SS represent the major COD fraction in domestic sewage, high removal of SS will lead to high removal of COD<sub>t</sub>. The removal of large particles in the SS fraction depends mainly on drag forces and settling characteristics. In an anaerobic reactor the removal of SS occurs by settling and by filtration through the sludge bed. The filtration mechanism includes entrapment, sorption and straining (Metcalf and eddy, 1991). The following factors affect the physical removal of SS in anaerobic reactors:

-Influent concentration: The sedimentation and filtration rate depend on the influent concentration. Grin *et al.*, (1985), Man *et al.*, (1986) and Wang (1994), found that the removal of COD<sub>t</sub> in an upflow anaerobic sludge blanket (UASB) reactor treating domestic sewage depends on influent COD<sub>t</sub>.

-*Temperature*: Grin *et al.*, (1985), Man *et al.*, (1986) and Wang (1994), found that the removal of SS in UASB reactors with both flocculent and granular sludge declines when the temperature of domestic sewage decreases. The higher viscosity of the wastewater at lower temperature reduces the sedimentation of particles, as the settling velocity of particles in domestic sewage is extensively proportional to the viscosity of the liquid according to Stokes law.

-Height of the reactor: Although the production of biogas causes mixing in the sludge beds and improves the contact between the wastewater and biomass, it also disturbs the sludge bed reduces the settling of SS in the reactor. In shallow reactors the rate of biogas production per unit area of cross section is lower than in tall reactors. Therefore, the removal of SS in shallow reactor is better than in tall reactor (Man *et al.*, 1986). *-HRT*: The HRT has physical and biological effects on the removal of SS. Increasing the HRT improves the settling and filtration in the reactor.

-Upflow velocity: with increasing of the upflow velocity at constant HRT and reactor height, the removal of SS by settling will decreases, as the drag force depends on the upflow velocity. Man *et al.*, (1986) found that application of an upflow velocity exceeding 0.5 m/h resulted in a significant decrease in SS removal in the treatment of domestic sewage in a UASB reactor at low temperatures.

-Particle size and density: wastewater particle size and density play an important role in separation of particles heavier and larger particles settle faster than lighter and smaller particles respectively. Also, the particle size of anaerobic sludge might be an important factor in SS removal as the filtration depends on the size of the filter media (Levin *et al.*, 1985; Odegraard, 1998). According to Lawler *et al.*, (1986) anaerobic digestion changes the particle size distribution of the sludge. They found that when digestion works well, particles of all size are destroyed, but there is a preferential removal of particles of small size. When digestion does not work well, large particles are destroyed but small particles are created.

#### 2.4.1.2 Removal of colloidal particles ( Diameter between 0.001 -1 µm)

The removal of colloidal particles, which represent 20-30% of the COD<sub>t</sub> in domestic sewage clearly is the limiting factor at low temperature treatment, (Yoda *et al.*, 1985; Mergaert *et al.*, 1992; Wang, 1994; Sayed and Fergala, 1995). According to Sayed and Fergala (1995) the entrapment mechanism involved in removing solids is not sufficient to remove colloidal particles through the sludge bed with high porosity and under upflow and gasification. Lattinga and Wang (1994) found that colloidal particles in domestic sewage could be removed in batch recirculation experiment with granular sludge at 20  $^{\circ}$ C, but only upon applying a long recirculation time (144 h).

The removal of colloidal particles occurs by filtration and/or sorption in the suspended and/or attached biomass present in the anaerobic reactor. As known, the stability of colloidal matter is governed by Van der Waals attraction and electrostatic repulsion forces.

#### 2.4.1.3 Removal of dissolved matter ( Diameter < 0.001 µm)

High-rate anaerobic systems can efficiently treat-strength soluble substrate as well as low-strength soluble substrates even at low temperature (Kato, 1994, Rebac, 1998). Although, the removal of dissolved matter is mainly a biological process, some physical aspects are also involved. A high concentration of active biomass and a good removal of SS are needed for optimization of soluble substrate removal at low temperatures (Agrawal *et al.*, 1997; Rebac, 1998; Lettinga *et al.*, 1999). As the solubility of gases increases at declining temperatures below 20 °C, a lower mixing will prevail in the sludge bed in systems operated at lower temperatures. Moreover, as also viscosity increases at lower temperatures, more energy is required for mixing and diffusion of soluble compounds (Perry and Green, 1984).

#### 2.4.2 Anaerobic conversion of domestic sewage

#### 2.4.2.1 Anaerobic conversion of particulate matter

Anaerobic digestion is a process that converts organic material to methane and carbon dioxide in the absence of oxygen. The anaerobic degradation of organic matter is a complex process that is accomplished by various groups of a micro-organisms. The degradation of proteins, carbohydrates and lipids takes place by different pathways. In general four different steps can be distinguished in the overall process; hydrolysis, acidogenesis, acetogenesis and methanogenesis (Vogelaar, 1997) as can be observed in Figure 2.6 (Sayed, 1987). Also the effect of temperature on the digestion process is briefly discussed.



Fig. 2.6 Anaerobic digestion of domestic sewage (Sayed, 1987).

#### Hydrolysis

Hydrolysis is the first step in the anaerobic conversion of organic substrate. In this process complex, undissolved polymeric substrates are transformed into plain, dissolved mono-or diametric substrates (Lattinga *et al.*, 1998). This process is merely a surface phenomena in which the polymeric particles are degraded to smaller compounds by extracellular enzymes. The polymers present in sewage can be divided in three groups; proteins, carbohydrates and lipids. The proteins are degraded into amino acids, the carbohydrates into soluble sugars and the lipids into long chain fatty acids and glycerin. In many cases the hydrolysis can be considered as the rate-limiting step in the overall process of anaerobic conversion (Haandel and Lettinga, 1994).

#### Acidogenesis

During hydrolysis the organic polymers were degraded outside the cell wall by exoenzymes. The reaction products (amino acids, simple sugars and long chain fatty acids) are now further degraded intra-cellular by the same micro-organisms that excerte the exo-enzymes. Most of these are *Clostridia* and *Bacteroides* species. The organic compounds are mainly converted into volatile fatty acids and is therefore indicated by Acidogenesis (Lettinga *et al.*, 1998).

#### Acetogenesis

The acetogenic bacteria convert the products of the acidogenesis bacteria into acetate, hydrogen and carbon dioxide. The oxidation to these compounds are thermodynamically unfavorable unless the partial hydrogen pressure is kept very low. This can be achieved by an effective uptake of the hydrogen by methanogens or sulfate reducing bacteria.

#### Methanogenesis

Methanogenesis bacteria accomplish the final stage in the overall anaerobic of organic material to methane and carbon dioxide. Methanogens are classified into two major groups: the acetate converting (acetoclastic) and the hydrogen utilising (hydrogentrophic) bacteria. The growth rate of the avetoclastic bacteria is low. This is the main reason for the fact that anaerobic treatment systems require a high biomass retention time. Generally 70-80% of the methane formed from the organic material originates from acetate (reaction A). The rest is mainly derived from hydrogen and carbon dioxide (reaction B).

The acetotrophic and hydrogenotrophic reactions are listed below (Miron, 1997).

A) Acetotrophic methanogenises:  $CH_3COO^- + H_2O \longrightarrow CH_4 + HCO_3^- (\Delta G^\circ = -31.0 \text{ kJ/mole})$ B) Hydrogentrophic Methanogenesis  $CO_2 + 4 H_2 \longrightarrow CH_4 + 2H_2O$  ( $\Delta G^\circ = -131.0 \text{ kJ/mole}$ )

# 2.4.3 Therotical calculation of biogas and methan production

The overall equation for anaerobic fermentation of organic matter is (Metcalf and eddy 1991):

 $C_n H_n O_b N_c + \left(n - \frac{a}{4} - \frac{b}{4} + \frac{3c}{4}\right) H_2 O \longrightarrow \left(\frac{n}{2} + \frac{a}{8} - \frac{b}{4} - \frac{3c}{8}\right) CH_4 + \left(\frac{n}{2} - \frac{a}{8} + \frac{b}{4} + \frac{3c}{8}\right) CO_2 + cNH_3$ Assuming organic matter is composed of  $C_5 H_7 O_2 N$  so:

 $C_5H_7O_2N + 3H_2O \longrightarrow 2.5CO_2 + 2.5CH_4 + NH_3$ 

113 g organic matter  $\longrightarrow 2.5$  mole CH<sub>4</sub> Under STP (Standard Temperature and Pressure), 1 mole of gas = 22.4 litre

So, 2.5 mole = 2.5\*22.4 = 56 litre methan and 2.5 mole = 2.5\*22.4 = 56 litre CO<sub>2</sub>

So the rate of methan production = 56 litre / 113 g organic matter =  $0.49 \text{ m}^3 \text{ CH}_4/\text{kg}$  organic matter

and the rate of biogas (CH<sub>4</sub>+CO<sub>2</sub>) production =(56 litre CH<sub>4</sub>+56 litre CO<sub>2</sub>)/113 =  $0.98 \text{ m}^3$  CH<sub>4</sub>/kg organic matter

#### 2.5 High rate anaerobic systems

High rate anaerobic systems are capable to retain concentration of active biomass, which allows the application of a relatively high loading rate and maintenance of a long SRT at a relatively low HRT. Several high-rate anaerobic systems were developed in the last three decades, e.g. the upflow anaerobic filter (AF) reactor (Young and McCarty, 1969), UASB reactor (Lettinga *et al.*, 1979).

#### 2.5.1 The UASB reactor

The UASB reactor is the most widely and successfully used high-rate anaerobic systems for several types of wastewater (Lettinga, 1996). The great success of the

UASB reactor can be attributed to its capability to retain a high concentration of active suspended biomass with simple and low cost means. Morever, the formation of granular sludge, which has a high methanogenic activity and is better settable than flocculant sludge, improves the maximum loading rate of the UASB system. Several versions of the upflow sludge bed system were developed all on the same basis, like the expanded granular sludge bed (EGSB) reactor (Man *et al.*, 1988).

The EGSB reactor, which uses granular sludge at upflow velocities exceeding 6 m/h, enables the application of treatment for very low strength wastewater and even for low strength cold wastewater (<10  $^{\circ}$ C). The EGSB system also looks very promising for rather toxic substrate.

#### 2.5.2 The Anerobic Filter (AF) reactor

#### 2.5.2.1 General

The AF reactor, a fixed-bed anaerobic reactor, retains the biomass in attached and suspended form. The methanogenic activity in the AF reactor is associated with both suspended and attached biomass to the packing material (PM) constitutes on the other hand an obstacle to horizontal mixing. The system traps large amounts of SS mainly in the lower part of the reactor, where mixing due to the gas production is lowest (Tilche and Vieira, 1991).

According to Iwai and Kitao (1994) the anaerobic biofilm present on PM offers the following advantages: 1) quicker start-up, 2) capability to remove slowly decomposable substrate, 3) higher tolerance to variation in temperature and loading rate, 4) improve the treatment efficiency of low concentration wastewater.

#### 2.5.2.2 Effect of the characteristics of PM on reactor performance

Miyahara and Noike (1994) studied the effect of PM on SS removal and hydrolysis by feeding an anaerobic reactor with a low strength synthetic wastewater (550 mg COD/l) containing cellulose as SS. They found that the effluent water quality is improved remarkably and also the hydrolysis of cellulose was promoted by packing

the reactor. Miyahara *et al.*, (1995) studied the relationship between the PM and the behavior of anaerobic bacteria. The number of suspended acidognic bacteria was found to be higher than the number attached to the PM. On the other hand, they also found more attached than suspended methanogenic bacteria. The accumulation of attached bacteria was promoted by decreasing the HRT of the reactor. The number of acidogenic bacteria in the reactor packed sparsely with PM was higher that in closely packed reactor. The number of methanogenic bacteria in the sparsely packed reactor was lower than that in the closely packed reactor.

#### 2.6 Practical application of anaerobic treatment of domestic sewage

## 2.6.1 Anaerobic treatment of domestic sewage under tropical condition

In tropical areas where the temperature ranges between 20-35 °C, high-rate anaerobic reactors offer a big prospect for the treatment of domestic sewage (Haandel and Lettinga, 1994). Lettinga and his co-workers (Lettiga *et al.*, 1983; Grin *et al.*, 1985; Man *et al.*, 1986; Lettiga *et al.*, 1987; Man *et al.*, 1988) started the research in the anaerobic treatment of domestic sewage in the late seventies. Subsequently, a considerable amount of research has been carried out in this field using a variety of anaerobic reactor types. Table 2.1 shows results of recent research for the anaerobic treatment of sewage in lab scale reactors at temperature >20 °C.

Reactor	Volume	Temp	HRT	Influent	Rem	ioval	Reference
	( m <sup>3</sup> )	(°C)	(h)	CODt	CODt	SS	
				(mg/l)			
UASB	64	24-26	4-6	267	65	70-85	Schellinkout et al., 1985
UASB	686	20-25	4-5	455	11-60	27-58	Nucci et al., 1985
UASB	120	-	4.7-9	315-265	50-70	56-79	Vierira, 1988
UASB	1200	20-30	6	563	74	75	Draaijer et al., 1992
UASB	6600	25	5.2	380	60-80	-	Schellinkout et al., 1992

Table 2.1: Summary of recent results for sewage treatment under tropical conditions  $(>20 {}^{\rm O}{\rm C})$  in pilot and full scale systems.

Based on the results obtained in a 64 m<sup>3</sup> UASB reactor in Caloi, Columbia for domestic sewage treatment (Schellinkhout *et al.*, 1985) full scale UASB reactors have been successfully put in operation. The UASB reactor has been considered as the most attractive anaerobic system for treating domestic sewage due to its simplicity, low investment and operation costs as well as the long favorable experience in the treatment of a wide range of industrial wastewaters (Lettinga and Hulshoff, 1991). According to Monory *et al.*, (2000) the anaerobic treatment of wastewater in Mexico started in 1987 and within one decade, 31 anerobic reactors, mainly UASB, treating domestic sewage have been installed. The performance and description of these reactors can be found in the article of Monory *et al.*, (2000). Table 2.1 presents the performance results of full and pilot scale UASB reactors treating domestic sewage. The presented results in Table 2.1 reveal that more than 70% of COD<sub>t</sub> reduction can be accomplished in treating raw domestic sewage.

#### 2.6.2 Anaerobic treatment of domestic sewage at low temperatures

The application of anaerobic treatment of domestic sewage is certainly not restricted to tropical regions. From results obtained by Lettinga and his co-workers (Lettinga *et al.*, 1983; Last and Lettinga, 1992; Wang; 1994), it is clear that anaerobic treatment of domestic sewage may also represent an attractive and feasible option for moderate climates. However, at low temperature, more sophisticated reactors are needed (Wang, 1994; Sayed and Fergala, 1995; Lettinga, 1996; Kalogo and Verstraete, 1999). Table 2.2 presents a summary of results of arecent research for the anaerobic treatment of domestic sewage at low temperature (<20 °C). Although the anarobic treatment of domestic sewage has been applied at full scale in several tropical countries, the process so far is not applied at full scale in countries in temperature climate regions, mainly as a result of the lower removal efficiencies. Morever, at low ambient temperatures, the amount of SS accumulating in the sludge bed increase (Imanori *et al.*, 1983; Genung *et al.*, 1985; Sanz and Fdz-Polanco, 1990).

Reactor	Slud	Sew	Т	HRT	Vup	Influe	% Rem	loval	Reference
_	ge		°C	h	m/h	CODt	CODt	SS	
	type					mg/l			
UASB	F	R	8-20	8	0.4	400	30-50	-	Grin et al., 1983
UASB	G	R	7-8	9-14	0.14	467-	57	72	Man et al., 1986
						700			
AF	F	R	10	6	0.7	529	54	-	Derycke et al.,
									1986
UASB	F	R	12-	18	-	465	65	73	Monroy et al.,
			18						1988
UASB	G	R	12-	7-8	0.23	190-	30-75	60	Man et al., 1988
			20			1180			
EGSB	G	S	>13	1-2	6	391	16-34	-	Last and
									Lettinga, 1992
UASB+	F,G <sup>b</sup>	R	18-	8-4,	-	200-	74-82	86-	Sayed and
<b>UASB</b> <sup>b</sup>			20	2 <sup>b</sup>		700		93	Fergalla, 1995
UASB+	F,G <sup>b</sup>	R	17	3,2 <sup>b</sup>	1.6 <sup>b</sup>	679	69	79	Wang, 1994
EGSB									
UASB+	F,G <sup>b</sup>	R	12	3,2 <sup>b</sup>	1.6 <sup>b</sup>	507	51	67	Wang, 1994
EGSB									
						1			

Table 2.2: Summary of recent results for treatment of sewage at low temperature (<20 °C)

<sup>a</sup>: two reactor operated in an intermitted mode; <sup>b</sup>: the second reactor; F: floculant sludge; G: granular sludge; R: raw sewage; S: pre-settled

Various investigation have been carried out to overcome the obstacles faced the anaerobic treatment of domestic sewage at low temperature by the following:

# 2.6.2.1 Use of granular seed sludge

Lettinga *et al.*, (1983) studied the effect of using granular sludge for the treatment of domestic sewage. Granular sludge has a higher methanogenic activity and a distinctly better settlability than flocculant sludge. However, as mentioned already above, the treatment of domestic sewage at low temperature in an one step granular sludge

UASB reactor leads to accumulation of SS, due to the slow hydrolysis resulting in deterioration of the methanogenic activity of the sludge (Man *et al.*, 1986).

# 2.6.2.2 Removing of SS prior to the anaerobic treatment by settling or physicalchemical pre-treatment

By treating pre-settled sewage at low temperature not only SS accumulation in the sludge bed is prevented, but it may also promote the formation granular sludge as was found by Vieira and Souza (1986). According to kalogo and Verstraete (1999) SS concentration in pre-settled sewage is still high to achieve a stable performance during anaerobic treatment. They report a pre-requested ratio CODdis to VSS in the domestic sewage of more than 10 for achieving an effective and stable treatment. Therefore, kalogo and Vertraete (2000) applied physical-chemical pre-treatment of domestic sewage prior to anaerobic treatment in a UASB reactor.

## 2.6.2.3 Applying high upflow velocities

In EGSB or AFB reactors, the negative effect of SS accumulation can be avoided and a good contact between wastewater and biomass at low temperature can be achieved as a result of the high upflow velocity. Last and Lettinga (1992) found a high removal of CODdis, viz. 84% of the maximum achievable COD<sub>dis</sub> removal, during the treatment of pre-settled sewage using an EGSB reactor at an HRT of 2 h at temperature > 13 °C. However, an additional settler is needed following an EGSB reactor. Morever, for achieving the required high upflow velocities in either an EGSB or an AFB reactor a tall reactor and/or recirculation of wastewater has to be applied. Therefore, the use of these systems will be accompanied with higher construction, operation and maintenance costs as compared to the UASB reactor, although the land requirements may be distinctly lower.

# 2.6.2.4 Treatment of raw domestic sewage in a two-step anaerobic system

In the first-step of a two-step system mainly removal of SS and partial hydrolysis and acidification will occur. Due to a slow rate of hydrolysis at low temperatures, the accumulated SS in the first-step need to be discharged regularly. As a result of that,

the SRT in the first reactor remains low, therfore preventing the sufficient development of slowly growing methanogens. As most of the SS are removed in this first-step, the removal and conversion of the remaining organic matter can be delegated to the seconed-step, the methanogenic reactor. As little SS will enter the second-step reactor, the specific methanogenic activity of the sludge remains at a high level. Wang (1994) used a combination of a high UASB+EGSB system for the treatment of domestic sewage at temperature ranging 12-17 °C (Table 2.2). In the flocculant sludge UASB reactor, a substantially higher CODss removal could be accomplished than in a conventional settler as a pre-treatment step. However, the removal of colloidal particles was found to be limited in this combined UASB+EGSB system. Sayed and Fergala (1995) studied the treatment of domestic sewage at temperatures of 18-20 °C in a two-step system, consisting of three reactors. The firststep consisted of two identical flocculant sludge UASB reactors, which were operated in intermittent mode for thr removal and digestion of SS. The second-step was granular sludge UASB reactor (Table 2.2). Tang et al., (1995) treated domestic sewage in atwo-step UASB+AF system st a temperatur of 20 °C (Table 2.2). Most of the COD removal was achieved in the first-step (UASB reactor), viz. 70% of the 80%, which can be attributed to the slightly higher operational temperature of 20 °C.

#### 2.7 Anaerobic degradation equations

The total CH<sub>4</sub> production in each serum bottle was summation of CH<sub>4</sub> in the headspace and the dissolve CH<sub>4</sub>. The dissolved CH<sub>4</sub> was calculated to Henry's law. Percentage of hydrolysis (H), acidification (A) and Methanogenesis (M) were calculated according to equation 2.1, 2.2 and 2.3 respectively (Elmitwalli *et al.*, 2000)

$$H(\%) = 100 * \frac{(CH_4 asCOD + effluentCOD_{diss} + inf \ luentCOD_{dis})}{inf \ luentCOD_t}$$
(2.1)

$$A(\%) = 100 \frac{(CH_4 asCOD + effluentVFAasCOD + inf luentVFAasCOD)}{inf luentCOD_t}$$
(2.2)

$$M(\%) = 100 \frac{CH_4 asCOD}{\inf luentCOD_t}$$
(2.3)

#### 2.8 Biodegradability

The converted COD fraction to CH4 gas can be calculated according to the gas law:

n = PV / RT

n: number of moles

P: 0.94 atmospheric pressure (at Birzeit University)

V: volume of collected NaOH solution (litre)

R: 0.082 gas constant (l\*atm / mole\*K)

T: temperature (K)

The number of moles can be used to calculate the weight of the produced methane (molar mass of  $CH_4$  by conversion factor of 4 (g COD/g  $CH_4$ ) according to:

 $CH_4 + 2O_2 \longrightarrow CO_2 + 2H_2O$ 

 $64 \text{ g COD} / 16 \text{ g CH}_4 = 4 \text{ g COD} / \text{ g CH}_4$ 

Biodegradability =  $(COD_{CH4} / COD_T) *100 \%$  (2.4)

Biodegradability also can be calculated on the basis of COD reduction by calculating the amount of digested COD as shown in the following equation:

 $COD_T degraded = \{ (COD_T, t=0 - COD_T, t=t) / COD_T, t=0 \} *100 \%$  (2.5)

# **Chapter 3**

## **Material and Methods**

#### 3.1 Site description

Wastewater is collected from the Faculty of Commerce to a manhole were wastewater is pumped through 2" plastic pipe to a holding tank connected to UASB setic-tank as shown in Figures 3.1 and 3.2. The wastewater is collected from 24 toilets and 32 sinks, all of them are connected to a single 4" riser and discharge out of the building through 6" PVC to the drop manhole. A submered pump is installed inside a 200 L plastic tank, it was constructed to collect black wastewater and work as an overflow. The average daily wastewater production was 4 cubic meter. The submersed pump has a level control, also a timer is installed in the main electrical box which found near UASB reactor to provide additional control utilities in order to set desired influent during the operation of the reactors.



Fig. 3.1: Layout of manhole (submersible pump), 2" line, holding tanks and UASB septic tanks



Fig. 3.2: Cross section from manhole (submersible pump) to wastewater treatment plant

# 3.2 UASB septic tank pilot plant experiments

Initially pilot plant experiments were conducted for on site treatment of black wastewater only, at the beginning  $0.35 \text{ m}^3$  pilot plant was installed near the duck weed ponds and fed by diluted black wastewater from Faculty of Commerce. The same research was also conducted to the treat gray wastewater by another MSc. Student at Birzeit University. For this purposes a second 0.35 m<sup>3</sup> reactor was put into operation near the first reactor. A schematic cross section of the pilot plant is shown in Figures 3.3 and 3.4.

The removal efficiency of the COD fractions of the black wastewater from the Faculty of Commerce was studied. Not only the physical removal of the suspended solids was considered but also the biochemical transformation of the COD was analyzed in order to determine the design and operational parameters for the application of this system.

# 3.3 Experimental Setup (UASB-septic tank)

The research setup consists of two units, the first is a holding tank and the other is a UASB septic tank as shown in Figures 3.3 and 3.4. The holding tank serves as a wastewater reservoir and a primary sedimentation tank. The incorporation of the holding tank provides a partial removal of the solids, which accumulates there for further stabilization. The partial solids removal in the holding tank will reduces the

solids loading rate in the UASB-septic tank and consequently reduce the desludging frequency as the UASB-septic tank reactor treats a pre-settled sewage. This kind of a system layout facilitates the operation of the system as desludging the holding tank is expected to be easier than the UASB-septic tank.



Fig. 3.3: Front view of two UASB septic-tanks and holding tanks



Fig. 3.4: UASB septic-tank for treated black wastewater details

#### **3.4 Inoculation**

According to the measured wastewater quality, the reactor needs to be started with a 1kg of inoculum. When inoculum of sludge of about 0.1-0.08 g COD/g VSS methanogenic activity is used (Lettinga *et al.*, 1991).

#### 3.5 Start up

The reactor was inoculated with 200 liters of anaerobic sewage sludge from anaerobic ponds present at BZU wastewater treatment plant. The specific methanogenic activity of the inoculation sludge amounted to 0.08-0.1 g COD/gVSS (Lettinga *et al.*, 1991). The specific methanogenic activity was 0.07 g COD/g VSS, and efficiency of UASB-septic tank is COD removal was 78 %.

#### **3.6 Analysis**

The collected samples were analyzed in duplicate used for  $NH_4^+$ , TKN, ortho phosphate and total phosphate, once reading for total suspended solid (TSS), Volatile suspended solid (VSS) and triplicate readings for COD<sub>t</sub> and it's fraction. Two types of filter papers were used to determine sample fractionation for analysis of COD: 4.4µm folded paper filter (Scheicher and Schnell ME 295<sub>1/2</sub> Germany) samples for COD<sub>f</sub> and 0.45 µm membrane filtered (Scheicher and Schnell ME25 Germany) samples for COD<sub>d</sub>. COD<sub>t</sub> was divided into COD<sub>s</sub>, COD<sub>c</sub> and COD<sub>d</sub>. The COD<sub>s</sub> and COD<sub>c</sub> were calculated by the differences between COD<sub>t</sub> and COD<sub>f</sub>, COD<sub>f</sub> and COD<sub>d</sub>, respectively. In addition, these filter papers were used to determine ortho-phosphate. For physical analysis, pH, temp, and solids were measured.

#### 3.6.1 Chemical Analysis

The collected samples were analyzed in triplicate used for total COD, COD fraction, kjeldahl nitrogen, total phosphorus, orthophosphate and ammonia nitrogen.

1- Chemical Oxygen Demand (COD): measurments were carried out using reflux method (acid destruction at 150 °C for 120 minutes), then measuring absorbance by

spectrophotometer at 600 nm wave length according to Standard Methods ABHA (1995).

2- Biological Oxygen Demand  $(BOD_5^{20})$ : measurment were carried out using idometric method, with aprob or DO sensor for five days at temperature 20 °C. The sensor where calibrated and compared with the result of winkler titration method.

2- Kjeldahl Nitrogen (TKN): it is the nitrogen present in the wastewater in the form of organic and ammonium. It was determined by using macro-kjeldahl method (digestion, distillation and titration) according to APHA (1995).

3- Ammonia: it was determined by Nesslerization spectrophotometer according to the Standard Methods APHA (1995). Sample absorbance was measured at 425 nm wavelength.

4- Total phosphorus (Total P): it was determined for raw wastewater samples digested by auto-claving at 120 C for 30 minutes-to get one bar pressure-according to the Standard Methods APHA (1995) and measuring absorbance by spectrophotometer at 880 nm wave length.

5- Ortho-phosphate ( $PO_4^{-3}$ -P): it was determined for filtered wastewater sample through 4.4 µm filter according to the Standard Methods APHA (1995) and measuring a bsorbance by specrophotometer at 880 nm wavelength.

#### **3.6.2 Physical Analysis**

1-Total suspended solids (TSS): it was measured according to the Standards Methods APHA (1995) by drying at  $105 \,^{\circ}$ C oven.

2- Volatile suspended solids (VSS): it was measured according to the Standards Methods APHA (1995) by burning at 550 °C oven.

3- Settleable solids: it was measured by using Imhoff Cone for 30 minutes settling time according to the Standards Methods APHA (1995).

4- pH: it was determined for the total sample by EC pH meter (HACH).

5-Temperature: the wastewater was determined by alcohol thermometer for sample in the site. Weather temperature was taken from BirNabala organization for weather forcasting.

6-Color: it was determined by visual appearance.

#### 3.7 Sampling

Part of waste management is sampling and analyzing the waste and wastewater to know what you have. According to Hunter et al (1965), an adequate sampling and composting procedure can give wastewater of a truly average composition. Information from samples can be valuable for selecting treatment technology and properly operating the system. If results of analysis of waste samples are to be meaningful, then the sample collection and preservation must be done properly. A good sampling program should standard methods, APHA (1995):

-Ensure that the sample is representative of the waste stream.

-Use proper sampling technique.

-Protect the samples until they are analyzed.

#### 3.7.1 Types of samples

There are many types of samples: The discrete and the composite. The nature and the composition of the system plus the information required will determine the type of collected samples. Since sampling and analysis are costly, so,when circumstances allow, composting the discrete samples is recommended. Composite samples may be prepared in number of ways to produce information. The following categories clarify sample types (Muttamara, 1996):

*I-Discrete samples(Grab or catch samples)*: it is collected randomly and is retained separately for analyses a sample collected at particular time and place represent only the composition of the source at that time and place. It is the simplest form of

sampling and gives flow conditions at the sampling time. If a series of discrete samples are taken at constant increments of time or flow, the collection of samples is known as discrete sequential sampling. These samples are useful for detecting fluctuation in composition and discharge of pollutants. Discrete samples may be more desirable than composite samples when the flow to be sampled does not flow on a continuous basis or the waste characteristics are relatively constant.

2- Composite samples: In most cases, the term "composite sample" refers to a mixture of grab samples collected at the same sampling point at different times.

*3-Time-interval composite samples*: it is the simplest from of composite samples, prepared by collecting equal volume sub-samples at regular time intervals and then combined in a bulked samples.

4-Flow-proportional composite samples: it is prepared in approximation to the flow rate during the sampling period. There are two variable schemes: first by taking samples at constant time intervals but varying the volume of the aliquot composited in proportional to the flow. Alternatively, equal volume sub-samples are collected at variable time intervals that are inversely proportional to the volume of flow.

5- Sequent composite samples: the technique allows for the collection of a succession of short-period composites, each composite being held in an indivitual container. For example, individual samples taken every 10 minutes during one-hour period are collected in one bottle to give a one-hour composite. This technique provides the advantage of a more representative composite, especially during rapidly fluctuation conditions while retaining the discrete sample information.

Determination	Minimum Sample Size ml	Preservation	Maximum Storage Recommended Regulatory 6h/48 h	
BOD	1000	Refrigerate		
COD 100		Analyze as soon as possible, or add H <sub>2</sub> SO <sub>4</sub> to pH <2: refrigerate	7 d /28 d	
Nitrogen:				
Ammonia	500	Analyze as soon as possible or add H <sub>2</sub> SO <sub>4</sub> to pH <2, refrigerate	7d/28 d	
Nitrate	100	Analyze as soon as possible or refrigerate	28 h/ 48 h	
Nitrite	100	Analyze as soon as possible or refrigerate	None/48 h	
Organic,kjeldahl	500	Refrigerate; add H <sub>2</sub> SO <sub>4</sub> to pH<2	7d/28 d	
Oxygen, dissolved: Electrode Winkler	300	Analyze immediately ,Titration may be delayed after acidification		
pH	-	Analyze immediately	2h /start	
Phosphate	100	For dissolved phosphate filter immediately; refrigerate		
Solids	-	Refrigerate	7d/2-7d	
Sulfate	-	Refrigerate	28d/28d	
Sulfide	100	Refrigerate; add 4 drop 2N zinc acetate /100ml ; add NaOH to pH>9	28 d /7 d	
Temperature	-	Analyze immediately	Stat/stat	
Turbidity	-	Analyze same day; store in dark up to 24 h, refrigerate	24 h/48 h	

# Table 3.1 Summary of special sampling of handling requirements APHA (1995)

#### Chapter 4

#### **Results and Discussion**

#### 4.1 General

The experiments were carried out in the experimental UASB-septic tank described in chapter 2. Prior to start the full scale experiment some preliminary tests were carried out in water lab. In the expriments emphasis was put on assessing the performance of the system with respect to treatment efficiency (COD and COD fractions, BOD, TSS, TKN, NH<sub>4</sub><sup>+</sup>-N, ortho PO<sub>4</sub><sup>-3</sup>-P & Total P, pH adjustment) and sludge retention (i.e. filling up time).

#### 4.2 Wastewater characteristics

Table A1 of Annex A shows the average values for characteristic of the black wastewater. The COD<sub>t</sub> is 644 mg/l and the BOD<sub>t</sub> is 324 mg/l while the ratio between COD<sub>t</sub>/BOD<sub>t</sub> is 1.98. The TKN is 46 mg/l and the  $NH_4^+$ -N is 37 mg/l while the ratio between  $NH_4^+$ -N/TKN is 80%, where organic nitrogen is 20%. The  $NO_3^-$  -N and  $NO_2^-$ -N is 0.17, 0.28 mg/l respectively, values of  $NO_3^-$  -N and  $NO_2^-$  -N is low because septic condition in sewer line. The TSS is 425 mg/l and the VSS is 287 mg/l while the ratio between TSS/VSS (ash content ) is 67%. The ortho  $PO_4^{-3}$ -P is 12 mg/l while total P is 14 mg/l while the ratio between ortho  $PO_4^{-3}$ -P/total P is 86%. The pH is 8.1 and the temperature is 17.5 °C.

#### **4.3 Inoculation**

According to the measured wastewater quality, the reactor needs to be started with a 1kg of inoculum. When inoculum of sludge of about 0.1-0.08 g COD/g VSS methanogenic activity is used (Lettinga *et al.*, 1991).

Table A2 of Annex A shows specific methanogenic activity during inoculation from return sludge of activated sludge then from anaerobic pond from wastewater treatment plant at Birzeit University, the result were taken from 25/3/01 to 29/5/01.

#### First inoculation at 8/4/2001

UASB was inoculated with activated sludge (Return sludge) from the wastewater treatment plant at Birzeit University, the volume of the inoculum was 70 L. This attempt failed due to immediate washout, while ( $COD_{inf}/VSS$  is 0.27) was not good.

#### Second inoculation at 7/5/2001

UASB was inoculated with activated sludge (Return sludge) from the wastewater treatment plant at Birzeit university, the volume of the inoculun was 70 L while ( $COD_{inf}/VSS$  is 0.6) was not good. The sludge inoculum proceed the following characteristics:

COD<sub>t</sub>=26844 mg/l TSS=416000 mg/l VSS=16420 mg/l SVI = 460 ml/l

#### Third inoculation at 27/5/2001

UASB septic tank was inoclated from nearby anaerobic bond as shown in photo C10 in Annex C, while ( $COD_{inf}/VSS$  is 0.07) is rather good. The property of the seed sludge were :

COD<sub>t</sub>= 61400 mg/l TSS = 34780 mg/l VSS= 13915 mg/l SVI= 480 ml/l

#### 4.4 Influent measurement

The daily influent flow rates were measured over the whole experimental period as shown in Figure 4.1.



Figure 4.1 shows influent flow rate over a period of batch and continuos period.

From 1/5/2001 to 30/5/2001, batch flow =50 L/d From 1/6/2001 to 30/6/2001, 0.5L/min (2hr operation) = 60 L/d From 1/7/2001 to 15/8/2001, 400 L/d

Table 4.1 shows parameter of influent which taken from 29/5/01 to 10/7/01. The fraction of suspended solids in the influent accounts for 50% of the total COD, while 8% is present as dissolved COD, 38.6% is present as suspended COD and 4% is present as colloidal COD. The  $COD_t/BOD_t$  ratio a mounts is 2.21, while Ammonia nitrogen is 52% of the total available nitrogen and ortho  $PO_4^{-3}$ -P/ total  $PO_4^{-3}$  ratio is 90% that mean 90% of the total  $PO_4^{-3}$  is present as ortho  $PO_4^{-3}$ . The ash content of the influent is 56% of the TSS.

Parameter	Unit	Average	Standard deviation	Period
CODt	mg/l	1013	262	29/5/01 to 10/7/01
COD <sub>dis</sub>	mg/l	155	143	29/5/01 to 10/7/01
COD <sub>sus</sub>	mg/l	784	162	29/5/01 to 10/7/01
COD <sub>coll</sub>	mg/l	162	50	29/5/01 to 10/7/01
BODt	mg/l	458	109	16/6/01 to 10/7/01
pH	-	8.02	0.37	29/5/01 to 10/7/01
TKN(N-total)	mg/l	41	3	29/5/01 to 4/6/01
NH4 <sup>+</sup> -N	mg/l	21	2	29/5/01 to 10/7/01
P-total	mg/l	11	3	29/5/01 to 10/7/01
PO <sub>4</sub> - <sup>3</sup> -P	mg/l	10	3	29/5/01 to 10/7/01
TSS	mg/l	715	285	29/5/01 to 10/7/01
VSS	mg/l	401	212	29/5/01 to 10/7/01
Fecal-coliform	Count/ 100ml	Uncountable		29/5/01 to 10/7/01
Total-coliform	Count/100ml	Uncountable		29/5/01 to 10/7/01

Table 4.1 Influent characteristics of anaerobic reactor

#### 4.5 Performance of reactor

The reactor was inoculated with 200 L. Seed sludge was taken from anaerobic bond which located in the Birzeit university wastewater treatment plant and was monitored for a period 12 weeks. Initially the reactor performed well COD<sub>t</sub> and TSS reduction up to (75 & 50%) respectively. The seed sludge improved when its methanogenic activity between 0.1-0.08 (gCOD/gVSS). However after 8 weeks a continuos operation. During the last weeks the performance of the reactor was rather good; COD<sub>t</sub> and TSS removal efficiencies of 86 and 85% respectively (Lettinga *et al.*, 1991).

The biogas production could not be measured at this reactor, because methonogensis activity takes time, In the UASB reactor at Biofarma, Indonesia gas production reading started after 150 days 4 l/hr (Lettinga *et al.*, 1991).

The property of the seed sludge were: TSS = 34780 mg/l VSS= 13915 mg/l  $COD_t= 61400 \text{ mg/l}$ SVI= 480 ml/l

Assuming a daily influent flow of 400 liters and an average influent  $COD_t$  of 900 mg/l, the prevailing start-up conditions are as shown in Table 4.2.

Parameter	Unit	UASB septic tank-BZU	
Hydraulic retention time	hr	11.6	
Sludge concentration	kg VSS/m <sup>3</sup>	13.9	
Organic sludge loading	kg COD/kg VSS.d	0.11	
Methanogenic activity	g COD/g VSS	0.07	
Upward velocity	m/h	0.10	

Table 4.2 Start-up conditions of the UASB septic tank at BZU.

#### 4.5.1 COD

Tables A3, A4, A5, A6, A7 of Annex A show the  $COD_t$  and fractions in all taps (tap1 = effluent), while Figures from B1 to B10 of Annex B show COD balances for the period from 29/5/2001 to 10/7/2001 in each taps as percentage of the average  $COD_t$  in mg/l and in percentage divided over  $COD_{coll}$ ,  $COD_{sus}$ ,  $COD_{diss}$ .

Figures from B11 to B14 of Annex B show the  $COD_t$ ,  $COD_{coll}$ ,  $COD_{sus}$ ,  $COD_{diss}$  at different heights of the reactor (taps) and Figure A2.20 show removal efficiency of total COD as a function of time.

Figure 4.2 shows the evolution of the  $COD_t$  in effluent in comparison with the influent, During the period 29/5/01 to 15/7/01, the first COD was a bout 200 to 600 mg/l, after that steady state occure for COD effluent values and reached 150 mg/l. The average  $COD_{effluent}$  is 246 mg/l and standard deviation is 199.The overall efficiegncy is 76% for COD removal.



Figure 4.2 Influent and effluent CODt for triplicate reading.

#### 4.5.2 BOD

Table A12 of Annex A shows BOD for influent and effluent from period 16/6/01 to 10/7/01 while Figure B21 of Annex B shows the removal efficiency of the total BOD as a function of time.

Figure 4.3 shows the evolution of the BOD total in effluent in comparison with the influent, During the period 16/6/01 to 10/7/01, in the first BOD in influent was a bout 400 mg/l, after that steady state occured for BOD effluent values and reached about 250 mg/l. The average BOD effluent is 190 mg/l and standard deviation is 53. The overall efficiency is 59% for BOD removal.



Figure 4.3 Influent and effluent BOD for duplicate reading.

#### 4.5.3 TSS

Table A8 of Annex A shows TSS concentration in all taps, Figure B16 of Annex B shows VSS at different heights of the reactor (taps) and Figure B26 of Annex B shows removal efficiency of TSS as function of time.

Figure 4.4 shows the evolution of the TSS in effluent in comparison with the influent During the period 29/5/01 to 15/7/01, in the first TSS in effluent increases up to 600 mg/l, after that steady state occurred for TSS effluent values and reached 100 mg/l. The average TSS effluent is 301 mg/l and standard deviation is 200.The overall efficiency is 58% for TSS removal.



Figure 4.4 Influent and effluent TSS.

#### 4.5.4 VSS

Table A9 of Annex A shows TSS concentration in all taps, Figure B17 of Annex B shows VSS at different heights of the reactor (taps) and Figure B27 of Annex B shows removal efficiency of TSS as function of time.

Figure 4.5 shows the evolution of the VSS in effluent in comparison with the influent During the period 29/5/01 to 15/7/01, in the first VSS in effluent was a bout 100 mg/l and increased up to 250 mg/l, after that steady state occurred for VSS effluent values and reached about 100 mg/l. The average VSS effluent is 158 mg/l and standard deviation is 63. The overall efficiency is 53% for VSS removal.



Figure 4.5 Influent and effluent VSS.

# 4.5.5 Total PO<sub>4</sub>-<sup>3</sup>-P

Table A14 of Annex A shows Total  $PO_4^{-3}$ -P concentration in all taps, Figure B18 of Annex B shows Total  $PO_4^{-3}$ -P at different heights of the reactor (taps) and Figure B25 of Annex B shows removal efficiency of Total  $PO_4^{-3}$ -P as function of time.

Figure 4.6 shows the evolution of the total  $PO_4^{-3}$ -P in effluent in comparison with the influent, During the period 19/5/01 to 15/7/01, in the first Total  $PO_4^{-3}$ -P in effluent was a bout 8.5 mg/l, after that steady state occurred for Total  $PO_4^{-3}$ -P effluent values and reached 6 mg/l. The average Total  $PO_4^{-3}$ -P in effluent is 8 mg/l and standard deviation is 2. The overall efficiency is 25% for  $PO_4^{-3}$ -P removal.



Figure 4.6 Influent and effluent total PO<sub>4</sub><sup>-3</sup>-P for duplicate reading.

# 4.5.6 Ortho-PO4-3-P

Table A13 of Annex A shows Total  $PO_4^{-3}$ -P concentration in all taps, Figure B19 of Annex B shows ortho  $PO_4^{-3}$ -P at different heights of the reactor (taps) and Figure B24 of Annex B shows removal efficiency of ortho  $PO_4^{-3}$ -P as function of time.

Figure 4.7 shows the evolution of the Ortho  $PO_4^{-3}$ -P in effluent in comparison with influent, During the period 29/5/01 to 15/7/01, in the first Ortho  $PO_4^{-3}$ -P in effluent was a bout 9 mg/l, after that steady state occurred for Ortho  $PO_4^{-3}$ -P effluent values and become 6 mg/l. The average Ortho  $PO_4^{-3}$ -P in effluent is 8 mg/l and standard deviation is 2. The overall efficiency is 22% for Ortho  $PO_4^{-3}$ -P removal.



Figure 4.7 Influent and effluent Ortho PO<sub>4</sub>-<sup>3</sup>-P for duplicate reading.

# 4.5.7 NH4<sup>+</sup>-N

Table A10 of Annex A shows  $NH_4^+$ -N concentration in all taps, Figure B15 of Annex B shows  $NH_4^+$ -N at different heights of the reactor (taps) and Figure B23 of Annex B shows removal efficiency of  $NH_4^+$ -N as function of time.

Figure 4.8 shows the evolution of the  $NH_4^+$ -N in effluent in comparison with the influent, During the period 29/5/01 to 15/7/01, the first  $NH_4^+$ -N in influent was a bout 25 mg/l, after that steady state occurred for  $NH_4^+$ -N in effluent values and reached 15 mg/l. The average  $NH_4^+$ -N in effluent is 17 mg/l and standard deviation is 2. The overall efficiency is 21% for  $NH_4^+$ -N removal.



Figure 4.8 Influent and effluent NH4<sup>+</sup>-N for duplicate reading.

#### 4.5.8 TKN

Table A11 of Annex A shows TKN concentration in influent and effluent and Figure B22 of Annex B shows removal efficiency of TKN as function of time.

Figure 4.9 shows the evolution of the TKN in effluent in comparison with the influent. During the period 29/5/01to10/6/01, the first TKN in influent was a bout 43 mg/l, after that steady state occurred for TKN in effluent values and reached 31 mg/l. The average TKN in effluent is 30 mg/l and standard deviation is 2. The overall efficiency is 27 % for TKN removal.



Figure 4.9 Influent and effluent TKN for once reading.

## 4.5.9 pH

Table A15 of Annex A shows the variation of pH from 29/5/01 to 10/7/01 for each taps. Figure 4.10 shows the evolution of pH at influent of range over 8 and in effluent reduces to become around 7.5.



Figure 4.10 pH variation in influent and effluent.

# 4.5.10 wastewater and weather temperature

Table A16 of Annex A shows wastewater temperature in celcius degree in each taps.

Wastewater temperature was measured on site by thermometer for influent and wastewater in taps when samples were taken from 29/5/01 to 10/6/01.



Fig. 4.11 Variation of wastewater temperature in influent and effluent.

Wheather temperature were taken from Ministry of transport, Meteorological office, BirNabala, values were measured every 3 hours, the maximum temperature is at 12 o'clock midnoon, the minimum temperature is at 12 o'clock at mid night or 7 o'clock at night (see table A1.17).



Fig. 4.12 Variation of ambient temperature.

# 4.5.11 Color

The color of wastewater in influent is dark browish and dark black in tap5, gradually decreases, dark in tap4, medium black in tap3, light in tap2, and clear in tap1 (effluent) as shown in photo A11 of Annex C.
### 4.5.12 Summary of results

Table 4.3: Summary of overall results, where it shows the average and standard

deviation for different parameter for all taps.

Parameter	T	ap1	T	ap2	T	ap3	Ta	np4	Та	p5	Influ	lent
	Avg	SD	Avg	SD	Avg	SD	Avg	SD	Avg	SD	Avg	SD
CODt	246	199	947	740	173 5	1769	2457	1867	72221	25690	1013	262
COD sus	110	150	365	285	748	680	1206	1414	5335	27008		
COD coll	62	61	284	337	488	1034	296	342	5411	6034		
COD diss	75	58	298	301	484	489	450	1061	13461	14324	-	
%(CODss/CODt)	45	75	38.5	38.5	43	38.4	49	76	8	105		
%(CODco/CODt)	25	31	30	45.5	28	58.5	12	18	8	23		
%CODdiss/CODt	31	29	31	40.5	28	27.5	38.5	57	18	55		
BOD t	190	53									458	109
CODt/BODt	1.3										2.21	
NH4 <sup>+</sup> -N	17	2	19	2	21	2	24	3	42	7	21	2
TKN	30	2									41	3
%(NH4 <sup>+</sup> -N/TKN)	56										52	
Ortho-PO <sub>4</sub> - <sup>3</sup>	8	2	10	5	12	5	16	6	29	11	10	3
Total PO <sub>4</sub> - <sup>3</sup>	8	2	10	2	12	4	16	7	54	3	11	3
$\%(0-PO_4^{-3}/Total)$	100		100		100		100		54		90	
TSS	301	200	637	337	107 5	429	1749	856	27231	12450	715	285
VSS	158	63	329	181	897	451	1574	1529	18685	8964	401	212
%(VSS/TSS)	52		52		83		90		68		56	-
РН	7.5	0.2	8.6	4.5	8.6	4.5	8.6	4.5	8.1	4.1	8.0	0.4
Tww	27.9	2.0	26.9	1.9	26.7	1.6	26.9	1.7	27.0	2.1	28.8	2.2

SD: Standard Deviation; Avg: Average value

#### 4.6 Previous Study: UASB Biofarma Reactor, Indonisia.

#### 4.6.1 Site description

The 0.86  $\text{m}^3$  on site UASB reactor treating black wastewater was installed at the housing comlex of Biofarma, Two housholds were connected to the reactor (including a total of 4 adults and 5 children living in units for low-income housing (Lettinga *et al.*, 1991).

#### **4.6.2 Influent Measurments**

The average flow rate over the whole experimental period was 60 L/d. Dring the experiments feeding of the reactor was interupted several times for a number of days to enable the operators to collect influent for carrying out relevant chemical analyses. The fraction of suspended solids accounts for 62% of the total COD, while 17% is present as colloidal material and 21% as soluble COD. The COD/BOD ratio amounts to 2.5. The total phosphate 59% is present as orthphosphate. The ammonia-nitrogen represents about 42% of the total available nitrogen (Lettinga *et al.*, 1991).

Parameter	Unit	Average	Standard deviation	Period
CODt	mg/l	5988	1471	3 years
COD <sub>dis</sub>	mg/l	1263	438	3 year
COD <sub>sus</sub>	mg/l	3730	1334	3 year
COD <sub>coll</sub>	mg/l	995	282	3 year
BODt	mg/l	2381	598	3 year
PH	-	6.9	0.3	3 year
TKN(N-total)	mg/l	646	129	3 year
NH4 <sup>+</sup> -N	mg/l	268	73	3 year
P-total	mg/l	65	17	3 year
PO <sub>4</sub> - <sup>3</sup> -P	mg/l	38	13	3 year
TSS	mg/l	2678	997	3 year
VSS	mg/l	2484	942	3 year

Table 4.4: Influent charecteristics of anerobic reactor-Biofarma (Lettinga et al., 1991).

#### 4.6.3 Performance of reactor

This reactor was inocculated with 110 L seed sludge from a septic tank and was monitored for a period of 14 weeks. Initially the reactor performed well:  $COD_t$  and TSS reductions up to 83 and 79 % respectively. The seed sludge improved in its methanogenic activity (from 0.026 to 0.061 kg CH<sub>4</sub>-COD/kg VSS.d). However after 10 weeks of continous operation, a significant wash-out of solids occured. During the last weeks the performance of the reactor therefore was rather poor; CODt and TSS-removal efficiencies of 68 and 36 % respectively. The biogas production could not be measured in this reactor (Lettinga *et al.*, 1991).

After the 25 th of september 1985 (corresponding to day 0 of the monitoring program) the reactor of Biofarma was seeded with 166 liter sludge and fed with influent. The start up conditions are mentioned in table 4.5. The slidge used for inoculation partly originated from the small 200 liter reactor, and partly from a nearby septic tank. The properties of the seed sludge were:

TSS = 21.4 g/l VSS = 13.5 g/l COD<sub>t</sub> = 15836 mg/l SVI = 445 ml/l

Assuming a daily black wastewater inflow of 51 litre/day (which is the average flow during the first 3 months of operation) with a COD content of 5300 mg/l (Lettinga *et al.*, 1991).

Parameter	Unit	UASB-Biofarma
Hydraulic retention time	hr	15.7
Sludge concentration	kg VSS/m <sup>3</sup>	2.8
Organic sludge loading	kg COD/kg VSS.d	0.121
Methanogenic activity	kg COD/kg VSS	0.083
Upward velocity	m/h	0.01

Table 4.5 : Start-up condition of a UASB-Biofarma (Lettinga et al., 1991).

#### 4.6.3.1 Gas production

The average gasproduction rate over the whole experimental period amounts to 4.9 L/hour. Analysis showed that the biogas contained 84.5 % methane (Lettinga *et al.*, 1991).

#### 4.7 Comparison of results

Table 4.6 shows comparison between UASB septic tank in Birzeit University, Palestine and Biofarma in Indonesia.

Table 4.6: Comparison between UASB septic tank at BZU and UASB septic tank Indonesia.

Item	Birzeit University,	Biofarma, Indinesia
	Palestine	
Reactor volume (m <sup>3</sup> )	0.35	0.86
	Influent characteristics	
COD <sub>t</sub> (mg/l)	1013	5988
COD <sub>dis</sub> (mg/l)	155	438
COD <sub>sus</sub> (mg/l)	784	1334
COD <sub>coll</sub> (mg/l)	162	282
BOD <sub>t</sub> (mg/l)	458	598
pH	8.02	6.9
TKN(N-total) (mg/l)	41	646
NH4 <sup>+</sup> -N (mg/l)	21	268
P-total (mg/l)	11	65
$PO_4^{-3}-P (mg/l)$	10	38
TSS (mg/l)	715	2678
VSS (mg/l)	401	2484
	Inoculation characteristics	
TSS (mg/l)	34780	21400
VSS (mg/l)	13915	13500
COD <sub>t</sub> (mg/l)	61400	15836
SVI (ml/l)	480	445
	Start-up condition	
Hydraulic retention time (hr)	11.6	15.7

Sludge concentration	13.9	2.8
(Kg VSS/m <sup>3</sup> )		
Organic sludge loading	0.11	0.121
(Kg COD/kg VSS.d)		
Upward velocity (m/h)	0.125	0.083
Methanogenic activity	0.07	0.01
(kg COD/kg VSS)		
l	Efficiency of UASB septic tank	
COD <sub>t</sub> removal %	76	93
BOD <sub>t</sub> removal %	59	92
TSS removal %	58	97
VSS removal %	53	92
TKN removal %	27	Not significant
NH4 <sup>+</sup> -N removal %	21	Not significant
P-total removal %	25	Not significant
PO <sub>4</sub> - <sup>3</sup> -P removal %	22	Not significant
Fecal and total coliform %	0.0	Not significant

#### 4.8 Sludge Blanket of UASB: Mathematical Simulation

#### 4.8.1 General

Blanket height is an important aspect of UASB reactor design. It regulates the solid concentration reaching the gas-solid separator (GSS) and thereby, prevents sludge washout. The maximum organic load which a reactor can assimilate depends on proper proportioning of the reactor height into the bed and blanket. (Narnoli *et al.*, 1997).

UASB reactor consists of a sludge bed, a sludge blanket and an internal settler supplemented with a physical device called the gas-solid separator (GSS). The reactor is provided with a bottom inlet for wastewater which passes through the bed and the blanket and finally comes out of the effluent channel provided around the settling zone. During the process, gas and solids are separated from water by the GSS (Narnoli *et al.*, 1997).

The conditions for sludge granulation blanket formation, sludge retention and sludge washout in the reactor are governed mostly by fluid dynamics (Narnoli *et al.*, 1997).

Variation of sludge concentration from the bed to GSS depends on 1) gas production or COD load per unit area, 2) upward water velocity or hydraulic retention time (HRT) and 3) sludge settling characteristics (Narnoli *et al.*, 1997).

### 4.8.2 Concentration of solids at different height

Two mechanisms are proposed for the transport of sludge particles from the bed to the blanket. The first mechanism is based on wake formation and was proposed by Heertjes and Van der meer (1978). The other, which was proposed by Bolle *et al.*, (1986a), is based on the concept of gas induced buoyancy. This means that the gas bubbles are attached to the surface of sludge particles and lift them from bed to the blanket (Narnoli *et al.*, 1997).

$$C = C_a * \exp\left[-\frac{(V_s - V_w)(y - a)}{20.q^2.(1 - e^{-0.55/q})^2}\right]$$
(4.1)

a= height of transmition zone (m).

C= solid concentration at any height y (kg m<sup>-3</sup>).

Ca=solid concentration at height a (kg  $m^{-3}$ ).

q= gas production rate per unit area per unit time ( $m^3 m^{-2} h^{-1}$ ).

Vs= settling velocity of a solid particle.

Vw=upward velocity of wastewater(hydraulic loading rate per unit area) (m .h<sup>-1</sup>).

#### 4.8.3 Physical model of an UASB-septic tank system

-phase separation and effluent
-sludge bed completely mixed
-water phase completely mixed
-no evaporation of water
-no gas in water phase
-model of height of the sludge bed

$$SludgeBedHeight = \frac{Q * t(SS_{settled} - SS_{hydrolysed} + SS_{biomass} + SS_{inoculum})}{SS_{max}} * \frac{h_{reactor}}{V_{reactor}}$$
(4.2)

with:

Q = influent flow(1/d)

T=time (days)

V<sub>reactor</sub>=reactor volume (l)

H<sub>reactor</sub>=reactor height (cm)

SS<sub>inoculum</sub>=SS of inoculum (g/l)

SS<sub>max</sub>=maximum SS conc. that can be reached in reactor (g/l)

SS<sub>hydrolysed</sub>=hydrolysed SS (g/l)

SSbiomass= amount of SS present as biomass (g/l)

SS<sub>settled</sub>=All SS from the influent that is settled, entrapped or adsorbed in the sludge bed (g/l)

#### 4.8.4 Biological model

From literature review it has been shown that for accumulation systems treating night soil or black water at a long retention time either the hydrolysis or the methane formation is rate limiting. Methane formation can be rate limiting at the start of the system, when no or too little methanogenic inoculum is supplied. When enough methanogic activity is present hydrolysis can be designated as the rate-limiting step of the process. This makes it possible to describe the whole biological the whole biological process by modeling only the methane formation and the hydrolysis.

#### 4.8.5 Model description

A schematical presentation of the model is presented in figure 4.13. The base unit of the model is COD. In the model the five types of COD and biomass can be recognized:

- suspended and colloidal COD (COD<sub>ss+coll</sub>)
- dissolved non acidified COD (COD<sub>diss(non vfa)</sub>)
- dissolved acidified COD (COD<sub>vfa</sub>)
- methane COD (COD<sub>ch4</sub>)
- refractory COD(COD<sub>refractory</sub>)
- Biomass

#### COD<sub>ss+coll</sub>

This is all COD that doesn't pass through a filter with pore size 0.45 micrometer. This COD fraction has to be colonized by bacteria and broken into soluble compounds by the enzymes which are excreted by the bacteria. This process is called hydrolysis. During the hydrolysis process monomeric and dimeric sugars, amioacids and peptides are formed. The non-biodegradable COD is left in the form of refractory COD.

At constant temperature the hydrolysis process can be modeled by a first order relation between the hydrolysis rate (dF/dt) and the amount of biodegradable substrate present, equation (4.3).

dF/dt = -kh.F

(4.3)

with: t=time (days) kh=first order hydrolysis constant (l/d) F= amount of degradable COD<sub>ss+coll</sub> (g COD/l)

The next step of the anaerobic digestion process is the acidification. During the acidification the products of the hydrolysis are taken up the bacteria and converted to volatile fatty acids, such as acetic acid, propionic acid and butyric acid. The acidification however is considered not to be rate limiting at any time during operation of low loaded reactors such as the AC system. Therefore the acidification process is left out of the model and it is assumed that during the hydrolysis volatile fatty acids are formed.

#### **COD**<sub>diss</sub>

This is all COD that does pass through a filter with pore size 0.45 micrometer but is not the form of volatile fatty acids. This COD has to be acidified. However as stated above the formation of volatile fatty acids is considered not to be rate limiting. Therefore in the model it is assumed that this acidification happens instantaneously upon entering the reactor. Not all dissolved COD is biodegradable, therefore also refractory COD is formed.

#### **COD**<sub>vfa</sub>

This is all COD that is present in the form of volatile fatty acids. Volatile fatty acids are not only present in the wastewater but also produced from suspended and colloidal COD and non-acidified dissolved COD. In 'real life' the volatile fatty acids which are not yet in the form of acetic acid (the substrate for the methane bacteria) are converted to this compounds during the acetogenesis. However, this acetogenesis, as the acidification, is considered not to be rate limiting at any time during operation of the AC system. Therefore also the acetogenesis is left out of the model and all volatile fatty acids are assumed to be readily present as acetic acid.

#### COD<sub>ch4</sub>

This is all COD converted from acetic acid to CH<sub>4</sub>. Usually in anaerobic digestion two conversion processes which produced methane are distinguished. One from acetic

65

acid and one from carbondioxide and hydrogen. However, for simplicity in this model it is assumed that all CH<sub>4</sub> is produced via acetic acid. The methane production is modelled via Monod kinetics.

#### **COD**<sub>refractory</sub>

This is all non-biodegradable COD. In the model it is assumed that refractory COD from suspended and colloidal COD remains suspended and refractory COD from dissolved COD remains dissolved.

#### Biomass

During all steps of the anaerobic process biomass is formed. Upon death and lysis it is assumed that this biomass produces volatile fatty acids and refractory COD.Moreover the present biomass attributes to the total COD of the system.



Figure 4.13: The biological model.

#### 4.8.5.1 Model input parameters

#### 4.8.5.1.1 Influent

-concentration suspended and colloidal COD

-biodegradability of suspended and colloidal COD

-first order hydrolysis constant
-concentration dissolved COD
-biodegradability of dissolved COD
-concentration volatile fatty acids

#### 4.8.5.1.2 Physical parameters

- wastewater flow
- reactor volume
- reactor height
- temperature(profile)

-UASB-septic tank systems

-Maximum sludge concentration

-Fraction of SS which can be settled, entrapped or adsorbed

-maximum growth rate methanogens

-Ks methanogens

-Yield methanogens

-Yield acidogens

#### 4.8.5.1.3 Inoculum sludge

-Volume

-VS and TS

-Methanogenic activity

#### 4.9 Calculation model for HRT from required SRT

The SRT represents one the most important (design) parameter for both methanogenic and acidifying reactors treating domestic sewage (Zeeman and Lettinga, 1999). According to Elmitawalli, (2000) studying the role of SRT on anaerobic digestion of primary sludge in completely stirred tank reactor (CSTR) system at 25°C, a SRT exceeding 8 days results in methanogenic conditions, where the hydrolysis was the rate-limiting step. Zeeman and Lettinga (1999) recently proposed a model for calculating the HRT for achieving the needed SRT in the treatment of a complex wastewater, like domestic sewage, using the following equation:

HRT = (C\*SS/X)\*R\*(1-H)\*SRT(4.4) Where C=influent COD<sub>t</sub> (mg/l), 1 mg VSS/l =1.4 mg COD/l) R= fraction of the COD<sub>ss</sub> removed H=hydrolyzed fraction of removed SS.

SRT = X/Xp (4.5) X: sludge concentration in the reactor (g COD/l); 1g VSS =1.4 g COD Xp: sludge production (g COD/l.d)

Xp = O\*SS\*R(1-H)(4.6) O: organic loading rate (kg COD/m<sup>3</sup>.d); SS=COD<sub>ss</sub>/CODinf

HRT=C/O (days)(4.7)

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

The following input data were taken into consideration:

R=0.8, Miron (1997) found around 85% TSS removal efficiency at  $V_{up}$  =0.56 m/h, H=0.15, 15% of the TS are hydrolysed under winter conditions (Elzen and Koppes, 2000)

X=15 g VSS/l =21 g COD/l

SS= 0.77 (this study: Birzeit University UASB)

C=1,013 g COD/l (this study: Birzeit University UASB)

SRT = 75 days, minimum SRT to achieve methanogenic conditions during winter time (Amarneh, 2001)

#### **Chapter 5**

### **Conclusion and Recommendations**

#### 5.1 Conclusion

Considering the results obtained in the reasearch project, the following conclusion can be made:

- Measurments of different parameters were started after achieving methanogenic activity (COD/VSS) of 0.1-0.08, based on the range recommended by Lettinga *et al.*, (1991) for operation of UASB septic tank.
- UASB-septic tank represents an effective on-site pre-treatment for black wastewater in tropical condition (T >20 °C), since it achieved 76% removal of COD<sub>t</sub>, 61% of BOD, 58% of TSS, and 53% of VSS.
- UASB septic tank is not efficient for removal nutrients since the efficiency of removal were 21% for NH<sub>4</sub><sup>+</sup>-N, 27% for TKN, 22% for ortho-PO<sub>4</sub><sup>-3</sup>-P and 25% for total P.
- The effluent from the reactor is still quite hazardous, particularly because the pathogen removal is poor i.e, no reduction of total and fecal coliform were achieved.
- Methan gas production CH<sub>4</sub> is very low since it was not detected in the start up phase.

Morever, it is worthwile to mention that there were some limitation for UASB-septic tank like:

• The efficiency of the system was droped when channeling in the sludge bed occurs or when relatively high load was imposed.

• In some cases, the high velocity of the inflow was more than 0.9 m/s, and this high velocity caused high concentration of pollutant in the effluent. To solve this problem, the velocity should be between 0.3-0.9 m/s.

#### **5.2 Recommendations**

- The experiments were conducted in outdoor under ambient temperature. It is recommended to conduct a UASB reactor in a lab to note the differences in findings.
- Since UASB-septic tank is very efficient for COD removal only, post treatment will be required to reduce NH<sub>4</sub><sup>+</sup>-N, TKN, ortho PO<sub>4</sub><sup>-3</sup>-P, and total P.
- Post treatment will be required to eliminate remaining pollutants, particularly pathogens. It is highly recommended to eliminate investigationss on the development of proper low cost post treatment systems, e.g. like lagoons, gravel filtration 'Trickling filter', facultative lagoons to meet higher degree of treatment in order to remove pollutant.
- It is recommended to use anaerobic sludge instead of aerobic sludge or facultative sludge to decrease time for start-up of UASB reactor.
- It is recommended to use a pump befor the influent tap for upward flow to controll up-ward velocity to prevent wash out of sludge.

#### References

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### Annex Annex A : Tables

Item	10/04/01	12/04/01	17/04/01	11/05/01	12/05/01	AVR	STDEV
CODt (mg/l)	724	656	600	700	538	643.6	75.58
BOD <sub>5</sub> <sup>20</sup> (mg/l)	371	330	286	363	274	324.8	43.89
TKN (mg/l)	56	42	54	39	37	45.6	8.79
NH₄ <sup>+</sup> -N(mg/l)	45	32	45	32	31	37	7.31
NO <sub>3</sub> -N(mg/l)	0.174	0.03	0.187	0.27	0.18	0.1682	0.09
NO <sub>2</sub> -N(mg/l)	0.328	0.228	0.283	0.323	0.239	0.2802	0.05
Total P(mg/l)				15	13	14	1.41
Ortho-P(mg/l)				13	11	12	1.41
TSS (mg/l)	466	412	397	550	301	425.2	91.70
VSS (mg/l)	351	268	254	374	189	287.2	75.36
рН	7.42	8.56	7.75	8.35	8.41	8.098	0.49
Tww <sup>o</sup> C	22	24	22	23	23.3	22.86	0.86
Twth <sup>O</sup> C	16	14.5	18	19	20	17.5	2.24

### Table A1: Characterization of black wastewater

Table A2: Start-up observation results

Date	TS	TSS	VSS	CODin	CODeff	VSS/TSS	Eff COD	COD/VSS
	(g/L)	(g/L)	(g/L)	(mg/L)	(mg/L)	%	%	g COD/gVSS
25/03/01	7.96	6.73	4.72	814	277	70	66	0.17
27/03/01	7.63	6.89	4.85	650	284	70	56	0.13
29/03/01	7.87	7.7	5.02	822	297	65	64	0.16
12/04/01	3.42	3.11	2.89	770	277	93	64	0.27
09/05/01	1.96	1.6	1.2	626	242	75	61	0.52
10/05/01	1.856	1.23	0.93	570	213	76	63	0.61
12/05/01	1.036	0.35	0.402	852	283	115	67	2.12
14/05/01	1.34	1.22	0.45	712	217	37	70	1.58
16/05/01	1.302	0.512	0.394	668	245	77	63	1.70
17/05/01	1.914	1.244	1.202	622	236	97	62	0.52
18/05/01	2.018	1.916	0.944	910	184	49	80	0.96
9/05/2001	2.43	2.22	1.38	826	301	62	64	0.60
21/05/01	1.102	1.052	0.484	940	284	46	70	1.94
22/05/01	1.814	0.76	0.648	650	171	85	74	1.00
23/05/01	1.737	1.217	0.734	540	178	60	67	0.74
24/05/01	1.421	0.987	0.512	835	177	52	79	1.63
26/05/01	1.327	1.106	0.612	580	190	55	67	0.95
29/05/01		34.78	13.915	1013	196	40	81	0.07

	CODt mg/l	CODdiss mg/l	CODsus mg/l	CODcol mg/l	Efficiency
	Average	Average	Average	Average	%
13/02/01	<u>_</u>		<u>v</u>	, in the second s	
Influent	1333	467	563	437	
15/02/01					
Influent	562	254	299	11	
17/02/01	502	204	200		
	605	216	204	04	
Influent	005	210	204	34	
25/02/01					
Influent	/48				
29/05/01					
Tap1	212	84	/4	53	73
Tap2	1500	102	1098	311	
Тар3	2633	191	2289	156	
Tap4	6367	80	6111	178	
Tap5	42833	956	39667	2222	
Influent	787				
30/05/01					
Tap1	387	58	156	178	56
Tap?	1013	347	378	289	
Tap2	2813	569	1911	333	
Taps	4103	1080	2556	556	
Tap4	4195	1000	36889	5667	
Tapo	47500	4930			
Influent	070				
01/06/01		10	70	183	
Tap1	303	48	200	1156	
Tap2	1/93	430	200	400	
ТарЗ	2167	569	1200	400	
Tap4	3660	2102	1267	209	
Тар5	99167	12511	82222	44 <b>44</b>	
Influent	889				
4/6/01/01					
Tap1	507	218	133	156	68
Tap2	2440	969	622	844	
Tap3	3060	1591	711	756	
Tap4	3613	2880	356	378	
Tap5	97167	11733	84333	1111	
Influent	1575				
07/06/01					
Tap1	563	240	156	167	39
Tap1	1097	418	522	156	
Tapz	2422	696	978	489	
Tap3	2433	836	1533	289	
Tap4	2000	8622	76333	1778	
Tap5	86767	0022	10000		
Influent	929				
11/06/01		60	689	78	18
Tap1	827	02	009	000	10
Tap2	2550	1118	600	000	
Tap3	7747	1591	1600	4556	
Tap4	6440	3413	1511	1511	
Tap5	71833	14844	34444	225 <b>56</b>	
Influent	1008				
minuom		and the second			

#### Table A3 : COD total and its fractions

	CODt mg/l	CODdiss mg/l	CODsus mg/l	CODcol mg/l	Efficiency
	Average	Average	Average	Average	%
13/06/01					
Tap1	65	21	19	25	92
Tap2	1462	423	641	398	
Tap3	1947	1172	224	550	
Tap4	3499	2643	570	286	
Tan5	79367	18667	53867	6833	
Influent	862				
14/06/01					
Tan1	178	64	78	36	88
Tap1	422	266	137	18	
Tap2	518	183	92	243	
Tapa	1182	762	292	133	
Tap4	80156	42400	27867	9889	
Influent	1522	72700			
15/06/01	1555				
Top1	120	70	30	21	86
Tapi	130	142	41	17	
Tapz	201	143	76	39	
Тарз	257	143	172	138	
Тар4	604	293	16533	1567	
Тарэ	39500	41400	10555	1007	
Influent	939				
16/06/01			20	21	86
Tap1	130	/9	30	12	
Tap2	160	115	52	10	
ТарЗ	322	1/8	94	277	
Tap4	1208	587	344	1567	
Tap5	59500	41400	16533	1507	
Influent	939				
20/06/01		40		17	87
Tap1	92	46	29	17	07
Tap2	907	218	233	400	
Тар3	1596	340	967	209	
Tap4	1/40	640	933	7444	
Tap5	56733	4/33	44889	/	
Influent	/20				•••••
21/06/01		AE	04	04	80
Tap1	87	45	21	21	09
Tap2	186	113	46	21	
Тар3	615	139	230	241	
Tap4	847	320	400	070	
Tap5	103944	2900	100167	0/0	•••••
Influent	813				
26/06/01			00	06	00
Tap1	108	61	22	20	00
Tap2	437	70	210	09	
Тар3	1684	240	1344	100	
Tap4	2873	140	2322	411	
Tap5	47844	1733	45111	1000	
Influent	892				

	CODt mg/l	CODdiss mg/l	CODsus mg/l	CODcol mg/l	Efficiency
	Average	Average	Average	Average	%
27/06/01					
Tap1	138	76	29	33	85
Tap2	390	215	47	129	
Tap3	687	287	258	142	
Tap4	648	431	133	83	
Tap5	96733	6178	77889	12667	
Influent	917				
30/06/01	}				
Tap1	294	70	141	83	78
Tap2	895	215	556	124	
Tap3	972	536	262	173	
Tap4	2120	642	1106	372	
Tap5	114956	21622	78889	14444	
Influent	1358				
04/07/01					
Tap1	107	38	34	34	91
Tap2	518	7	378	133	
Tap3	736	52	587	98	
Tap4	1100	69	951	80	
Tap5	85511	1622	82000	1889	
Influent	1140				
07/07/01					
Tap1	130	48	71	10	82
Tap2	727	74	556	98	
Тар3	692	96	449	147	
Tap4	972	96	831	44	
Tap5	18400	2289	15111	1000	
Influent	710	52	602	56	
09/07/01					
Tap1	227	73	124	30	77
Tap2	348	121	213	13	
Тар3	357	146	191	20	
Tap4	499	77	320	102	
Tap5	52067	3733	47556	778	
Influent	981	95	840	47	
10/07/01					
Tap1	198	14	179	6	
Influent	1367	318	911	138	

STDEV	199	740	1769	1007	20000		
AVERAGE	246	947	1735	1967	25690	262	19
		0.17	4725	2457	72221	1013	76
10/07/01	198					·····	
09/07/01	227	348	357	+00	02007	1367	86
07/07/01	130	727	092	199	52067	981	77
04/07/01	107	518	/30	972	18400	710	82
30/06/01	294	895	972	1100	85511	1140	91
27/06/01	138	390	687	2120	114956	1359	78
26/06/01	108	437	1684	618	96733	917	85
21/06/01	87	186	615	047	17844	892	88
20/06/01	92	907	1596	017	103944	813	89
16/06/01	130	160	322	1740	56733	720	87
15/06/01	130	201	257	1208	59500	939	86
14/06/01	178	422	518	1183	59500	939	86
13/06/01	65	1462	1947	3499	80156	1553	89
11/06/01	827	2550	7747	6440	70367	862	92
07/06/01	563	1097	2433	2660	71922	1008	18
04/06/01	507	2440	3060	3613	9/16/	020	39
01/06/01	303	1793	2167	3660	99167	009	68
30/05/01	387	1013	2813	4193	4/500	070	66
29/05/01	212	1500	2633	6367	42833	/0/	56
	TULAT	Tapz	Tape		10000	707	73

#### alved ma/l -11

Table A5:	CODdisso	Tang/L	Top3	Tap4	Tap5	Influent
	Tap1	Tap2	101	80	956	
29/05/01	84	102	191	1090	1956	
30/05/01	58	347	569	1000	4950	
01/06/01	48	436	569	2102	12511	
04/06/01	218	969	1591	2880	11/33	
07/06/01	240	418	696	836	8622	
07/00/01	62	1118	1591	3413	14844	
11/06/01	21	423	1172	2643	18667	ļ
13/06/01	61	266	183	762	42400	
14/06/01	04	143	143	293	41400	ļ
15/06/01	79	145	178	587	41400	ļ
16/06/01	79	019	340	640	4733	
20/06/01	46	218	120	326	2900	
21/06/01	45	113	138	140	1733	
26/06/01	61	70	240	121	6178	ĺ
27/06/01	76	215	287	431	21622	<u></u>
20/06/01	70	215	536	642	1622	
30/00/01	38	7	52	69	1022	52
04/07/01	48	74	96	96	2289	05
07/07/01	72	121	146	77	3733	95
09/07/01	15		(·····			318
10/07/01	14					ļ
		000	484	950	13461	155
AVERAGE	75	298	180	1061	14324	143
STDEV	58	301	409	100.		

	Tan1	Tan2	Tan3	Tant	Tan5	Influent
	тарт	Tapz	1403	1ap4	1 apo	midem
29/05/01	74	1098	2289	6111	39667	
30/05/01	156	378	1911	2556	36889	
01/06/01	72	200	1200	1267	82222	
04/06/01	133	622	711	356	84333	
07/06/01	156	522	978	1533	76333	
11/06/01	689	600	1600	1511	34444	
13/06/01	19	641	224	570	53867	
14/06/01	78	137	92	292	27867	
15/06/01	30	41	76	172	16533	
16/06/01	30	32	94	344	16533	
20/06/01	29	233	967	933	44889	
21/06/01	21	46	236	408	100167	
26/06/01	22	278	1344	2322	45111	
27/06/01	29	47	258	133	77889	
30/06/01	141	556	262	1106	78889	
04/07/01	34	378	587	951	82000	
07/07/01	71	556	449	831	15111	602
09/07/01	124	213	191	320	47556	840
10/07/01	179					911
10/07/01						
AVERAGE	110	365	748	1206	53350	784
STDEV	150	285	680	1414	27008	162

Table A6: COD suspended mg/L

#### Table A7: COD colloidal mg/L

	Tap1	Tap2	Тар3	Tap4	Tap5	Influent
29/05/01	53	311	156	178	2222	
30/05/01	178	289	333	556	5667	
01/06/01	183	1156	400	289	4444	
04/06/01	156	844	756	378	1111	
07/06/01	167	156	489	289	1778	
11/06/01	78	833	4556	1511	22556	
13/06/01	25	398	550	286	6833	ļ
14/06/01	36	18	243	133	9889	ļ
15/06/01	21	17	39	138	1567	
16/06/01	21	13	49	277	1567	
20/06/01	17	456	289	167	7111	ļ
21/06/01	21	27	241	113	878	
26/06/01	26	89	100	411	1000	ļ
27/06/01	33	129	142	83	12667	
30/06/01	83	124	173	372	14444	
04/07/01	34	133	98	80	1889	
07/07/01	10	98	147	44	1000	56
09/07/01	30	13	20	102	778	47
10/07/01	6					138
10/07/01						<u>.</u>
AVERAGE	62	284	488	300	5411	80
STDEV	61	337	1034	332	6034	50

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### Table A8 : TSS (mg/L)

	Tap1(effluent)	Tap2	Тар3	Tap4	Тар5	Influent	Efficiency %
29/05/01	145	210	1200	2510	34780	290	50
30/05/01	186	725	1375	2195	16345	320	42
01/06/01	475	485	1360	2450	20620	720	34
04/06/01	635	1250	1375	2950	22040	1325	52
07/06/01	364	824	1100	1344	48450	824	56
11/06/01	520	872	1720	2280	40853	860	40
16/06/01	492	980	1400	2384	12867	880	44
20/06/01	148	340	488	700	17408	796	81
27/06/01	136	500	715	860	37640	620	78
07/07/01	152	664	504	700	13327	560	73
09/07/01	56	156	584	870	35207	668	92
AVERAGE	301	637	1075	1749	27231	715	58
STDEV	200	337	429	856	12450	285	19

### Table A9 : VSS(mg/L)

	Tap1(effluent)	Tap2	Тар3	Tap4	Тар5	Influent	Efficiency %
29/05/01	95	115	675	1115	13915	215	56
30/05/01	110	425	445	1530	11165	295	63
01/06/01	160	365	960	1785	12715	650	75
04/06/01	245	380	1025	1400	16480	840	71
07/06/01	212	450	808	1108	34660	400	47
11/06/01	228	336	984	1392	26693	448	49
16/06/01	240	120	2000	6000	29800	280	14
20/06/01	84	220	690	1114	10144	108	22
27/06/01	128	116	640	1015	14660	212	40
07/07/01	88	720	368	200	9000	480	82
09/07/01	152	372	1272	660	26300	480	68
	-						
AVERAGE	158	329	897	1574	18685	401	53
STDEV	63	181	451	1529	8964	212	22

## Table A10: NH4<sup>+</sup> mg/L

	Tap1(effluent)	Tap2	Тар3	Tap4	Tap5	Influent	Efficiency %
29/05/01	17	22	24	28	36	20	15
04/06/01	18	18	21	27	40	25	28
10/06/01	20	21	23	25	42	26	23
20/06/01	18	21	22	24	45	21	14
26/06/01	17	20	23	26	57	20	15
27/06/01	16	18	20	22	34	19	16
30/06/01	16	16	19	24	43	20	20
07/07/01	16	17	19	20	36	21	24
09/07/01	14	17	19	20	42	19	26
10/07/01	15					20	25
AVERAGE	17	19	21	24	42	21	21
STDEV	2	2	2	3	7	2	5

### Table A11: TKN in mg/L

	influent	Tap1(effluent)	Efficiency %
29/05/01	43	31	28
31/05/01	38	28	26
04/06/01	42	31	26
AVERAGE	41	30	27
STDEV	3	2	1

Table A12: BOD<sub>5</sub><sup>20</sup> mg/L

	influent	Tap1(effluent)	Efficiency %
16/06/01	420	148	65
26/06/01	393	180	54
04/07/01	480	176	63
07/07/01	315	135	57
09/07/01	510	222	56
10/07/01	632	280	56
AVERAGE	458	190	59
STDEV	109	53	4

# Table A13 :Ortho PO<sub>4</sub>-<sup>3</sup>-P (mg/L)

STDEV	2	5	5	6	11	3	8
AVERAGE	8	10	12	16	29	10	22
09/07/01	4.2	2.6	3.3	3.6	18	5.1	18
07/07/01	6.4	8.4	9.2	14.1	39.2	9.7	34
30/06/01	6.6	5	7.1	15.1	49.9	9.8	33
26/06/01	6	7.6	13.8	14.8	30	8	25
20/06/01	8.6	11.7	13.9	19.8	24.6	10.7	20
16/06/01	9.5	11.8	12.5	18.4	23.6	11.8	19
12/06/01	9	15.6	18.9	21.5	21.7	11.7	23
07/06/01	12	14.7	16.6	20.7	21.1	13	8
	Tap1(Effluent)	Tap2	Tap3	Tap4	Tap5	Influent	Efficiency %
and the second se		the second s			and the second se		

Table A14: Total PO<sub>4</sub>---P (mg/L)

	Tap1(Effluent)	Тар2	Тар3	Tap4	Тар5	Influent	Efficiency %
01/06/01	8.5	9.6	12.9	15.3	51	10.3	17
15/06/01	9.5	9.6	10.5	13.6	53	14.1	33
30/06/01	9.7	12.9	18	27.1	54	15	35
07/07/01	8.4	9.8	10.1	15	58	12	30
09/07/01	5.8	6.7	7.2	8.5	52	7.2	19
10/07/01	7					8.4	17
AVERAGE	8	10	12	16	54	11	25
STDEV	2	2	4	7	3	3	8

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	Tap1(effluent)	Tap2	Tap3	Tap4	Tap5	Influent
29/05/01	7.16	7.03	7.1	7.07	6.98	8.41
30/05/01	7.09	7.07	7.09	7.14	6.99	8.47
01/06/01	7.19	7.12	7.19	7.34	6.97	8.26
04/06/01	7.31	7.23	7.29	7.38	6.88	7.68
07/06/01	7.43	7.58	7.7	7.48	8.2	7.08
11/06/01	7.52	7.51	7.93	7.55	7.02	7.61
13/06/01	7.35	7.37	7.33	7.4	6.85	8.32
14/06/01	7.58	7.43	7.38	7.4	7.48	8.15
15/06/01	7.57	7.44	7.34	7.42	7.09	8.22
16/06/01	7.53	7.42	7.45	7.51	7.3	8.19
21/06/01	7.85	7.56	7.47	7.55	7.3	8.37
26/06/01	7.43	7.6	7.96	7.82	7.74	8.1
27/06/01	7.29	7.42	7.42	7.34	6.84	8.28
29/06/01	7.3	8.24	8.15	8.19	7.18	7.63
30/06/01	7.5	8.24	8.15	8.19	7.18	7.63
04/07/01	7.7	8.32	8.26	8.27	6.95	8.27
07/07/01	7.8	26.7	<b>26</b> .5	26.6	24.6	8.06
09/07/01	7.5	7.82	<b>7</b> .7	7.68	6.83	7.8
10/07/01	7.6					7.8
AVERAGE	7.5	8.6	8.6	8.6	8.1	8.0
STDEV	0.2	4.5	4.5	4.5	4.1	0.4

### Table A16: Temperature <sup>O</sup>C degree

	Tap1(effluent)	Tap2	Тар3	Tap4	Tap5	Influent
29/05/01	27.9	27.2	26.9	27.4	27.2	27.9
30/05/01	24.4	24.3	25.2	25.1	24.4	25.1
01/06/01	26.3	24.2	25.1	24.7	23.5	29.5
04/06/01	24.9	24.3	24.3	25	24.7	25.6
07/06/01	27	25.6	26.2	25.6	26.2	26.3
11/06/01	25.7	25.2	26.5	26.2	27	28.1
13/06/01	30.7	31.1	30.2	30.1	30.9	32.2
14/06/01	32	29.2	29.2	29.2	30.4	32.2
15/06/01	29.5	28	27.4	27.3	28.1	28.9
16/06/01	30.7	28	27.7	28.4	29.5	32.9
21/06/01	26	26.5	25.8	26.7	27.6	30.1
26/06/01	27.5	29.3	28.7	30.4	27.8	30.1
27/06/01	28.1	27.7	27.2	27.1	28.5	30.3
30/06/01	27.2	26.9	25.6	25.8	26.3	26.2
04/07/01	27.3	26.3	26.3	26	25.9	28.9
07/07/01	27	26.7	26.5	26.6	24.6	26.4
09/07/01	25.9	26.7	25.2	25.8	26	29.2
09/07/01	28.5					28.2
10/07/01	28.5					28.2
10/07/01						
AVERAGE	27.6	26.9	26.7	26.9	27.0	28.8
STDEV	2.0	1.9	1.6	1.7	2.1	2.2

### Table A17 Ambient temperature

March, 2001			April, 2001		May, 2001		June, 2001		July, 2001	
Davs	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.
1	25.2	17	24.2	20.7	23.4	18	34.5	22	27.6	17.9
2	15.2	10	21.4	14.5	16.8	13	31.4	28.4	26.3	17
3	12	8.4	17	12.6	19	12.6			27	15
4			18.4	10	21.8	12			26	17
5			14.7	9.6	24	12.8			25	16
6			17	8.9	28.6	17.6			26.2	15.6
7			19.2	8.6	18	12.4			28	19
8	19.2	12.5	14.3	10	17.6	11			27	19
9	16.5	10.6	21	8.8	22	11.4			27.4	17.6
10	17.5	9.2	22	14	18	11.2			30	18
11	12.4	22	16.5	10.3	20	11			31.4	21.8
12	22.8	14	17.5	10	26.6	11			31.2	20.5
13	15	12	21.2	8.2	26	23.6			29.7	18
14	13.5	9.6	21	15	20	14.4	32.5	27	29.5	20.8
15	16.8	8.5	18.8	10	17.8	11.5			29.5	18
16	16	8.5	21.5	8.6	20	11			28.4	19.5
17	21.4	9	23.8	14	21	11			28	20
18	24.5	12	29	16.2	22.6	11.5			29	19
19	25.4	15	19.2	13.6	24	12.6			28	21
20	26.6	17.8	23.6	9	27.8	15	33	29.1		Į
21	24.7	17.8	28.4	17.6	31	20.6	32.1	28.2	31.8	22.8
22	18	11.8	28	19.6	26.5	18			32.2	21.5
23	19	11	24.2	20.6	31	17.2	23	18.2	30	21.4
24	12.6	10.6	18	11	30.5	23.6	25.4	17.4	29.5	18
25	16.4	9	16.5	10	28	20.3	23.5	15.6	27.4	18.2
26	22.4	16.7	22	10	31.5	18.4	24.4	20.5	26	18
27	27.1	15.8	26.5	11	35.4	24	25.3	20.4		<u> </u>
28	27.6	19.6	29.5	18.4	34.4	25.4				
29	22.8	17.8	30.4	21.5	22.6	16				
30			27.6	21.6	24	14	29.2	20	30.9	25.9
31	29.4	23.8			27.4	15.6				
<u> </u>										
AVR	20.0	13.5	21.7	13.1	24.4	15.4	28.6	22.4	28.6	19.1
STDEV	5.3	4.5	4.6	4.4	5.2	4.5	4.3	4.9	2.0	2.4

#### **Annex B : Figures**

B: Mean values of (COD balance,concentrations in taps, efficiency curves) for COD and fractions, BOD, TSS, VSS, NH4<sup>+</sup>-N, TKN, *o*-PO4<sup>-3</sup>-P, total P, pH



Figure B1: COD balances per one half a month in tap1, as percentage of the average CODt in mg/l divided over COD coll, COD sus, CODdiss.



Figure B2: COD balances per one half a month in tap2, as percentage of the average CODt in mg/l divided over COD coll, COD sus, CODdiss.







Figure B4: COD balances per one half a month in tap4, as percentage of the average CODt in mg/l divided over COD coll, COD sus, CODdiss.



Figure B5: COD balances per one half a month in tap5, as percentage of the average CODt in mg/l divided over CODcoll, COD sus, CODdiss.



Figure B6: COD balances per one half a month in tap1, as percentage of the average CODt divided over CODcoll, COD sus, CODdiss.


Figure B7: COD balances per one half a month in tap2, as percentage of the average CODt divided over CODcoll, COD sus, CODdiss.



Figure B8: COD balances per one half a month in tap3, as percentage of the average CODt divided over CODcoll, COD sus, CODdiss.



Figure B9: COD balances per one half a month in tap4, as percentage of the average CODt divided over CODcoll, COD sus, CODdiss.



Figure B10: COD balances per one half a month in tap5, as percentage of the average CODt divided over CODcoll, COD sus, CODdiss.







Figure B12: COD colloidal at different heights of the reactor (sampling point heights in each profile :17, 48, 107, 132 and 157 cm from the top respectively)



Figure B13: COD suspended at different heights of the reactor (sampling point heights in each profile :17, 48, 107, 132 and 157 cm from the top respectively)



Figure B14: COD dissolved at different heights of the reactor (sampling point heights in each profile :17, 48, 107, 132 and 157 cm from the top respectively)



Figure B15:  $NH_4^+$ -N at different heights of the reactor (sampling point heights in each profile :17, 48, 107, 132 and 157 cm from the top respectively)



Figure B16: TSS at different heights of the reactor (sampling point heights in each profile :17, 48, 107, 132 and 157 cm from the top respectively)



Figure B17:VSS at different heights of the reactor (sampling point heights in each profile :17, 48, 107, 132 and 157 cm from the top respectively)



Figure B18: Total P at different heights of the reactor (sampling point heights in each profile :17, 48, 107, 132 and 157 cm from the top respectively)



Figure B19:  $o-PO_4^{-3}$ -P at different heights of the reactor (sampling point heights in each profile :17, 48, 107, 132 and 157 cm from the top respectively)







Figure B21: Removal efficiency of total BOD as function of time.



Figure B22: Removal efficiency of TKN as function of time.



Figure B23: Removal efficiency of  $NH_4^+$ -N as function of time.



Figure B24: Removal efficiency of o- PO<sub>4</sub><sup>-3</sup>-P as function of time.



Figure B25: Removal efficiency of total  $PO_4^{-3}$  as function of time.



Figure B26: Removal efficiency of TSS as function of time.



Figure B27: Removal efficiency of VSS as function of time.

## **Annex C : Photos**



Photo C1: Collected manhole from faculty of commerce.



Photo C2: Submersiple pump in collected manhole



Photo C3: line 2" from manhole to holding tank for black wastewater.



Photo C4 : Holding tanks and UASB tanks for black and grey wastewater.



Photo C5 : Holding tank ,UASB tank and gas meter for black wastwater.



Photo C6 : Influents and effluent for UASB tank and taps for black wastwater.

2.



Photo C7 : Effluent for UASB tank and cover the reactor.



Photo C8 : Effluent of treated wastewater from UASB tank to treatment plant.



Photo C9: Gas meter and its connection.



Photo C10: Anaerobic bond which take from it seed sludge.



Photo C11: Tube shows concentration of solids and color of wastewater in taps.

## الخلاصة

إن التطبيقات و التجارب التي أجريت على المفاعل اللاهوائي UASB-Septic tank في جامعة بير زيت في الفترة ما بين افتراير 2001 إلى استمبر 2001 أعطت نتائج مشجعة من حيث الفعالية الجيدة في معالجة المياه العادمة سواء كانت مياه عادمة سوداء أو متزلية، حيث إن العامل الاقتصادي الذي تمتاز بما هذه الطريقة جعلت البلدان النامية تتبنى تطبيقها، حيث أنها توفر الاكتفاء الذاتي في هذا الجال.

إن الهدف الرئيسي من هذا البحث هو تشغيل المفاعل اللاهوائي UASB-Septic tank ومراقبة أداؤه طوال هذه الفترة. من خلال التجارب التي أجريت و النتائج التي حصلنا عليها\_ ورغم تشغيل المفاعل اللاهوائي بشكل متقطع لعدم ثبات طبقة الحمأة و تشغيل المفاعل اللاهوائي بالنظام المستمر لمدة ثلاثة شهور فقط \_ أظهرت التحاليل المخبرية أن المفاعل ذو فعالية جيدة في إزالة المواد العضوية من المياه العادمة. إن إزالة المواد العضوية كانت كنتيجة لعمليات فيزو حيوية بفعل الترسيب والتحليل المكتيري لهذه اللوتات فقد تم إزالة 76% من COD الإجمالي، 86% من CODs، 23% من CODs و 25% من CODdiss.

إن المياه المعالجة الخارجة من المفاعل اللاهوائي UASB-Septic tank تتميز بشفافية جيدة تسهل عمليات المعالجة اللاحقة، حيث إن عملية التمثيل الضوئي وزيادة pH "درجة الحموضة" في مراحل المعالجة المتقدمة تزيد من استهلاك ثاني أكسيد الكربون و تسرع في نسبة موت الجرائيم المرضة و البكتيريا و توسع أيضا من إمكانية إزالة النيتروجين و الفوسفور الزائد في المياه العادمة التي لم يتم إزالتها في المفاعل اللاهوائي. إن ميزة نظام UASB-Septic tank من حيث صغر حجمه و تجميع الغازات النابحة من التحلل الحيوي ( غاز الميثان وغاز ثاني أكسيد الكربون) و عدم وجود روائح يجعل من هذا النظام صديق للبيئة، كما إن سهولة تشغيله يجعل من تطبيقه لمعالجة المياه العادمة للمناطق السكانية بشكل مركزي فكرة مستحسنة إضافة إلى تطبيقه في المناطق الريفية بشكل لامركزي لمعالجة مياه كل بيت على حده. تعزيز التنقية الأولية للمياه العادمة السوداء الخارجة من كلية التجارة-جامعة بيرزيت بواسطة

التنقية اللاهوائية

رسالة ماجستير مقدمة من

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قدمت هذه الرسالة استكمالا لمتطلبات درجة الماجستير في هندسة المياه من كلية الدراسات العليا.

في جامعة بيرزيت-فلسطين

## ديسمبر – 2001